A SIMPLIFIED METHOD TO PREDICT PEAK VALUE AND TREND OF SEISMIC RELATIVE MOTION BETWEEN ADJACENT BUILDINGS

地震時における隣接建物間の相対変位の最大値やその傾向の簡易予測法

Kazuhiko KASAI* and Binh T. TRAN**

笠井和彦, チャン タン ビン

Seismic pounding of two closely-spaced buildings can be a serious hazard, and adequate building separation distance must be used to preclude pounding. It can be calculated as the peak value of the relative displacement between the buildings through time history analysis. For an alternative and more practical estimate, the writers proposed the “spectral difference (SPD) rule” that simply uses the buildings’ peak displacements. The present paper proposes a simplified method combining the rule with elastic response spectrum to approximate peak inelastic displacement of each building. Utilizing this SPD-based method, the paper clarifies the relative displacement trends based on building periods, damping ratios, ductility demands, and earthquake spectral characteristics. The method is validated and various trends discussed by using numerous code-compatible building pairs and 33 earthquakes. The SPD-based method is shown to be much more accurate than other simplified methods, due to its ability to account for the inelastic vibration phase of two buildings. The results are also readily applicable to the problem of finding a necessary sliding seat distance to prevent falling of a bridge, which is supported by two separate structures developing relative motion.

Keywords: adjacent inelastic structures, relative displacement, required separation, inelastic vibration phase, spectral difference (SPD) rule, response spectrum.

隣接弾塑性構造, 相対変位, 建物要求間隔, 弾塑性振動位相, スペクトル差法, 応答スペクトル

1. INTRODUCTION

1.1 Relative Displacement Problem

In recent years, there has been increased awareness of the potential impact of buildings during moderate to strong earthquakes due to insufficient separation distance^{1) - 3)}. Pounding can occur between the two buildings adjacent to each other, or between the segments of a building connected via an expansion joint or a bridge component.

Magnitude of adequate building separation distance s (Fig. 1) can be estimated by calculating the buildings’ peak relative displacement through time history analysis. However, since one cannot determine specifically the future earthquake time history, a spectrum approach that uses an ensemble from the past and potential earthquakes would be more reliable and preferred. It also has an advantage of ease of use over the time history approach.

The peak relative displacement between the two buildings depends not

Figure 1. Relative Displacement of Adjacent Buildings.

only on the peak displacement of each separate building, but also on the “vibration phase” associated with their elastic and inelastic responses. Therefore, the effects from key parameters such as adjacent buildings’ vibration periods, damping ratios, heights, ductility demands, and even hysteretic types must be considered in order to understand the relative displacement problem. Obviously, time history approach is not appropriate for such a task.

1.2 Objectives and Scopes

Kasai et al.^{4), 5)} presented a method called “spectral difference (SPD) rule” using response spectrum to estimate the maximum relative displacement. The method provides a closed-form solution that relates those key parameters with the buildings’ relative motion. Nevertheless, using directly this SPD rule requires either inelastic response spectrum or time history analysis to obtain peak inelastic displacement of each separate building.

The objective of this paper, therefore, is to propose a practical SPD-based method by implementing more simplifications. The proposed method employs only elastic response spectrum to approximate peak inelastic displacement of each structure.

Validation study is conducted considering various adjacent building pairs having stiffness and strength consistent with the Japanese seismic code and an ensemble of 33 earthquakes scaled to different levels. For each case, the peak relative displacement is estimated using the SPD-based method and others. Time history analysis is also used to obtain an ‘exact’ solution. The accuracy of the SPD-based method will

* Prof., Struct. Eng. Research Center, Tokyo Institute of Technology, Ph. D.

** Graduate Student, Dept. of Built Environment, Tokyo Institute of Technology

東京工業大学建築物理研究センター 教授・Ph. D.
東京工業大学人間環境システム専攻 大学院生

be exemplified by comparing with other methods.

Furthermore, the trend of the peak relative displacement will be clarified by explaining the complex effect of the above-mentioned key factors through the SPD-based method. Unlike the previous studies, this work aims to emphasize on the significant effect of yielding on buildings' phase motion and relative displacement. The study would be useful for understanding and controlling the relative displacement.

The results are also readily applicable to the problem of finding a necessary sliding seat distance to prevent falling of a bridge, which is supported by two separate structures developing relative motion.

2. RELATIVE DISPLACEMENT AND VARIOUS ESTIMATES

2.1 Past Rules

From Fig. 1, the relative displacement $u_{rel}(t)$ between buildings A and B is expressed as

$$u_{rel}(t) = u_A(t) - u_B(t) \quad (1)$$

where $u_A(t)$, $u_B(t)$ = displacement time histories at potential pounding locations. From now on, the subscripts "A" and "B" shall refer to buildings A and B, respectively.

Peak relative displacement between the buildings is determined as

$$u_{rel}(TH) = \max [u_{rel}(t)] \quad (> 0) \quad (2)$$

'TH' indicates results from time history analysis, and pounding is avoided if the separation distance $s > u_{rel}(TH)$. Note that one needs to consider both directions of earthquake, it is equivalent to utilize a single direction earthquake and additionally consider $u_B(t) - u_A(t)$ in Eqs. 1, 2.

Two other methods for estimating the peak relative displacement are the absolute-sum (ABS) rule and the square-root-of-sum-of-squares (SRSS) rule such as

$$u_{rel}(ABS) = u_A + u_B, \quad u_{rel}(SRSS) = \sqrt{u_A^2 + u_B^2} \quad (3a, b)$$

where u_A , u_B = the absolute peak displacements of the buildings, which can be obtained from the response spectrum. The use of the SRSS rule is stipulated in the U.S. seismic code⁶⁾.

2.2 Spectral Difference (SPD) Rule

Unlike the ABS and SRSS rules, the SPD rule uses a cross correlation coefficient ρ_{AB} and provides average estimate also for both directions of earthquake as just mentioned above.

$$u_{rel}(SPD) = \sqrt{u_A^2 + u_B^2 - 2\rho_{AB}u_Au_B} \quad (4)$$

Especially when Eq. 4 gives very small u_{rel} , although theoretically valid, Ref. 4 provided a lower bound of its value for a practical and conservative reason. In the present paper, however, such a consideration is omitted in order to clarify the accuracy of the present concept.

