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REVIEW ARTICLE

# Comprehensive Review on Seismic Pounding Between Adjacent Buildings and Available Mitigation Measures

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**Abstract**
Seismic pounding has taken place in several earthquake events since adjacent structures that lack adequate separation distance usually suffer from repetitive, severe collisions. These collisions result in considerable impact forces in addition to acceleration spikes, thus dealing damage to both structural and non-structural elements. So, a meaningful effort has been widely directed towards the investigation of that phenomenon, leading to a considerable number of publications that are related to that field of study. A review of these publications has thus become a matter of interest. Accordingly, this paper mainly aims to present a detailed state-of-the-art review concerned with seismic pounding between adjacent buildings. Firstly, general definitions, types, and causes of seismic pounding are addressed. Later, facts and statistics of historical earthquake incidents that reflect the scale of the threat caused by seismic pounding are clarified. Moreover, the effect of seismic pounding on fixed-base and base-isolated buildings is discussed. Furthermore, the effect of soil-structure interaction is also presented. Additionally, alternative mitigation methods for seismic pounding are presented. Their classification, types, efficiency, and applicability are also discussed. Eventually, different impact analytical models that can be used to simulate seismic pounding in theoretical studies are discussed. By the end of this paper, deficiencies in previous studies are clarified in order to be taken into account throughout future studies.

## 1 Introduction

Seismic pounding (also known as earthquake-induced pounding) is a phenomenon that occurs as a result of collisions between adjacent buildings with insufficient gap distances. In detail, the sudden transition of the ground beneath structures causes them to vibrate. However, structures do not freely vibrate since, in most cases, particularly in metropolitan cities, they are closely spaced because of financial and architectural considerations. This leads to seismic pounding. Small to null gap sizes increase the chances of interference between adjacent buildings during ground excitations. Consequently, these interactions result in high impact forces with short-duration acceleration pulses, known as spikes, on each structure. The problem that arises herein is that neither these forces nor accelerations were taken into account during the structural design process. In other words, each structure has been individually designed to resist both gravity and lateral loads, including earthquake loads, without paying attention to the pounding scenario. Accordingly, improperly designed adjacent structures vulnerable to pounding commonly suffer from both local and global damages.

During earthquakes, adjacent structures may either exhibit in-phase vibrations (Fig. 1a) or out-of-phase vibrations (Fig. 1b). The latter is the general case since it is common for structures to have different dynamic properties such as fundamental period, lateral stiffness, damping ratio, etc. In overall, seismic pounding is amplified in the case of out-of-phase vibrations [1–3].

It is worth pointing out that seismic pounding does not exclusively occur between adjacent buildings; it can also take place on bridges. [4]. As is custom, bridges are

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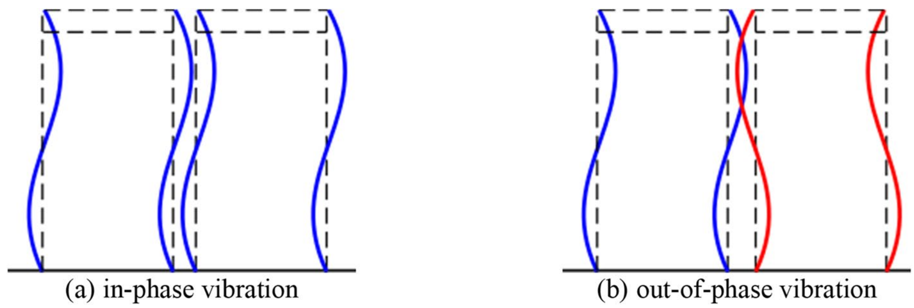



**Fig. 1** Seismic vibration response of adjacent structures

commonly provided with expansion joints to overcome thermal change effects. Therefore, the whole bridge is divided into several segments that are located at certain distances, usually within a few centimetres, apart from each other. During earthquakes, collisions may occur between the decks in each segment or between the decks and the adjacent abutment. Nevertheless, seismic bounding between adjacent bridge segments is out of the scope of this paper.

Due to the risk and serious consequences that come with seismic pounding, numerous research papers and books have been published considering this issue. In light of this, the current review article aims to comprehensively highlight and discuss the historical incidents in which seismic pounding was involved and provide a critical overview of the studies found in the literature related to that field.

## 2 Types of Seismic Pounding Between Adjacent Buildings

There exist two main types of seismic pounding. Typically, slab-to-slab (floor-to-floor) pounding or slab-to-column (floor-to-column) pounding, as illustrated in Fig. 2. In the first one, the colliding buildings have equal storey heights. Thus, slabs of each building crash into one another. On the other hand, corresponding stories in each building in the case of slab-to-column pounding vary in height. Consequently, the collision herein occurs between the slab of one building and the column of the other one. For sure the latter case is more dangerous since the columns are subjected to high shear forces somewhere through their height [1].

Cole et al. [5] indicated that several scenarios can accompany the aforementioned types of pounding. For instance, a heavier building may collide with a lighter building (Fig. 3a), or a taller building may collide with a shorter building (Fig. 3b). Moreover, they also demonstrated that colliding buildings are not always ideally aligned with each other, causing translational (symmetric) pounding (Fig. 3c), but they may be eccentrically positioned, resulting in a particular type of pounding called torsional (asymmetric) pounding, as depicted in Fig. 3d. As elucidated by Karayannis and Naoum [6, 7], buildings subject to seismic vibrations and asymmetric restraints due to neighbouring structures often experience torsional oscillations. This particular phenomenon represents torsional (asymmetric) pounding. It usually occurs in densely populated urban centres, where it is typical to find large blocks comprising multistorey buildings closely situated to one another. It is noteworthy that the ownership and geometric layout of land within these blocks vary. Consequently, it is highly probable that adjacent structural systems within a block come into partial and asymmetric contact. That phenomenon can also take place in non-uniform shaped buildings such as L-plan buildings (Fig. 3d), which are particularly problematic. Owing to the pronounced disparity in lateral stiffness along their two axes, these buildings tend to sway differentially during seismic events. This discrepancy often results in structural damage or complete failure, especially at the common corner juncture, under severe earthquake conditions. To address this issue, L-plan buildings are sometimes constructed as two distinct entities in contact, thereby eliminating the configuration problem. However, this leads to the occurrence

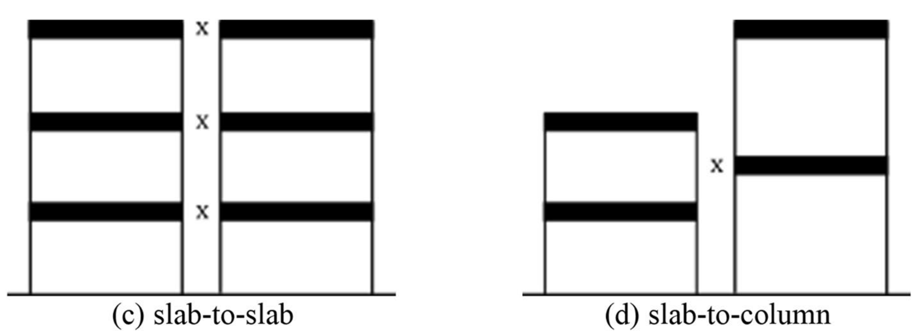
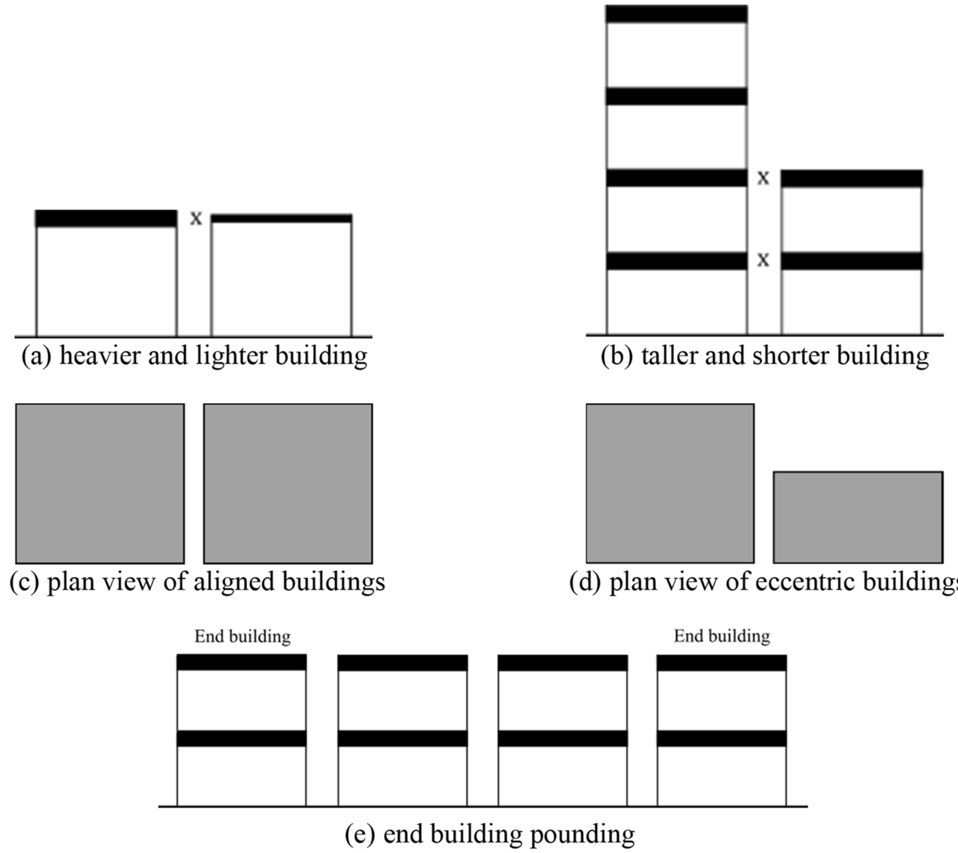



**Fig. 2** Types of seismic pounding

**Fig. 3** Scenarios of seismic pounding




of torsional (asymmetric) pounding. It is also noted that seismic oscillations induced by earthquakes in adjacent structural subsystems of a building complex can frequently induce torsional oscillations in one another due to torsional (asymmetric pounding) [8]. The last scenario of the seismic pounding is the external building pounding (Fig. 3e).

## 3 Historical Incidents, Facts, and Statistics

Several field observation studies have been carried out previously to evaluate the damage that adjacent buildings have suffered because of pounding. The most common factor among these previous studies is that the adjacent buildings did not have sufficient separation distance. As an example, Doğan and Günaydin [9] reported that around only 36% of adjacent buildings in Eskişehir, Turkey, were provided with adequate gap distance.

Looking back, the vulnerability of adjacent buildings to pounding was first observed during the Alaska earthquake in 1964 [10]. Afterward, the damage resulting from seismic pounding has been seriously taken into design consideration. After the San Fernando earthquake in 1971, it was noted that the buildings located at the end of a row of buildings were the most vulnerable to seismic pounding; the collapse of the external stair tower of Olive View Hospital emphasised that since it fully collapsed [11].

The largest ever damage in history, caused by seismic pounding, dates back to 1985, when the Mexico earthquake took place. In 15% of fully and partially collapsed buildings, seismic pounding was found to be involved [1, 12–15]. The estimated number of people killed was about 10,000, while 50,000 were injured and 250,000 became homeless [2].

Moreover, 40% of inspected buildings that were damaged during the Loma Prieta earthquake in 1989 were found to be affected by seismic pounding [16].

Several buildings were also affected by seismic pounding during the Chi-Chi earthquake in 1999. These effects were more obvious in school buildings, which had expansions through building new adjacent ones. Surely, these adjacent buildings differed in their dynamic properties. Thus, they experienced an out-of-phase response. As a result of this, around 2500 people were killed, whereas more than 53,000 buildings were subjected to damage [17].

The city of Bhuj suffered catastrophic destruction in 2001 due to a major earthquake claiming approximately 20,000 lives, with 350 children perishing under the rubble of collapsed school buildings. Post-inspection data revealed moderate damage in a significant number of school buildings, while approximately 4–5% sustained severe damage.

These observations suggest various structural failures were responsible, with pounding identified as a key contributing factor [18]. Combined survey and investigative efforts demonstrated widespread infill wall damage, column shear failures, and a potential for pounding-induced collapse in numerous closely spaced buildings [19]. Moreover, bridges experienced pounding-related damage alongside buildings, as they underwent failure of girder ends and damage to bearings due to pounding between adjacent spans [20]. In addition, dislodgement of a significant portion of steel bearings at pier supports caused by the seismic pounding at deck joints was observed, which led to severe spalling of the concrete coating and cover, particularly around joints [21].

Eccentric pounding, previously discussed in Sect. 2, was observed in the Kaliningrad earthquake in 2004. The buildings that experienced that type of pounding displayed plaster spalling because of the large torsion strains that developed throughout the contact region [22].

Distinctive seismic pounding types and scenarios between adjacent buildings were also noted in the Wenchuan earthquake of 2008 [23]; this includes slab-to-column pounding, pounding between taller and shorter buildings, and end-building pounding.

After the 2010 Darfield earthquake, most of the buildings in Christchurch Central Business District suffered only low damage because of pounding [24, 25]. However, few masonry buildings underwent moderate to severe damage. Additionally, it was obvious that most of the pounding damage was concentrated in vertical elements. Christchurch Central Business District was again struck by another earthquake in 2011 called the Christchurch earthquake [26, 27]. Out of 376 surveyed buildings, 6% were severely damaged. 22% of the surveyed buildings, as well, displayed some signs of pounding damage. It was also detected that even low-rise buildings (definitely masonry ones) suffered from pounding damage. With respect to the buildings that had their separation gaps filled with solid architectural flashings, they were vulnerable to seismic pounding.

In the Lorca 2011 earthquake, several structural and non-structural elements experienced damage due to pounding [28]. One of the most significant cases that caused shear failure of the columns was slab-to-column pounding. Structural elements were not the only ones affected by seismic pounding; exterior masonry walls also suffered from apparent damage.

Pounding damage was later detected in the Gorkha earthquake in 2015 [29–34]. Damage in adjacent buildings ranged from low to severe. In addition, some closely spaced buildings almost completely collapsed because of the insufficient gap size.

More recently, seismic pounding was also reported to take place in the Sivrice-Elazığ earthquake in 2020 [35–38]. Adjacent buildings, especially those having different storey heights, exhibited fragile behaviour. Of course, this threatened the integrity of the affected buildings. In that earthquake, pounding damage either took place in beams, columns, or masonry walls.

