Effect of impact model on the response of base-isolated building during moat wall pounding

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ABSTRACT

Base isolation is a seismic protection strategy that has consistently proven to effectively reduce the damage to buildings in earthquakes. However, the large displacements induced in the isolation layer during shaking can potentially result in pounding between the structure and surrounding moat walls. This phenomenon, known as moat wall pounding, is a step-change in response that can lead to an increased probability of collapse. The simulation of moat wall pounding is necessary to understand its consequences on the superstructure and inform building design. Moat wall pounding is typically simulated using the contact force-based approach, and various impact models have been developed to capture the relationship between pounding force and deformation.

This study investigates the influence of various impact models on the seismic response of a reinforced concrete base-isolated building through numerical simulations. Six commonly used impact models, linear elastic, Hertz, Hertzdamp, Muthukumar, linear viscoelastic, and nonlinear viscoelastic models, are considered. A suite of near-fault ground motions from the PEER strong ground motion database, representing different peak ground acceleration (PGA) levels, is utilised for nonlinear time history analysis on a 2D numerical model developed in OpenSees software.

The results reveal that neglecting superstructure nonlinearity leads to underestimating inter-storey drift ratios and overestimating floor accelerations during pounding. The selection of impact models has minimal effect on the superstructure response. Therefore, the linear elastic model, being the simplest, is appropriate for predicting the effects of moat wall pounding on superstructure responses. However, the Muthukumar model is not recommended for elastic superstructures due to its tendency to predict excessively large indentations that exceed the gap size.

1 INTRODUCTION

Base isolation techniques have gained widespread application in structural engineering, serving as highly effective measures for controlling vibrations under earthquakes. Base-isolated buildings have demonstrated superior seismic performance compared to fixed-based buildings, as observed during past earthquake events, such as the 2010 Darfield earthquake and the 2011 Christchurch earthquake in New Zealand (Gavin & Wilkinson, 2010; Mcintosh et al., 2012), 2016 Kumamoto earthquake in Japan (Morita & Takayama, 2017) and the 2023 Turkey earthquake in Turkey (Qu et al., 2023). Reduced floor accelerations and superstructure inter-storey drifts of base-isolated buildings compared to fixed-base buildings are the main advantages of base isolation, which also results in reduced structural and non-structural damage.

The flexibility of the base isolation systems leads to large displacements in the isolation layer, which may result in pounding of the base level (i.e. structural plane immediately above the isolators) to surrounding moat walls if the moat clearance is insufficient. While structural damage from moat wall pounding is rarely observed in past earthquake events, numerous studies have demonstrated its effects through numerical simulations and experimental tests (e.g. Komodromos et al., 2007; Polycarpou & Komodromos, 2010; Masroor & Mosqueda, 2012; Pant & Wijeyewickrema, 2013). In these studies, moat wall pounding has been observed to significantly amplify superstructure floor accelerations, especially at the base level, and excite higher vibration modes in the superstructure, increasing inter-storey deflections. Additionally, the base shear demand increases during pounding (Qu et al., 2013; Mavronicola et al., 2020), in some cases exceeding the weight of the building (Malhotra, 1997). Shaking table tests by Masroor and Mosqueda (2012) showed that maximum inter-storey drift ratios can increase by 1.5 to 2 times, and base-level accelerations can increase from 0.4 g to over 3.0 g. Such response amplification has been demonstrated to increase the probability of collapse (Masroor & Mosqueda, 2015; Bhagat & Wijeyewickrema, 2018).

Simulation of moat wall pounding is essential for understanding its consequences and developing mitigation measures. It is critical to include the simulation of moat wall pounding in the assessment of existing buildings. Most research on structural pounding utilises two modelling techniques: the stereomechanical approach and the contact force-based approach. The latter is preferred in civil engineering due to its ease of integration with finite element analysis software and its ability to simulate the entire impact process (Guo et al., 2012). In this method, a contact element is activated during pounding, and the contact force is computed based on the chosen impact model, which accounts for either elastic or inelastic behaviour. When energy dissipation is considered, a dashpot is introduced parallel to the contact spring.