The ρ_{AB} reflects vibration phase of buildings A and B, and it was derived from a random vibration theory as follows^{5), 7)}:

$$\rho_{AB} = \frac{8\sqrt{\xi_A^*\xi_B^*}(\xi_B^* + \beta^*\xi_A^*)\beta^{1.5}}{(1 - \beta^{*2})^2 + 4\xi_A^*\xi_B^*(1 + \beta^{*2})\beta^* + 4(\xi_B^{*2} + \xi_A^{*2})\beta^{*2}} \quad (5)$$

where β^* = ratio of effective vibration periods T_B^*/T_A^* , ξ_A^* and ξ_B^* = effective damping ratios. Note that $0 \leq \rho_{AB} \leq 1$, and larger ρ_{AB} means more in-phase motion, and consequently smaller u_{rel} (Eq. 4).

Eq. 5 explains that T^* and ξ^* play a key role in vibration phase. When T_B^*/T_A^* is close to 1, and/or ξ_A^* and ξ_B^* are large, ρ_{AB} approaches 1, and in-phase motion develops. Inclusion of damping comes from the fact that damping tends to eliminate a free vibration portion of the seismic response, and mainly a forced vibration portion remains, making the two buildings vibrate similarly to the ground motion^{4), 8)}.

From Ref. 4, for the buildings with bilinear hysteresis (Fig. 2a) and stiffness degrading hysteresis (Fig. 2b), the above effective properties for μ up to 6 are given as:

$$\text{Bilinear: } T^* = T[1 + 0.09(\mu - 1)] ; \quad \xi^* = \xi + 0.084(\mu - 1)^{1.3} \quad (6a)$$

$$\text{Degrading: } T^* = T[1 + 0.18(\mu - 1)] ; \quad \xi^* = \xi + 0.16(\mu - 1)^{0.9} \quad (6b)$$

where T , ξ , μ = initial elastic vibration period, initial viscous damping ratio, and peak ductility demand, respectively. Note that these effective period and damping differ from those used in predicting peak displacement of a single inelastic structure. Besides, effect of residual displacement (one-sided movement) was already statistically accounted for in Eqs. 6 via consideration of both directions of many earthquakes⁴⁾.

Figure 2. Hysteretic Behavior: (a) Bilinear Building Model, and (b) Stiffness Degrading Model.

3. TRENDS OF RELATIVE MOTION AND PHASE

Kasai et al.⁴⁾ discussed the trends of ρ_{AB} and u_{rel} for adjacent inelastic buildings having the same ductility demand μ . In this chapter, a more general case of different μ 's will be investigated.

3.1 Trends of Relative Motion

Consider two single-degree-of-freedom (SDOF) systems A and B with initial vibration periods T_A and $T_B = 1.0s, 1.3s$, and initial viscous damping ratios $\xi_A = \xi_B = 0.02$. The stiffness degrading hysteretic model (Fig. 2b) is used with strain-hardening ratio 5%. The systems are designed under the three cases described below, and they are subjected to the 1940 Imperial Valley earthquake (117 El Centro station, 0.35g).

Figure 3. Time History Responses of the Systems in Three Cases.

Case 1: Systems are elastic, and only small damping is given. Thus, they must vibrate mostly out-of-phase.

Case 2: Systems are inelastic, and designed to develop $\mu_A = \mu_B = 3$. In-phase motion is promoted due to the hysteretic damping.

Case 3: Systems are inelastic, and designed to develop distinct values of $\mu_A = 6$ and $\mu_B = 3$. Like case 2, in-phase motion develops due to hysteretic damping. In addition, although $T_B/T_A \approx 1.3$, different μ_A and μ_B causes $T_B^*/T_A^* \approx 1$, leading to strong in-phase motion (Sec. 3.2).

Table 1. Results of Three Considering Cases.

Case	Ductility		Peak Disp. (cm)			u_{rel}	
	μ_A	μ_B	u_A	u_B	u_{rel}	$\max(u_A, u_B)$	$u_A + u_B$
1	1	1	16.81	11.09	20.02	1.19	0.72
2	3	3	9.70	11.77	6.50	0.55	0.30
3	6	3	11.84	11.77	4.23	0.57	0.18

As Fig. 3a shows, Case 1 develops out-of-phase movement between the elastic systems A and B due to their different periods. In contrast, the responses of inelastic systems in Cases 2 and 3 are significantly in phase (Figs. 3b, and 3c). Fig. 3d plots $u_{rel}(t)/(u_A + u_B)$, which highlights increasing trend of the in-phase motion in the order of Cases 1, 2, and 3. Table 1 lists magnitudes of each response quantity.

The SPD, SRSS, and ABS rules are used to estimate u_{rel} of the above three cases by using the peak displacements u_A , u_B obtained from the time history analyses. In case of the SPD method, the values are also divided by the yield displacements to calculate μ_A , μ_B , and the cross correlation ρ_{AB} (Eqs. 5 and 6).

Table 2 indicates superior prediction of u_{rel} by the SPD rule. The errors of the SRSS and ABS rules increase, especially when buildings exhibit large inelastic deformations. Eqs. 4 to 6 of the SPD rule, therefore, could clarify the complex effects from the initial vibration periods, viscous damping ratios, and ductility demands varied herein. This point will be further demonstrated below.

Table 2. Approximation Results Using SPD and Other Methods.

Case	Effective Parameters				ρ_{AB}	u_{rel}		
	T_A^*	T_B^*	ξ_A^*	ξ_B^*		$u_{rel}(\text{SPD})$	$u_{rel}(\text{SRSS})$	$u_{rel}(\text{ABS})$
1	1.00	1.30	0.02	0.02	0.02	1.00	1.01	1.39
2	1.36	1.77	0.32	0.32	0.85	0.96	2.35	3.30
3	1.90	1.77	0.70	0.32	0.91	1.19	3.95	5.58

3.2 Trends of Vibration Phase

Fig. 4 plots cross correlation coefficients ρ_{AB} (Eq. 5) in order to illustrate general trends of systems' phase.

For elastic systems (Fig. 4a), the ρ_{AB} is high only if both $T_B/T_A \approx 1$ and $\xi_B/\xi_A \approx 1$, and it is very low otherwise (Case 1).

