

Article

Impact Resistance Performance and Damage Characteristics of Mortise-and-Tenon Joint Prefabricated Bridge Piers

Shukun Li ¹, Jiahe Mi ¹, Xingjun Qi ^{1,*}, Yudong Jin ² and Yupeng Guo ²

¹ School of Traffic Engineering, Shandong Jianzhu University, Jinan 250101, China

² JiQing High Speed Railway Co., Ltd., Jinan 250100, China

* Correspondence: qixingjun@sdjzu.edu.cn

Abstract: The mortise-and-tenon joint prefabricated connection combines the assembly form of mortise-and-tenon joints and cast-in-place wet joints. It achieves reliable joint connections through small joint depths and lap-spliced reinforcement lengths. To study the impact resistance and damage characteristics of the assembled pier, a nonlinear finite element analysis was performed on the assembled and monolithic pier model piers to study the effects of mortise-and-tenon joint depths, lap reinforcement, and grout on the response of the piers to vehicle impact. The results showed that, after impact, the damage to the prefabricated pier was similar to that of the monolithic one. The failure mode involved opening of the seam at the impact face-pier bottom junction and localized concrete compression at the back-impact face pier bottom, and damage accumulated from the column base towards the column centerline. The mortise-and-tenon joint provided substantial horizontal constraint for the pier, imparting excellent resistance to lateral stiffness. Consequently, both piers showed nearly identical peak impact forces, yet the prefabricated pier exhibited a lesser degree of bending deformation compared to the monolithic one. The depth of the mortise-and-tenon joints was a critical factor affecting the impact response of the prefabricated bridge pier. When the depth reached 0.4D or more, it ensured good impact resistance and joint connection, enhancing energy absorption capability and reducing pier damage. The length of lap-spliced reinforcement significantly affected the overall integrity of prefabricated component connections. Lap lengths of 10d or more greatly reduced the probability of failure in the connection between pier columns and cap beams, lowering damage to the pier columns, joints, and pier cap beams, thus ensuring good impact resistance. The diameter of the lap-spliced reinforcement and the elastic modulus of the grouting material affected the local stiffness near the joints. Increasing the diameter of the lap-spliced reinforcement appropriately prevented excessive local damage, while altering the elastic modulus had minimal impact on improving pier damage.

Keywords: collision; prefabricated bridge pier; damage characteristics; impact resistance performance; UHPC



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1. Introduction

With the rapid development of the transportation industry, the number of motor vehicles and their carrying capacity continue to increase, leading to frequent accidents involving vehicles colliding with bridge piers, often resulting in pier damage. In severe cases, this can cause the collapse of bridge structures, leading to traffic paralysis, casualties, and other serious consequences. This issue has attracted widespread attention from experts and scholars around the world [1–4]. Prefabricated bridge piers meet the modern bridge construction and development needs for rapid, green, and civilized construction [5], and are widely used in practical engineering projects. However, there is little research on the impact of vehicles on prefabricated bridge piers [6]. Therefore, there is an urgent need to study the collision mechanism and crashworthiness of prefabricated bridge piers, which creates significant engineering implications.

Mortise-and-tenon connections involve directly inserting prefabricated pier columns into pile caps or cap beams, and injecting grouting material into the gap to form an integrated prefabricated connection. This type of connection is characterized by fast construction and low precision requirements. Insert-type connections have attracted significant attention from scholars both domestically and internationally due to their intuitive structural characteristics and stable connection effects. Osanai et al. [7] conducted lateral resistance studies on the insert-type connection structure between pier columns and foundations in frame structures, investigating the stress transfer mechanism between pier columns and foundations, and proposing design methods for insert-type connections and stress calculation formulas at connection nodes. Haraldsson et al. [8] designed and fabricated three specimens for horizontal cyclic loading tests to study the seismic performance of insert-type structures, with results indicating that the sensitivity of structural hysteresis performance to insert depth was not high, but the influence of insert depth on structural failure modes was significant. Xu et al. [9,10] conducted pseudo-static test analysis on prefabricated specimens with different insert depths, with results showing that the seismic performance of specimens with different insert depths was essentially equivalent to cast-in-place structures, and the increase in grout elastic modulus had an optimizing effect on the horizontal anti-push stiffness of specimens; Han et al. [11] conducted model tests on bridge piers subjected to vehicle impact, studying the dynamic response of piers under lateral impact forces, with results indicating that insert depth was an important factor affecting the extent of damage to insert-type bridge piers caused by vehicle impacts.

Due to significant differences in structural forms between monolithic and prefabricated bridge piers, the integrity of prefabricated bridge piers has become a research focus. The key factors affecting the reliability of mortise-and-tenon joint connections are the insertion depth [9,12] and grouting material [10,11]. Reasonably designing the insertion depth of pier columns and the amount of grouting material can ensure good overall performance. However, mortise-and-tenon connections also face drawbacks such as large insertion depths and poor durability of pier column–pile cap connections. The emergence of UHPC (ultra-high performance concrete) provides new ideas for optimizing mortise-and-tenon connections. Its excellent durability and workability can compensate for the lack of durability in prefabricated connection joints [13] and effectively protect pier columns and reinforcement from external forces and natural environments. Since the bond strength between UHPC and reinforcement is eight times that of ordinary concrete materials [14,15], using UHPC as a wet joint grouting material can ensure segmental connections with shorter reinforcement overlap lengths [16]. Therefore, the mortise-and-tenon bridge pier is a prefabricated form that combines mortise-and-tenon connections and cast-in-place wet joint connections. Its construction requirements include longitudinal reinforcement of prefabricated pier columns overlapped inside and outside the recesses of the cap beam, connecting adjacent prefabricated components into a whole using ultra-high performance concrete (UHPC). The basic requirement for this connection is that the overlapped length of the adjacent components' reinforcement should not be less than 10 times the diameter of the longitudinal reinforcement. The dimensions of the recesses must ensure that there is sufficient cover thickness for the embedded overlapped reinforcement in the cap beam. Using UHPC as the grouting material to connect the tendon heads and overlapped reinforcements has achieved good results in optimizing the seismic performance of prefabricated bridge piers [17,18]. The tendon head insert-type connection can achieve reliable joint connection and good load-bearing performance through shorter insert depths and overlapped reinforcement lengths [19]. However, the crashworthiness of this connection form and its influencing factors are still unclear.

Bridge piers are primarily composed of steel reinforcement and concrete, providing extremely high compressive, tensile strength, and rigidity. In contrast, the bodies of small vehicles are often made of steel, aluminum, or composite materials. Although lightweight and possessing a certain level of strength, they cannot compare to reinforced concrete, so the stiffness and strength of small vehicles are usually much lower than that of

bridge piers and the pier columns are generally not severely damaged under their impact. Therefore, this article establishes a nonlinear finite element model for heavy vehicle-pier collisions to compare the damage characteristics and dynamic responses of mortise-and-tenon bridge piers and monolithic bridge piers under impact. The effects of insertion depth, reinforcement overlap length, overlap reinforcement diameter, and grouting material elastic modulus on the vehicle impact response of this prefabricated bridge pier are considered, providing a certain reference for practical engineering applications.

2. Mortise-and-Tenon Interlocking Connection Structure

The reliability of connections at joints of prefabricated components directly impacts the overall safety and usage of assembled bridges. Among various assembly connection techniques, the tenon interlocking connection are widely employed in prefabricated assembly construction due to their advantages of rapid construction and simple procedures. However, the large depth of the tenon in the pier column leads to heavy prefabricated piers, making transportation and hoisting difficult, and the durability at the connection site is insufficient. On the other hand, the cast-in-place wet joint connection exhibits good overall integrity and durability at joints, with reliable load-bearing performance, but construction is slow.

Therefore, while retaining the good load-bearing performance, this article proposes to combine the strengths of both assembly connection methods: reducing the tenon depth, lowering hoisting weight, improving construction speed and portability, and optimizing the durability of connections at joints. The proposed solution involves using UHPC connections for dovetail joint and mortise-and-tenon bridge pier structures. In this structure, the concrete tenon at the bottom of the pier is placed as the tenon part in a groove. During construction, the pier is centered based on the tenon, and the pre-embedded reinforcing bars of the pier cap footing and the longitudinal bars of the pier column are arranged to overlap inside and outside. Subsequently, UHPC is injected to connect the pier column and the pier cap footing into a whole, with the overlapping reinforcing bars connected only by UHPC grout, eliminating the need for tying or welding. The specific connection structure is illustrated in Figure 1. In the mortise-and-tenon interlocking connection structure, the tenon not only provides a positioning function but also acts as a temporary support and reinforces the assembly node connection together with the overlapping reinforcing bars.

