# Assessment of the 1991 NEHRP Provisions for Nonstructural Components and Recommended Revisions

by

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# NOTICE

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# PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.

| ELEMENT I<br>BASIC RESEARCH | ELEMENT II<br>APPLIED RESEARCH | ELEMENT III<br>DEMONSTRATION PROJECTS |
|---|---|---|
| • Seismic hazard and ground motion<br><br>• Soils and geotechnical engineering<br><br>• Structures and systems<br><br>• Risk and reliability<br><br>• Protective and intelligent systems<br><br>• Societal and economic studies | • The Building Project<br><br>• The Nonstructural Components Project<br><br>• The Lifelines Project<br><br>• The Bridge Project | Case Studies<br>• Active and hybrid control<br>• Hospital and data processing facilities<br>• Short and medium span bridges<br>• Water supply systems in Memphis and San Francisco<br>Regional Studies<br>• New York City<br>• Mississippi Valley<br>• San Francisco Bay Area |

**ELEMENT IV**
**IMPLEMENTATION**

• Conferences/Workshops
• Education/Training courses
• Publications
• Public Awareness

Research tasks in the **Nonstructural Components Project** focus on analytical and experimental investigations of seismic behavior of secondary systems, investigating hazard mitigation through optimization and protection, and developing rational criteria and procedures for seismic design and performance evaluation. Specifically, tasks are being performed to: (1) provide a risk analysis of a selected group of nonstructural elements; (2) improve simplified analysis so that research results can be readily used by practicing engineers; (3) protect sensitive equipment and critical subsystems using passive, active or hybrid systems; and (4) develop design and performance evaluation guidelines.

The end product of the **Nonstructural Components Project** will be a set of simple guidelines for design, performance evaluation, support design, and protection and mitigation measures in the form of handbooks or computer codes, and software and hardware associated with innovative protection technology.

*The work presented in this report represents one part of the 1994 update effort of the 1991 NEHRP provisions. The seismic design formulas for nonstructural components as they exist in 1991 NEHRP are critically reviewed and various levels of improvements to these formulas are recommended based on analyses and experiments, performed by NCEER researchers and elsewhere, as well as on observation data from past earthquakes. The recommended revisions thus bring the existing formulas more in line with the state-of-the-knowledge in the area of seismic behavior of nonstructural components. Also proposed in this report is a set of displacement equations which can be useful in the design process.*

# ABSTRACT

As one part of the 1994 update effort of the 1991 NEHRP provisions, the seismic design force formulas for nonstructural components as they exist in the 1991 provisions are critically assessed and some of their shortcomings are identified. Various levels of improvements to these formulas are then presented which, on the one hand, preserve the equivalent lateral force format for design applicability and, on the other, correct some of their deficiencies on the basis of analyses, experimental results and observation data from past earthquakes.

Based on different interpretations of the component seismic coefficients as well as different degrees of simplicity required in practical design, three recommendations are proposed. The first recommended revision is the most comprehensive in that both effects of nonstructural component anchorage detailing and its supporting structural characteristics are taken into account. The second recommendation is a structure-driven type of modification of the current provisions and is motivated by the possibility that nonstructural component information during a design process is not available. The third revision, however, mainly concentrates on the effect of nonstructural component characteristics on the design force although it partially implies structural effects in the process of determining the response modification coefficient. The maximum and minimum design forces in the three recommendations are compared with those produced by the 1991 NEHRP provisions, the 1991 UBC, and the 1985 Tri-Service codes. Case studies of a parapet, a storage rack and a general equipment attached to a reinforced concrete shear wall structure are provided to show the relative conservatism involved in different codes and the importance of the factors ignored in the current provisions.

Simple displacement equations are also developed in this report to provide deformation information needed in some cases of practical design.

## ACKNOWLEDGEMENTS

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# SECTION 1

# INTRODUCTION

Nonstructural components anchored or attached to a building structure are generally considered as those elements that are not a part of the load-bearing structural system. They include mechanical and electrical equipment, architectural elements, and building contents. The importance of nonstructural component issues in seismic design and performance evaluation is now well recognized by researchers as well as practicing engineers. The subject received special attention after the San Fernando earthquake of 1971 when it became clear that nonstructural damage not only can result in major economic loss, but also can pose real threat to life safety.

Today, major building codes and seismic design guidelines exist which address seismic design forces for various nonstructural components. The 1991 NEHRP provisions [7] and 1991 UBC code [13], for example, are widely used for seismic design standards for nonstructural components, from which local jurisdictions and some federal agencies have developed similar seismic design requirements. In these provisions, the design force for nonstructural components is formulated as an equivalent static lateral force applied to the approximate center of gravity of the component being analyzed. While simple formulas are necessary for the sake of design applications, they contain considerable amount of arbitrariness and subjectivity which produce ambiguous results and inconsistent design forces among different codes and provisions. Furthermore, they do not reflect the level of understanding of the behavior of nonstructural components that has been achieved through theoretical analyses, experiments, and observation data from past earthquakes.

The thrust of this work is to critically assess current design force formulas for nonstructural components as they exist in the 1991 NEHRP provisions, to identify their shortcomings, and to recommend revisions which would bring them more in line with current state-of-the-knowledge in this area. These revisions are recommended within the framework of the equivalent lateral force format for practical applicability.

The 1991 NEHRP design force formulas for nonstructural components are reviewed in Section 2 and their shortcomings are identified and commented upon. Three revision recommendations based on incorporation of different levels of consideration of structural and component effects are advanced in Sections 3-5. It is shown that these revisions can

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be justified on the basis of simple dynamic analyses, experimental results, and experience data.

In Section 6, these three recommended design force formulas are compared among themselves as well as in relation to existing codes and provisions. These results are further examined through case studies.

Since excessive displacements on the part of nonstructural components are causes of failure in a large number of cases during past earthquakes, the derivation of simple displacement equations for support deformation and sliding is considered in Section 7. It is recommended that this type of displacement equations be added to future codes and provisions in order to achieve a more complete seismic design scenario for nonstructural components.

## SECTION 2

## AN ASSESSMENT OF 1991 NEHRP PROVISIONS

### 2.1 Brief Summary of Current Design Forces

In 1991 NEHRP Provisions [7], as in other codes such as the 1991 UBC [13] or the 1985 Tri-Service Manual [25], seismic design forces for nonstructural components are specified in terms of static equivalent forces acting through their centers of gravity. They are described below for architectural components and for mechanical and electrical components.

#### 2.1.1 Architectural Components

Architectural components and their means of attachment are designed for seismic forces $(F_p)$ determined in accordance with the following equation:

$$F_p = A_v C_c P W_c \tag{2.1}$$

in which:

$F_p$ is the seismic force applied to a component of a building at its center of gravity;

$A_v$ is the seismic coefficient representing effective peak-velocity related acceleration from Sec. 1.4.1 of [7];

$C_c$ is the seismic coefficient for architectural components from Table 8.2.2 of [7];

$P$ is the performance criteria factor from Table 8.2.2 of [7]; and

$W_c$ is the weight of the architectural component.

The force $(F_p)$ is applied independently at vertical, longitudinal, and lateral directions in combination with the static load of the element.

#### 2.1.2 Mechanical and Electrical Components

Mechanical and electrical components and systems are designed for seismic forces determined in accordance with the following equation:

$$F_p = A_v C_c P a_c W_c \tag{2.2}$$

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in which:

$F_p$ is the seismic force applied to a mechanical or electrical component at its center of gravity;

$C_c$ is the seismic coefficient for mechanical and electrical components from Table 8.3.2a of [7];

$P$ is the performance criteria factor from Table 8.3.2a of [7];

$a_c$ is the amplification factor determined in accordance with Table 8.3.2b of [7]; and

$W_c$ is the operating weight of the mechanical or electrical component or system.

Alternatively, the seismic forces $(F_p)$ can be determined by a properly substantiated dynamic analysis subject to approval by the building code official.

### 2.1.3   Comments on the Seismic Coefficient ($C_c$)

The specification of the component seismic coefficient $(C_c)$ in Eq. (2.1) was originally based on the use of the working stress design and was similar to the $C_p$ factors specified in the UBC (1991) [13] and Title 24 of the California Administrative Code [24]. The values of $C_p$ in the UBC (1991) [13] were set basically by professional judgement, primarily from examining the performance of equipment in past earthquakes. Some results from analyses of linear elastic multistory buildings were used to justify general relationships among the various values. Additional capacity and ductility reservations were permitted in the assignment of the $C_p$ values [20]. Based on these observations, the determination of the $C_c$ values is basically a subjective result from experts in the related fields and their assigned values are somewhat arbitrary.

For the design of the mechanical and electrical equipment, the seismic coefficient $(C_c)$ in Eq. (2.2) was originally introduced to represent an amplification of the effective peak acceleration coefficient for coefficient $A_v$ equal to or greater than 0.2 ([7], Part 2, p. 183). In order to bring $C_c$ into conformance with other sections of the provisions. The concept was changed by defining $C_c$ as a numerical dimensionless factor related to that for mechanical and electrical components in Table 8.3.2a of [7]. The values presented in the table were developed by adopting an analogy to the $C_p$ values in Table T17-23-3 of Title 24 of the California Administrative Code [24] with the consistent performance criteria level of the 1991 NEHRP Provisions taken into account.

To sum up, the component seismic coefficient $(C_c)$ in Eqs. (2.1) and (2.2) appears to be a subjectively-assigned, reflecting in part the component performance during past

strong earthquakes. The determination of this value may involve many factors, including the attachment and constraint detailing effect, transient characteristics of the seismic input, uncertainties in the determination of amplification factor $(a_c)$, interaction between the component and its supporting structure, soil property and structural behaviors, etc. Based on different interpretations of these effects on the determination of $C_c$, three approaches are proposed in this report to revise formulation of the seismic design force for nonstructural components.

## 2.2 Shortcomings of Current Provisions

A design provision provides minimum legal design requirements for building structures. Without sacrificing simplicity for applicability to practical design, the design guidelines should reflect the state-of-the-knowledge as well as accumulated experience. As mentioned in the Introduction, the 1991 NEHRP Provisions in the area of nonstructural components have not kept in pace with current understanding of their seismic behavior through analyses, experiments, and field observations. This implies inadequacies of the current provisions in providing basic design requirements such as the minimum design forces. The major deficiency in the design force formulas for nonstructural components appears to be the negligence of their supporting structural behaviors. In particular, effects of soil type, structural period, component location and structural yielding on the nonstructural element design are not included. The anchorage detailing of the nonstructural component might be implied in the seismic coefficient $(C_c)$ but has not been interpreted on physical grounds.

### 2.2.1 Soil Type Effect

In the current provisions, the design force on the nonstructural element attached to a building structure is considered to be independent of the soil type, as can be seen in Eqs. (2.1) and (2.2). Obviously, this is inconsistent with design requirements for building structures. As evidenced in the design of a building structure [7,13], soil condition has a great influence on the design base shear force for the building structure. The softer the soil layer on which the building structure is constructed, the more stringent the design requirement for the building structure. Since the seismic input to a nonstructural element is the dynamic response at its supporting locations in the building structure, its seismic response clearly depends on the properties of soils supporting the building structure. Furthermore, effects of site conditions on the design for building structures have also been amply demonstrated by past earthquake observations [26].

### 2.2.2 Location Effect

According to current provisions, nonstructural components located at different floors of a supporting structure are designed for the same level of force. In practice, however, nonstructural elements attached to different floors of the supporting building structure will experience different levels of acceleration. The floor amplification effect can be observed from past earthquakes as well as from analytical results as shown in Section 3.2. The inclusion of the floor amplification effect in the design force formulas of the two earlier editions of NEHRP (1985 [5] and 1988 [6]) also demonstrates the need for distinguishing design forces on nonstructural elements situated on different floors, though the effect was discarded in the 1991 edition in favor of a somewhat arbitrarily defined $C_c$ coefficient.

### 2.2.3 Structural Period Effect

For the design of architectural components, the design force $(F_p)$ in Eq. (2.1) is not considered to be related to the structural period $(T_s)$. However, design forces on the architectural components obviously require the distinction between flexible and stiff building structures as can be seen in the base shear force formula for building structural design. For instance, a flexible structure may receive a small amount of inertia force so that an architectural component or mechanical equipment rigidly mounted to the structure are only slightly excited. As discussed in Section 3.2.3, the distribution of the acceleration or inertia force along the building height is also different for building structures with different periods.

The same is also true of the structural period effect on the design force for mechanical and electrical equipment, Eq. (2.2). Although the structural period $(T_s)$ is incorporated into the amplification factor $(a_c)$ in Eq. (2.2), it accounts for only partial effect that structural period has on the equipment response.

### 2.2.4 Structural Yielding Effect

Effects of structural yielding on the structural and nonstructural component design are considerably different. A building structure may experience inelastic deformation during severe earthquakes and, indeed, yielding of the building structure is considered as a limit state for its own design in the current provisions. However, this may not be the case for the design of nonstructural components. Failure of a nonstructural component attached to a building structure situated in a high seismic zone can occur under either of the following two cases: (1) the structure behaves inelastically under the maximum earthquake input at

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the site; or (2) the structure remains elastic during a small or moderate earthquake input at the same site. Yielding of the building structure will absorb a substantial amount of energy and hence reduce the seismic force imposed on the structure, however, the seismic input energy into the nonstructural component depends both on the direct seismic energy through seismic inertia force and the transferred energy from the supporting building structure. Under a strong earthquake, the direct seismic energy from the earthquake may be large but the transferred energy from the supporting structure is usually small due to structural yielding. On the other hand, the direct seismic energy under a small or moderate earthquake is relatively small and the transferred energy can be relatively large so that the total energy received by the nonstructural component might be greater than that under a strong earthquake.

### 2.2.5   Anchorage Detailing Effect

Finally, anchorage detailing of nonstructural components can also have a significant influence on the design forces. As has been observed, damage of a nonstructural component in many cases results from the failure of its anchorage due to excessive stress. An appropriate design for anchorage detail can thus significantly improve the nonstructural component's performance during earthquakes. For example, the introduction of ductility capacity in the anchorage will reduce the design force.


These observations have led to three recommendations for the revision of 1991 NEHRP Provisions for architectural, mechanical, and electrical components as described in the following sections.

# SECTION 3

# FIRST RECOMMENDATION

In this approach, the value of $C_c$ is considered to be affected by the interaction between the component and its supporting structure, soil properties, structural behavior, uncertainties in determination of the amplification factor, and transient characteristics of seismic input as well as the equipment detailing effect or yielding potential of mechanical or electrical equipment.

It is noted that the first four factors discussed in the shortcomings of nonstructural component design formulas outlined in Section 2.2 have been incorporated into the base shear of structural design in the current provisions. It is thus natural to take into account these effects in mechanical or electrical equipment design by transferring the vertical distribution of the base shear along the building structural height to the equipment.

## 3.1 Suggested Revision of the Design Force

### 3.1.1 Design Force Equation

Mechanical and Electrical Equipment. It is proposed that mechanical and electrical components and system be designed for seismic force determined in accordance with the following equation:

$$F_p = C_p W_c \tag{3.1}$$

in which $F_p$ and $W_c$ are, respectively, the seismic force applied to a nonstructural component at its center of gravity and the weight of the component as defined in the current provisions, and $C_p$ is the seismic design coefficient of the nonstructural component which can be calculated from

$$C_p = \frac{A_v a_x a_c P}{R_s R_c} \tag{3.2}$$

where $a_x$ is the floor amplification factor which can be determined by (see Fig. 3-1)

$$a_x = 1 + \frac{h_x}{h_n}(a_n - 1) \tag{3.3}$$

with

$$a_n = 1.5\beta_s \geq 1 \tag{3.4}$$

and

$$\beta_s = \frac{1.2S}{T_s^{2/3}} \leq \frac{2.5A_a}{A_v} \qquad (3.5)$$

in which $S$ is the site coefficient from Table 3-2 in [7], $A_v$ and $A_a$ are, respectively, the effective peak velocity-related acceleration and the effective peak acceleration from Section 1.4.1 of [7], and $T_s$ is the structural period determined in Section 4.2.2 of [7].

The factor $a_c$ in Eq. (3.2), a function of period ratio $(T_c/T_s)$ between equipment and its supporting structure, is the equipment amplification factor which can be calculated by (see Fig. 3-2)

$$a_c = \begin{cases} 1.0, & T_c/T_s \leq 0.5 \text{ and } T_c/T_s \geq 2.0 \\ 2.5, & 0.7 \leq T_c/T_s \leq 1.4 \\ -2.75 + 7.5(T_c/T_s), & 0.5 < T_c/T_s < 0.7 \\ 6 - 2.5(T_c/T_s), & 1.4 < T_c/T_s < 2.0 \end{cases} \qquad (3.6)$$

The factor $R_s$ in Eq. (3.2) is the response modification coefficient for component design due to structural yielding which can be estimated from the response modification coefficient for structural design $(R)$ in Table 3-3 of [7] by

$$R_s = 1 + \frac{R-1}{7} \qquad (3.7)$$

Its values are tabulated in Table 3-1 in this report for different types of buildings. In Eq. (3.2), the performance criteria factor $(P)$ takes different values than those in the current provisions as shown in Table 3-2 and $R_c$ is the response modification coefficient due to component yielding which is tentatively presented in Table 3-2.

Architectural Components.   For architectural components, the design force $(F_p)$ is again expressed by Eq. (3.1) except that the equipment amplification factor $(a_c)$ is assigned to be 1.0 and the performance criteria factor $(P)$ and the response modification coefficient $(R_c)$ take different values as presented in Table 3-3.

### 3.1.2  Development of the Design Force Equation

In this section, the formulation of $C_p$ in Eq. (3.2) is discussed in detail.  Since $C_p$ is closely related to the base shear and force distribution of the supporting structure, formulas for their calculations are also given.

Base Shear Force for Structure.   The seismic base shear force $(V)$ of the building structure in a given direction is provided by Eq. (4-1) in [7], i.e.,

$$V = C_s(W_s + W_c) \qquad (3.8)$$

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in which $W_s$ is the total dead load and applicable portion of other loads on the building structure and the seismic design coefficient $C_s$ can be written as (Eq. (4-2) of [7])

$$C_s = \beta_s \cdot \frac{A_v}{R_s} \tag{3.9}$$

Here, $R_s$ is the modified response modification coefficient ($R$) for building structures defined in Table 3-3 of [7]. This modification is necessary because the reserve capacity for building structure design due to nonstructural element constraint and unnecessary redundancy no longer exists for nonstructural element design and the ductility capacity for structural design is not totally transferable to the nonstructural component design. A very simple modification would be to shrink the response modification coefficient ($R$'s) for structural design into the range from 1.0 to 2.0 for nonstructural component design as given in Eq. (3.7). The quantity $\beta_s$ is a coefficient which is actually related to the seismic response spectrum as given by Eq. (3.5), which is derived from Eq. (4-2) of [7].

Vertical Distribution of Base Shear Force.    As shown in Fig. 3-3, the total base shear force ($V$) is balanced by the seismic force on mechanical and electrical equipment ($F_p$) and the inertia force on building floors ($F_i$, $i = 1, 2, \ldots, n$), i.e.,

$$V = F_p + \sum_{i=1}^{n} F_i \tag{3.10}$$

Assuming that the inertia force acting on a given floor is proportional to the floor height multiplied by its floor weight as in the structural design in the current provisions, the inertia force acting on the floor at $h_x$ level can be then formulated as

$$F_x = C_{vx}(V - F_p) \tag{3.11}$$

in which the vertical distribution factor ($C_{vx}$) can be expressed as

$$C_{vx} = \frac{W_x(h_x + h_0)}{\sum\limits_{i=1}^{n} W_i(h_i + h_0)} = \frac{W_x a_x}{\sum\limits_{i=1}^{n} W_i a_i} \tag{3.12}$$

and

$$h_0 = \frac{h_n}{a_n - 1} \tag{3.13}$$

as denoted in Fig. 3-1. The quantity $h_0$ is introduced because the acceleration distribution along building height is trapezoidal instead of triangular as will be illustrated in Section 3.2.

<u>Design Force on Equipment at $h_x$ Level.</u>  The floor acceleration can be calculated by dividing the lateral force $(F_x)$ by its floor mass.  This acceleration is amplified by $a_c$ to obtain the equipment acceleration which is then multiplied by equipment mass to arrive at the design force on the equipment attached to the floor at $h_x$ level, i.e.,

$$F_p = \left(\frac{F_x}{W_x}\right) a_c W_c \tag{3.14}$$

or

$$F_x = \frac{W_x F_p}{W_c a_c} \tag{3.15}$$

As one can see from Eqs. (3.11) and (3.12), the ratio of the inertia forces between the $i$th floor and the $x$th floor is

$$\frac{F_i}{F_x} = \frac{W_i a_i}{W_x a_x} \tag{3.16}$$

By substituting Eq. (3.15) for $F_x$ in Eq. (3.16), the inertia force on the $i$th floor $(F_i)$ can be expressed as

$$F_i = \frac{W_i a_i}{W_x a_x} F_x = \frac{W_i a_i}{W_c a_c a_x} F_p \tag{3.17}$$

Consequently, the seismic force on the equipment $(F_p)$ can be determined by introducing Eq. (3.17) into Eq. (3.10), giving

$$F_p = \frac{V}{1 + \frac{1}{a_c a_x} \sum\limits_{i=1}^{n} \frac{W_i}{W_c} a_i} = \frac{A_v a_c a_x W_c}{R_s} \alpha_1 \alpha_2 \tag{3.18}$$

in which

$$\alpha_1 = \frac{\beta_s}{\sum\limits_{i=1}^{n} \frac{W_i}{W_s} a_i} \tag{3.19}$$

$$\alpha_2 = \frac{\left(1 + \frac{W_c}{W_s}\right) \sum\limits_{i=1}^{n} \frac{W_i}{W_s} a_i}{\frac{W_c}{W_s} a_c a_x + \sum\limits_{i=1}^{n} \frac{W_i}{W_s} a_i} \tag{3.20}$$

where Eqs. (3.8) and (3.9) have been employed.

