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**Experimental and numerical study of axially-loaded RC columns and frame
columns under lateral impact loading**

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Draft

Abstract

The pendulum impact tests were carried out on one RC frame column and four RC free-standing columns. The effect of axial compression ratio and reinforcement ratio on the impact resistance of columns were compared by means of dynamic time curves of framed and freestanding columns under impact. The test results show that with the same impact load, though the presence of axial loads can play a positive role (e.g., reducing the residual displacement), it may lead to more severe local damage. Also, compared with free-standing columns, the frame column can be considered as a protective structure for its greater lateral stiffness and stronger crashworthiness. The corresponding finite element models are developed, and the influence of the axial loads on cross section force responses under impact excitation is deeply explored. The axial loads can significantly affect the distribution of the moment, the shear force, and also the damage for the column.

Keywords: Pendulum test; Frame column; Axial load effects; Failure mode; Numerical simulation

1 Introduction

As a typical axial load-carrying structure in bridge piers and offshore buildings, RC columns are conventionally suffered from lateral impact loads in the occasion of the vessel or vehicle collision, which is very risky. Numerous catastrophic accidents involving the collapse of the RC column due to impact loading have been widely reported (AASHTO 2009; Buth et al. 2010). Therefore, the effects of various load-related and structure-related parameters on the failure behavior of the columns against the impact loads should be carefully examined in the structural design of the columns.

In the experiment field, most of the previous tests adopt drop hammer impact to explore the dynamic responses, failure modes, and crashworthiness of RC structural members, including beams and columns (Pham et al. 2017; Fujikake et al. 2009; Xiao et al. 2016). However, unlike the horizontal beam members, the inertia effect of superstructure mass play an important role on the lateral shear resistant behavior of RC columns. Therefore, the pendulum impact testing system can be considered as a better method for the study on RC columns. When RC columns were impacted at different locations, it might suffer localized failure modes and damages such as brittle spalling, scabbing, perforation, and punching shear failure, or overall flexural failure modes. (Sha et al. 2013; Thilakarathna et al. 2010; Zhang, et al. 2018).

Moreover, the impact-resistant behaviors of the beams are different from those of the columns due to the axial load. The experimental and numerical investigations on the impact-resistant performances of axially-loaded circular RC columns with drop hammer impact test was conduct (Fan et al. 2019; Liu et al. 2017), which indicated that the applied axial loads significantly affect the impact responses (e.g., impact forces, maximum and residual deformations) and even the ultimate failure mode of the circular RC columns. A recent study (Gholipour et al. 2018) has indicated that the distance of plastic hinge locations from the column impact point decreases in proportion to the increase of axial load ratio, which causes column failure on the impact zone. Previous research (Al-Thairy et al. 2011; Chen et al. 2016; Pham et al. 2019) have found that failure modes caused by axial load and impact point have not been thoroughly explained, and more studies need to be carried out to understand the relationship between the internal force of the impact point and its failure mode as well as the mechanism behind. Furthermore, the frame column has been widely used in overpass bridge structures and high-rise pile cap foundations, but these structures also are vulnerable to

vehicle or vessel collisions. However, there is no relative study on the multiple-column structure under impact loading.

The pendulum impact tests on four free-standing columns and one frame column were carried out in this study. The main variable parameters were axial load level, reinforcement ratio, and impact energy. The response (e.g., impact forces, cracking characteristics, and structural deformations) and failure mode of the RC column with impact load were recorded and analyzed accordingly. Then, the numerical model was proposed and verified for the results of pendulum impact tests. With the calibration, the distribution of acceleration, bending moment, shear force, and axial force of RC columns were compared and analyzed, which showed a clear difference for the responses of the axially loaded free-standing column, no axially loaded free-standing column, and the frame column under impact loads.

2 Experiments

2.1 Component design

Four free-standing RC columns and one frame RC double column were designed and built in the laboratory. The pendulum impact system was utilized to generate the impact loading onto the column. The RC high-elevated pile, in which the free length of the elevated pile to the fixed end is 8m, was served as a prototype. Fig 1 shows the schematic view of the five specimens. The free-standing column was 800 mm in height, and the circular section was 150mm in diameter. The designated dimension of the tested columns represented 1/10 scale model of a 8 m tall high-elevated pile. The height of the tested columns was restricted by the dimension of the pendulum impact testing system for the current experiment. Previous studies by Zhang, X. and Hao, H. have demonstrated the suitability of 1/10 scale model for RC columns with similar dimensions subjected

to impact loading. (Zhang et al. 2018) The structural scaling factors of the present model tests are given in Table 1.

