Effect of Ground Motion Duration on Seismic Design Load for Civil Engineering Structures

—Development of Equivalent Ground Acceleration (EQA)—

By

Hiroyuki KAMEDA* and Kazunori KOHNO**

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Abstract

The effect of ground motion duration on seismic loads for structural design is studied in terms of the equivalent ground acceleration (EQA). EQA is obtained by multiplying the peak ground acceleration (PGA) by a reduction factor (EQA factor), which takes into account the peak response ratio relative to its standard values and the effective level of response affecting structural failure. Both are evaluated in connection with the duration of the input ground motion. EQA is considered to be a direct measure of destructiveness of earthquake ground motion, and will provide straightforward information for determining and interpreting the design seismic load.

An EQA model is proposed and formulated. On this basis, the effect of a strong motion duration on the seismic design load is discussed, using numerical results for recorded accelerograms. Statistical models are developed so that EQA can be estimated on a hand-caluculator basis.

1. Introduction

It is common in engineering practice for seismic design of structures to represent the seismic load in acceleration terms. In the Japanese seismic design specifications for highway bridges[4),7)], seismic loads are represented in the following forms or those equivalent to them.

i) elastic design:

$$k_{hm}=\beta k_h \qquad \cdots\cdots\cdots\cdots(1)$$

ii) ultimate design (deformation capacity):

$$d_a=\alpha k_{hd} \qquad \cdots\cdots\cdots\cdots(2)$$

in which k_{hm}=design response acceleration (%g), β=acceleration response ratio (acceleration response / ground acceleration), k_h=ground acceleration (%g) for

* School of Civil Engineering, Kyoto University
** Maeda Construction Co.

elastic design, d_d=design displacement α=displacement response ratio (displacement response / ground acceleration) for equivalent linear models corresponding to hysteretic structures, and k_{hd}=ground acceleration (%g) for ultimate design. The forms of Eqs. (1) and (2) are common in seismic design codes throughout the world, in that the structural response is given by ground acceleration times the dynamic response amplification.

The actual values for the design ground accelerations k_h and k_{hd} have been determined in a semi-empirical manner on the basis of the results of structural response analyses, combined with judgements in the light of damage data from past earthquakes. It is obvious that design ground accelerations can not be determined only from the peak ground acceleration (PGA) and the spectral response amplification. It is also affected by the duration of the ground motion. Structural failures are more or less progressive phenomena, so that the states of structural damage are strongly influenced by the duration of the earthquake excitation. Lack of rational techniques to incorporate the effect of the ground motion duration on the structural damage have kept the determination of seismic loads for a structural design highly judgemental. This has prevented the strong motion data, accumulated abundantly by this time, from being used for a direct determination of seismic design loads. Therefore, it is believed that the development of an appropriate technique to incorporate the ground motion duration will be a strong step-ahead toward the development of seismic load specifications on a more rational basis.

The objective of this study is to formulate the effect of the ground motion duration on structural response and damage, and to develop a ground motion intensity parameter in which these effects are incorporated. Specifically, the intensity parameter developed herein is called the equivalent ground acceleration (EQA). EQA is an acceleration value which in principle can be used directly for the hazard parameter to be employed for representing design ground accelerations, such as k_h and k_{hd} in Eqs. (1) and (2), respectively.

Works have been performed by several authors[1),2),10),11)] in order to reduce PGA into an acceleration level that is intended to conform with the structural response or failure. The reduced accelerations, defined in different manners, have been called the effective peak acceleration (EPA). In the Tentative Provisions for Buildings[1)] (ATC-3) developed by the Applied Technology Council, EPA has been defined as the smoothed acceleration response spectra divided by its magnification factor of 2.5. It has been defined for a short period range (0.1~0.5 sec), whereas for a larger period (1 sec), "effective peak velocity based acceleration" has been defined in a similar manner. These ideas are clear-cut as far as it is claimed that the seismic design should be based on the peak response values and if the corresponding response spectra can be uniquely defined. However, EPA as outlined above needs refinements, particularly regarding the effects of the ground motion duration. The necessity for such improvements has been pointed out by ATC-3 itself.

The widely recognized idea that PGA does not necessarily have significant effects on the structural effects of ground motion has been demonstrated by Blume[2], and later by Watabe and Tohdo[10]. They used strong motion accelerograms whose peaks were cut off above certain prescribed levels. Then it has been shown that a decrease in the spectrum intensity through these modifications is much smaller than the decrease in PGA. The cut-off level of acceleration has been called the effective acceleration[2], or EPA[10]. These results demonstrate qualitatively that high frequency acceleration spikes in ground motions have little effect on the structural response. However, EPA defined in this manner consequently depends on an arbitrary cut-off level, which makes its quantification on an objective basis difficult. The EPA values using the above definition should be strongly affected by the ground motion duration. However, no discussion regarding this has been made.

It may further be pointed out that the definitions of EPA in references 1), 2), and 10) have been developed only in connection with the peak value of the structural response. The design seismic load should be determined on the basis not only of the response amplitude but also of the limit states, or more generally, the damage states of structures.

Whitman[11] defined EPA in relation to the intensity and the duration of earthquake motions. This definition has been proposed for soil liquefaction susceptibility. When it is extended to structural problems, not only the structural response but the process of damage growth should also be taken into account.

In view of the problem as stated above, this study is aimed at developing a straightforward method to determine the effective level of the acceleration of a recorded strong motion accelerogram. This method will give due consideration to the effect of the ground motion duration on the state of the progressive structural failure as well as that on the response amplitude. These effects are incorporated in a reduction factor to be multiplied with PGA to generate a reduced value (in some cases increased value) of acceleration. The terminology "equivalent ground acceleration", (EQA), is used for this reduced (or increased) acceleration in a sense that using EQA for the ground acceleration in semi-static seismic design formats of the form of Eqs. (1) and (2), either elastic or inelastic, will be equivalent to performing a dynamic analysis by which the design response level is determined.

Although it has been emphasized repeatedly that EQA is to conform with Eqs. (1) and (2), it certainly does not mean that Eqs. (1) and (2) should be the best form of a seismic design format. They may even be misleading in that the acceleration terms appearing there can be misunderstood to represent PGA. This paper is intended to provide a quantitative tool to understand the physical meaning of the acceleration terms in those design formats, and to convert an instantaneous peak value of acceleration, PGA, into a static design acceleration by using the concept of EQA. It will be useful not only for developing methodologies to determine design parameter values on a more rational basis within the frame of current design formats, but also for understanding a need for new design methods that implement the effects of ground motion duration.

Following in this paper is Chapter 2 which deals with the basic definition and formulation of EQA. In Chapter 3, a more practical definition of EQA is presented. As another acceleration measure which is convenient for general discussion, the "average equivalent acceleration", (AEQA), is also defined. In Chapter 4, two statistical models are developed for estimation of EQA and AEQA. One is to estimate those for specific accelerograms, and the other for given values of earthquake magnitude and distance. Following Chapter 5 presenting comments on future developments, the results and conclusions of this study are summarized in Chapter 6.

2. Basic Formulation of Equivalent Ground Acceleration

2.1 Definition of EQA and the EQA Factor

The equivalent ground acceleration (EQA), denoted by A_e, may be defined by the following simple form.

$$A_e = C_e A_p \qquad \cdots\cdots\cdots\cdots(3)$$

where A_p=peak ground acceleration (PGA), and C_e=reduction factor to convert PGA into EQA (hereafter called the EQA factor). From the definition of EQA by Eq. (3), it is the main subject herein to develop a rational method to evaluate the EQA factor C_e.

The basic relationship between A_p and A_e may be illustrated by Fig. 1. When there are two accelerograms with equal PGA's but very different values for the duration T_d, a number of experiences in past earthquakes demonstrate that the ground motion with a longer duration has much larger destructive effects on structures than

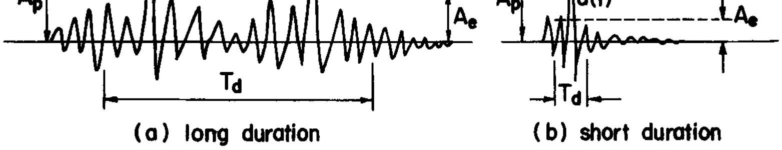

Fig. 1. Ground Motion Accelerograms and Illustration of EQA.

the one with a shorter duration. Therefore, as illustrated in Fig. 1, the value for A_e for a long duration should be larger than that for a short duration. The EQA factor C_e must be formulated in this direction through a quantitative analysis of physical processes of structural response and failure.

The EQA factor C_e is represented as a function of relevant parameters in the following manner.

$$C_e = C_e(T_d,\ T_0,\ h,\ \mu,\ \text{structural capacity}) \qquad \cdots\cdots\cdots\cdots(4)$$

where T_d=ground motion duration, T_0=undamped natural period, h=damping factor, and μ=maximum ductility factor of inelastic response. The ground motion duration T_d represents the effect of nonstationarity as explained by Fig. 1. The natural period T_0 certainly represents the effect of the response amplitude, and through this, it also incorporates the effect of the frequency content of the ground motion. The maximum ductility factor μ has been included as a ruling parameter to consider the nonlinear characteristics of the inelastic response. The structural capacity, particularly that for repeated response peaks, will affect the value of EQA. No specific quantitative expressions have been provided in Eq. (4), since there are a variety of modes of progressive failure depending on the type of structures and structural members. In this study, a simple damage model is employed for illustration, which will be explained in 2.3.

Several definitions have been proposed for the ground motion duration T_d. Herein, the following formula developed by Vanmarcke and Lai[9] is employed.

$$T_d = 7.5\frac{P_t}{A_p^2} \qquad \cdots\cdots\cdots\cdots(5)$$

where

$$P_t = \int a^2(t)\,\mathrm{dt} \qquad \cdots\cdots\cdots\cdots(6)$$

in which a(t)=acceleration time history. This definition is used since it represents the duration of relatively short time ranges including only the strong part of accelerograms which is of our concern in developing EQA. Vanmarcke and Lai have also developed a more detailed definition of the ground motion duration which includes the effect of the predominant period. The simplified formula of the form of Eq. (5) has been developed, assuming a constant peak factor.

Structural damage under earthquake motion is governed by two kinds of structural behavior. One is how large the peak response will be, and the other is how long the response will last. Both of these two factors are greatly affected by the ground motion duration. To discuss these effects separately, the EQA factor C_e in Eq. (4) may be divided into two parts as

$$C_e(T_d,\ T_0,\ h,\ \mu,\ \text{structural capacity})$$
$$=\gamma(T_d,\ T_0,\ h)\ \eta\ (T_d,\ T_0,\ \mu,\ \text{structural capacity}) \qquad \text{.............(7)}$$

where γ=peak response factor, and η=effective response factor. The parameters γ and η are discussed in detail in the following.

2.2 Formulation of Peak Response Factor γ

Let ξ generally denote the response ratio; i.e.,

$$\xi=\frac{S_p}{A_p} \qquad \text{.............(8)}$$

where S_p=response spectrum. To be specific, ξ will be written as ξ_A when S_p stands for acceleration response, and as ξ_D when S_p stands for displacement response.

The effect of ground motion duration on the response spectrum is straightforward. For a ground motion with a short duration, the structural response amplitude is suppressed due to the transient effect, whereas if the ground motion duration is large, the structural response will follow the variation of the ground motion intensity in a quasi-stationary state making the response amplitude fully developed. The acceleration response ratio ξ_A has been computed for strong motion data with various values of duration. The average response ratio $\bar{\xi}_A$ has been obtained by averaging ξ_A over a range of the natural period of 0.1~5.0 sec. Fig. 2 shows the relation between $\bar{\xi}_A$ and the ground motion duration T_d, in which $\bar{\xi}_A$ clearly tends to increase with T_d. A considerable scatter of data in Fig. 2 is caused by the difference in the frequency content among the strong motion data. This scatter is remarkably reduced by going through the site classification, as will be seen

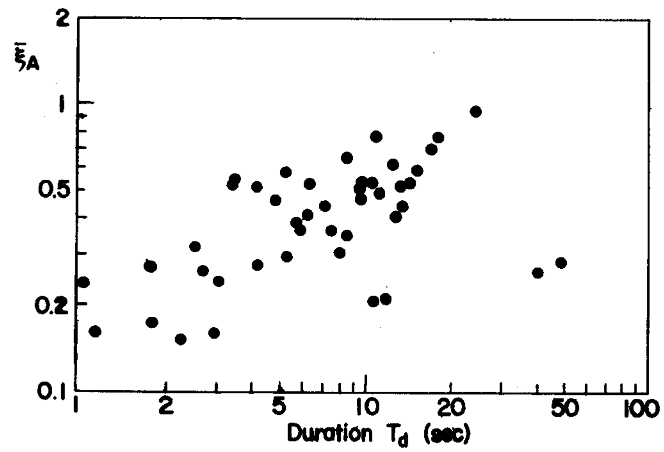

Fig. 2. Relation between Average Acceleration Response Ratio and Ground Motion Duration.

later.

From these arguments, it is clear that the effect of ground motion duration on the response ratio should be measured in a relative manner by comparing ground motions with different durations. For this purpose, it is convenient to introduce standard values for $\bar{\xi}$. These standard values may be obtained by averaging ξ over a number of strong motion records.