For inelastic systems (Figs. 4c and 4d), however, $T_B/T_A \neq 1$ can lead to the largest ρ_{AB} , depending on μ_A and μ_B (Cases 2 and 3). This is because the effective period and damping, instead of initial period and damping, governs ρ_{AB} (Eq. 5) when systems are inelastic. Case 3 gives a typical example, where $T_B^*/T_A^* \approx 0.93$ (Table 2) in contrast to $T_B/T_A = 1.3$, and it gave the smallest u_{rel} among all cases (Tables 1 and 2, Fig. 3).

Since T^* and ξ^* are affected by the ductility demand μ , Fig. 4 provides the direct and useful information regarding the effect of μ . The peaks of the ρ_{AB} -curves are close to 1.0 for a wide range of T_B/T_A when $\mu > 2$, indicating importance of including even moderate amount of yielding.

Thus, large ρ_{AB} results even when the initial period ratio $T_B/T_A \neq 1$, and maximum ρ_{AB} is obtained when systems A and B have different μ 's. This is the reason why Case 3 shows more in-phase motion than Case 2.

4. SIMPLIFIED SPD-BASED METHOD

As explained in Sec. 3.1, the SPD rule can perform well when peak

Figure 4. General Trends of Buildings' Phase (ρ_{AB}).

displacements u_A and u_B are accurately given. Inelastic spectrum provides these values, but elastic spectrum is preferred from a practical reason. This chapter, therefore, proposes a "simplified SPD-based method" that combines the SPD rule and the elastic spectrum.

4.1 Inelastic Displacement Prediction by Elastic Spectrum

Studies have been conducted worldwide to predict inelastic response via elastic spectrum. They utilize the strong correlation between the ductility demand μ and the strength reduction factor R_μ such as (Fig. 5):

$$R_\mu = \frac{Q_e}{Q_y} = \frac{\Delta_e}{\Delta_y} \quad (7)$$

where Q_y , Δ_y = yield shear and yield displacement of the system given, and Q_e , Δ_e = base shear and displacement when the system is presumed to behave elastically. Note that the above displacements are defined at the effective height⁹⁾ H_{eff} of the building.

The $\mu - R_\mu$ relationships have been proposed by Newmark and Hall¹⁰⁾, Uang¹¹⁾, and others. In the present study we will utilize Nassar and Krawinkler's rule¹²⁾ as follows:

$$\mu = 1 + \frac{R_\mu^c - 1}{c} \quad (8a, b)$$

$$c = \frac{T^a}{1 + T^a} + \frac{b}{T}$$

Figure 5. Building Model and Elastic Approximation.

where a and b depend on the strain-hardening ratio α . By applying linear regression to the α -values tabulated in Ref. 12,

$$a = 1.02 - 2.18\alpha, \quad b = 0.41 - 1.21\alpha \quad (\alpha \leq 0.1) \quad (9)$$

Unlike the equal energy rule¹⁰⁾ or the equal displacement rule^{10), 11)}, Eq. 8 is applicable to the whole range of short to long period, and considers the fact^{11) - 14)} that for a short period R_μ is less than μ . Eq. 8 was already validated¹²⁾ for μ -value up to 8.

Once Δ_e is estimated from an elastic spectrum, μ is obtained from Eqs. 7 and 8, and the peak inelastic displacement Δ (Fig. 5) at height H_{eff} is

$$\Delta = \mu \cdot \Delta_y \quad (10)$$

Following Refs. 12, 11 and 16, the damping ratio shall be 5% when using the elastic spectrum.

4.2 Step-by-Step Procedure for Simplified SPD-Based Method

For simplified prediction of u_{rel} without conducting time history analysis, we combine the SPD rule with elastic response spectrum. The height of building A is set equal to or greater than that of building B, i.e., $H_A \geq H_B$ will be considered (Fig. 1).

The procedure obtains the following parameters in order:

- (1) Elastic displacements Δ_{eA} and Δ_{eB} from elastic spectrum.
- (2) Inelastic displacements Δ_A and Δ_B using Eqs. 7 to 10.
- (3) Inelastic displacements u_A and u_B at the common critical height.
- (4) Effective periods T_A^* and T_B^* , damping ratio ξ_A^* and ξ_B^* from Eqs. 6.
- (5) Cross correlation ρ_{AB} from Eq. 5, and $u_{rel}(SPD)$ from Eq. 4.

Note that Δ = displacement at each building's effective height which may differ from each other, and u = displacement at a common critical elevation ($= H_B$) of the two buildings where pounding is likely (Fig. 1).

In the present study, a simple straight-line building deformation mode is assumed. For step (3) above, therefore, effective height $H_{eff} = 2H/3$ is considered for each building, and the following relationship is used:

$$u_A = 1.5(H_B/H_A) \Delta_A, \quad u_B = 1.5 \Delta_B \quad (11)$$

4.3 Building Models Consistent with Code

Building models to be used in the following sections will be defined here. The preliminary data required for the application of the SPD-based method are the initial period T and the yield displacement Δ_y of each building. From now on, T is assumed to coincide with that given by the Japanese Seismic Code¹⁵⁾, i.e.,

$$T = 0.03H \quad (\text{Steel Bldg.}), \quad T = 0.02H \quad (\text{Concrete Bldg.}) \quad (12)$$

where H = total height of the building in meters.

Building yield shear is set to ΩQ_y , where Ω = overstrength factor, and Q_y = yield shear required by the code such as (Fig. 5):

$$Q_y = C_0 R_f W \quad (13)$$

In Eq. 13, $C_0 = 0.25$ and 0.30 are assumed for the steel and concrete buildings, respectively. The required values are somewhat arbitrary as long as $C_0 \geq 0.2$, and they are made equal to the D_s -factors¹⁵⁾. Indeed, these higher values may better approximate the actual behavior, as suggested by Fig. 5. Note also that the overstrength factor Ω will be varied later (Fig. 5), making the specific C_0 -value less significant in such a study. Also, all the buildings in the present study appear to have enough stiffness to satisfy the code drift angle limit $1/200$ (Fig. 5).