The frame column had two columns: the front column and the back column, whose height and cross-sections were consistent with that of the free-standing column. The upper RC platform was connected by the frame column to enhance the integrated lateral stiffness of the structure. The horizontal distance between the two columns in the frame model was set at least $3D$. A footing of 200 mm in depth with 400 mm $\times$ 400 mm cross-sectional area (800mm $\times$ 400 mm for frame column) was built for each column to bolt them onto the floor. A 400 mm $\times$ 400 mm $\times$ 200 mm concrete block (400 mm $\times$ 800mm $\times$ 200mm for frame column) with five 30 kg steel plates was placed on top of the columns, which gave a total weight of about 180 kg on the column. For the axially-load column, the designed value of the axial force was 67kN when the axial compression ratio was 0.1. The axial compression was applied by unbonded prestressed tension of the steel tendon through PVC pipes.

Table 2 summarizes the main design parameters of the tested specimens. The longitudinal reinforcing bars of columns were six HRB335 bars with a diameter of 6/8mm. The stirrups were HPB300, and the stirrup space was 100/50mm.

The concrete strength grade of the specimens was C30. According to the standard test procedure, the average compressive strength of the cube specimen was 38MPa. Details on the material parameters (including reinforced bar and tendon) can be found in Table 3.

2.2 Testing Device

Given that the axial force and lateral impact force need to be active at the same time, the test system of the pendulum-column impacting can be divided into two parts: (1) the axially loaded

system is shown in Fig 2(a), (2) the pendulum loading system is presented in Fig 2(b). The axially loaded system was mainly composed of additional mass blocks, high-strength prestressing tendons, anchor devices, and PVC pipes. The additional mass blocks were installed at the top of the platform to reflect the actual transmission of inertia force on the superstructure mass. As seen in Fig 2(a), the RC-P0 columns and RC2 frame column had an additional mass block in the upper platform only with no axial force, while the RC-P1 columns were applied with axial force and additional mass block simultaneously, and the axial force was applied in both sides with a high-strength prestress tendon.

The pendulum loading system is presented in Fig 2(b). The pendulum and the rod were made of high strength low carbon steel and installed on the self-balancing frame. The pendulum was pulled by the crane to a specific angle and then fell. The pendulum rod was equipped with an angle dial at the upper end. The pendulum was released from 15° , 30° , 45° , 60° and 75° respectively to obtain corresponding impact velocities and energies. The columns were tested at multiple impacts by increasing the impact speed until the specimens were completely failed. The pendulum weight was 150kg, and the length of the rod was 1.26m. The impact force was obtained by the pressure sensor installed in the head of the pendulum. The parameters of the impact condition are summarized in Table 4. Five dynamic displacement sensors were installed on each column to observe its deformation, as shown in Fig 2(d).

3 Test Results and Analysis

Quantitative results, such as failure modes of specimens, impact load time history, column displacement time history, and the relationship between input energy and column peak response

were presented as follows.

3.1 Failure Model

The impact velocity is increased gradually until the specimens were completely failed. The failure modes of the RC column specimens under the ultimate failure scenario were measured, as shown in Fig 3. The RC-P0-1 specimen was destroyed at impact 3, while the remaining specimens were destroyed at impact 5. Also, Fig.4 shows that the local damage extents in the base and the top of columns.

Fig 3 (a)(b) and Fig.4 (a)(b) depicts the overall damages as well as the local damages of the columns with no axial load. For the non-axial columns, concrete opening can be observed at the bottom of the column, indicating the flexural induced tensile failure of concrete. When the top of the columns RC-P0-01 is impacted, the column base will be subjected to more significant flexural actions. As the level of impact load increased, crack in the rear side of the column base is observed, which develop into critical cracks. The tensile cracks will lead to flexural failure when the impact load is rose continually. Figure 3(a)(b) shows that in the final failure mode, the frontal crack at the base of column RC-P0-02 exceeded 15mm in width and penetrated with the back crack forming a plastic hinge. Notably, Fig 3(a)(b) indicates that the residual displacement of the column RC-P0 with no axial load is considerably large under ultimate failure. For example, the maximum residual displacement of the RC-P0-02 column was recorded as 40.21mm under the ultimate failure, which makes it almost impossible to restore the column after a disaster. Meanwhile, the shear damage is also induced at the connection zones in the upper platform, and apparent shear cracks are observed in the concrete section of the column.

