Influence of Isolation Gap Size on the Collapse Performance of Seismically Base-Isolated Buildings

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Pounding with the retaining walls forms a potential risk of degrading the performance of seismically base-isolated buildings subjected to strong, especially near-fault, earthquake ground motions. Incremental dynamic analysis is employed to generate the so-called ‘gap graph’, in which two characteristic gap sizes of a base-isolated building are related with the isolation period of the building and the strength of the superstructure. The gap graph can be used to evaluate the required gap size for a base-isolated building to have certain collapse performance. This paper examines the interdependent relations of gap size with other factors that influence the seismic performance of the base-isolated building. Specifically, it is observed from the analysis results that near-fault pulse-like ground motions are likely to impose much higher demand for the isolation gap than far-field ones.

INTRODUCTION

Modern base isolation technique has been widely applied to residential and office buildings in Japan ever since the devastating 1995 M6.9 Kobe earthquake. During the same period of time, the technique has evolved and allows more flexible isolation layers so that the strength demand for superstructure can be further decreased. On the other hand, worries about the safety of base-isolated buildings subjected to strong earthquakes, especially near-fault motions, have arisen because a considerable part of them are located near major active faults. It is obvious that strong pulse-like near-fault motions are capable of subjecting flexible buildings to much greater displacement demand than expected (Hall et al., 1995).

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which may subject the superstructure to impact against the surround retaining walls.

![Figure 1. Statistics of isolation gap width of 979 seismically base-isolated buildings in Japan: (a) isolation gap width, $\delta_{\text{gap}}$ and (b) ratio of isolation gap width to maximum displacement, $\delta_{\text{gap}}/\delta_{\text{max}}$.](image)

Even if incidents of significant pounding between base-isolated buildings and their retaining walls were not reported so far, Nagarajaiah and Sun (2001) presented a particular case that the base-isolated FCC building impacted against its entry bridge in the 1994 Northridge earthquake. Although it was attributed to the incorrect construction, rather than unexpected strong ground motion or inadequate isolation gap, this incident raised questions about the proportioning of the isolation gap, which is the most significant design parameter that addresses the risk of pounding, and it seems that rational methods have not yet been developed and implemented. Gap size, $\delta_{\text{gap}}$, is generally considered adequate as long as it is no less than the total maximum displacement at isolation layer, $\delta_{\text{max}}$, which is usually determined through nonlinear time history analysis or equivalent linear analysis of the base-isolated building of interest. In design practice, engineers are left quite arbitrary rather than rational in dealing with such a criterion. Figure 1 plots the isolation gap size of 979 base-isolated buildings constructed in Japan from 1995 to 2012. It seems that 60 cm gap size is gradually becoming an empirical standard regardless of $\delta_{\text{max}}$ as long as $\delta_{\text{gap}} \geq \delta_{\text{max}}$ is satisfied. On the other hand, the decision of $\delta_{\text{gap}} / \delta_{\text{max}}$ ratio is subject to significant uncertainty and the engineers, probably encouraged by the aforementioned fact, tend to select smaller $\delta_{\text{gap}} / \delta_{\text{max}}$ ratio in recent years, without evaluating the risk of pounding at base mat in future earthquakes.

A significant effect of the impact between the base mat of the superstructure and its surrounding retaining walls is that higher vibration mode of the superstructure will be excited
by the traveling of the impact energy along the height of the superstructure, resulting in much
greater acceleration response and thus higher shear demand for the superstructure. This effect
and the influence of various parameters, such as type of isolators, isolation gap width,
retaining wall stiffness and impact model, has been investigated through numerical studies
with uniform elastic shear beam models (Malhotra (1997)) or lumped mass elastic shear
spring models (Matsagar and Jangid (2003), Komodromos et al. (2007)).

The higher than expected shear demand is very likely to push the superstructure of a
base-isolated building far beyond its elastic limit. This may even eliminate the advantage of
base-isolated buildings over conventional fixed base ones (Hall et al., 1995). It is worth
noting that the superstructure of a base-isolated building is elastically designed and is not
seismically detailed for ductility from the very beginning of the development of modern base
isolation technique in Japan. The exceedance of elastic limit may immediately trigger
collapse of the superstructure. It is also significant to notice that the inelastic deformation of
the superstructure tends to concentrate at the lower stories when the superstructure is
subjected to pounding at its base mat (Tsai, 1997). This observation is confirmed by Pant and
Wijeyewickrema (2012) in their analysis with more sophisticated inelastic frame model and
also by Wang et al. (2006) in their hybrid test on an 8-story base-isolated building model.
Such soft story behavior is unfavorable for the seismic performance of base-isolated
buildings.