For a uniform building structure, i.e., $W_s = nW_1$ and $h_n = nh_1$, coefficients $\alpha_1$ and $\alpha_2$ can be respectively written as

$$\alpha_1 = \frac{2\beta_s}{a_n + 1 + (a_n - 1)/n} \tag{3.21}$$

$$\alpha_2 = \frac{\left(1 + \frac{W_c}{W_s}\right)\left(\frac{a_n+1}{2} + \frac{a_n-1}{2n}\right)}{\frac{W_c}{W_s} a_c a_x + \frac{a_n+1}{2} + \frac{a_n-1}{2n}} \tag{3.22}$$

When a moment-resistent frame structure with the estimated fundamental period $(T_s)$ equal to 0.1n sec (Eq. (4-4a) of [7]) is constructed on soil layer of type 1, coefficient $\alpha_1$ is plotted in Fig. 3-4 against structural period $T_s$. As one can see, $\alpha_1$ almost remains constant and is approximately equal to 0.9. Therefore, coefficient $\alpha_1$ can be approximately set to 1.0 for simple design purposes for this type of uniform structures.

Equation (3.22) shows that coefficient $\alpha_2$ is a function of mass ratio $W_c/W_s$, period ratio $T_c/T_s$ implied in $a_c$, and position $x$ expressed by $a_x$, which can be defined as an interaction factor. When $a_x \cong [a_n + 1 + (a_n - 1)/n]/2$, interaction factor $\alpha_2$ is plotted in Fig. 3-5 as a function of mass ratio $W_c/W_s$ for both detuned and tuned cases, indicating that seismic force acting on an equipment $(W_c = 0.1W_s)$ can be reduced by about 12% due to the interaction effect in the tuned case. Following simplicity requirements for practical design, the interaction effect can be neglected for light equipment and therefore the seismic force on the equipment is simply represented by

$$F_p = \frac{A_v a_x a_c W_c}{R_s} \tag{3.23}$$

Ductility Capacity of Equipment. As in structural design, the seismic force in Eq. (3.23) can be reduced by a factor $R_c$ for equipment design due to potential ductility capacity in the equipment anchorage, hence,

$$F_p = \frac{A_v a_x a_c W_c}{R_s R_c} \tag{3.24}$$

Performance Criteria Factor. By considering the performance criteria factor $(P)$ for equipment, the design force on the equipment can be finally formulated as

$$F_p = \frac{A_v a_x a_c P W_c}{R_s R_c} = C_p W_c \tag{3.25}$$

### 3.1.3 Performance Criteria Factor (P) and Response Modification Coefficient (R_c)

As illustrated by the derivations given above, characteristics of an integrated system consisting of nonstructural elements and building structure show that the nonstructural element performance during earthquakes can be considerably improved. Impact of the nonstructural element importance (or function) on the design of the nonstructural element should therefore be not as significant as that in the current provisions. The values of 0.8, 1.0, 1.2 for the performance criteria factor $(P)$ are thus suggested for architectural components as well as mechanical and electrical components with seismic

hazard exposure group I, II, and III, respectively. The specific values of $P$ for different nonstructural elements can be obtained by modification of the number in Tables 8-2-2 and 8-3-2(a) of [7] and are retabulated in Tables 3-2 and 3-3 here.

The response modification coefficients ($R_c$'s) can be determined by following two steps: (1) direct transfer of the seismic coefficients ($C_c$) in the current provisions to $\overline{R}_c$ and (2) modification of the obtained values ($\overline{R}_c$) to obtain $R_c$. Here, $\overline{R}_c$ accounts for effects of both structure and component on the design force of the nonstructural component; whereas $R_c$ takes into account the nonstructural component effects only. The relationship between $R_c$ and $\overline{R}_c$ can be simply established as

$$R_c = 1 + \frac{2}{3.5}(\overline{R}_c - 1)$$  (3.26)

which compresses the range (1 - 4.5) of $\overline{R}_c$ into the range (1 - 3) of $R_c$. The determination of $\overline{R}_c$ is discussed in Section 5.1.3.

### 3.1.4 Comments on Structural Yielding Effect

As pointed out in Section 2.2.4, structural yielding does not necessarily cause the maximum seismic force acting on a nonstructural component attached to the structure. This phenomenon is not reflected in the design force formula (Eq. (3.1)) for simplicity. If warranted, this effect can be included in the design force computation by adding the constraint

$$\frac{A_v}{R_s} \geq \min(A_v, A_{vy})$$  (3.27)

in which $A_{vy}$ denotes the peak velocity-related acceleration resulting in structural yielding. In an earthquake-prone area, $A_v$ could be larger than $A_{vy}$ and Eq. (3.27) basically means that the larger of the input accelerations of a structure in the inelastic state and in the initial yielding state should be used as the input for nonstructural component design.

## 3.2 Justifications

In formulating the recommended revision to the current provisions, efforts have been made to incorporate into the revision the latest theoretical research results and available experimental results as well as building response observation data and reconnaissance reports of recent earthquakes.

### 3.2.1 Theoretical Analyses

Modal Shapes and Mass Distribution Factor.   For a uniform moment-resisting frame structure as shown in Fig. 3-1, the equation of motion is given by

$$\mathbf{M}\ddot{\mathbf{y}}(t) + \mathbf{C}\dot{\mathbf{y}}(t) + \mathbf{K}\mathbf{y}(t) = -\mathbf{M}\mathbf{e}\ddot{x}_g(t) \tag{3.28}$$

in which $\mathbf{M}$, $\mathbf{C}$, and $\mathbf{K}$ are the mass, damping, and stiffness matrices of an $n$-story structure; $\mathbf{e}$ is the index vector of earthquake input; and $\mathbf{y}(t)$ is the relative displacement of the structure with respect to the ground. The fundamental period of the structure can be analytically calculated by [8]

$$T_s = \frac{T_0}{2\sin\left(\frac{\pi}{2(2n+1)}\right)} \tag{3.29}$$

in which $T_0$ is the period of its associated one-story building. The mode shape associated with the first mode can be analytically formulated as

$$\phi(n) = 1 \tag{3.30}$$

$$\phi(n-1) = 1 - \lambda_1 \tag{3.31}$$

$$\phi(i) = (2 - \lambda_1)\phi(i+1) - \phi(i+2), \quad i = 1, 2, \ldots, n-2 \tag{3.32}$$

where

$$\lambda_1 = \left(\frac{T_0}{T_s}\right)^2 \tag{3.33}$$

The first mode shape is shown in Fig. 3-6 for buildings with different numbers of stories $(n)$. It can be observed that the mode shape of a frame system is quite stable as the number of story increases and can be approximated by a straight line.

When the first mode shape is assumed to be a straight line, the $i$th element of mode shape vector $\phi$ can be simply expressed as

$$\phi(i) = \frac{i}{n} \tag{3.34}$$

and the modal mass and participation factor can be formulated as

$$m_1 = \phi^T \mathbf{M}\phi = m \sum_{i=1}^{n} \left(\frac{i}{n}\right)^2 = \frac{m(n+1)(2n+1)}{6n} \tag{3.35}$$

$$\phi \mathbf{M}\mathbf{e} = m \sum_{i=1}^{n} \frac{i}{n} = \frac{m(n+1)}{2} \tag{3.36}$$

$$\Gamma_1 = \frac{\boldsymbol{\phi}^T \mathbf{M} \mathbf{e}}{\boldsymbol{\phi}^T \mathbf{M} \boldsymbol{\phi}} = \frac{3n}{2n+1} \tag{3.37}$$

The mass distribution factor $\phi(n)\Gamma_1$ can then be formulated as

$$\phi(n)\Gamma_1 = \frac{3n}{2n+1} \tag{3.38}$$

which approaches 1.5 for large $n$, i.e., the factor used in Eq. (3.4).

<u>Absolute Acceleration or Inertia Force Distribution over Building Height</u>. The equation of motion of the first-mode representation of a multi-degree-of-freedom (MDOF) building structure can be expressed by

$$\ddot{y}_x(t) + 2\xi_1\omega_1\dot{y}_x(t) + \omega_1^2 y_x(t) = -\phi(x)\Gamma_1\ddot{x}_g(t) \tag{3.39}$$

in which $y_x(t)$ is the relative displacement of the structure at $h_x$ level and can be further expressed by

$$y_x(t) = \phi(x)\Gamma_1 q(t) \tag{3.40}$$

$$\ddot{q}(t) + 2\xi_1\omega_1\dot{q}(t) + \omega_1^2 q(t) = -\ddot{x}_g(t) \tag{3.41}$$

The absolute acceleration of the structure at $h_x$ level can thus be formulated as

$$\ddot{y}_x(t) + \ddot{x}_g(t) = \phi(x)\Gamma_1[\ddot{q}(t) + \ddot{x}_g(t)] + [1 - \phi(x)\Gamma_1]\ddot{x}_g(t) \tag{3.42}$$

from which one can readily observe that vertical distribution of the absolute acceleration is trapezoidal if the first mode shape $\phi(x)$ is assumed to be a straight line. At the base of the building structure, $\phi(x) = 0$ and $\ddot{y}_x(t) + \ddot{x}_g(t) = \ddot{x}_g(t)$, which is exact. The floor amplification factor $(a_0)$ in this case is equal to unity. At the top of the structure, the first term in Eq. (3.42) is predominant and therefore the floor amplification factor $a_n$ is approximately equal to the acceleration response spectrum calculated by Eq. (3.41) multiplied by a mass distribution factor $\phi(n)\Gamma_1$. This is the theoretical foundation of Eq. (3.4).

<u>Amplification Factor $(a_c)$ and Mass Ratio Effect</u>. The equation of motion for a coupled system with a single-degree-of-freedom (SDOF) representation of the equipment and a SDOF representation of the MDOF structural system can be written as

$$m_x[\ddot{y}_x(t) + 2\xi_1\omega_1\dot{y}_x(t) + \omega_1^2 y_x(t)] - m_c[2\xi_c\omega_c\dot{z}(t) + \omega_c^2 z(t)] = -\phi(x)\Gamma_1 m_x\ddot{x}_g(t) \tag{3.43}$$

$$m_c[\ddot{z}(t) + 2\xi_c\omega_c\dot{z}(t) + \omega_c^2 z(t)] = -m_c[\ddot{y}_x(t) + \ddot{x}_g(t)] \tag{3.44}$$

in which the modal mass of the structure represented by the relative displacement at $h_x$ level is calculated by

$$m_x = \frac{\boldsymbol{\phi}^T \mathbf{M} \boldsymbol{\phi}}{\phi^2(x)} \tag{3.45}$$

$m_c$, $\xi_c$, and $\omega_c$ are the equipment mass, damping ratio, and frequency, respectively, and $z(t)$ is the relative displacement of the equipment with respect to the building floor at $h_x$ level.

The root-mean-square ratio between absolute accelerations of the equipment and the first mode representation of a six-story uniform moment-resisting frame structure subjected to seismic excitation with the Kanai-Tajimi spectrum (an indication of amplification factor for equipment subjected to random loadings) is plotted in Fig. 3-7 as a function of the period ratio $(T_c/T_s)$ and for different mas ratios $(m_c/m_x)$ and different $\phi(x)\Gamma_1$'s due to different equipment locations. The Kanai-Tajimi spectrum has the form

$$S_{\ddot{x}_g}(\omega) = S_o \frac{1 + 4\xi_g^2 \left(\frac{\omega}{\omega_g}\right)^2}{\left(1 - \frac{\omega^2}{\omega_g^2}\right)^2 + 4\xi_g^2 \left(\frac{\omega}{\omega_g}\right)^2} \tag{3.46}$$

with parameters $S_0 = 1$, $\xi_g = 0.64$, $\omega_g = 15.6$ rad/sec [12].

Further examination of Eqs. (3.43) and (3.45) shows that both the mass ratio $(m_c/m_x)$ and the mass distribution factor $(\phi(x)\Gamma_1)$ increase when floor number on which the equipment is installed increases. The increase of the mass ratio means a reduction of the equipment amplification factor $(a_c)$ due to interaction effect while the increase of the mass distribution factor results in greater equipment amplification factor. The question as to whether the floor amplification factor $(a_x)$ and the equipment amplification factor $(a_c)$ in the design force formula can be separately considered is therefore posed here. A very preliminary conclusion can be drawn from Fig. 3-7, which shows that the equipment amplification factor $(a_c)$ can be approximately determined independent of location of the equipment. This supports the derivation process for the design force in the suggested approach.

Uncertainty Effect on the Determination of Amplification Factor $(a_c)$. In the above, the equipment amplification factor $(a_c)$ of a six-story building structure has been evaluated individually. For design purposes, however, a simpler but more general formulation for $a_c$

is needed. The simplest model for $a_c$ would include the determination of both amplitude and broadened band associated with the equipment response spectrum in the tuned case.

The response of mechanical or electrical equipment located on ground or on a very rigid structure is mainly a function of the frequency content of the postulated earthquake, whereas the response of the equipment attached to a relatively flexible structure is mainly a function of the structure's natural periods. The supporting structure in this case acts as a filter amplifying the seismic motion at its own natural periods. The statistical characteristics of the equipment response during a seismic disturbance in the first case can be well described by the design response spectrum value $(\beta_s)$, while the equipment response on a flexible structure can be simply described as the harmonic response oscillating at the fundamental structural period. The reality for the determination of $a_c$ is between the above two extreme cases. Based on these observations, we consider the amplitude of $a_c$ for all period ratios between the equipment and the structure to be not less than 2.5.

Due to uncertainties involved in the structural parameters such as mass, stiffness, and damping ratio, the peak value of $a_c$ (commonly called floor response spectrum) needs to be broadened for design purposes. According to the analyses performed for nuclear power plant design [1], coefficients of variation (Cov) of 5-10% for mass determination and about 34% for stiffness determination are appropriate. In what follows, coefficients of variation of the structural period and the equipment period are simply evaluated by assuming perfect correlation between masses at different floors or between stiffnesses at different floors. In this case, the fundamental period of the structure can be calculated by Eq. (3.29) but $T_0$ in the formula is a random variable.

When mass and stiffness of a uniform structure are considered as two independent lognormal random variables, the coefficient of variation for its fundamental period, $\text{Cov}(T_s)$, can be calculated from the coefficient of variation of mass, $\text{Cov}(M)$, and of stiffness, $\text{Cov}(K)$, by the following equation:

$$\text{Cov}(T_s) = \sqrt{[1 + (\text{Cov}(M))^2]^{1/4}[1 + (\text{Cov}(K))^2]^{1/4} - 1} \qquad (3.47)$$

Substituting $\text{Cov}(M) = 0.10$ and $\text{Cov}(K) = 0.34$ into Eq. (3.47), we can calculate the coefficient of variation for the structural period, i.e., $\text{Cov}(T_s) = 0.174$. The structural period $(T_s)$ in this case is also lognormal. Assuming that the equipment period $(T_c)$ has the same coefficient of variation as the structural period but is independent of $T_s$, the coefficient of

variation of the period ratio between the equipment and the structure can be subsequently calculated as

$$\text{Cov}(T_c/T_s) = \sqrt{[1 + (\text{Cov}(T_c))^2][1 + (\text{Cov}(T_s))^2] - 1} \qquad (3.48)$$

$$= \sqrt{(1 + 0.174^2)^2 - 1} = 0.248 \qquad (3.49)$$

Considering nonuniformity of mass and stiffness distributions along the building height, imperfectly-correlated properties associated with masses or stiffnesses at different floors, possible higher-mode effects, and other uncertainty factors such as damping coefficients as well as the fact that both $\text{Cov}(K)$ and $\text{Cov}(M)$ might be larger for regular building structures, a coefficient of variation of 30% for the period ratio $(T_c/T_s)$ is suggested. That is, the peak value of $a_c$ can be broadened into the range of 0.7 to 1.3. However, sensitivity of the period ratio to uncertainties existed in the equipment-structure system is often stronger for flexible equipment than for stiff equipment and the amplification factor always skews toward the larger period ratio $(T_c/T_s)$ as illustrated in Fig. 3-7. The peak value is finally recommended to be broadened to the range of 0.7 to 1.4. From recent research on decoupling criteria between equipment and structure [8] and other related research results, the interaction effect outside the range of 0.5 to 2.0 for the period ratio is negligible. Consequently, the amplification factor $(a_c)$ can be calculated by Eq. (3.6) and is shown in Fig. 3-2.

Structure Yielding Effect.   Limited investigations on the structure yielding effect [9,22] consistently demonstrate a reduction of the dynamic response of an equipment when the supporting structure behaves inelastically during severe earthquakes. The introduction of the response modification coefficient $(R_s)$ is attributed to the different degrees of ductility capacities of structures. On the other hand, information on equipment response reduction due to structural yielding is less than that available for the structure itself [17]. It seems reasonable to define the available reduction coefficient $(R_s)$ for nonstructural component design as a value ranging from 1.0 to 2.0, which is Eq. (3.7).

### 3.2.2  Experimental Results

Compared with extensive analytical work on the structure-equipment system interaction, small-scale or full-scale experimental evidence seems to be scant [9]. Experimental results reported in [14-16,18] played a role in the development of the proposed procedure in this report.

The experimental model used in [15] was a five-story, one-third scale frame structure with first three periods of 0.308, 0.180, and 0.082 seconds. This model structure was subjected to four typical earthquake input signals based on records of historical California earthquakes, i.e., the El Centro 1940 NS component, the Pacoima Dam 1971 S16E component, the Taft 1950 S69E component, and the Parkfield 1966 N65E component. Each signal was run in real time and time-scaled by a factor of $\sqrt{3}$, which corresponds to the geometrical scale of the model. The response of the model to these time-scaled inputs should correspond to that of a full-scale structure to the historical earthquakes. Three oscillators, simple vertical cantilevers, were used in the test to simulate light equipment. Oscillators 1 and 2 were attached to the top floor and oscillator 3 was supported at the second floor of the model structure. Their vibrational periods were respectively taken as 0.308, 0.180, and 0.67 seconds, the first two of which were tuned to the first and second modes of the model structure.

The floor amplification factors $(a_x)$ at the top and second floors of the model structure as well as the equipment amplification factors $(a_c)$ of the three oscillators are calculated and tabulated in Table 3-4 for both real time and time-scaled signals. As one can see, the top floor amplification factor $(a_5)$ varies from a low of 2.7 for both the Pacoima Dam time-scaled and the Parkfield real time signals to a high of 4.8 for the El Centro and Parkfield time-scaled signals. These amplifications have a mean of 3.46 and a coefficient of variation of 0.276, which support the maximum value of 3.75 for $a_n$ in Eq. (3.4). Furthermore, the values of ratio $a_2/a_5$ in the last column of Table 3-4 have a mean of 0.639, which roughly agrees with the value 0.56 calculated from Eq. (3.3). The larger statistical value of $a_2/a_5$ from the experimental results may be due to the straight line assumption used for the first mode shape of the experimental structure in Eq. (3.3) as well as higher-mode contribution which is especially significant to the response at lower floors. It can also be observed that values of the equipment amplification factor $(a_c)$ have a mean of 2.996 for oscillator 1 tuned to the structural fundamental mode and a mean of 1.913 for oscillator 2 which is detuned to the fundamental mode but tuned to the second mode of the structure. The larger equipment amplification factor (2.431) for oscillator 3 over oscillator 2 further illustrates the higher-mode involvement in lower floor's response.

In the experiment reported in [18], a three-story, one-quarter scaled frame was used to model the building structure and a cantilevered damper was used to represent the component. The shaking table test results show that the interaction effect between structure and equipment is significant in the tuned cases and a numerical calculation scheme can predict equipment response that agrees well with the experimental results.

The floor response spectrum for a full-scaled equipment converted from the scaled model was calculated numerically for different equipment locations and is shown in Fig. 3-8. Obviously, the floor response spectrum strongly depends on the equipment location in tuned cases, which supports the proposed formula of the design force here.

### 3.2.3 Observations on Past Earthquakes

Many observations on the structural behavior during earthquakes have been conducted in the past two decades. Most of them were made in California during a few major earthquakes such as the San Fernando earthquake on February 9, 1971, the Whittier earthquake on October 1, 1987, and the Loma Prieta earthquake on October 17, 1989. These observation data are used here to perform a statistical analysis on the amplification factor $(a_n)$ for different types of buildings. Figure 3-9 presents the amplification factors $(a_n)$ for different types of buildings (steel frame, reinforced concrete frame, and R-C shear wall) with various structural periods $(T_s)$. It can be seen from this plot that the observed data are quite dispersive. This dispersion mainly results from the fact that the soil layer on which the structure is located and the seismic intensity to which the structure is subjected are not distinguished. Different degrees of structural yielding involved during these earthquakes may further complicate the distribution of the observed data. Nevertheless, the proposed floor amplification factor $(a_n)$ in Eq. (3.4) for soil conditions of type IV (solid line in Fig. 3-9) almost envelopes the dispersive observation data.

Figure 3-10 presents the same set of observed amplification factors $(a_n)$ as a function of building structural period $(T_s)$ relative to the different earthquake events. It can be seen that most observed data are from the more recent earthquakes such as the Loma Prieta earthquake.

Similar statistical analyses for the amplification factor have been conducted elsewhere [10,21] and the trapezoidal acceleration distribution along a building height has also been observed.

### 3.2.4 Related Design Codes

As mentioned in Section 2.2, the earlier NEHRP provisions (1985, 1988) employed an amplification factor $(a_x)$ to distinguish different degrees of response magnification when an equipment is installed at different floors of a building. The deletion of this factor in the current provisions only reinforces difficulties in reaching a good understanding of the component seismic coefficient $(C_c)$. As more experience data about component behavior

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during earthquakes are accumulated, a better understanding of the floor amplification factor $(a_x)$ can be achieved.

The Japanese code [11] for nonstructural component design also introduces the floor amplification factor $(a_x)$ as expressed in Eq. (3.3) but the amplification factor $(a_n)$ at the top of a building structure is bounded by a factor of 10/3 instead of $1.5 \times 2.5 (= 3.75)$ in this report, which is more justifiable. In addition, the expression for $a_n$ in the Japanese code is independent of soil conditions.