Concrete crushing on the backside of the collision are shown on the axially loaded columns

RC-P1, as shown in Fig 3(c)(d) and Fig 4(c)(d). The axial load plays a positive role in the RC column's failure under impact load. For example, the width of crack opening in the RC-P1 is less than those of the RC-P0. The axial load can remarkably enhance the stiffness of columns and reduce the cracking width of the concrete on the front side of the collision as well as reduce the residual deformation after impact. However, the axially loaded columns under ultimate failure exhibit larger area and depth of local damage compared with those of the column with no axial load, particularly on the compression side of the column bottom. Cracks can be found extending across through the column bottom, and plastic hinges are formed at the column bottom for specimens RC-P1. As the column bent, the local base concrete was severely crushed due to the extreme flexural bending. Meanwhile, when larger reinforcement ratios are provided, the overall and local failures of the column will be significantly reduced, as shown in Fig 3(c)(d) and Fig 4(c)(d). This may be because that the longitudinal and transverse reinforcements can help to increase the resistance of RC columns subjected to impact load. In the previous impact-resistant behaviors of RC beams, cracking of specimens in all series tended to concentrate at the impact point, inclined cracks initiated at the top near the impact point and propagated down at a fixed angle on both sides, these inclined cracks developed into major cracks that controlled the specimen behavior and the failure mode changed from flexure to shear. All of the above differ significantly from the impact-resistant behaviors in this paper. This is due to the fact that the difference in impact location caused the above differences. Experiments in this paper impact at the top of the column rather than midspan. From the comparison, when the specimens are impacted at the midspan (such as vehicle collisions on RC bridge columns), the shear response can become the dominant response mode. The column is easier to deform and tend to flexural failure when the impact loaded is applied at the column top (such as vessel collisions

on flexible piles) rather than that at the column bottom.

The damage and failure patterns of the frame column RC02 differed from those of the free-standing column, as presented in Fig 3(e) and Fig 4(e). It can be observed that the considerable concrete cracking is formed at the column-foundation interface and column-cap beam joints after Impact 05. However, the column RC02 is survived after the impact and maintains integrity. Compared with the test result of the free-standing columns above, the frame column suffered more serious local concrete damage on both ends of column. This is probably because that the frame column was more rigid due to restraining lateral deformation. As a result, flexural and compressive stress is concentrated on the smaller local areas, such as the column end.

On the basis of the observed crack formations, the failure modes of RC columns failing in flexural under impact loading can be grouped into three categories: (1) Brittle failure characterized by tensile cracking of the column at the base such as non-axial columns RC-P0, (2) Ductile failure characterized by crush of the column at the base such as axial columns RC-P1, (3) Ductile failure characterized by crush of the column, and plastic hinge are formed simultaneously at the top and bottom of the columns such as columns RC02. The plastic strain in the FEA result is illustrated in Fig 3 as well to be compared with the column damage in the experimental test. Major flexural cracks and a few shear cracks are both observed in the numerical model and the experimental test, where the flexural cracks were at the column bottom while the shear cracks were at the column top. In conclusion, the dynamic behaviors of the column in the experimental test are well simulated in the numerical model.

3.2 Impact Force

Fig 5 shows the impact of force-time history curves of columns under ultimate failure.

According to the variation trend characteristics of impact time-history curves, the curves can be divided into three stages as follows.

Stage I: The peak stage of impact force. Like the earlier RC beam test with drop weight impact (Pham et al. 2017; Fujikake et al. 2009; Xiao et al. 2016), the peak impact force occurred in a short time interval (<2 ms) after the pendulum contacted the specimen. The shape of the impact load on the nonaxial-loaded specimens was primarily single wave which differed from those without an axial load. Multiple peaks may appear on the load time history, which results from the restrain of axial load. The multiple contact interactions are formed between the pendulum and the column specimens after the first peak.

Stage II: The plateau stage of impact force. This stage is characterized by its long duration (more than 10 ms) and its fluctuated impact force in a specific range. With the same energy impact in the plateau phase, the force platform duration of the axially loaded column is much longer than that with no axial load, as shown in Fig 5(a)(c). In particular, it needs to be noted that the RC-P0-01 exhibits almost no plateau phase. This is attributed to the rapid reduction in the lateral stiffness of the columns for the influences posed by the lateral impact load. Also, compared with the free-standing column, the frame column has half the duration of the force plateau phase.

Stage III: Unloading phase. The motion direction of the column top in this phase is opposite to that of the stage II, and the column specimens will rebound. In this stage, the duration of the axially loaded column is much longer than that with no axial load as well.

3.3 Impact Displacement

The lateral displacement time-history curves of the column top under ultimate failure are shown in Fig 6. Similar to the result of the impact force, the lateral displacement time-history curve

of the column top can be divided into three stages as follows.