In this paper, the influence of gap size on the collapse performance of base-isolated
multistory buildings is evaluated through extensive incremental dynamic analysis. Two
characteristic gap sizes of a base-isolated building are defined based on the analysis results.
Their dependence on various structural parameters as well as ground motion characteristics is
investigated by means of the so-called ‘gap graph’, which can itself assist in choosing the gap
size in the design practice of base-isolated buildings.

**COLLAPSE PERFORMANCE OF BASE-ISOLATED BUILDINGS**

Following the proposed methodology of determining the collapse performance of
building structures in FEMA P695 (FEMA, 2009), the incremental dynamic analysis (IDA)
suggested by Vamvatsikos and Cornell (2002, 2004) is used to evaluate the probability of exceeding an assumed collapse limit-state for base-isolated buildings. In IDA, the seismic response of a building to a set of earthquake ground motions, which are scaled incrementally by an intensity measure (IM), is calculated through nonlinear dynamic analysis, and the fragility curve which relates IM to the probability of exceedance can be obtained. The IM that produces 50% probability of exceedance is then the median collapse capacity of the building corresponding to the assumed collapse limit-state. In order to allow for parametric study, a simple two degrees-of-freedom (DOF) lumped mass model is adopted in this study. The two DOFs are connected by two springs, one representing the isolation layer and the other the superstructure. The use of a single DOF to represent the superstructure imposes a major limitation that the effect of higher vibration mode, a major consequence of impact in base-isolated buildings, cannot be included in this study. However, a single DOF model is adequate to simulate the inelastic behavior of a multistory building exhibiting soft story mechanism, which is, as mentioned above, most common in base-isolated buildings subjected to pounding at the base.

The ductility factor $\mu$ of the superstructure, which is the ratio of the maximum to the yield lateral drift of the superstructure, is used to define the collapse limit-state. Two extreme cases for the ductility capacity of superstructures are considered here: (1) non-ductile case, in which the superstructure is assumed to collapse as long as its strength is reached, that is, when $\mu = 1$, and (2) ductile case, in which the superstructure is assumed to be seismically detailed to sustain very large plastic deformation. There is not yet consensus on ultimate ductility of structures within the research community, partly because of the various definitions of yield and ultimate displacement. For simplicity, $\mu = 8$ is taken herein as the collapse limit-state criterion for ductile RC superstructures. As suggested by Priestley et al. (2007), the yield drift of RC moment-resisting frames is of the order of 1/100. As a result, ductility of eight would result in 1/12.5 ultimate story drift, which is usually large enough to cause dynamic instability for RC structures.

It is worth noting that non-ductile superstructure is very common in the practical design of seismically isolated buildings in Japan, although it has long been excluded by related laws
in Japan for fixed-base earthquake-resistant buildings.

ANALYSIS MODEL

The 2DOF model is illustrated in Figure 1. The lower lumped mass, \( m_0 \), represents the mass of the base and the upper one, \( m_1 \), represents that of the rest of the superstructure. The superstructure is assumed to exhibit trilinear peak-oriented hysteretic behavior as shown in Figure 2(a), where \( F \) and \( \Delta \) are the shear force and lateral deformation of the superstructure, respectively. The hysteretic rules follow the suggestion of Takeda et al. (1970).

![Figure 1. Lumped mass shear model](image)

The isolation layer is assumed to consist of natural rubber bearings (NRBs) and hysteretic dampers. The trilinear elastic model suggested by Nakazawa et al. (2011) is adopted for modeling NRB (see Figure 2(b), where \( F \) and \( \varepsilon \) are the shear force and the shear strain of the rubber bearings, respectively). By calibrating with existing test data on NRBs subjected to large shear strain, they proposed that the bearing begins to exhibit strain hardening at 250% shear strain and the tangent stiffness becomes 2 times the initial stiffness, \( k_i \), and then it further increases to 7\( k_i \) beyond 350% shear strain. Furthermore, the bearing fractures at 450% shear strain. The post-fracture behavior of the bearing is not modeled in this study. Instead, a large stiffness is assigned to the bearing when it exceeds 450% shear strain. The hysteretic dampers are modeled by an elastic-perfectly plastic spring and the retaining wall by a linear elastic spring with stiffness \( k_w \) and initial gaps on both sides of the origin (see Figure 2(c), where \( F \) and \( \Delta \) are the reaction force of the retaining wall and the deformation of the isolation layer, respectively). The natural vibration period of the fixed-base superstructure, \( T_i = 2\pi\sqrt{(m_1/K_0)} \), is assumed to be 0.3 s, representing typical short period multi-story RC buildings. The inherent damping of the building is assumed 2% of the critical damping. Among all the other parameters to fully define the structure, the following
five are to be investigated.