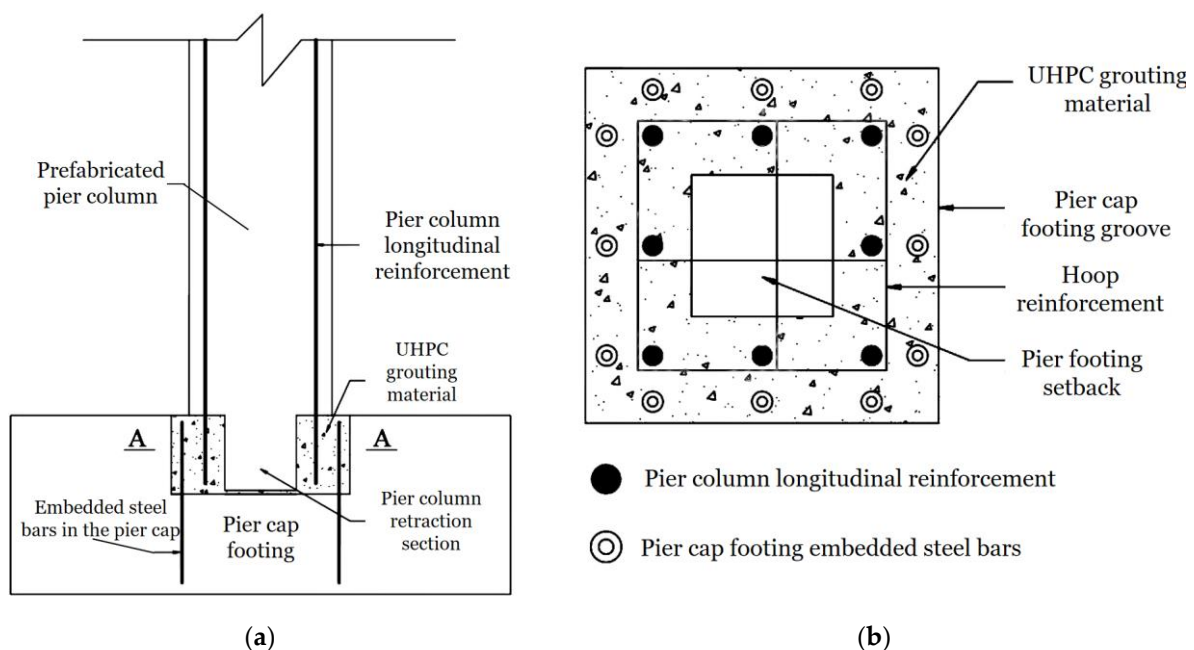


Figure 1. Schematic diagram of mortise-and-tenon joint connection. (a) Mortise-and-tenon joint connection structure and (b) A-A sectional structure.

3. Finite Element Model

3.1. Pier Model

To investigate the most unfavorable position and damage conditions of bridge piers under heavy vehicle collision, this study established a nonlinear finite element model based on ABAQUS/Explicit to simulate the impact process of heavy trucks colliding with bridge piers. Vertical degrees of freedom were released above the deck beam, and a vertical uniformly distributed load with an axial compression ratio of 0.1 was applied to simulate the constant load of the upper structure. The response of pier collision under the gravitational effect of the upper structure was analyzed. To simplify the analysis, when the distance between the pier cap footing and the ground was short, the lateral constraint effect of the soil on the pier column was ignored.

The construction and computational model of prefabricated bridge piers are shown in Figure 1, where ordinary concrete adopts C40, and UHPC grouting material is used in the groove of the prefabricated bridge pier cap footing. In the process of loading, concrete structures often exhibit nonlinear characteristics. In order to accurately simulate plastic deformation, cracking, and failure of concrete structures, especially during the initiation and propagation of cracks, it was essential to provide precise responses while ensuring computational efficiency and convergence under the premise of result accuracy. Therefore, conventional concrete and UHPC were modeled using C3D8R solid elements, and the constitutive model adopted the concrete damaged plasticity (CDP) model to simulate the collision–failure process of concrete [20], referring to the Code for Design of Concrete Structures (GB 50010-2010) [21] to determine the uniaxial stress–strain relationship of concrete, and introducing the damage evolution parameters d_c and d_t when concrete was subjected to uniaxial compression and tension, respectively. Based on the pseudo-static tests of pier columns in reference [19], this study employed a full-scale model. The total height of the mortise-tenon pier used in this paper was 11.2 m, with a pier column height of 9.2 m, a section size of 1.6 m × 1.6 m, and longitudinal reinforcement, lap steel, and pier cap footing steel all using HRB400. The diameter of the longitudinal reinforcement was 32 mm, with a reinforcement ratio of 1.01%; stirrups used HPB300, with a diameter of 16 mm and a spacing of 200 mm, with the spacing halved in the confinement zone; the steel constitutive model adopted a bilinear model, all simulated by the T3D2 truss element and embedded in the concrete solid element.

3.2. Vehicle Model

Referencing the Dongfeng Tianjin cargo truck (Table 1), a simplified model is established based on [22]. Given that the engine and cargo compartment directly influence the variation of collision forces during secondary vehicle impacts, it was necessary to consider simulation methods for these components when analyzing and calculating. Therefore, the heavy vehicle model included the cab, chassis, cargo compartment, engine, and cargo. The engine and cargo compartment were simulated using elastic solid elements, while the cab and chassis, being the main parts subject to deformation upon contact, were modeled using S4R elements (four-node reduced integration shell elements), with material selection of Q345 low-carbon steel. To accurately simulate the damage to the pier after the vehicle collision, the mass of the cab and body was distributed according to the actual proportion of the truck body mass, with the total mass of the vehicle and cargo kept constant at 30 tons.

Table 1. Dongfeng Tianjin cargo truck parameters.

Length/m	Width/m	Height/m
9.0	2.5	3.5

The collision contact type between the vehicle and the bridge pier was defined as surface-to-surface contact, with the impact point located 1.8 m above the road surface.

Tangential behavior between the vehicle and the bridge pier contact surfaces was modeled using penalty functions with a friction coefficient of 0.3 [23], while normal behavior was treated as hard contact, allowing separation after contact. When conducting analyses on different impact velocities and bridge pier design parameters, the vehicle–pier collision location consistently involved a frontal collision with zero eccentricity. The vehicle finite element model was established by defining surface-to-surface contacts between various components of the vehicle model. The impact velocity of the vehicle was achieved by altering the vehicle’s horizontal translational velocity. The cab and body were partitioned using quadrilateral mesh elements, while solid elements including the engine and cargo were partitioned using hexahedral mesh elements. The contact area was meshed finely. The simplified numerical model of the vehicle–bridge pier interaction is illustrated in Figure 2.

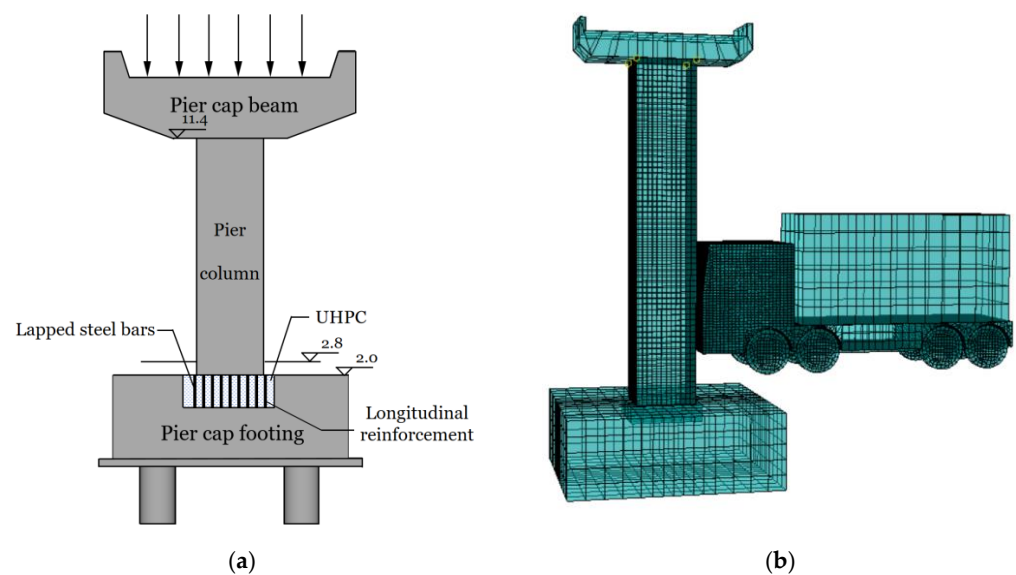


Figure 2. Vehicle–bridge pier model. (a) Mortise-and-tenon joint bridge pier structure and (b) numerical calculation model.

To scientifically conduct numerical calculations on the impact resistance performance of the prefabricated bridge pier, material parameters for ordinary concrete, UHPC, and steel reinforcement were obtained through compression and tension tests, as depicted in Figure 3. The specific material parameters are detailed in Table 2.