Stage I: The loading stage, where the displacement increases from 0 to the peak value.

Stage II: The unloading stage, where the column displacement decreases from the maximum level to the residual value.

Stage III: The free vibration stage, where the column displacement oscillates back and forth near the residual value to the final residual displacement.

Furthermore, when the displacement time-history curve enters the free vibration stage, the RC-P0-01 will get close to a smooth straight line (Fig 6(a)). This is attributed to the rapid deflection of columns due to the rapid reduction of lateral stiffness under strong collision. On the contrary, the displacement vibration curve of frame column RC02 is quite different from that of the free-standing RC column in this stage. The variation of displacement is characterized by the shakeup and down, and the frequency of free vibration response is relatively high (Fig 6(e)).

3.4 Analysis and discuss of column response

The test results of each column at the different impact velocities are summarized in Table 5.

The peak tendency of the impact load with the gradual increase of the impact energy is summarized in Fig 7 (a). With the comparison of different columns from Table 4 and Fig 7(a), it can be found that the PIF (peak impact force) increases with the rise of impact energy under the condition of low-energy impact. However, under the high-energy impact condition, RC columns are damaged by the impact. The contact stiffness is diminished, and PIF remains unchanged and even decreases. Meanwhile, under the same impact energy, the PIF of the column with axially load is greater than that with no axial load. The reason behind is that the initial contact stiffness of the RC column is enhanced with an axial load. The overall stiffness of the frame column is much greater

than that of free-standing columns, so as the impact force.

From the comparison on the maximum and residual deflections of the five columns top, as shown in Figs 7(b) and (c), it can be observed that in the impacted test, the trend of maximum lateral displacement $D_{\max}$ and residual displacement R_{esi} of the column top is consistent with the results of the column damage degree. $D_{\max}$ and R_{esi} increase with the rise of impact energy E_k , and the lateral displacements tend to decrease on account of the axial load or the rise of the reinforcement ratio. With the strong impact of the pendulum, the $D_{\max}$ and R_{esi} of the frame column RC02 are notably lower than those of RC free-standing columns, which indicates that the overall stiffness of the frame column remains good after being impacted and damaged, representing good crashworthiness.

4. Impact FE Simulations of the Experiment

The verification has preliminarily shown that the FE model above yields reliable predictions of the column responses to impact load, and it can observe the impact force, lateral displacement, and the failure modes of the tested column. In this section, some issues regarding the FE model, including material models, strain rate effect, and the method of axial load, are discussed in detail. Numerical results are further analyzed, and the column internal force features with impact load are also explored. Notably, the column collision in the experiment can be revealed with these FE modeling results for internal force.

4.1 Finite element model

The FE model was built to simulate the experimental pendulum impact test mentioned above, as shown in Fig 8. A hexahedral element was employed to represent the concrete elements, steel impactor, and steel plates. A 3-nodes beam elements was adopted for steel reinforcement. The CONSTRAINED BEAM IN SOLID modeling option is employed to embed rebar and grout sleeve beam elements into concrete solid elements. The nodes of beam elements are constrained by the solid elements along the normal and tangential directions by using this coupling method. The CONTACT ERODING SINGLE SURFACE modeling option was adopted to simulate contact

between the pendulum and the RC columns, considering the possible eroding (loss) of concrete elements at the loading point. Penalty scale factors SFS and SFM at this contact provide a means of increasing or decreasing the contact stiffness. Increasing values of SFS and SFM leads to a higher contact stiffness, thereby increasing the peak impact force. The values of SFS and SFM were determined through calibration of the computed peak impact force with that measured in the experiment results ($SFS = SFM = 0.15$). Slip between the foundation and the bottom platform was not observed in the impact tests, hence in the numerical model, all degrees of the platform bottom nodes were constrained as boundary conditions.

