ABSTRACT

This study aims to evaluate seismic performance of base-isolated buildings with insufficient seismic gaps that do not conform to minimum codified gap size requirements. A wide range of isolated and fixed-base elastic superstructures has been subjected to analytical near-field pulse-type motions to evaluate seismic demands in the superstructure during impact with moat/stop walls. Different values for stiffness of moat walls has been considered adopting a stereomechanical impact model for a mass-spring-dashpot representation of the moat walls. The results of the parametric study show that the developed seismic demands in the superstructure depends on the pulse duration, gap size, and the ratio of the isolated period to the fixed-base period. This study implies that by using available gap sizes, and utilizing lower cost isolation devices with a smaller displacement capacity compared with that of the current code-based design requirements, a more economical performance-based seismic design for base-isolated structures is achievable resulting a gradual transition from the immediate occupancy performance level to the collapse prevention one.

Keywords: Base-isolation; gap size; pounding; near-field pulse type motion; Seismic performance

1. INTRODUCTION

Increasing number of seismically isolated structures mostly in seismic-prone regions of the world has raised a concern on seismic performance of these structures in severe earthquake ground motions. On one hand, base-isolated buildings should have a clear certain gap with adjacent structures to avoid pounding, otherwise developed seismic force and displacement demands during impact with neighboring buildings or surrounding moat/stop walls may be far beyond the capacity of the superstructure. In this case, the advantage of using base-isolation technology over traditional seismic design may be totally lost. On the other hand, considering the high cost of lands and other urban limitations do not allow for large seismic gaps to completely eliminate the risk of pounding. In contrast to fixed-base buildings, lateral displacement demand is mostly concentrated at the isolation layer, and the superstructure moves laterally almost as a rigid body. Moreover, isolation hardwares should remain stable under vertical loads at their maximum horizontal displacement demand under maximum considered earthquake ground motion. This results in expensive isolation devices to accommodate safely a large displacement demand as an important design consideration. This may prevent the use of isolation technology for a wide range of residential constructions (Skinner et al., 1993).

Additionally, the structural performance level of base-isolated buildings designed in accordance with modern seismic codes has a sharp change from immediate occupancy (IO) for moderate seismic hazard levels to collapse prevention (CP) for a large seismic hazard level. Consequently, structural elements of the superstructure usually do not experience a damage-control performance level which falls between IO and life safety (LS) performance levels. Aforementioned performance objectives may not be compliant with the performance goals considered for residential buildings with a normal importance. According to the US seismic design code, ASCE/SEI 7-16 (American Society of Civil Engineers, 2017) a small response modification coefficient equal to the maximum of 2.0 and ⅜ of the value of $R$ associated with the structural system above the isolation level should be used to reduce seismic forces in the