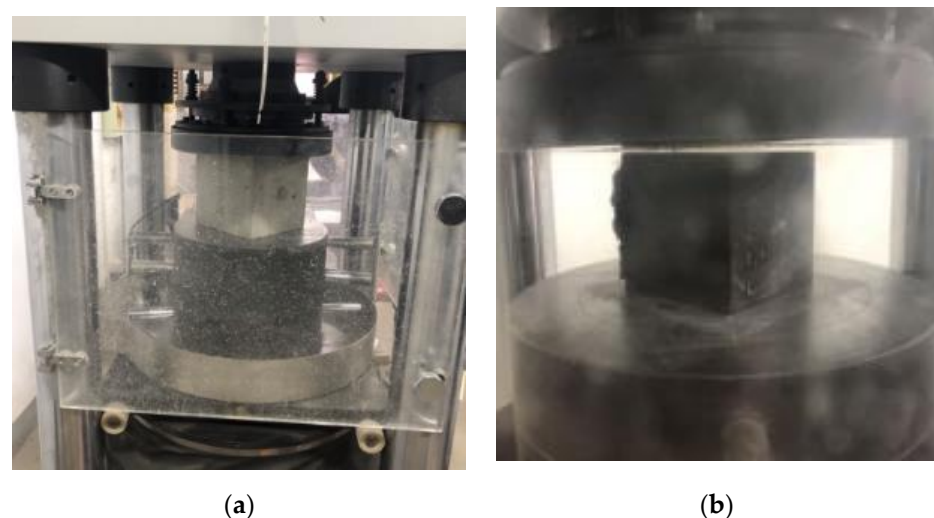


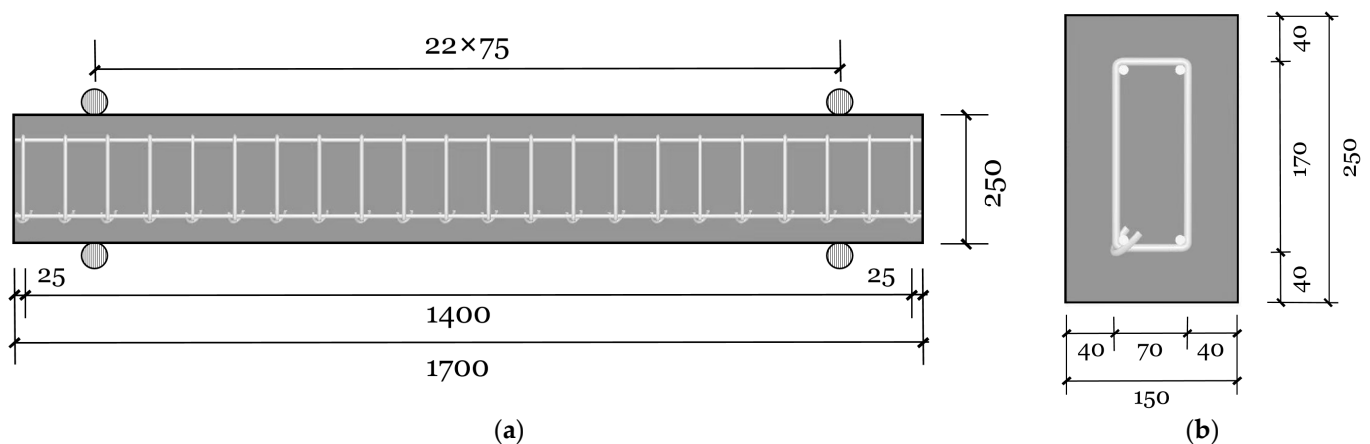
Figure 3. Mechanical properties test of materials. (a) C40 specimen; (b) UHPC specimen.

Table 2. Material parameters.

ABAQUS Material Model	Material Parameters	Value	
C40/UHPC	Density/kg·m ^{−3}	2400	2600
	Elastic Modulus/MPa	30,000	45,000
	Poisson's Ratio	0.2	0.2
	Compressive Strength/MPa	45.56	113.17
HRB400/HPB300	Density/kg·m ^{−3}	7800	7800
	Elastic Modulus/MPa	210,000	210,000
	Poisson's Ratio	0.3	0.3
	Yield Strength/MPa	439.1	382.4
	Ultimate Strength/MPa	603.9	523.3

3.3. Verification of Finite Element Model

In [24], numerical simulations were conducted on the impact test of reinforced concrete beams struck by a falling hammer to verify the accuracy and reliability of numerical simulation methods in impact dynamic analysis. Figure 4 illustrates the dimensions and reinforcement cross section of the test beam, with a length of 1.7 m, a span of 1.4 m, and a cross-sectional dimension of 150 mm × 250 mm. The beam was made of concrete with a water–cement ratio of 0.445, and the compressive strength of the concrete was measured at 42 MPa. Two D16 steel bars with a yield strength of 426 MPa were used in both the compression and tension zones of the beam. Additionally, 10 mm diameter shear reinforcement with a yield strength of 295 MPa was placed at longitudinal intervals of 75 mm.

**Figure 4.** Specimen dimensions and reinforcement (unit: mm). (a) Front view; (b) cross-section view.

The steel reinforcement adopted a bilinear elastoplastic model, taking into account the increase in steel strength with strain rate. Under the action of impact loads, it is necessary to consider the influence of strain rate. Therefore, the Cowper–Symonds model (CS model) was introduced, which enhances the yield strength of the steel by incorporating a strain rate factor with power exponent. Its formula is as follows:

$$\frac{\sigma_d}{\sigma_s} = 1 + \left(\frac{\dot{\epsilon}^p}{D} \right)^{\frac{1}{p}} \quad (1)$$

In the equation, σ_d represents the stress value of the steel reinforcement at a plastic strain of ϵ^p , while σ_s represents the stress value of the steel reinforcement under static loads. ϵ^p is the plastic strain value of the steel reinforcement. D and P are parameters related to material type and strain hardening, respectively, with values of 40.4 and 5 [25].

The impact simulation was based on the ABAQUS/Explicit solver, utilizing the concrete damaged plasticity (CDP) model to simulate the behavior of concrete in the test beam.

The damage factor d was introduced to model the evolution of concrete material damage, with material failure occurring when the damage factor reaches 1. The damage factor was determined using the energy equivalence method proposed by Sidoroff based on the principle of energy equivalence. Considering that the compressive and tensile strengths of concrete increase significantly with the increase in material strain rate [26,27], dynamic increase factors C_{DIF} and T_{DIF} (Equations (2)–(7)) were introduced based on the CEB code recommendations to account for strain rate effects.

$$C_{DIF} = \frac{f_{cd}}{f_{cs}} = \begin{cases} 1 & \varepsilon < \varepsilon_s \\ \left(\frac{\varepsilon}{\varepsilon_s}\right)^{1.026\alpha} & \varepsilon_s < \varepsilon \leq 30 \text{ s}^{-1} \\ \gamma_s \varepsilon^{0.33} & \varepsilon > 30 \text{ s}^{-1} \end{cases} \quad (2)$$

$$\alpha = \left(5 + \frac{3\sigma_{cu}}{4}\right)^{-1} \quad (3)$$

$$\log \gamma_s = 6.156\alpha - 0.492 \quad (4)$$

$$T_{DIF} = \frac{f_{td}}{f_{ts}} = \begin{cases} \left(\frac{\varepsilon}{\varepsilon_s}\right)^{1.016\delta} & \varepsilon_s < \varepsilon \leq 30 \text{ s}^{-1} \\ \gamma_s \left(\frac{\varepsilon}{\varepsilon_s}\right)^{0.33} & \varepsilon > 30 \text{ s}^{-1} \end{cases} \quad (5)$$

$$\delta = \frac{1}{10 + \frac{\sigma_{cu}}{2}} \quad (6)$$

$$\gamma_s = 10^{(7.11\delta - 2.33)} \quad (7)$$

In the equation, f_{cd} and f_{cs} represent the dynamic compressive strength and static compressive strength of concrete, respectively; ε and ε_{stat} represent the strain rate values under dynamic and static loads, with ε_{stat} set to $30 \times 10^{-6} \text{ s}^{-1}$; σ_d denotes the compressive strength of concrete cubes; f_{td} and f_{ts} represent the dynamic tensile strength and static tensile strength of concrete, respectively.

The steel reinforcement was modeled using a bilinear elastoplastic model, and strain rate effects were also considered under impact loading, leading to the adoption of the Cowper–Symonds model. The impacting head of the hammer was a hemisphere with a diameter of 90 mm and a mass of 400 kg, dropped from a free-fall height of 1.2 m. To reduce computation time, an initial velocity was imparted to the hammer based on $v = \sqrt{2gh}$ ($g = 9.8 \text{ m/s}^2$) and the drop height was reduced. Both the support and the hammer were constrained as rigid bodies, with all degrees of freedom of the support fully constrained. A surface-to-surface contact simulation was employed between the impacting head of the hammer and the surface of the concrete beam, defining a friction coefficient of 0.1 between the concrete and the support.