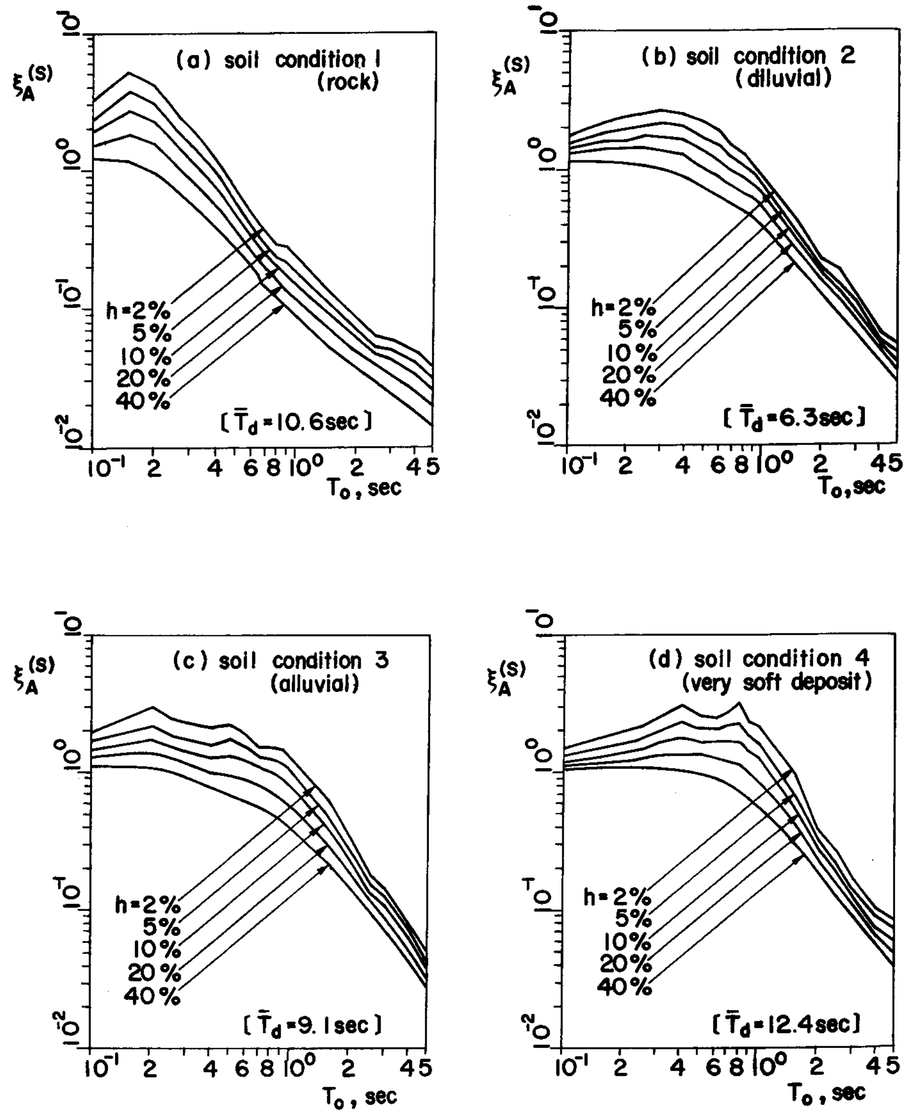

Fig. 3. Standard Acceleration Response Ratio Based on Strong Motion Data in Table A.1. (Model-I: See Table A.2 also.)

Let $\xi^{(s)}$ denote such a standard value for ξ. The design response ratios β and α in Eqs. (1) and (2), respectively, can be regarded as examples of $\xi^{(s)}$, specifically of $\xi_A^{(s)}$ and $\xi_D^{(s)}$, respectively.

Throughout this chapter, the mean values for ξ_A computed from the Japanese strong motiom data, listed in Table A.1 in Appendix A, will be used as the standard values $\xi_A^{(s)}$. They are plotted in Fig. 3, and their numerical values are listed in Table A.2. They have been obtained in terms of the harmonic mean, since it gives the least systematic bias of the scatter of individual data[8]. The four data groups in Fig. 3 correspond to the site classification employed in the Japanese seismic design specifications for highway bridges[4]. The soil conditions 1~4 roughly stand for rock, diluvial, alluvial, and very soft deposite sites, respectively. They suggest that the ground will be softer in this order. As Table A.1 indicates, ten strong motion data have been used to obtain $\xi_A^{(s)}$ for each soil condition. The geometric mean ground motion duration $\bar{T}_d$ for each data group is also indicated in Fig. 3. These

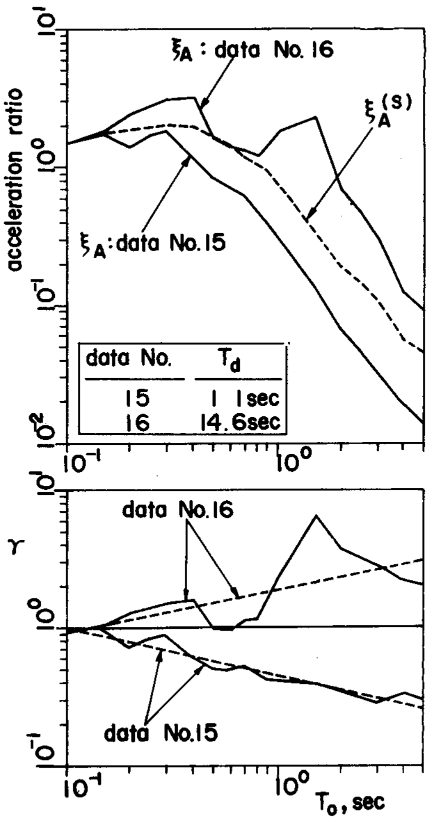

Fig. 4. Examples of Acceleration Response Ratio and Peak Response Factor γ. (soil condition 2; h=0.05)

strong motion data have been selected so that the ground motion duration will be distributed in a wide range within each group.

With the standard response ratio thus obtained, the peak response factor $\gamma(T_d, T_0, h)$ in Eq. (7) can be evaluated by

$$\gamma(T_d, T_0, h)=\frac{\xi}{\xi^{(s)}} \qquad \cdots\cdots\cdots\cdots(9)$$

For a large value of the ground motion duration T_d, relative to $\bar{T}_d$, ξ will be generally larger than $\xi^{(s)}$, making the values of γ larger than unity, and vise-versa for a small T_d. This effect is observed clearly in Fig. 4, Fig. 4(a) showing ξ_A and $\xi_A^{(s)}$, and Fig. 4(b) showing γ. Note also that the effect of T_d on γ becomes stronger as the natural period T_0 increases. This is a consequence of the fact that long period structures repeat fewer response cycles than short period structures, so that the transient nature of the response predominates. By using $\gamma(T_d, T_0, h)$ defined as above, the effect of the ground motion duration on the peak response amplitude can be incorporated in EQA.

Suppose that the peak response factor $\gamma(T_d, T_0, h)$ has been determined numerically on the basis of a certain standard response ratio $\xi^{(s)}$. If one wishes to discuss the same subject by referring to another standard response ratio $\xi^{(s)\prime}$, then a new peak response factor $\gamma'(T_d, T_0, h)$ corresponding to $\xi^{(s)\prime}$ will be obtained through the following simple modification of $\gamma(T_d, T_0, h)$.

$$\gamma'(T_d, T_0, h)=\frac{\xi^{(s)}}{\xi^{(s)\prime}}\gamma(T_d, T_0, h) \qquad \cdots\cdots\cdots\cdots(10)$$

2.3 Formulation of Effective Response Factor η

The effective response factor takes into account the effect of the ground motion duration on the progressive failure of structures.

Suppose that $x(t)$ in Fig. 5 shows a structural response time history, either in

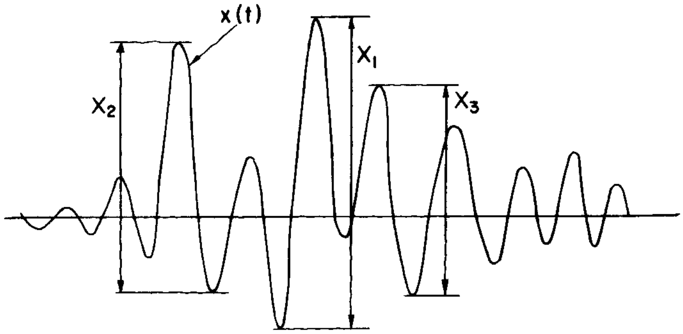

Fig. 5. First, Second, Third, etc. Largest Response Excursions.

terms of acceleration or displacement. Let X_1, X_2, X_3, ... represent the largest, second largest, third largest, ... response excursions in the time history of $x(t)$. The greatest effect of $x(t)$ on the structural failure will certainly be attributed to X_1. However, not only that, X_2, X_3, etc. will have joint effects on the progressive failure of the structure. To deal with this, consider an equivalent level of response at which response peaks with a constant amplitude are repeated by a certain prescribed number of times so that they have the same damaging effect as the original response $x(t)$. This equivalent response level, denoted by S_e, will be called the "effective response".

The effective response factor η to be used in Eq. (7) is defined as the ratio of the effective response level S_e to the value of the response spectrum S_p; i.e.,

$$\eta=\frac{S_e}{S_p} \qquad \text{............(11)}$$

The kind of response quantity that should be used for determining η depends on what part of the structure is to be discussed. Take the seismic design of highway bridges, for example. Major seismic failures of highway bridges are due to a bending failure in the lower part of piers, and a brittle shear failure of the supports at girder-pier connections. The former is dominated by the pier-top displacement, and the latter is dominated by the acceleration response of girders. Response quantity to be dealt with should be selected on this basis. Within the range of elastic response, which of the displacement or the acceleration response is used to determine η will not cause much difference. In the case of an inelastic response, however, the displacement response and the acceleration response will generate considerably different values of η as will be seen later.

A practical determination of η will depend on the characteristics of progressive failure which will vary with the type of structures and structural members, and the type of structural materials used. It is required that a damage rule should be developed for each mode of progressive failure. In this study, the following simple damage model is employed for an illustration of EQA.

It is common that the test results on the cyclic-load carrying capacity of RC members of a bending type are presented in terms of the allowable value of ductility factor μ_u and the allowable number of deformation cycles n_e (at $\mu=\mu_u$). This idea is combined with the concept of cumulative damage in fatigue. A linear law of cumulative damage is represented by

$$D=\sum_{i=1}^{n}(pZ_i^q) \qquad \text{...........(12)}$$

where D=cumulative damage, n=number of load cycles, Z_i=amplitude of i-th load reversal, and p, q=constants. Eq. (12) together with the allowable number of cycles n_e are applied to formulate the effective response factor.

Instead of using the i-th of the successive response peaks, we employ the largest, second largest, third largest, ...response excursions X_1, X_2, X_3, ...in Fig. 5, for Z_1, Z_2, Z_3, etc. in Eq. (12). Also, the summation in Eq. (12) will be taken up to n_e instead of n. Then the effective response level X_e in terms of the response excursion is given by

$$n_e p X_e^q = \sum_{i=1}^{n_e} (p X_i^q) \qquad \text{............(13)}$$

from which we have

$$X_e = \sqrt[q]{\frac{1}{n_e} \sum_{i=1}^{n_e} X_i^q} \qquad \text{............(14)}$$

From this, the effective response factor can be defined in a manner analogous to Eq. (11). Specifically, the effective response factor for the displacement response is written as

$$\eta_D = \sqrt[q]{\frac{1}{n_e} \sum_{i=1}^{n_e} X_{Di}^q} \Big/ X_{D1} \qquad \text{............(15)}$$

and that for acceleration response as

$$\eta_A = \sqrt[q]{\frac{1}{n_e} \sum_{i=1}^{n_e} X_{Ai}^q} \Big/ X_{A1} \qquad \text{............(16)}$$

in which X_{Di} and X_{Ai}=values for X_i corresponding to the displacement response and the acceleration response, respectively.

The parameter q should be determined from extensive experimental works. In this study, $q=1$ is used for numerical examples in Chapters 2 and 3. The statistical models developed in Chapter 4 deals with a range of $q=1\sim3$.

Numerical examples of η_D and η_A for elasto-plastic systems with $q=1$ are shown in Fig. 6. The maximum ductility factor is $\mu=3$, and the damping factor h =0.05. The value of n_e has been chosen at 1, 3, 6, 10, and 15 considering that $n_e=10$ is normally the allowable number of deformation cycles for the ductile RC flexural members subjected to cyclic deformation at an ultimate amplitude.

It may be observed in Fig. 6 that the effective response factor for displacement, η_D, assumes values smaller than those for acceleration, η_A. This is a consequence of a plastic drift in the displacement response which makes the maximum excursion X_{D1} considerably larger than X_{D2}, X_{D3}, ... When $\mu=1$, elastic response, it has been found that η_D almost coincides with η_A. It may also be noted in Fig. 6 that the values of the effective response factors η_D and η_A are large for a long ground motion duration and small for a short ground motion duration. This is an effect

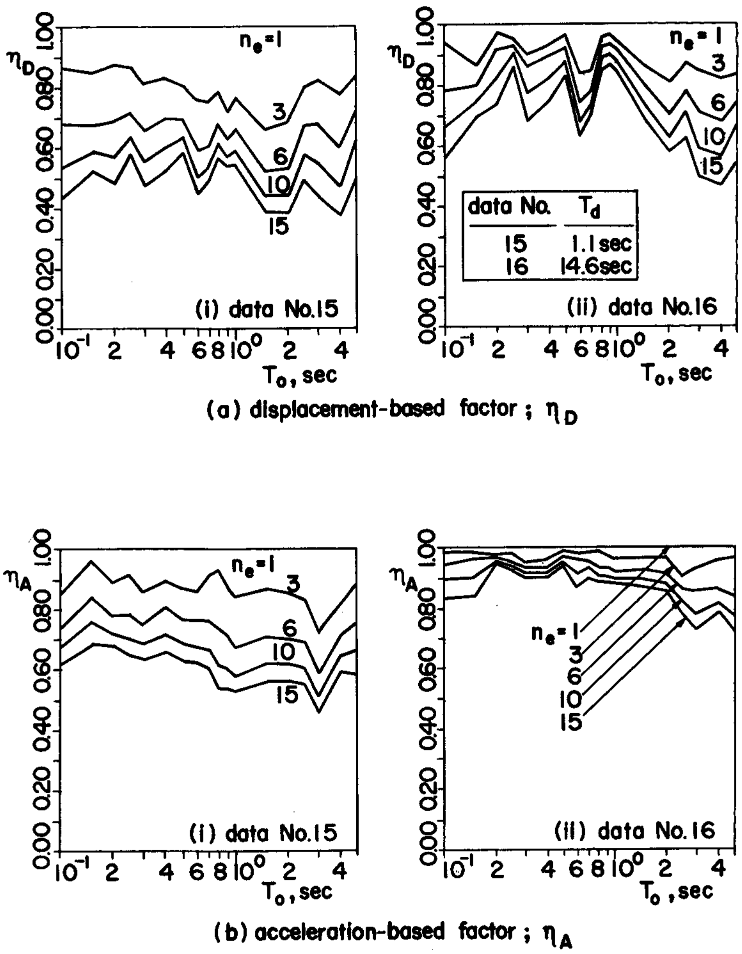

Fig. 6. Examples of Effective Response Factor η. ($\mu=3$, $q=1$)

of the ground motion duration on structural damage particularly in relation to its progressive failure; i.e., the shorter the ground motion duration, the smaller the damaging effect on structures.