![Figure 2](image_url)

**Figure 2.** Hysteretic models for the base-isolated building: (a) superstructure, (b) natural rubber bearing and (c) retaining wall.

(1) The isolation period, $T_i$, which is defined as the period of the isolator (the NBR) and superstructure system, and is calculated by Equation (1). It neglects the stiffness of the dampers and assumes the superstructure to be rigid. It is considered one of the most important parameters in evaluating the effectiveness of seismic base-isolation. It is commonly higher than 3.5 s in the design practice in Japan (Pan et al., 2005).

$$T_i = 2\pi \sqrt{\frac{m}{k_i}}$$

where $m = m_0 + m_1$, is the total mass of the building and $k_i$ is the initial stiffness of the isolators.

(2) The base shear coefficient, $C_0$, which is the ratio of the shear strength to the weight of the superstructure, that is, $C_0 = F_y / (m_1g)$, where $F_y$ is the shear strength (see Figure 2(a)) and $g$ is the gravity acceleration. The required $C_0$ for multistory fixed-base RC buildings is generally greater than 0.3 in Japan. However, much smaller $C_0$ has been frequently used for the superstructure of base-isolated buildings. As reported by AIJ (2001), over 90% of the base-isolated buildings by the year of 2000 were designed with $C_0$ equal to or less than 0.15.

(3) The damper strength ratio, $\alpha_s$, which is the ratio of the total yield strength of the dampers to the total weight of the structure. It is usually in the range of 0.03 to 0.05 (Pan et al., 2005).

(4) The retaining wall period, $T_w$, which is the period of the retaining wall and superstructure system, assuming the superstructure is rigid and neglecting the mass of the retaining wall. Similar to the isolation period $T_i$, it is calculated by Equation (2). It is merely a
measure of the retaining wall stiffness, which is believed important for the impact response of the base-isolated building.

\[ T_w = 2\pi \sqrt{\frac{m}{k_w}} \]  

(2)

where \( m = m_0 + m_1 \), is the total mass of the building and \( k_w \) is the stiffness of the retaining wall.

(5) The total rubber thickness of NRB, \( n_t R \), which is directly related to the limit deformations at which the bearings begin to exhibit strain hardening and to fracture. \( n \) is the number of rubber layers and \( t_R \) is the thickness of a single rubber layer. The common value of \( n_t R \) is in the range of 120mm to 250mm in the engineering practice in Japan.

There are still several other parameters that cannot be derived from the above five. Among them, the gap size is the major parameter to be discussed later and some assumptions are made upon the others. For the trilinear hysteresis of the superstructure, it is assumed that the cracking strength, \( F_c \), is one third of the yield strength, \( F_y \), that the secant stiffness at the yield point is 0.3 times the initial stiffness, that the post yield stiffness ratio is 0.001 and that the unloading stiffness ratio that was defined by Takeda et al. (1970) is 0.4. In addition, the base mass, \( m_0 \), is assumed one fifth of \( m_1 \). The yield deformation of the hysteretic dampers is assumed to be 30 mm, a small value as compared to the expected displacement of the isolation layer.

**GROUND MOTION RECORDS**

Two ground motion record sets, a far-field and a near-field set, are composed for the incremental dynamic analysis. All the records are selected from the PEER ground motion database (PEER, 2012) and rules similar to those used in FEMA P695 report (FEMA, 2009) are followed in selecting the records except that (1) the valid frequency content is limited to at least 6 s to be applicable for the evaluation of base-isolated buildings with very long period of vibration and (2) the maximum number of records per event is increased to three to allow for more records for the analysis. In addition, the records in the far-field set should contain no pulse in both the two horizontal components while those in the near-field set should contain a pulse in at least one or both the two horizontal components and only the component
containing a pulse is used in the analysis. Whether a record component contains a pulse or not is based on the judgment of the latest version of the PEER database. As a result, 18 records with 36 non-pulse-like horizontal components and 22 records with 31 pulse-like horizontal components are selected for the far-field and the near-field set, respectively. Details of these records are given in the appendix.

As the analysis model is two dimensional, the two components of each record are normalized and used independently. Individual components are normalized by their respective peak ground velocities (PGVs) to the median of all their PGVs. The two sets are normalized separately. It is obvious from the median velocity spectra \( S_v \) in Figure 3 that the medium- and long-period component of the near-field set is much more significant than the ones of the far-field set while the short-period components of the two sets are close to each other after normalization.