Figure 5 presents a comparison between experimental data and numerical simulations for the impact of a falling hammer on the beam at a height of 1.2 m. This includes comparison of the impact force-time history curve, mid-span deflection curve, and beam damage. It can be observed that the simulated impact force-time history curve and displacement-time history curve fitted well with the experimental data. The errors in peak impact force, impact duration, and maximum mid-span displacement were small, at 6.59%, 6.82%, and 1.24% respectively. The simulated locations of beam damage corresponded well with the failure mode observed in the experiments, particularly showing significant damage in the area of contact with the impact head and at the bottom of the beam. In summary, the numerical simulation method based on ABAQUS/Explicit and the chosen model parameters accurately replicated the behavior of reinforced concrete structures under low-speed impact.

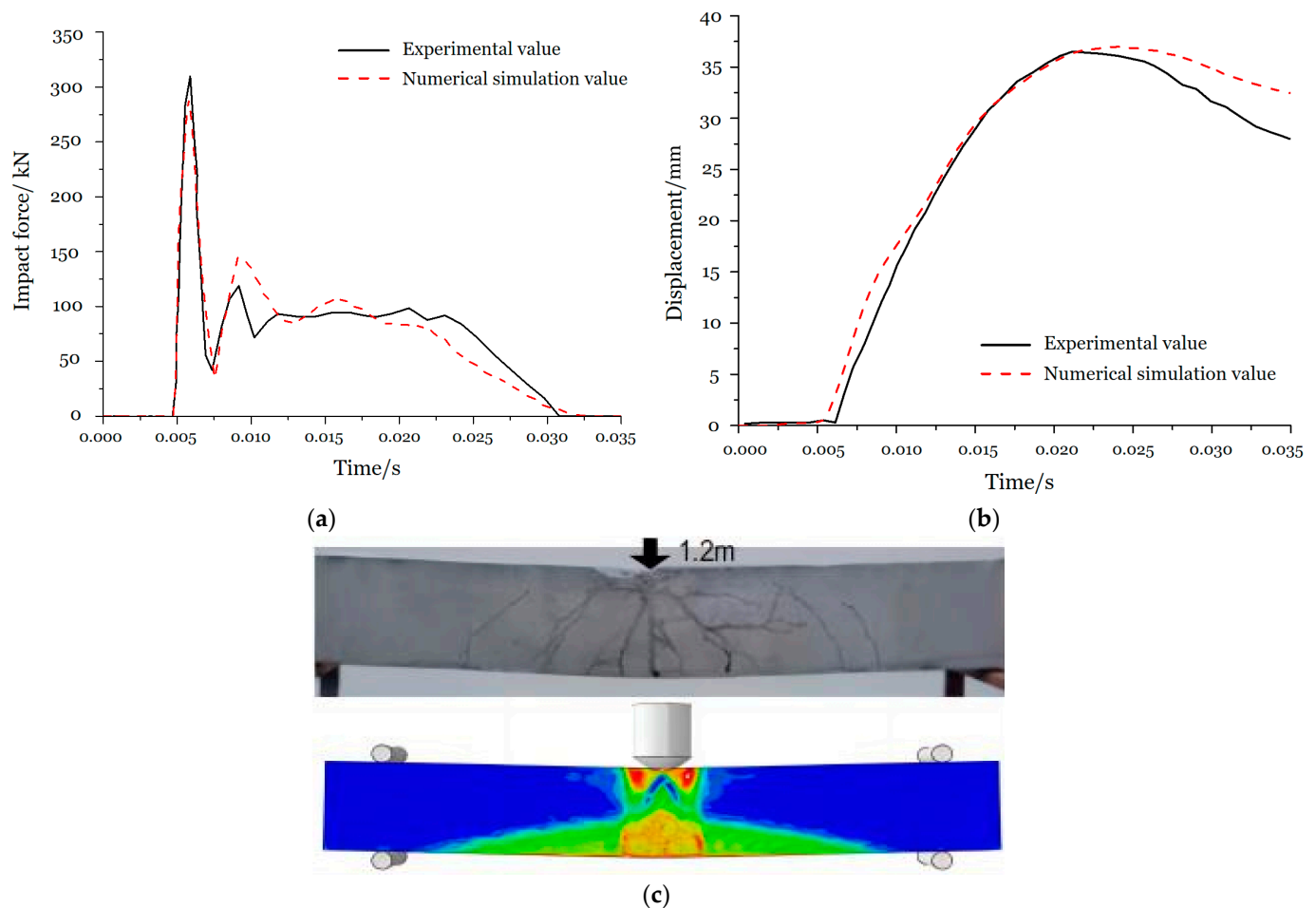


Figure 5. Experimental values and numerical simulation. (a) Impact force time-history curve, (b) displacement time-history curve, and (c) comparison between impact test and numerical simulation on failure modes.

3.4. Vehicle Impact Analysis

Energy Analysis

Figure 6 shows the energy-time curves of a 30-ton cargo truck impacting both monolithic and prefabricated bridge piers at a speed of 60 km/h. From the graph, it can be observed that before the collision, the total system energy consists only of the kinetic energy of the truck. Upon contact between the truck and the pier body, the kinetic energy begins to convert into internal energy. By the time the collision reaches 0.15 s, the energy conversion is essentially complete, and various types of energy stabilize. Throughout the entire collision process, over 90% of the kinetic energy is converted into internal energy, while the remaining energy is primarily frictional energy and hourglass energy.

Due to the use of reduced integration in solid elements in this study, hourglass effects may occur, causing some elements to undergo deformation while the calculated stress remains zero. Obviously, such results are invalid. Therefore, it was essential to control the occurrence of hourglass effects during computation. This was achieved by refining the mesh, setting hourglass controls, or changing the element type to minimize the impact. The percentage of hourglass energy in this study did not exceed 10% of the internal energy and total energy, indicating effective control of hourglass energy during simulation. This further validates the effectiveness of the numerical simulation in this study [28,29].

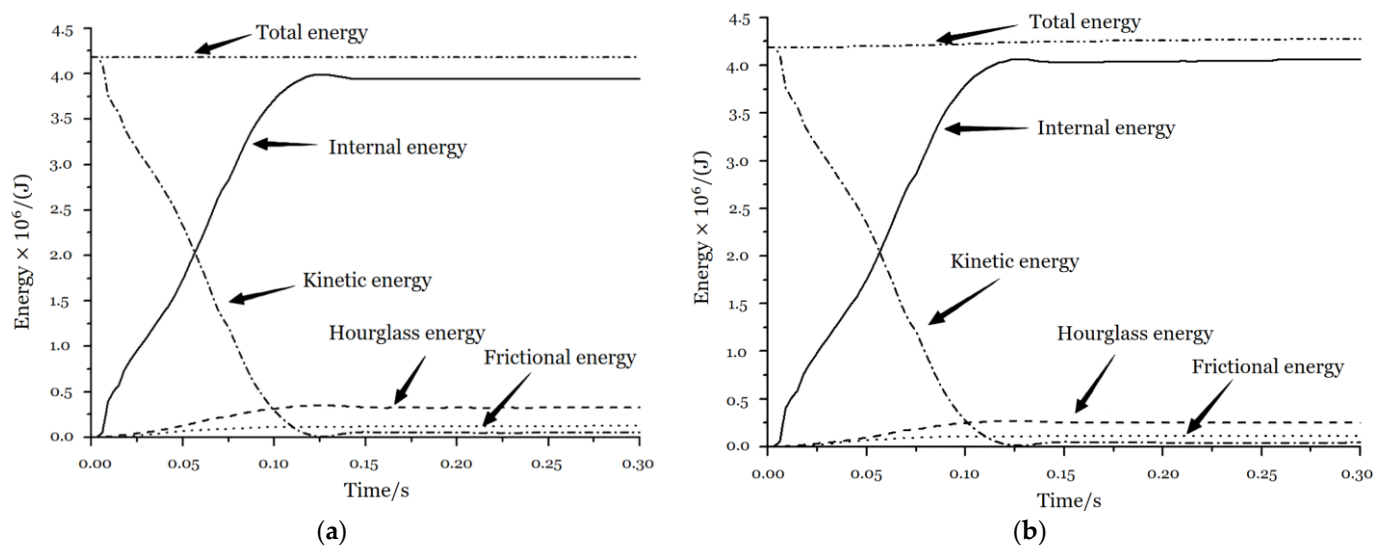


Figure 6. Energy variation time-history curve. (a) Monolithic structure; (b) prefabricated structure.