2.4 Numerical Examples of EQA and Effective Response Spectra

Numerical examples for the foregoing discussion are shown in Figs. 7 and 8. The values of $\xi_A^{(s)}$ in Fig. 3 have been used for the standard acceleration response ratio.

The solid lines in Fig. 7 are the plots of the EQA factor C_e. The results for data no. 15, Figs. 7(a) (i) and 7(b) (i), are for a short ground motion duration $T_d=1.1$ sec, relative to $\bar{T}_d=6.3$ sec. In this case, the value of C_e decreases with the natural period T_0. In contrast, the value of C_e for data no. 16 with $T_d=14.6$ sec increases

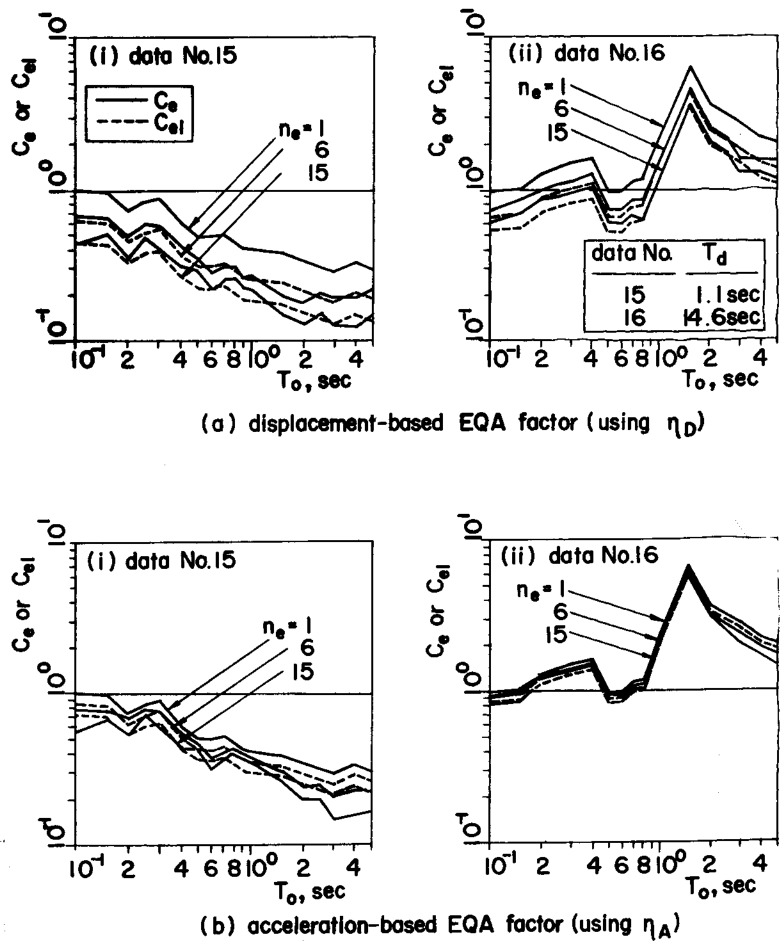

Fig. 7. Examples of EQA Factor (soil condition 2; $\mu=3$, $q=1$).

with T_0. This is obviously the effect of the peak response factor γ discussed in regard to Fig. 4. The decreasing trend of C_e values with increasing n_e is the consequence of the effect of the effective response factor η. Observe that this effect of n_e on C_e is stronger for shorter values of T_d. It may be observed also that the value of n_e has a larger effect on the displacement-based C_e than the acceleration-based C_e. These results are consistent with the characteristics of η discussed in 2.3.

From Eqs. (3), (7), (8), (9) and (11), the effective response S_e is represented by

$$S_e = \xi^{(s)} A_e \qquad \text{...........(17)}$$

or

$$S_e = \xi^{(s)} C_e A_p \qquad \text{...........(17')}$$

The solid lines in Fig. 8 show example results for the effective response in acceleration,

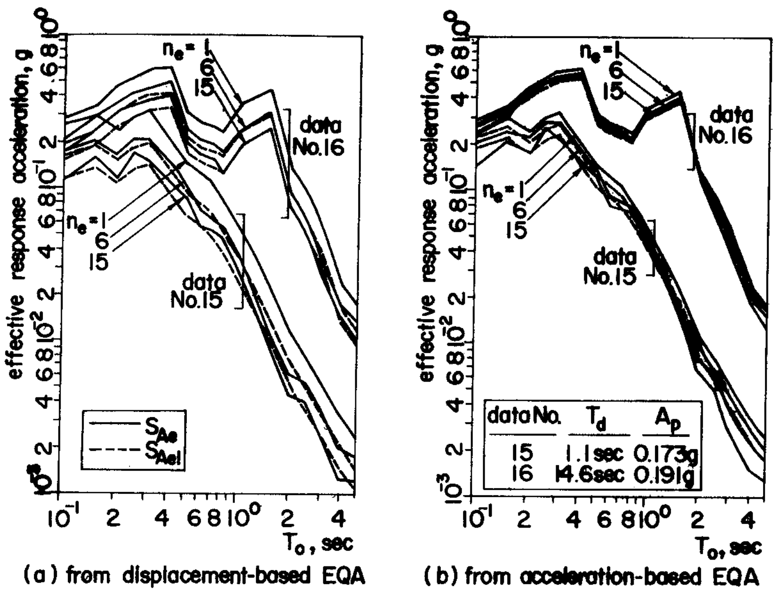

Fig. 8. Examples of Effective Response ($S_{Ae}=\xi_A^{(s)}A_e$, $S_{Ae1}=\xi_A^{(s)}A_{e1}$; soil condition 2, $\mu=3$, $q=1$, $h=0.05$).

denoted by S_{Ae}, with $h=0.05$. The two strong motion data used for Fig. 8 have similar values of PGA, A_p. However, the values of S_{Ae} for these two data are quite different, particularly for large values of the natural period. It will be verified later in connection with Fig. 20 that this difference comes rather from the effect of the ground motion duration T_d, than from the difference in the frequency content of the ground motions. As to the effect of n_e on S_{Ae}, the same argument as that regarding the effect of n_e on η holds.

3. Practical and Simplified Definitions of EQA

3.1 EQA Based on Average Effective Response Factor

Within the range of the natural period dealt with in this study, $T_0=0.1\sim5.0$ sec, it may be assumed that the effective response factor η does not vary systematically with T_0. The numerical results in Fig. 6 and the results for other strong motion data in Table A.1 seem to justify this assumption.

From this, the following average effective response factor η_a may be used instead of η.

$$\eta_a=\int_{T_1}^{T_2}X_e(T_0)dT_0\Big/\int_{T_1}^{T_2}X_1(T_0)dT_0 \qquad \text{...........(18)}$$

in which $T_1=0.1$ sec, and $T_2=5.0$ sec. Specifically, the average effective response factors for the displacement response and the acceleration response are represented, respectively, by

$$\eta_{Da}=\int_{T_1}^{T_2} X_{De}(T_0)dT_0 \Big/ \int_{T_1}^{T_2} X_{D1}(T_0)dT_0 \qquad \text{............(18')}$$

and

$$\eta_{Aa}=\int_{T_1}^{T_2} X_{Ae}(T_0)dT_0 \Big/ \int_{T_1}^{T_2} X_{A1}(T_0)dT_0 \qquad \text{............(18'')}$$

It will be practical and significant to replace the definition of EQA given by Eqs. (3) and (7) by the following form.

$$A_{e1}=C_{e1}A_p \qquad \text{............(19)}$$

where

$$C_{e1}=\gamma(T_d,\ T_0,\ h)\eta_a(T_d,\ \mu,\ n_e) \qquad \text{............(20)}$$

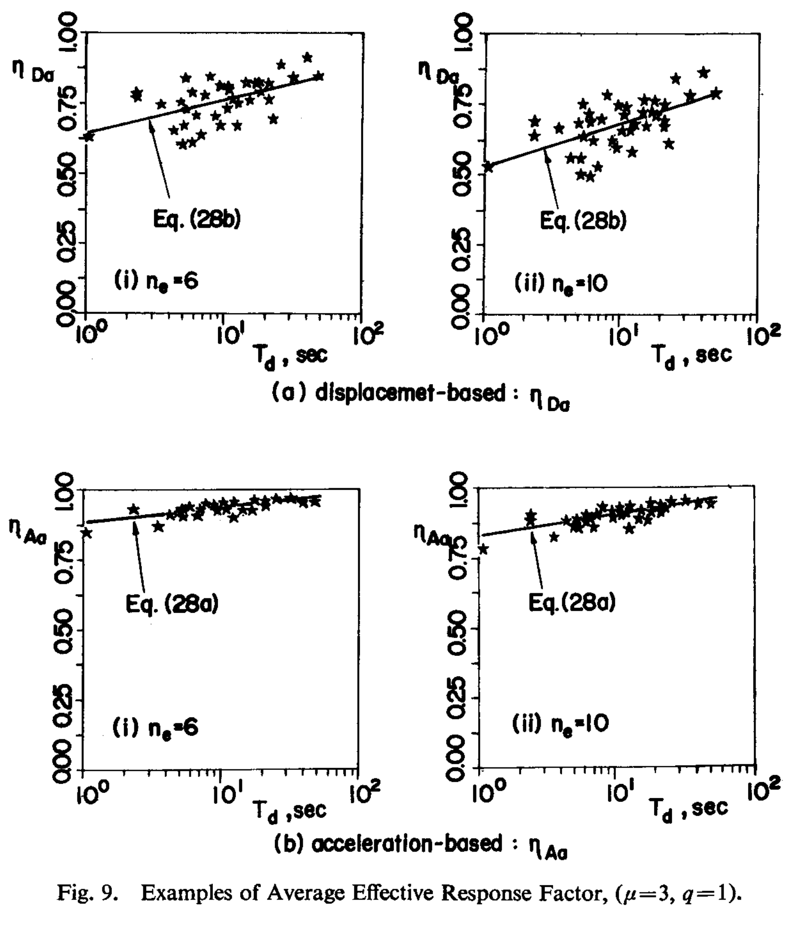

Fig. 9. Examples of Average Effective Response Factor, (μ=3, q=1).

In this way, the effects of the dynamic properties of the structures, T_0 and h, and those of the structural capacity, μ and n_e, are incorporated separately in γ and η_a, respectively.

The values of η_{Da} for $\mu=3$ and $n_e=10$ for the 40 strong motion data used so far are listed in Table A.1. The values of η_{Da} and η_{Aa} for $q=1$ are plotted against the ground motion duration T_d in Fig. 9. Observe that they increase with T_d, which clearly indicates that a longer ground motion duration will cause more severe damage to structures requiring a higher level of design seismic load.

The effective response based on A_{e1} is represented by

$$S_{e1}=\xi^{(s)}A_{e1} \qquad \text{...........(21)}$$

or

$$S_{e1}=\xi^{(s)}C_{e1}A_p \qquad \text{...........(21')}$$

The numerical results for C_{e1} and S_{e1} are shown by dashed lines in Figs. 7 and 8, respectively. When $n_e=1$, C_{e1} and S_{e1} coincide with C_e and S_e, respectively, as $\eta\equiv1$ in that case.

In the following part of this paper, discussion will be made on the basis of A_{e1} when EQA is concerned.

3.2 Average Equivalent Ground Acceleration (AEQA)

The equivalent ground acceleration, A_{e1}, is obviously a function of the natural period T_0. By averaging the peak response factor γ over a range of T_0, an average peak response factor is defined as

$$\gamma_a(T_d, h)=\int_{T_1}^{T_2}\xi(T_0)\,dT_0 \Big/ \int_{T_1}^{T_2}\xi^{(s)}(T_0)\,dT_0 \qquad \text{...........(22)}$$

where again $T_1=0.1$ sec and $T_2=5.0$ sec are used. For the acceleration response ratio ξ_A, the average peak response factor is represented by

$$\gamma_{Aa}(T_d, h)=\int_{T_1}^{T_2}\xi_A(T_0)\,dT_0 \Big/ \int_{T_1}^{T_2}\xi_A^{(s)}(T_0)\,dT_0 \qquad \text{...........(22')}$$

Likewise, for the displacement response,

$$\gamma_{Da}(T_d, h)=\int_{T_1}^{T_2}\xi_D(T_0)\,dT_0 \Big/ \int_{T_1}^{T_2}\xi_D^{(s)}(T_0)\,dT_0$$

$$=\int_{T_1}^{T_2}T_0^2\xi_A(T_0)\,dT_0 \Big/ \int_{T_1}^{T_2}T_0^2\xi_A^{(s)}(T_0)\,dT_0 \qquad \text{...........(22'')}$$

The values of γ_{Aa} for individual ground motions are indicated in Table A.1.

Using the concept of an average peak response factor, the average equivalent ground acceleration (AEQA) is defined as

$$A_{ea} = C_{ea} A_p \qquad \text{...........(23)}$$

in which C_{ea}=AEQA factor, which is represented by

$$C_{ea} = \gamma_a(T_d, h)\, \eta_a(T_d, \mu, n_e) \qquad \text{...........(24)}$$

AEQA is no longer a function of T_0, so that it does not directly represent an effective hazard level of ground motion for a particular structure. However, AEQA or the AEQA factor is a convenient measure for making a comparative observation of the over-all effect of the ground motion duration on the seismic design load. It will also be used in the statistical models for estimating EQA in the next chapter.

Fig. 10 shows the AEQA factor C_{ea} for the strong motion data in Table A.1, plotted against the ground motion duration. It may be observed that for all soil conditions, except for rock sites, the effect of the ground motion duration T_d on C_{ea} is remarkable. For rock sites (soil condition 1), there are only small effects of T_d on C_{ea}. This big difference between the rock sites and soil sites should be important in evaluating seismic design loads.

In Fig. 10, C_{ea} has been obtained as a product of γ_{Aa} and η_{Da}, the former from acceleration and the latter from displacement. What combination to employ, namely which of γ_{Aa} or γ_{Da} and which of η_{Aa} or η_{Da} is a matter of choice. The choice for η_{Aa} or η_{Da} should be determined from the design purposes as discussed in 2.3.