4.2 Material Model

The CSCM material model was selected for modeling the concrete in this study, considering the plasticity, shear failure damage, and strain-rate effect (Murray 2007). Under extreme dynamic loads, the reliability for simulating and predicting the behavior of concrete structures of this material model has been confirmed by previous studies (Jiang et al. 2015). In this study, the unconfined compressive strength (f_{cu}) tested from a $150\text{mm} \times 150\text{mm} \times 150\text{mm}$ concrete cube was 38MPa. Usually, the uniaxial compressive strength f'_c of the cylinder concrete is equal to 0.79 times f_{cu} . As a result, $f'_c = 38 \times 0.79 = 30\text{MPa}$, which was applied to define the uniaxial compressive strength in the CSCM model. For the cover concrete of the columns, the default values suggested by the CSCM model were employed for the given compressive strength. However, the material parameters related to the confinement effect were calibrated in line with the Mander model (Mander et al. 1988) for considering the confined concrete adopted in columns. The corresponding stress-strain curve is presented in Fig 8. The Piecewise Linear Plasticity material model was employed to model the steel reinforcements (Hallquist et al. 2007). The mass density and Young's modulus were 7800 kg/m^3 and 200GPa , respectively. The yield strength of the longitudinal steel was 335MPa , while the corresponding value of the transverse reinforcement was 210MPa . The effect of strain rate on the behavior of the material was considered in compliance with previous studies. For the steel pendulum impactor and anchor plate, the elastic material model was applied, in which the steel yield strength, mass density, and Young's modulus were 300 MPa , 7800 kg/m^3 , and 200 GPa , respectively. In addition, the erosion modeling option is employed to automatically remove the distorted elements

based on the predefined criteria. The maximum principal strain of 0.7 is determined by trial and error in this study to yield fairly good agreement with the experimental results (Hallquist et al. 2007).

4.3 Strain rate effect

Since concrete and steel are strain rate sensitive materials, the strength of these materials can enhance in varying degree depending on loading rates. The strain rates involved in the impact problem of reinforced concrete generally fall into the category of low to medium strain rates, with material strain rates lower than those of the blast impact action. After preliminary trial calculations, it can be determined that the concrete strain rate in this study does not exceed 30/s. The enhancement of material strength is defined by the dynamic increase factor (DIF) that is the ratio of dynamic to static material strength. To date, extensive researches have been conducted to determine the strain rate effect of concrete and steel based on the experimental data. In this study, DIFs for concrete compressive and tensile strength proposed by Hao are employed because they are derived by removing the lateral inertial and end friction confinement effects from impact test data, therefore better represent the true dynamic strain rate effect of concrete material (Hao et al. 2015).

The compressive DIF of concrete at the strain rate $\dot{\epsilon}_d$ given by the following equation:

$$CDIF = \frac{f_{cd}}{f_{cs}} = \begin{cases} 0.0419(\log \dot{\epsilon}_d) + 1.2165 & \dot{\epsilon}_d \leq 30s^{-1} \\ 0.8988(\log \dot{\epsilon}_d)^2 - 2.8255(\log \dot{\epsilon}_d) + 3.4907 & \dot{\epsilon}_d > 30s^{-1} \end{cases}$$

in which CDIF is the DIF for the concrete in compression; f_{cd} is the dynamic compressive strength at the strain rate $\dot{\epsilon}_d$, and f_{cs} is the static compressive strength.

The DIF of concrete tensile strength is

$$CDIF = \frac{f_{td}}{f_{ts}} = \begin{cases} 0.26(\log \dot{\epsilon}_d) + 2.06 & \dot{\epsilon}_d \leq 1s^{-1} \\ 2(\log \dot{\epsilon}_d) + 2.06 & 1s^{-1} < \dot{\epsilon}_d \leq 2s^{-1} \\ 1.4433(\log \dot{\epsilon}_d) + 2.2276 & 2s^{-1} < \dot{\epsilon}_d \leq 150s^{-1} \end{cases}$$

where TDIF is the DIF for the concrete in tension; f_{td} is the dynamic tensile strength at the strain rate $\dot{\epsilon}_d$, and f_{ts} is the static tensile strength. The relationship between the tensile and compressive DIF of steel and strain rate is defined by the following equation:

$$DIF = \left(\frac{\dot{\epsilon}}{10^{-4}} \right)^{\alpha}$$

$$\alpha = 0.074 - \frac{0.04 f_y}{414}$$

where f_y is the yield strength of steel in MPa. It should be noted that in this study DIF is held as constant when the strain rate is higher than 160 s^{-1} to prevent overestimation of the DIF of steel material at very high strain rate.

4.4 Method of stress initialization

It was necessary to conduct a primary or quasi-static analysis before the transient simulation, such as the self-weight or axially loaded analysis. The abrupt application of the axial static load as the preload will lead to undesirable dynamic responses of the structures in the transient analysis (Pham et al. 2018). Thus, the dynamic relaxation modeling option (DR) was applied to calculate the initial stress of the FE model before transferring the result on initialization stress data for subsequent transient analysis (Hallquist et al. 2007). The modeling procedure was divided into two phases, namely the DR phase and the transient phase. In the DR phase, the self-weight or axial load was applied firstly by gradually getting close enough to the target load and then holding the load constantly until the solution converges. In the transient phase, the axial load was applied as a constant curve. The simulation is presented in Fig 10.