## TABLE 3-1. Response Modification Coefficients

| Basic Structural System and Seismic Force Resisting System | for Component Design, $R_s$ | for Structural Design, R |
|---|---|---|
| **Bearing Wall System** | | |
| Light frame walls with shear panels | 1.8 | 6.5 |
| Reinforced concrete shear walls | 1.5 | 4.5 |
| Reinforced masonry shear walls | 1.4 | 3.5 |
| Concentrically braced frames | 1.4 | 4.0 |
| Unreinforced masonry shear walls | 1.0 | 1.25 |
| **Building Frame System** | | |
| Eccentrically braced frames, moment resisting connections at columns away from link | 2.0 | 8.0 |
| Eccentrically braced frames, non-moment resisting connections at columns away from link | 1.9 | 7.0 |
| Light frame walls with shear panels | 1.9 | 7.0 |
| Concentrically braced frames | 1.6 | 5.0 |
| Reinforced concrete shear walls | 1.6 | 5.5 |
| Reinforced masonry shear walls | 1.5 | 4.5 |
| Unreinforced masonry shear walls | 1.0 | 1.5 |
| **Moment Resisting Frame System** | | |
| Special moment frames of steel | 2.0 | 8.0 |
| Special moment frames of reinforced concrete | 2.0 | 8.0 |
| Intermediate moment frames of reinforced concrete | 1.4 | 4.0 |
| Ordinary moment frames of steel | 1.5 | 4.5 |
| Ordinary moment frames of reinforced concrete | 1.1 | 2.0 |
| **Dual System with a Special Moment Frame Capable of Resisting at Least 25% Prescribed Seismic Forces** | | |
| Eccentrically braced frames, moment resisting connections at columns away from link | 2.0 | 8.0 |
| Eccentrically braced frames, non-moment resisting connections at columns away from link | 1.9 | 7.0 |
| Concentrically braced frames | 1.7 | 6.0 |
| Reinforced concrete shear walls | 2.0 | 8.0 |
| Reinforced masonry shear walls | 1.8 | 6.5 |
| Wood sheathed shear panels | 2.0 | 8.0 |
| **Dual System with an Intermediate Moment Frame of Reinforced Concrete or an Ordinary Moment Frame of Steel Capable of Resisting at Least 25% of Prescribed Seismic Forces** | | |
| Concentrically braced frames | 1.6 | 5.0 |
| Reinforced concrete shear walls | 1.7 | 6.0 |
| Reinforced masonry shear walls | 1.6 | 5.0 |
| Wood sheathed shear panels | 1.9 | 7.0 |
| **Inverted Pendulum Structures--Seismic Force Resisting System** | | |
| Special moment frames of structural steel | 1.2 | 2.5 |
| Special moment frames of reinforced concrete | 1.2 | 2.5 |
| Ordinary moment frames of structural steel | 1.0 | 1.25 |

**TABLE 3-2. Mechanical and Electrical Component and System Response Modification Coefficient ($R_c$) and Performance Criteria Factor (P)**

| Mechanical and Electrical Component or System | Response Modification Coefficient ($R_c$) | Performance Criteria Factor (P) [new(old)] Seismic Hazard Exposure Group | | |
|---|---|---|---|---|
| | | I | II | III |
| Fire protection equipment and systems | 1.1 | 1.2(1.5) | 1.2(1.5) | 1.2(1.5) |
| Emergency or standby electrical systems | 1.1 | 1.2(1.5) | 1.2(1.5) | 1.2(1.5) |
| Elevator drive, suspension system, and control anchorage | 1.6 | 1.0 | 1.0 | 1.2(1.5) |
| General equipment<br><br>Boilers, furnaces,incinerators, water heaters, and other equipment using combustible energy sources or high temperature energy sources chimneys, flues, smokestacks, and vents<br><br>Communication systems<br><br>Electrical bus ducts, conduit, and cable trays<br><br>Electrical motor control centers, motor control devices, switchgears, transformers, and unit substations<br><br>Reciprocating or rotating equipment<br><br>Tanks, heat exchangers, and pressure vessels<br><br>Utility and service interfaces | 1.1 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Manufacturing and process machinery | 2.7 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Pipe systems<br>Gas and high hazard piping<br>Fire suppression piping<br>Other pipe systems | <br>1.1<br>1.1<br>2.7 | <br>1.2(1.5)<br>1.2(1.5)<br>NR | <br>1.2(1.5)<br>1.2(1.5)<br>1.0 | <br>1.2(1.5)<br>1.2(1.5)<br>1.2(1.5) |
| HVAC and service ducts | 2.7 | NR | 1.0 | 1.2(1.5) |
| Electrical panel boards and dimmers | 2.7 | NR | 1.0 | 1.2(1.5) |
| Lighting fixtures | 2.7 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Conveyor systems (nonpersonnel) | 2.7 | NR | NR | 1.2(1.5) |

## TABLE 3-3. Architectural Component Response Modification Coefficient ($R_c$) and Performance Criteria Factor (P)

| Architectural Component | Response Modification Coefficient ($R_c$) | Performance Criteria Factor (P) [new(old)] Seismic Hazard Exposure Group | | |
|---|---|---|---|---|
| | | I | II | III |
| Exterior nonbearing walls | 2.1 | 1.2(1.5) | 1.2(1.5) | 1.2(1.5) |
| Interior nonbearing walls | | | | |
| Stair enclosures | 1.5 | 1.0 | 1.0 | 1.2(1.5) |
| Elevator shaft enclosures | 1.5 | 0.8(0.5) | 0.8(0.5) | 1.2(1.5) |
| Other vertical shaft enclosures | 2.1 | 1.0 | 1.0 | 1.2(1.5) |
| Other nonbearing walls | 2.1 | 1.0 | 1.0 | 1.2(1.5) |
| Cantilever elements | | | | |
| Parapets, chimneys, or stacks | 1.0 | 1.2(1.5) | 1.2(1.5) | 1.2(1.5) |
| Wall attachments (see Sec. 8.2.3) | 1.0 | 1.2(1.5) | 1.2(1.5) | 1.2(1.5) |
| Veneer connections | 1.0 | 0.8(0.5) | 1.0 | 1.0 |
| Penthouses | 3.0 | NR | 1.0 | 1.0 |
| Structural fireproofing | 2.1 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Ceilings | | | | |
| Fire-rated membrane | 2.1 | 1.0 | 1.0 | 1.2(1.5) |
| Nonfire-rated membrane | 3.0 | 0.8(0.5) | 1.0 | 1.0 |
| Storage racks more than 8 feet in height (content included) | 1.5 | 1.0 | 1.0 | 1.2(1.5) |
| Access floors (supported equipment included) | 1.1 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Elevator and counterweight guiderails and supports | 1.6 | 1.0 | 1.0 | 1.2(1.5) |
| Appendages | | | | |
| Roofing units | 3.0 | NR | 1.0 | 1.0 |
| Containers and miscellaneous components (free standing) | 1.5 | NR | 1.0 | 1.0 |
| Partitions | | | | |
| Horizontal exits including ceiling | 2.1 | 1.0 | 1.2(1.5) | 1.2(1.5) |
| Public corridors | 2.1 | 0.8(0.5) | 1.0 | 1.2(1.5) |
| Private corridors | 3.0 | NR | 0.8(0.5) | 1.2(1.5) |
| Full height area separation partitions | 2.1 | 1.0 | 1.0 | 1.2(1.5) |
| Full height other partitions | 3.0 | 0.8(0.5) | 0.8(0.5) | 1.2(1.5) |
| Partial height partitions | 3.0 | NR | 0.8(0.5) | 1.0 |

**TABLE 3-4. Floor and Equipment Amplification Factors of a Test Structure**

| signal | input | $a_5$(5th fl) | $a_2$(2nd fl) | $a_c$(osc. 1) | $a_c$(osc. 2) | $a_c$(osc. 3) | $a_2/a_5$ |
|---|---|---|---|---|---|---|---|
| real time | El Centro | 3.890 | 1.940 | 2.815 | 2.030 | 2.362 | 0.4986 |
| | Taft | 3.454 | 1.923 | 5.385 | 1.643 | 1.090 | 0.5567 |
| | Pacoima | 3.549 | 2.126 | 2.449 | 2.998 | 2.547 | 0.5990 |
| | Parkfield | 2.685 | 1.750 | 4.215 | 1.466 | 3.306 | 0.6519 |
| time-scaled | El Centro | 4.737 | 2.267 | 1.626 | 1.247 | 0.952 | 0.4785 |
| | Taft | 1.855 | 1.879 | 3.687 | 2.984 | 4.036 | 1.0129 |
| | Pacoima | 2.706 | 1.820 | 1.558 | 1.503 | 2.446 | 0.6728 |
| | Parkfield | 4.776 | 3.058 | 2.235 | 1.430 | 2.709 | 0.6403 |
| mean | | 3.460 | 2.095 | 2.996 | 1.913 | 2.431 | 0.639 |
| coefficient of variation | | 0.276 | 0.189 | 0.418 | 0.344 | 0.396 | 0.244 |
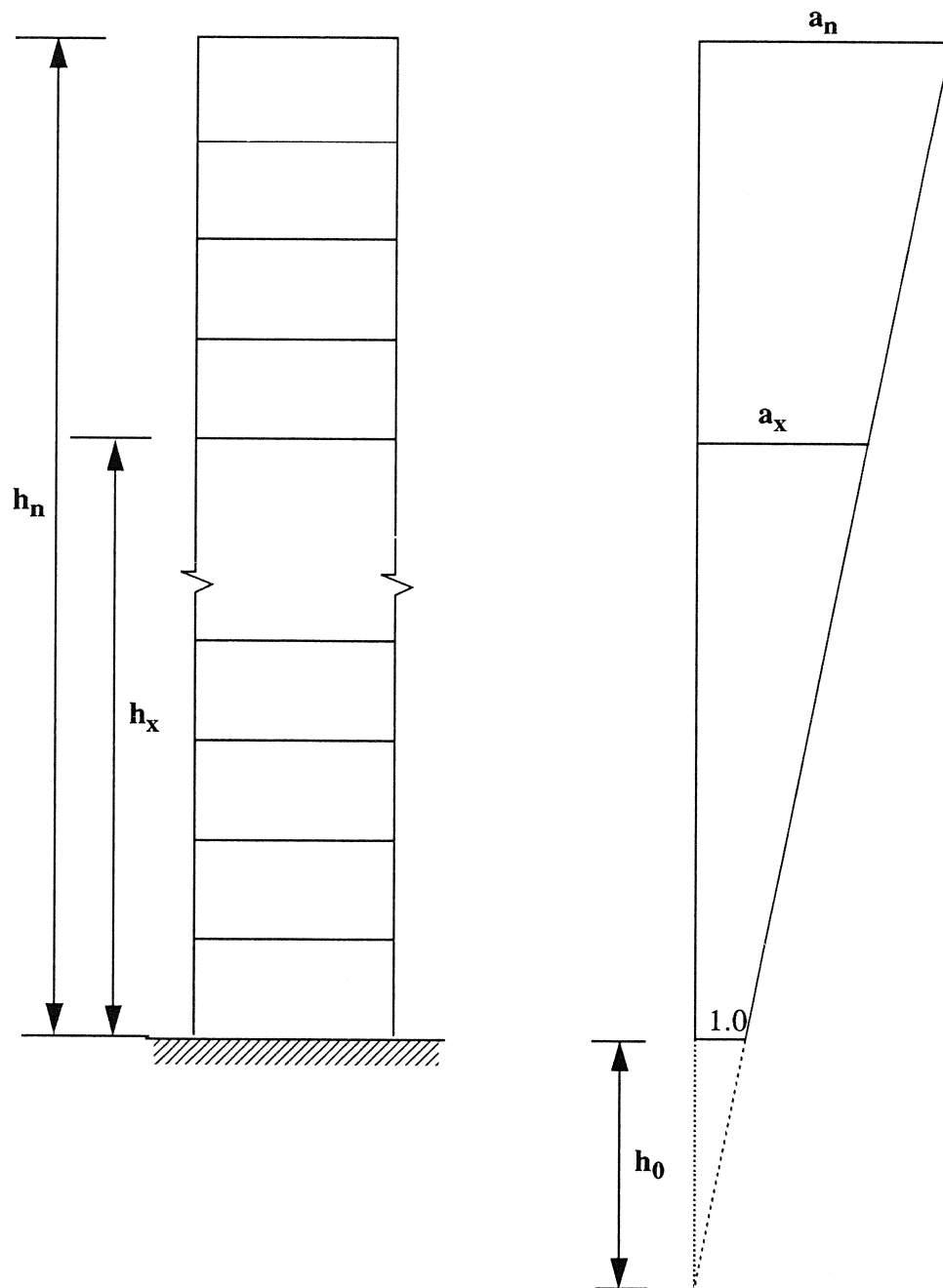
Fig. 3-1 Building Structure and Floor Amplification Factor
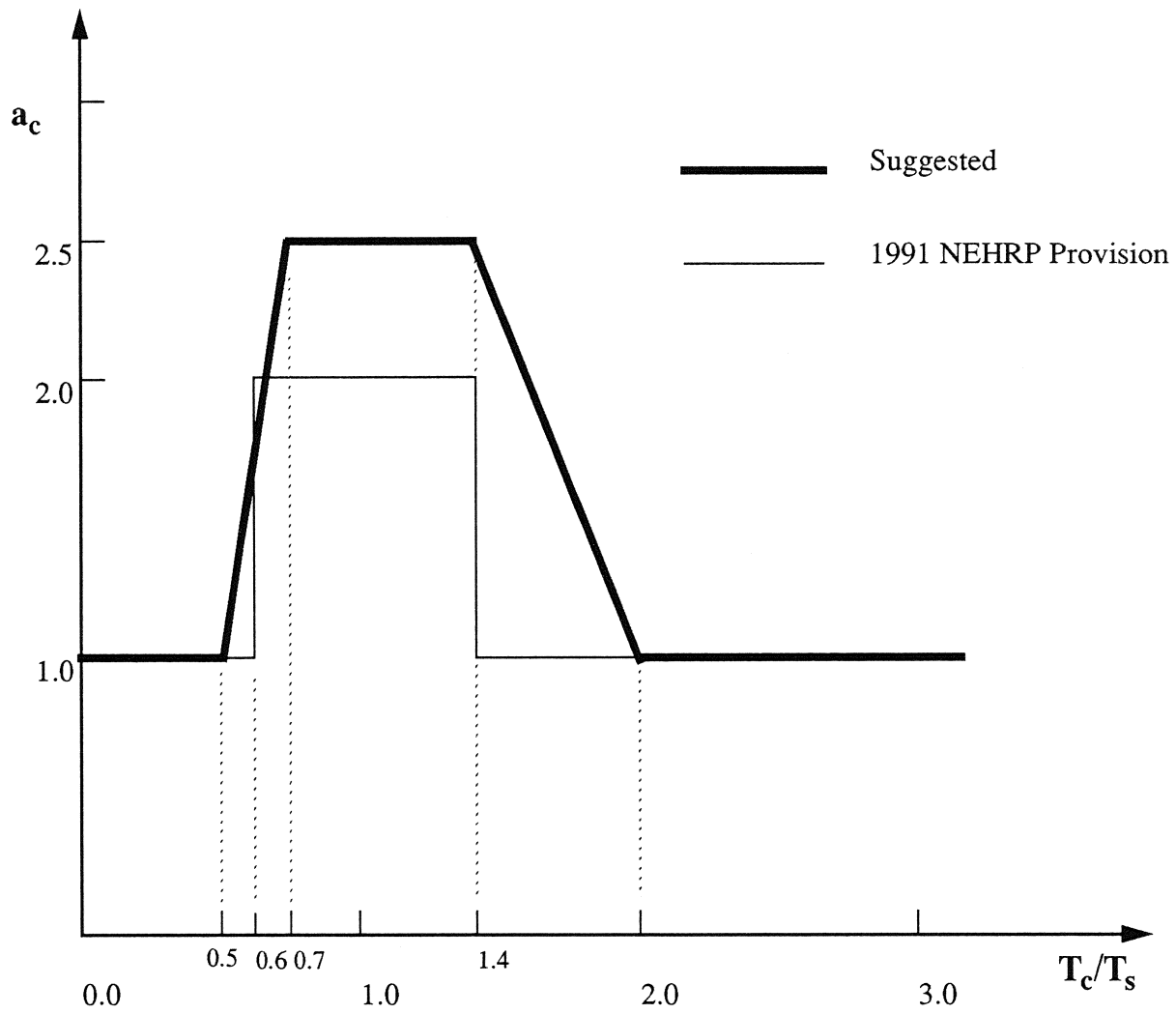
**Fig. 3-2 Equipment Amplification Factor**
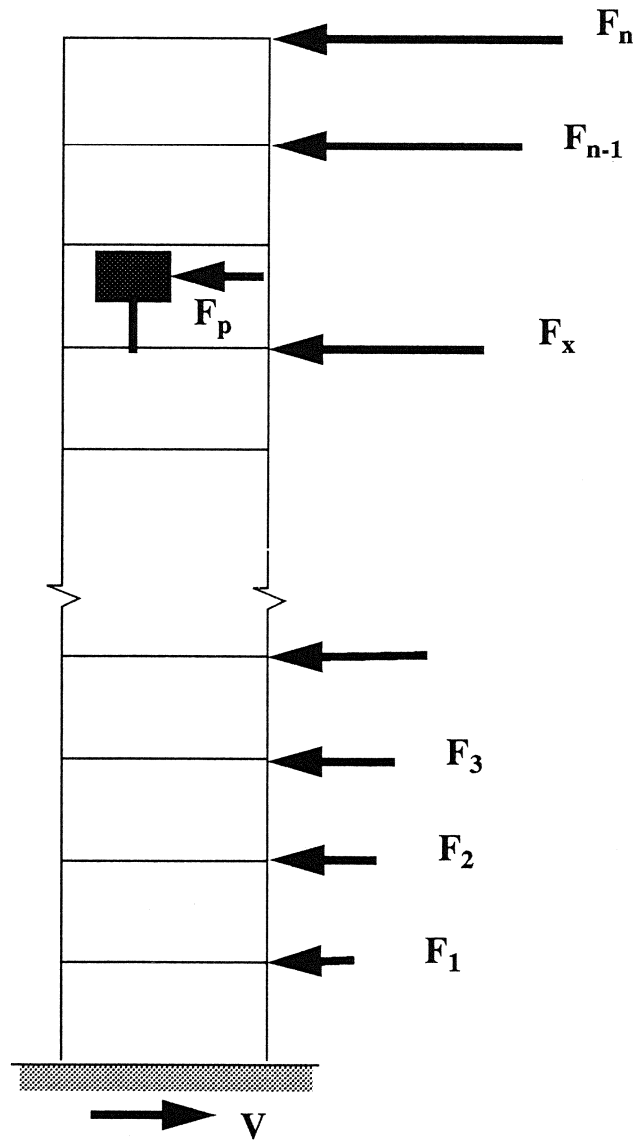
**Fig. 3-3 Force Balance Between Structure and Component**
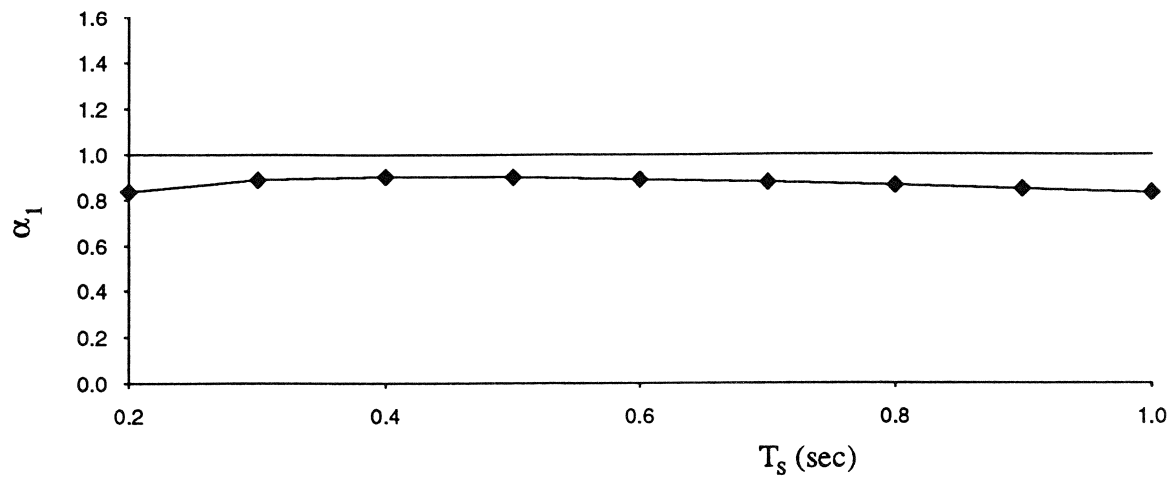
Fig. 3-4 Coefficient $\alpha_1$
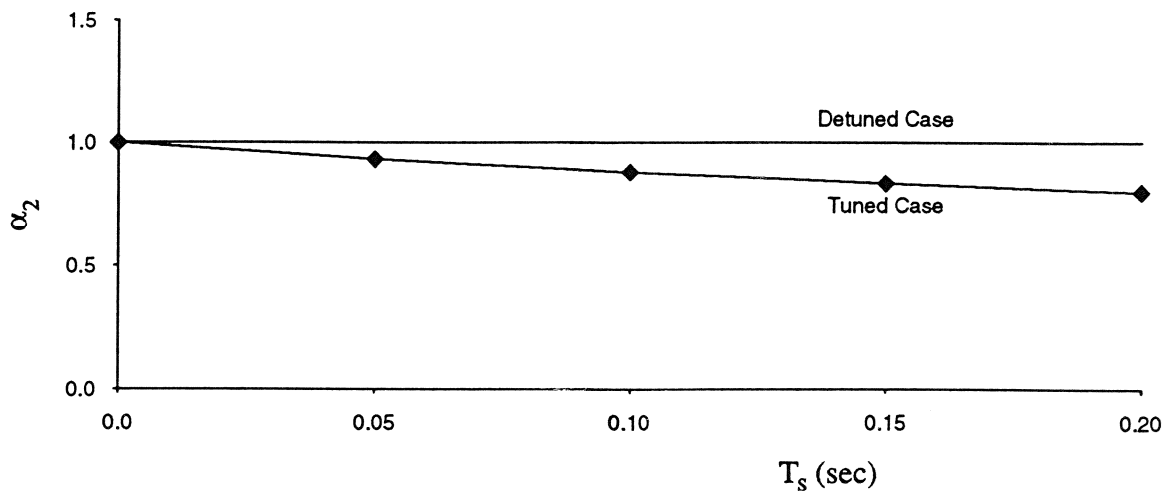