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superstructure while all the seismic design requirements including ductile detailing of a fixed-base structural system should be satisfied for its base-isolated counterpart. This implies that a large reserved seismic capacity remains almost useless until the isolation layer is pushed to its displacement capacity which results in pounding with the moat/stop walls or engagement of lateral displacement restrainers in case of an earthquake scenario with a 2500-year return period. Within the scope of performance-based seismic engineering, a preliminary investigation has been done in this study to determine whether it is possible to design a low-cost base-isolated building with a less than code-prescribed gap size while maintaining a better seismic performance of the isolated building than its fixed-base counterparts. Strong ground motions may pose a larger displacement demand on flexible structural systems including seismically isolated structures particularly in seismic regions suspected to experience near-field pulse-like motions (Hall et al. 1995). However, due to a limited number of base-isolated structures compared with a much larger stock of conventional fixed-base constructions, more rigorous and strict design requirements of base-isolated structures and a smaller probability of exposure to near-field ground motions, reports on impact and pounding of seismically isolated buildings with moat/stop walls or adjacent structures are quite rare. Nagarajaiah and Sun (2001) studied a base-isolated fire command and control (FCC) building in Los Angeles during 1994 Northridge earthquake, and found that in the EW direction pounding of the base floor above the isolation system to the entry bridge of the building resulted in an increase in shear and drift demand. Qu et al. (2013) examined the influence of gap size on the seismic performance of base-isolated buildings by introducing characteristic gap graphs to evaluate the required gap distance to achieve a certain collapse performance. They considered the spectral velocity corresponding to a 50% probability of exceeding the collapse limit-state as a measure of capacity. It was shown that the demand for the gap size is larger for near-field pulse-type motions than that of the far-field ones. Consequently, for large isolated periods the adopted practical gap size of 600 mm in Japan may not be satisfactory for achieving a collapse performance corresponding to the constant spectral velocity of the medium to long period range in accordance with the Japanese building code. Moreover, in contrast to ductile superstructures increasing the yield strength of the superstructure may not be effective to reduce the minimum gap size required for achieving the abovementioned collapse performance for non-ductile superstructures. Masroor and Mosqueda, (2013) examined performance of seismically isolated buildings under strong ground motions including pounding against a moat wall. They found that pounding and contact forces can amplify the accelerations at all stories for gap sizes less than the code-specified gaps. Ye et al. (2009) showed that lower floors are affected by pounding induced forces more than other stories. Therefore, ductility demand of lower stories is larger than that of other stories, particularly for near-field ground motions. Mavronicola et al. (2017) showed the combination of ground motion directionality considerations, mass eccentricity effects and pounding to adjacent structures may significantly exacerbate peak inter-story drift ratios of base-isolated buildings. Therefore, rational amplification factors should be studied to address this issue. Komodromos et al., (2007) assumed that pounding occurs at the isolation level. In their numerical studies, they found that pounding generally increases floor accelerations as well as interstorey drift ratios. Moreover, in contrast to peak interstorey drifts, floor accelerations are highly sensitive to the impact stiffness. Additionally, where seismic gap distances is limited, using an isolation system with excessive flexibility may lead to pounding vulnerability which may eliminate the benefit of using the base-isolation technology to reduce seismic demands in the superstructure. Other studies by Malhotra (1997), Tsai, (1997), Dimova (2000), Matsagar and Jangid (2003), Agarwal et al. (2007) and, Pant and Wijeyewickrema (2012) showed the importance of pounding and induced contact forces in base-isolated structures. They investigated the most effective parameters like e.g., gap size, moat wall stiffness, and different assumptions for damping as well as the significance of different types of isolation systems.

2. ANALYTICAL PULSES

Research works done by Makris (1997), Makris and Chang (1998), Vassiliou et al. (2013), Gazetas et al. (2009), Bao et al. (2017) and, others show the significance of near-fault ground motions on the seismic response of base-isolated structures.
The existence of specific pulses, mainly at the beginning of the near-field motions either a single-pulse or a combination of different pulses, has led to recognition and significance of different fundamental behavior of structures when subjected to near-field and far-field earthquake ground motions. Since the evaluation of the performance of structures under near-field earthquakes is usually a complex and time-consuming task, it is common to use a series of simplified pulses to simulate near-field motions. Garini et al. (2015) examined responses of four idealized dynamic systems under different ground motions and two analytical wavelets fitted to the recorded ground motions. These four systems are: an elastic single-degree-of-freedom (SDOF) oscillator; an elastic–perfectly-plastic SDOF oscillator; a rigid block resting in simple frictional contact on a horizontal base; and a rigid block resting on a sloping plane. Results show that for two first systems response under both recorded and wavelet motions are acceptably similar, but for two latter cases, the seismic response is affected by number of cycles, and the detail of the real ground motion. Thus, for the system considered in this paper, i.e., elastic two-degree-of-freedom systems analytical wavelets can be used without significant reservation instead of using real ground motions. The analytical expressions of considered pulse-like ground motions are listed in Table 1.