4.5 Damping

The damping has a great influence on the impact resistance of columns, especially non-realistic oscillations of columns are prevented during the free vibration phase after the pier has been impacted. To properly incorporate damping into impacted dynamic analysis, LS-dyna provided mass-weighted damping method defined as:

$$F_{damp} = D_s m v$$

where D_s is the damping constant. Generally, the damping constant for the system to reach static equilibrium is based on the critical damping factor for the lowest frequency mode of interest

$$D_s = 2\omega_{\min}$$

where ω_{min} is the lowest natural circular frequency of a structural system. The natural frequencies of the structure can be derived from the fast Fourier transform (FFT) analysis of the axial force time history with no damping.

To explore the influence of damping to collision analysis, the comparison of the dynamic response of an RC column with damping or not under impact is shown in Figure 11. The damping has little effect on the impact force. This is attributed to the fact that the impact force is mainly dependent on the resistance of the impacting pendulum, rather than the impacted structure. However, damping plays an important role in predicting the impact -induced displacement response, not only by reducing the maximum displacement response, but, more critically, by enabling the columns to reach displacement equilibrium as soon as possible during the free vibration phase.

5 Analysis and discussion about FEA internal force

It is well-known that the stress wave from an impact event propagates from the impact area to the ends of structures. During the vibration of the stress wave, the dynamic internal force will be formed in each section of the column. However, the relations between the lateral impact force and the dynamic internal force in the concrete column remains unknown. With this as the context, the critical sections is adopted to evaluate the axial force, bending moment, and shear force located at different parts of the concrete column. A set of critical sections is defined accordingly, as shown in Figure 12.

5.1 Axial forces response

Fig. 13 shows the time history of the dynamic axial force in impact 03. It should be noted that the negative value in the figure stands for the compression force in the cross-section. When the impact force reaches the peak, the axial compression force gets close to its maximum value. The axial force fluctuates notably with the impact force in the duration of impact due to the propagation and reflection of the stress wave. After the force phase, the axial force returns to the original condition and vibrates around its initial level.

The dynamic axial force response of the free-standing column with or without axial load is compared in Fig 13(a) and (b). The non-axial column RC-P0-02 is characterized by alternatively positive and negative variations, whose highest level of the compression axial force and axial tension force are about 50kN and 30kN, respectively. Compared with the RC-P0-02, there is no tension force in the axially column RC-P1-02, and the highest level of the compression axial force is about 120kN. The lowest level of the compression axial force is about 30 kN in the column RC-P0-02 with no axial load. These results indicate that axial load can play a decisive role in avoiding tension force so that concrete cracks will not occur in the initial force phase. The dynamic axial force variation for the frame column RC02 is shown in Fig 13 (c). There are several vital differences in dynamic axial force response between the free-standing column and the frame column. Firstly, both the front column and back column withstand the axial compression force during the initial force phase. However, the critical sections of the back column withstand the axial tension force mainly during the subsequent vibration phase. On the contrary, the axial compression force may occur in the front column sections. Secondly, the peak axial force appears in the vibration phase rather than the force phase, which means that the effect of dynamic axial load lasts longer in the frame column RC02 than others. Moreover, the highest level of the axial compression force in the front column section (about 150kN) is slightly larger than the highest level of the axial tension force (about 100kN) in the back column section. Since the top platform connected by a rigid beam increases the stiffness of the column and restrains the axial deformation for the frame column RC02, the axial force with a large amplitude is induced and accompanied by the high-frequency vibration in this phase.

5.2 Shear force response

The shear force time history compared with the impact force for part of the critical sections in the free-standing column RC-P0-02 are represented in Fig14(a). In the initial phase, the impact force, increases to the peak value of 80 kN, and the maximum shear force at the column top rises to about 60 kN simultaneously. At the moment, the maximum shear force at the column top is approximately equal to the peak impact force, and the shear force at the column base becomes trivial. This is because the impact force is balanced by the inertial force near the impact point. Thus, the impact force is primarily bore by the column top. Furthermore, the peak shear force of the frame column RC02 reaches the maximum value of about 40 kN at the front-column top. However, the maximum shear force at the back-column top is only 20kN under the same conditions, as shown in Fig 14 (b). These results illustrate that the front and back column will bear the impact together, and that is why a decrease in shear failure is formed. In this perspective, the frame column can become one of the good choices for shear impact resistance. Besides, the top of the front column is a critical section for the shear impact resistance and needs to be paid more attention.