Impact models with linear or nonlinear force-deformation relationships, with and without dashpots, have been proposed in previous studies (e.g. Anagnostopoulos, 2004; Jankowski, 2005; Muthukumar & DesRoches, 2006). The efficiency and accuracy of these forced-based impact models in numerical simulations have been investigated. Jankowski (2005) compared linear elastic, nonlinear elastic, and nonlinear viscoelastic models, finding that nonlinear models produced smaller simulation errors in pounding force time histories. Muthukumar and DesRoches (2006) examined the seismic response of two elastic 2DOF systems using the Hertzdamp and Kelvin models, reporting that displacement amplification due to pounding varied by up to 12% across different models. Mate et al. (2015) evaluated spring-dashpot models in MDOF systems, while Khatiwada et al. (2013) validated various impact models against experimental data from steel portal frames, concluding that impact model selection had minimal influence on predicting seismic responses. Similarly, Hughes and Mosqueda (2020) investigated the effects of impact model selection in base-isolated buildings and found little impact on overall seismic response.

Most studies analysed two or three impact models on a linear system. While Hughes and Mosqueda (2020) provided a more comprehensive comparison between five impact models, linear elastic, linear viscoelastic, nonlinear viscoelastic, Hertzdamp, and improved Hertzdamp models, the variations of seismic response due to varying impact models of each storey were not presented. Additionally, their conclusions were based on the average maximum response across multiple ground motions. This approach may have obscured variations between impact models under varying ground motion intensities. Muthukumar and DesRoches

(2006) grouped ground motions by peak ground acceleration (PGA) and found that the influence of impact models on the response varied from PGA level.

This study investigates the effect of six commonly used impact models (linear elastic, Hertz, Hertzdamp, Muthukumar, linear viscoelastic, and nonlinear viscoelastic model) on the superstructure response of an RC base-isolated building. The effect of the nonlinearity of superstructure is considered. Numerical models with elastic and inelastic superstructures are developed in the OpenSees platform to account for the nonlinearity of superstructure. A suite of near-fault ground motions from the PEER strong ground motion database (PEER, 2014), covering a range of peak ground acceleration (PGA) levels, is utilised for nonlinear time history analysis. Building on the findings of previous research by Muthukumar and DesRoches (2006), this research adopts a similar approach of dividing ground motions by PGA to investigate the influence of ground motion intensity on seismic response variations between different impact models. Seismic response of inter-storey drift ratio, floor accelerations and velocities are mainly studied. The response of each storey is analysed to provide a more detailed understanding of how impact models influence the superstructure behaviour.

2 IMPACT MODELS

Six impact models are selected for the simulation of moat wall pounding, as illustrated in Figure 1. The horizontal axis (Δ) represents the relative displacement between pounding bodies after contact, and the vertical axis (F) corresponds to the resulting pounding force. The selected models, i.e. linear elastic model, Hertz model, Hertzdamp model, Muthukumar model, linear viscoelastic model, and nonlinear viscoelastic model, simulate the unidirectional point-to-point pounding behaviour of two objects. Regarding the moat wall pounding scenario, the pounding occurs between the base level and the moat wall. When the distance between the base slab and moat wall exceeds the initial gap Δ_g, a pounding force is generated based on impact stiffness k, and indentation $\delta = \Delta - \Delta_g$. Relative velocity and damping coefficient c will also influence the pounding force if a viscous damper is incorporated. Here, δ represents the material deformation of the base slab and the moat wall. During the approaching period of pounding (when $\delta > 0$), the pounding objects compress each other, causing material deformation. During the restitution period (when $\delta < 0$), pounding objects separate.