The design spectral coefficient R_f is obtained from the formula below

$$R_f = \begin{cases} 1 & ; T < T_c \\ 1 - 0.2(T/T_c - 1)^2 & ; T_c \leq T < 2T_c \\ 1.6T_c/T & ; 2T_c \leq T \end{cases} \quad (14a-c)$$

where medium soil is assumed and the critical period $T_c = 0.6s$. Based on these, the yield displacement Δ_y at height H_{eff} is calculated as:

$$\Delta_y = C_0 \cdot R_f \cdot g(T/2\pi)^2 \quad (15)$$

4.4 Example Application

This section demonstrates an example application of the SPD-based simplified method to estimate the relative displacement u_{rel} .

Consider 15-story building A and 12-story building B. The buildings' heights are $H_A = 60$ m and $H_B = 48$ m, respectively. They are designed as ductile steel moment frames. Accordingly, Eqs. 12 to 15 give

$$\begin{aligned} T_A &= 0.03H_A = 1.80s, & \Delta_{yA} &= 10.7cm, \\ T_B &= 0.03H_B = 1.44s, & \Delta_{yB} &= 8.6cm. \end{aligned}$$

Initial damping ratio $\xi = 0.02$, and strain-hardening ratio $\alpha = 0.05$ is considered. The buildings are subjected to the 1994 Northridge earthquake (Newhall NS record, 0.59g) scaled to 0.4g peak acceleration. See the following steps:

- (1) The 5% damping elastic displacement spectrum (Sec. 4.1) of the scaled Newhall record gives

$$\begin{aligned} \Delta_{eA} &= S_d(T_A, 0.05) = 28.1cm, \\ \Delta_{eB} &= S_d(T_B, 0.05) = 29.3cm. \end{aligned}$$

- (2) Using the strength reduction factor of Eq. 7, ductility demands and inelastic displacements are estimated from Eqs. 8 and 10.

$$\begin{aligned} \mu_A &= 1 + \{(\Delta_{eA}/\Delta_{yA})^{0.82} - 1\}/0.82 = 2.47, & \Delta_A &= \mu_A \Delta_{yA} = 26.5cm \\ \mu_B &= 1 + \{(\Delta_{eB}/\Delta_{yB})^{0.83} - 1\}/0.83 = 3.13, & \Delta_B &= \mu_B \Delta_{yB} = 26.9cm \end{aligned}$$

- (3) The critical height is H_B (Fig. 1a), and from Eq. 11

$$\begin{aligned} u_A &= 1.5(H_B/H_A) \Delta_A = 31.8cm \\ u_B &= 1.5 \Delta_B = 40.2cm \end{aligned}$$

- (4) Effective periods and effective damping are estimated from Eq. 6a.

$$\begin{aligned} T_A^* &= T_A [1 + 0.09(\mu_A - 1)] = 2.04; & \xi_A^* &= \xi_A + 0.084(\mu_A - 1)^{1.3} = 0.16 \\ T_B^* &= T_B [1 + 0.09(\mu_B - 1)] = 1.72; & \xi_B^* &= \xi_B + 0.084(\mu_B - 1)^{1.3} = 0.24 \end{aligned}$$

- (5) Substituting the above into Eq. 5, we calculate $\rho_{AB} = 0.84$. The peak relative displacement $u_{rel}(SPD)$ is estimated as

$$u_{rel}(SPD) = \sqrt{u_A^2 + u_B^2 - 2\rho_{AB}u_Au_B} = 22.04cm = 1.02u_{rel}(TH)$$

The result of inelastic time history analysis $u_{rel}(TH) = 21.60cm$, after averaging 21.45cm and 21.74cm obtained using positive and negative directions of the earthquake. If we use ABS- and SRSS-based methods,

$$\begin{aligned} u_{rel}(ABS) &= u_A + u_B = 72.1cm = 3.34u_{rel}(TH) \\ u_{rel}(SRSS) &= \sqrt{u_A^2 + u_B^2} = 51.3cm = 2.38u_{rel}(TH) \end{aligned}$$

These indicate that the SPD-based method provides by far the most accurate estimate for u_{rel} . The ABS-based method gives excessive conservatism. Clearly, the errors from the ABS- and SRSS-based methods stem from their inability to account for the vibration phase of the two buildings.

4.5 Relative Displacement under Code Design Basis Spectrum

It is assumed that the code-compatible steel buildings are subjected to the earthquake whose pseudo-acceleration spectrum S_{pa} has a magnitude of scaled $R_f g$. This section will consider a variety of buildings having $H_A = 12m \sim 96m$, $H_B/H_A = 0.4 \sim 0.8$, and overstrength factor $\Omega = 1 \sim 3$, as well as various scaling factors of $R_f g$. Since the SRSS- (adopted by IBC2000⁶⁾) and ABS-based methods neglect the vibration phase, trends of relative displacement u_{rel} are discussed below by using only the SPD-based method. Yet, the following graphs clearly illustrate again the inaccuracy of the two methods compared to the SPD-based method.

Fig. 6a shows a case where H_A is varied. Apparently, when H_B/H_A approaches 1, the dynamic properties of buildings become similar due to the code specifications, and they have the tendency to vibrate in-phase. SPD-based method captures this remarkable aspect, and explains why the u_{rel} of two very tall buildings is similar and somewhat less than that of a very tall building and a short building. Moreover, at the critical elevation H_B , the u_{rel} is almost proportional to H_A (Fig. 6) regardless of H_B/H_A as mentioned above. With $S_{pa} = R_f g$ and the overstrength $\Omega = 1$, the important relationship $u_{rel} \approx 0.006H_A$ is obtained from Fig. 6a.

Fig. 6b illustrates a case where $H_A = 60m$ and S_{pa} is scaled to different levels. This figure again reinforces the fact that inelastic displacement contributes importantly to the in-phase vibration between adjacent buildings. Under the stronger earthquake, u_A and u_B increase obviously,

but since ρ_{AB} also increases, Eq. 4 gives almost the same relative displacement u_{rel} (Fig. 6b). Thus, the aforementioned rule of $u_{rel} \approx 0.006H_A$ is still applicable for the earthquake exceeding the code design basis spectrum, as long as the building overstrength $\Omega = 1$.

