Research Article

Collapse-Pounding Dynamic Responses of Adjacent Frame Structures under Earthquake Action

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There are a large number of adjacent buildings in practical engineering application. The structure will collapse and impact the adjacent structures once the weak column is destroyed under seismic action, and, finally, the earthquake damage is aggravated. The numerical simulation method is undoubtedly an important analysis method to study the whole collapse process due to the large volume and complex system of engineering structures. Therefore, the numerical simulation and analysis of the collapse of the structure and the study of the collapse mechanism of the structure can provide the basis for the anticollapse design of the structure and effectively reduce or even avoid the occurrence of such accidents, which has practical significance. There have been cases of adjacent structures impact in previous earthquakes. Much research has been carried out on the dynamic response of collapse. Moustafa and Mahmoud [1] evaluated the collapse probability of a planar RC frame structure using both static and dynamic nonlinear analyses and obtained that the most important parameters affecting the collapse fragility appear to be the concrete compressive strength and

1. Introduction

There are a large number of closely arranged multistory and high-story buildings in many cities. The gap size between these buildings is very small or even not. It is easy to suffer serious damage in the future earthquake due to the danger of pounding.

Local damage of the structure occurs under the action of strong earthquake, impact, explosion, and fire, it will form a certain degree of initial damage, which will cause the redistribution of internal force of the structure and even cause the damage of other parts, then produce chain reaction, and eventually lead to the collapse of part or the whole part of the structure. It will often cause a wide range of damage and serious loss of life and property once the collapse of the structure occurs.

The numerical simulation method is undoubtedly an important analysis method to study the whole collapse process due to the large volume and complex system of engineering structures. Therefore, the numerical simulation and analysis of the collapse of the structure and the study of the collapse mechanism of the structure can provide the basis for the anticollapse design of the structure and effectively reduce or even avoid the occurrence of such accidents, which has practical significance. There have been cases of adjacent structures impact in previous earthquakes.
Young's modulus for concrete. Luo et al. [2] researched the complete process of the typical lateral increment collapse modes like domino and failure mechanism of RC frame structures located at high seismic intensity region induced by Wenchuan and Yushu earthquakes; the experimental findings indicate that the complete collapse process can be divided into two parts, that is, horizontal increment collapse and vertically progressive collapse. Jia et al. [3] conducted the nonlinear infinite laminated element analysis of three reinforced concrete space frame structural models with different seismic precautionary and obtained that the slab-framed beam collaboration can increase the vertical progressive collapse capability. Zhang and Li [4] conducted a progressive collapse test of three 1/3-scaled reinforced concrete frames and established a refined solid finite element model to validate the test results; the results indicated that the collapse resistance of RC structures was substantially improved with the increase in the slab thickness and seismic design intensity. Cai et al. [5] researched the seismic collapse of frame structures by using the THUFIBER program of MSC Marc and obtained that the failure of the structure under the earthquake action is a layer failure mode. Much research has also been carried out on the dynamic responses of adjacent building structures. Yang et al. [6] calculated the seismic response of adjacent buildings considering collision, the influence of pounding to the building seismic response became slight with the increase of building mass, and the influence was larger to the lighter building. Polycarpou et al. [7] performed numerical simulations for the investigation of the effect of the angle of seismic incidence on the response of adjacent buildings, and the results revealed that it was essential to consider the arbitrary direction of the ground motion with respect to the structural axes of the simulated buildings, especially during pounding. Ghandil and Aldaikh [8] studied the impact problem of adjacent plane symmetric buildings under unidirectional earthquake; it was found that the floor shear and damage index changed significantly due to the serious earthquake impact of adjacent buildings. Mate et al. [9] comparatively studied two adjacent single-degree-of-freedom structures with and without pounding and found TMD reduced the pounding forces in the adjacent structures. Chenna and Ramancharla [10] studied pounding responses and impact effects on structures subjected to ground motions and found that the damage would be more for buildings with unequal heights and less for buildings with equal heights. Crozet et al. [11] studied the sensitivity of the response of pounding buildings with respect to structural and earthquake excitation parameters and found that the frequency ratio is the most important parameter as far as the maximum force and nonlinear demand are concerned. Luo et al. [12] established constitutive models of two unequal height adjacent ten-story and six-story reinforced concrete frame structures based on OpenSees software and obtained that when two adjacent buildings were not connected with a damper, the distance of adjacent structures was suggested to be 0.3 m under moderate and strong earthquakes and the distance of adjacent structures was suggested to be a specified value of 0.24 m under rare earthquakes. Nazri et al. [13] combined two four-story and two ten-story frames, regular and irregular, to represent nine different cases of structural pounding, and proposed a minimum spacing of 1 m for buildings in areas that experienced repeated earthquakes to avoid structural pounding. Bybordiani and Arici [14] investigated the interacting effects of adjacent buildings; it was obtained that, for identical low-rise structures, SSSI effects could be negligible, while for neighboring closely spaced high-rise structures or building clusters with a large stiffness contrast, the true seismic demands would be underestimated if SSSI effects were not considered. Miari et al. [15] discussed the pounding effect on buildings with fixed bases, isolated buildings, and buildings resting on soft soils and proposed aspects of a sufficient recommended gap and appropriate mitigation measures. Raheem et al. [16] formulated a numerical simulation to estimate the pounding effects on the seismic response demands of three adjacent buildings in series with different alignment configurations; it was concluded that the severity of the seismic pounding effects depends on the vibration characteristic of the adjacent buildings and the input excitation characteristic. Wu et al. [17] proposed a probabilistic performance-based procedure for determining the critical separation distance (CSD) with a designed pounding probability for adjacent buildings under different structural properties and seismic hazard sites and derived the specific CSDs with a target probability of pounding during the design life of the given adjacent buildings using a proposed piecewise fitting iterative search algorithm.