### Table 1. Analytical wavelets.

<table>
<thead>
<tr>
<th>Wavelet</th>
<th>Analytical Expression</th>
<th>Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sinusoidal</td>
<td>$A \sin \left( \frac{2\pi}{T_p} t \right)$</td>
<td>$A$: amplitude</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_p$: pulse period</td>
</tr>
<tr>
<td>Küpper</td>
<td>$A \left( \sin \left( \frac{m \pi t}{T} \right) - \left( \frac{m}{m+2} \right) \sin \left( (m+2) \frac{\pi}{2} \right) \right)$</td>
<td>$A$: amplitude</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$m$: controls the number of half-cycles</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T$: duration</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_p$: pulse period</td>
</tr>
<tr>
<td>Ricker (symmetric)</td>
<td>$a_p \left( 1 - \frac{2\pi^2 t^2}{T_p^2} \right) e^{-\frac{2\pi t}{T_p}}$</td>
<td>$a_p$: amplitude</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$T_p$: pulse period</td>
</tr>
<tr>
<td>Ricker (Antisymmetric)</td>
<td>$\frac{a_p}{\beta_R} \left( \frac{4\pi^2 t^2}{3T_p^2} - 3 \right) e^{-\frac{2\pi t}{T_p}}$</td>
<td>$a_p$: amplitude</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$\beta_R$: set to 1.38 (has a maximum equal to $a_p$)</td>
</tr>
</tbody>
</table>

Five ground acceleration wavelets which were used in this study has been plotted in Figure 1.

![Figure 1](image-url)
Displacement and acceleration amplification factors, i.e., $R_d$ and $R_a$ spectra for analytical near-field pulse-like motions shown in Figure 1 were calculated and compared with their corresponding response quantities of base-isolated systems in the following sections. For a wide range of $T_p/T_s$ ratio, increase in damping results in significant reduction of amplification factors. However, for large $T_p/T_s$ ratios maximum response occurs within the pulse duration, hence, damping effects on the value of maximum response is negligible.

3. STRUCTURAL MODEL OF BASE-ISOLATED SYSTEMS INCLUDING IMPACT

A two-degree-of-freedom representation of the base-isolated structures (Kelly, 1990) depicted in Figure 3 has been widely used to study the most fundamental aspects of seismic response of seismically isolated structures. This model has been modified to include pounding effects with adjacent moat walls. The analogy has been studied with stiff and flexible superstructures in the following sections. The system parameters of this representation are defined as follows.

Figure 2. Parameters of the general two-degree-of-freedom base-isolated structure with moat/stop walls

### 3.1 Equations of Motion and System Parameters

- **Fixed-base period and cyclic frequency:**
  \[ T_s = 2\pi \sqrt{\frac{m_s}{k_s}}, \quad \omega_s = \sqrt{\frac{k_s}{m_s}} \]

- **Isolation period and cyclic frequency:**
  \[ T_b = 2\pi \sqrt{\frac{m_s + m_b}{k_b}}, \quad \omega_b = \sqrt{\frac{k_b}{m_s + m_b}} \]

- **Moat wall period and cyclic frequency:**
  \[ T_m = 2\pi \sqrt{\frac{m_m}{k_m}}, \quad \omega_m = \sqrt{\frac{k_m}{m_m}} \]

- **Cyclic frequency ratios:**
  \[ \varepsilon = \frac{\omega_s^2}{\omega_b^2}, \quad \eta = \frac{\omega_m^2}{\omega_s^2} \]

- **Mass ratios:**
  \[ \gamma_s = \frac{m_s}{m_b + m_s}, \quad \gamma_m = \frac{m_m}{m_b + m_s} \]

where $m_s$ and $m_b$ and $m_m$ are the superstructure mass, the mass of base slab above the isolation layer and moat wall mass, respectively. $k_s, k_b$ and $k_m$ are stiffnesses of the fixed-base superstructure, isolation layer and the moat wall, respectively. In this study, we assumed that $\gamma_s = 0.5$ which usually represent a light superstructure (Vassiliou et al., 2013b). Moreover, $\gamma_m$ was considered equal to 0.5 which represent the mass ratio of the moat wall and its back soil masses to the total mass of the isolated system.