Fig. 6c illustrates a case where $H_A = 60\text{m}$ and overstrength Ω is varied to more than 1. Since larger Ω leads to less inelastic deformation, the trend of in-phase vibration reduces, which means that stronger buildings have larger u_{rel} . Based on this and considering the above findings (Figs. 6a, b), regardless of building heights H_A , H_B , and earthquake intensity level, $u_{\text{rel}} \approx 0.01H_A$ may be used conservatively as long as $\Omega \leq 2$.

5.1 Parameters Considered for Validation

Buildings have 8 different heights of 3, 6, 9, 12, 15, 18, 21, and 24 stories with a common story height of 4m. For building pairs A and B, all possible combinations of 8 different heights are considered with the condition of $H_A \geq H_B$ (Fig. 1). The total number of building pairs is 36.

Stories: 3 ~ 24
 H_B/H_A : 0.13 ~ 1.0
 T_B/T_A : 0.08 ~ 1.5
 Bldg. Types: Steel and Concrete

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Bldg. A Stories
 Bldg. B Stories
 H_B/H_A

T_B/T_A →

C vs. S
 S vs. C
 S vs S, C vs. C
 (S=Steel, C=Concrete)

Table 4. Building Pairs A and B Used in Validation Study.

Rec No.	Earthquake		PGA (g)	Rec No.	Earthquake		PGA (g)
1	Imperial Valley 1940	El Centro NS	0.349	17	Iran 1978	Tabas N074E	0.880
2		El Centro EW	0.214	18		Tabas N344E	0.938
3	Kern County 1952	Taft N021E	0.156	19	Northridge 1994	Newhall NS	0.590
4		Taft N111E	0.179	20		Newhall EW	0.583
5	Tokachi oki 1968	Hachinohe NS	0.278	21		Sylmar NS	0.844
6		Hachinohe EW	0.204	22		Sylmar EW	0.605
7	Hyogoken Nanbu 1995	JMA Kobe NS	0.837	23	Chi Chi (Taiwan) 1999	TCU065 NS	0.575
8		JMA Kobe EW	0.632	24		TCU065 EW	0.790
9		Fukiai N240E	0.701	25		TCU068 NS	0.369
10		Fukiai N330E	0.818	26		TCU068 EW	0.512
11		JR Takatori NS	0.618	27		TCU074 NS	0.376
12		JR Takatori EW	0.670	28		TCU074 EW	0.598
13	Miyagiken oki 1978	Tohoku NS	0.263	29	Kocaeli (Turkey) 1999	Ypt NS	0.329
14		Tohoku EW	0.207	30		Ypt EW	0.235
15	Kushiro oki 1993	Kushiro N063E	0.707	31		Sakarya	0.415
16		Kushiro N153E	0.588	32	Artificial	BCJ-L1	0.212
				33		BCJ-L2	0.363

For each building pair, 31 past earthquakes and 2 artificial earthquakes (Table 3) are used, and each record is applied in both positive and negative directions. The records cover a variety of seismic intensities, and it is reasonable to scale them to the same peak ground acceleration (PGA). Thus, 4 different PGA scales of 0.2g, 0.4g, 0.6g, and 0.8g are considered for each record. In total, the number of earthquakes considered for each building pair is 264 [(33 earthquakes) x (2 directions) x (4 PGA scales)].

Using the elastic spectrum of each earthquake, inelastic peak displacement is estimated, and used for all the SPD-, SRSS-, and ABS-based methods. Also, in order to obtain exact solution for u_{rel} ,

Figure 7. Mean and Deviation of 33 Acceleration Spectra (PGA=0.4g, 5% Damping), Design Spectrum, and Strengths of Designed Buildings:

dynamic time history analyses are conducted using a SDOF nonlinear analysis program NONSPEC¹⁷⁾. In summary, this validation study examines a total of 38,016 cases [(144 building pairs) $\times$ (264 scaled records)].

5.2 Validation Results

For each of the cases mentioned above, the ratios of $u_{\text{rel}}(\text{SPD})$, $u_{\text{rel}}(\text{SRSS})$, and $u_{\text{rel}}(\text{ABS})$ to the $u_{\text{rel}}(\text{TH})$ are obtained, and they are averaged over 36 height combinations and 2 earthquake directions per combination of material type, earthquake, and PGA scale. The average values are shown in Fig. 8. Next, such values are further averaged over 33 earthquakes, and the results are summarized in Table 5.

Fig. 8 shows that $u_{\text{rel}}(\text{SPD})/u_{\text{rel}}(\text{TH}) \approx 1$ for most cases, indicating superior accuracy of the SPD-based method. ABS- or SRSS-based method appears to be largely conservative, as the PGA scale increases. This is because they do not account for the important effect of inelastic deformation, which, under stronger earthquake causes more in-phase motion between the two buildings. The standard deviation in Table 5 (number in parentheses) clearly strengthens the stable degree of SPD-based method. Although we observe some scattering of its estimates for stronger earthquakes in 0.6g and 0.8g PGA ensembles, the scattering is still much less than those of SRSS- or ABS-based method.

The important effect of inelastic in-phase motion is seen especially for the strong earthquakes like Kobe Japan (earthquakes 7 to 12), Iran and Northridge (earthquakes 17 to 22). These earthquakes, even scaled to the same PGA as other earthquakes, force each building to deform larger due to their higher spectral values over a wide period range, but they also produce at the same time more in-phase motion between the two

buildings. Such tendencies are accurately predicted by the simplified SPD-based method.

The SPD-method is also accurate for less severe earthquakes like Imperial Valley, Kern County and Hachinohe (earthquakes 1 to 6) or Kocaeli and BCJ (earthquakes 29 to 33). When PGA is 0.2g, the SRSS-based method is almost as accurate as the SPD-based method. This is because the average spectrum of 0.2G earthquakes approaches the buildings' design strength spectra, as can be imagined from Fig. 7. Thus, the buildings responded almost elastically, resulting in the small effective damping and consequently the small correlation ρ_{AB} , which made Eq. 3b and 4 produce almost the same estimate for u_{rel} . This trend, therefore, differs from what is discussed in Sec. 3.2.

Table 5. Average Accuracy of SPD, SRSS and ABS Methods[†].