Although many researchers have investigated structure collapse responses and adjacent structure pounding responses, there is a lack of research on the dynamic responses of adjacent structures which collapse first and then impact. When the gap size of the adjacent structures is small, pounding will be caused in case of structure collapse under earthquake, and the above research did not involve this problem. Material nonlinearity and contact nonlinearity are considered in this paper and a three-dimensional numerical model of the adjacent unequal height frame structure is established based on LS-DYNA. It is assumed that weak columns exist in different floors of the higher structure, the collapse-pounding dynamic responses of the adjacent structures after the column failure are studied, and the influence of initial collapse position and gap size on dynamic response is discussed.

2. Nonlinear Contact Pounding

The pounding problem is dealt with using the penalty function method [18], the driven node is defined \( b_s \), the active node \( a_s \) closest to the driven node \( b_s \) is searched on the active surface, and the active surface elements associated with the active node are marked as \( A_1, A_2, A_3, \) and \( A_4 \), respectively.

When the driven node satisfies the following conditions, it can be ensured that the projection of the driven node \( b_s \) on the active surface does not fall on the boundary line of the element, and the driven node \( b_s \) does not coincide with the active node \( a_s \) [18]:

\[
\begin{align*}
&\text{when } \alpha_s \neq 1, \quad b_s \neq a_s, \\
&\text{and } \alpha_s \neq 3, \quad b_s \neq a_s, \\
&\text{and } \alpha_s \neq 4, \quad b_s \neq a_s.
\end{align*}
\]
\[
\begin{align*}
\mathbf{S}_i \times \mathbf{t} \cdot \left( \mathbf{S}_i \times \mathbf{S}_{i+1} \right) > 0, \\
\mathbf{S}_i \times \mathbf{t} \cdot \left( \mathbf{t} + \mathbf{S}_{i+1} \right) > 0,
\end{align*}
\]

where \( \mathbf{S}_i \) and \( \mathbf{S}_{i+1} \) are the two vector edges of the main plate on the active surface:

\[
\mathbf{t} = \mathbf{b} - \left( \mathbf{b} \cdot \mathbf{n}_1 \right) \mathbf{n}_1, \quad \mathbf{n}_1 = \frac{\mathbf{S}_i \times \mathbf{S}_{i+1}}{|\mathbf{S}_i \times \mathbf{S}_{i+1}|}
\]

where \( \mathbf{n}_1 \) vector is the external normal unit vector at the active node \( \alpha_n \) of the element \( A_i \) on the principal plane. The vector \( \mathbf{b} \) is a vector directed from the active node \( \alpha_n \) to the driven node \( \alpha_S \). Vector \( \mathbf{t} \) is the projection vector of vector \( \mathbf{b} \) on the active surface, as shown in Figure 1.