In IDA, the normalized set of ground motions is collectively scaled by some intensity measure (IM). The energy equivalent velocity at the isolation period, \( V_E(T_i) \), is adopted herein as the IM. It is the velocity which makes the kinematic energy of the total mass of a system of natural period \( T_i \) equals the input energy, \( E_i \), to the system under the ground motion. It has been widely used in Japan as a basis for the design of base-isolated buildings (AIJ, 2000).

The input energy, \( E_i \), is calculated by Equation (3) (Akiyama, 1985), which is also referred to as the ‘relative input energy’ because it neglects the effort of the rigid body motion
of the building (Uang and Bertero, 1988). 10% damping ratio is assumed in calculating the input energy.

\[ E_I = \int ma_g v dt \]  

(3)

where \( a_g \) is the ground acceleration and \( v \) is the relative velocity of the mass.

The input energy, \( E_I \), can be easily converted to \( V_E \) by Equation (4). One of the advantages of using \( V_E \) as the IM instead of \( E_I \) is that it has the unit of velocity and thus keeps linearly proportional to other commonly used IMs such as the spectral acceleration.

\[ V_E = \sqrt{\frac{2E_I}{m}} \]  

(4)

Figure 4 compares the median \( V_E \) spectra of the two ground motion sets after being scaled to \( V_E(4 \text{ s}) = 165.4 \text{ cm/s} \). The selection of this velocity value will be explained later. The spectra within the range of period greater than about 2.5 s are very close to each other while the spectrum of the far-field set is much greater than that of the near-field set for shorter period.

![Figure 4. Median \( V_E \) spectra of the two sets scaled to \( V_E(4 \text{ s}) = 165.4 \text{ cm/s} \) (10% damping)](image)

**EVALUATION OF MEDIAN COLLAPSE CAPACITY**

In IDA, each normalized ground motion is scaled starting from a scale factor of 0.1, and the “hunt & fill” algorithm (Vamvatsikos and Cornell, 2002) is adopted to search for the scale factor with which the assumed collapse limit-state (expressed by the ductility factor of the superstructure, \( \mu \), herein) is about to be reached. The required resolution for this target scale factor, that is, the difference between the target scale factor and the consequential one which
causes the building to exceed the limit-state, is set to be 0.05. The thus-obtained scale factor can then be converted to the intensity measure $V_E(T_i)$ by simply having it multiplied by the median $V_E(T_i)$ of the normalized, but un-scaled, ground motion set.

Thus $V_E(T_i)$ at the assumed limit-state can be found for each ground motions in the set. By assuming a log-normal distribution for $V_E(T_i)$ of all the ground motions, the $V_E(T_i)$ corresponding to 50% probability of exceeding the collapse limit-state can be found and is taken as the median collapse capacity, $V_{EC}$. Figure 5 demonstrates the above process of evaluating the capacity, which shows the IDA curves of the far-field set for ductile superstructure. The hollow box on each IDA curve indicates the target point obtained by the ‘hunt & fill’ algorithm. The ductile median collapse capacity, $V_{EC}$, indicated by the solid circle is read from the lognormal distribution curve at 50% probability of exceedance.

![Lognormal dist.](image)

**Figure 5.** Median collapse capacity of ductile base-isolated building with $C_0 = 0.15$, $T_i = 4$ s, $\alpha_s = 0.04$, $T_w = T_f$, $n_{tr} = 200$ mm and gap size = 500 mm under far-field set.

**CHARACTERISTIC GAP SIZES**

**RELATIONSHIP OF GAP SIZE VERSUS MEDIAN COLLAPSE CAPACITY**

A change in any of the structural parameters described above will have more or less an influence on $V_{EC}$ of a base-isolated building. Particularly, the influence of the gap size is of great interest of the present paper. It is expected that the increase of gap size would substantially increase the collapse capacity by reducing both the possibility of impact with the retaining wall and the impact velocity, if the impact does occur. To give an example, Figure 6 depicts the relationship of the gap size versus the median collapse capacity of base-isolated buildings of $C_0$ from 0.1 to 0.35, $T_i = 4$ s, $\alpha_s = 0.04$, $T_w = T_f$, $n_{tr} = 200$ mm,
subject to the far-field set ground motions. The results for both non-ductile and ductile cases are demonstrated. Almost linear relationship between the gap size and $V_{EC}$ can be observed until a certain threshold size, beyond which the gap size has no more influence on the capacity. It is seen that $C_0$ has a great influence on the value of the threshold gap size. It has also significant effect on $V_{EC}$ of ductile system when the gap size is less than the threshold. It is, however, quite reverse for the non-ductile cases.