4. Result Analysis

4.1. Analysis of Damage Characteristics

Collision between the bridge piers and the vehicle resulted in significant localized stresses at the contact surface, with substantial tensile stresses appearing on the opposite side of the pier. Due to the restraint effect of the pier cap footing and the pier cap beam on the pier column, local damage occurred at the column base and top, especially noticeable at the connection between the bottom of the pier column and the pier cap footing, where significant shear deformation of the concrete occurred. When the impact velocity is low, the stress at the contact points between bridge piers and vehicles is significant. Cast-in-place bridge piers, with strong overall integrity between the pier columns and the cap beam, experience stress concentration at locations with pronounced changes in cross-sectional dimensions. Consequently, the bottom of the cast-in-place bridge piers suffered severe damage due to this stress concentration. Conversely, in prefabricated connection joints, the overall integrity was weaker than that of monolithic structures. Following a collision, the upper part of the tendon head exerts downward pressure on the UHPC grouting material. However, due to differences in material strength, the column base on the impacted side will undergo failure. The damage in the monolithic pier was mainly concentrated at the impact location and the pier base, while in the prefabricated pier, it was mainly concentrated at the impact location and the backside of the column base. Plastic strains generated in both piers were relatively small, and simple repair and reinforcement could restore them to normal use. However, as the impact velocity increased, significant plastic deformation occurred in the monolithic pier, indicating that higher impact velocities make monolithic piers more prone to irreversible plastic damage, and the severity of damage increases accordingly. In the prefabricated pier, the damage was primarily concentrated at the joint between the pier column and the pier cap footing or pier cap beam, and the degree of damage was generally consistent with that of the monolithic pier. This is because the prefabricated pier had joints, causing the pier columns to open at the joint with the pier cap footing, leading to plastic deformation and slippage of the lap reinforcement. Frictional slippage between the bottom of the pier column, the tenon structure, and the UHPC grout dissipated energy, and the UHPC grout increased the damage tolerance of the pier, significantly reducing plastic damage to the pier body.

In the concrete damaged plasticity (CDP) model, tensile and compressive damage criteria were defined to simulate the degradation of elastic stiffness caused by plastic strains. The range of development of the tensile damage factor (DAMAGEGT) and the compressive damage factor (DAMAGEGC) was 0 to 1, with larger numerical values indicating more severe concrete material damage. Plastic strain refers to the strain of material elements that

cannot recover after unloading. In Abaqus, the equivalent plastic strain (PEEQ) represents the cumulative plastic strain during the entire deformation process, and a value greater than 0 indicates that the material has entered the plastic stage. In this study, damage factors were used to evaluate the extent of concrete damage after the pier columns were impacted. The uniaxial stress–strain curve from the concrete code was introduced into the CDP model, and the damage evolution parameters were converted into damage factors suitable for Abaqus based on the energy equivalence principle. This method can be used to analyze concrete damage behavior under complex loading conditions [29]. The maximum compressive damage and tensile damage are represented by μ_c and μ_t , respectively. The degree of steel reinforcement damage is evaluated based on the maximum stress. The maximum longitudinal reinforcement stress within the impact height range is represented by σ_z , and the maximum stress value of the hoop reinforcement within the entire pier is represented by σ_g [30]. σ_z reflects the degree of deformation of the longitudinal reinforcement after bearing bending tensile stress, thereby revealing the deformation response after impact.

Figure 7 illustrates the damage conditions of the tenon joint pier after collision with a loaded truck at different speeds. It can be observed that the vehicle speed significantly affected the damage to the pier column. From the analysis of damage factors and equivalent plastic strain values, it is evident that the higher the vehicle speed, the more severe the damage to the pier. When the vehicle collided with the pier, a significant local compressive stress was generated at the contact surface, with substantial tensile stress appearing on the backside of the pier column at the contact surface. As the vehicle speed increased, the damage range of the pier column expanded accordingly. Due to the restraint effect of the pier cap footing and pier cap beam on the pier column, significant damage accumulated at the sections of the column base and top, particularly noticeable at the connection between the bottom of the pier column and the pier cap footing, where significant shear deformation of the concrete occurred.

Table 3 presents the impact responses of two piers under different vehicle speeds. It can be seen from the table that under the same initial kinetic energy, the two piers exhibited different impact responses due to structural differences. The greater the collision speed, the more significant the impact response. Under the impact of a 30-ton heavy vehicle at speeds of 80 km/h and 100 km/h, the maximum lateral displacement of the pier column of the prefabricated pier was 4.75% and 28.85% lower than that of the monolithic pier, respectively. Combined information from Figure 8 and the analysis of the maximum stress difference of the reinforcement shows a larger bending deformation of the impact area of the monolithic pier, which plays a certain buffering role in the process of vehicle impact. Although the stress of the longitudinal reinforcement was increased, the impact load on the stirrups was alleviated. However, due to the existence of joints at the bottom of the prefabricated pier, the restraint of the pier foundation on the pier column was weaker than that of the monolithic pier. Therefore, the displacement at the bottom of the pier was larger, and the bending deformation of the pier body was smaller than that of the monolithic pier, resulting in greater stress on the stirrups in the impact area. Under impacts at different speeds, the peak impact forces of the two piers differed by 3.09%, 4.75%, and 4.08%, respectively. The difference in peak impact forces between the two types of piers was not significant, indicating that the impact resistance of the tenon-and-mortise prefabricated pier was similar to that of the monolithic pier. This is because the prefabricated pier used UHPC grouting material to connect the components, which improved the local horizontal equivalent stiffness and stiffness retention capacity of the pier column. The deformation, stress–strain, and compression damage factors of the concrete in the contact area were all relatively large, indicating that there was severe damage to the concrete in this area. Concrete cracking strain was reached earlier on the back impact surface, pier bottom, and pier top, and both piers suffered severe tensile damage. In the prefabricated pier, the stress wave was transmitted from the contact surface to the pier cap footing through the grouting material. Due to the excellent mechanical properties of UHPC, the connection effect of the joints was stable and the grouting material was not damaged. However, the local stress

on the wall of the pier cap footing groove was relatively large, resulting in severe tensile damage and slight compression damage at the connection between the groove and the top surface of the pier cap footing.

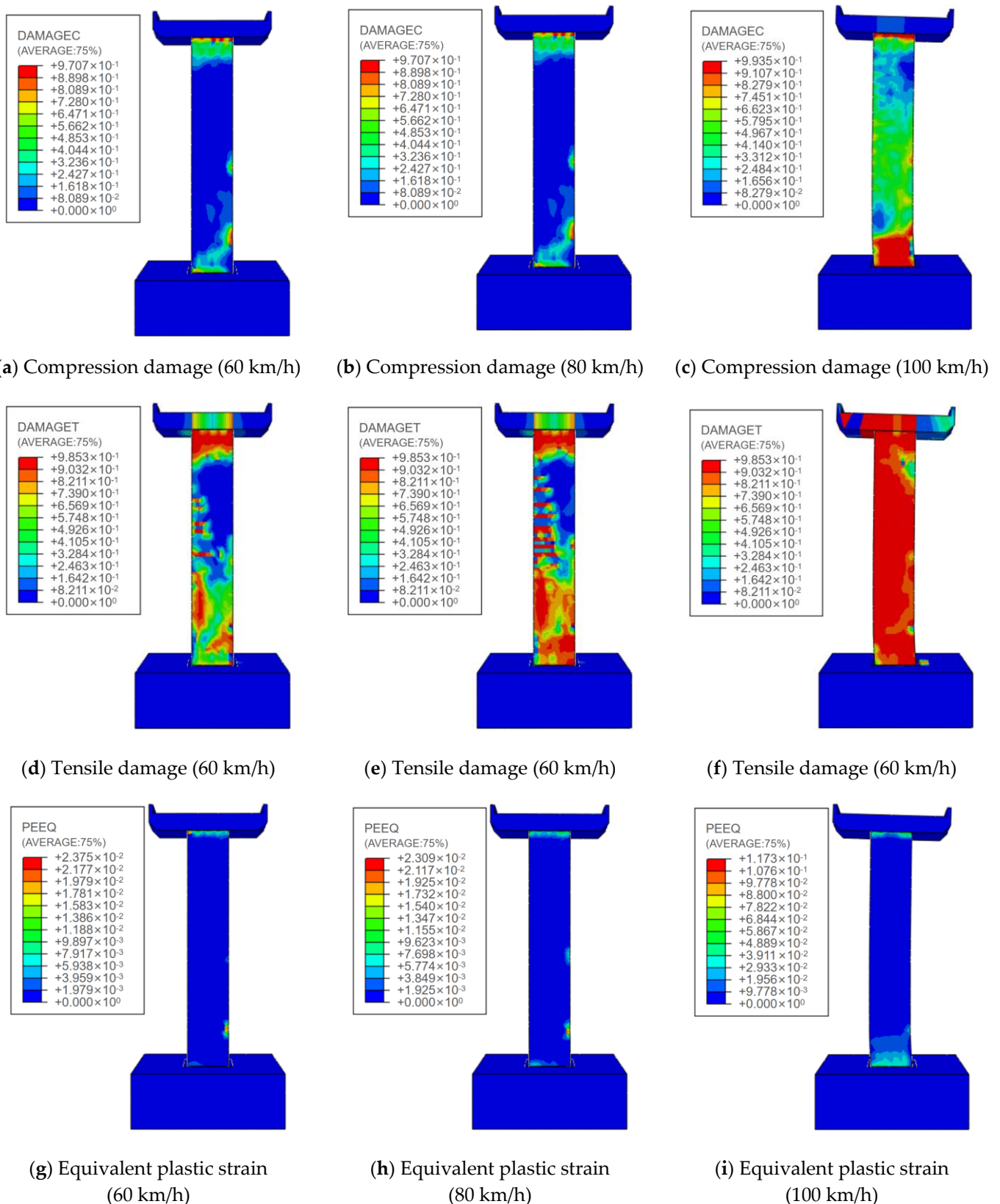


Figure 7. Vehicle-bridge pier collision damage.