When the projection of the driven node \( \alpha_S \) on the principal plane is on boundary line or adjacent to the boundary line of the principal surface element, the above inequality equations fail. When the following formulas reach the maximum value, the projection of driven node \( \alpha_S \) is on the boundary line:

\[
\max \left( \frac{\mathbf{b} \cdot \mathbf{S}_i}{|\mathbf{S}_i|} \right) = 1, 2, \ldots
\]

If the projection of the driven node \( \alpha_S \) on the nearest active chip \( \alpha_S \) is point \( M \), then point \( M \) is the contact point of \( \alpha_S \) node on the \( \alpha_S \) chip when the pounding occurs, as shown in Figure 2. The position vector \( \mathbf{m} \) of any point on the main chip \( \alpha_S \) can be expressed by the following formulas, and the \( x \) coordinate of any node can be expressed as follows [18]:

\[
f_i(\xi, \eta) = \phi_1(\xi, \eta) \xi^1 + \phi_2(\xi, \eta) \xi^2 + \phi_3(\xi, \eta) \xi^3 + \phi_4(\xi, \eta) \xi^4,
\]

\[
\phi_j(\xi, \eta) = \frac{1}{4} \left( 1 + \xi \xi \right) \left( 1 + \eta \eta \right), \quad (j = 1, 2, 3, 4).
\]

Therefore, vector \( \mathbf{m} \) can be expressed in the principal coordinate system as follows:

\[
\mathbf{m} = f_1(\xi, \eta) \mathbf{i}_1 + f_2(\xi, \eta) \mathbf{i}_2 + f_3(\xi, \eta) \mathbf{i}_3,
\]

where \( \xi^j \) is the \( x \)-coordinate of the \( j \)-th node of the element, \( \phi_j(\xi, \eta) \) is the two-dimensional shape function of the element on the main chip, and \( \mathbf{i}_1, \mathbf{i}_2, \) and \( \mathbf{i}_3 \) are the unit direction vectors in the principal coordinate system.

If the normal direction of the main chip \( \alpha_S \) at the contact point \( M(\xi_m, \eta_m) \) is \( \mathbf{e}_n \), then \( \mathbf{e}_n \) can be expressed as follows [18]:

\[
\mathbf{e}_n = \frac{\partial \mathbf{m} / \partial \xi \left( \xi_m, \eta_m \right) \times \partial \mathbf{m} / \partial \eta \left( \xi_m, \eta_m \right)}{\left| \partial \mathbf{m} / \partial \xi \left( \xi_m, \eta_m \right) \times \partial \mathbf{m} / \partial \eta \left( \xi_m, \eta_m \right) \right|}.
\]

The driven node \( \alpha_S \) does not contact with the main chip \( \alpha_S \) when \( g = \mathbf{e}_n \cdot \left| \mathbf{d} - \mathbf{m} \left( \xi_m, \eta_m \right) \right| > 0 \). The driven node \( \alpha_S \) is on the main chip \( \alpha_S \) when \( g = \mathbf{e}_n \cdot \left| \mathbf{d} - \mathbf{m} \left( \xi_m, \eta_m \right) \right| = 0 \). There is no need to deal with the node in these two cases. The search of the driven node \( \alpha_S \) is finished, the node \( \alpha_S+1 \) enters a new search cycle, and the method is the same. It indicates that the node \( \alpha_S \) penetrates the main chip containing the contact point \( M(\xi_m, \eta_m) \) when \( g = \mathbf{e}_n \cdot \left| \mathbf{d} - \mathbf{m} \left( \xi_m, \eta_m \right) \right| < 0 \). The following equations are combined to solve the specific coordinate \( \mathbf{M} \left( \xi_m, \eta_m \right) \) of the contact point:

\[
\begin{align*}
\frac{\partial \mathbf{m}}{\partial \xi} \left( \xi_m, \eta_m \right) \cdot \mathbf{p} &= 0, \\
\frac{\partial \mathbf{m}}{\partial \eta} \left( \xi_m, \eta_m \right) \cdot \mathbf{p} &= 0, \\
\mathbf{p} &= \mathbf{d} - \mathbf{m} \left( \xi_m, \eta_m \right).
\end{align*}
\]