![Figure 6. Influence of gap size on limit-state capacity ($T_i = 4$ s, $\alpha_s = 0.04$, $T_w = T_f$, $n_t = 200$ mm, far-field set): (a) non-ductile cases and (b) ductile cases](image)

**CHARACTERISTIC GAP SIZES**

Based on the above observation, the gap size versus median collapse capacity relationship obtained by the analysis is fitted into two straight lines; a horizontal line representing the portion when the gap size has no more influence and an ascending one representing the rest portion. The intersection of these two straight lines is referred to as the maximum gap size, $\delta_{max}$, hereafter. If a base-isolated building is provided with an isolation gap wider than $\delta_{max}$, its collapse would have nothing to do with the collision with the retaining wall.

$\delta_{max}$ is sometimes too great to be practical for some applications because of the limited site area or some other economic considerations. In such cases, isolation gaps narrower than $\delta_{max}$ should be allowed and hence the impact with the retaining walls should be considered a source of deterioration of the performance of the base-isolated building in the design. However, the collapse performance should by no means be degraded to be worse than some minimum requirement, for example, the median collapse capacity, $V_{EC}$, should by no means be less than some minimum energy-equivalent velocity demand, $V_{ED}$. According to this, the gap size corresponding to $V_{ED}$ on the fitted straight line is referred to as the minimum
required gap size, $\delta_{\text{min}}$ (Figure 7(a)). It represents a criterion where the collapse margin ratio is unity, that is, $V_{\text{EC}}$ equals $V_{\text{ED}}$.

As far as the collapse performance is concerned, the minimum demand, $V_{\text{ED}}$, can be determined based on the Level II, that is, the maximum considered intensity level, energy spectrum suggested by BCJ (2006) for buildings on firm site (soil type II). The suggested energy spectrum is expressed in terms of energy-equivalent velocity and is obtained by simply dividing the design spectral acceleration, which is stipulated in the Building Standard Law of Japan (BCJ, 2004), by the circular vibration frequency of the building. Because the stipulated design spectral acceleration in the medium- and long-period range is proportional to the vibration frequency, the corresponding energy spectrum is constant in the medium- and long-period range. For firm sites (soil type II), it is constant at 165.4 cm/s for periods greater than 0.87 s.

**Figure 7.** Characteristic gap sizes of base-isolated building: (a) when $\delta_{\text{min}}$ is less than $\delta_{\text{max}}$ and (b) when $\delta_{\text{min}}$ is greater than $\delta_{\text{max}}$

By their physical meaning, the maximum gap size, $\delta_{\text{max}}$, should always be no less than $\delta_{\text{min}}$. However, it is possible that $\delta_{\text{min}}$ goes even greater than $\delta_{\text{max}}$ if it is determined from the fitted straight lines of the analysis results, no matter if from the part within the data range or from that beyond it (Figure 7(b)). This simply means that the studied base-isolated building cannot meet the requirement even if it has infinite isolation gap, or in other words, even if pounding is not taken into account. In this case, the design of the base-isolated building itself, rather than the selection of appropriate isolation gap size, should be carefully revised. In other cases, the two characteristic gap sizes, $\delta_{\text{max}}$ and $\delta_{\text{min}}$, present the upper and lower bound for appropriate gap size. It is not economical if the gap size exceeds $\delta_{\text{max}}$; on the other hand, it is not safe if the gap size is less than $\delta_{\text{min}}$. In practical design, the lower bound, $\delta_{\text{min}}$, is usually more useful. It ensures that isolated buildings sustain uniform collapse performance (in terms of energy-equivalent velocity) like other types of buildings even if major earthquakes subject isolated buildings to pounding incidents.
GAP GRAPH