Case	PGA = 0.2g			PGA = 0.4g			PGA = 0.6g			PGA = 0.8g		
	SPD	SRSS	ABS	SPD	SRSS	ABS	SPD	SRSS	ABS	SPD	SRSS	ABS
Steel vs. Steel	1.01 (0.18)	1.05 (0.24)	1.36 (0.38)	1.07 (0.25)	1.29 (0.48)	1.69 (0.73)	0.99 (0.31)	1.41 (0.66)	1.86 (0.99)	0.90 (0.35)	1.47 (0.84)	1.95 (1.24)
Concrete vs. Concrete	1.00 (0.16)	1.07 (0.32)	1.38 (0.50)	1.07 (0.27)	1.45 (0.81)	1.91 (1.20)	0.98 (0.37)	1.63 (1.21)	2.17 (1.75)	0.90 (0.45)	1.70 (1.36)	2.27 (1.96)
Steel vs. Concrete	0.99 (0.19)	1.02 (0.23)	1.36 (0.31)	1.08 (0.30)	1.30 (0.49)	1.75 (0.69)	1.00 (0.39)	1.43 (0.70)	1.94 (0.97)	0.89 (0.45)	1.50 (0.84)	2.05 (1.17)
Concrete vs. Steel	1.03 (0.22)	1.28 (0.58)	1.76 (0.80)	1.04 (0.32)	1.65 (0.72)	2.28 (1.00)	0.93 (0.40)	1.80 (0.89)	2.47 (1.24)	0.84 (0.45)	1.83 (1.00)	2.52 (1.38)

[†] () = standard deviation.

In average, the SPD-based method fulfills the expectations of accuracy as shown in Table 5. However, by careful inspection we also noticed that in the aforementioned strong earthquakes (earthquakes 7 to 12), the degree of accuracy has somehow exhibited rather wide

Figure 8. Average Ratios of $u_{\text{rel}}(\text{SPD})$, $u_{\text{rel}}(\text{SRSS})$, and $u_{\text{rel}}(\text{ABS})$ to Exact Solution $u_{\text{rel}}(\text{TH})$; Each Point Indicates Average over 72 Cases. (Thick and thin horizontal lines represent average accuracy and average $\pm$ standard deviation of the SPD-based method)

Figure 9. Estimated Values vs. Actual Peak Inelastic Displacements: 0.4g Ground Motion Ensemble.
(66 data point per graph)

variability. Fig. 9 explains this phenomenon using 0.4g PGA earthquakes: peak inelastic displacement estimated from the elastic spectrum and Eqs. 7 to 10 are plotted against the accurate time history analysis result, where each data point corresponds to one earthquake of one direction. For short period structures such as 3 and 6 story buildings the accuracy of the equations varies considerably, depending on earthquake (Fig. 9).

For the structures more flexible, Eqs. 7 to 10 seem to give good approximation. However, at a few earthquakes they produced significant errors (Fig. 9), reducing the accuracy of the SPD-based method. This was seen from the record like Chi Chi (earthquakes 24 to 26), under which the 18 to 24 story buildings showed biased motion, shifting to one side and developing much larger displacement than predicted by the elastic approach.

In order to maintain a reasonably conservative performance for practice, the required separation distance s_{req} based on $u_{\text{rel}}(\text{SPD})$ should be scaled by a conservative factor γ as follows

$$s_{\text{req}} = \gamma \cdot u_{\text{rel}}(\text{SPD}) \quad (16)$$

here $\gamma = 1.2$ to 1.45 for earthquake from moderate (0.2g) to catastrophic (0.8g) level, and linear interpolation may be applied for other levels.

Statistically, Eq. 16 would give more than 85% confidence for the s_{req} to be greater than the 'exact' value obtained from time history analysis. Note that SRSS and ABS methods require much higher γ values as well as $u_{\text{rel}}(\text{SRSS})$ and $u_{\text{rel}}(\text{ABS})$, resulting in much larger estimates to achieve the same confidence level.

6. APPLICATION OF SIMPLIFIED SPD-BASED METHOD

The previous sections indicated that the peak relative displacement of adjacent inelastic buildings could be approximated by the use of elastic SDOF systems. The writers also performed some numerical experiments using adjacent multi-degree-of-freedom (MDOF) systems, from which the straight-line deformation mode used for the SDOF model is still found to produce good estimate for u_{rel} . This modeling is not recommended, however, for buildings having very irregular deformed shape (e.g. soft or weak story building), unless such deformation is accounted for in the SDOF model. In spite, practitioners may find the SPD-based method simple and useful for preliminary design, since the method gives them full control over the relative displacement problem.

Making use of attractive features of the SPD-based method, an even simpler version of the method is presented herein. This graph-oriented version will equip users with a handy tool that can help them quickly judge the peak relative displacement between adjacent buildings.

If the building belongs to the constant spectral velocity domain, its peak elastic displacement becomes proportional to initial vibration

period. Moreover, since Eq. 8 for such a period range becomes similar to Newmark and Hall's equal displacement rule¹⁰⁾, elastic displacement is assumed to be the same as inelastic displacement. Based on these,

$$\frac{u_A}{u_B} = \frac{1.5(H_B/H_A)\Delta_A}{1.5\Delta_B} = \frac{H_B/H_A}{T_B/T_A} \quad (17)$$

Substituting the above relationship into Eq. 4 and by manipulations,

$$u_{\text{rel}}(\text{SPD}) = u_B \sqrt{1 + \left(\frac{H_B/H_A}{T_B/T_A}\right)^2 - 2\rho_{AB} \frac{H_B/H_A}{T_B/T_A}} \quad (18)$$

where u_B = peak elastic displacement at the top level of building B. The cross correlation ρ_{AB} still can be obtained by Eq. 5, using initial periods, damping ratios, and buildings' ductility demands.

From Eq. 12, let's introduce the material type combination factor β_c as

$$\beta_c = \frac{H_B/H_A}{T_B/T_A} = \begin{cases} 1.0 & \text{Steel vs. Steel, or Conc. vs. Conc.} \\ 1.50 & \text{Steel vs. Conc.} \\ 0.67 & \text{Conc. vs. Steel} \end{cases} \quad (19)$$

Then substituting this β_c factor into Eq. 18,

$$u_{\text{rel}}(\text{SPD})/u_B = \sqrt{1 + \beta_c^2 - 2\rho_{AB}\beta_c} \quad (20)$$

Eq. 20 is plotted in Fig. 10 for the buildings having initial damping ratio 2%, ductility demands 1 to 6, and H_B/H_A of 0.4, 0.7, and 1.0, respectively. Inappropriate estimates from the SRSS- and ABS-based methods are also shown by the red broken line and solid line, respectively. They appear to be constant with respect to not only ductility demands but also height ratios.