Vector \( \mathbf{p} \) is the vector of contact point \( M \) pointing to the driven node \( \alpha_S \), vector \( \mathbf{p} \) is perpendicular to the main chip \( \alpha_S \) when node \( \alpha_S \) is an out-of-plane point, and vector \( \mathbf{p} \) is a zero vector when node \( \alpha_S \) is an in-plane point. Penalty function algorithm works when the penetration occurs, and a normal contact force vector \( \mathbf{f}_n \) is added between the contact node \( \mathbf{M} \left( \xi_m, \eta_m \right) \) and the driven node \( \alpha_S \). The value of the penalty function can be expressed as follows:

\[
\mathbf{f}_n = -g \mathbf{e}_n, \quad (9)
\]
where $k_i$ is the stiffness factor of main chip $S_p$, $\bar{c}_i$ is the external normal unit vector at the contact point $M(\xi_m, \eta_m)$, and $k_i$ can be expressed as follows:

$$k_i = \frac{\lambda K_i D_i^2}{V_i} \tag{10}$$

where $\lambda$ is the scale factor of contact stiffness to control the punishment degree of contact; it is necessary to select appropriate values for different analysis structures. $K_i$ is the bulk modulus of the element where the contact point $M(\xi_m, \eta_m)$ is located, $D_i$ is the area of $S_p$, and $V_i$ is the volume of the element where node $M$ is located.

The normal contact force $f_j$ acting on the driven node $h_j$ must produce the reaction force $-f_j$ on the contact node $M(\xi_m, \eta_m)$ which interacts with it. The force on the contact node $M(\xi_m, \eta_m)$ can be equivalent to the four main nodes of the main chip $S_p$, and the contact force $f_j (j = 1, 2, 3, 4)$ of the element node can be obtained, which can be expressed as follows [18]:

$$f_j = -\phi_j(\xi_m, \eta_m) f_j = -\phi_j(\xi_m, \eta_m) gk_j \bar{c}_j, \tag{11}$$

where $\phi_j(\xi_m, \eta_m)$ is the shape function corresponding to the main chip of $S_p$ and $\phi_j(\xi_m, \eta_m)$ must satisfy the following equation [18]:

$$\sum_{j=1}^{4} \phi_j(\xi_m, \eta_m) = 1, \tag{12}$$

$$\phi_j(\xi, \eta) = \frac{1}{4} \left(1 + \xi \xi \right) \left(1 + \eta \eta \right) \quad (j = 1, 2, 3, 4). \tag{13}$$

### 3. Numerical Example

**3.1. Calculation Model.** The collapse-pounding problem of adjacent building structures involves large displacement, large strain, large rotation, and material nonlinearity. It is very feasible to select the explicit finite element program LS-DYNA for analysis. The integral model is adopted for reinforced concrete. The reinforcement and concrete are included in one element and combined into a composite element stiffness matrix in the element analysis.

The 3D Solid 164 element is used for beam and column, and the brittle failure model *MAT_BRITTLE_DAMAGE is used to simulate the structural failure, which is a composite material model of reinforcement and concrete. It is an anisotropic brittle damage model designed primarily for concrete though it can be applied to a wide variety of brittle materials. It admits progressive degradation of tensile strength and shear strength across smeared cracks that are initiated under tensile loadings. The stress–strain relationship of the material model is as follows [19]:

$$\sigma = C : \left[ \dot{\varepsilon} - \sum_{k=1}^{M} \gamma_k \partial \phi_k \right], \tag{14}$$

where $C$ is the rank 4 stiffness tensor of the material, $\sigma$ is the stress tensor, $\dot{\varepsilon}$ is the infinitesimal strain tensor, $\gamma_k$ is Lagrange multipliers or consistency parameters, $M$ is the set of stresses, $\partial \phi$ is the gradients, and $\phi_k$ is damage function. Rebar is defined by the reinforcement ratio and the reinforcement ratio of beam and column is 0.021 and 0.009. The material parameters of concrete and reinforcement [20] are shown in Tables 1 and 2.