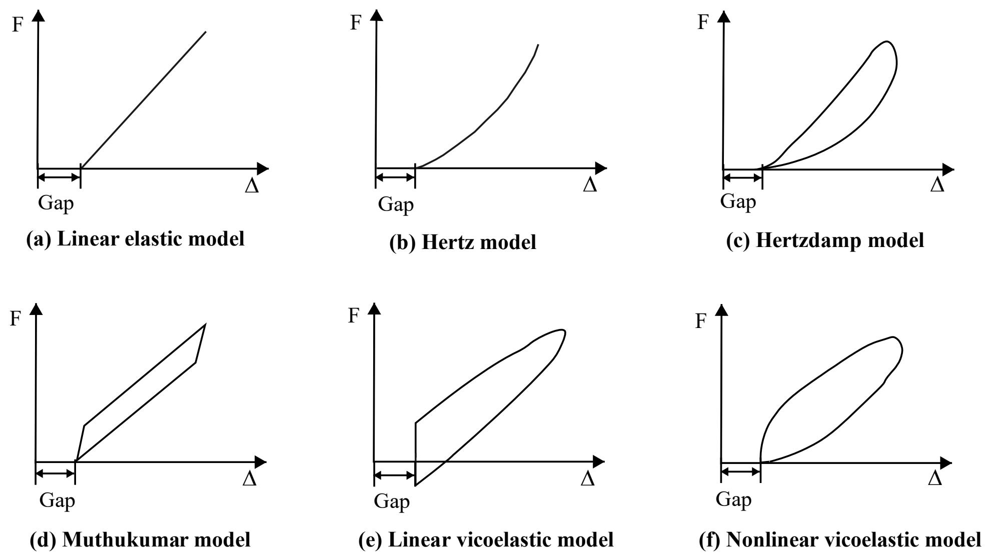


Figure 1: Force and relative displacement relationship of impact models.

These six models offer a range of behaviours from simple linear elastic to more complex viscoelastic damping characteristics and can be divided into two groups: linear and nonlinear models. Linear models, including linear elastic and linear viscoelastic models (refer to Figures 1a and e), exhibit a linear pounding force and indentation relationship during pounding. The linear elastic model describes an elastic pounding process without considering energy dissipation, assuming that all energy induced by pounding is conserved. In contrast, the linear viscoelastic model incorporates a viscous damper to account for energy dissipation during pounding. Nonlinear models, i.e. Hertz, Hertzdamp, and nonlinear viscoelastic models, demonstrate a nonlinear relationship between pounding force and indentation (Figures 1b, c, and f). The Hertz model, derived from the Hertz contact law, assumes an elastic pounding and the energy induced by pounding is ideally transferred to the pounding objects (Goldsmith, 2001). Hertzdamp model extends the Hertz model by introducing a viscous damper to capture energy dissipation. However, Hertzdamp model applies a constant viscous damping throughout the entire pounding process, which is considered unrealistic since minor energy loss occurs in the restitution period (Ye et al., 2009). The nonlinear viscoelastic model has been proposed with no energy dissipation during the restitution period (Jankowski, 2005). The Muthukumar model (Figure 1d) is a bilinear approximation of Hertzdamp model, developed for easier implementation in OpenSees software (Muthukumar, 2003). It defines initial and secondary stiffness values by assuming the dissipated energy during pounding at maximum indentation (i.e., the area under the curve) matches that of the Hertzdamp model. It is noted that the expected yielding and maximum indentations, δ_y and δ_m, should be provided as input parameters in OpenSees (Hughes & Mosqueda, 2020). However, the actual δ_y and δ_m may differ from input values. The Hertz stiffness, k_h, for nonlinear models is calculated as follows:

$$k_h = \frac{4}{3\pi}\left(\frac{1}{\lambda_1 + \lambda_2}\right)\sqrt{R_1} \tag{1}$$

where R_1 is the radius of a sphere colliding with a massive plane surface, which is taken as 0.6 according to Bustamante (2024). λ_i is a material parameter defined as:

$$\lambda_i = \frac{1-\nu_i^2}{\pi E_i} \tag{2}$$

where E_i and λ_i are the modulus of elasticity and Poisson's ratio of the pounding material, respectively. E_i and λ_i are adopted as 25742 MPa and 0.2 for C30 concrete, so k_h is calculated as 1.38e7 kN/m$^{3/2}$. Impact stiffness for the Muthukumar model is defined as the effective stiffness of the Hertzdamp model, calculated as k_h multiply the square root of δ_m. Impact stiffness for linear models, k_l, is taken as the axial stiffness of the base slab (Muthukumar & DesRoches, 2006), calculated as 6.18e6 kN/m.