FORMATION

The two characteristic gap sizes, $\delta_{\text{max}}$ and $\delta_{\text{min}}$, and two of the major influential parameters, the isolation period, $T_i$, and the base shear coefficient, $C_0$, are integrated into a single graph, referred to as the ‘gap graph’. It is a convenient tool in studying the interdependence of the two characteristic gap sizes and various parameters. The horizontal and the vertical axis of a gap graph are $\delta_{\text{min}}$ and $\delta_{\text{max}}$, respectively. A pair of $\delta_{\text{min}}$ and $\delta_{\text{max}}$ forms a data point on the graph. Such points corresponding to base-isolated buildings of various $C_0$ and $T_i$ are plotted and connected to form a net. A straight line starting from the origin representing $\delta_{\text{min}} = \delta_{\text{max}}$ is also plotted on the graph. Any data point falling below this line, that is, $\delta_{\text{min}}$ is greater than $\delta_{\text{max}}$, indicates that the design of the base-isolated building requires revision. Figure 8 gives an example of the gap graph for base-isolated buildings with ductile superstructures and $\alpha_s = 0.04$, $T_w = T_i$, $nt_R = 200$ mm, subjected to the far-field ground motion set. The point of $C_0 = 0.1$, $T_i = 3.5$ s and $T_i = 4.0$ s falls below the diagonal line of $\delta_{\text{min}} = \delta_{\text{max}}$, indicating that these combinations of $C_0$ and $T_i$ are not adequate to meet the capacity demand even if the isolation gap is infinite. Both $\delta_{\text{min}}$ and $\delta_{\text{min}}$ generally increase with the increase of the isolation period $T_i$, indicating that wider gaps should be provided for more flexible isolation systems. On the other hand, $\delta_{\text{max}}$ increases with the increase of $C_0$ while $\delta_{\text{min}}$, on the contrary, decreases rapidly with it. This indicates that the disadvantage of using narrower isolation gap can be compensated by increasing the strength of the superstructure as far as the superstructure is ductile. To be noted that it is quite reverse if the non-ductile cases are concerned. This will be shown later.
Figure 8. Gap graph of ductile base-isolated buildings with \( \alpha_s = 0.04, T_w = T_f \) under far-field set

GAP GRAPHS WITH VARIOUS AMOUNTS OF DAMPERS

The gap graphs may serve as guidance for engineers in determining the size of the isolation gap of a base-isolated building in their design. A spectrum of gap graphs covering major parameters of a base-isolated system, for both non-ductile and ductile cases and subjected to both far-field and near-field ground motion sets, is provided in Figure 9. It is worth emphasizing that these results should only be used as only rough estimates and with full awareness of the assumptions and limitations upon which they are obtained. As the same as in Figure 8, each of the gap graphs covers the range of \( C_0 = 0.10 \) to 0.35 from bottom to upper part and \( T_i = 3.5 \text{ s} \) to 6 s from left to right.

Besides \( C_0 \) and \( T_i \), which are inherently included in a gap graph, the structural parameter recognized in Figure 9 is the amount of hysteretic dampers in the isolation layer in terms of \( \alpha_s \). The other important parameters, \( T_w \) and \( nt_R \), are fixed at \( T_w = T_f \) and \( nt_R = 200 \text{ mm} \) in Figure 9. Their effect on the gap graph will be discussed later. The following observations can be made:

(1) As already shown in Figure 8, \( \delta_{\text{min}} \) decreases rapidly with stronger superstructures, that is, greater \( C_0 \) when the superstructure is ductile. On the contrary, \( C_0 \) has little effect in decreasing \( \delta_{\text{min}} \) for non-ductile cases even if the superstructure is as strong as a fixed-based building, for example, \( C_0 = 0.3 \). This is true for buildings of various \( \alpha_s \) and various \( T_i \) subjected to either far-field or near-field ground motions.

(2) Greater \( T_i \) requires greater \( \delta_{\text{min}} \) for both the non-ductile and the ductile cases. In the
practical design of base-isolated buildings, greater $T_i$, which indicates a more flexible isolation layer, is usually favorable because it generally leads to lower strength demand for the superstructure and thus makes the design more economically attractive. However, the increased flexibility of the isolation layer and the consequent potential decrease in the strength of the superstructure impose much greater demand for the isolation gap by increasing $d_{\text{min}}$. In such a manner, the flexibility of the isolator and the size of the isolation gap may form kind of trade-off in the design of a base-isolated building.

(3) The increase of the amount of dampers in the isolation layer, or in other words, the value of $\alpha_s$, may help reducing $\delta_{\text{min}}$. For example, for a building of $C_0 = 0.15$, $T_i = 5$ s and subject to far-field motions, the corresponding points on the gap graphs are marked by hollow circles in Figure 9. The value of $\delta_{\text{min}}$ corresponding to non-ductile cases decreases from 559 mm when $\alpha_s = 0.02$, to 472 mm when $\alpha_s = 0.04$ and further to 439 mm when $\alpha_s = 0.06$. Similarly, $\delta_{\text{min}}$ corresponding to ductile cases decreases from 489 mm when $\alpha_s = 0.02$, to 419 mm when $\alpha_s = 0.04$ and further to 380 mm when $\alpha_s = 0.06$. To be noted that greater $\alpha_s$ would increase the strength demand for the superstructure. Again, it is a trade-off for the selection of the value of $\alpha_s$. 
Figure 9. Gap graphs of base-isolated buildings with various amount of dampers subjected to either far-field or near-field motions \( (T_w = T_i, n_{TR} = 200\text{mm}) \): (a) non-ductile cases and (b) ductile cases.