To examine the shear force during the peak impact force in detail, the inertia distribution and the shear force diagram along the column at the peak time in different velocities are presented in Fig 15 and Fig 16. It is observed that the peak shear force and its acceleration increases with the rise of impact velocity ($V_1=0.93\text{m/s}$, $V_2=2.72\text{ m/s}$, and $V_3=4.35\text{ m/s}$). It is particularly necessary to point out that the acceleration and shear force along the column occupy the same varying tendency. Still, the direction of variations is opposite to each other. As seen in Fig 15, the stress wave is propagated from the impact point to the column ends so that only part of the section along the column reacts to the impact of the peak impact force at its peak moment. The length of the active part is almost half of the column (about 400mm). Besides, the value of peak shear force under the same impact conditions is not affected by whether there is an axial load or not, as is shown in Fig 16(a). These results illustrate that the shear force response is not affected by the increased stiffness due to axial load, and the column tends to show a local response rather than a complete response during the force phase.

5.3 Bent moment response

The bending moment time history of the columns in impact 01 is presented in Fig 17. Both positive and negative bending moments are observed in the critical section of the free-standing column under the impact, which is different from the static case. The time history curves show the fluctuation of the bending moment. The bending moment time history of the column with no axial load is presented in Fig 17(a). The maximum positive bending moment at section 4 is about 450 kN m, while the maximum negative bending moment at the column base is 670kN m. With axial load, the positive bending moment at the column base reaches the maximum value of 370 kN m. In contrast, the corresponding maximum negative bending moment is 570kN m, as presented in Fig 17(b). It can be seen that the peak of bending moment response decreases with the rise of axial load, which results from the positive influence of axial load on the column's flexural stiffness and the reduction of deflection for the column with the same impact energy. In addition, the maximum moment of column occurs in the free vibration phase rather than the impact phase. Therefore, the bending deflection will be the main feature in the vibration phase, and the maximum bending moment governed by the greater overall stiffness will decrease in this phase.

The dynamic bending moment of critical section variation for the frame column is shown in Figs. 17c-d. The bending moment time history of each section fluctuates around the zero levels, and the positive and negative bending moment in rapid variations is shown alternatively in each critical section. It is evident that the bending moment time history of the frame column RC-02 fluctuates in a high frequency, while that of the free-standing column is much slower, which results from the top platform connected by the rigid beam that can restrain the lateral deformation in frame column RC02. The bending moment is therefore induced and accompanied by the high-frequency vibration during this phase. Moreover, the positive bending moment of the base section (about 1115kN) of the front column is nearly twice as much as the positive bending moment of the base section (about 700kN) of the back column. These results explain why the front column, rather than the back column, is more seriously destroyed in the collision.

Fig. 18 describes the distribution of peak bending moment along the column with different impact velocities. It can be seen that the shape of the distribution diagram is changed due to the

restrain condition of the frame column top. The maximum bending moment of the frame column occurs at the top and base of column, while the peak bending moment at the section-4 is close to that of the free-standing column base. Moreover, the maximum bending moment response in the frame column is larger than that of the free-standing column under the same impact. When the impact velocity (V_3) is increased to 4.35 m/s, the peak bending moment of the free-standing column is only 1000 kNm, while the maximum bending moment of frame columns is about 2100 kNm. Furthermore, the plastic hinges are formed at both ends of the column.

As mentioned above, the shear response of the frame column is rather small compared with that of the free-standing column. However, the bending moment of the frame column is much larger than that of the free-standing column with the same impact load. In other words, although the strictly constrained frame column shows a positive influence on the dynamic shear resistance at the first force phase, it plays a negative role in column response during the vibration phase, which may be induced by the formation of plastic hinges.

6 Conclusion

The pendulum impact tests are performed in this study to investigate the failure modes and impact resistance capacity of the free-standing column and frame columns with or without axial load. Numerical models, which is corresponding to the experimental impact conditions, have been developed with nonlinear finite element software LS-DYNA. The effects of the dynamic impact load on the axial force, the bending moment, the shear force, and the energy distribution of the column are examined, respectively. With these experimental and numerical results, the following conclusions can be drawn:

- 1) When impact load is applied at the top of the free-standing column, the concrete bottom can be poorly damaged with ultimate impact, and bending failure is more likely to occur at the column bottom than shear failure at the column top.

2) The applied axial load has a remarkable impact on the dynamic responses of the columns. Although the axial load plays a positive effect (e.g., reducing the residual displacement and increasing the impact-resistant capacity), it also has adverse effects (e.g., increasing the local damage at the column bottom). However, the impact-resistant performance of columns can be enhanced with the increase of the reinforcement ratio. Thus, the overall and local damages can be greatly reduced.