Fig. 10 can be used to quickly estimate the relative displacement u_{rel} . When the values of H_B/H_A , μ_A and μ_B , and peak elastic displacement u_B are known for a specific problem, a preliminary assessment of the u_{rel} can be carried out with these graphs.

For small H_B/H_A , effect of ductility demand on $u_{\text{rel}}(\text{SPD})/u_B$ is not very significant (Fig. 10 left). When H_B/H_A increases, u_{rel} is influenced not only by ductility variation but also on how the material combined.

If $H_B/H_A = 1$ and the buildings are made of the same structural material, the graph is symmetric with respect to μ_A and μ_B . Noticeably, $u_{\text{rel}} = 0$ when $\mu_A = \mu_B$, simply because the two buildings become identical hence they must show identical motion at any time. In contrast, a pair with different material combination always give $u_{\text{rel}} \neq 0$ even when $H_B/H_A = 1$ and $\mu_A = \mu_B$.

For $H_B/H_A < 1$, the graphs are asymmetric and indicate a better outcome in reducing u_{rel} is obtained by increasing μ_B rather than μ_A . Since $T_B/T_A < 1$, making $\mu_B > \mu_A$ will make T_B^*/T_A^* closer to 1 (Eq. 6), thereby forcing more in-phase motion (Eq. 5)

7. CONCLUSIONS

This paper proposed the new method to estimate seismic peak relative displacement between two inelastic buildings, by combining the writers' spectral difference (SPD) rule and elastic response spectrum. The method is applied to create a unique tool that can be used for preliminary and rapid assessment of the required separation to preclude pounding. They can be used also to find a necessary sliding seat length to prevent falling of a bridge supported by two separate structures.

The conclusions are as follows:

(1) The method is validated through extensive numerical experiments using numerous code-compatible building pairs with different heights and material combinations, as well as 33 earthquakes of 2 directions, scaled to 4 different levels. The method is found to accurately estimate the relative displacement, with a narrow variability of error. As with the appropriate range of μ -value mentioned in Sec. 2.2 and 4.1, this method

Figure 10. Relative Displacement Normalized by Peak Displacement (u_B) at Top of Building B, Constant Velocity Spectrum Assumed.

is believed to hold for a ductility ratio μ up to 6.

(2) Determination of relative displacement requires considerations of many factors such as; building heights, elastic vibration periods, initial yield strengths, hysteretic types, and spectrum characteristics as well as intensities of the earthquakes. Only the SPD-based method explicitly accounts for and clarifies the complex effects of these key parameters, and its use is simple.

(3) The absolute sum (ABS)-based method is excessively conservative for the level of earthquake as well as building stiffness and strength, specified in the current code. The square-root-of-sum-of-square (SRSS)-based method used in the US Code IBC2000⁶⁾ gives reasonably conservative estimate for moderate earthquakes, but remains incorrect for strong earthquakes because of not accounting for the relevant effect of hysteretic damping.

More research is motive, especially in consideration of the effects from high supplementary damping, scattering of initial vibration period and yield strength of structure; as well as multi-degree-of-freedom modeling of the buildings with various irregularities.

Lastly, this study assumed no time lag of excitations against adjacent structures. The assumption, which is based on the writers' previous study about effects of seismic travel wave on elastic structures⁵⁾, follows considerations that buildings' dimensions are not so spacious, their vibration periods are often long enough, and the soil is not too soft. Nevertheless, more in-depth study will be conducted so that the proposed method here for inelastic structures can include effect of travel wave, for covering a wider range of application.

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和文要約

1 はじめに

1.1 相対変位問題

地震時に隣接建物が異なる揺れ方をすると深刻な衝突被害が起こり得る(図1)。また、エキスパンションジョイントや連結橋をもつ建物でも同様である。衝突回避に必要な建物間隔は、非衝突を前提とした時刻歴解析で両建物間の最大相対変位を求め決定できるが、それでは、ある地震入力での特解を得るだけである。一方、応答スペクトルを用いる手法があれば、様々な地震や建物の特性の影響を包括的に捉えて相対変位問題を解明でき、評価も容易になる。

1.2 目的および全体概要

Kasaiらは、各建物の振動位相を建物周期・減衰の関数として表し、これと最大変位の値のみから最大相対変位を求める「SPD(スペクトル差)則」を提案した。しかし、各建物の最大変位の評価は、弾塑性スペクトルまたは弾塑性時刻歴解析を必要としていた。

本論では、より有用な手法として、SPD則に弾性スペクトルを併用した簡易評価法を提案する。日本の規準に則った隣接建物を2体の1自由度モデルとして多数作成し、様々に増幅した33地震波を用いて本手法の精度を実証し、スペクトルに基づく他手法とも比べる。なお本手法は、2つの構造物が離れるため連結橋が落下するような問題にも有効で、橋端のローラー支持部長さの決定にも用いられる。

2 相対変位とその様々な評価法

2.1 既往評価則

建物A, Bの衝突が予想される位置(図1)でのそれぞれの最大変位絶対値 u_A , u_B を用い、ABS(絶対和)則やSRSS(2乗和平方根)則により、最大相対変位 u_{rel} を求める(式3a, b)。

2.2 SPD則

SPD則は、 u_A , u_B のほかに両建物間の相関係数 ρ_{AB} を含む(式4, 5)。 ρ_{AB} は建物の周期・減衰定数で陽に表されるが、弾塑性建物では等価周期・等価減衰を用い、一方それらは初期(弾性)周期、初期減衰、塑性率で表される。また、 $0 \leq \rho_{AB} \leq 1$ であり、 ρ_{AB} が大きいほど両建物の位相が類似することを示し、 u_{rel} の評価値が小さくなる。両等価周期が類似する場合や等価減衰が高い場合に ρ_{AB} が大きい。