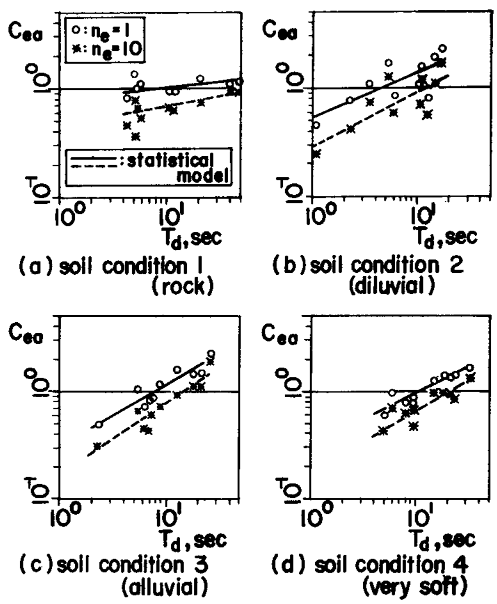

Fig. 10. AEQA Factors, (h=0.05, μ=3, n_e=10, q=1; $C_{ea}=A_{ea}/A_p$; $\gamma_{Aa}\eta_{Da}$ is used for A_{ea}).

The choice for γ_{Aa} or γ_{Da} is more arbitrary. When γ_{Aa} is used, the response characteristics in relatively short period ranges, say $T_0=0.1\sim2$ sec, will dominate the values of A_{ea}, whereas when γ_{Da} is used, a long period range, $T_0=1\sim5$ sec, will have large effects.

Besides, in the statistical models developed in the next chapter, the average peak response factor is calculated first, and on this basis the period-dependent peak response factor is determined. In such cases, whether to use γ_{Da} or γ_{Aa} does not affect the result.

4. Statistical Models for Estimation of EQA

4.1 General Features of the Models

Determination of EQA using the method developed so far requires certain computational efforts. In order to simplify this procedure, two statistical models, Model-I and Model-II, are developed on the basis of Japanese strong motion data. By using these statistical models, an estimation of EQA can be performed on a hand-calculator basis.

Model-I is intended to simplify the estimation of EQA referring to the 40 strong motion data in Table A.1, which have been used for numerical evaluation throughout the foregoing part of this paper. Therefore, it can be said to be a statistical model for EQA based on the standard acceleration response ratio $\xi_A^{(s)}$ in Fig. 3 or Table A.2. The model is developed so that EQA for any individual accelerogram can be estimated according to the classification of the soil condition for the recording station.

Model-II is developed for an estimation of EQA linked with ground motion attenuation and microzonation. For the given values of earthquake magnitude and epicentral distance, EQA can be estimated from this model. If, in addition, the blow count profile (N-value profile) is available from the standard penetration test at a specific site, a site-dependent EQA can be estimated. This enables one to perform microzonation of EQA.

4.2 Model-I, EQA for Specific Accelerograms

Suppose that one wishes to determine EQA for a specific accelerogram. Information for starting a calculation using this model is provided by the peak acceleration A_p, ground motion duration T_d, and the classification of soil condition[4] as specified in Fig. 3. One may also wish to determine AEQA and the effective response S_e. Model-I developed herein can be used for such purposes.

The statistical model is developed by finding the relation of the peak response factor γ and the average effective response factor η_a to the ground motion duration T_d.

(1) Peak Response Factor

From Fig. 4(b) and a graphical observation for other strong motion data, it is considered reasonable to assume a linear relation between log γ and log T_0 within the range $T_0=0.1\sim5.0$ sec which is of interest in this study. It would also be appropriate to assume that γ is equal to unity for $T_0\leqq0.1$ sec. Therefore, the statistical estimate for γ may be expressed as*

$$\hat{\gamma}=(10T_0)^{a_\gamma},\ 0.1\leqq T_0\leqq5.0 \text{ sec} \qquad \cdots\cdots\cdots\cdots(25)$$

The regression error in terms of the coefficient of variation (COV) δ_{U_γ} of $U_\gamma=\gamma/\hat{\gamma}$ ranges between $0.115\sim0.597$. The average value for δ_{U_γ} over the 40 accelerograms in Table A.1 is 0.301. These analyses have been made with the damping factor $h=0.05$. This value is also maintained in the following.

The regression coefficient a_γ is a key parameter to represent the effect of the ground motion duration on the peak response factor. To see this effect, a_γ has been plotted against T_d in Fig. 11. There is some tendency that a_γ increases with T_d. However, the scatter of data is too large to derive a statistical model out of this figure.

As an alternative way, a_γ has been plotted against the average peak response factor γ_{Aa} in Fig. 12. It may be observed that there is a high correlation between

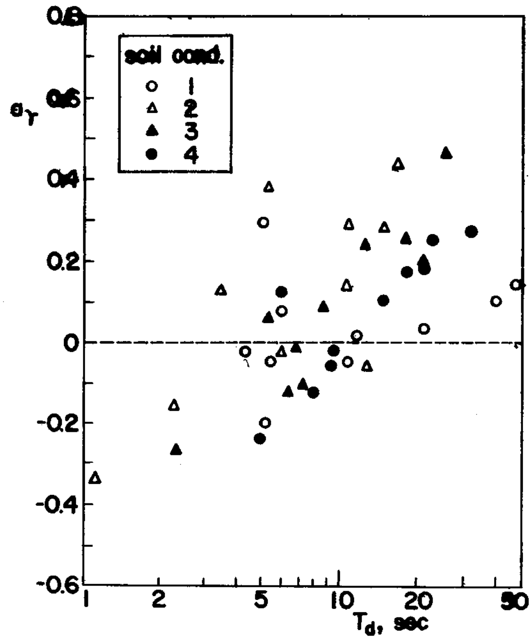

Fig. 11. Dependence of a_γ on T_d.

* All notations with ^ stand for statistical estimates.

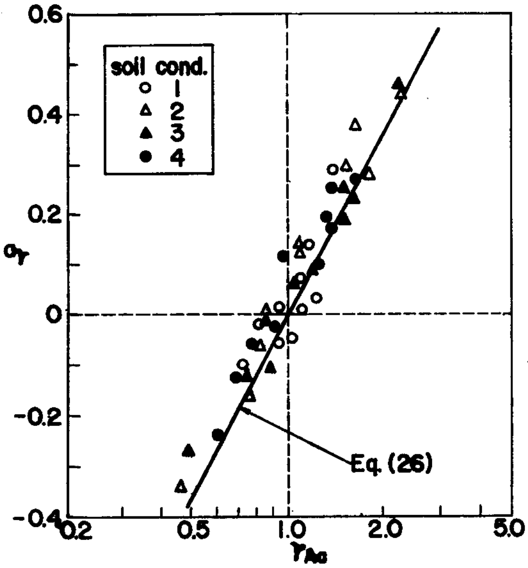

Fig. 12. Dependence of a_γ on γ_{Aa}.

a_γ and γ_{Aa}. Regression on this figure, taking into consideration that $\hat{a}_\gamma$ should vanish for $\gamma_{Aa}=1$, have lead to

$$\hat{a}_\gamma = 1.196 \log \gamma_{Aa} \qquad \text{............(26)}$$

The standard error for a_γ is $\sigma_{a_\gamma}=0.063$.

Dependence of γ_{Aa} on T_d is obvious from Fig. 10, observing the case with $n_e=1$, since in this case, C_{ea} coincides with γ_{Aa}. From regression for these results, we have

soil condition 1 (rock):

$$\hat{\gamma}_{Aa}=0.811\ T_d^{0.092} \qquad \text{............(27a)}$$

soil condition 2 (diluvial):

$$\hat{\gamma}_{Aa}=0.531\ T_d^{0.402} \qquad \text{............(27b)}$$

soil condition 3 (alluvial):

$$\hat{\gamma}_{Aa}=0.306\ T_d^{0.577} \qquad \text{............(27c)}$$

soil condition 4 (very soft deposit):

$$\hat{\gamma}_{Aa}=0.313\ T_d^{0.481} \qquad \text{............(27d)}$$

The regression error in terms of COV of $U_{\gamma_{Aa}}=\gamma_{Aa}/\hat{\gamma}_{Aa}$ is $\delta_{U\gamma_{Aa}}=0.188$, 0.348, 0.172, and 0.129, respectively.

(2) Effective Response Factor

From Fig. 9 and similar results for other values of μ and n_e, the linear regression has been performed between η_a and $\log T_d$. Specifically, the regression has been made for η_{Da} and η_{Aa}, based on the displacement response and the acceleration response, respectively, i.e.,

$$\hat{\eta}_{Da}=\begin{cases} a_{\eta D}+b_{\eta D}\log T_d, & 0<T_d\leq T_{cD} \\ 1 & , \quad T_d>T_{cD} \end{cases} \qquad \text{............(28a)}$$

and

$$\hat{\eta}_{Aa}=\begin{cases} a_{\eta A}+b_{\eta A}\log T_d, & 0\leq T_d\leq T_{cA} \\ 1 & , \quad T_d>T_{cA} \end{cases} \qquad \text{............(28b)}$$

in which $T_{cD}=10^{(1-a_{\eta D})/b_{\eta D}}$ and $T_{cA}=10^{(1-a_{\eta A})/b_{\eta A}}$.

The coefficients $a_{\eta D}$ and $b_{\eta D}$ for $q=1$ are plotted in Figs. 13 and 14, against the various values of μ and n_e. The values of $a_{\eta A}$ and $b_{\eta A}$, also for $q=1$, are shown in Figs. 15 and 16. The numerical values of these coefficients for various values of q are listed in Tables B.1 and B.2 in Appendix B, along with the standard error of regression by using Eq. (28). By these results, we may observe that the allowable number of deformation cycles n_e is a major parameter that affects both $a_{\eta A}$ and $b_{\eta A}$. The ductility factor μ has a minor effect insofar as there is a considerable difference between the value of $a_{\eta A}$ for $\mu=1$, the linear response, and that for $\mu\geqq 2$, the inelastic response. In contrast, $a_{\eta D}$ and $b_{\eta D}$, particularly the latter, are affected very much by the ductility factor μ as well as by n_e, which was not the case for $a_{\eta A}$ and $b_{\eta A}$. The large dependence of $b_{\eta D}$ on μ is caused by a plastic drift of the displacement response which makes the difference between the major response excursions larger than in the

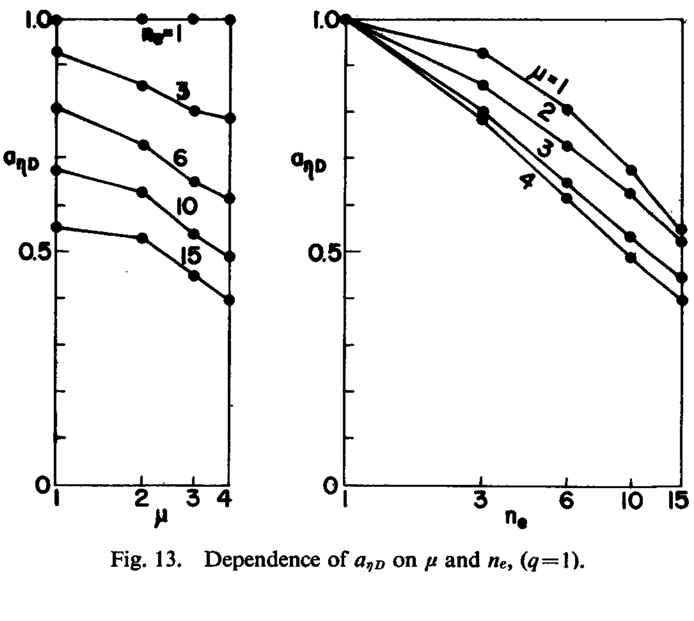

Fig. 13. Dependence of $a_{\eta D}$ on μ and n_e, $(q=1)$.

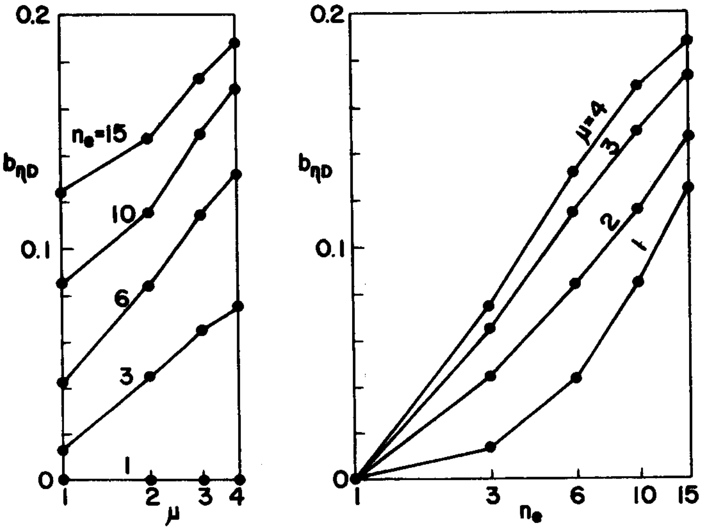

Fig. 14. Dependence of $b_{\eta D}$ on μ and n_e, (q=1).

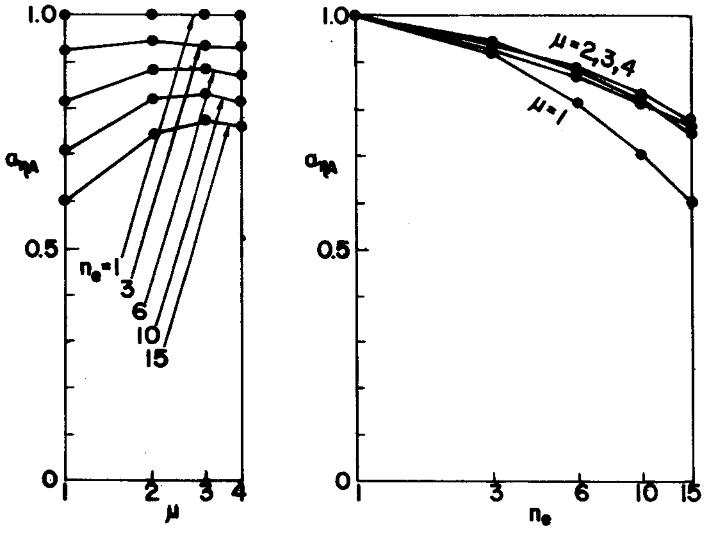

Fig. 15. Dependence of $a_{\eta A}$ on μ and n_e, (q=1).

case of the acceleration response.