3) The frame column structure has a relatively larger lateral stiffness, and its impact-resistance and energy-absorbing capability are almost two times as much as that of the free-standing column structure. The damage of the frame columns under ultimate impact is shown on both ends of the column, which reflects as developing flexural failure in the vibration phase.

4) The study is performed with the validated FE modeling to explore the transmission and distribution of the internal force along with columns under the impact load. The axial load can cause an increase of peak impact force of the column during the impact. However, the shear response generated by the impact event is less sensitive to the axial load, and the maximum bending response even decreases. The negative axial force under tension can be avoided because the axial load increases the lateral resistance of the column. The impact force can cause the alternatively positive and negative characteristic on the end moment of the frame column. Thus, it needs to be paid extra attention to in the design.

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(Note: RC02-B is the back column in the double frame columns, while RC02-F is the front column in the double frame columns.)

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Tab 1. Structural Scaling Factors of Model Tests

	Parameter	Symbol	Relational Expression	Scaling Factor
Geometry	Length	SL	*	1/4
Property	Displacement	SL	*	1/4
Material Property	Strain	S_{ϵ}	S_{σ} / S_E	1
	Stress	S_{σ}	*	1
	Elasticity Modulus	S_E	$S_E = S_{\sigma}$	1
	Mass	SM	$S_{\sigma} \times S_{2L} / S_a$	1/16
Load	Force	SF	$S_{\sigma} \times S_{2L}$	1/16

Property	Moment	SMO	$S \sigma \times S3L$	1/100
Dynamic Property	Load Rate	SV	$(SL \times Sa)0.5$	$(1/4)0.5$
	Frequency	Sf	$SL-0.5 \times Sa0.5$	$(1/4)-0.5$
	Acceleration	Sa	*	1

(Three physical quantities, length, stress and acceleration, were chosen as control scale parameter)

Tab 2. Summary of tested column specimens

Specimens	Height /mm	Section diameter /mm	Axial compression ratio	Longitudinal reinforcement diameter/mm	Reinforcement ratio	Stirrup spacing /mm	Stirrup ratio
RC-P0-1	800	150	0	6	0.96	100	0.47
RC-P0-2	800	150	0	8	1.71	50	0.94
RC-P1-1	800	150	0.1	6	0.96	100	0.47
RC-P1-2	800	150	0.1	8	1.71	50	0.94
RC02	800	150	0	8	1.71	50	0.47

Tab 3. Material properties

Materials	ρ /kg/m ³	f_c /MPa	f_y /MPa	E /GPa
Concrete	2400	38	-	30
Longitudinal reinforcement	7800	-	235	200
Stirrup	7800	-	235	200
Prestressed tendon	7850	-	785	195

Tab 4. Summary of impact parameters

No	m/kg	θ / °	v_a /m/s	E_k /J
1	150	15	0.93 (Impact01)	47
2	150	30	1.84 (Impact02)	183
3	150	45	2.72 (Impact03)	400
4	150	60	3.55 (Impact04)	684
5	150	75	4.35 (Impact05)	1027

(Note: m is the pendulum mass, θ is the release angle of the pendulum, v_0 is the reduced velocity, E_k is the impact energy)

Table 5. Summary of test results

No	RC-P0-1			RC-P0-2			RC-P1-1		
	F_{max}	D_{max}	Resi	F_{max}	D_{max}	Resi	F_{max}	D_{max}	Resi
	/kN	/mm	/mm	/kN	/mm	/mm	/kN	/mm	/mm
1	60	10.42	3.52	91	8.35	2.36	77	5.24	2.09
2	132	19.5	12.20	178	13.7	8.19	186	12.48	6.12
3	157	29.2	23.99	180	24.61	20.14	199	22.46	14.67
4				200	38.21	33.14	208	34.27	26.04
5				169	45.21	40.21	199	39.22	30.12

No	RC-P1-2			RC02		
	F_{max}	D_{max}	Resi	F_{max}	D_{max}	Resi
	/kN	/mm	/mm	/kN	/mm	/mm
1	98	4.77	2.35	137.0	3.09	0.44
2	189	10.32	6.44	208.4	9.41	3.85

3	190	18.85	12.28	212.5	15.73	7.26
4	199	26.18	21.02	215.1	20.01	10.31
5	195	35.12	28.12	230.3	24.36	13.35

(Note: $F_{\max}$ is the peak impact force; $D_{\max}$ is the peak displacement of pier top; R_{esi} is the residual displacement at the column top after impact; E_a is the energy dissipation of the column, obtained by integrating the impact time history curve with the displacement time history curve of the column top.)

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