Consequently, it is realised that seismic pounding damage that affected adjacent buildings ranged from local damage in non-structural elements (such as masonry walls) up to extreme damage (such as column shear failure or complete collapse). For that, seismic pounding is considered to be of great importance. It should also be considered in the design process of adjacent buildings to avoid such damage.

## 4 Response of Adjacent Buildings Exposed to Seismic Pounding

Numerous investigations in the literature have been involved with the effect of seismic pounding on the response of adjacent buildings. In addition, several studies have examined how the dynamic properties of the buildings (such as number of storeys, mass, total height, etc.) influence the response of adjacent buildings subjected to seismic pounding (in terms of storey acceleration, storey displacement, inter-storey drift, impact force, etc.). Those investigations can be classified into three main categories or aspects. The first one includes studies concerned with buildings having fixed bases; the second one considers buildings equipped with base isolation systems; and finally, studies that take soil-structure interaction into consideration fall under the third category.

### 4.1 Buildings with Fixed Bases

Anagnostopoulos [39], and Anagnostopoulos and Spiliopoulos [40] numerically explored the effect of different parameters on seismic pounding. Single-degree-of-freedom systems were utilised in the first research, whereas the second involved multi-degree-of-freedom systems. They detected that, in a row of adjacent buildings, the pounding-induced response of the interior buildings was amplified when the ratio of their fundamental period to that of the adjacent ones was less than unity. However, this amplified response was still lower than that of the outer buildings. On the other hand, if the prementioned ratio exceeded unity, the interior buildings noticeably exhibited reduced response. Furthermore, significant differences in masses and heights of adjacent buildings also led to an increase in inter-storey drift, storey acceleration, and impact force.

Likewise, Abdel Raheem [41, 42], conducted a numerical study on two single-degree-of-freedom adjacent buildings to assess their response due to pounding. Similar to the work in [39, 40], the results emphasised that the large difference in the fundamental periods of adjacent buildings amplified the pounding-induced response. He also found that storey acceleration, storey displacement, and storey shear were increased in the case of pounding.

Elwardany et al. [43] carried out a similar study to investigate the seismic pounding between four adjacent buildings with different fundamental periods. They demonstrated that the arrangement of adjacent buildings notably affected the pounding-induced response. For instance, the existence of flexible buildings at the outer edges of the cluster caused an increase in the pounding forces. This is consistent with the findings in Ref. [40].

Otherwise, Maniatakis et al. [44, 45] selected the central church of the Kaisariani Monastery as a case study. This church has been built in two different constructional phases; in the first one, during the eleventh and twelfth centuries, the main building of the church had been built; in the second phase, during the sixteenth and seventeenth centuries, a narthex and a chapel were built. Thus, it can be regarded as two adjacent buildings, rather than a single one. They found that pounding seemed to be significant even though the two buildings nearly had equal fundamental periods.

Rojas and Anderson [46] carried out a numerical case study on a ten-storey office building existing in Los Angeles and the four-storey parking building next to it. These buildings experienced seismic pounding during the San Fernando earthquake of 1971. The results proved that the effect of pounding on storey displacement and storey shear vanished after about four storeys. Moreover, pounding was found to limit the maximum storey displacement of the office building as the parking building tended to restrain it from further translation beyond it.

In the same manner, Efraimiadou et al. [47] numerically studied the pounding-induced response of a row of buildings with different arrangements and parameters. They compared pounding and no-pounding cases in terms of maximum and residual inter-storey drift, as well as maximum storey acceleration. Of course, pounding was found to seriously detriment the response of the buildings. Nevertheless, it was found beneficial in some buildings that had their inter-storey drift dropped. The latter remark is consistent with the findings in [46].

This led Sołtysik and Jankowski [48], based on their numerical work, to conclude that although pounding raises the storey displacement and storey acceleration of one of the colliding buildings, it may play a vital role in lowering the response of the second one.

Seismic pounding between adjacent 3-D buildings was numerically investigated by Jameel et al. [49]. They observed that the pounding-induced response of the buildings was mainly affected in the pounding direction. In contrast, the response in the transverse direction was mostly insignificant. They also demonstrated, like [46–48], that pounding may reduce storey displacement as adjacent buildings block each other from translation. The lighter-weight building was, in addition, found to have higher storey acceleration and storey displacement compared to the heavier one. So, it may be more vulnerable to damage.

In addition, the numerical simulations of Inel et al. [50] on inadequately separated buildings subjected to pounding showed that they had a high demand for storey displacement.

Jankowski [51, 52] presented the concept of the pounding force response spectrum in order to facilitate the design process for pounding-related issues. The pounding force response spectrum is similar to typical acceleration response spectra, which are commonly used in seismic design. So, it can be defined as a plot of the maximum impact force caused by seismic pounding in terms of the fundamental periods of the colliding buildings. The generated response spectra did not only act as a useful tool for the design of closely spaced buildings, but they also provided helpful remarks regarding the pounding-induced response of those buildings. For instance, variations in the gap size, fundamental period, damping coefficient, mass, etc. were found to considerably affect the pound-induced response of the buildings.

To evaluate the influential parameters on seismic pounding, Crozet et al. [53, 54] conducted a sensitivity analysis on adjacent buildings based on Monte Carlo simulations. It was noticed that the ratio of the frequencies of adjacent buildings was the predominant parameter in determining the impact force.

The numerical work of Karayannis and Favvata [55, 56] demonstrated that the columns located around the contact region exhibited an extensive increase in ductility requirements to the extent of exceeding existing ones. In the case of slab-to-column pounding of two non-equal-height buildings, they deduced that the ductility and shear requirements of the columns of the taller building could be reduced if a significant difference in storey numbers exists. Alternative numerical investigations conducted also by Karayannis et al. [57, 58] revealed that non-equal storey heights resulted in an excess of the shear requirement in the case of torsional pounding.

To further analyse the effect of slab-to-column pounding, Dogan et al. [9] carried out numerical investigations on buildings with distinctive heights. They demonstrated that pounding near the end of the column was better for building safety than taking place in the column at mid-height. Rajaram and Kumar [59], as well, confirmed the same observation.

The experimental work in [60] besides the numerical work in Ref. [61–67] showed that, overall, a flexible building pounded with another stiff one would be vulnerable to response amplification. Meanwhile, the response of the stiff building is only slightly influenced. The rationale behind this is that impact forces transfer from the building with high stiffness to the flexible one. The same remark was also emphasised in the prior case studies carried out by Jankowski [68, 69] and the subsequent case studies of Maniatakis et al. [44, 45].

Speaking of which, Jankowski [68, 69] pointed his attention to numerically investigating the pounding between Olive View Hospital and its adjacent stairway towers, which had been discussed earlier in Sect. 3. He confirmed that care should be taken to properly design a weaker building that is located beside a stiffer one since seismic pounding could be destructive to the weaker building.

On the contrary, in some reported cases, the pounding-induced response of the stiff building could be amplified. At the same time, this stiff building might limit the response of the adjacent flexible building [70–73].

To settle this, Dimitrakopoulos et al. [74, 75], in their dimensional analysis of single-degree-of-freedom systems, clarified that building response amplification was dependent on the frequency range of the acting ground motion. Specifically, the flexible structure was vulnerable to response amplification in the case of low-frequency ground excitations. In contrast, the stiff structure response was amplified if the acting ground excitation had a high frequency range.

Moreover, Changhai et al. [76] conducted a dimensional analysis of multi-degree-of-freedom systems. They indicated that the pounding-induced displacement and velocity of the flexible system were directly proportional to the mass ratio.

Along the same lines, Chenna and Ramancharla [77] performed numerical investigations on adjacent buildings with equal and unequal heights. They revealed that the stiff building had its pounding-induced response amplified if both buildings had a frequency that was comparable to the dominant frequency of the ground excitation, regardless of the heights of the buildings. The response of the flexible building, on the other hand, became amplified if the frequency of it as well as the stiff building were not comparable to that of the ground excitation.

As shown before, like in the discussion of [74, 75], the seismic pounding process is not only dependent on the characteristics of the buildings but also on the properties of the earthquake [78]. Chitte et al. [79] numerically made a comparison between near-field and far-field earthquakes in terms of their induced pounding. As precited, near-field excitations caused impact force, acceleration, base shear, and storey displacement to rise significantly in comparison with far-field excitations.

However, based on the experimental tests of Fujii and Sakai [80], the general trend of flexible and stiff buildings' responses to pounding is dependent on their heights. So, a flexible or stiff building might have its response amplified or limited in accordance with the height of the adjacent stiff or flexible building. This is also coincident with the results of Abdel Raheem et al. [81]. Accordingly, no general conclusion can be obtained herein.

Nazri et al. [82] studied the pounding-induced response of adjacent buildings subjected to repeated earthquakes. The analysed buildings were either regular or irregular in their elevation. They noticed that in the regular buildings under consideration, the damage was significant at the bottom floors. Meanwhile, irregular buildings exhibited considerable damage to the floors near the bottom and top of the buildings.

Moreover, Jing et al. [83] numerically analysed the responses of adjacent buildings, that one of them experience collapse first then they collide into each other. This was achieved through the placement of weak columns in different locations throughout the building. They noticed that the impact force was minimised if the weak column was in the bottom storeys. The impact force increased if the weak column was placed in a higher storey. This was because it got near the collision.

Based on the work of Folhento et al. [84], it was deduced that in the case of a collision between a taller and a shorter building, the response of the taller building in the region located below the height of the shorter one was decreased.

In order to study the effect of the material of colliding buildings on the overall response, Jankowski [60] experimentally tested four cases for the pounding-induced response of two adjacent frames. Each pair of frames per case differs in their material of construction. To clarify, the first case was involved with the pounding of steel-to-steel frames; the second case was involved with the pounding of concrete-to-concrete frames; and the third case was involved with the pounding of timber-to-timber frames. Finally, the fourth case involved the pounding of ceramic-to-ceramic pounding. The results illustrated that steel frames had the highest peak displacement. It was then followed by concrete, ceramic, and timber, respectively. Also, Favvata et al. [85] numerically carried out a similar study in which they analysed the pounding between a reinforced concrete building and an adjacent steel one subjected to slab-to-column pounding. The results indicated that the columns subjected to the hit had increased shear and ductility demands.

Although masonry infill has usually been neglected in most numerical studies, Elwardany et al. [86, 87] proved that taking masonry infill into account can reduce the pounding effect compared to bare buildings. This was attributed to the fact that higher vibration mode shapes are involved in the case of seismic pounding. At the same time, these mode shapes were characterised by high deformation of the storeys. So, infilled panels were, for sure, subjected to lower deformations because of their higher stiffness compared to bare panels. This is contradictory to most simple buildings, for which only their first vibration mode shape is dominant in no-pounding cases. Ismail et al. [88], in their numerical study, also found similar findings. They even indicated that the contribution of masonry infills to the reduction of seismic pounding could be exploited by adopting a smaller seismic gap.

Despite this, the columns of infilled frames subjected to slab-to-column pounding still have their shear requirements exceeding the existing capacity [89]. In the same vein, the numerical results of Favvata and Karayannis [90] illustrated that these shear requirements might be large to the extent of surpassing those of bare frames.

This prompts to now delve into the discussion regarding pilotis. Pilotis, or open storeys, can exacerbate seismic pounding between buildings due to their lower stiffness compared to upper storeys. This stiffness mismatch can amplify collision forces, leading to increased damage in columns, shear walls, and connections. In their numerical study, Manoukas and Karayannis [8] meticulously explored the seismic interaction between reinforced concrete buildings, focusing notably on structures with open first storeys (pilotis) and the impact of asymmetric pounding. They demonstrated that pilotis configurations significantly amplified the overall structural response, particularly evident in floor rotations which could increase by up to almost ten times. Notably, asymmetric pounding, both slab-to-slab and slab-to-column pounding, exacerbated torsional vibrations and induced shear failures in columns, even surpassing maximum shear strength by significant margins. Their findings underscored the critical importance of considering these factors in seismic design and retrofitting strategies. A prior investigation by the same authors [91] explored the significant influence of infill panels on how structures respond during earthquake-induced pounding events. This study distinguished itself by examining the impact of pilotis configuration compared to structures with fully infilled frames. Additionally, it investigated the influence of the direction of seismic excitation on structural response, particularly regarding the shear behaviour of columns subjected to collisions. Furthermore, Karayannis and Naoum [6] studied seismic-induced interaction between adjacent reinforced concrete buildings, focusing on asymmetric pounding, where one building was taller and symmetric while the other was shorter and asymmetric. They found that this interaction induced significant torsional oscillations, especially as the height of the shorter structure increased. The severity of pounding was influenced by the period of the adjacent structure, with stiffer adjacent structures inducing higher torsional moments. This asymmetric pounding led to high shear forces in columns, altering ductility requirements, particularly increasing the demands on columns experiencing high displacement due to rotational movement. Moreover, Ambiel et al. [92] conducted a study focusing on earthquake-induced pounding within the framework of safety-related devices in nuclear plants. They demonstrated the effectiveness of the explicit CD-Lagrange scheme, a computational method grounded in non-smooth contact dynamics, in accurately capturing high-frequency phenomena. Initial defects were introduced to simulate asymmetric pounding and it was observed that eccentric pounding transient computation provided more precise acceleration spikes compared to symmetric pounding computation, which tended to overestimate response spectra due to abrupt acceleration spikes. Notably, the explicit CD-Lagrange approach obviated the need for contact parameters, and the number of time steps employed did not compromise the relevance of the results. The authors underscored the significance of appropriately tuning the Rayleigh viscous damping matrix to mitigate spurious frequencies.

The potential for torsional pounding in buildings with asymmetrical configurations was also acknowledged in the work of Karayannis and Naoum [57, 58], Fiore et al. [93], and Wei et al. [94] with emphasis placed on its ability to exacerbate collisions and elevate demands on displacement, shear, torsion, and ductility. Rajaram and Kumar [59] studied the torsional pounding of two reinforced concrete buildings, numerically, with different setback levels. They found that the impact force was directly proportional to the setback level. Many other studies also dealt with torsional pounding [57, 93–96]. They generally reported that torsional pounding results in an increase in collision numbers, storey displacements, and shear and torsion requirements.