Fig. 3-5 Coefficient $\alpha_2$
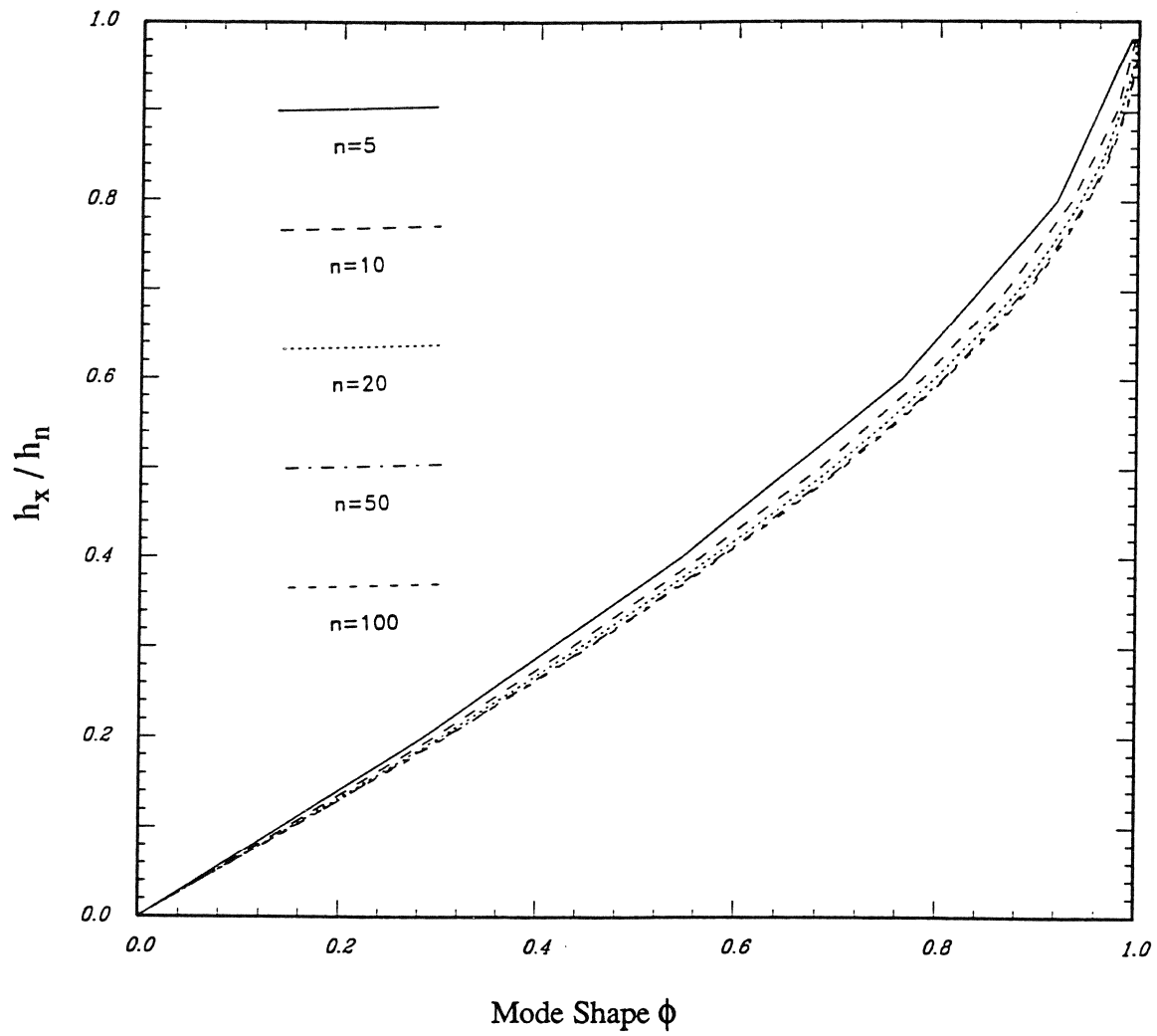
Fig. 3-6 First Mode Shape of a Uniform Moment-Resisting Frame Structure
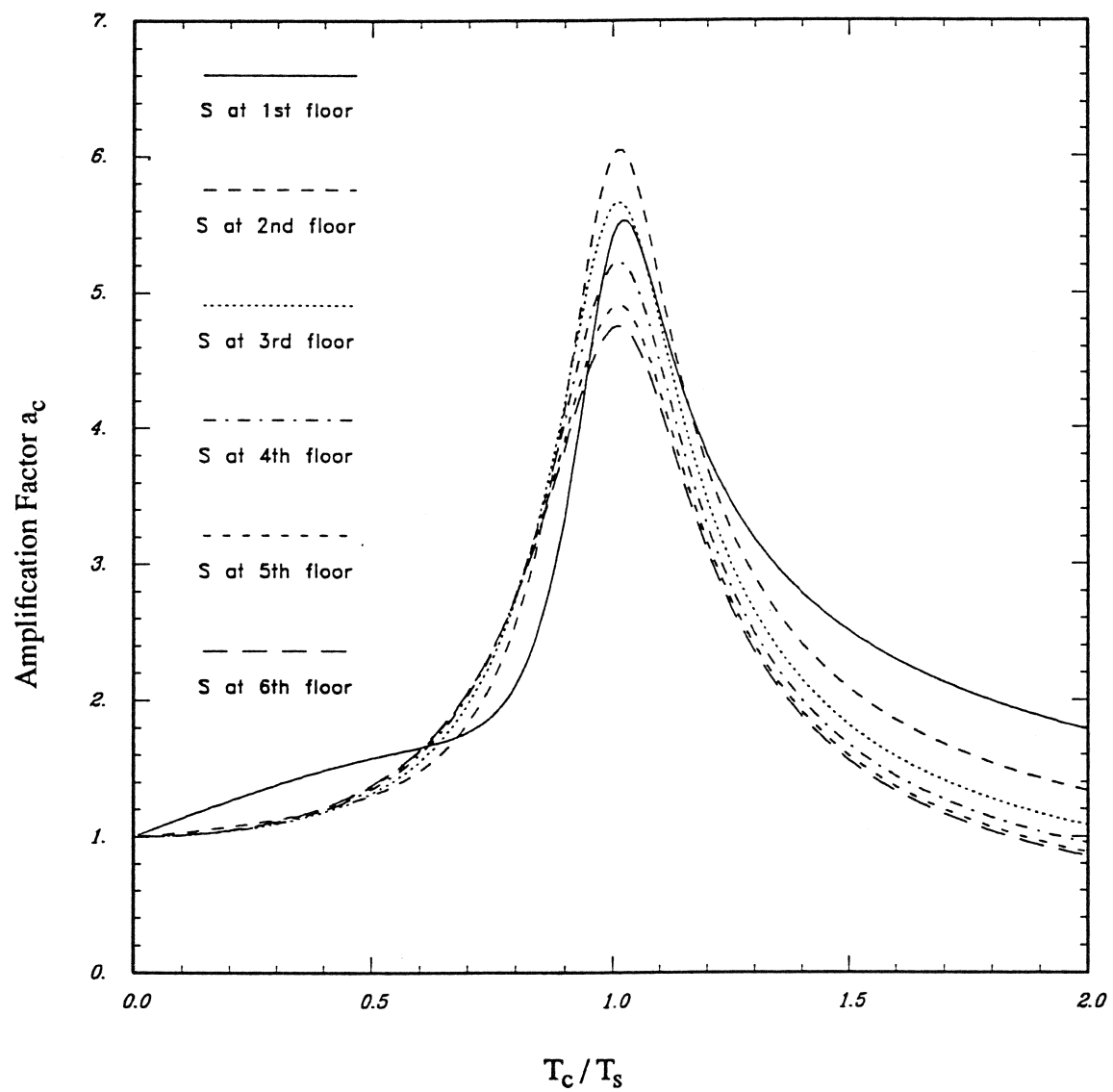
Fig. 3-7 Equipment Amplification Factor of a Six-Story Building:
5% Damping Ratios for Equipment and Building Structure
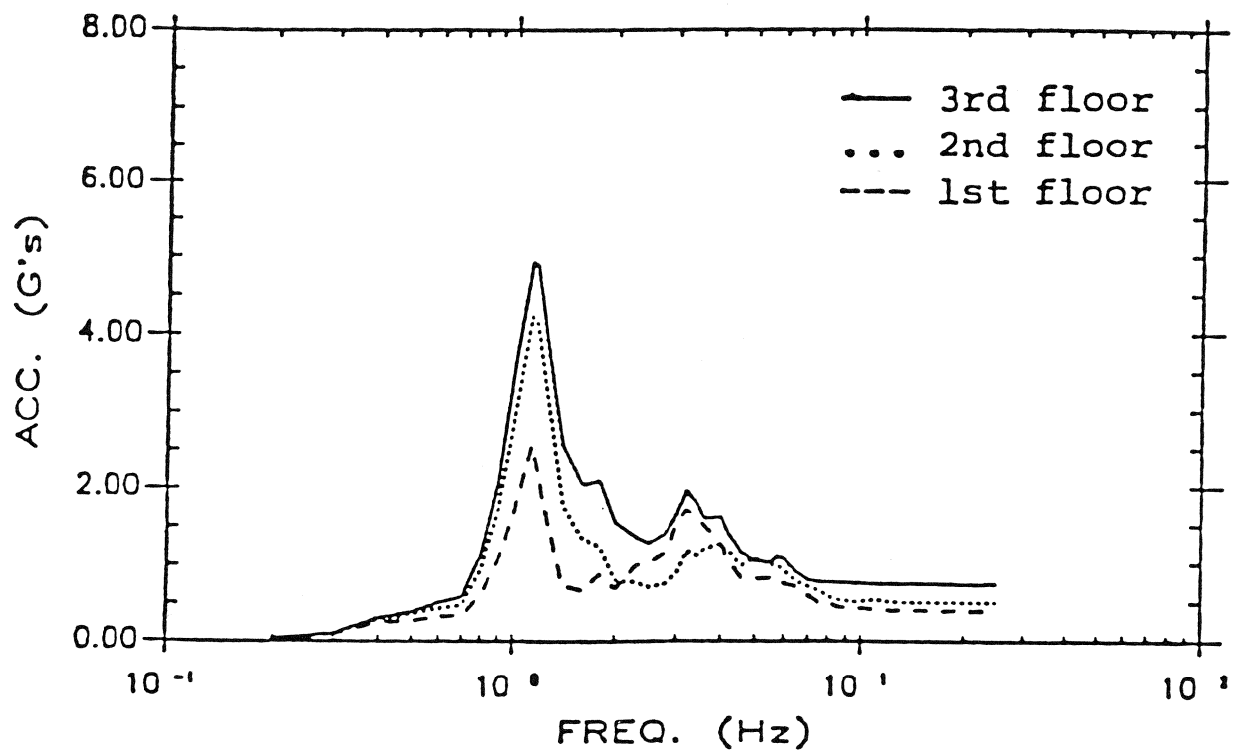
Fig. 3-8 Floor Response Spectrum for Secondary System Attached
to Full-Scale Frame Under El-Centro 1940 Earthquake:

Location Effect (Taken from [18])

Base to Roof Acceleration Amplification Factor--Observation from California Earthqaukes
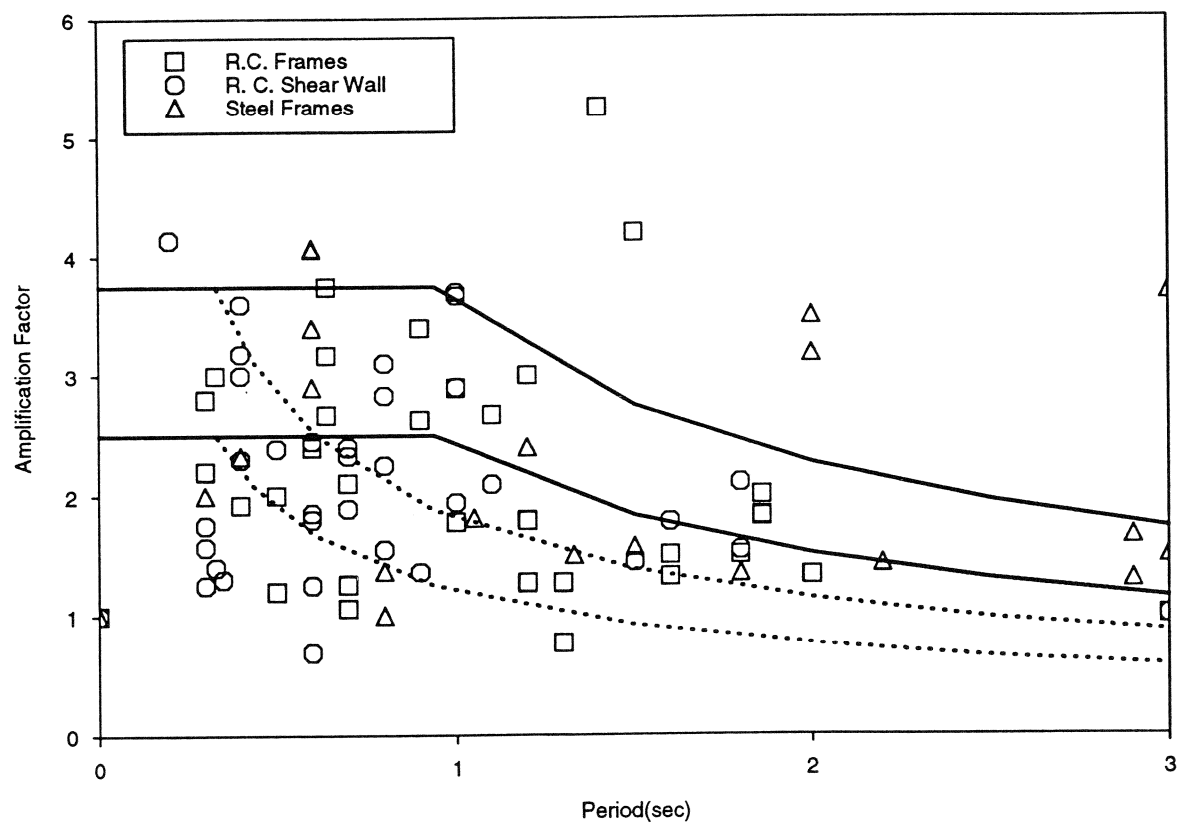


Fig. 3-9 Observation Data (Different Structures)

# Base to Roof Acceleration Amplification Factor--Observation from California Earthqaukes
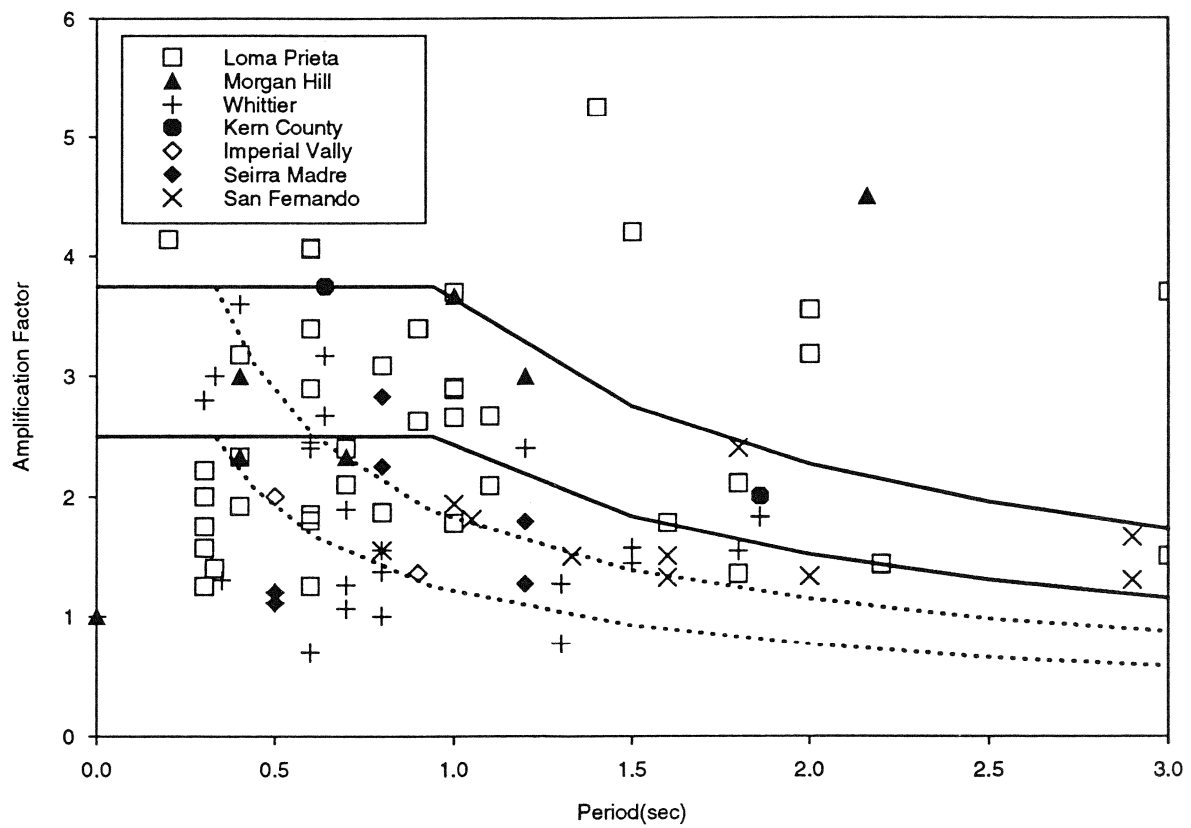


Fig. 3-10 Observation Data (Different Earthquakes)

# SECTION 4

## SECOND RECOMMENDATION

The integrated behavior of the nonstructural component and its supporting structure has been considered in the formulation of the first recommendation. Hence, explicit information concerning the component, such as its response modification coefficient $(R_c)$ and its period $(T_c)$ is required. In certain situations, component-specific information may not be available. Therefore, a completely structure-driven design methodology for nonstructural components is useful in practice, i.e., parameters such as $R_c$ and $T_c$ do not appear explicitly in the design force equation. This methodology is also supported by the argument that two different light equipment with equal weight, attached to a heavy building floor, will receive approximately the same amount of inertia force except for perfectly tuned cases which have a low probability of occurrence in practice.

By following the same procedure as in the first recommendation, the design force on a mechanical or electrical equipment can be required to satisfy

$$F_p = \frac{A_v a_x a_c P W_c}{R_s} \tag{4.1}$$

where

$$a_c = \begin{cases} 1, & \text{for rigidly-mounted equipment} \\ 2, & \text{for flexibly-mounted equipment} \end{cases} \tag{4.2}$$

which accounts for the tuning effect as well as the reduction effect due to potential component yielding. All the remaining factors in Eq. (4.1) are exactly the same as those in Eq. (3.1)

For the design of architectural components, coefficient $a_c$ in Eq. (4.1) is simply assigned to be unity and the performance criteria factor $(P)$ is taken from Table 3-3.

## SECTION 5

## THIRD RECOMMENDATION

In this approach, we consider equipment detailing such as specific anchorage design and constraints imposed by the supporting structure, possible ductility capacity in the equipment design, and the transient characteristics of input motion as having major contributions to the assignment of $C_c$. The supporting building structure effect on the design of mechanical and electrical equipment is considered to be implicitly covered in the determination of $C_c$ values and will not be considered elsewhere in this approach. Therefore, the response modification coefficient $(R_c)$, to be introduced below, is closely related to the $C_c$ values from Tables 8.2.2 and 8.3.2a in [7] for architectural component and equipment design but its values are more uniform than the $C_c$ values for the various components. This is due to the fact that other factors such as location effect on the design force have been sorted out from somewhat arbitrarily assigned $C_c$ values.

### 5.1  Suggested Revision of the Design Force

#### 5.1.1  Design Force Equation

<u>Mechanical and Electrical Equipment</u>.  In this approach, it is proposed that mechanical and electrical equipment and systems be designed for seismic force determined in accordance with the following equation:

$$F_p = C_p W_c \tag{5.1}$$

where

$$C_p = \frac{A_v a_x a_c P}{R_c} \tag{5.2}$$

in which $A_v$, $W_c$, $a_x$ and $a_c$ are the same as in the first recommendation. The performance criteria factor $(P)$ is the same as in the current provisions.  The response modification coefficient $(R_c)$ is directly transferred from the values of $C_c$ in the current provisions which includes the effect that the supporting structure has on the performance of equipment although this effect is considered to be small.  Both the performance criteria factor and the response modification coefficient are tabulated in Table 5-1.

<u>Architectural Components</u>.  For architectural component design, the design force required can also be determined by Eq. (5.1) but the amplification factor $(a_c)$ is assigned

to be 1.0. The response modification coefficient $(R_c)$ and the performance criteria factor $(P)$ are different from their values for mechanical and electrical components and systems as demonstrated in Table 5-2.

### 5.1.2 Development of the Design Force Equation

Floor and Equipment Acceleration Coefficients.   Floor acceleration is considered to be linearly distributed along the structural height as shown in Fig.  3-1.  The floor acceleration coefficient at $h_x$ level is then denoted by $A_v a_x$ and the equipment acceleration coefficient at the same level can subsequently be expressed as $A_v a_x a_c$.

Inertia Force on Equipment.   The inertia force acting on the equipment at $h_x$ level can be written as

$$F_p = A_v a_x a_c W_c \tag{5.3}$$

Response Modification Coefficient.   As in the design of building structures, the design force for the equipment can be reduced by a factor $R_c$, i.e.,

$$F_p = \frac{A_v a_x a_c W_c}{R_c} \tag{5.4}$$

due to earthquake variabilities and detailing construction of equipment including small amounts of allowable yielding in equipment anchorage and possible redundant constraint to the equipment.

Performance Criteria Factor.   After the performance criteria factor has been considered, the design force on the equipment expressed in Eq. (5.4) becomes

$$F_p = \frac{A_v a_x a_c P W_c}{R_c} \tag{5.5}$$

### 5.1.3 Determination of Response Modification Coefficient ($R_c$)

As has been briefly discussed above, the value of $R_c$ is related to the determination of the value of $C_c$ in the current provisions but need to be modified subject to further investigations. In fact, $R_c$ in this recommendation is close to the intermediate result $(\overline{R}_c)$ of the first recommendation in deriving the response modification coefficient. In what follows, we simply compare $a_x/R_c$ with $C_c$ in the current provisions to determine the corresponding response modification coefficient $(R_c)$. More specifically, we assign the minimum value of $R_c$ to the most vulnerable components with maximum $C_c$ values. For example, when a communication system is installed on the top floor of a building, the $C_c$ value can be taken

from Table 8.3.2a of the current provisions as 2.0 and the amplification factor on the top floor $(a_n)$ is considered to be 3.75. The response modification coefficient is therefore equal to 3.75/2.0 = 1.88. All the $R_c$ values for different mechanical and electrical equipment and architectural components are tabulated in Tables 5-1 and 5-2. For some nonstructural components, such as elevator shaft enclosures, stair enclosures, etc., which can span two or more floors in a building, the response modification coefficient $R_c$ is considered to be the floor amplification factor $a_x$ at 3/5 height divided by the corresponding $C_c$ value. The 3/5 height approximately represents node of the second mode of the building structure and the corresponding response modification coefficient $(R_c)$ implicitly accounts for the second mode contribution to the design force of nonstructural components attached to the upper 2/5-height floors of the building on the conservative side.