(3) Procedure of Statistical Estimation of EQA and Effective Response Using Model-I

On the basis of the statistical models for the peak response factor and the effective response factor as developed above, EQA and/or AEQA, and also the

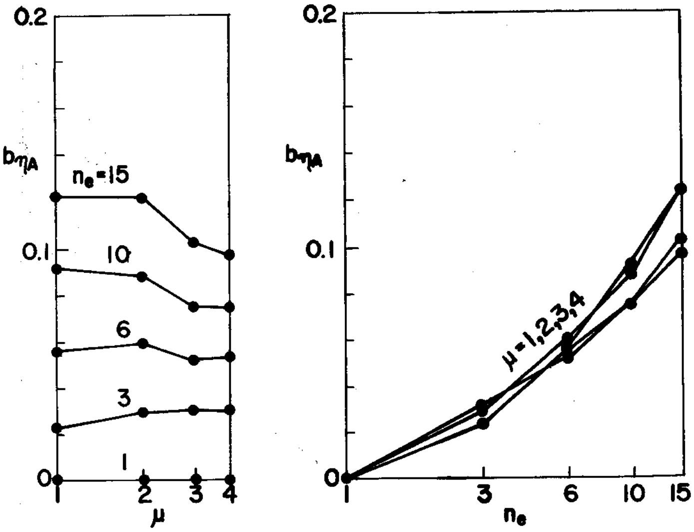

Fig. 16. Dependence of $b_{\eta A}$ on μ and n_e, $(q=1)$.

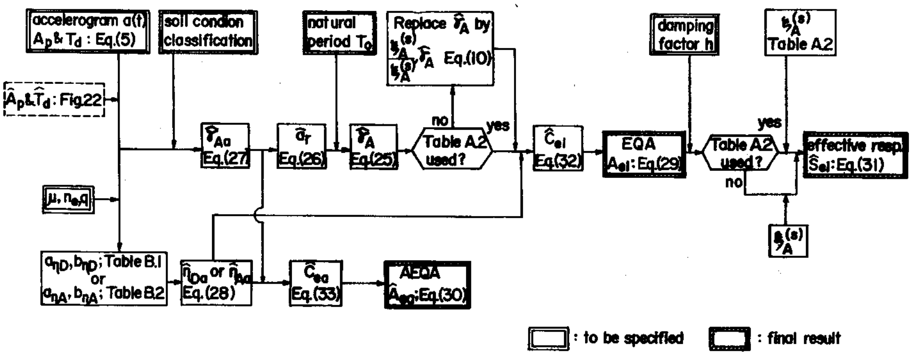

Fig. 17. Statistical Estimation EQA and Effective Response Using Model-I. (As to dashed arrows, see 4.3.)

effective response S_e can be estimated by following the flow chart in Fig. 17.

Information to be prescribed are (1) the PGA, A_p, and the ground motion duration T_d determined from a specific accelerogram $a(t)$, (2) soil condition classification for the strong motion station where $a(t)$ was recorded, (3) the maximum ductility factor μ, the allowable number of deformation cycles n_e and the constant q, and (4) the natural period T_0 and the damping factor h.

The final results of the estimation are as follows:

$$\hat{A}_{e1}=\hat{C}_{e1}A_p:\ \text{EQA} \qquad \cdots\cdots\cdots\cdots(29)$$

$$\hat{A}_{ea}=\hat{C}_{ea}A_p:\ \text{AEQA} \qquad \cdots\cdots\cdots\cdots(30)$$

$$\hat{S}_{e1}=\xi_A^{(s)}A_{e1}:\ \text{effective response} \qquad \text{...........(31)}$$

where

$$\hat{C}_{e1}=\hat{\gamma}\hat{\eta}_a:\ \text{EQA factor} \qquad \text{...........(32)}$$

$$\hat{C}_{ea}=\hat{\gamma}_a\hat{\eta}_a:\ \text{AEQA factor} \qquad \text{...........(33)}$$

When the acceleration response ratio, other than that in Table A.2, denoted by $\xi_A^{(s)\prime}$, is used, we have somewhat different results. The procedure for such cases is also shown in Fig. 17.

Fig. 18 shows the statistical estimates for A_{e1} for the two accelerograms dealt with in Figs. 4, 6, and 7. The actual values of A_{e1} are shown by dashed lines. Likewise, Fig. 19 shows the statistical estimates $\hat{S}_{e1}$ for the effective response S_{e1}. It may be observed that the statistical estimate $\hat{S}_{e1}$ follows quite well the general

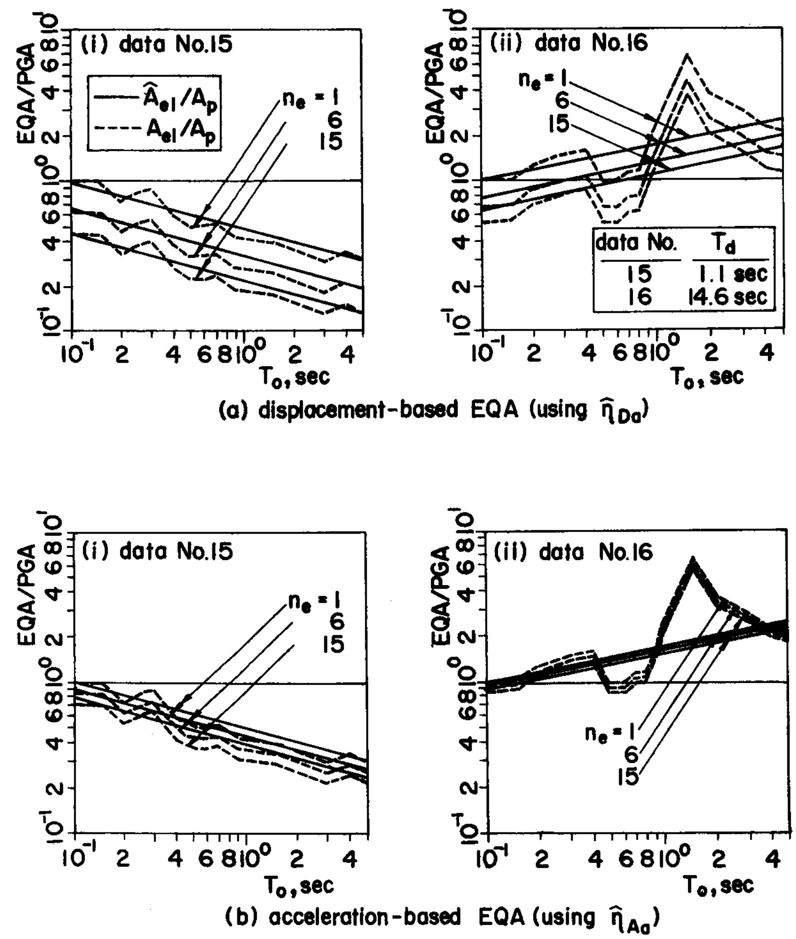

Fig. 18. Examples of Statistical Estimates for EQA, (soil condition 2; μ=3, q=1).

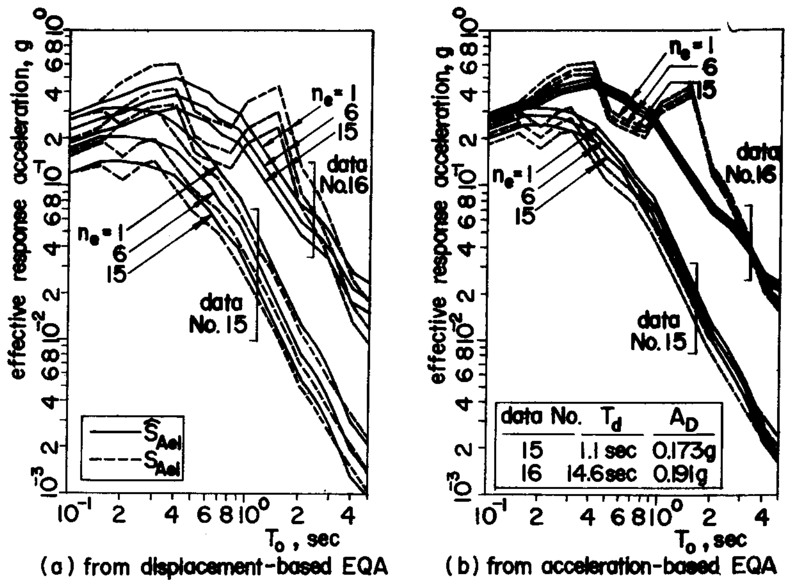

Fig. 19. Examples of Statistical Estimates for Effective Response. (See Fig. 8 for definition of S_{Ae1}; soil condition 2, $\mu=3$, $q=1$, $h=0.05$.)

trend of the actual value for S_{e1}. It should be noted that the statistical models for γ and η are based solely on their dependence on the ground motion duration. The frequency content has been considered in terms of $\xi^{(s)}$, whose values are used commonly within each soil classification. Therefore, the results in Fig. 19 demonstrate that the difference in the values of S_{e1} for the two strong motion data is primarily caused by the difference in their ground motion durations. An example from the statistical model is also seen by the dashed lines in Fig. 10, which shows the statistical estimates of the AEQA factor $\hat{C}_{ea}=\hat{A}_{ea}/A_p$.

It may be noted that the statistical model for the peak response factor γ and the average effective response factor η_a has been developed with a fixed value for the damping factor of $h=0.05$, which allows one to consider the effect of the damping factor only in $\xi_A^{(s)}$ in Eq. (31). For the different values of h, there will be different values for the model parameters for γ and η_a. Therefore, an exact way is to develop models for the various damping levels. However, as the primary effect of the damping factor is seen in the value of the standard response ratio $\xi_A^{(s)}$ and its effect on γ and η_a would be somewhat secondary. Therefore, for a first order approximation, it would be reasonable to regard $\hat{A}_{e1}$ and $\hat{A}_{ea}$ as common statistical estimates for EQA and AEQA. In this case, the effective response $\hat{S}_e$ for an arbitrary damping is estimated by using the values for $\xi_A^{(s)}$ corresponding to the damping of interest in Table A.2.

4.3 Model-II, Attenuation and Microzonation of EQA

It is desirable to be able to estimate EQA for given values of earthquake magnitude and source-to-site distance. This can be achieved through combining the EQA model with the attenuation rules of PGA and the magnitude-and-distance relation of the ground motion duration. It is also desired that the effect of the detailed site condition be incorporated in the procedure of estimating EQA, so that an appropriate site-dependent EQA can be estimated. This will enable one to perform microzonation of EQA. Model-II, developed in the following section, is constituted for these purposes.

(1) Attenuation and Microzonation of PGA

Given the earthquake magnitude M and the epicentral distance Δ, the attenuation rule for PGA for general Japanese diluvial and alluvial sites (soil conditions 2 and 3, respectively) has been developed by Kameda, Sugito, and Goto[5]. Denoting the magnitude-and-distance dependent PGA by A_0, its statistical estimate is represented by

$$\hat{A}_0=\begin{cases}349\times 10^{0.232M}/(\Delta+30)^{0.959}, & \Delta\geqq\Delta_0(M)\\ 330 & , \quad \Delta<\Delta_0(M)\end{cases} \qquad \cdots\cdots\cdots\cdots(34)$$

in which $\hat{A}_0$ is given in gals (cm/sec^2), Δ is given in km, and $\Delta_0(M)$ in km is given by

$$\Delta_0(M)=\begin{cases}1.06\times 10^{0.242M}-30, & M<6.0\\ 0 & , \quad M\leqq 6.0\end{cases} \qquad \cdots\cdots\cdots\cdots(35)$$

The attenuation uncertainty in terms of the COV of $U_a=A_0/\hat{A}_0$ is $\delta_{U_a}=0.593$.

A technique of microzonation for peak ground motion has also been developed[5], in which the site-dependent estimate for PGA is obtained from

$$\hat{A}_p=C_a(S_n)\hat{A}_0 \qquad \cdots\cdots\cdots\cdots\cdots(36)$$

in which $C_a(S_n)$=correction factor to incorporate site effects, and S_n=site parameter.

The correction factor $C_a(S_n)$ is given by

$$C_a(S_n)=\begin{cases}2.09^{S_n}, & S_n<0.6\\ 1.56 & , \quad 0.6\leqq S_n<1\end{cases} \qquad \cdots\cdots\cdots\cdots(37)$$

in which S_n is determined from

$$S_n=0.264\int_0^{d_s}\exp\{-0.04N(x)\}\exp(-0.14x)\,dx-0.883 \qquad \cdots\cdots\cdots\cdots(38)$$

where $N(x)$=blow-count at a depth of x (in meters) from a standard penetration test, and d_s=maximum depth of the blow-count profile.

The site parameter S_n is a practical measure of the softness of the ground. In an extreme case where $N(x)\equiv 0$, we have $S_n=1$. For an intermediate case with $N(x)\equiv 19$, we have $S_n=0$, whereas for $N(x)\equiv 50$, $S_n=-0.628$. For sites with soil condition classifications 2 (diluvial), 3 (alluvial) and 4 (very soft deposit), used in the foregoing discussion, S_n varies roughly in ranges, respectively, of $-0.4\sim 0.3$, $0\sim 0.6$ and $0.5\sim 0.8$.

As standard penetration test data are commonly available from civil engineering construction sites, the method of microzonation described above is practical and useful. The attenuation uncertainty of the site-dependent PGA, A_p, is measured by the COV $\delta_{U_{a1}}$ of $U_{a1}=A_p/\hat{A}_p$. It has proved $\delta_{U_{a1}}=0.481$, which is a considerable reduction compared to $\delta_{U_a}=0.593$.

It should be pointed out that these results for attenuation and microzonation are applicable for soil sites where standard penetration tests have an engineering significance. Indeed, the dataset of strong motion records on which the attenuation equation, Eq. (34), is based excludes the strong motion data from rock sites.

(2) Dependence of Ground Motion Duration on Magnitude and Distance

The ground motion duration T_d, defined by Eq. (8), has been determined for each of 91 major accelerograms recorded on Japanese diluvial and alluvial grounds. On this basis, regression of T_d on M and Δ has been performed, and the following result has been obtained.

$$\hat{T}_d=\begin{cases}0.0325\times 10^{0.168M}(\Delta+30)^{0.572}, & \Delta\geqq\Delta_0(M) \quad \cdots\cdots\cdots\cdots(39)\\ 0.0336\times 10^{0.306M}, & \Delta<\Delta_0(M) \quad \cdots\cdots\cdots\cdots(39')\end{cases}$$

in which $\Delta_0(M)$ has been defined by Eq. (35). The COV of $U_{Td}=T_d/\hat{T}_d$ proved to be $\delta_{U_{Td}}=0.505$.

The statistical estimate $\hat{T}_d$ (in sec) in Eq. (39) demonstrates that the ground motion duration increases both with magnitude and distance. This obviously agrees with the established seismological knowledge.