A critical aspect of pounding analysis involves the meticulous simulation of the nonlinear behaviour exhibited by members within interacting structures. Of particular significance are beam-column joints, pivotal structural components that significantly influence the seismic response of framed buildings and serve as points of impact between them. For this reason, Karayannis et al. [97] investigated the impact of exterior joint capacity degradation on the failure mechanisms of reinforced concrete buildings. Employing a specialised rotational spring element with a tailored behaviour model, the study assessed the influence of exterior joint damage on the seismic behaviour of bare and infilled framed buildings, including infilled frames without infills at the base storey (pilotis frame). It was demonstrated that neglecting the local damage of exterior joints could yield erroneous conclusions and compromise safety in design and seismic evaluations. Notably, they emphasised that the degradation of exterior joints significantly affects the behaviour of pilotis frames, which underscored the importance of considering the response of exterior beam-column joints in understanding failure mechanisms and ensuring structural safety. Favvata et al. [98] studied the seismic-induced interaction between multistorey buildings with unequal storey heights, focusing on interstorey pounding while considering the local response of exterior beam-column joints. The results revealed that while the local inelastic response of exterior joints could sometimes benefit the seismic behaviour of the impacted column, it generally led to increased demands for joint deformation and severe damages due to pounding.

Moreover, the presence of masonry infill panels emerged as a crucial factor influencing the response of exterior beam-column joints and the overall safety of the building. However, despite the presence of infills, excessive demands for shear and ductility of the impacted column persisted in all examined interstorey pounding cases, indicating the need for further mitigation measures.

The damage index serves as a crucial metric and common method for assessing the structural integrity of buildings following seismic events. It provides a quantitative measure of the extent and severity of damage incurred by structural elements. So, it offers valuable insights into the structural performance and vulnerability of buildings [99]. Therefore, Jeng and Tzeng [100], based on a field survey, determined damage indices for existing buildings in Taipei City and used them to classify the damage level of the buildings located there. Hosseini et al. [101] conducted a study on nonlinear damage detection in adjacent reinforced concrete buildings considering seismic pounding effects using three different damage indices. The results indicated that pounding between the buildings led to the occurrence of nonlinear damage at lower seismic intensities. Increasing the separation distance generally decreased the damage index, while shorter buildings experienced more significant damage due to pounding, with higher values of damage indices observed in shorter buildings with smaller fundamental periods. Yang et al. [102] determined that the extent of damage from pounding does not diminish entirely as the separation distance between buildings increases. This conclusion was drawn from their examination of the variations in the damage index of a certain beam-column element within the buildings.

In almost all studies, earthquake components are applied to buildings in orthogonal directions that are either parallel or perpendicular to their sides. Nonetheless, Polycarpou et al. [103] followed another way by numerically studying the effect of the ground motion orientation angle on the pounding-induced response of adjacent buildings. They emphasised that the arbitrary direction of the earthquake should be considered in pounding-related simulations as it may result in a four-fold greater response. Unfortunately, the angle wherein the response was maximally amplified was not always constant; it was dependent on the properties of the building as well as the ground motion.

Moreover, most studies in the literature dealt with the horizontal components of the earthquake. But Hatzigeorgiou and Pnevmatikos [104] numerically examined the effect of the vertical component on the seismic pounding of adjacent buildings. The vertical component was reported to have a minor effect on the global response of colliding buildings. In contrast, it meaningfully affected the local damage to structural members.

Also, distinctive studies considered other parameters that might affect seismic pounding. For instance, Abdel-Mooty et al. [78] revealed that the cracking of reinforced concrete cross-sections considerably influenced the pounding forces. So, cross-sections should be modelled with an effective moment of inertia in lieu of a gross moment of inertia. Mouzakis and Papadrakakis [78] studied the effect of friction forces that are induced during impact, despite the fact that this latter did not receive much attention in previous investigations. They concluded that the effect of friction forces on the flexible building was significant, whereas it was negligible for the stiff building. Additionally, Maniatakis et al. [44, 45] demonstrated that response spectrum analysis is conservative for seismic pounding simulations. Consequently, nonlinear time history analysis should be used instead. Neglecting the P-delta effect was also found to underestimate the pounding-induced forces, according to Kazemi et al. [105] and Mohebi et al. [106].

While previous studies predominantly relied on time-history analysis to evaluate building response during seismic pounding, a recent shift has seen the adoption of fragility analysis as a complementary and powerful tool. Unlike the detailed, event-specific nature of time-history analysis, fragility analysis takes a probabilistic approach. It estimates the likelihood of a building exceeding a pre-defined damage state given a specific level of seismic excitation. This shift from deterministic assessment to probabilistic evaluation offers several advantages. Firstly, it acknowledges the inherent uncertainties in both ground motion and structural behaviour. Secondly, it allows for the development of fragility curves, which visually depict the relationship between seismic intensity and damage probability. These curves serve as valuable tools for decision-making, enabling engineers to assess the vulnerability of buildings across a range of potential seismic scenarios and prioritize mitigation efforts accordingly. In essence, fragility analysis provides a more comprehensive and nuanced understanding of building performance under seismic pounding, moving beyond single-event simulations to assess the overall resilience of structures to earthquake threats.

Sinha and Rao [107] recently investigated methodologies for developing fragility curves to assess the seismic performance of adjacent reinforced concrete buildings experiencing structural pounding. Their study focused on displacement-based fragility curves for critical slab-to-column interactions between an eight-storey frame and a three-storey frame with varying storey heights. The fragility curves were generated using multiple approaches, with the High Dimensional Model Representation method emerging as highly suitable due to its consistent accuracy and remarkable computational efficiency compared to standard approaches. This method is recommended for accurate fragility curve generation and the subsequent estimation of pounding risks in adjacent reinforced concrete buildings. Flenga and Favvata [108] carried out a similar study considering five different methodologies for constructing fragility curves. The study revealed discrepancies among methods.

Adjacent six- and nine-storey moment-resisting steel frames were numerically analysed by Yazdanpanah et al. [109] to implement different approaches for fragility curves development. It was demonstrated that that fragility curves obtained via the methods that consider contribution of higher modes, exhibited lower damage probabilities and greater efficiency compared to the estimated fragility curves based solely on the approaches that considered only the period of the first mode. Additionally, owing to the pounding phenomenon, the six-storey moment-resisting frame experienced a higher likelihood of damage.

Liu et al. [110] proposed a reliability-based method for assessing seismic pounding fragility and risk of nonlinear adjacent buildings using subset simulation. The method was tested on multi-storey reinforced concrete buildings with varying storey heights and separation distances, compared to an incremental dynamic analysis-based method. The study found that slab-to-column pounding cases generally had higher fragility than slab-to-slab cases, which agrees with the findings of Mohamed and Romão [111] who attributed this to the shear failure that occurs in the columns. For larger separation distances, the incremental dynamic analysis-based method yielded larger fragility estimates than the subset simulation-based method. However, for small separation distances, the difference between the fragility curves was minor. It was therefore suggested that the larger fragility estimate of the two methods could be used for practical purposes.

Bantilas et al. [112] used fragility analysis to investigate the impact of pounding on the seismic behaviour of multi-storey reinforced concrete buildings. It was indicated that pounding led to higher seismic demands, particularly with taller adjacent buildings, smaller separation gaps, and stronger ground motions.

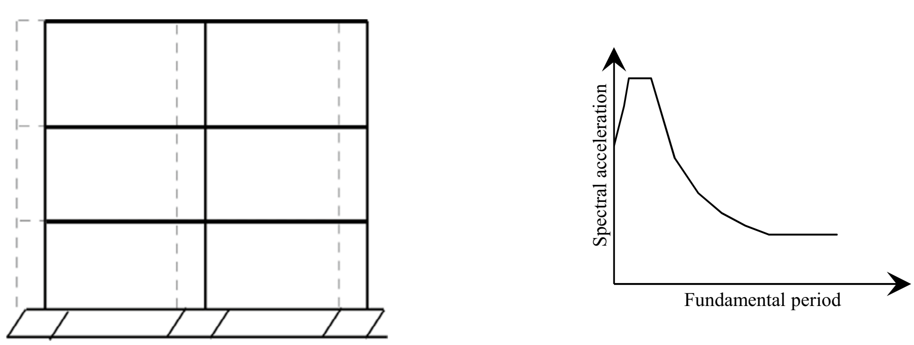
Flenga and Favvata [113] investigated the influence of earthquake magnitude and distance to rupture on the fragility assessment of an eight-story reinforced concrete frame experiencing slab-to-slab structural pounding. They indicated that variations in earthquake magnitude and distance to rupture led to shifts in fragility curves, impacting the likelihood of exceeding performance levels, especially with the distance to rupture.

Ebrahimiyan et al. [114] focused on the seismic pounding effects of neighbouring reinforced concrete buildings with series arrangements and different heights (number of storeys). Comparing results to single building analyses, they found that the series arrangement significantly affected collapse capacity. Moreover, the fragility analysis showed improved performance levels for certain configurations, particularly in arrangements with ascending height order, while reduced performance levels were evident when shorter buildings were between taller ones. They noted that even when allowable separation distances were considered, the impact of arrangement on performance remained significant.

### 4.2 Buildings with Base Isolation Systems

To protect structures from earthquake ground motions, several resisting methods have been proposed in past research. One of these methods represents the installation of a base isolation system between the superstructure and the foundations [115], as shown in Fig. 4a. Since the superstructure becomes isolated from its foundation, its fundamental period is increased. This causes a drop in the spectral acceleration in accordance with a typical response spectrum (see Fig. 4b). Therefore, a base-isolated building is subjected to lower interstorey drifts and storey shear despite experiencing large displacements at the isolation level. However, these large displacements at the isolation level may lead to seismic pounding between the base-isolated building and its surroundings. These surroundings might be another base-isolated building, a fixed-base building, or a moat wall [116].

Based on the numerical studies in Refs. [117–119], isolated buildings exposed to pounding with moat walls experienced an increase in their acceleration, interstorey drift, and base shear. Each of these response parameters was found to be sensitive to the flexibility of the isolation system. As




(a) base-isolated building (b) typical response spectrum

**Fig. 4** Base isolation system for a typical building the isolation system got more flexible, these parameters increased.

The experimental results of Masroor and Mosqueda [120] also showed that impact forces were dependent on the stiffness of the moat wall into which the base-isolated building collide. It was, as well, deduced by Bao and Becker [121], based on their numerical study, that a stiff isolated building pounded with a moat wall had higher impact forces and ductility requirements compared to a corresponding flexible isolated building.

To evaluate the effect of seismic pounding location on the response of isolated buildings, Polycarpou et al. [122] carried out a numerical study on two cases as follows: (a) a building without a basement in which pounding occurred at its base, and (b) a building with a basement that was subjected to pounding at its base in addition to the first storey. In each case, lead rubber bearings were adopted as a base isolation system. The second case yielded higher acceleration accompanied by lower displacement than the first.

The numerical work of Mavronicol et al. [123] concentrated on pounding between a building isolated with laminated rubber bearings and moat walls. The orientation angle of the ground motion was found to noticeably affect the pounding-induced response of the building. Thus, it should be considered to appropriately select the adequate gap size. More recently, Mavronicola et al. [124] also confirmed the same observations for buildings with lead rubber bearings.

To clarify the difference between the pounding of isolated buildings with moat walls and with fixed-base buildings, Polycarpou et al. [125–127] conducted a numerical study in which the isolated building was installed with rubber isolators. It was thus seen that pounding an isolated building with a moat wall was less dangerous than pounding a fixed-base building. This was because impact occurred at the isolation level only in the case of moat walls, while it occurred at the isolation level as well as in storeys in fixed-base building cases. On this basis, the required gap between an isolated building and a moat wall was less than that between an isolated building and a fixed-base building.

Mahmoud and Janowski [128] numerically examined the pounding-induced response of isolated and fixed-base buildings. The isolated buildings were installed with high-damping rubber bearings. They illustrated that if an isolated building collided with another isolated or fixed-base building, the storey response of the isolated building would be nearly constant along its height. Meanwhile, the fixed-base building had its storey response vary throughout its height.

In the numerical work of Pant and Wijeyewickrema [129], the performance of an isolated building, a fixed-base building, and a moat wall subjected to slab-to-column pounding was assessed. The adopted base isolation system was the lead rubber bearing. It was noted that columns of the buildings failed due to flexure rather than shear, even though the shear requirement was amplified because of the slab-to-column pounding.

The numerical investigation of Uz and Hadi [130] presented an evaluation of the pounding-induced response of two isolated buildings with different weights. They reported that the response of the lighter building was considerably influenced compared to the heavier one. This may cause permanent deformation of that lighter building even after the excitation comes to an end.

To compare distinctive base isolation systems, Liu et al. [131] studied the pounding of buildings with lead rubber bearing isolators and friction pendulum isolators numerically. They indicated that friction pendulum isolators caused a higher amplification of the pounding-induced response of the building compared to lead rubber bearing isolators.

The study conducted by Masroor and Mosqueda [132] used fragility analysis to examine the collapse probability of different base-isolated reinforced concrete moment-resisting framed and concentrically braced framed buildings considering pounding to moat wall. The results showed that, in the absence of the moat wall, the concentrically braced framed building had a more conservative collapse probability in comparison to the moment-resisting framed building. However, considering moat walls in the collapse evaluation analysis changed these collapse margin ratios considerably.

### 4.3 Buildings with Soil-Structure Interaction Considered

Though all the studies discussed earlier have neglected the effect of soil, several investigations have considered the effect of soil-structure interaction on seismic pounding. The reason behind this is that neglecting soil-structure interaction is only valid for rock soils. Other types of soils, particularly soft ones, notably influence the pounding-induced response of buildings [133].

Buildings are located on different soil strata that have variable properties. So, they evidently alter the characteristics of the ground motion that passes through the strata to reach the ground surface [134]. This, in turn, modifies the seismic response of buildings. Accordingly, the interaction between the building and the soil beneath it should be taken into account [135].

During earthquakes, the underlying soil moves into translational motion. This, of course, causes the foundations of the building to also translate; thus, demolishing the general idea that buildings are restrained at base level. Consequently, the stiffness of the building is actually lower, in the case of considering soil-structure interaction, than the original assumption [136].

Naserkhaki et al. [137] performed numerical analysis on the pounding-induced response of adjacent buildings, considering soil-structure interaction. Impact forces were reduced in the case of considering soil-structure interaction compared to the fixed base assumption. Nevertheless, this corresponded to higher storey displacement, which required a larger seismic gap. It was also found that the pounding process had a short duration and low magnitude if the colliding buildings had comparable heights. Unless the colliding buildings had an insignificant difference in height, the pounding process lasted for a longer duration. Moreover, seismic pounding was most critical in the case where the height of one building was half that of the adjacent building. These findings are consistent with those reported by Ghandil and Aldaikh [138], who also investigated the damage index of adjacent buildings considering soil-structure interaction and revealed that the damage index exhibits a greater sensitivity compared to conventional structural response parameters to seismic events, underscoring its importance in structural assessment and analysis. Furthermore, the numerical simulations found in [138–141] illustrated that the response of flexible buildings was more sensitive to seismic pounding than stiff buildings if soil-structure interaction was considered.