Table 3. Pier column impact response.

Number	Types of Bridge Pier	Vehicle Speed/(km/h)	Maximum Horizontal Displacement/mm	Maximum Impact Force/MN	σ_z /MPa	μ_c	μ_t
1	Monolithic	60	6.79	8.85	213.59	0.97	0.98
2	Prefabricated	60	7.00	8.91	138.85	0.97	0.98
3	Monolithic	80	13.27	11.73	227.71	0.98	0.98
4	Prefabricated	80	12.64	14.08	197.05	0.97	0.99
5	Monolithic	100	156.87	34.09	537.72	0.99	0.99
6	Prefabricated	100	111.62	32.70	515.51	0.99	0.99

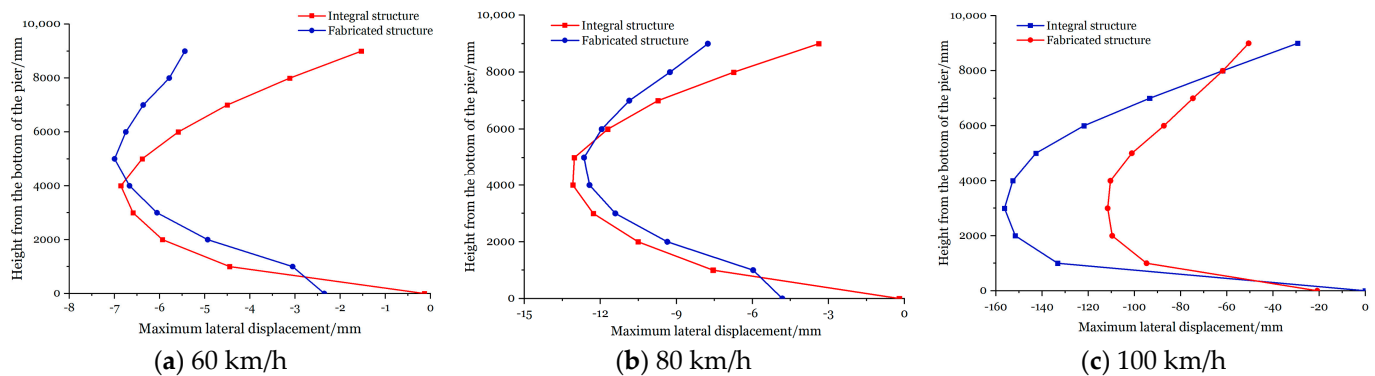


Figure 8. Maximum lateral displacement at different heights of the pier column.

4.2. Analysis of the Impact of Different Mortise-and-Tenon Joint Depths

Table 4 lists the peak impact force, maximum lateral displacement of the pier column, and energy absorption capacity of prefabricated bridge piers at different impact velocities under various mortise-and-tenon joint depths.

Table 4. Pier column impact response at different mortise-and-tenon joint depths.

Socket Depth	60 km/h			80 km/h			100 km/h		
	Peak Impact Force /MN	Maximum Displacement /mm	Energy Consumption /kJ	Peak Impact Force /MN	Maximum Displacement /mm	Energy Consumption /kJ	Peak Impact Force /MN	Maximum Displacement /mm	Energy Consumption /kJ
0.2D	8.67	6.62	177.88	12.47	10.98	222.67	24.07	100.52	391.50
0.3D	8.89	7.12	142.03	12.60	10.77	204.20	24.77	104.56	401.35
0.4D	8.91	7.00	142.26	14.08	12.64	210.86	32.70	111.63	412.82
0.5D	8.87	6.83	142.87	13.95	12.72	218.44	26.87	114.85	441.07

From the table, it is evident that at an impact velocity of 60 km/h, there was little difference in the peak impact force, maximum lateral displacement, and energy absorption capacity of prefabricated bridge piers under different mortise-and-tenon joint depths. However, when the impact velocity reached 80 km/h, there was a negative correlation between the joint depth and the maximum lateral displacement of the pier column. When the joint depth exceeded 0.3D (where D represents the diameter or width of the pier column), both the peak impact force and the maximum lateral displacement increased compared to when the joint depth was 0.3D. This phenomenon became more pronounced at higher impact velocities.

At an impact velocity of 100 km/h, the peak impact force of the 0.4D joint depth was higher than that of the 0.2D, 0.3D, and 0.5D joint depth piers. This is because joint depths below 0.3D resulted in unstable connections at the joints of prefabricated bridge piers. During the transmission of impact loads from the pier column to the pier cap footing foundation via grouting material, excessive local stresses occurred at the pier column

mortise-and-tenon joints, grouting material, and pier cap footing grooves, leading to severe damage to the pier column and affecting impact resistance. Larger joint depths provide greater constraint for the pier column mortise-and-tenon joints, exhibiting characteristics similar to those of monolithic bridge piers when the joint depth was large.

Except for the 0.2D joint depth prefabricated bridge piers, under different impact velocities, total energy absorption increased with increasing joint depth. This is because larger joint depths resulted in greater bonding and frictional forces between the pier cap footing grooves and grouting material. Insufficient constraint of the pier column mortise-and-tenon joints by the pier cap footing and grouting material in the 0.2D joint depth piers led to severe pier column vibration and increased frictional cumulative energy consumption at the joints when subjected to impact loads.

At different mortise-and-tenon joint depths, the local stress generated by the same kinetic energy on unit area grouting material varied, with smaller joint depths leading to easier grouting material damage. Despite this, UHPC grouting material did not exhibit significant damage under various vehicle impact speeds. However, the pier cap footing grooves suffered varying degrees of damage to their walls and bottoms after bearing the stress transmitted by the pier column. Figure 9 compares the damage factors of the pier cap footings and grooves under different mortise-and-tenon joint depths and vehicle impact conditions. It can be observed that under different collision speeds, the compressive and tensile damage to the concrete near the pier cap footing grooves decreased with increasing joint depth, and the damage to the pier cap footings and grooves became more severe with higher speeds.

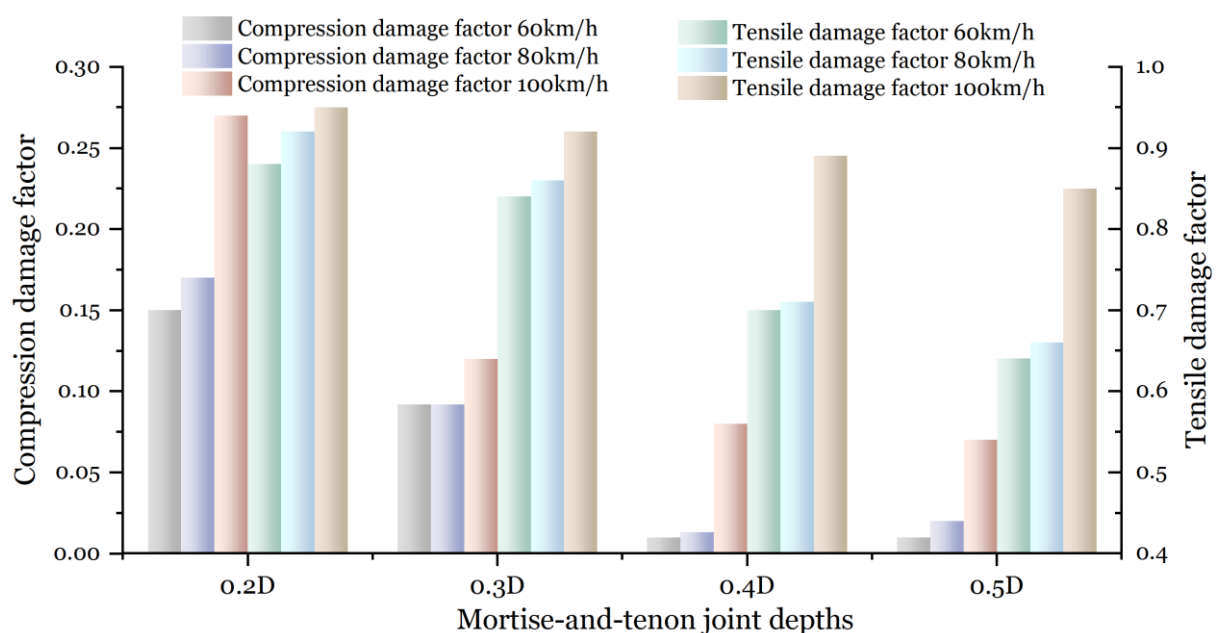


Figure 9. Comparison of damage to pier cap footing grooves at different mortise-and-tenon joint depths.