Eq. (39′) has been derived by analogy to Eq. (34) in order to take care of the characteristics for epicentral regions, assuming that the ground motion duration within the epicentral region depends basically only on the magnitude. This result may be compared with a result of seismological theory, which follows.

It has been shown by Geller[3] that the relation between the source area S (in km^2) and the 20 second surface wave magnitude M_s is given by

$$\log S=\frac{2}{3}M_s-2.28, \qquad M_s<6.76 \qquad \cdots\cdots\cdots\cdots(40)$$

and

$$\log S=M_s-4.53, \qquad 6.76\leqq M_s<8.12 \qquad \cdots\cdots\cdots\cdots(40')$$

Assume roughly that the earthquake source is a rectanglar fault plane whose edges have lengths with a 1:2 ratio, and that a fault rupture propagates in the longitudinal direction either in a uni-lateral or symmetric bi-lateral manner at a speed of 2.5~3.5 km/sec. Then, the duration T_d' of the fault rupture is represented by

$$T_d'=0.021\sim0.041\times10^{M_s/3}, \quad M_s<6.76 \qquad \text{.......….(41)}$$

and

$$T_d'=0.0016\sim0.0031\times10^{M_s/2}, \quad 6.76\leqq M_s<8.12 \qquad \text{...........(41')}$$

It would be reasonable to assume that there is a high correlation between the ground motion duration T_d, as defined by Eq. (5), and T_d' given by Eqs. (41) and (41′) within the epicentral region. From this, and taking into account that the magnitude values used for the strong motion data to develop Eq. (39′) are approximately equal to the surface wave magnitude, Eqs. (41) and (41′) can be compared with Eq. (39′) as to the dependence on the magnitude.

In Fig. 20, the numerical values for $\hat{T}_d$ calculated from Eq. (39′) and those for T_d' from Eqs. (41) and (41′) are compared. It may be noted that the dependence of $\hat{T}_d$ on M agrees quite well with that of T_d' for a range of $M<6.76$. This will support the use of Eq. (39′) to estimate the ground motion duration for epicentral regions. For $M>6.76$, the value for T_d' depends more strongly on M than does the

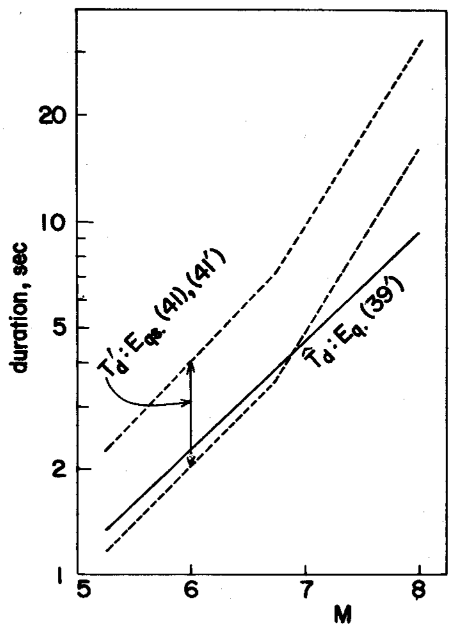

Fig. 20. Comparison of Ground Motion Duration in Epicentral Regions from Various Definitions.

value for $\hat{T}_d$. However, this does not mean that Eq. (39′) should be modified according to Eq. (41′). This is a region of the earthquake magnitude where the size of the source area is very large. Therefore, the strong part of the ground motion at a site within the epicentral region is affected only by the part of the source relatively close to the site. As the ground motion duration $\hat{T}_d$ from Eq. (39′) represents the strong part of the site ground motion, it is reasonable that it depends on M more weakly than T_d' which stands for the total duration of the fault rupture propagation. For these reasons, it has been judged appropriate to use Eq. (39′) for an estimation of the near-source ground motion duration.

(3) Model Parameters for Estimation of EQA Using Model-II

As proposed above, site conditions are evaluated in terms of a continuous site parameter S_n in the statistical estimation of EQA using Model-II. This will enable one to incorporate site conditions on a more rational basis compared to a qualitative classification of soil conditions which has so far been used in this study. In the following, statistical model parameters are reevaluated for a consistent use of the site parameter S_n.

It has been stated above that the values of S_n for soil conditions 2 (diluvial) and 3 (alluvial) vary in overlapping ranges of $S_n=-0.4\sim0.3$, and $S_n=0\sim0.6$, respectively. Moreover, the standard acceleration response ratios $\xi_A^{(s)}$ differ little between soil conditions 2 and 3, as can be seen in Fig. 3. Therefore, it is difficult to distinguish the effect of one of these two categories of site classification from the other, while the use of the correction factor $C_a(S_n)$ enables one to make a more detailed evaluation of the site condition.

For these reasons and considenring that $S_n=-0.628$ for $N(x)\equiv50$, Model-II will employ site classifications, determined in the following manner.

$$\text{Normal site condition: } -0.63<S_n<0.6 \qquad \text{(42a)}$$

$$\text{Very soft site condition: } 0.6\leqq S_n<1 \qquad \text{(42b)}$$

Rock sites and very firm grounds outside the above range of S_n are beyond the scope of Model-II.

The model parameters which need to be reevaluated for these new site classifications are the standard acceleration response ratio $\xi_A^{(s)}$ and the estimate for the average peak response factor $\hat{\gamma}_{Aa}$. For other parameters, Eqs. (25), (26), and (28), and Tables B.1 and B.2 apply.

For the standard acceleration response ratio, the harmonic mean of $\xi_A^{(s)}$ for soil conditions 2 and 3, Table A.2, will be employed for $\xi_A^{(s)}$ for the normal site condition. For a very soft condition, the values for the soil condition 4 in Table A.2 are used, since for the soil condition 4 the values of S_n fall in the range specified in Eq. (42b).

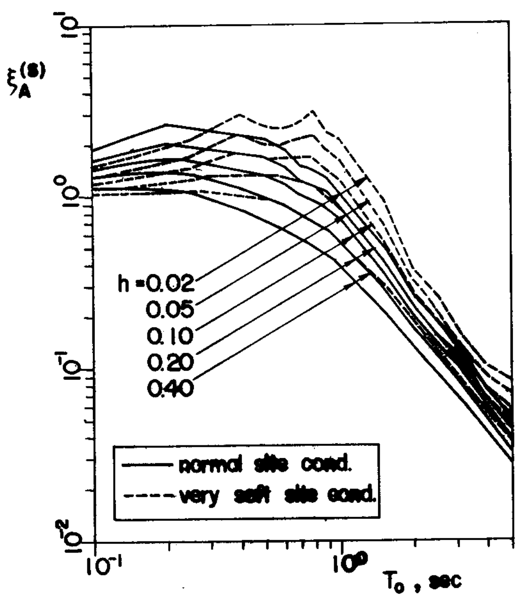

Fig. 21. Standard Acceleration Response Ratio. (Model-II: See Table C.1 also.)

The values of $\xi_A^{(s)}$ for the above new site classifications are shown in Fig. 21, and also tabulated in Table C.1.

The statistical estimate for the average peak response factor $\hat{\gamma}_{Aa}$ for the normal site condition is determined from the geometric mean for those soil conditions 2 and 3, Eqs. (27b) and (27c). For the very soft site condition, the result for the soil condition 4, Eq. (27d) is used without any changes. These yield

Normal site condition $(-0.6<S_n<0.6)$:

$$\hat{\gamma}_a=0.403\ T_d^{0.490} \qquad \text{............(43a)}$$

Very soft site condition $(0.6\leq S_n<1)$:

$$\hat{\gamma}_a=0.313\ T_d^{0.481} \qquad \text{............(43b)}$$

The model errors in terms of the COV of $U_{\gamma_{Aa}}=\gamma_{Aa}/\hat{\gamma}_{Aa}$ are $\delta_{U_{\gamma_{Aa}}}=0.274$ and 0.129, respectively.

(4) Procedure of Statistical Estimation of EQA and Effective Response Using Model-II

Fig. 22 is a flow chart showing the procedure for estimating EQA and the effective response. The biggest difference of Fig. 22 from Fig. 17 is that the earthquake magnitude M and the epicentral distance Δ are to be specified instead of A_p and T_d for an actual accelerogram.

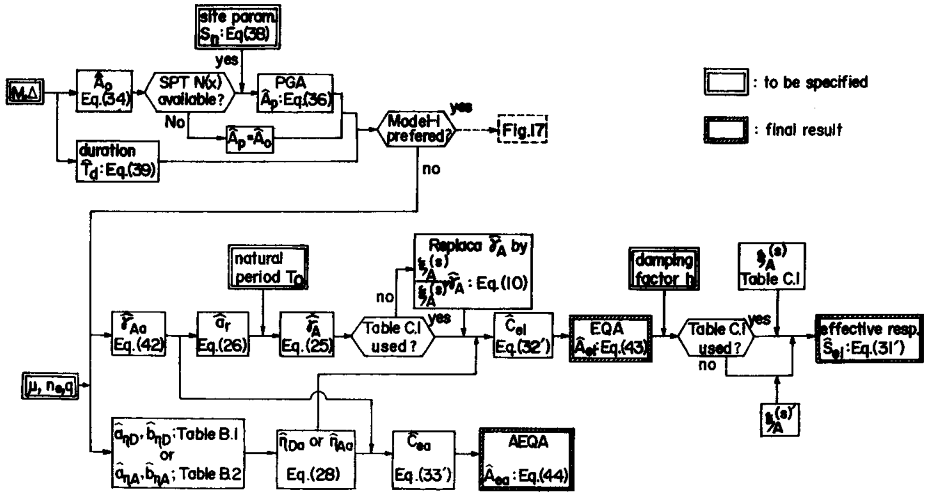

Fig. 22. Statistical Estimation of EQA and Effective Response Using Model-II.

If one still wishes to use Table A.2 and go basically through Model-I, it suffices to follow the dashed arrow and transfer to Fig. 17.

The final results of the estimation are represented in the same way as Eqs. (29)~(33), except that $\hat{A}_p$ is used instead of A_p, which follows:

$$\hat{A}_{e1}=\hat{C}_{e1}\hat{A}_p\text{: EQA} \qquad \text{............(44)}$$

$$\hat{A}_{ea}=\hat{C}_{ea}\hat{A}_p\text{: AEQA} \qquad \text{............(45)}$$

$$\hat{S}_e=\xi_A^{(s)}\hat{A}_e\text{: effective response} \qquad \text{............(31')}$$

where

$$\hat{C}_{e1}=\hat{\gamma}\hat{\eta}_a\text{: EQA factor} \qquad \text{............(32')}$$

$$\hat{C}_{ea}=\hat{\gamma}_a\hat{\eta}_a\text{: AEQA factor} \qquad \text{............(33')}$$

(5) Verification of Model-II in Terms of Spectral Acceleration

It was suggested in regard to Fig. 19 (4.2 (3)) that the dependence of the effective response on the natural period is dominated primarily by the ground motion duration. This aspect of the problem is examined in further detail by comparing some numerical results from Model-II developed herein with the results of direct regression on magnitude and distance. In the following, the spectral acceleration S_A is dealt with. S_A is regarded as a special case of the effective response S_{e1}, with $\mu=1$ and $n_e=1$, which means $\eta_a=1$.

The spectral acceleration S_A has been analyzed statistically for the 91 strong motion data used to develop Eqs. (34) and (39). Statistical estimates for S_A from direct regression on the magnitude M and the epicentral distance Δ have been obtained. Consideration of the effect of epicentral regions was made in a manner similar to Eq. (34). The numerical results are shown by the dashed lines in Fig. 23 for some typical combinations of M and Δ.

The spectral acceleration obtained from the EQA technique by using Model-II is shown by the solid lines in Fig. 23 for the same combinations of M and Δ. Observe that the agreement between the results of Model-II and those from direct regression is satisfactory.

There is a widely recognized bahavior of linear response spectra that a change in the magnitude M, compare (i) and (iii) in Fig. 23, has a larger influence on the values of S_A in a long period range than on those in a short period range, whereas a change in the epicentral distance, compare (i) and (ii), has a larger influence on the values of S_A in a short period range than on those in a long period range. This behavior has often been explained as being a result of a quick attenuation of S_A with distance for short periods, and a slow attenuation for long periods. Such an effect should certainly exist, and indeed this is observed if the attenuation characteristics of the peak ground acceleration and those of the peak ground velocity[5] are compared.

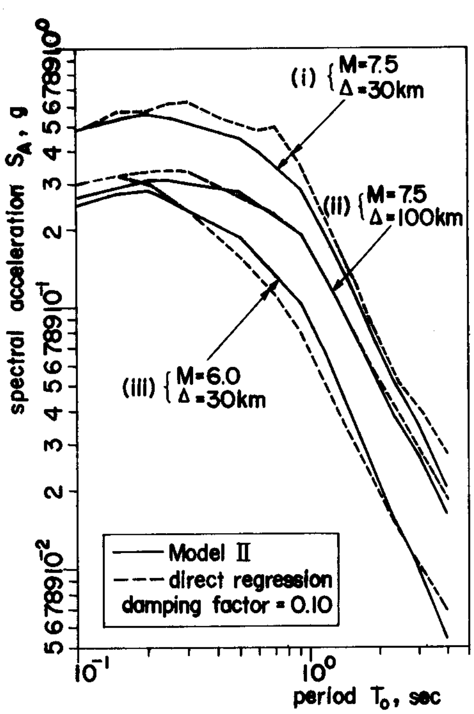

Fig. 23. Response Spectra from EQA Technique and Those from Direct Regression (I).

It should be pointed out, however, that similar results of Model-II in Fig. 23 can be explained only from the effect of the ground motion duration T_d. This is because Model-II uses the same values of $\xi_A^{(s)}$ for all combinations of M and Δ. Hence, the only parameter affecting the difference in the spectral shape for varying M and Δ is the peak response factor γ which is estimated from T_d as a function of M and Δ. From these results, it would be reasonable to say that the difference in the spectral shapes for various combinations of M and Δ can be primarily explained as an effect of the ground motion duration varying with M and Δ.