Fatahi et al. [142] numerically tested seismic pounding of adjacent buildings resting on piles. Thus, a particular form of soil-structure interaction, called soil-pile-structure interaction, was considered. They recommended that the combined effect of seismic pounding and soil-pile-structure interaction should be considered, in practice, upon the design of adjacent buildings, as it had an evident effect on the results.

To study the effect of soil type on the response of buildings prone to seismic pounding, Miarai and Jankowski [143] conducted shaking table experimental tests on two adjacent steel buildings with different separation distances under the effect of different earthquakes that were scaled such that their response spectra match up with the response spectra found in [144] for different soil types (hard rock, rock, etc.). It was demonstrated that soil type significantly affected the response of the adjacent buildings. Nevertheless, the results for different soil types varied depending on the scenario of pounding and ground motion. So, it was concluded that there is no specific soil type that amplifies the response of the buildings when seismic pounding takes place. The same authors [145] also investigated the impact of soil type on three buildings with varying heights (4, 6, and 8 storeys) subjected to earthquake-induced pounding using incremental dynamic and fragility analyses. Different soil types were incorporated as per [144] that ranged from hard rock to soft clay. It was revealed that while generally detrimental, pounding can sometimes mitigate damage under specific circumstances. In addition, the fragility analysis revealed a stark connection between soil type and the likelihood of damage. The softer the soil, the more vulnerable the building was to exceed various performance levels, with probabilities reaching 100% at higher ground motion intensities. Conversely, buildings on firmer soil exhibited significantly lower susceptibility, even under strong quakes. This trend held true across all studied scenarios, highlighting the critical role of soil type in pounding vulnerability. In simpler terms, buildings founded on soft clay soil were most susceptible to pounding damage, followed by stiff soil, very dense soil/soft rock, and rock/hard rock.

Furthermore, Shakya et al. [146] confirmed that near-fault earthquake ground motions were more influential than far-fault earthquake ground motions. This is compatible with other studies that have neglected soil-structure interaction (see Sect. 4.1). The results indicated that considering soil-structure interaction caused a decrease in storey displacement and storey shear while causing an increase in storey acceleration. This is consistent with the study conducted by Mahmoud et al. [139]. On the other hand, the results of other studies showed that the pounding-induced response of the buildings was amplified when soil-structure interaction was considered [137, 138, 147–150].

Elwardany et al. [151] took on a rarely addressed topic focusing on the effect of soil-structure interaction on seismic pounding between steel buildings with or without masonry infills. It was demonstrated that considering soil-structure interaction increased the flexibility of buildings with or without masonry infills. For instance, omitting soil-structure interaction in the case of buildings with masonry infills was accompanied by no interference between the buildings, and no pounding-involved response was observed. Taking soil-structure interaction into account altered this observation, resulting in seismic pounding and a significant increase in the pounding-induced response.

In general, these contradictory results may be a result of soil type, as it affects the pounding-induced response of the buildings. Typically, utilising flexible soil results in an increase in storey displacement, whereas stiff soil leads to a decrease in storey displacement [116].

For base-isolated buildings, Mahmoud and Gutub [152] studied their response in the case of pounding with moat walls while considering soil-structure interaction. The base isolation system adopted in the studied buildings was the rubber bearings. They detected that considering soil-structure interaction led to an increase in storey displacement, storey acceleration, and the number of collisions. Meanwhile, the isolated base was subjected to lower acceleration. Also, they reported that the response of the building increased for soft soils.

Recently, Naseri et al. [153] investigated a rarely addressed issue. It is, typically, the effect of the earthquake ground motion duration on the seismic pounding of adjacent buildings. Of course, soil-structure interaction was considered. It was obviously seen that as the duration of the seismic action got longer, the impact forces increased. Moreover, the models subjected to longer excitations were found to require a wider seismic gap.

## 5 Mitigation Measures

As it has been seen throughout the previous sections, seismic pounding has a dangerous effect on adjacent buildings. Thus, several mitigation techniques have been proposed in the literature to reduce this effect. These mitigation measures mainly aim to limit impact forces, thus keeping the whole building relatively safe. Generally, mitigation measures can be classified into four categories, as follows:

(a) Providing a sufficient seismic gap.
(b) Installation of impact-absorbing materials.
(c) Installation of earthquake-resisting systems.
(d) Coupling of buildings.

Discussions on each method of mitigation are introduced in the upcoming subsections.

### 5.1 Seismic Gap Distance

In fact, providing a sufficient seismic gap between adjacent buildings does not only mitigate seismic pounding; it completely eliminates it. So, the seismic gap should be large enough to account for the peak displacements of each building. The results of Gong and Hao [71], and Jameel et al. [49] revealed that enlarging the seismic gap did not have a major effect on pounding unless the buildings were adequately separated. Nonetheless, this is contradictorily to the results reported by Kamel [154], and Abdel-Mooty et al. [78] who indicated that the pounding force and number of collisions are significantly sensitive to the change in the seismic gap distance. In general, the results revealed that amplifying the seismic gap distance by eightfolds caused average change in the peak pounding force and number of hits by 32 and 93%, respectively. This illustrates the high sensitivity of the number of collision to the seismic gap distance compared to the peak pounding force.

Consequently, several studies have provided distinctive formulas to predict the required seismic gap between adjacent buildings. In addition, many seismic codes have dealt with the same topic. However, there are various techniques that can be adopted to determine the sufficient seismic gap. The absolute sum of the peak displacements, the square root of the sum of the squares of the peak displacements, and the double difference method (complete quadratic combination method) all represent examples of seismic gap calculation techniques. Table 1 summarises the formulae provided by several worldwide code of practice for seismic gap distance calculations. Throughout this section, these formulae are extensively discussed alongside other formulae suggested in the literature.

To calculate the sufficient seismic gap using the absolute sum of the peak displacements method, the following formulas can be used:

$$S = U_1 + U_2 \tag{1}$$

where $S$ denotes the required seismic gap, while $U_1$ and $U_2$ denote the seismic peak displacement of each building when separately analysed.

That previous formula has been used in numerous codes, such as the UBC [155]. Nonetheless, since it is not likely that both buildings reach their peak displacements at the same time, this method is regarded as conservative; it overpredicts the seismic gap.

Accordingly, the seismic gap was proposed by Anagnostopoulos [39] to be calculated based on the method of square root of the sum of the squares of the peak displacements as follows:

$$S = \sqrt{{U_1}^2 + {U_2}^2} \tag{2}$$

This equation was widely adopted in later prints of the UBC [155], FEMA 356 [156], EN 1998–1 [157], ECP 201 [158], NBCC [159], ASCE/SEI 7–10 [160], and IBC [161], due to its relatively better accuracy compared to Eq. (1). Note that in EN 1998–1 [157], the terms $U_1$ and $U_2$ are taken as $qd_c$, where $q$ is the behaviour factor provided in [157], while $d_c$ is the storey displacement, as calculated through a linear analysis using the design response spectrum. EN 1998–1 [157] also allows to reduce Eq. (2) by a 30% in the case of slab-to-slab pounding. ECP 201 [158] typically follows the same approach of EN 1998–1 [157] except for the terms $U_1$ and $U_2$, as they are taken herein as 0.7 $Rd_c$, where $R$ is the response modification factor reported in ECP 201 [158]. Otherwise, ECP 201 [158], as well, permits to reduce Eq. (2) by 30% in the case of slab-to-slab pounding. On the other hand, in ASCE/SEI 7–10 [160] and IBC [161], the terms $U_1$ and $U_2$ are replaced with the inelastic maximum response displacements ($U_{M,1}$ and $U_{M,2}$) which can be calculated for each building as the following:

$$U_M = \frac{C_d \Delta_{max,e}}{I} \tag{3}$$

where $C_d$, $\Delta_{max,e}$, and $I$ represent deflection amplification factor, maximum elastic displacement, and importance factor, respectively.

Yet, the numerical work of Pantelides and Ma [162], and Kumar and Kumar [163] indicated that this technique was still conservative and somehow overpredicted the required seismic gap. On the other hand, this technique became unconservative, and it underpredicted the required seismic

**Table 1** Requirements of worldwide codes of practice for seismic gap distance

| Code of practice | Seismic gap distance | Comments |
|---|---|---|
| Early prints of UBC [155] | $U_1 + U_2$<br>$U$: seismic peak displacement | It is improbable that both structures will experience their maximum displacements simultaneously, making this approach considered conservative as it tends to overestimate the seismic gap distance |
| Later prints of UBC [155], FEMA 356 [156], and NBCC [159] | $\sqrt{U_1{}^2 + U_2{}^2}$<br>$U$: seismic peak displacement | This approach is conservative and somewhat tends to overestimate the seismic gap distance |
| EN 1998–1 [157] | $\sqrt{\left(q_1 d_{c1}\right)^2 + \left(q_2 d_{c2}\right)^2}$<br>$q$: behaviour factor, $d_c$: storey displacement | Seismic gap distance can be multiplied by a reduction factor of 0.7 in the case of slab-to-slab pounding |
| ECP 201 [158] | $\sqrt{\left(0.7R_1 d_{c1}\right)^2 + \left(0.7R_2 d_{c2}\right)^2}$<br>$R$: Response modification factor | Seismic gap distance can be multiplied by a reduction factor of 0.7 in the case of slab-to-slab pounding |
| ASCE/SEI 7–10 [160], and IBC [161] | $\sqrt{U_{M,1}{}^2 + U_{M,2}{}^2}$<br>$U_M = \frac{C_d \Delta_{max,e}}{I}$, $C_d$: amplification factor, $\Delta_{max,e}$: elastic displacement, $I$: importance factor | The seismic peak displacements ($U_1$ and $U_2$) are replaced with the inelastic maximum response displacements ($U_{M,1}$ and $U_{M,2}$) |
| Taiwan building code [172] | $0.6\left(\Delta u_1 + \Delta u_2\right)$<br>$\Delta u = 1.4\alpha_y R \Delta e$, $\alpha_y$: amplification factor, $R$: allowable ductility factor, $\Delta e$: elastic displacement | This approach depends on the absolute sum of the peak displacements method but the seismic gap distance is reduced herein by 40% as it is rare for neighbouring buildings to experience their maximum displacements simultaneously |
| AS 1170.4 [173] | $0.01H_{max}$<br>$H_{max}$: the maximum of the adjacent buildings heights | This approach also suffered from being conservative as it overpredicts the seismic gap |
| Iranian code [174] | $0.05\left(h_1 + h_2\right)$<br>$h$: height of the building | Relates the seismic gap distance to the buildings' heights |

gap if the adjacent buildings had different structural systems [164], or they were analysed in the near-collapse limit state [165].

To predict seismic gap more accurately, Jeng et al. [166] developed the double difference method (also known as the complete quadratic combination), as shown in the following formula:

$$S = \sqrt{U_1{}^2 + U_2{}^2 - 2\rho U_1 U_2} \tag{4}$$

where $\rho$ is a factor that accounts for the uncertainties during seismic pounding. It is determined as follows:

$$\rho = \frac{8\sqrt{\xi_1\xi_2}\left(\xi_2 + \xi_1\frac{T_2}{T_1}\right)\left(\frac{T_2}{T_1}\right)^{1.5}}{\left[1 - \left(\frac{T_2}{T_1}\right)^2\right]^2 + 4\xi_1\xi_2\left[1 + \left(\frac{T_2}{T_1}\right)^2\right]\left(\frac{T_2}{T_1}\right) + 4\left(\xi_1{}^2 + \xi_2{}^2\right)\left(\frac{T_2}{T_1}\right)^2} \tag{5}$$

where $\xi_1$ and $\xi_2$ represent damping ratios of the buildings, whereas $T_1$ and $T_2$ are the fundamental periods of the buildings ($T_1 < T_2$).

Lopez-Garcia [167] investigated the accuracy of the double difference method in predicting seismic gap. He reported that the prementioned method was inaccurate in the case that the adjacent buildings had convergent fundamental periods. Later, Lopez-Garcia and Soong [168] extended the study using different scenarios affecting the accuracy of the double difference method. The observations were consistent with the findings in Refs. [169–171].

Shretha [175] made a comparison between the seismic gaps predicted by the absolute sum method, the square root of the sum of the squares method, and the double difference method against the analytically predicted seismic gap. It was demonstrated that the double difference method surpassed the absolute sum of the peak displacements method and the sum of the squares of the peak displacements method in terms of seismic gap prediction accuracy. Barbato and Tubaldi [176], as well, figured out similar findings.

The method embedded in Taiwan building code [172], which is dependent on the absolute sum of the peak displacements method, takes inelastic deformations into account as follows:

$$S = 0.6\left(\Delta u_1 + \Delta u_2\right) \tag{6}$$

where $\Delta u_1$ and $\Delta u_2$ are the inelastic displacements of the buildings that can be determined based on Eqs. (7) and (8). It is noted that the absolute sum of the peak displacements is reduced herein by 40% since adjacent buildings seldom reach their peak displacements at the same time, as previously discussed.

$$\Delta u_1 = 1.4\alpha_y R_1 \Delta e_1 \tag{7}$$

$$\Delta u_2 = 1.4\alpha_y R_2 \Delta e_2 \tag{8}$$

where 1.4 is the overstrength factor, $\alpha_y$ is an amplification factor, $R_1$ and $R_2$ denote the allowable ductility factors, and $\Delta e_1$ and $\Delta e_2$ denote the elastic displacements of the buildings when subjected, separately, to seismic loading. However, the studies of Lin and Weng [177], and Lin [178] showed that the formula of Taiwan building code [172] was conservative and it overpredicted the required seismic gap.

The AS 1170.4 [173] followed another approach to determine the seismic gap by relating it to the heights of the adjacent buildings, as shown below:

$$S = 0.01H_{max} \tag{9}$$

where $H_{max}$ is the height of the tallest building. Nevertheless, this formula also suffered from being conservative as it overpredicts the seismic gap [179, 180]. Therefore, a modified formula was suggested in [180] as follows:

$$S = \sqrt{{U_1}^2 + {U_2}^2 - 2SFU_1U_2} \tag{10}$$

where $SF$ is a separation factor that can be determined by the following formula:

$$SF = \left(\frac{T_2}{T_1}\right) - 10.5\left(T_2 - T_1\right) \tag{11}$$

where $T_1 < T_2$.