The frame structure will inevitably produce large deformation and displacement in the process of collapse, which will lead to the failure of the element. Many constitutive models of materials themselves in LS-DYNA do not allow failure and erosion failure, but LS-DYNA provides a method to simulate material failure by adding *MAT_ADD_EROSION command to keyword file. The solid element is controlled by the principal strain in this paper, the concrete is considered to be invalid, and the element will be deleted from the whole structural model when the principal strain in the concrete exceeds 0.4 [20].

The symmetrical penalty function method is used to solve the contact impact problem, the automatic surface-to-surface contact (*AUTOMATIC_SURFACE_TO_SURFACE) is selected as the contact type, and the contact surface direction can be defined automatically. The program will automatically check each side of the element to judge the contact surface and the dynamic and static friction coefficients are 0.5 and 0.6, respectively [21]. The calculation model information is shown in Figure 3; $D$ is the gap size in Figure 3(a).

To avoid severe vibration caused by load application, the load is divided into two stages. Gravity load is imposed on the calculation model during 0–1 s. At the end of 1 s, the structure tends to be stable. The second stage is to apply the seismic action. At 1 s, the Northridge wave is applied at the column bottom. The duration of the earthquake wave is 15 s, the peak ground acceleration (PGA) is adjusted to 0.40 g and 0.62 g, respectively, and the acceleration time-history curve of Northridge wave is shown in Figure 4.

**3.2. Effect of Weak Column Position on Collapse-Pounding Dynamic Responses**

**3.2.1. Peak Ground Acceleration is 0.40 g.** The weak columns located in the first, third, fifth, and seventh floors of KJ-A are named GK-1, GK-2, GK-3, and GK-4, respectively, to facilitate the subsequent analysis, as shown in Figure 5. The green columns in Figure 5 are the weak columns and the column number in KJ-A is WC-A on the outside and WC-B on the inside.

Impact force is one of the important dynamic responses of adjacent building structures under earthquake action, which has an important influence on the collapse of structures. When the PGA is 0.40 g and the weak column at different positions is damaged, the variation law of the maximum impact force is shown in Figure 6.

It can be seen from Figure 6 that the impact force is the largest when the weak columns are located in the seventh floor of KJ-A; the impact force is the second when the weak column is located in the fifth floor and is the smallest when the weak column is located in the third floor. The maximum impact force first decreases and then increases when the
weak column is from the first floor to the seventh floor. The impact force is small when the weak columns are located in the lower floors of KJ-A, and the impact force is significantly greater than that of the lower floors when the weak column is located in the upper floors of KJ-A.

Interlayer displacement angle is an important index of frame structure under earthquake action. The structure is easy to collapse once the limit value is exceeded. The interlayer displacement angles of KJ-A and KJ-B when the weak column located at different positions is shown in Figure 7.

According to Figure 7(a), the maximum value of the interlayer displacement angle without impact is smaller than that with impact. The maximum value of the interlayer displacement angle of the higher structure is increased due to the impact, the interlayer displacement angle of part of

| Table 1: Parameters of concrete material. |
|------------------------------------------|
| Name | Density (kg/m³) | Elastic modulus (MPa) | Poisson’s ratio | Ultimate tensile strength (MPa) | Fracture toughness (Pa) | Viscosity (Pa) |
|------|-----------------|-----------------------|-----------------|-------------------------------|------------------------|---------------|
| C30  | 2500            | 3.2 × 10⁴             | 0.2             | 1.50                          | 144                    | 0.003         |

| Table 2: Parameters of reinforcement materials. |
|-----------------------------------------------|
| Name        | Density (kg/m³) | Elastic modulus (MPa) | Poisson’s ratio | Yield strength (MPa) | Failure strain |
|-------------|-----------------|-----------------------|-----------------|----------------------|----------------|
| HRB400      | 7850            | 2.0 × 10⁵             | 0.3             | 400                  | 0.25           |

Figure 3: Model information. (a) Plane. (b) Numerical calculation model.