## TABLE 5-1. Mechanical and Electrical Component and System Response Modification Coefficient ($R_c$) and Performance Criteria Factor (P)

| Mechanical and Electrical Component or System | Response Modification Coefficient ($R_c$) | Performance Criteria Factor (P) | | |
|---|---|---|---|---|
| | | Seismic Hazard Exposure Group | | |
| | | I | II | III |
| Fire protection equipment and systems | 1.25 | 1.5 | 1.5 | 1.5 |
| Emergency or standby electrical systems | 1.25 | 1.5 | 1.5 | 1.5 |
| Elevator drive, suspension system, and control anchorage | 2.0 | 1.0 | 1.0 | 1.5 |
| General equipment<br><br>Boilers, furnaces,incinerators, water heaters, and other equipment using combustible energy sources or high temperature energy sources chimneys, flues, smokestacks, and vents<br><br>Communication systems<br><br>Electrical bus ducts, conduit, and cable trays<br><br>Electrical motor control centers, motor control devices, switchgears, transformers, and unit substations<br><br>Reciprocating or rotating equipment<br><br>Tanks, heat exchangers, and pressure vessels<br><br>Utility and service interfaces | 1.25 | 0.5 | 1.0 | 1.5 |
| Manufacturing and process machinery | 4.0 | 0.5 | 1.0 | 1.5 |
| Pipe systems | | | | |
| Gas and high hazard piping | 1.25 | 1.5 | 1.5 | 1.5 |
| Fire suppression piping | 1.25 | 1.5 | 1.5 | 1.5 |
| Other pipe systems | 4.0 | NR | 1.0 | 1.5 |
| HVAC and service ducts | 4.0 | NR | 1.0 | 1.5 |
| Electrical panel boards and dimmers | 4.0 | NR | 1.0 | 1.5 |
| Lighting fixtures | 4.0 | 0.5 | 1.0 | 1.5 |
| Conveyor systems (nonpersonnel) | 4.0 | NR | NR | 1.5 |

## TABLE 5-2. Architectural Component Response Modification Coefficient ($R_c$) and Performance Criteria Factor (P)

| Architectural Component | Response Modification Coefficient ($R_c$) | Performance Criteria Factor (P) Seismic Hazard Exposure Group | | |
|---|---|---|---|---|
| | | I | II | III |
| Exterior nonbearing walls | 3.0 | 1.5 | 1.5 | 1.5 |
| Interior nonbearing walls | | | | |
| Stair enclosures | 1.8 | 1.0 | 1.0 | 1.5 |
| Elevator shaft enclosures | 1.8 | 0.5 | 0.5 | 1.5 |
| Other vertical shaft enclosures | 3.0 | 1.0 | 1.0 | 1.5 |
| Other nonbearing walls | 3.0 | 1.0 | 1.0 | 1.5 |
| Cantilever elements | | | | |
| Parapets, chimneys, or stacks | 1.0 | 1.5 | 1.5 | 1.5 |
| Wall attachments (see Sec. 8.2.3) | 1.0 | 1.5 | 1.5 | 1.5 |
| Veneer connections | 1.0 | 0.5 | 1.0 | 1.0 |
| Penthouses | 4.5 | NR | 1.0 | 1.0 |
| Structural fireproofing | 3.0 | 0.5 | 1.0 | 1.5 |
| Ceilings | | | | |
| Fire-rated membrane | 3.0 | 1.0 | 1.0 | 1.5 |
| Nonfire-rated membrane | 4.5 | 0.5 | 1.0 | 1.0 |
| Storage racks more than 8 feet in height (content included) | 1.8 | 1.0 | 1.0 | 1.5 |
| Access floors (supported equipment included) | 1.25 | 0.5 | 1.0 | 1.5 |
| Elevator and counterweight guiderails and supports | 2.0 | 1.0 | 1.0 | 1.5 |
| Appendages | | | | |
| Roofing units | 4.5 | NR | 1.0 | 1.0 |
| Containers and miscellaneous components (free standing) | 1.8 | NR | 1.0 | 1.0 |
| Partitions | | | | |
| Horizontal exits including ceilings | 3.0 | 1.0 | 1.5 | 1.5 |
| Public corridors | 3.0 | 0.5 | 1.0 | 1.5 |
| Private corridors | 4.5 | NR | 0.5 | 1.5 |
| Full height area separation partitions | 3.0 | 1.0 | 1.0 | 1.5 |
| Full height other partitions | 4.5 | 0.5 | 0.5 | 1.5 |
| Partial height partitions | 4.5 | NR | 0.5 | 1.0 |

## SECTION 6

## COMPARISON OF DESIGN FORCES

In this section, two architectural components (parapet and storage rack) and one mechanical or electrical equipment are chosen to demonstrate that the recommended approaches overcome much of the shortcomings in the current provisions as discussed in Section 2.2. They are also used to compare the relative conservativeness of various provisions and design codes. All the comparisons are made based on the determination of the seismic design coefficient for components $(C_p)$ as a function of structural period $(T_s)$.

### 6.1 Maximum and Minimum Design Forces

It is instructive to make a simple comparison among maximum and minimum design forces specified in different provisions and codes. The maximum and minimum design forces of nonstructural components among three recommended provisions are compared in Table 6-1 and those of the 1991 NEHRP, the 1991 UBC, and the 1985 Tri-Service codes are tabulated in Table 6-2. It can be observed that the recommended force formulas yield maximum forces which are higher than those specified in the UBC and the current NEHRP provisions but are less than those given in the Tri-Service Code. On the other hand, the minimum design forces are consistently less than the values specified in the other provisions and codes. These larger variations in the design force exist since more factors such as soil property, equipment location, and supporting structural characteristics have been taken into account in the recommended formulas, which is a major contribution of the suggested formulation for nonstructural element design. In addition, these extreme forces can not be reached in most cases of practical design because the combination of all the factors contributing to the extreme values hardly occurs in practice.

### 6.2 Case Studies (Parapets, Storage Racks and General Equipment on Reinforced Concrete Shear Walls)

#### 6.2.1 Effects of Soil Type, Structural Period and Component Location

It is instructive to know how significant the effects of the soil condition, structural period, and component location on nonstructural component design are before comparing design forces of the three recommended approaches with those of current available provisions and codes.

6-1

The parapet chosen here is considered to be atop a building structure. The storage rack and mechanical or electrical equipment are considered to be installed either at the top floor or at the middle floor of the building. The parameters used in these case studies are given in Table 6-3.

The seismic design coefficients $(C_p)$ for a parapet determined by the three recommended revisions are presented in Fig. 6-1. It can be seen that $C_p$ is a function of structural period and the soil type. As the building structure becomes more flexible or the soil layer under the structure becomes stiffer, the design force on the parapet decreases.

Figures 6-2, 6-3 and 6-4 present the seismic design coefficients determined by the three recommended approaches for a storage rack installed at the top or middle of the building. As one can see from these plots, the storage rack location in the building has a significant influence on the seismic design force imposed on it. The higher the storage rack location, the larger the required design force. When the storage rack is anchored to the very flexible building structure, the location effect on the design force becomes less significant, since the floor amplification factor $(a_x)$ approaches unity in the limit. As in the case of parapet design, similar effects of structural period and soil type on the design force for the storage rack can be observed.

For seismic design of the mechanical or electrical equipment, the seismic design coefficients determined in accordance with the first recommended revision are respectively shown in Figs. 6-5(a,b) when it is installed at the top and middle of a building structure; those with the second recommended revision are shown in Figs. 6-6(a,b); and those with the third recommended revision are shown in Figs. 6-7(a,b), respectively. An examination of these figures indicates that the tuning effect on the design force is significant and effects of structural period, soil type and equipment location on the design forces are consistent with those found for the parapet and storage rack designs.

### 6.2.2 Comparisons Among the Three Recommendations

Comparisons among the three recommended revisions may shed more light on their relative merits toward improvement over the 1991 NEHRP provisions. The seismic design coefficients for the parapet, the storage rack, and the mechanical or electrical equipment are presented in Figs. 6-1, 6-8 and 6-9. It can be observed from Figs. 6-1 and 6-9 for the parapet and equipment design that the seismic design coefficient determined by the third recommendation is the highest while the corresponding value produced by the second recommendation is the smallest. This mainly results from the reduction effect

6-2

due to structural yielding in these examples, i.e., $R_s = 1.5$. In contrast, the second recommendation provides a larger design force for the storage rack design since the response modification coefficient of the structure for the storage rack design ($R_s = 1.5$) is smaller than that of the component ($R_c = 1.8$). It should be noted that the seismic design coefficient calculated by the first recommendation may be larger in other cases than those of the remaining two recommendations. In other words, comparisons among the three recommendations can only be made on individual cases. Anyone of them can produce the most or least conservative design force for a given nonstructural component.

### 6.2.3  Recommended Revisions vs. U.S. Codes/Provisions

As can be seen from Fig. 6-1, the first and second recommended revisions provide smaller design forces than that given by the 1991 NEHRP provisions for design of the parapet atop a building structure with various flexibility while the third revision requires a larger design force for the same parapet attached to a stiff building structure and a smaller design force to a flexible building than that required by the current NEHRP provisions. Compared with the 1991 UBC or the 1985 Tri-Service Code, the recommended revisions require larger seismic design forces in a broad range of structural periods.

The seismic design coefficients for the storage rack calculated from the recommended revisions are compared in Figs. 6-2, 6-3 and 6-4 with the 1991 NEHRP provisions, the 1991 UBC and 1985 Tri-Service codes when it is installed at the top or middle of a building structure. The seismic design coefficients for the storage rack attached to the top floor, calculated by the recommended formulas, are greater in the case of a stiff building and smaller in the case of a flexible building than the corresponding coefficient provided in the current NEHRP provisions. The second recommended revision requires a slightly greater seismic design force for the storage rack design than that given by the 1991 UBC for the middle attachment as in the case of top attachment; whereas the first and third recommended revisions require smaller seismic design forces than the current NEHRP values but larger design forces than the 1991 UBC or the 1985 Tri-Service values when the storage rack is supported by a stiff building.

The seismic design coefficients of the mechanical or electrical equipment installed at the top of a building, calculated by the recommended revisions, are compared in Figs. 6-5(a), 6-6(a), 6-7(a) and 6-9 with those specified in the current NEHRP provisions, UBC and Tri-Service codes. In the tuned case, as shown in Fig. 6-9, the seismic design coefficients from the suggested approaches are larger for stiff building structures and

smaller for flexible building structures than the corresponding values provided by the 1991 NEHRP and the 1985 Tri-Service codes but consistently larger than the design coefficient given by the 1991 UBC code. In the detuned case as shown in Figs. 6-5(a), 6-6(a) and 6-7(a), the recommended seismic design coefficients are also larger for a stiff structure and smaller for a flexible structure than the 1991 NEHRP values, but those provided by the second and third recommendations are consistently larger than the 1991 UBC and the 1985 Tri-Service values.

It is worth noting that, for a nonstructural component attached to a very flexible structure, the recommended design forces are in many cases smaller than those provided by the 1991 UBC and the 1985 Tri-Service codes. This seems contradictory to the design philosophies employed in the NEHRP provisions and UBC (or Tri-Service) Code (strength design vs. stress design). In fact, this phenomenon only indicates the over conservativeness involved in the UBC and Tri-Service codes for this case since the recommended revisions in this report are justified to a certain degree by analyses, experimental results and observation data from past earthquakes.

## TABLE 6-1. Maximum and Minimum Design Forces (1)

| Design Code | first recommendation | | second recommendation | | third recommendation | |
|---|---|---|---|---|---|---|
| | Architectural | Mechanical & Electrical | Architectural | Mechanical & Electrical | Architectural | Mechanical & Electrical |
| Basic Equation | $F_p = \dfrac{A_v a_x P W_c}{R_s R_c}$ | $F_p = \dfrac{A_v a_x a_c P W_c}{R_s R_c}$ | $F_p = \dfrac{A_v a_x P W_c}{R_s}$ | $F_p = \dfrac{A_v a_x a_c P W_c}{R_s}$ | $F_p = \dfrac{A_v a_x P W_c}{R_c}$ | $F_p = \dfrac{A_v a_x a_c P W_c}{R_c}$ |
| Basis | Strength Design | Strength Design | Strength Design | Strength Design | Strength Design | Strength Design |
| Maximum Value | $\dfrac{0.4 \times 3.75 \times 1.2 W_c}{1.0 \times 1.0}$ $= 1.80\ W_c$ | $\dfrac{0.4 \times 3.75 \times 2.5 \times 1.2 W_c}{1.0 \times 1.1}$ $= 4.09\ W_c$ | $\dfrac{0.4 \times 3.75 \times 1.2 W_c}{1.0}$ $= 1.80\ W_c$ | $\dfrac{0.4 \times 3.75 \times 2.0 \times 1.2 W_c}{1.0}$ $= 3.60\ W_c$ | $\dfrac{0.4 \times 3.75 \times 1.5 W_c}{1.0}$ $= 2.25 W_c$ | $\dfrac{0.4 \times 3.75 \times 2.5 \times 1.5 W_c}{1.25}$ $= 4.50 W_c$ |
| Minimum Value | $\dfrac{0.05 \times 1.0 \times 0.8 W_c}{2.0 \times 3.0}$ $= 0.007\ W_c$ | $\dfrac{0.05 \times 1.0 \times 1.0 \times 0.8 W_c}{2.0 \times 2.7}$ $= 0.007\ W_c$ | $\dfrac{0.05 \times 1.0 \times 0.8 W_c}{2.0}$ $= 0.020 W_c$ | $\dfrac{0.05 \times 1.0 \times 1.0 \times 0.8 W_c}{2.0}$ $= 0.020 W_c$ | $\dfrac{0.05 \times 1.0 \times 0.5 W_c}{4.5}$ $= 0.006 W_c$ | $\dfrac{0.05 \times 1.0 \times 1.0 \times 0.5 W_c}{4.0}$ $= 0.006 W_c$ |

## TABLE 6-2. Maximum and Minimum Design Forces (2)

| Design Code | 1991 NEHRP | | 1991 UBC | 1985 Tri-Service | |
|---|---|---|---|---|---|
| | Architectural | Mechanical & Electrical | Nonstructural | Architectural | Mechanical & Electrical |
| Basic Equation | $F_p = A_v C_c P W_c$ | $F_p = A_v C_c P a_c W_c$ | $F_p = Z I C_p W_p$ | $F_p = Z I C_p W_p$ | $F_p = Z I A_p C_p W$ |
| Basis | Strength Design | Strength Design | Allowable Stress | Allowable Stress | Allowable Stress |
| Maximum Value | $0.4 \times 3.0 \times 1.5 W_c$ $= 1.80\ W_c$ | $0.4 \times 2.0 \times 1.5 \times 2.0 W_c$ $= 2.40\ W_c$ | $0.4 \times 1.5 \times 2.0 W_p$ $= 1.20\ W_p$ | $1.0 \times 1.5 \times 0.8 W_p$ $= 1.20\ W_p$ | $1.0 \times 1.5 \times 5.0 \times 0.8 W_p$ $= 6.00\ W_p$ |
| Minimum Value | $0.05 \times 0.6 \times 0.5 W_c$ $= 0.015\ W_c$ | $0.05 \times 0.67 \times 0.5 \times 1 W_c$ $= 0.017\ W_c$ | $\dfrac{2}{3} \times 0.05 \times 1.0 \times 0.75 W_p$ $= 0.025\ W_p$ | $\dfrac{3}{16} \times 1.0 \times 0.3 W_p$ $= 0.056\ W_p$ | $\dfrac{3}{16} \times 1.0 \times 1.0 \times 0.3 W_p$ $= 0.056\ W_p$ |