3 CASE STUDY OF A PROTOTYPE BUILDING

3.1 Information on the prototype building

A 10-storey reinforced concrete (RC) moment frame building from Red Book (NZCS & CCANZ, 2008) is adopted as the prototype for this study, with an overview provided in Figure 2. The building is designed in accordance with NZS 3101 (NZS 3101:2006), and earthquake loadings are determined following NZS 1170.5 (NZS 1170.5:2004). The building has a square plan of 29.45 m x 29.45 m. The typical storey height is 3.6 m, with a height of 4 m for the first storey. This building is designed with a ductility factor of 4 and it is designed in the Central Business District of Christchurch. The floor mass is 645 and 636 tonnes for the first and top floors, respectively, and 638 tonnes for the other floors. A two-dimensional (2D) model is built to simulate the frame along grid F, assuming that it resists half of the building's total lateral demands. The fixed-base structure has a natural period of 2.0 s. Lead rubber bearings (LRBs) are added under each column as the isolation layer. The properties of the LRBs are selected from the product catalogue of an industrial company, DIS (DIS, n.d.), to achieve an effective period at ULS of approximately 3 times the natural period as specified by the Guidelines for the Design of Seismic Isolation Systems for Buildings (NZSEE, 2019). The resulting base-isolated building has a natural period of 3.27 s and an effective period of 6.05 s at ULS design displacement. The moat clearance is set to 0.1 m to increase the likelihood of pounding induced by ground motions.

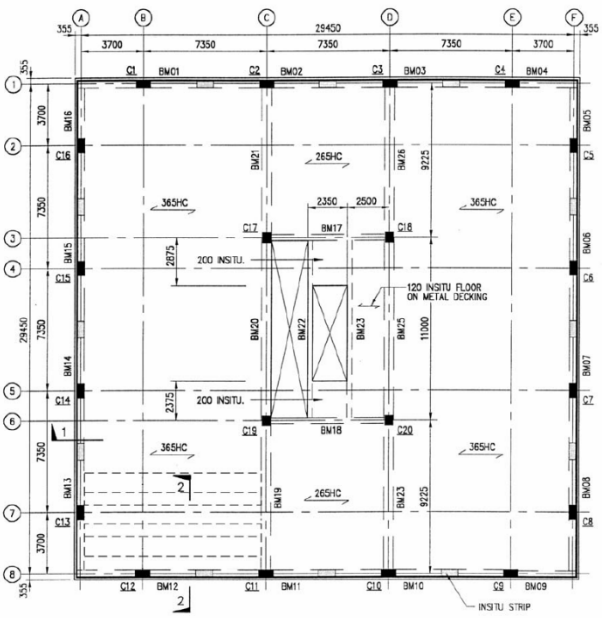


Figure 2: Frame building plan.

3.2 Numerical model with elastic and inelastic superstructures

The 2D numerical models with elastic and inelastic superstructures are developed in OpenSees software. A leaning column is added to account for the P-Δ effect of the internal gravity frames. Regarding the elastic model, beams and columns are simulated as elastic elements. The LRBs are modelled using zero-length elements, with bilinear material behaviour assigned in the horizontal direction and elastic behaviour in the vertical direction. The moat wall is simulated as a fixed node. Impact models are applied between the moat wall and the base slab. This simulation method differs from that of Hughes and Mosqueda (2020), who accounted for the dynamic behaviour of the moat wall. The base of the LRBs is fixed, and the soil effect is not considered. 2% Rayleigh damping is applied to the model. The inelastic superstructure is modelled by adding rotation springs at the ends of beams and columns to simulate plastic hinges using bilinear backbone curves. The yielding point is determined by NZS 3101 (NZS 3101:2006), and the post-yield stiffness is determined by ASCE 41-17 (ASCE/SIE 41-17, 2017).