The viscous damping coefficients of the superstructure, the isolation layer and the moat wall system are $c_s, c_b$ and $c_m$, respectively. Therefore, damping ratios and equations of motion are expressed as follows.
Fixed-base, isolation and moat wall damping ratios: 
\[ \zeta_i = \frac{c_i}{2m_i\omega_i}, \quad \zeta_b = \frac{c_b}{2(m_i + m_b)\omega_b}, \quad \zeta_m = \frac{c_m}{2m_m\omega_m} \]

Equation of motion before impact,
\[
\begin{bmatrix}
1 - \gamma_s & 0 \\
0 & \gamma_s
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_b \\
\ddot{u}_i
\end{bmatrix}
+ 2\omega_i
\begin{bmatrix}
\zeta_i\omega_i^2 & 0 \\
0 & \zeta_s\omega_s^2
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_i
\end{bmatrix}
+ \omega_i^2
\begin{bmatrix}
\zeta_s\omega_s^2 + \eta_s & \gamma_s \\
-\gamma_s & \gamma_s
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_i
\end{bmatrix}
= -
\begin{bmatrix}
1 - \gamma_s & 0 \\
0 & \gamma_s
\end{bmatrix}
\begin{bmatrix}
u_s \\
u_i
\end{bmatrix}
\]

Equation of motion after impact,
\[
\begin{bmatrix}
1 - \gamma_m & 0 \\
0 & \gamma_m
\end{bmatrix}
\begin{bmatrix}
\ddot{u}_b \\
\ddot{u}_i
\end{bmatrix}
+ 2\omega_i
\begin{bmatrix}
\zeta_i\omega_i^2 & 0 \\
0 & \zeta_m\omega_m^2
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_i
\end{bmatrix}
+ \omega_i^2
\begin{bmatrix}
\zeta_m\omega_m^2 + \eta_m & -\gamma_m \\
-\gamma_m & \gamma_m
\end{bmatrix}
\begin{bmatrix}
\dot{u}_b \\
\dot{u}_i
\end{bmatrix}
= -
\begin{bmatrix}
1 - \gamma_m & 0 \\
0 & \gamma_m
\end{bmatrix}
\begin{bmatrix}v_s \\
v_i
\end{bmatrix}
\]

where, \( u_s \) and \( u_b \) are superstructure and the base slab displacements with respect to the ground displacement, \( u_g \).

The force–displacement relation of a linearly elastic stop and isolators are shown in Figure 4, where \( F_m \) is the spring force, \( \text{gap} \) is the gap size, \( k_m \) is the moat wall stiffness, \( F_b \) is the isolator force and \( k_b \) is the stiffness of isolation layer.

Figure 3. Force-displacement relationship (a) equivalent linear isolator (b) equivalent linear-elastic stop

### 3.2 Impact Simulation

Seismic or shock impact, in which the collision forces appear in short time as heat and energy, are typically modeled in two general categories stereomechanics and continuous force (DesRoches and Muthukumar, 2002; Ruangrassamee and Kawashima, 2001).

To investigate the basic features of an extensive parametric study in its preliminary stages, instead of using complex impact models we adopt the most basic one, i.e., the stereomechanical approach. In this approach impact is considered to be instantaneous and it is based on the principle of conservation of energy and momentum. It uses a coefficient of restitution, \( e \), to modify the velocities of colliding bodies with masses \( m_1 \) and \( m_2 \) after impact to their velocities before impact, \( v_1 \) and \( v_2 \).