3 相対変位と振動位相

3.1 相対変位の傾向

異なる弾性周期をもち隣接する2体の1自由度系で、1)両者が低減衰の場合、2)両者が弾塑性で塑性率3のため等価減衰が高い場合、3)両者が弾塑性で短周期、長周期構造それぞれで塑性率6と3の場合を考える。応答時刻歴から、位相差や u_{rel} はこれらの順番に小さくなる(図3)。また、時刻歴から得た各構造の最大変位を上記評価則に代入した場合、SPD則が u_{rel} を正確に予測し、SRSS則、ABS則は相関係数 ρ_{AB} をそれぞれ一定値0と1とするため誤差が大きい。

3.2 振動位相の傾向

相関係数 ρ_{AB} と弾性周期、減衰定数、塑性率の関係をプロットした(図4)。両建物が弾性だと、弾性周期・減衰定数が両建物でほぼ一致しない限り ρ_{AB} が低く、位相がかなり異なる。弾塑性時には塑性率が高いほど ρ_{AB} が高くなる。また、特に両建物のうち短周期構造の塑性率が大きくて等価周期が長周期構造とほぼ等しくなると、 $\rho_{AB} \approx 1$ 、つまり、位相が酷似する(表2)。これが、前節3)が最小の相対変位を示した理由である。このように、SPD則によれば、隣接する弾塑性構造における位相を良好な精度で予測できる。

4 SPD則に基づく簡易手法

4.1 弾性スペクトルによる個々の建物の弾塑性変位予測

弾性スペクトルから弾塑性構造物の最大応答を予測する手法に関し、多くの研究がなされてきた。これらからNassar-Krawinkler式(8-10)を採用する。弾性と仮定して求めた最大変位(以後、弾性変位)と構造物の降伏変位の比、つまり強度低減係数を用いて、弾塑性構造のそれぞれ最大変位(以後、弾塑性変位)を近似的に求める(図5)。

4.2 SPD則に基づく簡易な相対変位評価の手順

両建物の弾性周期、初期減衰定数、降伏変位が既知とする。手順として、1)両建物の等価高さでの弾性変位を弾性応答スペクトルにより求める。2)同位置における弾塑性変位と塑性率を4.1節手法で求める。3)衝突が予想される高さ位置での両建物相対変位を、直線変形モードを仮定し幾何学から求める。4)等価周期・減衰定数を求める。5)相関係数 ρ_{AB} および最大相対変位 u_{rel} をSPD則から求める。

4.3 耐震規準に則った建物モデル

日本の一次設計に則った建物モデルを考える。鉄骨および鉄筋コンクリートのラーメン構造2種を考慮し、それぞれの弾性周期の規準予測式を用いる。また、降伏力も R_L 曲線に基づく規準要求値を満たす。剛性の要求値は満たされている。

4.4 簡易手法の適用例

15階建てと12階建ての鉄骨建物を例にとり、4.2節の手順を詳細に示す。これらが、0.4gに基準化されたNorthridge地震(Newhall波)をうけたとして、SPD則、SRSS則、ABS則に基づく簡易手法で、弾性スペクトルから u_{rel} 値をそれぞれ評価したところ、やはりSPD則に基づくものが格段に高い精度を示した。

4.5 耐震規準に則った隣接建物の相対変位

我国のレベル2設計荷重を標準として、入力荷重、建物高さ、余剰耐力係数を変化させ、 u_{rel} の傾向を図6に示す。余剰耐力係数1の場合、0.4g以上の大地震レベルでは、両建物高さの比によらず、 u_{rel} がほぼ一定で、高層側高さ(60m)の0.006倍となる。余剰耐力係数がより大きいと u_{rel} が増すが、2以下であれば、高層側高さの0.01倍以下である。建物強度が大だと u_{rel} が増える理由は、塑性化しにくいため等価減衰が少なく、よって両建物の位相が異なるためである。

5 SPD則に基づく簡易手法の精度検証

5.1 検証に用いたパラメータ

隣接建物の高さ組み合わせ36種、鉄骨またはコンクリートの組み合わせ4種からなる計144種を考慮する(表4)。地震波は33種、正負方向の2種、最大加速度0.2g~0.8gの4種であり、計264種を考慮する。よって、組み合わせは総計38,016となり、これらに対しSPD則、SRSS則、ABS則に基づく簡易手法を適用した。

5.2 検証結果

予想通りSPD則に基づく簡易手法が、誤差平均・バラツキ共に格段に小さい(図8)。SRSS則で0.2gの地震で同様な精度を得たのは、特に中長周期建物が概ね弾性であったからである。これ以上の地震ではSRSS, ABSの順に安全側だが、誤差平均・バラツキとも非常に大きい(表5)。なお、時刻歴解析では反対2方向の地震入力における u_{rel} を算定し、その最大値で必要建物間隔 s_{req} を決めることができるが、SPD則はその平均値を予測する。これと様々な地震での本簡易手法のバラツキを考慮し、確率85%の範囲で衝突を防ぐ建物間隔 s_{req} の簡易式を導いた(式16)。他の確率でも同様に導くことができる。

6 SPD則に基づく簡易手法の応用

中・長周期建物をふまえ速度スペクトル一定を仮定する。この周期帯でNassar-Krawinkler式はNewmark-Hall変位一定則と類似し、弾塑性変位と弾性変位が等しいと仮定できる。これを用い、低層側建物頂部の弾性変位をスペクトルから求め、その割合として弾塑性の隣接建物の相対変位が簡易に表される(式20)。これを用い、隣接建物における相対変位の値や傾向の理解を促す図10も作成できた。

7 結論

- (1) 多数の数値実験により、SPD則と弾性スペクトルを併用する簡易手法の精度が統計的に実証された。これによる予測と厳密解の比の平均値は、ほぼ1であり、標準偏差も低い。
- (2) SPD則に基づく手法のみが、相対変位問題の重要因子(2.2節)の影響を明確に表すことができ、また、その適用は易しい。
- (3) ABS則に基づく手法は合理的でなく、誤差も大きい。SRSS則は、建物履歴減衰が小さい小地震で精度が高いが、大地震時では大きな誤差をもつ。両者とも誤差傾向が一貫しない。

なお、高い付加減衰、周期・降伏力ばらつき、多質点系の変形モード、両建物の入力位相差などの影響が、今後の課題である。

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