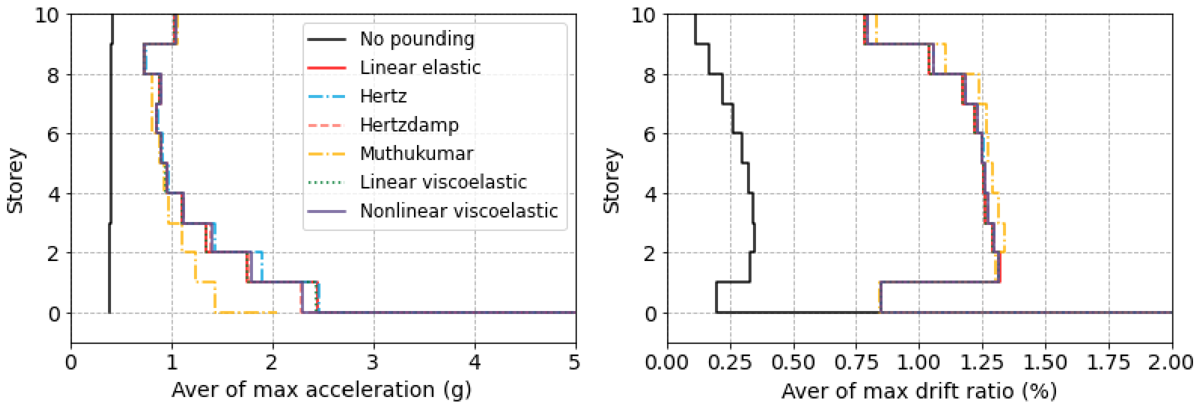
4 GROUND MOTIONS AND RESULTS

4.1 Selected ground motions

A set of 100 near-fault ground motions is used for dynamic time history analysis, including a range of intensities to consider the ground motion variations. The original, unscaled ground motions are employed as input excitations. Results of ground motions that fail to converge in the dynamic analysis or do not induce pounding are excluded in the following sections.

4.2 Effect of impact models on the elastic superstructure

The average of the maximum floor acceleration and inter-storey drift ratio of the superstructure is shown in Figures 3a and b, respectively. The black line indicates the response in the case of no pounding. Compared to the 'No pounding' case, moat wall pounding induces significant amplifications in floor accelerations and inter-storey drift ratio. While floor accelerations in the lower storeys show some variation between different impact models, floor accelerations of upper storeys are not affected by the impact model used in the simulation. In contrast, the impact model has a negligible influence on the inter-storey drift ratio of the superstructure. Muthukumar model leads to the smallest floor accelerations and a slightly larger inter-storey drift ratio compared to the others.

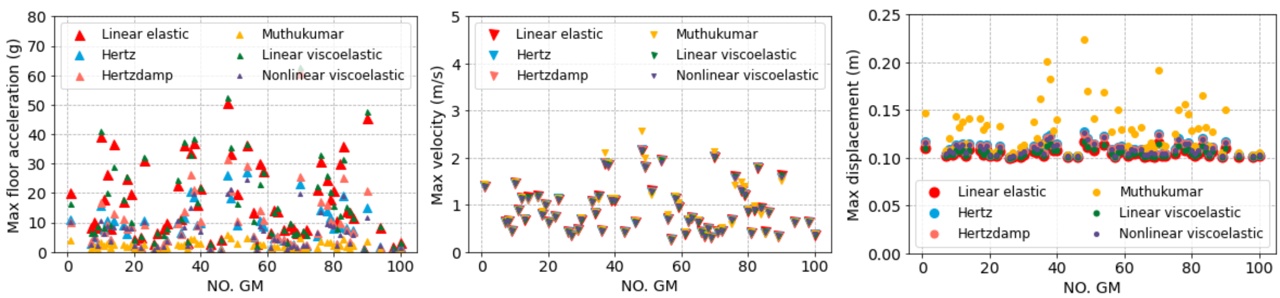


(a) Average floor acceleration (b)Average inter-storey drift ratio

Figure 3: Average of the maximum response with and without pounding.

The maximum responses at the base level, including accelerations, velocities, and displacements, are shown in Figure 4. Floor accelerations exhibit significant variations across different impact models. Linear models cause the largest accelerations, and Muthukumar model leads to the smallest accelerations. However, all impact models except for Muthukumar model result in similar maximum displacements at the base level. This outcome is expected because indentations relative to moat space are anticipated to be small. Base velocities using different models are similar, indicating that almost the same amount of energy transfers into the superstructure. Varying accelerations in the base level are only spikes over a short. Consequently, the superstructure responses are similar when using different models. The observation that Hertzdamp model predicts lower accelerations than the linear viscoelastic model is consistent with the results from Muthukumar and DesRoches (2006). Muthukumar model results in larger base velocities in some cases compared to the other models. Consequently, the superstructure's inter-storey drift ratio (Figure 3b) is higher under Muthukumar model compared to the other models, which predict similar inter-storey drift ratios.

Muthukumar model generally leads to a higher base displacement due to its smaller impact stiffness than the other models. Additionally, the Muthukumar model requires an assumption on the expected indentation. If the actual indentations exceed the input expected values, this model cannot give reasonable predictions of pounding response (Hughes & Mosqueda, 2020). Comparatively, the maximum indentations for other models are all similar.