Fig. 24 compares the results of Model-II with those from direct regression for a fixed combination of M and Δ, but for various values of the damping factor. Except for a very short period range, the results from the two procedures agree fairly well with each other. As pointed out earlier, all numerical calculations for determining the peak response factor γ and the effective response factor η_a were done by using a fixed damping factor of h=0.05. The different damping values are taken into account in the final estimation of the effective response by using Eqs. (31) and (31′) in terms of the damping-dependent values of $\xi_A^{(s)}$. It may, therefore, be concluded from these results that the values of γ and η_a determined from h =0.05, can be used

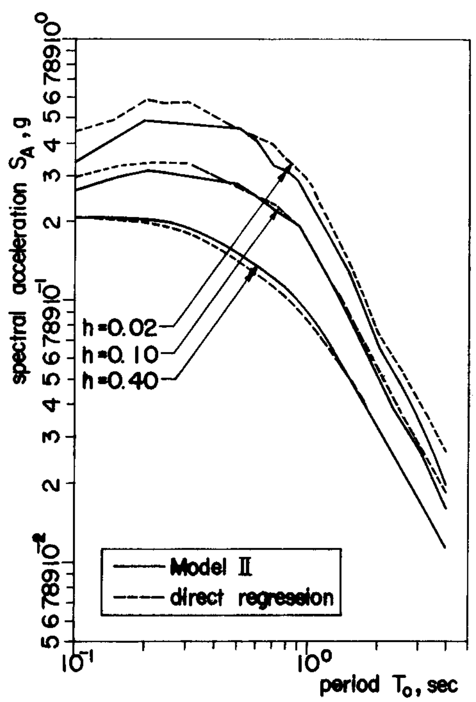

Fig. 24. Response Spectra from EQA Technique and Those from Direct Regression (II), (M=7.5, Δ=100 km).

for other values of h as a first order approximation. Hence, it suffices to consider the damping in determining the effective response by multiplying EQA by the damping-dependent standard response ratio $\xi_A^{(s)}$.

5. Future Extension and Application

In this paper, the concept of the equivalent ground acceleration (EQA) has been proposed, and its formulation has been presented. Statistical models have been developed for an estimation of EQA on a hand calculator basis. In each step, discussion has been made regarding the significance of EQA in earthquake engineering, particularly in connection with the effect of the ground motion duration in the evaluation of design seismic loads. Through these means, it has been demonstrated that EQA can be used as an appropriate earthquake hazard parameter that incorporates the structural effects not only of the intensity and the frequency content of the ground motion, but also of its duration. On this basis, research subjects are identified in this chapter that are considered useful for further verification of the EQA concept, and its application for determining design seismic loads. Studies along this line are now underway, and their results will be presented elsewhere.

Two topics are being studied for the verification of the significance of EQA. One is to estimate the regional distribution of EQA for known earthquakes, and compare them with existing structural damage data. The 1923 Great Kanto Earthquake, the 1978 Miyagiken-oki Earthquake, etc. are being analyzed. Similar analysis can be made for major earthquakes which occurred outside Japan, like the Montenegro, Yugoslavia Earthquake of 1979.

The second topic is to clarify the relation between EQA and other peak ground motion parameters. Some preliminary results demonstrate that depending on short- intermediate- and long period ranges, EQA is highly correlated with the peak ground acceleration, velocity and displacement, respectively. This is consistent with widely recognized characteristics of damageability of earthquake ground motions, which would suggest the usefulness of the EQA concept. The theoretical and statistical aspects of such relations are being studied.

For the extension of the EQA concept, introducing other realistic damage models for determining the effective response factor η is an important subject. Extensive discussion and evaluation of experimental results are required for this purpose.

As EQA will be useful for a seismic design of ordinary structures with standard sizes, it will not be meaningful to extend the evaluation of EQA to structural models with a large number of degrees of freedom. However, it will be useful to extend it to structures with a few degrees of freedom, since such structures commonly exist, and are designed using quasi-static seismic design procedures.

For a design seismic load evaluation, seismic risk analysis and microzonation in terms of EQA are essential. In this way, a rational basis will be laid for determining the design seismic load on a quantitative basis, or in other words, with less judgmental evaluation than it has so far been.

6. Conclusions

The results of this study may be summarized by the following conclusions.

(1) The consept of the "equivalent ground acceleration", (EQA), has been proposed for use as an appropriate earthquake hazard parameter that incorporates the structural effects of the ground motion duration as well as those of the ground motion intensity and frequency content.

(2) Formulation of EQA has been presented by considering the transient effect of the structural response (in terms of the peak response factor γ) and the contribution of repeated response pulses to successive structural failures (in terms of the effective response factor η or η_a). From this, EQA is represented by simply multiplying the EQA factor with the peak ground acceleration: Eqs. (3) and (7), or Eqs. (19) and (20).

(3) Once an EQA value is determined, the structural response level effective for a seismic design (effective response) can be determined as a product of EQA and the standard response ratio: Eq. (17) or Eq. (21).

(4) For discussion of the over-all effects of the ground motion duration, the "average equivalent ground acceleration", (AEQA), has been introduced: Eqs. (23) and (24).

(5) Two statistical models have been developed for an estimation of EQA, AEQA and the effective response on a hand calculator basis. Model-I is to perform such an estimation for any individual accelerogram whose peak ground acceleration and ground motion duration, Eq. (5), are known. Model-II includes attenuation and microzonation models, and can be used for estimating EQA and other related parameters for specified values of earthquake magnitude, epicentral distance, and a specific site condition given in terms of an STP blow counts (N-value) profile.

(6) It has been demonstrated from numerical results that the effect of the ground motion duration on the peak response factor is dominant in long period ranges, whereas that on the effective response factor is practically uniform over a period range of $T_0=0.1\sim5$ sec.

(7) The over-all effect of the ground motion duration on the equivalent ground acceleration, evaluated in terms of AEQA, proved to be so large that the variation of the ground motion duration in a range of $T_d=2\sim20$ sec can cause the AEQA factor to vary from $0.2\sim2.0$.

(8) For combinations of the earthquake magnitude M and the epicentral distance Δ of practical interest, the response spectra estimated by using the EQA technique have proved to agree fairly well with those from a direct statistical regression. This demonstrates within the numerical results dealt with herein that the variation of the spectral shape for various combinations of M and Δ can be attributed primarily to the variation of the ground motion duration.

(9) Further research subjects for future extension and application have been identified.

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Appendix A.

Table A.1 List of 40 strong motion data

(a) soil condition 1 (rock)

seq. No.	ID-No., comp., date recording station name of earthquake	M	Δ (km)	A_p (g)	V_p (cm/s)	T_d (sec)	γ_a ($h=$ 0.05)	η_{Da} ($\mu=3$, $n_e=10$)
1	S-236 N-S 1968- 5-16 Miyako 1968 Tokachi-Oki	7.9	189.0	0.170	6.8	48.9	1.158	0.793
2	S-236 E-W 1968- 5-16 Miyako 1968 Tokachi-Oki	7.9	189.0	0.164	5.7	40.1	1.113	0.868
3	S-271 N-S 1968- 5-16 Miyako 1968 Tokachi-Oki, (aftershock)	7.4	213.0	0.150	4.7	10.6	0.939	0.715
4	S-271 E-W 1968- 5-16 Miyako 1968 Tokachi-Oki, (aftershock)	7.4	213.0	0.124	4.1	11.6	0.940	0.660
5	S-537 N-S 1970- 4- 1 Miyako Iwate, Coast	—	—	0.174	5.6	5.1	0.715	0.504
6	S-1204 East 1978- 6-12 Miyako 1978 Miyagiken-Oki	7.4	166.7	0.164	6.3	21.1	1.225	0.612
7	S-1204 South 1978- 6-12 Miyako 1978 Miyagiken-Oki	7.4	166.7	0.228	7.3	5.4	0.989	0.670
8	D-723 LG 1978- 2-20 Kaihoku Bridge Miyagi, Off-shore	6.7	85.8	0.099	6.1	5.9	1.081	0.497
9	D-743 LG 1978- 6-12 Kaihoku Bridge 1978 Miyagiken-Oki	7.4	82.7	0.322	16.6	4.3	0.814	0.560
10	D-745 TR 1978- 6-12 Kaihoku Bridge 1978 Miyagiken-Oki	7.4	82.7	0.402	36.5	5.1	1.369	0.561

(b) soil condition 2 (diluvial)

seq. No.	ID-No., comp., date recording station name of earthquake	M	Δ (km)	A_p (g)	V_p (cm/s)	T_d (sec)	γ_a ($h=$ 0.05)	η_{Da} ($\mu=3$, $n_e=10$)
11	S-234 E-W 1968- 5-16 Muroran 1968 Tokachi-Oki	7.9	293.0	0.250	16.8	10.6	1.068	0.656
12	S-241 N-S 1968- 5-16 Muroran 1968 Tokachi-Oki, (aftershock)	7.4	196.0	0.120	6.2	12.7	0.808	0.684
13	S-252 N-S 1968- 5-16 Hachinohe 1968 Tokachi-Oki	7.9	235.0	0.269	35.7	11.0	1.528	0.744
14	S-252 E-W 1968- 5-16 Hachinohe 1968 Tokachi-Oki	7.9	235.0	0.207	35.0	16.9	2.279	0.726
15	S-647 E-W 1971-10-11 Kashima Chiba, North	5.2	13.4	0.174	7.1	1.1	0.462	0.532
16	S-733 N-S 1973- 6-17 Kushiro Nemuro Penn., Off-shore	7.4	141.0	0.191	26.7	14.6	1.805	0.602
17	S-813 E-W 1974- 3- 3 Kashima-ji Chiba, East	6.1	43.3	0.112	10.7	3.5	1.084	0.668
18	D-011 N-S 1962- 4-23 Kushiro Kisho-Dai Kushiro, Off-shore	7.0	94.0	0.244	13.6	6.0	0.847	0.693
19	D-613 E-W 1974-11-16 P.W.R.I. Kashima (906-GR-22) Chiba, Off-shore	6.1	55.0	0.174	11.6	2.3	0.767	0.551
20	S-1066 South 1978- 1-14 Shimizu-Miho 1978 Izu-Oshima-Kinkai	7.0	71.4	0.094	11.4	5.3	1.651	0.752

Table A.1 (continued)
(c) soil condition 3 (alluvial)

seq. No.	ID-No., comp., date recording station name of earthquake	M	Δ (km)	A_p (g)	V_p (cm/s)	T_d (sec)	γ_a (h=0.05)	η_{Da} (μ=3, n_e=10)
21	S-213 E-W 1968- 4-1 Hososhima 1968 Hyuganada	7.5	110.0	0.288	27.5	5.3	1.048	0.636
22	S-235 E-W 1968- 5-16 Aomori 1968 Tokachi-Oki	7.9	247.0	0.196	31.6	25.6	2.256	0.842
23	S-264 E-W 1968- 5-16 Aomori 1968 Tokachi-Oki, (aftershock)	7.4	193.0	0.101	9.2	17.8	1.476	0.767
24	S-340 E-W 1968- 7- 1 Shinagawa Saitama, Center	6.1	49.7	0.134	9.0	6.3	0.736	0.624
25	S-585 E-W 1971- 1- 5 Kinuura Aichi, Off-shore	6.1	54.2	0.091	6.7	7.2	0.869	0.701
26	D-408 Ha. 1970-10-16 Yuuhei Bridge (301-GR-7) Akita, South-East	6.2	24.1	0.250	9.2	2.3	0.485	0.641
27	S-1201 West 1978- 6-12 Shiogama-Kojyo 1978 Miyagiken-Oki	7.4	100.0	0.287	53.1	12.5	1.606	0.582
28	S-1201 North 1978- 6-12 Shiogama-Kojyo 1978 Miyagiken-Oki	7.4	100.0	0.323	29.0	8.7	1.169	0.621
29	S-1210 E41S 1978- 6-12 Ofunado-Bochi 1978 Miyagiken-Oki	7.4	103.0	0.215	14.5	6.8	0.828	0.529
30	D-757 LG 1978- 6-12 Taira Bridge 1978 Miyagiken-Oki	7.4	165.4	0.096	12.9	21.1	1.499	0.753

(d) soil condition 4 (very soft deposit)

seq. No.	ID-No., comp., date recording station name of earthquake	M	Δ (km)	A_p (g)	V_p (cm/s)	T_d (sec)	γ_a (h=0.05)	η_{Da} (μ=3, n_e=10)
31	S-74 E-W 1965- 4-20 Shimizu Kojyo Shizuoka, Off-shore	6.1	135.0	0.148	9.5	7.9	0.783	0.785
32	S-211 N-S 1968- 4- 1 Kochi 1968 Hyuganada	7.5	167.0	0.073	10.6	18.2	1.359	0.718
33	S-211 E-W 1968- 4- 1 Kochi 1968 Hyuganada	7.5	167.0	0.106	15.9	14.7	1.239	0.769
34	S-577 N-S 1971- 1- 5 Yokkaichi-Chitose Aichi, Off-shore	6.1	74.2	0.093	6.9	9.4	0.780	0.599
35	S-577 E-W 1971- 1- 5 Yokkaichi-Chitose Aichi, Off-shore	6.1	74.2	0.106	6.8	4.9	0.607	0.686
36	D-233 N-S 1968- 5-16 Shinishikari Bridge (1303-GR-1) 1968 Tokachi-Oki	7.9	324.0	0.190	24.1	32.1	1.655	0.785
37	D-235 E-W 1968- 5-16 Shinishikari Bridge (1303-GR-1) 1968 Tokachi-Oki	7.9	324.0	0.190	25.3	22.7	1.376	0.612
38	D-245 N-S 1968- 5-16 Shinishikari Bridge (1303-GR-2) 1968 Tokachi-Oki, (aftershock)	7.4	224.0	0.102	13.8	21.1	1.344	0.694
39	D-247 E-W 1968- 5-16 Shinishikari Bridge (1303-GR-2) 1968 Tokachi-Oki, (aftershock)	7.4	224.0	0.196	19.3	9.5	0.878	0.750
40	S-1063 E06N 1978- 1-14 Shimizu-Kojyo 1978 Izu-Oshima-Kinkai	7.0	76.0	0.103	12.4	5.9	0.964	0.718

Table A.2 Standard acceleration response ratio (Model-I)

(a) soil condition 1 (rock)