Similar to AS 1170.4 [173], the Iranian code [174] provided a formula that relates the seismic gap to the buildings height as follows:

$$S = 0.05\left(h_1 + h_2\right) \tag{12}$$

where $h_1$ and $h_2$ are the adjacent buildings' heights.

GB50011-2001 [181] on the other hand, requires the seismic gap to be not less than 70 mm for adjacent buildings shorter than 15 m. The seismic gap correspondingly increases by 20 mm for each increase in building height in accordance with different seismic intensity levels.

It is worth mentioning that all the formulas discussed in this section are concerned with fixed-base buildings. For base-isolated buildings, a larger seismic gap is required. The adequate gap in this case might be up to three times the one predicted by the formulas in this subsection [138, 149, 150].

Even though providing an adequate seismic gap is considered the optimum solution to disallow seismic pounding, it does not usually appeal to buildings' owners. This is because they do not often prefer to sacrifice regions within their property line to provide the required seismic gap, especially due to financial issues and the high cost of land. Hence, other mitigation measures are favourable. Furthermore, providing seismic gap is not applicable in the case of existing buildings that need retrofitting to survive seismic pounding.

### 5.2 Impact-Absorbing Materials

Another approach to avoid the detrimental effects of seismic pounding is the placement of layers of soft materials between adjacent buildings to absorb shocks. The idea behind this approach is identical to that adopted in marine platforms, wherein soft bumpers are installed at the wharfs to evade the damage caused by docking ships [182].

The incorporation of polystyrene absorbing material between adjacent buildings was experimentally and numerically evaluated by Rezavandi and Moghadam [183, 184]. They pointed out that polystyrene can efficiently reduce the storey acceleration of the buildings.

Polycarpou et al. [182], and Takabatake et al. [185] numerically studied the incorporation of rubber shock absorbers between adjacent buildings. It was indicated that this particular type of impact-absorbing material can severely reduce the impact force. As the softness of the absorbing material increases, the impact force decreases.

Sołtysik et al. [186, 187] conducted an experimental study on adjacent buildings with polymeric-absorbing material between them. They revealed that the incorporation of polymers is capable of effectively reducing seismic pounding. Moreover, they recommended the absorbing material to completely fill the gap between the buildings since this case yielded the lowest pounding-induced response.

A novel polymer-metal composite material was proposed by Stręk et al. [188] to mitigate seismic pounding. This material consisted of polyurethane and closed-cell aluminium foam. The experimental results were found to be promising. However, further experimental testing is still required.

Khatami et al. [189] installed rubber bumpers between the storeys of adjacent buildings. They examined bumpers with different shapes and dimensions. This is to determine the optimum ones that can effectively reduce seismic pounding and absorb impact forces. They found that the thickness of the rubber bumpers greatly affects the amount of impact force absorbed and energy dissipation. In addition, bumpers with a circular cross-section performed better than those with a square cross-section in absorbing impact forces. Furthermore, this study evaluated structural damage using equations for calculating the damage index, revealing significantly lower damage indices for buildings equipped with rubber bumpers compared to those without, underscoring the significant influence of rubber bumpers on structural response to seismic excitation.

Although all the previous studies were involved with fixed-base buildings, several investigations in the literature addressed the effect of impact-absorbing materials on the seismic pounding mitigation of base-isolated buildings, such as the studies of Komodromos [118], and Polycarpou and Komodromos [190]. It was implied that impact-absorbing materials were able to reduce impact forces in spite of the increase in the number of collisions. This increase is attributed to the softness of the materials.

According to Anagnostopoulos [39], filling the gap between adjacent buildings with different materials surely limits the effect of seismic pounding. Nevertheless, this method is not as efficient as the installation of earthquake-resisting systems, whose role is to limit lateral deformations of buildings. As a consequence, this brings us to the next category of seismic pounding mitigation measures: the use of earthquake-resisting systems.

### 5.3 Earthquake-Resisting Systems

This method of seismic pounding mitigation relies on the implementation of certain elements in the building that are responsible for reducing lateral displacements that the storeys undergo. On this basis, the possibility that adjacent buildings collide is decreased, despite not being fully hindered. However, even if the adjacent buildings collide into each other, their storeys move lower distances because of the installed earthquake-resisting systems. Accordingly, the corresponding momentum, which is directly proportional to the distance that a moving body travels, is reduced. This, in turn, results in the mitigation of seismic pounding. Earthquake-resisting systems can be classified into two categories: (a) conventional protection systems and (b) innovative structural control systems [99].

Conventional protection systems involve adding structural walls, cores, or concentric braces to the building or adopting columns with large cross-sectional dimensions. So, conventional protection systems rely on increasing the strength and stiffness of the building in order to resist the acting earthquake ground motions and decrease the lateral displacements. In contrast, innovative structural control systems are typical devices that are installed into the building for the purpose of adjusting its dynamic properties rather than strength and stiffness, therefore guaranteeing a reduction in the seismic response. These innovative structural control methods can be either active, semi-active, passive, or hybrid. Several subsidiary devices fall under each of these prementioned groups.

Active control systems are computer-based devices that are installed into the building to transform it into a smart building. These active control systems are entirely adaptive. Put in another way, they perceive the surrounding earthquake loading by means of sensors. Then, they adjust the response of the whole building in order to tolerate the acting loads without excessive deformations. Note that these systems require a huge power source to operate. This is to be able to generate the needed control force that decreases structural response [191, 192]. Active tuned mass dampers and distributed actuators are among the most well-known active control systems [193].

Semi-active control systems work in a similar way to active control systems. Nevertheless, the most disadvantageous trait of the active control devices is disposed of herein. In particular, semi-active control systems operate only on battery power; there is no need for a huge power source [194]. Semi-active control systems include magneto-rheological dampers, semi-active stiffness dampers, etc. [193].

Nowadays, passive control systems are the most reliable method of structural control. This is because they do not require any external power source at all to operate. So, they are simple and can be practically used without many precautions [195]. Nonetheless, it should be kept in mind that one of the drawbacks of passive systems is their inadequate adaptivity to varying excitation since they are not equipped with any sensors, nor do they operate on external power sources. Such a disadvantage, though, can be justified by the ease of manufacturing and the relatively low price of these systems, as only mechanical devices are used [192]. Passive control systems include base isolation systems and energy dissipation devices [196]. However, conventional base isolation systems are not considered a seismic pounding mitigation technique due to the circumstances discussed in Sect. 4.2. Other special base isolation systems that can mitigate seismic pounding have been developed in several previous studies [197–202]. On the contrary, all conventional energy dissipation devices can be utilised for seismic pounding mitigation. Energy dissipation devices encompass numerous devices that can be used to control seismic response. Energy dissipation devices can be classified as displacement-dependent dampers (like metallic dampers and friction dampers), velocity-dependent dampers (like fluid viscous dampers and viscoelastic dampers), or dynamic vibration absorbers (like tuned mass dampers and tuned liquid dampers) [203].

As the name suggests, hybrid control systems combine active, semi-active, or passive control systems together in the same building to make use of the features of each of them. Accordingly, each individual system participates in resisting the actions induced by the earthquake ground motion [194]. Hybrid systems are cost-effective, and they are characterised by sufficient functionality and reliability [192]. Semi-active tuned liquid dampers with passive dampers and hybrid mass dampers represent examples for hybrid control systems [204].

In the numerical investigations of Jamal and Vidyadhara [205], and Abhina and Nair [206] structural walls were found to be efficient in the mitigation of seismic pounding. Hameed et al. [207] also detected that the structural walls can limit the out-of-phase deformations of adjacent buildings as well as concentric braces, thus mitigating seismic pounding. Furthermore, Barros and Khatami [208] pointed out that although structural walls can significantly reduce storey displacements and the number of collisions, they cause the impact force to increase. This may be attributed to the increase in the stiffness of the building, as discussed earlier in this subsection.

Hameed et al. [179] assessed the performance of adjacent buildings installed with Pall friction dampers. They showed that the adopted friction dampers can reduce storey displacement more than structural walls and concentric braces. So, they can be considered an efficient technique to mitigate pounding-induced damage.

Adding to this, fluid viscous dampers have been widely proposed to mitigate seismic pounding. Kazemi et al. [209] reported that fluid viscous dampers can decrease impact force and duration caused by seismic pounding. Hence, they delay the collapse of the buildings in which they are installed. Elwardany et al. [210] also numerically investigated the effect of fluid viscous dampers on seismic pounding mitigation. They illustrated that despite fluid viscous dampers significantly mitigate seismic pounding, they are subjected to a higher peak force in comparison with the design independent force. Hence, the local response of the elements surrounding the fluid viscous damper should be carefully analysed in the design process. Kazemi et al. [211] illustrated that fluid viscous dampers can be utilised to reduce the collapse possibility of adjacent buildings subjected to pounding.

Tuned mass dampers were also addressed in several studies in the literature to mitigate seismic pounding [212, 213]. They were found to reduce peak storey displacement. Tuned mass dampers were then modified via the addition of viscous material; their responsibility is to dissipate energy during pounding [214–218]. In comparison with conventional tuned mass dampers, the modified ones proved to be more efficient in mitigating seismic pounding.

As mentioned before, special base isolation systems can be installed in buildings to mitigate seismic pounding. The roll-in-cage isolators were presented in the extensive work of Ismail et al. [197–201]. They indicated that this type of base isolation systems can effectively mitigate both translational pounding and torsional pounding as well. Agarwal et al. [198] investigated friction-varying base isolators, and they showed that these isolators were capable of mitigating seismic pounding under certain circumstances. Likewise, Mazza and Laberanda [219] assessed the ability of concave surface sliders to mitigate seismic pounding of adjacent reinforced concrete buildings that were irregular in the plan.

### 5.4 Coupling of Buildings

As described in Sect. 1, seismic pounding mainly occurs as a result of the out-of-phase vibrations of adjacent buildings. For that, adjacent buildings can be connected together by means of special connections, as shown in Fig. 5, that transmit forces between them, which can be exploited to mitigate out-of-phase vibrations and compel the buildings to vibrate in-phase [220]. This method of seismic pounding mitigation is called building coupling. In general, the results in [221–223] illustrated that the pounding-induced response of a flexible building can be noticeably decreased by coupling it to a stiff building. Yet, the response of the stiff building was only slightly affected.

This technique was first introduced in 1972 by Klein et al. [224] to stabilise wind-induced oscillations in structures. Four years later, Kunieda [225] was the first to use the coupling concept to enhance the seismic response of two adjacent single-degree-of-freedom structures. Later, in 1980, Miller [226] investigated the resonance of coupled buildings subjected to seismic pounding. These early publications represented the pioneering first steps in the development of the coupling technique to mitigate seismic pounding.

The connection between coupled buildings can either be rigid or energy-dissipative [220, 227]. Rigid connections depend on their strength and stiffness to couple adjacent buildings with each other, therefore fully or nearly fully synchronising their response. So, they are intended to keep responding within the elastic stage during the seismic event without exhibiting any kind of inelastic behaviour. On the other hand, energy-dissipative connections are designed to synchronise the response of adjacent buildings to some extent while dissipating some quantity of input energy through the induced inelastic actions. Energy-dissipative connections are classified as active, semi-active, passive, or hybrid [220].

Rigid connections, in their simplest form, can be stiff beams [228] or stiff links [229]. Another alternative to stiff

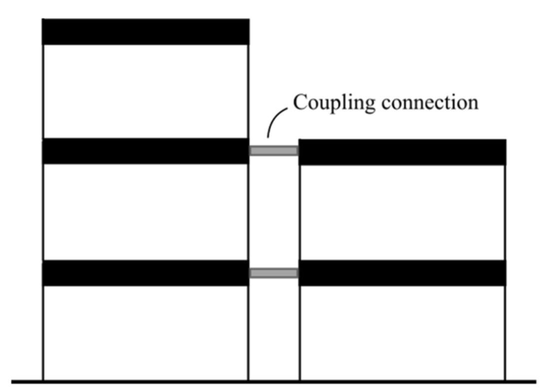



**Fig. 5** Two adjacent buildings coupled together

beams is the sky bridge connection [230]. On the other hand, energy-dissipative connections include a wide variety of devices, such as those listed in Table 2. It is noticeable that most of the structural control systems discussed in Sect. 5.3 can be used to couple adjacent buildings. As with previously discussed structural control systems, passive energy dissipation connections are the most used and studied in the literature [220]. Accordingly, this type of energy dissipation connections will be of interest throughout this subsection.

The coupling of adjacent buildings using passive energy dissipation connections causes them to behave as a single building. In general, the equation of motion for that coupled building is as follows:

$$[M]\{\ddot{u}(t)\} + [C]\{\dot{u}(t)\} + [K]\{u(t)\} = -[m]\{r\}\left\{\ddot{u}_g(t)\right\} \quad (13)$$

where $\ddot{u}(t)$, $\dot{u}(t)$, and $u(t)$ are the relative acceleration, velocity and displacement concerning the base, while $\ddot{u}_g(t)$ is a dimensionless vector, with dimensions $(n+m) \times 1$, that represents the earthquake ground motion. Note that $n$ and $m$ are the degrees of freedom of the adjacent buildings. Additionally, $[M]$, $[C]$, and $[K]$ represent the matrices of mass, damping, and stiffness, respectively. The aforementioned matrices have dimensions of $(n+m) \times (n+m)$ and they can be obtained as the following:

$$[M] = \begin{bmatrix} [M]_1 & [0]_{m\times n} \\ [0]_{n\times m} & [M]_2 \end{bmatrix} \quad (14)$$

$$[K] = \begin{bmatrix} [K]_1 & [0]_{m\times n} \\ [0]_{n\times m} & [K]_2 \end{bmatrix} + \begin{bmatrix} [k_d] & [-k_d] & [0]_{m\times n-n} \\ [-k_d] & [k_d] & [0]_{m\times n-n} \\ [0]_{m-n\times n} & [0]_{m-n\times n} & [0]_{m-n\times n-n} \end{bmatrix} \quad (15)$$

$$[C] = \begin{bmatrix} [C]_1 & [0]_{m\times n} \\ [0]_{n\times m} & [C]_2 \end{bmatrix} + \begin{bmatrix} [c_d] & [-c_d] & [0]_{m\times n-n} \\ [-c_d] & [c_d] & [0]_{m\times n-n} \\ [0]_{m-n\times n} & [0]_{m-n\times n} & [0]_{m-n\times n-n} \end{bmatrix} \quad (16)$$

in which $[M]_1$, $[M]_2$, $[K]_1$, $[K]_2$, $[C]_1$, and $[C]_2$ are the matrices of mass, stiffness, and damping for each of the adjacent buildings, respectively. Also, $[k_d]$, and $[c_d]$ are the stiffness and damping matrices for the dampers that can be expressed by the following equation:

$$[k_d] = [k_{d_1}, k_{d_2}, \dots, k_{d_i}] \quad (17)$$

$$[c_d] = [c_{d_1}, c_{d_2}, \dots, c_{d_i}] \quad (18)$$

where $k_{d_i}$ and $c_{d_i}$ are the stiffness and damping of the $i$ th damper, respectively.