In summary, the mortise-and-tenon joint depth of the pier column is a crucial factor affecting the impact resistance of the mortise-and-tenon joint prefabricated bridge pier. As the joint depth increases, the bonding strength, frictional resistance, and lateral resistance provided by the grouting material and pier cap footing also increase, enhancing the pier column's energy absorption capacity. Insufficient joint depth makes the connection at the joints of prefabricated bridge piers unstable, affecting the impact resistance of the pier column and resulting in severe damage to the pier cap footing, grooves, and embedded lap-spliced reinforcement in the pier cap footing. Appropriately increasing the joint depth

can improve the overall integrity of prefabricated bridge piers and reduce pier column damage under impact.

4.3. Analysis of the Impact of Overlapping Reinforcement

The bonding action between the pier column, the embedded reinforcement in the pier cap footing, and the grouting material provides a tight connection for prefabricated components. As the material used in the joint areas of the prefabricated pier, its performance has a significant impact on the connection effectiveness of the components and the integrity of the pier.

The length of the overlapping reinforcement directly affects the connection effectiveness of adjacent prefabricated components. UHPC can ensure reliable connections between prefabricated segments even with shorter reinforcement overlap lengths. Figure 10 illustrates the impact response of piers under different reinforcement overlap lengths, taking a vehicle speed of 100 km/h as an example, where d represents the diameter of the overlapping reinforcement between the pier column and the pier cap footing. It can be observed from the figure that when the overlap length was small, the maximum displacement of the pier body was significantly higher than the corresponding value when the overlap length was $7.5d$ or above. Especially at the bottom of the pier, excessive deformation occurred, indicating that an insufficient length of reinforcement overlap can lead to unreliable connections between adjacent components and poor stress transmission between the grouting material and the overlapping reinforcement. When stress was transmitted to the grouting material, it resulted in a significant accumulation of plastic damage at the pier bottom, greatly increasing the probability of failure in the connection between the pier column and the embedded reinforcement in the pier cap footing. Excessively short reinforcement overlap segments also caused excessive local stress in the grouting material and the pier cap footing groove, indicating that insufficient overlap length can cause severe damage to the grouting material and the pier cap footing, affecting overall safety and making it difficult to repair. There was a positive correlation between the peak impact force and the reinforcement overlap length, indicating that a reasonable reinforcement overlap length can ensure good impact resistance for prefabricated piers.

After the collision, some of the overlapping reinforcement in the pier cap footing groove entered the plastic stage, indicating that the overlapping reinforcement fully participated in force transmission and energy dissipation during the pier column's resistance to impact. Therefore, a parametric analysis was conducted on the diameter of the overlapping reinforcement. Figure 11 compares the impact response of piers with different diameters of overlapping reinforcement under a vehicle speed of 100 km/h. It can be observed from the figure that changes in the diameter of the embedded reinforcement in the pier cap footing enhanced the restraining effect of the pier cap footing on the pier column and the integrity of the assembled column. The increase in local stiffness of the groove and grouting material resulted in greater deformation of the upper pier column. The maximum equivalent stress in the grouting material increased as the diameter of the embedded reinforcement in the pier cap footing increased, while the maximum equivalent stress in the pier cap footing groove decreased accordingly. This indicates that larger-diameter overlapping reinforcement reduced local damage to the concrete in the pier cap footing groove. Therefore, leveraging the high strength and durability of UHPC, appropriately increasing the diameter of the overlapping reinforcement effectively protected the pier cap footing groove. Compared to changing the length of the overlap segment, increasing the diameter of the overlapping reinforcement did not significantly improve impact resistance performance.

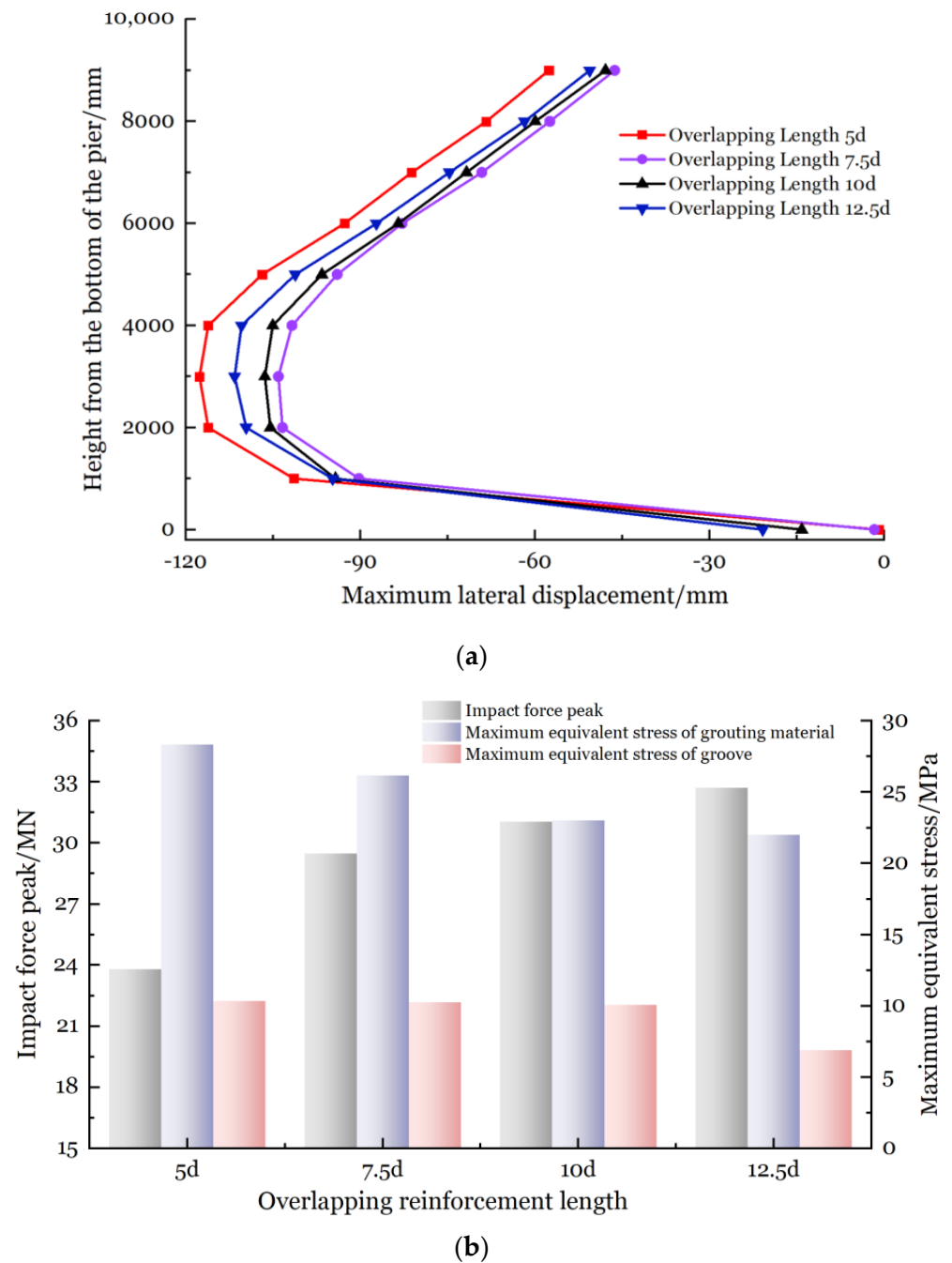


Figure 10. Bridge pier impact response at different overlapping reinforcement lengths. (a) Pier displacement response; (b) pier impact force response.