**TABLE 6-3. Parameters Used in Case Studies**

| Provisions/Codes | Parapet ($a_c = A_p = 1.0$) | Storage Rack ($a_c = A_p = 1.0$, $P = I = 1.0$) | Equipment ($P = I = 1.0$) ($a_c$ = detuned/ tuned) |
|---|---|---|---|
| first Recommendation | $A_v = 0.2$, $P = 1.2$, $R_s = 1.5$, $R_c = 1.0$ | $A_v = 0.2$, $R_s = 1.5$, $R_c = 1.5$ | $A_v = 0.2$, $R_s = 1.5$, $R_c = 1.1$, $a_c = 1.0/2.5$ |
| second recommendation | $A_v = 0.2$, $P = 1.2$, $R_s = 1.5$ | $A_v = 0.2$, $R_s = 1.5$ | $A_v = 0.2$, $R_s = 1.5$, $a_c = 1.0/2.0$ |
| third Recommendation | $A_v = 0.2$, $P = 1.5$, $R_c = 1.0$ | $A_v = 0.2$, $R_c = 1.8$ | $A_v = 0.2$, $R_c = 1.25$, $a_c = 1.0/2.5$ |
| 1991 NEHRP | $A_v = 0.2$, $P = 1.5$, $C_c = 3.0$ | $A_v = 0.2$, $C_c = 1.5$ | $A_v = 0.2$, $C_c = 2.0$, $a_c = 1.0/2.0$ |
| 1991 UBC | $Z = 0.2$, $I = 1.25$, $C_p = 2.0$ | $Z = 0.2$, $C_p = 0.75$ | $Z = 0.2$, $C_p = 0.75$ |
| 1985 Tri-Service | $Z = 0.5$, $I = 1.25$, $C_p = 0.8$ | $Z = 0.5$, $C_p = 0.3$ | $Z = 0.5$, $C_p = 0.3$, $A_p = 1.0/5.0$ |
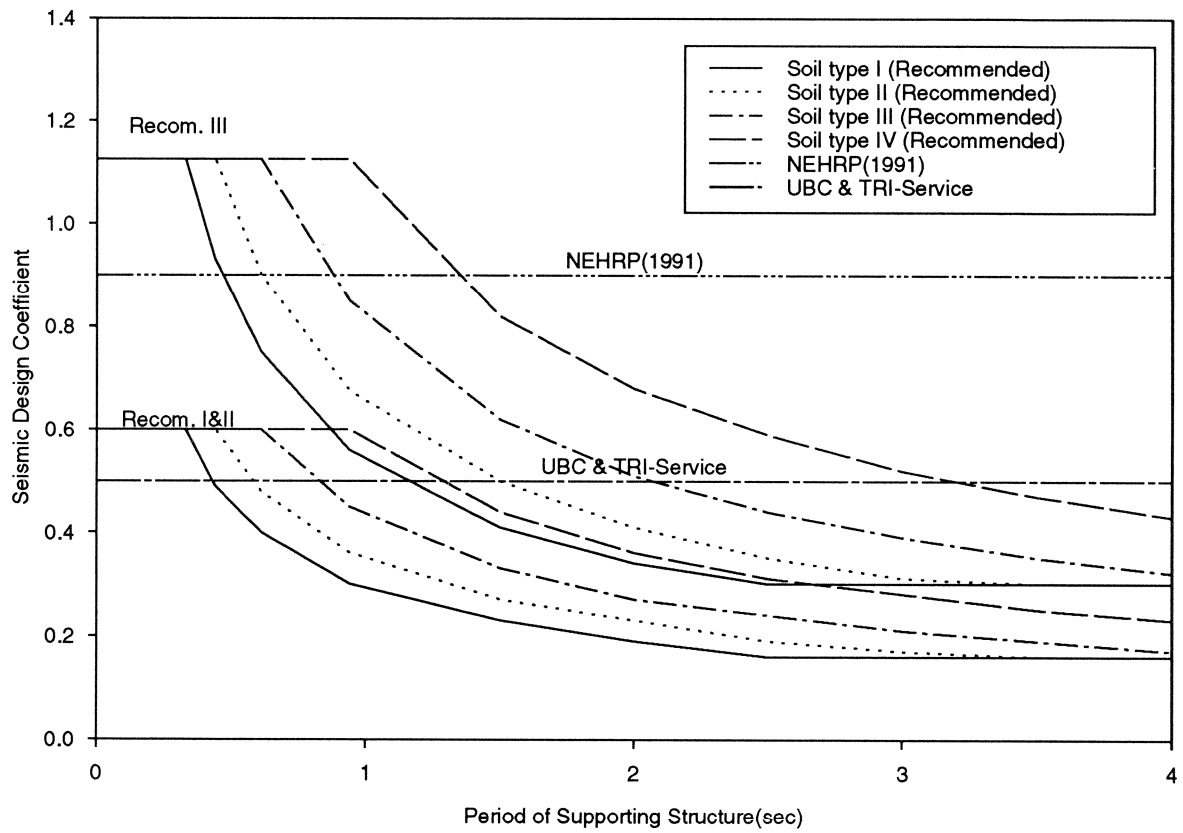
Fig. 6-1   Seismic Design Coefficient for Parapet at Top of Building
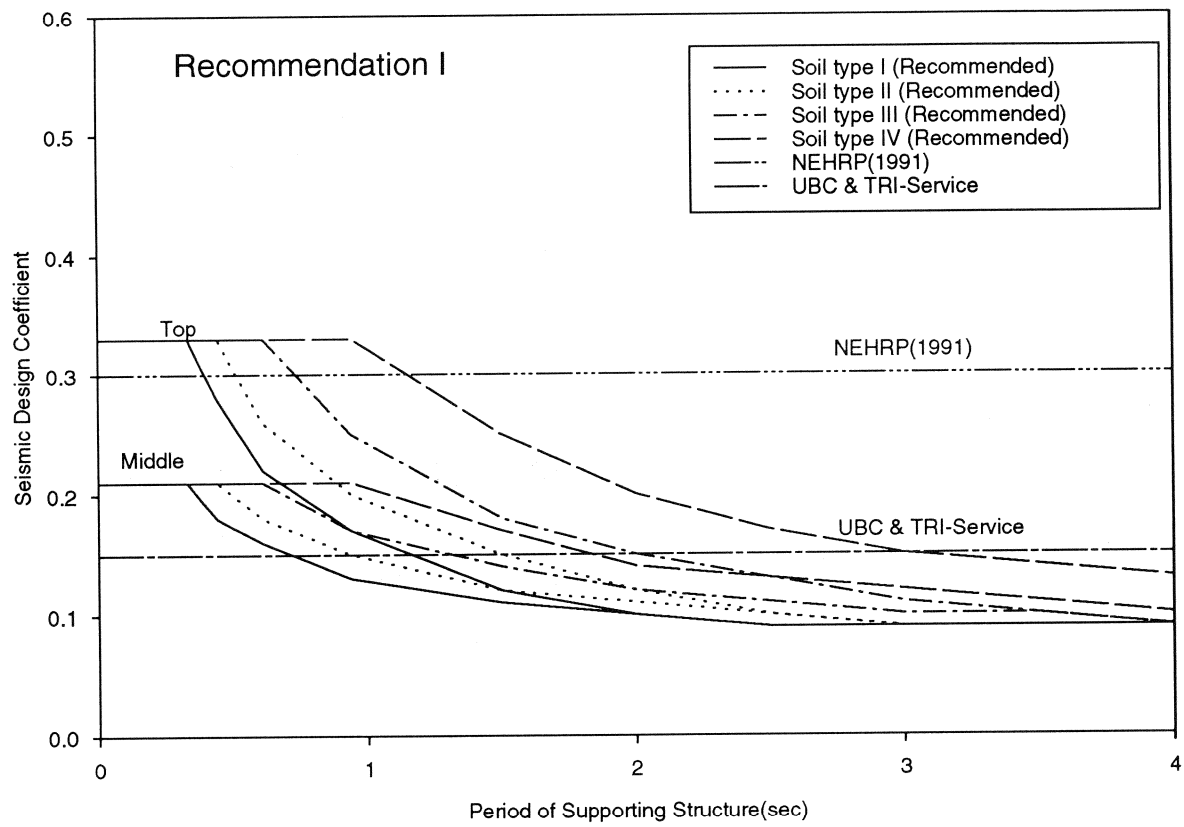
Fig. 6-2   Seismic Design Coefficient for Storage Rack at Different Locations
(First Recommendation)
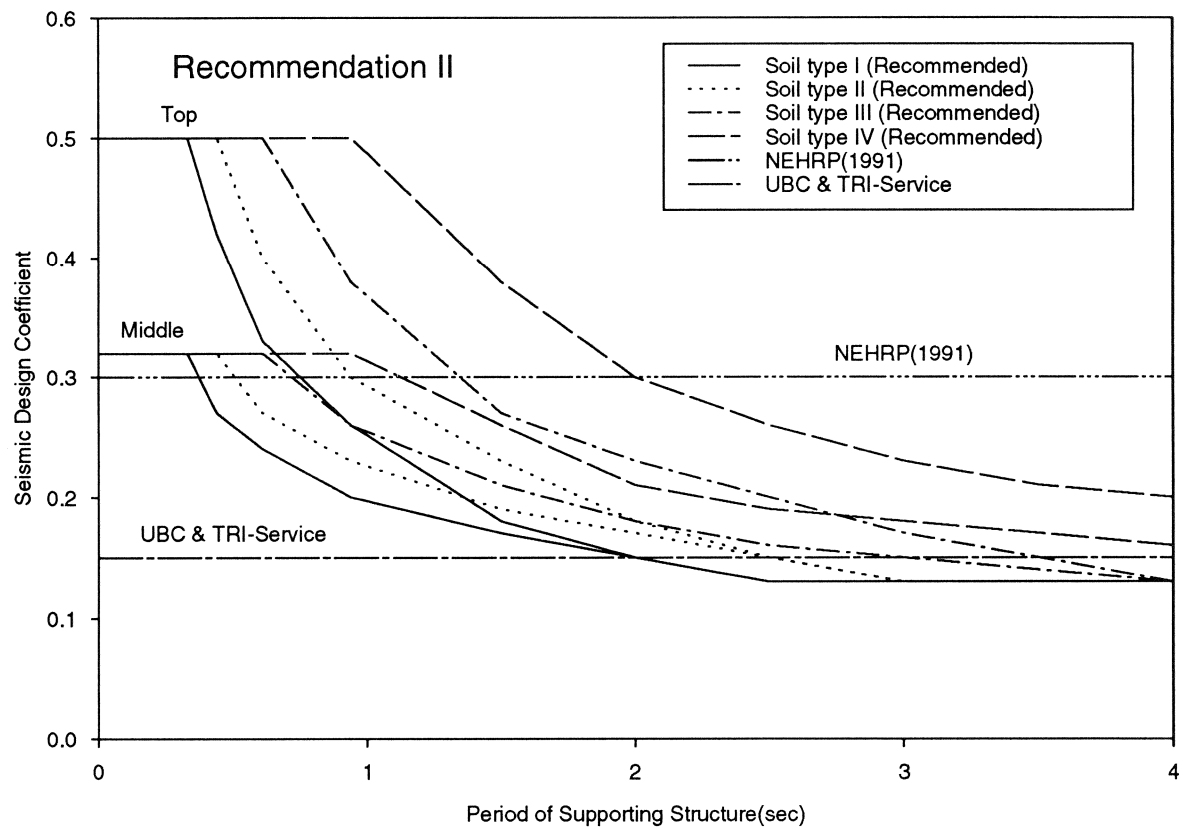
Fig. 6-3   Seismic Design Coefficient for Storage Rack at Different Locations
(Second Recommendation)
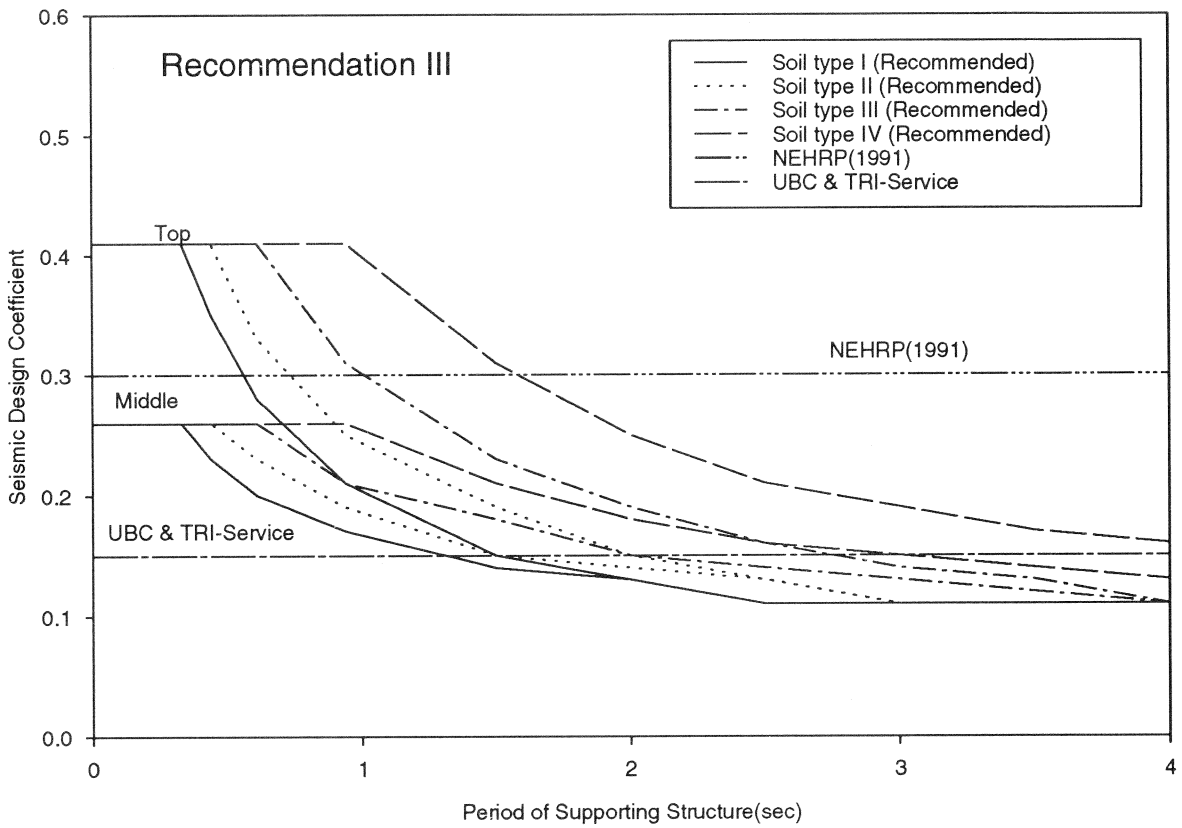
Fig. 6-4    Seismic Design Coefficient for Storage Rack at Different Locations
(Third Recommendation)

Fig. 6-5(a)   Seismic Design Coefficient of Equipment at Top of Building
(First Recommendation)

Fig. 6-5(b) Seismic Design Coefficient of Equipment at Middle of Building
(First Recommendation)

Fig. 6-6(a)   Seismic Design Coefficient of Equipment at Top of Building
(Second Recommendation)

Fig. 6-6(b)   Seismic Design Coefficient of Equipment at Middle of Building
(Second Recommendation)

Fig. 6-7(a)　Seismic Design Coefficient of Equipment at Top of Building
(Third Recommendation)

Fig. 6-7(b)   Seismic Design Coefficient of Equipment at Middle of Building
(Third Recommendation)
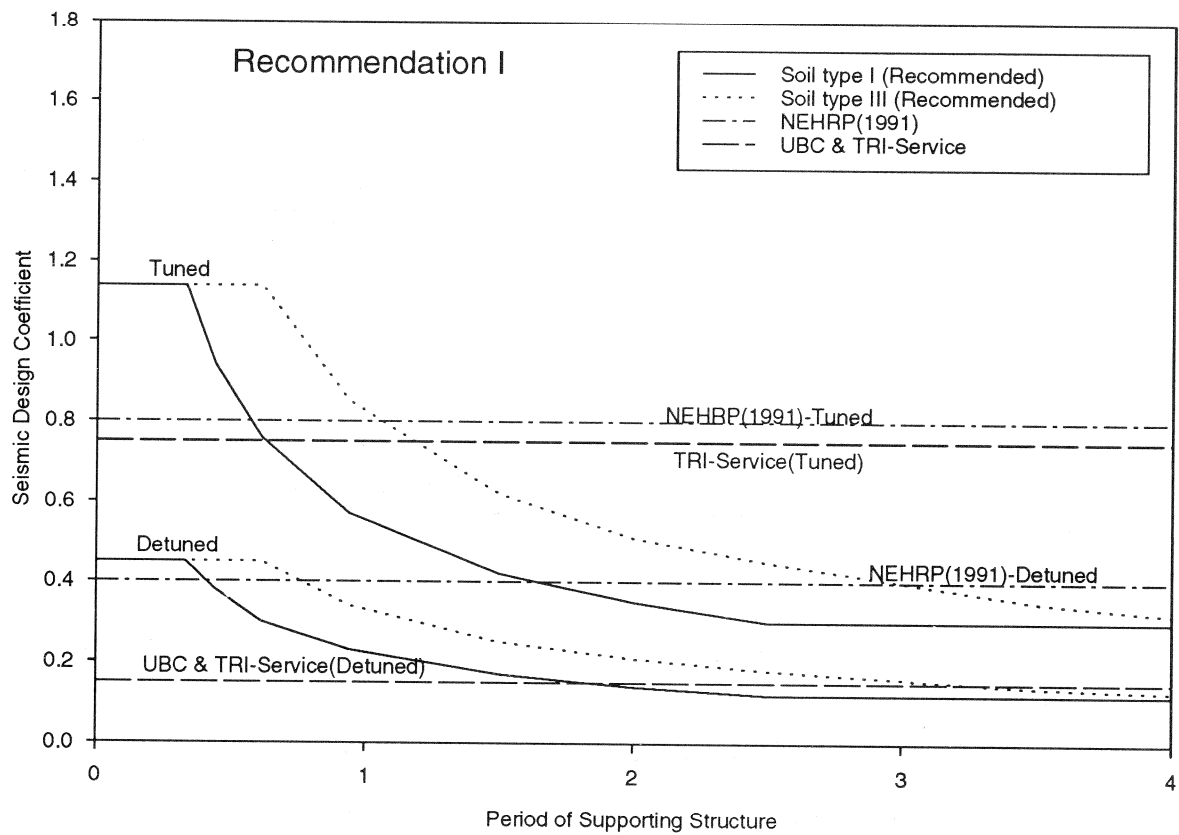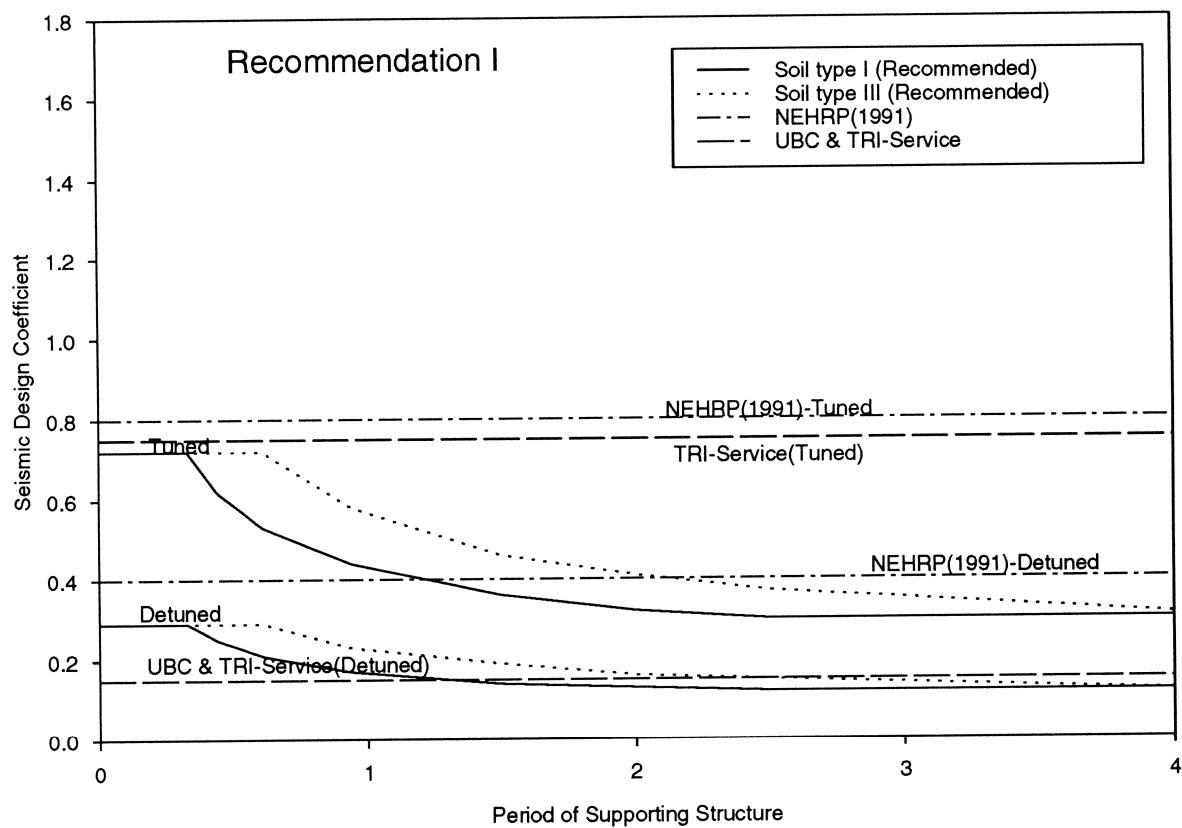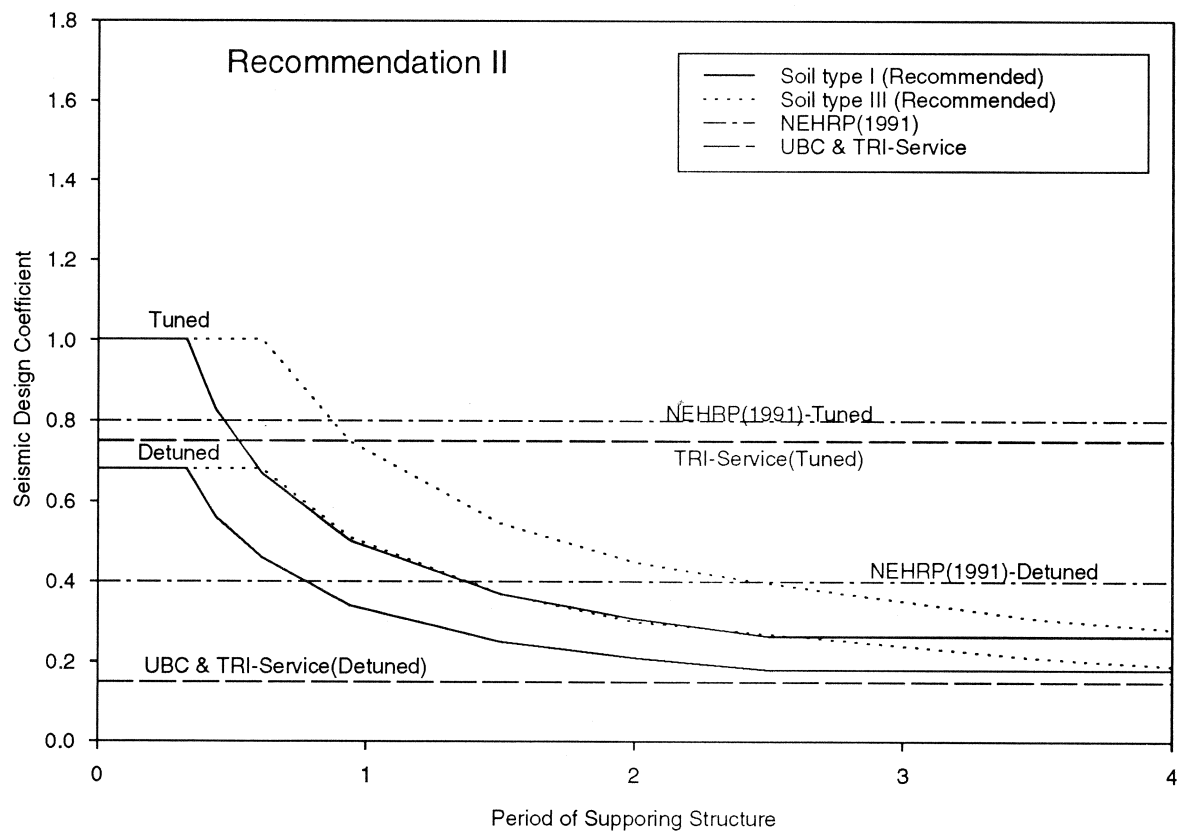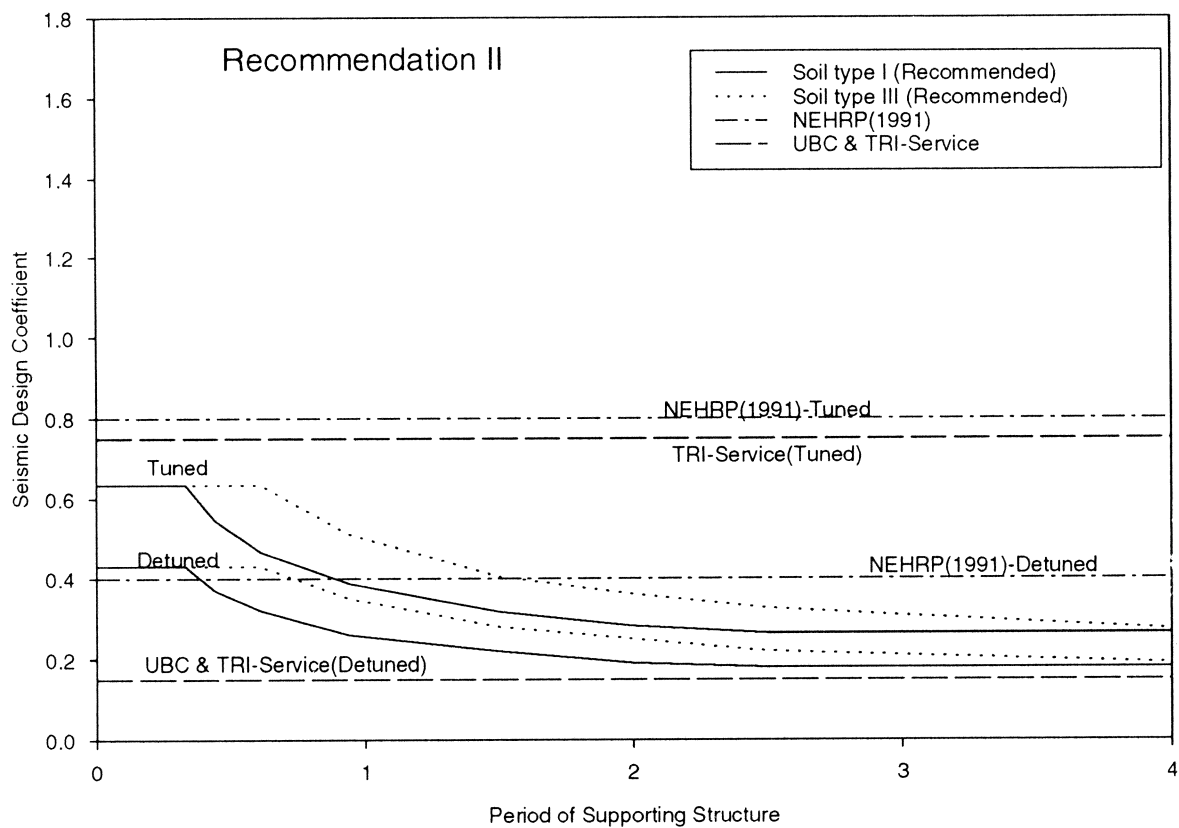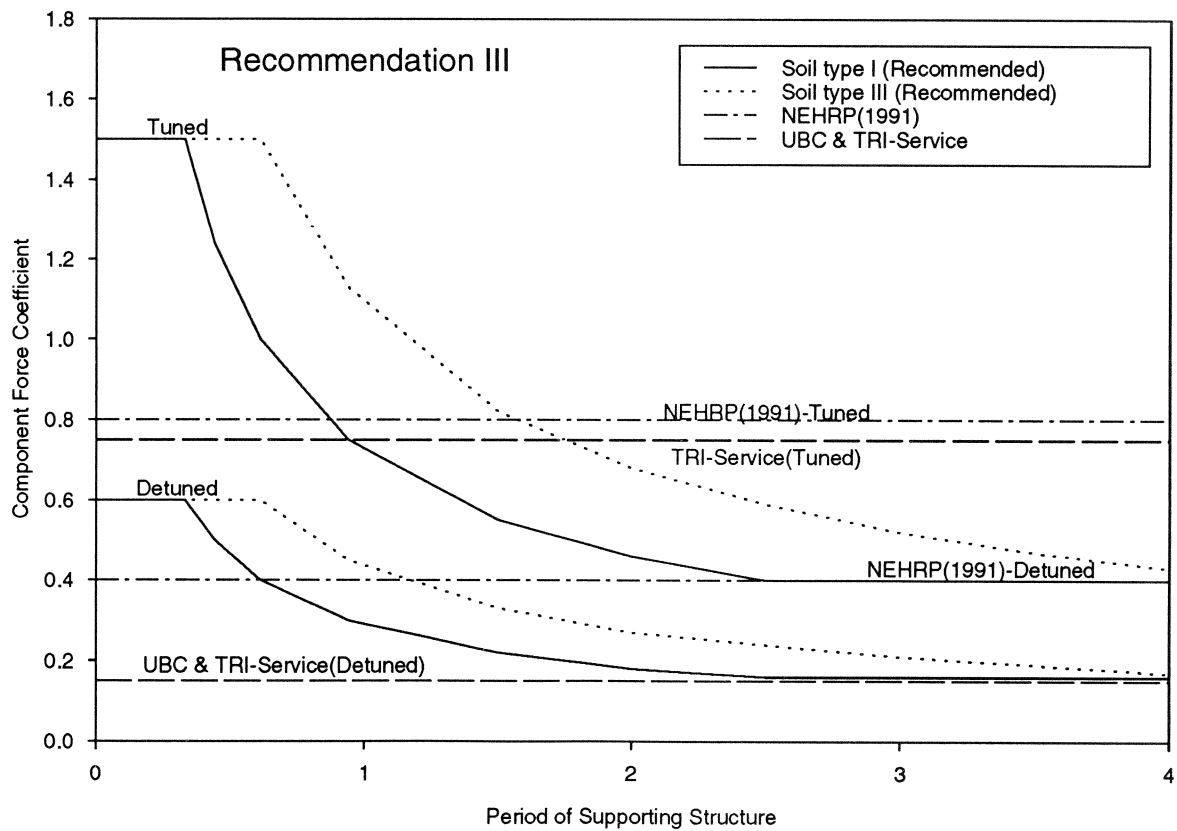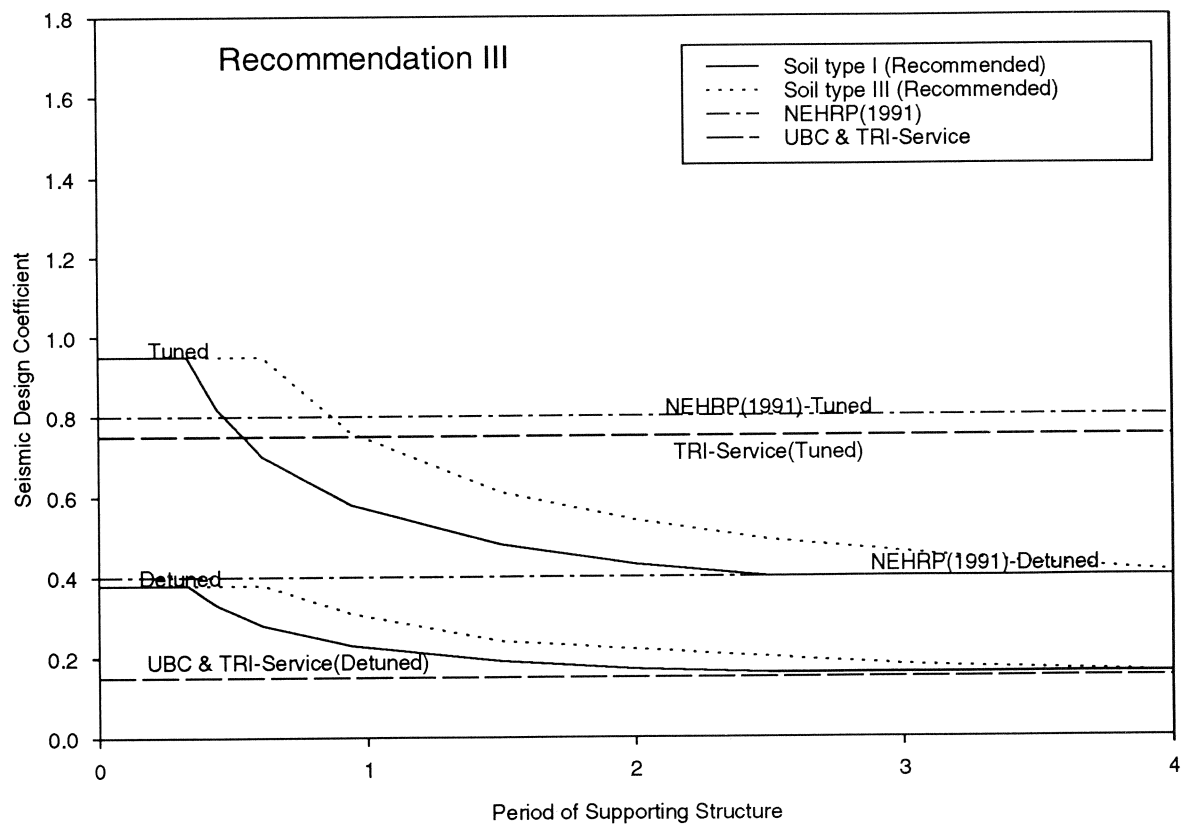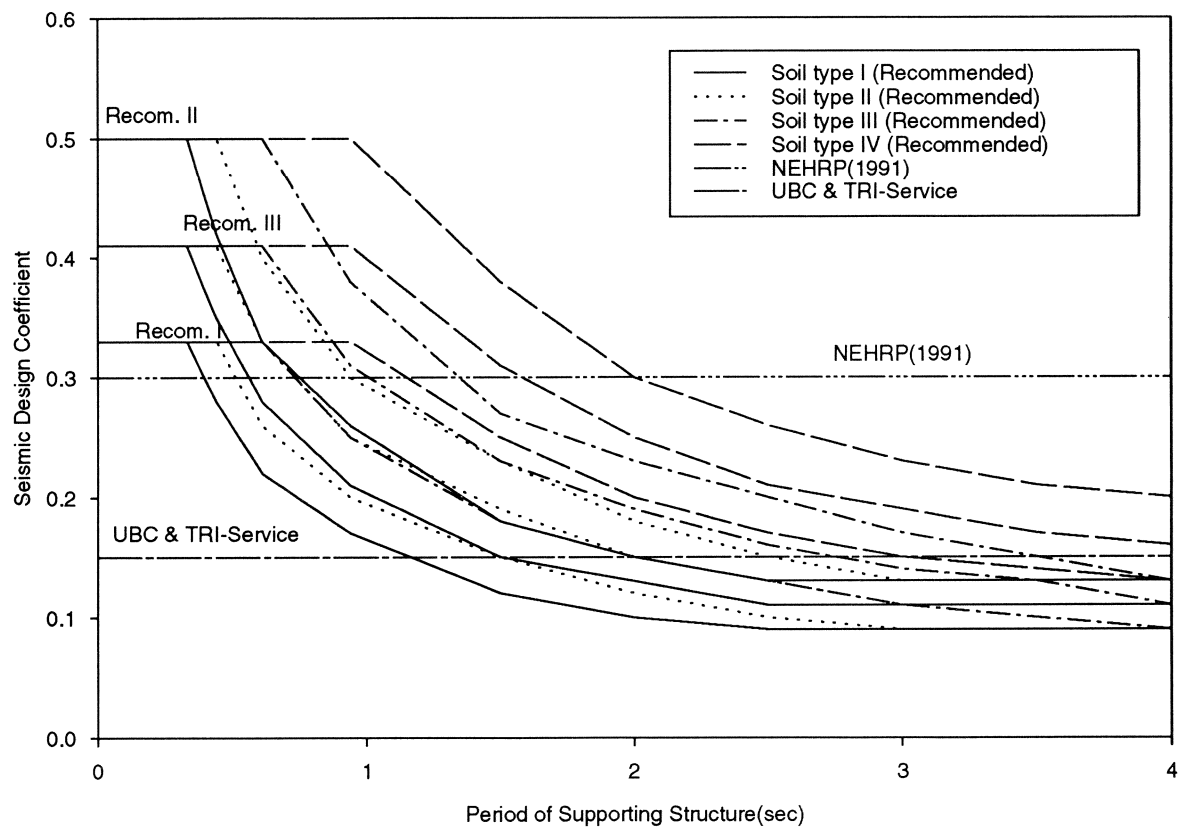
Fig. 6-8  Seismic Design Coefficient for Storage Rack at Top of Building
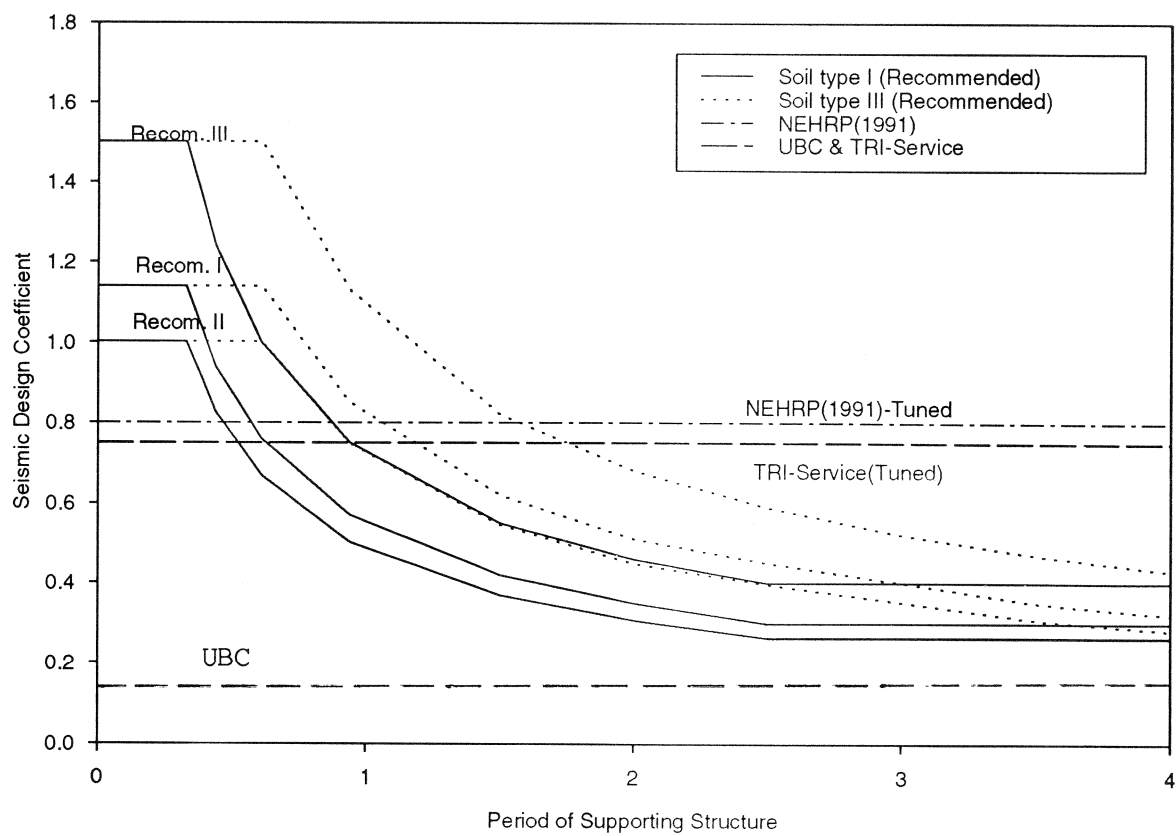
**Fig. 6-9   Seismic Design Coefficient of Equipment at Top (Tuned Case)**