As for $\{r\}$, it is a dimensional ground acceleration–mass transformation vector of the size $(n+m) \times 1$ that can be obtained as follows:

$$\{r\} = \begin{bmatrix} 1 & 1 & \cdots & 1 \end{bmatrix}^T \quad (19)$$

Connecting adjacent buildings with stiff beams was studied by Westermo [228] to avoid seismic pounding. More recently, Kamel [229] used reinforced concrete stiff beams to connect adjacent buildings with unequal heights. He, as well, deduced that the stiff beams were better placed in the top storey of the shortest building instead of the lower storeys in terms of pounding force mitigation.

Furthermore, Ni et al. [239] analysed the seismic response of adjacent buildings connected with yielding metallic dampers, and they implied that there was no need to install those dampers between all storeys of the buildings. Moreover, several investigations were conducted to optimally design adjacent buildings connected with yielding metallic dampers [249–251]. In addition, Sama and Gur [252] demonstrated that shape memory alloy hysteretic dampers surpassed yielding metallic ones in terms of storey displacements and acceleration reduction.

On the other hand, Bhaskararao and Jangid [240, 253, 254] extensively investigated friction dampers as a coupling system. They found that it could sufficiently reduce the seismic response of the buildings. Based on the experimental

**Table 2** Examples for each type of energy dissipation connections between coupled buildings

| Energy dissipation connection type | Examples |
|---|---|
| Active | Active tuned mass dampers [231], active negative stiffness devices [232], active viscous fluid dampers [233], and actuators [234] |
| Semi-active | Magneto-rheological dampers [235], semi-active friction dampers [236], semi-active viscous dampers [237], and semi-active variable stiffness dampers [238] |
| Passive | Metallic dampers [239], friction dampers [240], lead extrusion dampers [241], fluid viscous dampers [242], viscoelastic dampers [243], shared tuned mass dampers [244], and tuned liquid dampers [245] |
| Hybrid | Magnetorheological dampers with tuned mass dampers [246], actuators with base isolation systems [247], and fluid viscous dampers with tendon-type active control devices [248] |

tests of Ng and Xu [255], the friction coupling dampers can usefully reduce the seismic response of adjacent buildings compared to rigid connections.

Patel [241] followed a similar approach by connecting adjacent buildings using lead extrusion dampers. The results illustrated that this approach was able to mitigate the seismic response of coupled buildings, even if not all storeys were linked together.

Opposed to metallic dampers, friction dampers, and lead extrusion dampers, numerous publications that are related to velocity-dependent dampers, such as viscoelastic dampers and viscous dampers, are found in the literature. The latter, particularly, had the lion's share of the publications.

Zhu and Iemura [256] utilised viscous dampers to couple adjacent buildings. Moreover, Patel and Jangid [257], and Tubaldi et al. [176] carried out extensive investigations on the efficiency of viscous dampers in mitigating seismic pounding. They found that, generally, viscous dampers can efficiently limit seismic pounding. They, as well, demonstrated that there was no need to connect all the storeys of adjacent buildings together. For instance, only connecting half of the stories, according to [257], mitigated seismic pounding satisfactorily. Another application of viscous dampers, in which they were installed at the isolation level of base-isolated buildings, was presented by Polycarpou and Komodromos [125], and Abd-Elsalam et al. [258].

In addition, Roshan et al. [259] compared between viscous and viscoelastic coupling dampers in terms of seismic pounding mitigation. Both coupling dampers were seen to effectively mitigate seismic pounding. However, viscoelastic dampers slightly surpassed viscous dampers. Anyway, viscoelastic coupling dampers did not represent a value for money as the enhancement in seismic pounding mitigation was not consistent with their extra cost.

Soreci and Terenzi [260] adopted a similar approach wherein viscous dampers were placed in the gap between adjacent buildings. Similarly, Pratesi et al. [261, 262] carried out a case study on seismic pounding between the existing bell tower and church building of Chiesa del Sacro Cuore in Florence. They noticed that adjacent buildings were significantly vulnerable to seismic pounding. Consequently, they made a comparison between retrofitting the existing buildings with linking viscous dampers and rigid interconnection. Both retrofitting techniques managed to mitigate seismic pounding. Yet, viscous dampers exhibited a better effect as they additionally decreased axial forces, shear forces, and bending moments acting on the stressed columns. Moreover, Pratesi et al. [261, 262] adopted a one-meter-long viscous damper, but the gap between the buildings was not wide enough to accommodate it. Accordingly, they suggested demolishing around 1.6 m long of the nave walls to provide sufficient space for the viscous dampers to be housed in. This additional 0.6 m contains a reinforced concrete block that was connected to the steel plate at the end of the fluid viscous damper.

Karabork [263] carried out a numerical optimization analysis to determine the optimum damping of coupling viscous dampers for the sake of pounding avoidance. Moreover, Tubaldi et al. [264] suggested a simplified design strategy for coupling viscous dampers.

Jankowski and Mahmoud [223] compared between buildings coupled with springs, dashpots, and viscoelastic dampers. They observed that viscoelastic dampers were able to decrease the maximum storey displacements compared to springs and dashpots. As a result, a smaller seismic gap could be adopted.

Uppari and Chandrashekar [265] utilised coupling viscoelastic dampers to control seismic vibrations in adjacent buildings. They demonstrated that diagonal coupling viscoelastic dampers were more efficient than horizontal ones in mitigating the seismic response of the buildings. They also pointed out that coupling viscoelastic dampers need not be installed between all storeys of adjacent buildings; optimal placement of them can lead to better performance. In a slightly different study, Taleshian et al. [266] utilised viscoelastic dampers to mitigate seismic pounding between adjacent asymmetric-plan buildings. Adjusting the damping of the viscoelastic damper significantly affected the decrease in the displacements of adjacent buildings. This decrease reached 90% for certain damping values.

Kangda and Bakre [267] used viscous dampers to couple a fixed-base building with a base-isolated building equipped with lead rubber isolators. This combined mitigation technique was found to be efficient.

Kazemi et al. [268] evaluated the effect of coupling reinforced concrete and steel moment-resisting frames with viscous dampers. The existence of viscous dampers was seen to efficiently reduce impact forces during seismic pounding.

Kangda and Bakre [269] investigated the optimum location of coupling viscous dampers between adjacent buildings. They revealed that there was no need to install coupling viscous dampers between all colliding storeys. Installation of them at only the top colliding storey was sufficient to mitigate seismic pounding.

In a more recent study, Asgarkhani et al. [270] installed coupling viscous dampers between adjacent steel and reinforced concrete moment-resisting framed buildings. The results clarified that this technique can significantly enhance the capacity of buildings. Moreover, an optimal retrofit strategy was presented to mitigate seismic pounding between the prementioned buildings.

Licari et al. [271] innovated a novel coupling technique that consists of a damper that is connected in parallel with a spring. Thereby, this technique had varying damping with respect to time. They also detected that this technique was characterised by a considerable storey shear reduction, thus efficiently mitigating seismic pounding.

Dynamic vibration absorbers have also been investigated in early studies. For instance, instead of utilising a separate tuned mass damper for each building, as shown in Sect. 5.3, Abdullah et al. [272] developed the shared tuned mass damper; it is a single tuned mass damper that is installed into one of the buildings and connected to the adjacent one by means of a spring. They illustrated that the shared tuned mass dampers were better than conventional tuned mass dampers in terms of seismic pounding mitigation. However, this is contradictory to the findings in Refs. [244, 273].

Additionally, Wang et al. [274] used tuned liquid dampers to connect two adjacent high-rise buildings. The results implied that this method was capable of decreasing the seismic response of the buildings. Nevertheless, the efficiency of this method was dependent on the fundamental periods and frequencies of both buildings.

## 6 Impact Analytical Models

Since theoretical studies are more feasible than experimental tests from a time and cost point of view, many impact analytical models have been developed in the literature to accurately evaluate and simulate seismic pounding between structures. The general notion of impact modelling mainly relies on the interaction between the two colliding bodies before and after collision; mathematical formulas are thus used to describe the response of the colliding bodies.

To better understand impact models, it is first necessary to recognise the different phases associated with the collision process. Referring to Fig. 6, different phases of collision between two random bodies are illustrated. At the beginning, the two bodies translate towards each other with initial velocities, causing the separation distance to get smaller. Later, the two bodies get closer until the separation distance reaches zero. At this moment, the impact becomes imminent. So, this stage is called the approach phase. The end of this phase occurs when the relative velocity between the two bodies reaches zero. Beyond this point, the two bodies separate from each other, thus marking the beginning of another phase that is called the restitution phase. Next, the prementioned phase lasts till the two bodies completely get away from each other with final velocities that may differ from the initial velocities before impact [37].

Although numerous impact analytical models have been developed in the literature, as pointed out previously, they fundamentally follow one of two approaches. These approaches are typically the stereomechanical approach or the force-based (penalty) approach. Each of these two approaches is discussed in the following subsections.

### 6.1 The Stereomechanical Approach

The stereomechanical approach is considered one of the classical approaches to determine the post-impact velocity of colliding bodies. This approach is particularly based on the principle of momentum conservation [37, 116]. This principle states that the total momentum of two bodies colliding in an isolated system before impact is identical to their momentum after impact. In other words, the momentum lost by the first body is equal to that gained by the second body. Hence, the stereomechanical approach is only concerned with the relationship between the post-impact and pre-impact velocities; it does not pay attention to the direct estimation of impact forces. The post-impact velocities ($v_1{}'$ and $v_2{}'$) and pre-impact velocities ($v_1$ and $v_2$) can be related to each other as follows [275]:

$$v_1{}' = v_1 - (1+e)\frac{m_2 v_1 - m_2 v_2}{m_1 + m_2} \tag{20}$$

$$v_2{}' = v_2 - (1+e)\frac{m_1 v_1 - m_1 v_2}{m_1 + m_2} \tag{21}$$

where $m_1$ and $m_2$ denote the masses of the colliding bodies, while $e$ is the coefficient of restitution that can be defined as the square root of the ratio of the rebound distance ($h_2$) that a body travels upward after freefalling towards a rigid plate from a distance $h_1$. So, it describes the level of plasticity of the colliding bodies. For instance, if $e$ is equal to zero, this means that the collision is fully plastic. On the other hand, if $e$ is equal to unity, then the collision is fully elastic. That coefficient can be determined using either of the following formulas [116]:

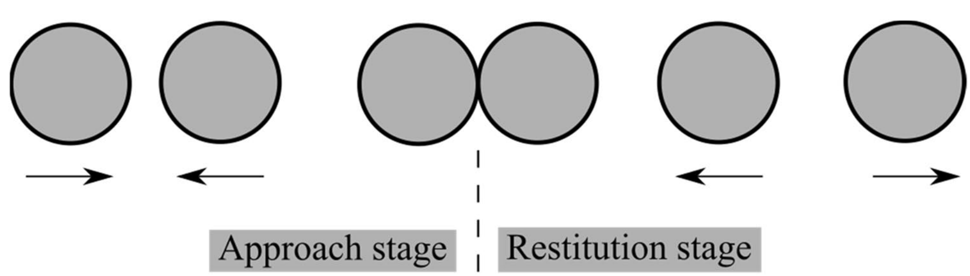



**Fig. 6** Different impact phases for two colliding bodies (direct central impact)

$$e^2 = \frac{h_2}{h_1} \tag{22}$$

$$e = \frac{v_2{}' - v_1{}'}{v_1 - v_2} \tag{23}$$

It is worth pointing out that the stereomechanical approach is seldom used in seismic pounding simulation, despite its precise theoretical base [116]. This is because that approach is mainly oriented towards the estimation of the post-impact velocities rather than accounting for complex interactions like material behaviour, energy dissipation, and structural damage caused by pounding events; it disregards the duration of impact (i.e., it is assumed to last for a negligible short time). Accordingly, the transformation of stresses and deformations between the bodies during impact is not considered. Besides, impact forces that are induced during contact cannot be directly estimated using that approach [37]. Furthermore, stereomechanical approach often do not accurately capture the full range of forces acting upon the buildings during pounding, leading to potentially misleading predictions about the extent of damage and the overall structural response [154]. To overcome these challenges, researchers have turned to more sophisticated methods and stochastic approaches, which better represent the intricate physics governing seismic pounding scenarios [37, 116].

## 6.2 The Force-Based Approach

Due to the limitations associated with the stereomechanical approach, the force-based approach (also known as the penalty approach) has been developed and extensively investigated. As its name suggests, this approach depends on the estimation of the impact forces that are induced during contact. Therefore, impact stiffness is taken into account. Various impact models fall under the force-based approach classification, such as:

(a) the linear spring model [116],
(b) the linear viscoelastic model (Kelvin-Voigt model) [39, 276],
(c) the modified linear viscoelastic models [117, 277, 278],
(d) the nonlinear Hertz model [70, 95, 279–281],
(e) the Hertzdamp model [282, 283],
(f) the nonlinear viscoelastic model [284–287], and
(g) the viscous elastoplastic model [288].

In the linear spring model [116], a spring connecting the two colliding bodies, as shown in Fig. 7, is added to take the impact stiffness into consideration. Note that the spring is activated only when the two bodies become in contact with each other. According to this mode, the impact force ($F(t)$) and the interpenetration distance ($\delta(t)$) can be expressed as follows:

$$F(t) = \begin{cases} k\delta(t) & \delta(t) > 0 \\ 0 & \delta(t) \le 0 \end{cases} \tag{24}$$

$$\delta(t) = u_1(t) - u_2(t) - d \tag{25}$$

where $k$ is the spring stiffness (which is equivalent to the impact stiffness at contact location), $u_1(t)$ and $u_2(t)$ are the displacements of the colliding bodies, and $d$ is the initial separation distance.