Figure 4: Acceleration time-history curve of Northridge wave.
Figure 5: Locations of weak column: (a) GK-1; (b) GK-2; (c) GK-3; and (d) GK-4.

Figure 6: Maximum impact force corresponding to different weak column positions (PGA = 0.40 g).
KJ-A higher than that of KJ-B is also increased, and the impact also makes the position of the maximum value of the interlayer displacement angle move up.

According to Figure 7(b), the maximum interlayer displacement angle when the weak column is located in the first floor, the fifth floor, and the seventh floor is less than the value under the condition of no impact. The deformation law of KJ-B changes due to the impact when the weak column is from the first floor to the seventh floor, and the maximum interlayer displacement angle appears on the top floor when the weak column is in the third floor of KJ-A.

Generally, the impact increases the interlayer displacement angle of the higher structure in the adjacent structure and decreases the interlayer displacement angle of the lower structure.

The weak column KG-3 is taken as an example because of the limitation of space, and the collapse-pounding process of the adjacent structures is shown in Figure 8.

It can be seen from Figure 8 that the adjacent structures are in elastic working state before 1.58 s and the plastic deformation of the structure gradually accumulates with the increase of earthquake time. The longitudinal beam end of the fourth floor of KJ-A is damaged by the joint action of earthquake and collision at 8.27 s, and then the scope expanded rapidly to the third and fifth floors. All the weak column heads and bases are damaged at 10.38 s which leads to progressive collapse of the corner of the frame where the weak column is located. Every floor of KJ-A produces beam end failure at the last moment of earthquake; the fourth, fifth, sixth, and seventh floors are seriously damaged; the first, second, and eighth floors are the second, while the failure beam of KJ-B is very few and the overall damage is light.

3.2.2. Peak Ground Acceleration of 0.62 g. KJ-A completely collapsed when PGA is 0.62 g, and the interlayer displacement angle of KJ-B is shown in Figure 9 under different weak column positions.

According to Figure 9, the maximum value of the interlayer displacement angle in case of impact is smaller than that in case of no impact, and the reason is that the deformation of KJ-B structure is limited by pounding. The interlayer displacement angle is the largest when the weak column is in the fifth floor, the interlayer displacement angle is the second when the weak column is in the seventh floor, and it is the smallest when the weak column is in the first floor in the case of impact. The analysis shows that the impact has a limited effect on the interlayer displacement angle of KJ-B. Since the failure of weak column in KJ-A does not necessarily cause the collapse of KJ-B, the influence of the weak column position in KJ-A on KJ-B is small.

When PGA is 0.62 g, the maximum impact force at different weak column positions is shown in Figure 10.

According to Figure 10, the impact force is the largest and the value is 2710 kN when the weak column is located in the fifth layer; the impact force is the second and the value is 2580 kN when the weak column is located in the seventh layer; the impact force is the least when the weak column is in the third story. The maximum impact force decreases first, then increases, and after that decreases. The impact force is large when the position of the weak column is close to the top of the low-rise building, which is harmful to KJ-B structure.

The weak column KG-3 is taken as an example when PGA is 0.62 g, the collapse-pounding process of the adjacent structures is shown in Figure 11.

It can be seen from Figure 11 that the structure is in an elastic state before 1.34 s, the plastic deformation of the structure begins to accumulate under the earthquake action. The beam end of the weak column in KJ-A starts to fail at 8.11 s and rapidly extends to other beams. The top of weak column WC-B
fails first, which weakens the stiffness of this story. At 8.37 s, the bottom column base element fails and exits the bearing capacity system. The decrease in bearing capacity makes the side frame become a movable mechanism and starts to collapse. Finally, the whole structure collapses due to local collapse. KJ-B does not collapse at the end of the earthquake, but most of the beam end elements fail and the serviceability of the structure is seriously reduced.