## SECTION 7

## DESIGN DISPLACEMENT

There are three types of displacements involved in the design of mechanical and electrical equipment, i.e., flexible support deformation, sliding displacement and interstory distortion. The flexible support deformation is defined as the relative displacement of the equipment with respect to its attached floor. The sliding displacement of an equipment with low center of gravity occurs when bolts (if exist) connecting the equipment anchorage to the building floor are damaged or when the maximum friction force between the equipment and floor (no bolts) is exceeded. The interstory distortion of one building structure or between two adjacent buildings is relevant to the design of equipment such as piping systems installed over two or more floors of the building structure or between two building structures.

For seismic design of architectural components, only interstory distortion is of interest.

### 7.1 Flexible Support Deformation

In order to suppress vibrational levels of a mechanical or electrical equipment, flexible anchorage of the equipment is generally required. When flexibility of the equipment anchorage is relatively large, the design force on the equipment may not be critical. However, the relative displacement of the equipment with respect to its attached floor may exceed an allowable level, which will also cause anchorage damage. In addition, when anchorage ductility is taken into account as in the first and third recommendations, the design force on the equipment would also be small. Nevertheless, the inelastic deformation in this case may be excessive. For these reasons, a displacement equation for flexibly-supported equipment which provides support deformation information is of practical value.

#### 7.1.1 Displacement Equation

From Eq. (3.1), the relative displacement of an equipment with respect to its supporting floor can be formulated as

$$\Delta_p = \frac{F_p}{K_c} = \frac{A_v a_x a_c P g T_c^2}{4\pi^2 R_s R_c} \tag{7.1}$$

in which $g$ is the gravitational acceleration. The response modification coefficient $(R_c)$ should be assigned to be 1.0 to obtain the maximum displacement.

7-1

### 7.1.2 Example: A General Equipment

The same type of general equipment as in Section 6 is used here to show the magnitude of its relative displacement with respect to the floor. The calculated displacements for different equipment periods are presented in Table 7-1. As one can observe, the relative displacement increases as the anchorage becomes flexible.

## 7.2 Sliding Displacement

As indicated in the brief introduction of this section, sliding of a rigid body which is not bolted to the floor occurs when seismic load acting on the rigid body exceeds friction force between the rigid body and its supporting floor. Sliding of a general equipment which is bolted to the floor could also occur when bolts fail when the seismic load is excessive.

It is noted that only equipment with low centers of gravity are considered here so that the possibility of overturning of the equipment is ignored.

### 7.2.1 Displacement Equation

The sliding distance of a rigid body along its supporting building floor can be approximately determined in accordance with

$$\Delta_p = C_\delta \delta_s \tag{7.2}$$

in which $\delta_s$ is the relative displacement of the building floor with respect to ground and can be determined from the building structural analysis exclusively. The sliding coefficient $C_\delta$ can be calculated from

$$C_\delta = \left| \frac{\eta \gamma^2}{2} - \frac{1 - \cos \gamma}{\eta} \right| \tag{7.3}$$

where

$$\gamma = \frac{2\pi(t_3 - t_1)}{T_s} \tag{7.4}$$

and satisfies the following equation:

$$\gamma = \frac{\sqrt{1 - \eta^2}(1 - \cos \gamma)}{\eta} + \sin \gamma \tag{7.5}$$

Here, $t_3 - t_1$ represents the time interval during which the rigid body moves within a half cycle of seismic input from the building floor. The parameter $\eta$ represents the relative strength of resistance (friction force) and load (seismic force) which can be simply expressed as

$$\eta = \frac{\mu(1 - a_v)}{A_v a_x} \tag{7.6}$$

7-2

in which $\mu$ is the friction coefficient between the rigid body and its supporting floor, and $a_v$ represents vertical acceleration of the supporting floor that is approximately equal to $A_v/3$ if floor amplification effect of the vertical acceleration is insignificant.

### 7.2.2   Development of the Displacement Equation

For the purpose of developing a simple sliding distance equation, the absolute acceleration of a building floor to which a mechanical or electrical equipment is attached can be simply considered as a harmonic motion with the fundamental period of the building structure. In the case of a mechanical or electrical equipment installed on the upper floors of a building structure, which is usually of practical interest, the contribution of the first mode of the structure to the seismic response of the equipment is predominant and therefore accelerations of the upper floors appear to be harmonic, i.e.,

$$\ddot{x}_f(t) = A_v a_x g \sin\left(\frac{2\pi t}{T_s}\right) \qquad (7.7)$$

The rigid body begins to slide when the inertia force $m_c \ddot{x}_f(t)$ acting on it exceeds the effective friction force and it stops again when $m_c \ddot{x}_f(t)$ is less than the friction force as shown in Fig. 7-1. The rigid body will move back and forth along a perfectly horizontal building floor during earthquakes and only a half cycle of seismic excitation $\ddot{x}_f(t)$ is needed to obtain maximum sliding distance of the rigid body.

The equation of motion of the rigid body $(W_c = m_c g)$ can be written as

$$\ddot{z}(t) = \begin{cases} 0, & m_c \ddot{x}_f(t) < F_f \\ \frac{F_f'}{m_c} - \ddot{x}_f(t), & m_c \ddot{x}_f(t) \geq F_f \end{cases} \qquad (7.8)$$

in which $z(t)$ denotes the sliding displacement of the rigid body, and $F_f$ and $F_f'$ are static and dynamic friction forces, respectively. The dynamic friction force $(F_f')$ is considered to be approximately equal to the static friction $(F_f)$ for simplicity. Both of them can be represented by

$$F_f' = F_f = \mu m_c g (1 - a_v) \qquad (7.9)$$

The initial conditions for sliding of the rigid body can be expressed as

$$z(t) = \dot{z}(t) = 0 \qquad (7.10)$$

By substituting Eq. (7.9) for $F_f'$ in Eq. (7.8), the equation of motion can be rewritten as

$$\ddot{z}(t) = \mu g (1 - a_v) - A_v a_x g \sin\frac{2\pi t}{T_s} \qquad (t_1 \leq t \leq t_3) \qquad (7.11)$$

7-3

with the solution of

$$\dot{z}(t) = \mu g(1 - a_v)(t - t_1) + \frac{T_s}{2\pi} A_v a_x g \left( \cos \frac{2\pi t}{T_s} - \cos \frac{2\pi t_1}{T_s} \right) \tag{7.12}$$

$$z(t) = 0.5\mu g(1 - a_v)(t - t_1)^2 + \frac{T_s}{2\pi} A_v a_x g \left[ \frac{T_s}{2\pi} \left( \sin \frac{2\pi t}{T_s} - \sin \frac{2\pi t_1}{T_s} \right) - (t - t_1) \cos \frac{2\pi t_1}{T_s} \right] \tag{7.13}$$

To determine the starting and ending time instants ($t_1$ and $t_3$), the following conditions are introduced:

$$\ddot{z}(t_1) = 0 \tag{7.14}$$

$$\dot{z}(t_3) = 0 \tag{7.15}$$

Solving these two equations simultaneously yields Eq. (7.5). The maximum sliding distance can then be calculated by

$$\Delta_p = |z(t_3)| = \frac{A_v a_x g T_s^2}{4\pi^2} C_\delta = C_\delta \delta_s \tag{7.16}$$

The values of $C_\delta$ for different values of the parameter $\eta$ in Eq. (7.6) are tabulated in Table 7-2.

## 7.3 Interstory Distortion

An architectural component such as a wall system is often subjected to distortion action due to story drift of its supporting building structure. A piping system inside a building structure is also restrained by story drift and a piping system attached to two adjacent buildings may be subjected to their differential movement. Therefore, interstory distortions inside a building structure or between two adjacent buildings are important to the design of this type of nonstructural components. However, constraint displacements of this type for nonstructural components can be exclusively calculated from structural analysis when the nonstructural components are considered to be relatively light. For this reason, explicit expressions for these interstory distortions are not discussed here.

**TABLE 7-1.   Displacement of Flexibly-Mounted Equipment at Top of Building (Soil Type I)**

| $T_s$ (seconds) | | 0.0 | 0.33 | 0.44 | 0.61 | 0.94 |
|---|---|---|---|---|---|---|
| Tuned $T_c=T_s$ | first recommendation | 0.0 | 0.034 | 0.050 | 0.077 | 0.137 |
| | second recommendation | 0.0 | 0.037 | 0.054 | 0.084 | 0.149 |
| | third recommendation | 0.0 | 0.051 | 0.075 | 0.115 | 0.201 |
| Detuned $T_c< 0.5\ T_s$ or $T_c>2.0\ T_s$ | first recommendation | 0.0 | 0.0034 | 0.0050 | 0.0077 | 0.0137 |
| | second recommendation | 0.0 | 0.0046 | 0.0067 | 0.0105 | 0.0186 |
| | third recommendation | 0.0 | 0.0051 | 0.0075 | 0.0115 | 0.0201 |
| $R_c$ in the first and third recommendations are taken as 1.0 for displacement determination to obtain the maximum elastic deformation presented above. | | | | | | |

**TABLE 7-2. Sliding Coefficient**

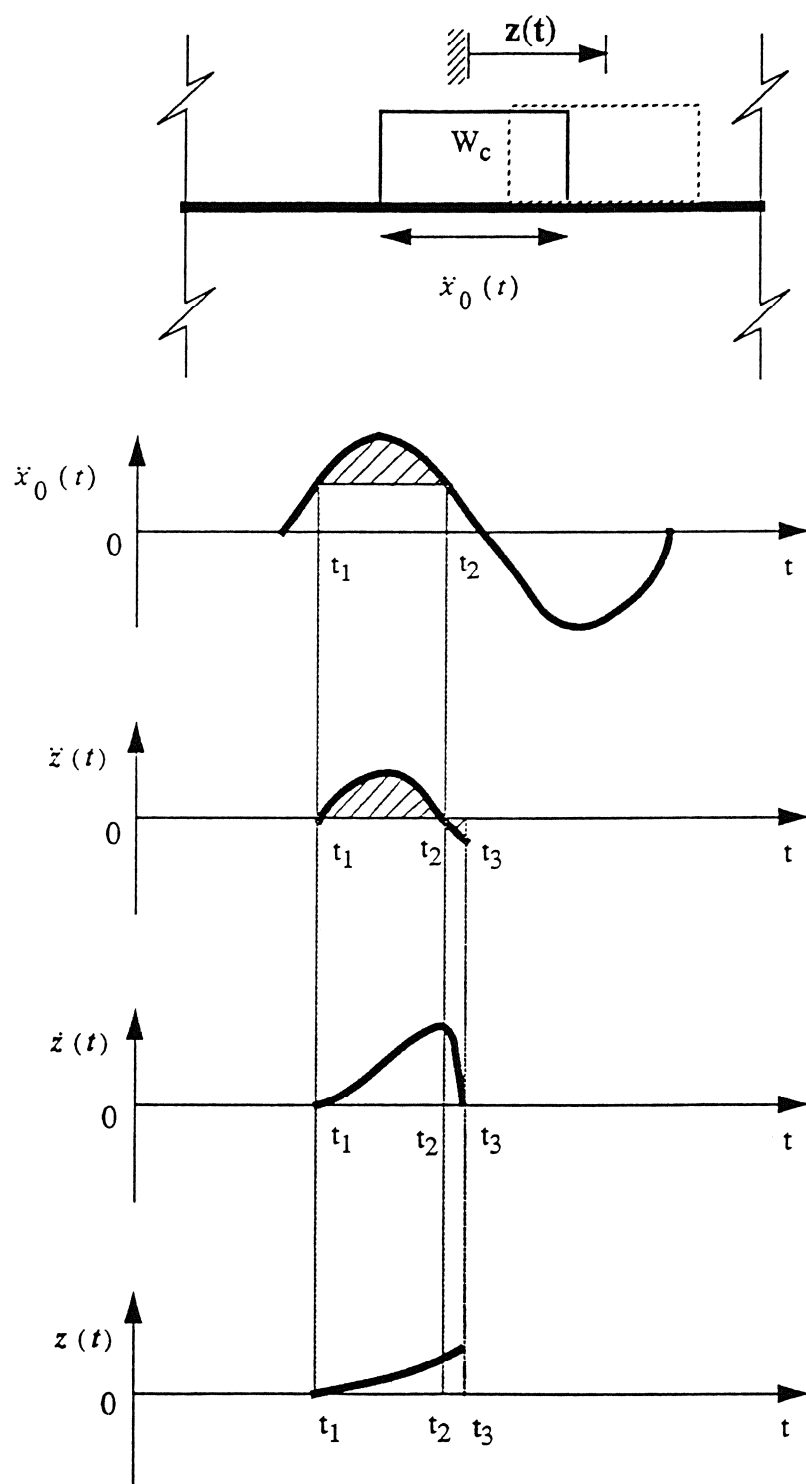| $\eta$ | 0.4 | 0.5 | 0.6 | 0.7 | 0.8 | 0.9 | 1.0 |
|---|---|---|---|---|---|---|---|
| $C_\delta$ | 1.8780 | 1.2658 | 0.7884 | 0.4325 | 0.1878 | 0.0460 | 0.0 |

Fig. 7-1 Sliding Displacement

# SECTION 8

## CONCLUDING REMARKS

Three approaches to the modification of the 1991 NEHRP design force equations for nonstructural components have been developed by incorporating different levels of consideration for structural and component effects. In the first recommendation, structural and component characteristics are jointly taken into account, which is no doubt the most comprehensive. The second recommendation is basically a structure-driven type modification to the current provisions. The third recommendation mainly concerns the effect of different types of components on the force, representing the least modified version.