(4) As mentioned above and as can be seen in Figure 4, the median \( V_E \) spectrum of the near-field set is generally smaller than that of the far-field set at the period equal to or less than \( T_i \) when they are scaled to the same IM. Notwithstanding this inferiority in response spectrum, the near-field set imposes much greater demand for \( \delta_{\text{min}} \) than the far-field set. As far as the far-field set is concerned, 600 mm gap size, which is generally required and actually adopted in most base-isolated buildings in Japan, would be large enough to exceed \( \delta_{\text{min}} \) of base-isolated buildings with moderate amount of dampers. However, it frequently becomes inadequate when the near-field set is used, especially for buildings of greater \( T_i \) and smaller \( C_0 \).

**INFLUENCE OF RETAINING WALL STIFFNESS**

The stiffness of the retaining wall comes from not only the retaining wall itself, which is usually made of reinforced concrete, but always from the soil backfill, too. A retaining wall with stiffness that makes \( T_w/T_i = 1 \) represents a quite practical case in the construction...
practice of base-isolation system, although much smaller values of $T_w/T_f$ ratio have been adopted by previous researchers. For example, it was assumed $T_w/T_f = 0.29$ by Tsai et al. (1997) and $T_w/T_f$ in the range of 0.3–1.3 was investigated by Komodromos et al. (2007). To give a realistic example, an impact test was carried out on a real 5-story base-isolated reinforced concrete building in Japan before being demolished (Sano et al., 2010). The height of the building, $h$, is 21.85 m above the ground. The superstructure was about 2576 ton in weight and was isolated by 14 natural rubber bearings. The fixed-base period, $T_f$, of the building can be estimated empirically to be $0.02h$, that is, 0.437 s according to AIJ (2004). The reinforced concrete retaining wall is 200 mm thick and 1.2 m high. In the test, the superstructure was pulled back a certain distance by jacks and released suddenly to collide with the retaining wall. The measured stiffness of the retaining wall was 104.6 kN/mm, leading to a retaining wall period $T_w = 0.986$ s. Thus, the $T_w/T_f$ ratio of this building is 2.2. Since this kind of test data is very limited and the stiffness of retaining wall is usually not considered in the design of base-isolated buildings, it seems appropriate to safely assume higher stiffness of retaining walls when considering the potential risk of pounding.

While the stiffness of retaining wall has much less effect on $d_{max}$, it would substantially decrease $d_{min}$ when it becomes much lower. This effect is especially significant for stronger and ductile superstructures, as can be seen in Figure 10. For a building of $C_0 = 0.15$, $T_i = 4s$ and subject to far-field ground motions, $d_{min}$ corresponding to non-ductile cases remains almost the same even if the retaining wall stiffness is reduced to one sixteenth (i.e., from $T_w/T_f = 1$ to $T_w/T_f = 4$). $d_{min}$ corresponding to ductile cases decreases from 406 mm at $T_w/T_f = 0.5$, to 386 mm at $T_w/T_f = 1$, 369 mm at $T_w/T_f = 2$ and 348 mm to $T_w/T_f = 4$. This difference becomes much more significant for buildings with stronger superstructures.

Although the cases of $T_w/T_f = 8$ seem unrealistically soft for an RC retaining wall, they are also depicted in Figure 10 to suggest the possibility of using some soft materials between the base of the superstructure and the retaining wall to allow a narrower isolation gap.
INFLUENCE OF RUBBER THICKNESS

The total rubber thickness, $n_{tr}$, may also have some effect on the gap graph because it represents the limit deformation at which the bearing begins to exhibit strain hardening. The analysis results show that it has little effect on $\delta_{\text{min}}$ while it does have considerable influence on $\delta_{\text{max}}$ (Figure 11).