(a) Acceleration (b) Velocity (c) Displacement

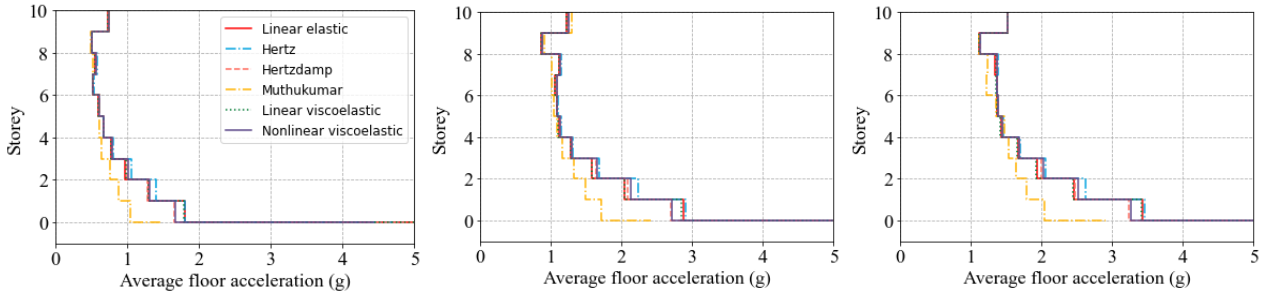
Figure 4: Maximum response of base level using different impact models.

To investigate the effect of ground motion intensity on the selection of impact models, ground motions are divided into different groups according to their PGA, as shown in Table 1.

Table 1. Range of divided ground motions.

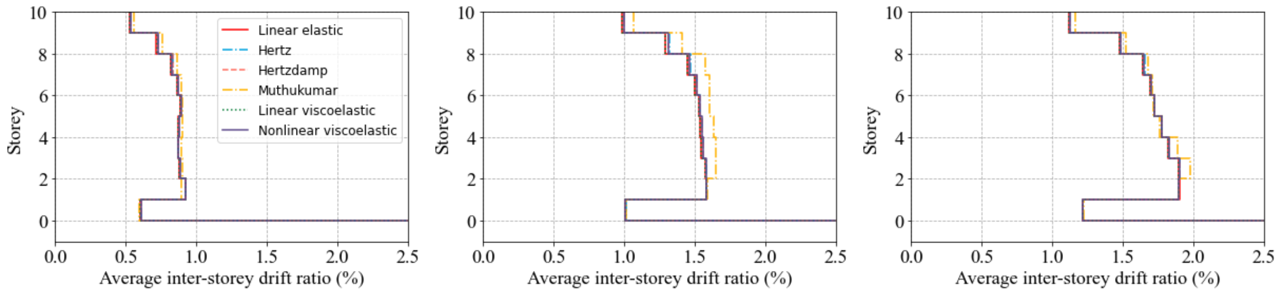
Groups	PGA1 (g)	PGA2 (g)	PGA3 (g)
Range	0 ~ 0.3	0.3 ~ 0.6	0.6 ~ 0.9
Number of ground motions	32	19	13

The average of the maximum floor accelerations and inter-storey drift ratio are shown in Figures 5-6. Muthukumar model leads to the smallest floor accelerations, while the other models produce identical floor accelerations in the upper storeys and exhibit slight variations in the lower storeys. The inter-storey drift ratios of the superstructure using Muthukumar model are slightly higher than those of the other models; however, this increase is negligible. Other impact models result in identical inter-storey drift ratios across three PGA groups, suggesting that PGA has little effect on the superstructure response variations between different impact models.



(a) PGA1 (b) PGA2 (c) PGA3

Figure 5. Average of the maximum floor accelerations of different PGA groups.