It deals with post impact velocities of colliding bodies based on the prior velocities. Suppose two colliding body velocities of then after impact their velocities can be obtained from the Equation 1, 2, and 3.

\[
v_i' = v_i - (1 + e) \frac{m_2 v_1 - m_1 v_2}{m_1 + m_2}
\]
\[ v_2' = v_2 + \left(1 + e\right) \frac{m_1 v_1 - m_1 v_2}{m_1 + m_2} \]  

\[ e = \frac{v'_1 - v'_2}{v_1 - v_2} \]

A value of \( e = 0 \) denotes a fully plastic collision. In this case, the two bodies after impact gain the same velocity and move together in the same direction. The value of \( e = 1 \) for a totally elastic one. Anagnostopoulos and Spiliopoulos, (1992) used a range of 0.5 to 0.75 for \( e \) to simulate collision between structures. Papadrakakis et al. (1991) and Jankowski et al. (1998) suggested \( e = 0.65 \) for typical concrete structures. Qu et al. (2013b) obtained \( e = 0.4 \) for steel structures by experimental study on a steel bridge girder model.

### 3.3 Damping Considerations

As the duration of contact between the base floor and the moat walls is very short after each impact, the effect of the moat wall damping in the dynamic response can be neglected. Tsai (1997) found that seismic response of base-isolated shear beams bumping against stops is insensitive to the amount of damping provided by the moat/stop wall system. Additionally, Nagarajaiah and Sun (2001) pointed out that the amount of the damping coefficient of the impact dashpot is overly unimportant in earthquake response of the FCC building which was mentioned earlier.

The damping ratio of the system attributed to the isolation layer was considered to be 0.2 and the damping ration of the superstructure was assumed to be 0.05.

### 4. RESPONSE TO ANALYTICAL PULSE MOTONS

Dynamic response of the abovementioned base-isolated structures was computed to the analytical pulse ground motions mentioned earlier in Section 2. In order to cover a wide range of systems, results of Equations 4 and 5 were classified by means of dimensional analysis. Dynamic response of the base-isolated structural model shown in Figure 2 is a function of \( T_s/T_p \), \( \text{Gap}/\text{Sd} \), \( T_m/T_s \), \( \gamma_s \), \( \gamma_m \) and damping ratios of the isolation layer and superstructure, where \( \text{Sd} \) is the spectral displacement of the fixed-base superstructure to an analytical pulse ground motion. All normalized response quantities can be expressed in terms of these dimensionless parameters. For example, the normalized superstructure acceleration demand to that of the fixed-base counterpart was plotted in Figure 4 for \( \text{Gap}/\text{Sd} = 0.3 \), \( T_m/T_s = 1.0 \), \( \zeta_1 = 0.2 \), \( \zeta_2 = 0.05 \), \( \gamma_m = 0.5 \), \( \gamma_s = 0.5 \) under different pulse excitations. It is evident that for \( T_s/T_p \) larger than 0.6, the maximum superstructure acceleration demands are less than or in the same order of their fixed-base counterparts while pounding has been taken into account in analyses.

#### 4.3.1 Effect of Moat Wall Period

Different values of \( T_m/T_s \) was adopted by previous researchers. For example, Tsai (1997) assumed that this parameter is equal to 0.29 and Komodromos et al. (2007) studied a range between 0.3-1.3 for this value. Qu et al. (2013b) adopted \( T_m/T_s \) equal to 1. Figure 5 shows the effect of \( T_m/T_s \) on the dynamic response for three different values of 0.6, 1 and 1.3. For all cases, \( T_s/T_p \) was assumed to be 0.7 and \( \text{Gap}/\text{Sd} \) equal to 0.3. It is evident that by increasing \( T_m/T_s \), acceleration will decrease, and also for a constant \( T_m/T_s \) for larger values of \( T_s/T_p \) accelerations are lower, except for the case of K"{u}pper wavelet with \( m \) equal to 3.
Figure 4. Superstructure acceleration spectra for elastic two-degree-of-freedom base-isolated structures for different $T_s/T_p$, and different analytical pulse-like excitations, $Gap/S_d = 0.3$, $T_m/T_s = 1.0$, $\zeta_1 = 0.2$, $\zeta_2 = 0.05$, $\gamma_m = 0.5$, $\gamma_s = 0.5$; (a),(b) Sinusoidal; (c),(d) Küpper $m = 2$; (e),(f) Küpper $m = 3$; (g),(h) Symmetric Ricker; (i),(j) Antisymmetric Ricker
Figure 5. Superstructure acceleration spectra for elastic two-degree-of-freedom base-isolated structures for different $T_m/T_s$ and different analytical pulse-like excitations, $Gap/S_d = 0.3$, $T_s/T_p = 0.7$, $\zeta_1 = 0.2$, $\zeta_2 = 0.05$, $\gamma_a = 0.5$, $\gamma_s = 0.5$: (a) Sinusoidal; (b) Küpper $m = 2$; (c) Küpper $m = 3$; (d) Symmetric Ricker; (e) Antisymmetric Ricker