Design force equations recommended in this report preserve the equivalent lateral force concept, a static procedure in which all the dynamic characteristics such as modal contribution, damping ratio, and response spectrum are not explicitly included. However, these statics-based equations can be justified based on dynamic analysis of the first mode representation of MDOF systems with 5% modal damping following the cascade (decoupled) procedure.

The values of the component response modification coefficient $(R_c)$ in this report are basically transferred from those of the seismic coefficient $(C_c)$ of the current provisions. They are subjected to further modification by practitioners.

An attempt was made to consider the structural yielding effect on the acceleration or inertia force distribution along building height. In general, the larger the structural inelastic deformation or the larger the seismic input, the more uniform the distribution of the maximum acceleration along the building height. However, it was decided in this revision to ignore the structural yielding effect on the acceleration distribution due to simplicity requirements for practical design and insufficient observed data for statistical analysis.

Overall, the recommended modifications of the 1991 NEHRP design force equations for nonstructural components represent a major effort which, on the one hand, preserves the equivalent lateral force format for practical applicability and, on the other, identifies and corrects deficiencies in the current provisions to the extent feasible. It has been shown that these recommended revisions can be justified on the basis of analyses, experimental

results, and observation data from past earthquakes; and they represent a significant improvement over the 1991 NEHRP design force formulas.

Current provisions do not include design guidelines based on displacements or deformations on the part of nonstructural components. Since excessive displacements or movements are causes of a significant number of past nonstructural failures, simple equations have also been presented which can be used to estimate flexible support deformation and the amount of sliding a nonstructural component can experience during a seismic event. The inclusion of this type of displacement equations in future codes and provisions is recommended.

# SECTION 9

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# APPENDIX

# OBSERVATION DATA FROM SELECTED PAST EARTHQUAKES

## Table A.1. Observation Data from Past Earthquakes

| Building Name | Hollywood Storage Building | | Hollywood Storage Building | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. frame 14/149.5' | R.C. frame 14/149.5' | R.C. frame 14/149.5' | R.C. frame 14/149.5' |
| Period (sec.) | 1.86 | 0.64 | 1.86 | 0.64 |
| Soil Conditions | Pile Foundation | Pile Foundation | Pile Foundation | Pile Foundation |
| Earthquake | Kern County July 21, 1952 | Kern County July 21, 1952 | Whitter Narrows October1, 1987 | Whitter Narrows October1, 1987 |
| Magnitude ($M_L$) | 7.2 | 7.2 | 5.9 | 5.9 |
| E. C. D (km) | 122 | 122 | 25 | 25 |
| Acc. at Base (g's) | 0.06 | 0.04 | 0.11 | 0.06 |
| Acc. at Top (g's) | 0.12 | 0.15 | 0.20 | 0.19 |
| Amplification Factor | 2.00 | 3.75 | 1.82 | 3.17 |

**Table A.1. (continued)**

| Building Name | Hollywood Storage Building | | Santa Clara County Office | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. frame 14/149.5' | R.C. frame 14/149.5' | Steel frame 13/186'7" | Steel frame 13/186'7" |
| Period (sec.) | 1.86 | 0.64 | 2.05 | 2.16 |
| Soil Conditions | Pile Foundation | Pile Foundation | ------ | ------- |
| Earthquake | Whitter Narrows October4, 1987 | Whitter Narrows October4, 1987 | Morgan Hill April 24, 1984 | Morgan Hill April 24, 1984 |
| Magnitude ($M_L$) | 5.3 | 5.3 | 6.2 | 6.2 |
| E. C. D (km) | 25 | 25 | 20 | 20 |
| Acc. at Base (g's) | 0.06 | 0.03 | 0.03 | 0.04 |
| Acc. at Top (g's) | 0.11 | 0.08 | 0.17 | 0.18 |
| Amplification Factor | 1.83 | 2.67 | 5.67 | 4.50 |

**Table A.1. (continued)**

| Building Name | Town Park Towers | | Great Western S & L Building | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. shear wall 10/95' | R.C. shear wall 10/95 | R.C. frame 10/124' | R.C. shear wall 10/124' |
| Period (sec.) | —— | —— | —— | —— |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Morgan Hill April 24, 1984 | Morgan Hill April 24, 1984 | Morgan Hill April 24, 1984 | Morgan Hill April 24, 1984 |
| Magnitude ($M_L$) | 6.2 | 6.2 | 6.2 | 6.2 |
| E. C. D (km) | 20 | 20 | 20 | 20 |
| Acc. at Base (g's) | 0.06 | 0.06 | 0.06 | 0.06 |
| Acc. at Top (g's) | 0.22 | 0.14 | 0.18 | 0.22 |
| Amplification Factor | 3.67 | 2.33 | 3.00 | 3.67 |

**Table A.1. (continued)**

| Building Name | Building 180   JPL, CalTech | | Bank of   California | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 9/146' | Steel Frame 9/146' | R.C. Frame 12/174' | R.C. Frame 12/174' |
| Period (sec.) | 1.33 | 1.05 | 1.41 | 2.52 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | San Fernado February 9,1971 | San Fernado February 9,1971 | San Fernado February 9,1971 | San Fernado February 9,1971 |
| Magnitude ($M_L$) | 6.5 | 6.5 | 6.5 | 6.5 |
| E. C. D (km) | 24 | 24 | 22.5 | 22.5 |
| Acc. at Base (g's) | 0.14 | 0.21 | 0.22 | 0.15 |
| Acc. at Top (g's) | 0.210 | 0.38 | 0.29 | 0.20 |
| Amplification Factor | 1.50 | 1.81 | 1.32 | 1.33 |

**Table A.1. (continued)**

| Building Name | Imperial County Service Building | | Freitas Building (Santa Babara) | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. frame 6/81.5' | R.C. wall 6/81.5' | R.C Shear wall 4/53' | R.C. Shear wall 4/53' |
| Period (sec.) | 0.44-0.61 | 0.64-1.25 | 0.5 | ____ |
| Soil Conditions | ____ | ____ | ____ | ____ |
| Earthquake | Imperial Valley October 15,1979 | Imperial Valley October 15,1979 | Santa Barbara August 13, 1978 | Santa Barbara August 13, 1978 |
| Magnitude ($M_L$) | 6.6 | 6.6 | 5.1-5.7 | 5.1-5.7 |
| E. C. D (km) | | | 20 | ----- |
| Acc. at Base (g's) | 0.29 | 0.33 | 0.23 | ----- |
| Acc. at Top (g's) | 0.58 | 0.45 | 0.55 | ----- |
| Amplification Factor | 2.00 | 1.36 | 2.39 | ____ |

## Table A.1. (continued)

| Building Name | Kajima Int. Building | | Holiday Inn | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 15/208.5' | Steel Frame 15/208.5" | R.C. Frame 7/66' | R.C. Frame 7/66' |
| Period (sec.) | 2.9 | 2.9 | 1.6 | 1.8 |
| Soil Conditions | ____ | ____ | ____ | ____ |
| Earthquake | San Fernado February 9,1971 | San Fernado February 9,1971 | San Fernado February 9,1971 | San Fernado February 9,1971 |
| Magnitude ($M_L$) | 6.5 | 6.5 | 6.5 | 6.5 |
| E. C. D (km) | 33.6 | 33.6 | 13 | 13 |
| Acc. at Base (g's) | 0.10 | 0.12 | 0.25 | 0.134 |
| Acc. at Top (g's) | 0.166 | 0.162 | 0.382 | 0.32 |
| Amplification Factor | 1.66 | 1.30 | 1.50 | 2.40 |

**Table A.1. (continued)**

| Building Name | Milllikan Library | | Watsonville Commercial | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Shear wall 9/144' | R.C. Shear wall 9/144' | Concrete shear 4/66.4' | Concrete shear 4/66.4' |
| Period (sec.) | 0.8 | 1.0 | 0.3 | 0.4 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | San Fernado February 9,1971 | San Fernado February 9,1971 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 6.5 | 6.5 | 7.0 | 7.0 |
| E. C. D (km) | 30.5 | 30.5 | 13 | 13 |
| Acc. at Base (g's) | 0.20 | 0.18 | 0.28 | 0.39 |
| Acc. at Top (g's) | 0.311 | 0.35 | 0.44 | 1.24 |
| Amplification Factor | 1.55 | 1.94 | 1.57 | 3.18 |

**Table A.1. (continued)**

| Building Name | Gilroy Historic Commercial | | San Jose Residential | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 3/49.6' | Steel Frame 3/49.6' | Concrete shear 10/90' | Concrete shear 10/90" |
| Period (sec.) | 0.6 | 0.6 | 1.0 | 0.6 |
| Soil Conditions | ——— | ——— | ——— | ——— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D  (km) | | | | |
| Acc. at Base (g's) | 0.20 | 0.20 | 0.10 | 0.13 |
| Acc. at Top (g's) | 0.67 | 0.58 | 0.37 | 0.24 |
| Amplification Factor | 3.4 | 2.9 | 3.7 | 1.85 |

**Table A.1. (continued)**

| Building Name | San Jose Govenrment | | San Jose Commercial | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 13/186.5' | Steel Frame 13/186.5' | R.C. frame 10/124' | R.C. frame 10/124' |
| Period (sec.) | 2.0 | 2.0 | 1.0 | 0.9 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.11 | 0.09 | 0.09 | 0.11 |
| Acc. at Top (g's) | 0.35 | 0.32 | 0.26 | 0.29 |
| Amplification Factor | 3.18 | 3.55 | 2.89 | 2.63 |

**Table A.1. (continued)**

| Building Name | S. San Francisco    Hospital | | San Francisco    Comm. Building | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 4/52.5' | Steel Frame 4/52.5' | Steel Frame 18/241.4' | Steel Frame 18/241.4' |
| Period (sec.) | 0.6 | 0.6 | 0.9 | 1.1 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.14 | 0.15 | 0.17 | 0.14 |
| Acc. at Top (g's) | 0.57 | 0.61 | 0.23 | 0.20 |
| Amplification Factor | 4.07 | 4.06 | 1.35 | 1.43 |

## Table A.1. (continued)

| Building Name | San Bruno Government | | San Bruno Office Building | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Concrete shear 9/104" | Concrete shear 9/104" | R.C. frame 6/78' | R.C. frame 6/78' |
| Period (sec.) | 1.1 | 1.0 | 1.0 | 1.1 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.11 | 0.11 | 0.14 | 0.12 |
| Acc. at Top (g's) | 0.23 | 0.32 | 0.25 | 0.32 |
| Amplification Factor | 2.09 | 2.91 | 1.78 | 2.67 |

**Table A.1. (continued)**

| Building Name | San Francisco Office Building | | Santa Rosa Residential | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 47/564' | Steel Frame 47/564' | R.C. frame 14/125' | R.C. frame 14/125' |
| Period (sec.) | 1.5 | 1.5 | 1.4 | 1.5 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D  (km) | | | | |
| Acc. at Base (g's) | 0.13 | 0..26 | 0.04 | 0.05 |
| Acc. at Top (g's) | 0.48 | 0.39 | 0.21 | 0.21 |
| Amplification Factor | 3.7 | 1.5 | 5.25 | 4.2 |

**Table A.1. (continued)**

| Building Name | Milpitas Industrial Bldg. | | Hayward Office Bldg. | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Concrete shear 2/31.5' | Concrete shear 2/31.5' | Concrete shear 6/72' | Concrete shear 6/72' |
| Period (sec.) | 0.33 | 0.2 | 0.7 | 0.8 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.10 | 0.14 | 0.10 | 0.11 |
| Acc. at Top (g's) | 0.14 | 0.58 | 0.24 | 0.34 |
| Amplification Factor | 1.4 | 4.14 | 2.4 | 3.09 |

**Table A.1. (continued)**

| Building Name | Hayward School Office Bldg. | | Oakland Residential Bldg | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel R.C. Frame 13/201" | Steel R.C.Frame 13/201' | Concrete shear 24/219' | Concrete shear 24/219' |
| Period (sec.) | 0.4 | 0.5 | 0.9 | 0.8 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.08 | 0.09 | 0.18 | 0.14 |
| Acc. at Top (g's) | 0.15 | 0.24 | 0.38 | 0.25 |
| Amplification Factor | 1.87 | 2.66 | 2.11 | 1.78 |

**Table A.1. (continued)**

| Building Name | Okland Office Bldg. | | Berkeley Hospital | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Shear wall 2/28' | R.C. Shear wall 2/28' | EB steel frame 2/25.3' | EB steel frame 2/25.3' |
| Period (sec.) | 0.3 | 0.6 | 0.4 | 0.3 |
| Soil Conditions | ——— | ——— | ——— | ——— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.20 | 0.25 | 0.12 | 0.13 |
| Acc. at Top (g's) | 0.25 | 0.45 | 0.28 | 0.26 |
| Amplification Factor | 1.25 | 1.8 | 2.33 | 2.0 |

**Table A.1. (continued)**

| Building Name | Richmond Govt. Office | | Walnut Creek Commercial Bld | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Frame 3/37.5' | R.C.Frame 3/37.5' | R.C. Frame 10/128.5' | R.C. Frame 10/128.5' |
| Period (sec.) | 0.4 | 0.3 | 0.7 | 0.9 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 7.0 | 7.0 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.12 | 0.09 | 0.10 | 0.05 |
| Acc. at Top (g's) | 0.23 | 0.20 | 0.21 | 0.17 |
| Amplification Factor | 1.92 | 2.22 | 2.1 | 3.4 |

**Table A.1. (continued)**

| Building Name | Pleasant Hill Commercial Blg. | | Watsonville Tel Building | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Concrete shear 3/40.5' | Concrete shear 3/40.5' | R.C Shear wall 4/66.3' | R.C Shear wall 4/66.3' |
| Period (sec.) | 0.3 | 0.6 | 0.4 | 0.4 |
| Soil Conditions | ——— | ——— | ——— | ——— |
| Earthquake | Loma Prieta Oct. 17, 1989 | Loma Prieta Oct. 17, 1989 | Morgan Hill April 24,1984 | Morgan Hill April 24,1984 |
| Magnitude ($M_L$) | 7.0 | 7.0 | 6.2 | 6.2 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.08 | 0.12 | 0.06 | 0.11 |
| Acc. at Top (g's) | 0.14 | 0.15 | 0.14 | 0.33 |
| Amplification Factor | 1.75 | 1.25 | 2.33 | 3.0 |

| Building Name | Kaiser Medical Center Bldg. | | CSULA Admst. Bldg | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C.Steel Frame 4/52.5' | R.C.Steel Frame 4/52.5' | R.C. shear wall 8/114' | R.C. shear wall 8/114' |
| Period (sec.) | 0.4 | 0.5 | 0.8 | 0.6 |
| Soil Conditions | ——— | ——— | ——— | ——— |
| Earthquake | Morgan Hill April 24,1984 | Morgan Hill April 24,1984 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 |
| Magnitude ($M_L$) | 6.2 | 6.2 | 6.1 | 6.1 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.02 | 0.03 | 0.31 | 0.39 |
| Acc. at Top (g's) | 0.11 | 0.17 | 0.48 | 0.27 |
| Amplification Factor | 5.5 | 5.6 | 1.55 | 0.69 |

**Table A.1. (continued)**

| Building Name | Burbank Federal Savings Bldg. | | Burbank Pacific Manor | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel frame 6/82.5' | Steel frame 6/82.5' | Concrete shear 10/88.7' | Concrete shear 10/88.7' |
| Period (sec.) | 0.8 | 0.8 | 0.7 | 0.6 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 |
| Magnitude ($M_L$) | 6.1 | 6.1 | 6.1 | 6.1 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.22 | 0.17 | 0.18 | 0.22 |
| Acc. at Top (g's) | 0.30 | 0.17 | 0.34 | 0.54 |
| Amplification Factor | 1.37 | 1 | 1.89 | 2.45 |

**Table A.1. (continued)**

| Building Name | N. Holloywood | Shearaton Hotel | CSULB | Engr. Bldg |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Frame 23/177' | Steel R.C.Frame 23/177' | Concrete shear 5/71' | Concrete shear 5/71' |
| Period (sec.) | 0.6 | 0.5 | 0.35 | 0.4 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 |
| Magnitude ($M_L$) | 6.1 | 6.1 | 6.1 | 6.1 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.11 | 0.09 | 0.1 | 0.1 |
| Acc. at Top (g's) | 0.17 | 0.13 | 0.13 | 0.36 |
| Amplification Factor | 1.54 | 1.44 | 1.3 | 3.6 |

**Table A.1. (continued)**

| Building Name | Long Beach Harbor Adm. | | UCLA Math Sci. Bldg | |
| --- | --- | --- | --- | --- |
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | Steel Frame 7/91' | Steel Frame 7/91' | R.C. Frame 7/94.5' | R.C. Frame 7/94.5' |
| Period (sec.) | 1.2 | 1.5 | 0.6 | 0.5 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 |
| Magnitude ($M_L$) | 6.1 | 6.1 | 6.1 | 6.1 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.05 | 0.07 | 0.05 | 0.04 |
| Acc. at Top (g's) | 0.12 | 0.11 | 0.12 | 0.05 |
| Amplification Factor | 2.4 | 1.57 | 2.4 | 1.2 |

**Table A.1. (continued)**

| Building Name | Union Bank Building | | Van Nuys Holiday Inn | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Frame 13/164' | R.C.Frame 13/164' | R.C. Frame 7/66' | R.C. Frame 7/66' |
| Period (sec.) | —— | —— | — | — |
| Soil Conditions | —— | —— | — | — |
| Earthquake | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 |
| Magnitude ($M_L$) | 6.1 | 6.1 | 6.1 | 6.1 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.18 | 0.11 | 0.16 | 0.16 |
| Acc. at Top (g's) | 0.14 | 0.14 | 0.20 | 0.17 |
| Amplification Factor | 0.77 | 1.27 | 1.26 | 1..06 |

**Table A.1. (continued)**

| Building Name | 1st Federal   Savings Blgd | | Pasadena   Office Bldg | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Frame 2/30' | R.C. Frame 2/30 | Masonary wall 6/82' | Masonary wall 6/82' |
| Period (sec.) | 0.33 | 0.3 | 0.5 | 0.5 |
| Soil Conditions | —— | —— | —— | —— |
| Earthquake | Whittier Oct. 1,1987 | Whittier Oct. 1,1987 | Sierra Madre,CA June 28, 1991 | Sierra Madre,CA June 28, 1991 |
| Magnitude ($M_L$) | 6.1 | 6.1 | 5.8 | 5.8 |
| E. C. D  (km) | | | | |
| Acc. at Base (g's) | 0.05 | 0.05 | 0.20 | 0.14 |
| Acc. at Top (g's) | 0.15 | 0.14 | 0.24 | 0.16 |
| Amplification Factor | 3 | 2.8 | 1.2 | 1.11 |

**Table A.1. (continued)**

| Building Name | Pasadena Comm. Bldg | | Burbank Residential | |
|---|---|---|---|---|
| | N-S | E-W | N-S | E-W |
| Configuration (story/ height) | R.C. Frame 9/117' | R.C. Frame 9/117' | R.C. shear wall 10/88' | R.C. shear wall 10/88' |
| Period (sec.) | 1.2 | 1.2 | 0.8 | 0.8 |
| Soil Conditions | ——— | ——— | ——— | ——— |
| Earthquake | Sierra Madre,CA June 28, 1991 | Sierra Madre,CA June 28, 1991 | Sierra Madre,CA June 28, 1991 | Sierra Madre,CA June 28, 1991 |
| Magnitude ($M_L$) | 5.8 | 5.8 | 6.6 | 6.6 |
| E. C. D (km) | | | | |
| Acc. at Base (g's) | 0.24 | 0.11 | 0.12 | 0.08 |
| Acc. at Top (g's) | 0.43 | 0.14 | 0.34 | 0.18 |
| Amplification Factor | 1.79 | 1.27 | 2.83 | 2.25 |

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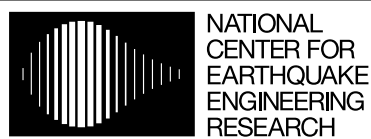
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