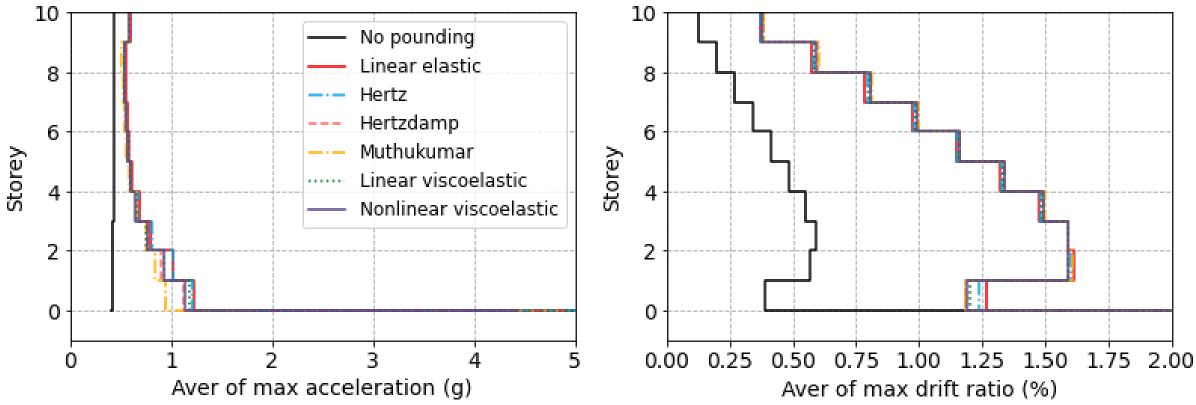


(a) PGA1 (b) PGA2 (c) PGA3

Figure 6. Average of the maximum inter-storey drift ratios of different PGA groups.

4.3 Effect of impact models on the inelastic superstructure

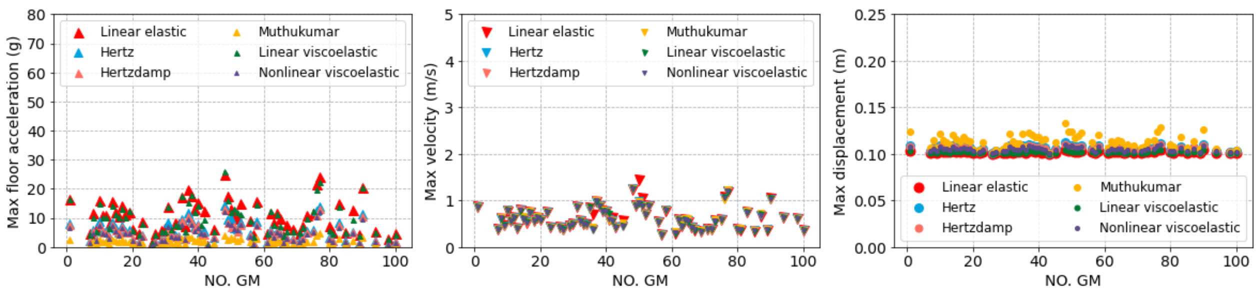
The average of the maximum response of inelastic superstructure for no pounding and pounding cases is illustrated in Figure 7. Moat wall pounding significantly amplifies floor accelerations and inter-storey drift ratios, particularly in the lower storeys. Regarding pounding cases using different impact models, floor accelerations and inter-storey drift ratios show minimal variation. These results reveal that the selection of impact models has an insignificant effect on the responses of the superstructure. This conclusion aligns with the findings reported by Hughes and Mosqueda (2020), suggesting that the influence of impact models on the superstructure response of base-isolated buildings is negligible regardless of whether the vibrational properties of the moat wall are considered or if it is modelled as rigid. Furthermore, the influence of impact models on superstructure responses follows the same trend for both elastic and inelastic superstructures.



(a) Average floor acceleration (b)Average inter-storey drift ratio

Figure 7. Average of the maximum response with and without pounding.

The maximum floor accelerations and inter-storey drift ratios at the base level are presented in Figure 8. Floor accelerations observed in the linear models are generally higher compared to those in the nonlinear models. Muthukumar model causes the largest base displacements and smallest accelerations, whereas the displacements for other models are nearly identical. For the inelastic superstructure, the base accelerations and displacements predicted by the impact models are smaller than those of the elastic superstructure. This reduction can be attributed to the yielding of plastic hinges in the inelastic model, which dissipates energy generated during pounding. These findings reveal that the nonlinearity of superstructure should be considered when analysing the consequences of moat wall pounding.



(a) Acceleration (b) Velocity (c) Displacement

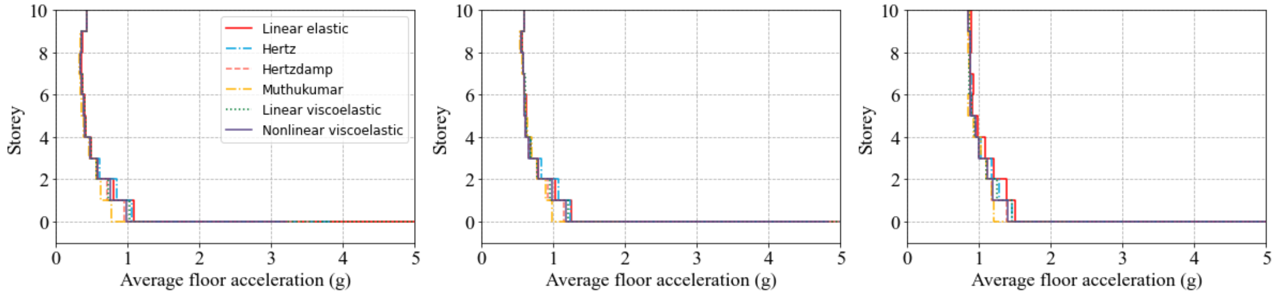
Figure 8. Maximum response of base level using different impact models.