3.3. The Influence of Gap Size on the Collapse-Pounding Dynamic Responses. Gap size is an important parameter in the design of adjacent structures, which needs to meet the requirements of normal use. Gap size will affect the impact dynamic response of adjacent structures under earthquake. To study the influence of gap size on the collapse-pounding dynamic responses, its values are set to 0.15 m, 0.17 m, 0.19 m, 0.21 m, and 0.23 m, respectively [22].

When the PGA is 0.40 g, the maximum impact force is shown in Figure 12 and the interlayer displacement angle is shown in Figure 13 corresponding to different gap sizes. As shown in Figure 12, when the gap sizes are 0.15 m, 0.17 m, 0.19 m, 0.21 m, and 0.23 m, respectively, the maximum impact forces are 3764.9 kN, 3050.9 kN, 3664.6 kN, 3525.2 kN, and 1900.4 kN. Generally speaking, the

![Figure 8](image-url)
Figure 9: KJ-B interlayer displacement angle (PGA = 0.62 g).

Figure 10: Maximum impact force at different weak column positions (PGA = 0.62 g).

Figure 11: Continued.
maximum impact force decreases at first, then increases, and after that decreases. When the gap size is too small, the impact force is larger, and when the gap size increases to a larger value, the impact force will decrease significantly. Therefore, gap size is a key parameter in the design of adjacent buildings.

As can be seen from Figure 13, when PGA is 0.40 g, the influence of gap size on KJ-A and KJ-B is significantly different. The maximum interlayer displacement angle for KJ-A is located in the middle floor whatever there is an impact or no impact. The interlayer displacement angle is the largest when the gap size is 0.15 m and the deformation mode of the structure also changes when the gap size increases due to the influence of impact. The distribution of the interlayer displacement angle of KJ-B has changed greatly when the gap size is 0.15 m, and the maximum interlayer displacement angle is located at the top of KJ-B compared with the case without impact. The impact reduces the interlayer displacement angle of KJ-B under the condition of other gap sizes.

The maximum impact force under different gap sizes is shown in Figure 14 when PGA is 0.62 g.
the gap size increases. The impact force is small when
$D = 0.15$ m between $D = 0.15$ m and $D = 0.19$ m, and the impact force gradually decreases with the gap size increasing when $D$ is greater than 0.19 m.

KJ-A collapsed completely when PGA is 0.62 g. The interlayer displacement angle of KJ-B is shown in Figure 15 under different gap sizes.

It can be seen from Figure 15 that the impact reduces the maximum value of the interlayer displacement angle of KJ-B compared with the case of no impact, and the distribution form of the interlayer displacement angle of KJ-B is changed greatly when the gap size $D$ is 0.21 m.

4. Conclusions

The 3D calculation models of 8-story and 6-story adjacent frames are established based on LS-DYNA, the collapse-pounding dynamic responses under the first failure of columns at different positions in the 8-story frame are studied, and the influence of the collapse position and the gap size of main factors of the system on the dynamic responses is discussed. The main conclusions are as follows:

(1) The impact force is small when the weak column is located at the lower floor of the KJ-A, the impact force is significantly greater than that of the lower floor when the weak column is located at the higher level of the KJ-A, and the impact force is larger when the weak column in the KJ-A is near the top of the KJ-B.

(2) Generally speaking, impact increases the interlayer displacement angle for the higher frame KJ-A with the weak column and reduces the interlayer displacement angle for the lower frame KJ-B.

(3) The collapse range of the higher frame KJ-A with weak columns is small when the PGA is 0.40 g, and
the higher frame KJ-A completely collapses when the PGA is 0.62 g.

(4) The dynamic response does not completely decrease with the increase in gap size. On the whole, it tends to decrease at first, then increase, and after that decrease. The impact force can still be reduced by designing the gap size as the intermediate value when the actual conditions are limited.

(5) Collapse-pounding of the higher frame KJ-A with weak columns has a great influence on the interlayer displacement angle distribution of the adjacent lower frame KJ-B.

Data Availability

The data used to support the findings of this study are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest regarding the publication of this paper.

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