T_o (sec)	$\xi_A^{(s)}$				
	h=0.02	h=0.50	h=0.10	h=0.20	h=0.40
0.1	3.231	2.403	1.915	1.532	1.236
0.15	5.240	3.799	2.758	1.844	1.179
0.2	4.201	3.098	2.299	1.586	0.975
0.25	2.759	2.062	1.606	1.155	0.758
0.3	2.131	1.599	1.223	0.882	0.584
0.4	1.255	1.001	0.765	0.565	0.392
0.5	0.743	0.593	0.482	0.379	0.279
0.6	0.499	0.408	0.337	0.264	0.202
0.7	0.377	0.295	0.244	0.197	0.151
0.8	0.291	0.240	0.204	0.164	0.124
0.9	0.288	0.223	0.174	0.136	0.105
1.0	0.253	0.190	0.155	0.123	0.092
1.5	0.132	0.110	0.089	0.071	0.052
2.0	0.089	0.074	0.062	0.050	0.038
2.5	0.065	0.053	0.048	0.040	0.030
3.0	0.061	0.052	0.044	0.034	0.025
4.0	0.050	0.041	0.034	0.025	0.018
5.0	0.037	0.031	0.025	0.019	0.014

(b) soil condition 2 (diluvial)

T_o (sec)	$\xi_A^{(s)}$				
	h=0.02	h=0.05	h=0.10	h=0.20	h=0.40
0.1	1.733	1.501	1.379	1.274	1.133
0.15	2.126	1.792	1.560	1.368	1.132
0.2	2.365	1.895	1.595	1.368	1.092
0.25	2.496	2.027	1.707	1.385	1.059
0.3	2.614	2.068	1.661	1.319	1.003
0.4	2.485	2.004	1.603	1.240	0.876
0.5	2.164	1.672	1.332	0.999	0.739
0.6	1.875	1.452	1.132	0.870	0.628
0.7	1.479	1.179	0.963	0.749	0.546
0.8	1.309	1.049	0.863	0.669	0.492
0.9	1.093	0.935	0.771	0.611	0.439
1.0	0.883	0.770	0.674	0.537	0.383
1.5	0.426	0.349	0.314	0.264	0.199
2.0	0.222	0.193	0.177	0.154	0.122
2.5	0.187	0.147	0.133	0.112	0.086
3.0	0.123	0.110	0.097	0.081	0.063
4.0	0.066	0.057	0.054	0.049	0.040
5.0	0.052	0.045	0.039	0.033	0.028

Table A.2 (continued)

(c) soil condition 3 (alluvial)

T_o (sec)	$\xi_A^{(s)}$				
	h=0.02	h=0.05	h=0.10	h=0.20	h=0.40
0.1	1.996	1.701	1.503	1.288	1.112
0.15	2.501	1.933	1.638	1.364	1.105
0.2	2.979	2.198	1.741	1.383	1.091
0.25	2.465	1.866	1.544	1.286	1.021
0.3	2.259	1.744	1.421	1.160	0.926
0.4	2.087	1.565	1.270	1.002	0.782
0.5	2.209	1.720	1.321	0.961	0.696
0.6	1.892	1.534	1.219	0.903	0.625
0.7	1.525	1.284	1.083	0.821	0.564
0.8	1.524	1.252	1.014	0.730	0.498
0.9	1.492	1.205	0.926	0.652	0.440
1.0	1.270	1.037	0.815	0.567	0.394
1.5	0.673	0.514	0.417	0.322	0.229
2.0	0.338	0.272	0.232	0.185	0.140
2.5	0.183	0.156	0.135	0.120	0.098
3.0	0.147	0.127	0.110	0.091	0.073
4.0	0.080	0.072	0.063	0.051	0.042
5.0	0.050	0.043	0.037	0.032	0.028

(d) soil condition 4 (very soft deposit)

T_o (sec)	$\xi_A^{(s)}$				
	h=0.02	h=0.05	h=0.10	h=0.20	h=0.40
0.1	1.499	1.298	1.188	1.115	1.045
0.15	1.718	1.451	1.278	1.149	1.054
0.2	1.981	1.600	1.387	1.203	1.062
0.25	2.119	1.707	1.453	1.235	1.066
0.3	2.448	1.968	1.618	1.318	1.064
0.4	3.018	2.307	1.792	1.322	1.014
0.5	2.553	2.011	1.649	1.329	0.959
0.6	2.440	1.997	1.669	1.278	0.890
0.7	2.691	2.166	1.683	1.195	0.809
0.8	3.180	2.231	1.661	1.138	0.722
0.9	2.333	1.835	1.385	0.982	0.638
1.0	2.158	1.627	1.250	0.856	0.558
1.5	0.990	0.693	0.556	0.432	0.309
2.0	0.375	0.322	0.273	0.231	0.190
2.5	0.265	0.211	0.186	0.155	0.124
3.0	0.168	0.138	0.124	0.110	0.090
4.0	0.101	0.087	0.074	0.065	0.056
5.0	0.083	0.072	0.059	0.049	0.039

Appendix B.

Table B.1 Parameters for estimation of η_{Da} and standard error

(a) $q=1$

μ	n_e	$a_{\eta D}$	$b_{\eta D}$	$\sigma_{\eta D}$
1	1	1.000	0.000	0.000
	3	0.923	0.013	0.021
	6	0.802	0.045	0.035
	10	0.667	0.091	0.045
	15	0.537	0.135	0.056
2	1	1.000	0.000	0.000
	3	0.850	0.048	0.037
	6	0.713	0.096	0.059
	10	0.606	0.129	0.070
	15	0.507	0.162	0.075
3	1	1.000	0.000	0.000
	3	0.799	0.063	0.045
	6	0.634	0.122	0.067
	10	0.518	0.159	0.075
	15	0.427	0.182	0.079
4	1	1.000	0.000	0.000
	3	0.776	0.078	0.052
	6	0.592	0.145	0.074
	10	0.468	0.181	0.080
	15	0.379	0.200	0.083

(b) $q=2$

μ	n_e	$a_{\eta D}$	$b_{\eta D}$	$\sigma_{\eta D}$
1	1	1.000	0.000	0.000
	3	0.926	0.012	0.019
	6	0.817	0.039	0.031
	10	0.700	0.075	0.039
	15	0.591	0.107	0.047
2	1	1.000	0.000	0.000
	3	0.861	0.043	0.034
	6	0.734	0.085	0.053
	10	0.635	0.114	0.063
	15	0.548	0.140	0.067
3	1	1.000	0.000	0.000
	3	0.817	0.054	0.039
	6	0.666	0.105	0.059
	10	0.558	0.137	0.067
	15	0.475	0.157	0.070
4	1	1.000	0.000	0.000
	3	0.800	0.066	0.045
	6	0.633	0.123	0.064
	10	0.518	0.155	0.070
	15	0.435	0.171	0.074

(c) $q=3$

μ	n_e	$a_{\eta D}$	$b_{\eta D}$	$\sigma_{\eta D}$
1	1	1.000	0.000	0.000
	3	0.930	0.011	0.018
	6	0.831	0.033	0.028
	10	0.729	0.061	0.034
	15	0.639	0.084	0.040
2	1	1.000	0.000	0.000
	3	0.871	0.038	0.030
	6	0.755	0.075	0.047
	10	0.665	0.099	0.055
	15	0.588	0.119	0.059
3	1	1.000	0.000	0.000
	3	0.834	0.046	0.034
	6	0.698	0.089	0.051
	10	0.601	0.115	0.058
	15	0.527	0.131	0.061
4	1	1.000	0.000	0.000
	3	0.822	0.055	0.039
	6	0.673	0.102	0.054
	10	0.571	0.128	0.060
	15	0.496	0.141	0.062

Table B.2 Parameters for estimation of η_{Aa} and standard error

(a) $q=1$

μ	n_e	$a_{\eta A}$	$b_{\eta A}$	$\sigma_{\eta A}$
	1	1.000	0.000	0.000
	3	0.919	0.024	0.011
1	6	0.810	0.059	0.021
	10	0.695	0.099	0.031
	15	0.587	0.136	0.039
	1	1.000	0.000	0.000
	3	0.939	0.033	0.012
2	6	0.871	0.070	0.021
	10	0.798	0.107	0.031
	15	0.719	0.147	0.040
	1	1.000	0.000	0.000
	3	0.933	0.032	0.010
3	6	0.876	0.059	0.017
	10	0.818	0.087	0.024
	15	0.757	0.118	0.032
	1	1.000	0.000	0.000
	3	0.927	0.031	0.012
4	6	0.863	0.058	0.020
	10	0.807	0.084	0.026
	15	0.753	0.108	0.033

(b) $q=2$

μ	n_e	$a_{\eta A}$	$b_{\eta A}$	$\sigma_{\eta A}$
	1	1.000	0.000	0.000
	3	0.923	0.022	0.011
1	6	0.823	0.052	0.019
	10	0.720	0.086	0.027
	15	0.625	0.116	0.033
	1	1.000	0.000	0.000
	3	0.941	0.031	0.011
2	6	0.876	0.066	0.020
	10	0.809	0.099	0.029
	15	0.736	0.135	0.037
	1	1.000	0.000	0.000
	3	0.935	0.030	0.010
3	6	0.880	0.056	0.016
	10	0.824	0.083	0.023
	15	0.768	0.111	0.030
	1	1.000	0.000	0.000
	3	0.929	0.030	0.012
4	6	0.867	0.056	0.019
	10	0.813	0.080	0.025
	15	0.763	0.102	0.031

(c) $q=3$

μ	n_e	$a_{\eta A}$	$b_{\eta A}$	$\sigma_{\eta A}$
	1	1.000	0.000	0.000
	3	0.926	0.020	0.010
1	6	0.834	0.046	0.017
	10	0.743	0.074	0.024
	15	0.661	0.097	0.029
	1	1.000	0.000	0.000
	3	0.944	0.030	0.011
2	6	0.882	0.062	0.019
	10	0.819	0.093	0.027
	15	0.753	0.124	0.034
	1	1.000	0.000	0.000
	3	0.937	0.029	0.009
3	6	0.884	0.054	0.015
	10	0.831	0.079	0.022
	15	0.779	0.103	0.028
	1	1.000	0.000	0.000
	3	0.932	0.028	0.011
4	6	0.872	0.053	0.018
	10	0.820	0.075	0.024
	15	0.772	0.096	0.029

Appendix C.

Table C.1 Standard acceleration response ratio (Model-II)

(a) Normal site condition ($-0.6<S_n<0.6$)

T_o (sec)	$\xi_A^{(s)}$				
	$h=0.02$	$h=0.05$	$h=0.10$	$h=0.20$	$h=0.40$
0.1	1.855	1.595	1.438	1.281	1.122
0.15	2.298	1.860	1.598	1.366	1.118
0.2	2.637	2.035	1.665	1.376	1.091
0.25	2.481	1.943	1.621	1.334	1.039
0.3	2.424	1.892	1.531	1.234	0.963
0.4	2.269	1.757	1.417	1.108	0.826
0.5	2.186	1.695	1.327	0.980	0.717
0.6	1.883	1.492	1.174	0.886	0.626
0.7	1.501	1.229	1.019	0.783	0.555
0.8	1.409	1.142	0.932	0.698	0.495
0.9	1.262	1.053	0.842	0.631	0.439
1.0	1.042	0.884	0.738	0.551	0.388
1.5	0.522	0.416	0.358	0.290	0.213
2.0	0.268	0.226	0.201	0.168	0.131
2.5	0.185	0.151	0.134	0.116	0.092
3.0	0.134	0.118	0.103	0.085	0.068
4.0	0.072	0.064	0.058	0.050	0.041
5.0	0.051	0.044	0.038	0.033	0.028

(b) Very soft site condition ($0.6\leqq S_n<1$)

T_o (sec)	$\xi_A^{(s)}$				
	$h=0.02$	$h=0.05$	$h=0.10$	$h=0.20$	$h=0.40$
0.1	1.499	1.298	1.188	1.115	1.045
0.15	1.718	1.451	1.278	1.149	1.054
0.2	1.981	1.600	1.387	1.203	1.062
0.25	2.119	1.707	1.453	1.235	1.066
0.3	2.448	1.968	1.618	1.318	1.064
0.4	3.018	2.307	1.792	1.322	1.014
0.5	2.553	2.011	1.649	1.329	0.959
0.6	2.440	1.997	1.669	1.278	0.890
0.7	2.691	2.166	1.683	1.195	0.809
0.8	3.180	2.231	1.661	1.138	0.722
0.9	2.333	1.835	1.385	0.982	0.638
1.0	2.158	1.627	1.250	0.856	0.558
1.5	0.990	0.693	0.556	0.432	0.309
2.0	0.375	0.322	0.273	0.231	0.190
2.5	0.265	0.211	0.186	0.155	0.124
3.0	0.168	0.138	0.124	0.110	0.090
4.0	0.101	0.087	0.074	0.065	0.056
5.0	0.083	0.072	0.059	0.049	0.039

Appendix D. Notations

A_e, A_{e1}=equivalent ground acceleration (EQA);
A_{ea}=average equivalent ground acceleration (AEQA);
A_p=peak ground acceleration (PGA);
a_γ=parameters for statistical estimation of γ;
a_η, $a_{\eta D}$, $a_{\eta A}$, b_η, $b_{\eta D}$, $b_{\eta A}$=parameters for statistical estimation of η_a, η_{Da}, η_{Aa};
$C_a(S_n)$=PGA correction factor for specific site condition;
C_e, C_{e1}=EQA factor (EQA/PGA);
C_{ea}=AEQA factor (AEQA/PGA);
D=damage function;
h=damping factor;
k_h, k_{hd}, k_{hm}=design acceleration coefficients;
M=earthquake magnitude;
n_e=allowable number of load reversals;
p, q=constants characterizing successive structural failure;
S_e, S_{e1}, S_{Ae}, S_{Ae1}=effective response;
S_n=site parameter;
S_p=response spectrum (peak response value);
T_d=ground motion duration;
T_0=undamped natural period;
X_i, X_{Di}, X_{Ai}=i-th largest response excursion;
X_e, X_{De}, X_{Ae}=response excursion corresponding to effective response;
$x(t)$=response time history;
Z_i=i-th largest response value;
α, β=design response ratios;
γ, γ'=peak response factors;
γ_a, γ_{Da}, γ_{Aa}=average peak response factors;
Δ=epicentral distance;
η, η_D, η_A=effective response factors;
η_a, η_{Da}, η_{Aa}=average effective response factors;
μ=ductility factor;
μ_d=design ductility factor;
ξ, ξ_D, ξ_A=response ratios;
$\bar{\xi}$=average response ratio;
$\xi^{(s)}$, $\xi^{(s)\prime}$, $\xi_D{}^{(s)}$, $\xi_A{}^{(s)}$=standard response ratios: and,
$\hat{}$=statistical estimate.