Ground motions are divided into different groups according to their intensities to investigate the effect of ground motion intensity on the selection of impact models, as shown in Table 2.

Table 2: Range of divided ground motions.

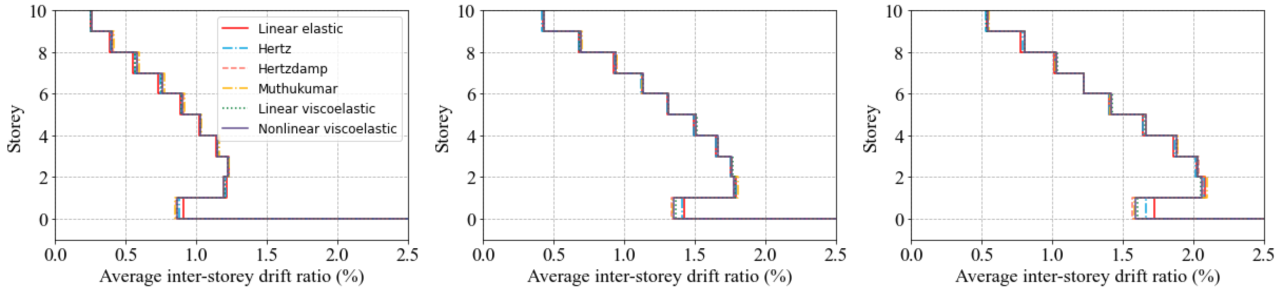
Groups	**PGA1 (g)**	**PGA2 (g)**	**PGA3 (g)**
Range	0 ~ 0.3	0.3 ~ 0.6	0.6 ~ 0.9
Number of ground motions	29	24	15

Figures 9-10 present the average of the maximum responses of the superstructure for different PGA levels. Overall, PGA has negligible influence on the variation in the superstructure responses between different impact models.



(a) PGA1 (b) PGA2 (c) PGA3

Figure 9. Average of the maximum floor accelerations of different PGA groups.



(a) PGA1 (b) PGA2 (c) PGA3

Figure 10. Average of the maximum inter-storey drift ratios of different PGA groups.

5 CONCLUSION

This study aimed to investigate the influence of various impact models: linear elastic, Hertz, Hertzdamp, Muthukumar, linear viscoelastic, and nonlinear viscoelastic models on the seismic response of a base-isolated reinforced concrete (RC) moment frame building, considering the effects of ground motion intensity and structural nonlinearity. Dynamic time history analyses were conducted on 2D numerical models using a suite of near-fault ground motions. The following conclusions can be drawn based on the results of this study.

1. The nonlinearity of the superstructure should be considered as the elastic model underestimates the inter-storey drift ratio and overestimates the floor accelerations when analysing pounding consequences. However, the selection of impact models has an insignificant effect on the superstructure response regardless of the nonlinearity of the superstructure.

2. While variations in peak base accelerations are observed across impact models, the peak velocities and displacements at the base level remain identical. This consistency indicates that the amount of pounding energy transferred into the superstructure is independent of the impact model used. Furthermore, the peak base displacement of base-isolated buildings is limited by the gap size, even when subjected to ground motions with high PGA. Consequently, the PGA level does not affect the variation in superstructure responses caused by different impact models.

3. The finding that superstructure responses of base-isolated buildings are not influenced by the choice of impact models is valid whether the vibrational behaviour of the moat wall is considered, or the wall is simulated as rigid.

4. Linear elastic model, being the simplest, is sufficient for simulating moat wall pounding of base-isolated buildings, as it predicts superstructure responses consistent with those of more complex models.

5. Muthukumar model is not recommended for elastic superstructures, as it can predict unreasonably high indentations that exceed the gap size.

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