The aforementioned model has a serious flaw as it assumes the impact to be elastic. This means that both plastic behaviour and energy dissipation through collision are ignored. To overcome this drawback, Anagnostopoulos [39, 276] developed the linear viscoelastic model (also known as Kelvin-Voigt model). In this model, a damper (dashpot) is added in parallel with the linear spring to include energy dissipation into calculations, as shown in Fig. 8. This model can be expressed by the following equation:

$$F(t) = \begin{cases} k\delta(t) + c\dot{\delta}(t) & \delta(t) > 0 \\ 0 & \delta(t) \le 0 \end{cases} \tag{26}$$

$$c = 2\xi\sqrt{k\frac{m_1 m_2}{m_1 + m_2}} \tag{27}$$

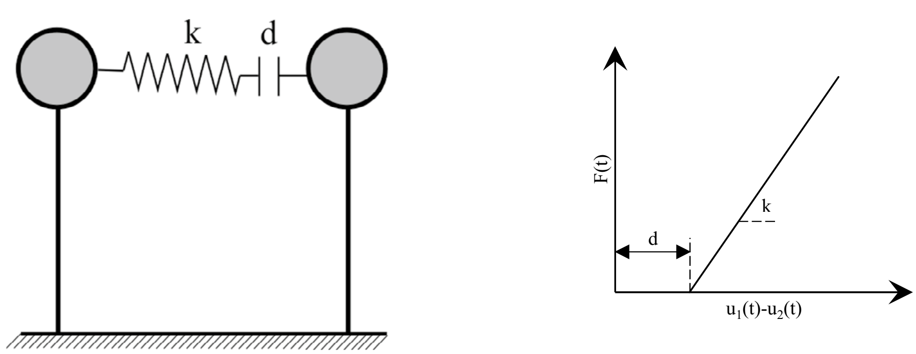



**Fig. 7** Schematic diagram of the linear spring model and its force–displacement relationship

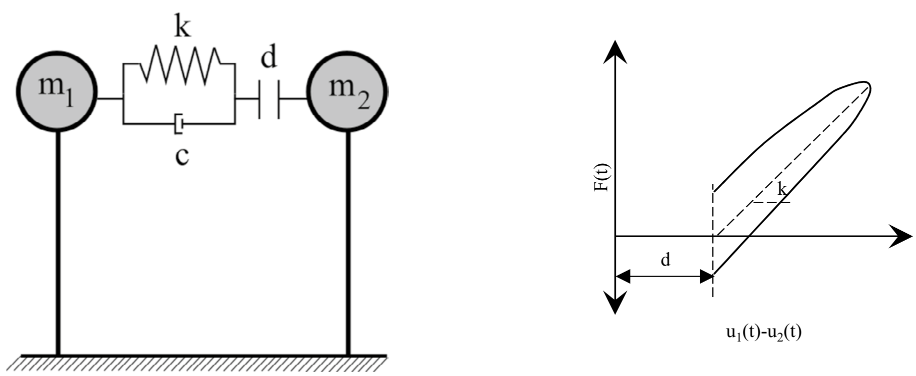



**Fig. 8** Schematic diagram of the linear viscoelastic model (Kelvin-Voigt model) and its force–displacement relationship

$$\xi = -\frac{\ln e}{\sqrt{\pi^2 + (\ln e)^2}} \tag{28}$$

where $\dot{\delta}(t)$ denotes the relative velocity between the colliding bodies, $c$ is the impact damping coefficient, while $\xi$ is the impact damping ratio.

The linear viscoelastic model, in its original form, has a certain disadvantage; definitely, it takes energy dissipation into account during both approach and restitution phases. This causes the development of tensile forces at the end of the impact process, which does not make sense [116]. Consequently, many modified linear viscoelastic models have been proposed in the literature to evade this issue. For example, Komodromos et al. [117] suggested ignoring these tensile forces by forcing $F(t)$ to always be larger than unity. Another modified linear viscoelastic model was developed by Pant et al. [277] which can be determined as follows:

$$F(t) = \begin{cases} k\delta(t) + c(t)\dot{\delta}(t) & \delta(t) > 0 \, and \, \dot{\delta}(t) > 0 \\ k\delta(t) & \delta(t) > 0 \, and \, \dot{\delta}(t) \leq 0 \\ 0 & \delta(t) \leq 0 \end{cases} \tag{29}$$

$$c(t) = \xi\delta(t) \tag{30}$$

$$\xi = -\frac{3k(1-e^2)}{2e^2(v_1 - v_2)} \tag{31}$$

In the same manner, in order to eliminate the tensile forces, Mahmoud and Jankowski [278] also presented the following modified model:

$$F(t) = \begin{cases} k\delta(t) + c\dot{\delta}(t) & \delta(t) > 0 \, and \, \dot{\delta}(t) > 0 \\ k\delta(t) & \delta(t) > 0 \, and \, \dot{\delta}(t) \leq 0 \\ 0 & \delta(t) \leq 0 \end{cases} \tag{32}$$

$$\xi = -\frac{1-e^2}{e(e(\pi-2)+2)} \tag{33}$$

With respect to the impact damping coefficient ($c$) in the latter modified model, it is determined in accordance with Eq. (27). Khatami et al. [302], on the other hand, proposed another formula to calculate impact damping ratio ($\xi$) as follows:

$$\xi = -\frac{1-e}{e^{\alpha+0.204} + 3.351\pi e} e^{0.204} \tag{34}$$

where $\alpha$ equals 1.05 $e^{0.653}$. The comparative analysis in [302] confirmed the accuracy of the proposed formula since it was validated across different scenarios.

All the aforementioned models only consider linear stiffness during impact. This inspired plenty of researchers, such as Davis [279] and Chau et al. [70, 95, 280, 281], to include nonlinear stiffness in impact model. Thus, the nonlinear Hertz

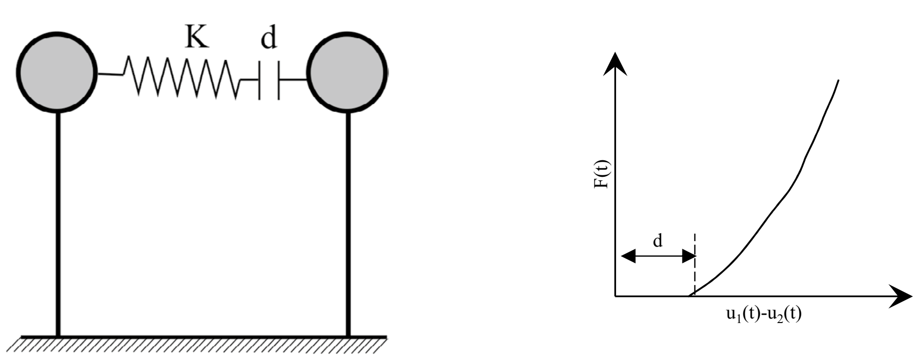



**Fig. 9** Schematic diagram of the nonlinear Hertz model and its force–displacement relationship

model has been developed, as shown in Fig. 9. The following formula can be used to obtain the impact force $F(t)$.

$$F(t) = \begin{cases} K\delta^{1.5}(t) & \delta(t) > 0 \\ 0 & \delta(t) \le 0 \end{cases} \tag{35}$$

where $K$ denotes the impact stiffness parameter.

Likewise, the Hertzdamp model, shown in Fig. 10, was developed by Muthukumar and DesRoches [282, 283] to include the nonlinear damping in the analytical expression as follows:

$$F(t) = \begin{cases} K\delta^{1.5}(t) + C(t)\dot{\delta}(t) & \delta(t) > 0 \\ k\delta(t) & \dot{\delta}(t) \le 0 \end{cases} \tag{36}$$

$$C(t) = \xi\delta^{1.5}(t) \tag{37}$$

$$\xi = \frac{3K(1-e^2)}{4(v_1 - v_2)} \tag{38}$$

where $C(t)$ is the impact damping parameter.

However, the theoretical work of Ye et al. [289, 290] demonstrated that utilising Eq. (38) to determine $\xi$ brings on unreliable results. As a result, they proposed a new formula to calculate $\xi$ as followw:

$$\xi = \frac{8K(1-e)}{5e(v_1 - v_2)} \tag{39}$$

Jankowski [284–287], as well, developed a nonlinear viscoelastic model to better model the nonlinear response during impact. This model can be described on the basis of the following formulas:

$$F(t) = \begin{cases} \overline{K}\delta^{1.5}(t) + \overline{C}(t)\dot{\delta}(t) & \delta(t) > 0 \; and \; \dot{\delta}(t) > 0 \\ \overline{K}\delta^{1.5}(t) & \delta(t) > 0 \; and \; \dot{\delta}(t) \le 0 \\ 0 & \delta(t) \le 0 \end{cases} \tag{40}$$

$$\overline{C}(t) = 2\xi\sqrt{K\sqrt{\delta(t)}\frac{m_1 m_2}{m_1 + m_2}} \tag{41}$$

$$\xi = \frac{9\sqrt{5}}{2} \bullet \frac{1-e^2}{e(e(9\pi - 16) + 16)} \tag{42}$$

where $\overline{K}$ and $\overline{C}(t)$ represent the impact stiffness parameter and the impact damping parameter, respectively. Subsequently, $\overline{C}(t)$ was modified by Naderpour et al. [291] as follows:

$$\overline{C}(t) = \alpha\frac{e^{\beta}(v_1 - v_2)(1-e)}{\ddot{\delta}(t)}\overline{K}\delta^{1.5}(t) \tag{43}$$

where $\ddot{\delta}(t)$ denotes the relative acceleration between colliding bodies, whereas $\alpha$ and $\beta$ are fitting parameters that have typical values of 0.01557 and 0.2706, respectively.

Khatiwada et al. [288] further modified the nonlinear viscoelastic model to consider elastoplastic behaviour. Thus, the viscous elastoplastic model was attained as follows:

$$F(t) = \begin{cases} \overline{K}\delta^{1.5}(t) + \overline{C}(t)\dot{\delta}(t) & \overline{K}\delta^{1.5}(t) + \overline{C}(t)\dot{\delta}(t) < F_E \; and \; \dot{\delta}(t) > 0 \\ F_E & \overline{K}\delta^{1.5}(t) + \overline{C}(t)\dot{\delta}(t) \ge F_E \; and \; \dot{\delta}(t) > 0 \\ \overline{K}\delta^{1.5}(t) & \overline{K}\delta^{1.5}(t) < F_E \; and \; \dot{\delta}(t) \le 0 \\ F_E & \overline{K}\delta^{1.5}(t) \ge F_E \; and \; \dot{\delta}(t) \le 0 \end{cases} \tag{44}$$

where $F_E$ represents the yield strength of the colliding element at the point of contact.

The impact models described earlier are only involved with 1-D and 2-D pounding problems. Hence, they neglect the friction in the direction perpendicular to the motion in spite of taking it in the direction of the motion into consideration. Accordingly, Polycarpou et al. [292–294] developed an extended impact model that is suitable for 3-D systems. Otherwise, alternative impact models have also been proposed, such as the equivalent linear impact model [295], sears impact model [296], and the nonlinear visco-elastoplastic model [297].

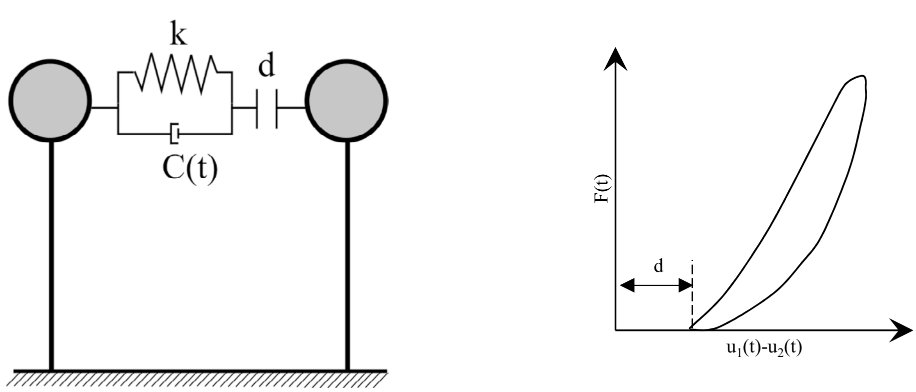



**Fig. 10** Schematic diagram of the Hertzdamp model and its force–displacement relationship

### 6.3 Parameters of the Impact Analytical Models

In the domain of impact analytical models, the parameters associated with the damping coefficient and spring stiffness garner significant attention due to their profound influence on the induced impact forces generated during pounding phenomena. By meticulously selecting judicious values for these parameters, the model's capacity to accurately capture critical localized effects is demonstrably enhanced. These effects encompass inelastic flexural deformations, yielding of flexural reinforcement, the inherent ductility of materials, and the potential for shear brittle failure within the impacted zone.

Many formulas have been developed in order to attain the stiffness-related parameters for the models described before. One of the most straightforward approaches to determining the impact stiffness coefficient ($k$) is to relate it to the axial stiffness of the colliding bodies as follows [77]:

$$k = \frac{EA}{L} \tag{45}$$

where $E$ represents the modulus of elasticity, while $A$ and $L$ are the cross-sectional area and length of the colliding element in pounding direction, respectively.

A distinctive formula was suggested by Cole et al. [298] in which $k$ was related to the mass, coefficient of restitution, and impact period of the colliding elements ($t_c$) as follows:

$$k = \frac{\left(\frac{m_1 m_2}{m_1+m_2}\right)\left(\frac{\pi}{t_c}\right)^2}{1-\left(\frac{-\ln e}{\sqrt{\pi^2+(\ln e)^2}}\right)^2} \tag{46}$$

Xu et al. [299] conducted a numerical study to check the capability of the current formulas to accurately predict the impact stiffness coefficient. They demonstrated that the formulas in [78] and [298] gave inaccurate estimations of the impact stiffness coefficient. Thereby, they developed a new formula (Eq. (46)) which proved to be more accurate.

$$k = \frac{m_2}{m_1 + m_2} k_1 e^{\frac{2\ln e}{\pi}\sin^{-1}\frac{\pi}{\sqrt{\pi^2+(\ln e)^2}}} \tag{47}$$

where $k_1$ is determined by Eq. (44).

Adopting the correct value for the impact stiffness coefficient could be tricky in some particular cases. The reason is that the seismic behaviour of pounded buildings is unresponsive to high values of the impact stiffness coefficient [116].