4.3.2 Effect of Gap Size

The effect of gap size on the superstructure acceleration demand has been examined for $T_s/T_p = 0.7$ and $T_m/T_s = 1$. As shown in Figure 6, results indicate that the superstructure acceleration demand does not change tremendously with variation of the gap size and remains in an acceptable range with respect to the fixed-base superstructure acceleration demand. For all types of pulse motions considered in this
study, the ratio of superstructure acceleration demand to its fixed-base counterpart remains almost less than 1.2 for the smallest \( Gap/S_d \) which is considered equal to 0.1. Moreover, generally by increasing \( Gap/S_d \) the superstructure acceleration ratio decreases over the all range of considered \( T_p/T_b \). It is clear that by adopting a larger \( Gap/S_d \) but smaller than unity, an isolated superstructure performs better than its fixed-base counterpart even after pounding to the moat wall.

![Graphs](image)

**Figure 6.** Superstructure acceleration spectra for elastic two-degree-of-freedom base-isolated structures for different \( Gap/S_d \) and different analytical pulse-like excitations, \( T_i/T_p = 0.7 \), \( T_m/T_s = 1.0 \), \( \zeta_i = 0.2 \), \( \zeta_s = 0.05 \), \( \gamma_s = 0.5 \); (a) Sinusoidal; (b) Küpper \( m = 2 \); (c) Küpper \( m = 3 \); (d) Symmetric Ricker; (e) Antisymmetric Ricker
5. CONCLUSIONS

Dynamic response of elastic base-isolated structures with insufficient seismic gap distances to surrounding moat walls has been investigated under analytical pulse ground motions. Insufficient gap sizes are expressed in terms of the normalized gap sizes to the spectral displacement of the isolated period, i.e., $\frac{Gap}{S_d} < 1$. Pounding of the base floor slab to the moat wall was simulated with a stereomechanical impact model and a mass-spring-dashpot assembly. By means of dimensional analysis it has been shown that for a practical range of pulse motion period to the isolated period, $\frac{T_p}{T_b}$ the maximum acceleration demands of the flexible superstructures do not tremendously amplify with respect to its fixed-base counterpart for $\frac{T_s}{T_p} > 0.7$. Additionally, for more flexible superstructure, deamplification of the acceleration demand is occurred. These findings imply that by careful selection of the dynamic properties of a base-isolated structure it is possible to adopt smaller gap sizes to keep the structure in the immediate occupancy performance level for seismic hazard levels correspond to the available gap size. However, after pounding to the moat wall the acceleration demand of the superstructure does not exceed significantly from its fixed-based spectral acceleration. Therefore, the structural performance level of the superstructure is not worse than that of its fixed-base counterpart after pounding. It should be noted that generally by increasing the stiffness of the moat wall and decreasing the gap size the superstructure acceleration demand increases. This requires a careful selection of dynamic properties of the base-isolated system to avoid excessive acceleration demands where a limited gap distance is available. In this way, an undamaged status may be attributed to a base-isolated structure before pounding occurs. After that, earthquake induced loss remains in the same order of its fixed-based counterpart.

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