The effect of $n_{tr}$ on $\delta_{\text{max}}$ can be easily estimated by a simple model without conducting any dynamic analysis. As already mentioned before, $\delta_{\text{max}}$ has the physical meaning that the failure of the superstructure would have nothing to do with the pounding if the gap size is greater than it. Instead, it should be due to the excessive shear force that is carried by the isolation layer and is transmitted to the superstructure when the rubber bearings are subjected to large deformation. Thus, it is natural to look for a shear deformation of the isolation layer, at which the shear force of the isolation layer equals the strength of the superstructure, and to make this deformation an estimation for $\delta_{\text{max}}$ of the building. This process is demonstrated in Figure 12 and the thus-obtained deformation is denoted as $\delta_{\text{cal}}$. This method can be easily carried out as long as the restoring force versus deformation relationship of the isolation layer
is known. In this method, the influence of $\alpha_s$, $nt_R$ and $C_0$ on $\delta_{\text{max}}$ is taken into account. $\delta_{\text{cal}}$ obtained by this method and $\delta_{\text{max}}$ by the above analysis for buildings of various properties subject to both far-field and near-field motions are compared in Figure 13. For non-ductile cases, $\delta_{\text{cal}}$ is practically equal to $\delta_{\text{max}}$ for all the cases. For ductile cases, however, $\delta_{\text{cal}}$ is generally less than $\delta_{\text{max}}$. It is probably due to the effect of the energy dissipation of the superstructure, which is not considered in the simple method of obtaining $\delta_{\text{cal}}$. As shown in Figure 13(b), $(\delta_{\text{cal}} + 300 \text{ mm})$ provides a practical estimate for the upper bound of the $\delta_{\text{max}}$.

Figure 11. Influence of rubber thickness on gap graphs ($\alpha_s = 0.04$, $T_w = T_t$, Far-field set): (a) non-ductile limit-state ($\mu = 1$) and (b) ductile limit-state ($\mu = 8$)

Figure 12. Model of calculating $\delta_{\text{cal}}$ as an estimate of $\delta_{\text{max}}$

Figure 13. Relationship of $\delta_{\text{max}}$ versus $\delta_{\text{cal}}$: (a) non-ductile limit-state and (b) ductile limit-state
CONCLUSIONS

Two characteristic gap sizes, $\delta_{\text{max}}$ and $\delta_{\text{min}}$, are defined in examining the influence of gap size on the collapse performance of seismically base-isolated buildings. By forming a gap graph, the dependence of these two characteristic gap sizes on major structural parameters of base-isolated buildings and ground motion characteristics is investigated. The following conclusions may be drawn from the above results and discussions.

(1) For isolated buildings with non-ductile superstructures, the demand for isolation gap size in terms of $\delta_{\text{min}}$ can be reduced either by increasing the amount of dampers in the isolation layers or by reducing significantly the stiffness of the retaining wall. At the same time, this demand would increase rapidly when the isolation layer gets more flexible. In addition, it seems not effective to reduce this demand by increasing the strength of the superstructure.

(2) For isolated buildings with ductile superstructures, the influence of major structural parameters on the demand for isolation gap size in terms of $\delta_{\text{min}}$ is similar to that for the non-ductile cases except that the increase of the superstructure strength would substantially reduce the demand for isolation gap size.

(3) The near-fault pulse-like ground motions may produce much greater $\delta_{\text{min}}$ than the far-field ones. For flexible isolation system, for example, $T_i$ equals 5.5 s or even 6 s, a gap as wide as 600 mm, which is currently a common practice in Japan, may still fail to meet the demand in terms of $\delta_{\text{min}}$. This suggests a potential risk for the many base-isolated buildings located close to major active faults.

(4) The maximum gap size, $\delta_{\text{max}}$, represents a threshold whether the performance of the base-isolated building is influenced by pounding. It can be satisfactorily estimated by the proposed simple method for non-ductile cases.

It is worth noting that the above conclusions are based on analytical models with single-degree-of-freedom system as the superstructures. Accelerations of the superstructure, which are also of major concern for the seismic performance of base-isolated buildings, and which may be greatly influenced by impact of the isolation layer, are not discussed in this
paper. Although similar ideas could be followed to produce gap graphs corresponding to some acceleration limit-states, much more computational effort might be required because multi-degree-of-freedom models might become necessary for the superstructure to capture the higher vibrations mode.

**REFERENCE**


## Appendix:

### Table A. Ground motion records in near-field set

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<thead>
<tr>
<th>ID</th>
<th>Earthquake Name</th>
<th>Mechanism</th>
<th>Fault Distance (km)</th>
<th>Shear wave velocity Vs30 (m/s)</th>
<th>Lowest usable frequency (Hz)</th>
<th>PGA (g)</th>
<th>PGV (cm/s)</th>
<th>PGA (g)</th>
<th>PGV (cm/s)</th>
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Shadowed components are excluded in the selection because they contain no pulse.
Table B. Ground motion records in far-field set

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<th>Earthquake Name</th>
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