Based on the numerical work of Ghandil and Aldaikh [138], as an example, the behaviour of pounded buildings is unresponsive to the impact stiffness coefficient if the latter exceeds $10^{10}$ N/m. On the other hand, the numerical work of Naserkhaki et al. [300] revealed that the impact stiffness coefficient is 50 to 100 times the lateral stiffness of the whole building. Furthermore, Jankowski [284], in experimental investigations, pointed out that the value of the impact stiffness coefficient ($k$) in the linear viscoelastic model is $9.35 \times 10^7$ N/m, while the impact stiffness parameter ($\overline{K}$) in the nonlinear viscoelastic model is $1.13 \times 10^9$. Nonetheless, these values deal exclusively with Jankowski's experimental tests [284]. So, they cannot be disseminated beyond these tests. Anagnostopoulos [39] investigated the effects of different values for the damping and stiffness coefficients of the impact analytical model on the pounding-induced response of single degree of freedom systems. The results demonstrated that the issue of building pounding can be investigated without necessitating precise estimations of the damping coefficient. In addition, a reduction of the stiffness coefficient yielded minimal impact, as the response amplifications resulting from pounding remained essentially unchanged across all scenarios. Anagnostopoulos [39] further illustrated that the previous remarks regarding the insensitivity of response to impact model properties pertain solely to displacements. Conversely, pounding-induced accelerations, and to a lesser extent, the corresponding velocities, are highly responsive to alterations in impact model properties, particularly changes in spring stiffnesses. These accelerations may result in structural damage but exhibit minimal influence on the displacement response of the colliding masses. This agrees with the findings of Mate et al. [301], which suggested that within a reasonable limit of the actual scenario, different impact analytical models can accurately anticipate the pounding response of closely situated buildings, contingent upon thorough investigation and appropriate utilisation of impact model properties. Nevertheless, this partially agrees only with the remarks of Karayannis and Naoum [57]. In this study, it was specified that the stiffness coefficient of the spring is conventionally presumed to be large. However, uncertainty prevails regarding the precise value of this spring, attributable to factors such as the undisclosed geometry of impact surfaces and the variability in material properties under differing impact velocities. So, Karayannis and Naoum [57] agreed with early literature studies suggesting that alterations in spring stiffness do not significantly affect system response [40, 65]. The work of Jaradat and Far [302] focused on determining the optimal stiffness coefficient values through numerical analysis on an existing experimental model, varying stiffness and evaluating its impact on pounding force magnitude and number of impacts. The results highlighted the number of impacts and maximum force as key factors for determining optimal stiffness range, presenting this range for the specific model used. This guidance aids in selecting realistic stiffness values for more accurate damage prediction, emphasising the importance of considering impact dynamics and experimental validation.

As to the coefficient of restitution, Jankowski [60] carried out experimental tests to find a formula that relates it to the pre-impact velocity ($v$). Accordingly, he developed the following formulas for different materials crashing into each other:

$$e = -0.0039v^3 + 0.044v^2 - 0.1867v + 0.72 \quad \textit{steel to steel impact} \tag{48}$$

$$e = -0.007v^3 + 0.0696v^2 - 0.2529v + 0.7929 \quad \textit{concrete to concrete impact} \tag{49}$$

$$e = -0.0043v^3 + 0.0479v^2 - 0.1971v + 0.7067 \quad \textit{timber to timber impact} \tag{50}$$

$$e = -0.004v^3 + 0.0474v^2 - 0.2116v + 0.8141 \quad \textit{ceramic to ceramic impact} \tag{51}$$

Regardless of the aforementioned detailed expressions for the coefficient of restitution, it has been assumed in many studies to be 0.65 without any calculations. This value corresponds to a damping ratio of 0.14 and 0.35 in the case of the linear and nonlinear viscoelastic models, respectively [138, 284]. Polycarpou and Komodromos [125] attributed this to the insensitivity of the results obtained from the analytical impact models to the coefficient of restitution in some cases. Nonetheless, the numerical analyses of Naderpour et al. [291] indicated that, in general, the impact forces are inversely proportional to the coefficient of restitution. In contrast, Anagnostopoulos [39] revealed that the displacements, in their theoretical work, are unresponsive to the coefficient of restitution, whereas the velocity and acceleration are entirely the opposite. It could be observed, as a consequence, that the results are contradictory and alter from one case to another.

In the recent work of Mosa et al. [303], the pounding forces were shown to be inversely proportional to the coefficient of restitution in the Hertz model. Also, they detected that the impact forces between adjacent buildings increased with the increase in the impact spring stiffness used in the simulation process.

### 6.4 Accuracy of Different Impact Analytical Models

Owing to the abundance of impact models, several studies have compared different impact models in terms of accuracy. For instance, Jankowski et al. [304–306] illustrated that the nonlinear viscoelastic model estimates the impact forces more accurately than the linear viscoelastic model, the nonlinear Hertz model, and the Hertzdamp model. However, with regard to the impact velocity, the Hertzdamp model is more precise than the other models.

In the same regard, Khatiwada et al. [307] investigated the pounding between adjacent portal frames both experimentally and numerically in order to assess the accuracy of different impact models in predicting the maximum displacements and forces developed due to pounding. The impact models under consideration were the linear viscoelastic, the modified linear viscoelastic, the nonlinear viscoelastic, the Hertzdamp, and the modified Hertzdamp models. The results showed that the Hertzdamp model was the most imprecise among the considered models. For the other models, they had a margin of error of up to 20% in predicting the displacements. Nevertheless, the linear viscoelastic model was more accurate than the other models, despite requiring less computational effort. On this basis, they recommended using the linear viscoelastic model for the simulation of seismic pounding after carrying out the necessary refinement. Yet, the conclusions of Khatiwada et al. [307] were contradictory to those of Jankowski et al. [304–306]. Another comprehensive comparison between different impact analytical models was carried out by Mate et al. [308] who investigated the behaviour of three adjacent multi degree of freedom buildings using six different impact models for impact simulation, namely linear spring model, linear viscoelastic (Kelvin-Voigt) model, modified linear viscoelastic (Kelvin-Voigt) model, nonlinear Hertz model, and Hertzdamp model. The findings revealed minimal change in peak displacement response across different simulation techniques, with exterior flexible buildings exhibiting unconventional shear force patterns. Linear spring stiffness models produced similar impact forces, while nonlinear models offer reduced pounding forces. Nonlinear techniques resulted in lower impact forces compared to linear techniques, but peak displacement remained similar across all models. Impact forces significantly increased spectral acceleration and spread peak spectral acceleration values over a longer structural period range. Furthermore, the study of Jaradat et al. [309] on the structural pounding between steel buildings post strong earthquakes. Five different impact analytical models were employed to capture pounding force, with parameters derived from experimental data. The used models were the linear spring, linear viscoelastic, Hertz, non-linear viscoelastic and Hertzdamp models. While the models tended to over-predict pounding response, they aligned closely with experimental results. Notably, the linear viscoelastic model exhibited the least variance and most accurate predictions, indicating its superiority for assessing pounding response in such scenarios.

More in-depth studies concerned with comparisons between the impact models were also presented in [277, 310, 311]. No final verdict, however, could be reported based on these comparative studies. The reason is that seismic pounding is a complex process that is affected by various parameters.

### 6.5 Applications

The comprehensive exploration of various impact analytical models in preceding discussions has greatly enhanced the capability to simulate adjacent building configurations, numerically and analytically, under seismic pounding. Consequently, theoretical studies necessitated the selection of the geometric modelling approach employed for neighbouring buildings. These buildings can be represented as stick masses, presenting single or multiple degrees of freedom buildings, 2-D (plane frames) models, or 3-D models. Furthermore, the pounding-induced response of such buildings can be analysed employing methods such as time-history analysis, incremental dynamic analysis, or reliability and fragility analysis. In summary, Table 3 encapsulates the methodological approaches adopted by select studies in the literature concerning these pertinent issues. While the stick mass model has found limited use in recent studies, primarily focused on time-history analysis, recent research has shown a preference for leveraging the increased detail and accuracy offered by 2-D (plane frame) and 3-D models. Notably, 3-D models have gained more traction compared to their 2-D counterparts, receiving extensive investigation through various analysis methods, including time-history, incremental dynamic, and reliability and fragility analyses. This shift suggests a growing interest in capturing the complex behaviour of structures using more sophisticated models, moving beyond the simplified approach of the stick mass model.

## 7 Concluding Remarks and Recommendations for Future Relevant Work

In this paper, an intensive review is conducted regarding most of the previous research that dealt with the seismic pounding phenomenon. On that basis, the most important remarks that could be drawn are as follows:

(a) The primary cause of seismic pounding is the insufficient separation distance between adjacent buildings since they experience repetitive collisions and usually exhibit out-of-phase vibrations during seismic events. Even for buildings with zero gap distance and similar dynamic characteristics, seismic pounding may still occur.

(b) In the case of pounding between a stiff building and a flexible building, the stiff one is usually not affected. In meantime, the flexible building usually has its response amplified. Moreover, although seismic pounding increase the storey displacements and accelerations of one of the colliding buildings, the vibration of the other building may, in some cases, be somehow blocked. In other words, one of the buildings may work on restraining the displacements that the stories of the other building undergo. However, numerous parameters have been seldom investigated in the literature despite noticeably affecting seismic pounding, such as the orientation angle of the earthquake ground motion and the vertical component of the seismic excitation. These parameters need further consideration.

(c) In numerical analyses, the bases of the buildings are assumed to be fixed, but this is only true if and only if the buildings are built on stiff soil (such as rock).

**Table 3** Methodological approaches adopted in most recent studies

| Performed analysis | Building model | | |
|---|---|---|---|
| | Stick mass | 2-D | 3-D |
| Time-history | Zhang et al. [312], Pote and Mate [313], Zhang and Zhang [314], Zhang et al. [315], Djerouni et al. [316], Kazemi et al. [317] | Sinha and Rao [107], Khatami et al. [189], Tena-Colunga and Sánchez-Ballinas [318], Yazdanpanah et al. [109], Cayci and Akpinar [319], Kazemi et al. [211], Mohebi et al. [320], Langlade et al. [321], Mazza and Labernarda [322] | Manoukas and Karayannis [8], Forcellini [323], Ambiel et al. [92], Isobe and Shibuya [324], Jiang et al. [325], Rayegani and Nouri [326], Kamal et al. [327], Kamal and Inel [328], Miari and Jankowski [145], Bodnar et al. [329], Kamal and Inel [330], Ambiel et al. [330], |
| Incremental dynamic | Kazemi et al. [317] | Sinha and Rao [107], Yazdanpanah et al. [109], Kazemi et al. [211], Mohebi et al. [320] | Forcellini [323], Rayegani and Nouri [326], Miari and Jankowski [145] |
| Reliability and fragility | Kazemi et al. [317] | Sinha and Rao [107], Yazdanpanah et al. [109] | Forcellini [323], Rayegani and Nouri [326], Miari and Jankowski [145] |

For buildings on softer soil, soil-structure interaction should be considered as it results in higher displacements of building storeys, therefore increasing the risk of seismic pounding.

(d) Some studies showed that the formulas provided by several codes of practice to determine seismic gap distance may either estimate or underestimate the required gap distance. Accordingly, more research related to this issue is required.

(e) Other than the seismic gap, there are a wide range of seismic pounding mitigation methods that differ in their operation mechanism and installation requirements, such as impact-absorbing materials, earthquake-resisting systems, and coupling techniques.

(f) The efficiency of alternative passive energy dissipation devices in mitigating seismic pounding should be addressed, such as shear links, friction dampers, buckling-restrained braces, etc. Although many passive energy dissipation devices have been addressed in the literature, the applicability of the aforementioned ones in mitigating seismic pounding has not been studied before.

(g) There are a variety of analytical models that can be used to simulate seismic pounding. However, different studies that compared these models demonstrated contradictory results for the pounding-induced response. So, in these models, the impact stiffness and coefficient of restitution need to be further investigated in order for them to be quantified for distinctive configurations of seismically pounding buildings.

(h) Despite the significant advancements in understanding seismic pounding through theoretical studies, future research efforts must prioritize comprehensive experimental testing to validate these findings and address remaining uncertainties. Real-world structures exhibit complexities beyond idealisations, and laboratory or field experiments can capture these intricacies and their influence on pounding behaviour. Such testing can illuminate the nuanced interplay between structural features, material properties, and earthquake ground motions, ultimately leading to more accurate design and mitigation strategies for structures susceptible to pounding. Additionally, experiments can guide the development of improved numerical models by providing crucial benchmarks for calibration and validation.

(i) The utilisation of artificial intelligence and machine learning-based approaches holds paramount importance in accurately predicting pounding forces and the resulting response of adjacent buildings during seismic events. These advanced methodologies offer significant potential for enhancing the understanding of structural dynamics and improving design strategies to mitigate pounding effects. It is strongly recommended that future studies explore these emerging techniques, as their application in this field remains relatively limited compared to traditional numerical and analytical methods, despite being featured in several early studies in the literature. Embracing these innovative methodologies can pave the way for more robust and efficient solutions to address the challenges posed by structural pounding in seismic regions.

(j) In addition to seismic forces, it is imperative to acknowledge that structural pounding may arise from other external influences, including wind, mining blasts, and surface blasts. While the current study primarily addresses seismic pounding, there exists a need for subsequent research endeavours to explore the ramifications of these alternative forces on structural pounding phenomena. Since pounding due to mining blasts was timidly investigated in early studies [72, 331], it is recommended that future investigations dedicate attention to these additional factors with the same scholarly rigor as seismic pounding, thus fostering a comprehensive comprehension of the multifaceted contributors to structural pounding and facilitating the development of comprehensive mitigation strategies.

(k) In the complex realm of seismic pounding, slab-to-column pounding deserves a spotlight. This specific form of collision intensifies the harmful effects of pounding. Unlike typical slab-to-slab pounding, slab-to-column pounding focuses immense forces onto smaller elements like columns. This results in magnified shear forces and ductility demands, which can easily overwhelm these members, triggering localised failures and potentially compromising the entire structure's integrity. Therefore, explicitly considering slab-to-column pounding scenarios in seismic design and retrofit strategies is crucial, as its neglection could lead to an underestimation of a building's true vulnerability during earthquakes.

(l) The presence of torsional pounding, prevalent in buildings with asymmetrical configurations, presents a significant concern during seismic events. Since it increases the frequency of collisions and heighten demands on displacement, shear, torsion, and ductility. These combined observations reveal the critical role of torsional pounding in amplifying the adverse effects of seismic pounding on adjacent buildings, necessitating careful consideration in design and assessment practices.

(m) Ensuring an adequate seismic gap between adjacent buildings not only alleviates seismic pounding but effectively eradicates it. Therefore, it is imperative that the seismic gap be sufficiently spacious to accommodate the peak displacements of each building. Findings from existing literature indicate that enlarging the seismic gap does not significantly influence pounding unless the buildings are adequately distanced from each other. However, this contrasts with findings presented in other studies, as it has been reported in some of them that expanding the separation distance between buildings changes the force of pounding experienced between them as well as the number of collisions.

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## Declarations

**Conflict of interest** The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.



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