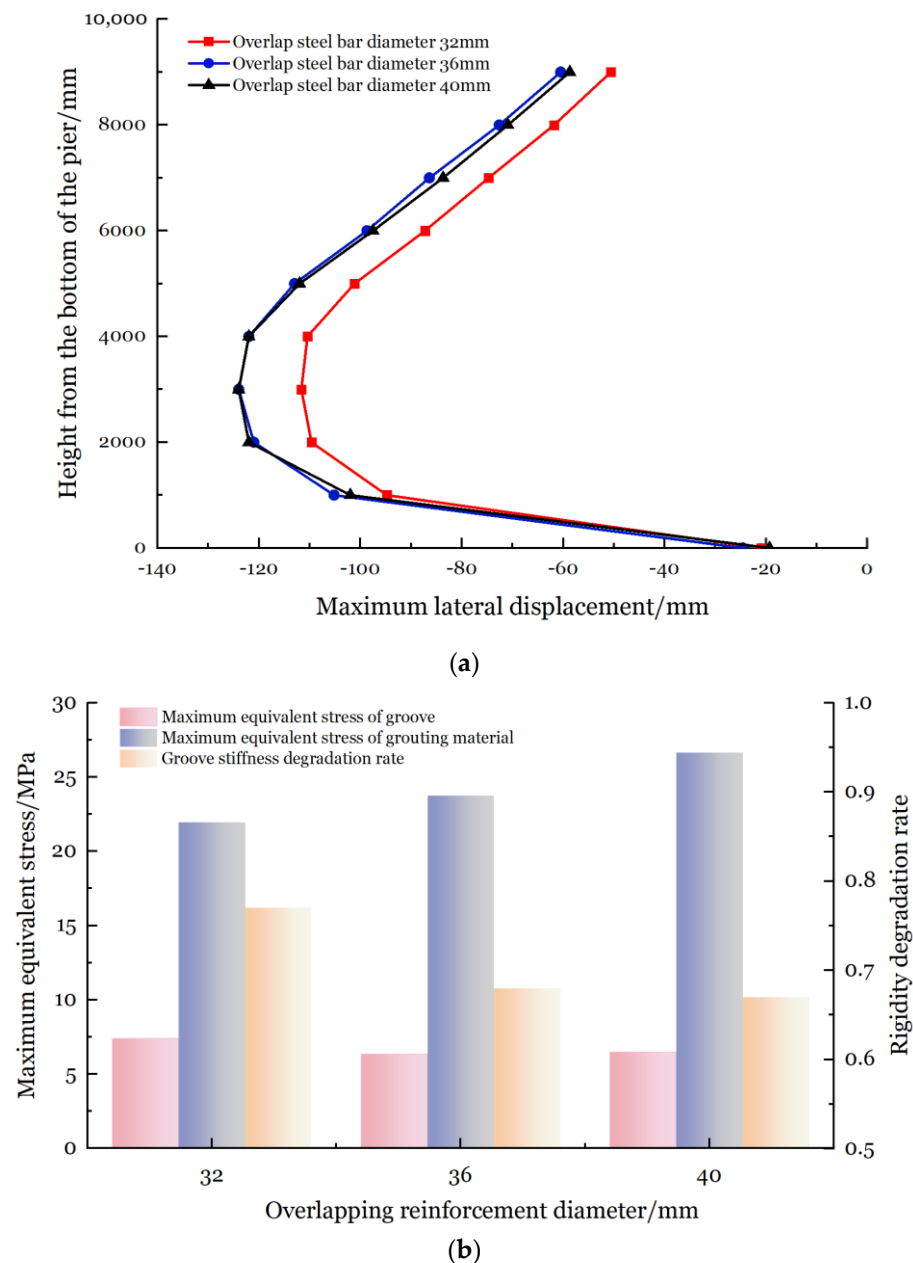


Figure 11. Bridge pier impact response at different overlapping reinforcement diameters. (a) Pier displacement response; (b) pier cap footing and grouting material impact response.

4.4. Analysis of the Impact of Grouting Material

The grouting material used in the mortise-and-tenon joint pier was UHPC, which has a significantly higher strength than ordinary concrete. Therefore, it was not likely to fail prematurely during operation and loading. Hence, the strength of UHPC grouting material is not discussed in this analysis. To investigate the impact of the elastic modulus of grouting material on impact resistance, a comparison was made between the anti-collision performance of piers with grouting material elastic moduli of 50 GPa, 55 GPa, and 60 GPa.

Figure 12 illustrates the impact response of piers under different elastic moduli of grouting material. It can be observed from the figure that as the elastic modulus of the grouting material increased, the deformation of the pier body also increased. The bending deformation acted as a buffer against the impact force, which was due to the enhanced horizontal constraint effect of the grouting material on the mortise-and-tenon joint. This improved the integrity at the joint and enhanced the horizontal stiffness near the groove.

Increasing the elastic modulus enhanced the grouting material's ability to resist elastic deformation, but it had no significant impact on its strength and stress. Therefore, changing the elastic modulus of the grouting material has limited influence on mitigating the damage to pier cap footings and grooves.

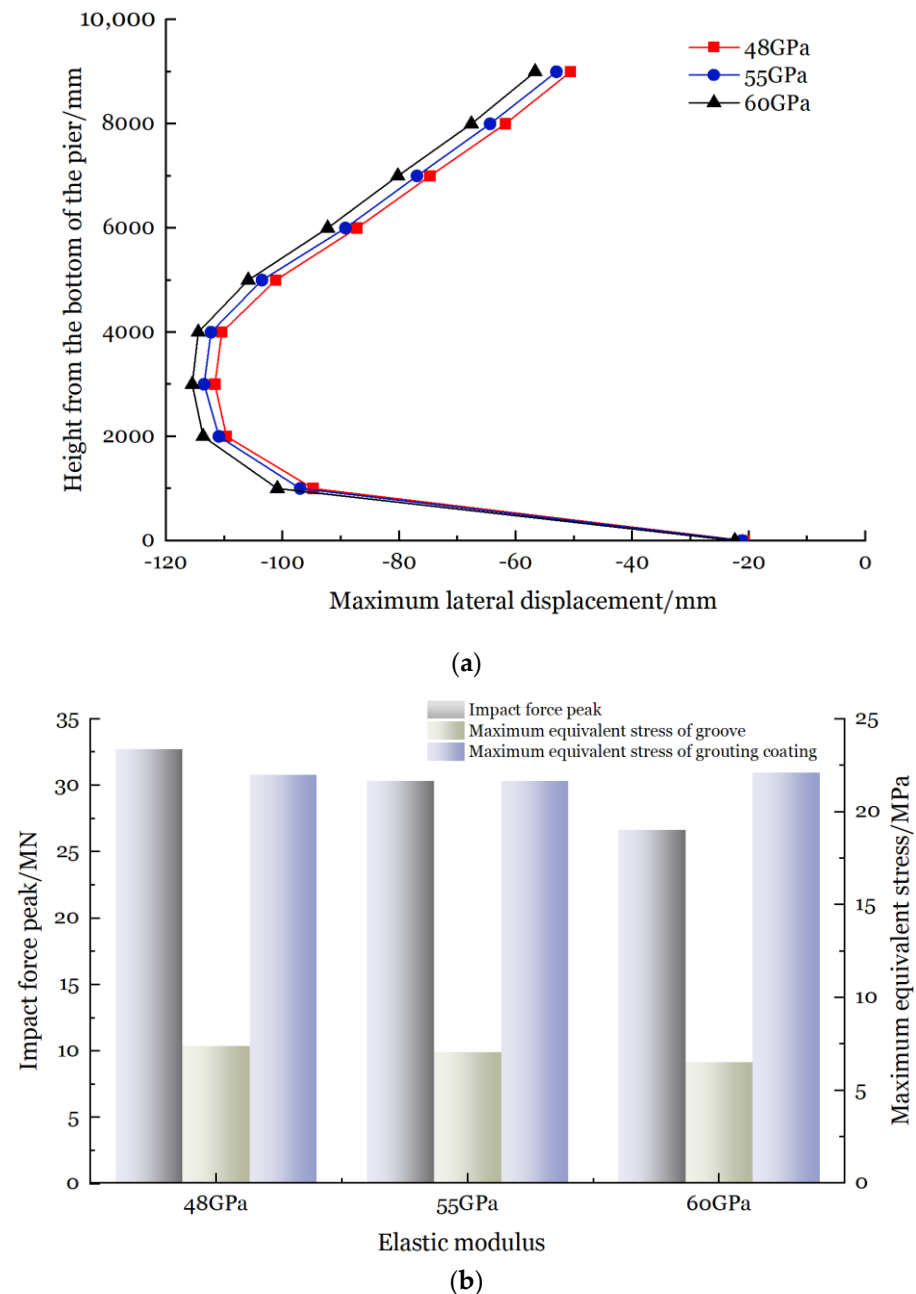


Figure 12. Bridge pier impact response at different grouting material elastic moduli. (a) Displacement response; (b) impact force response.

5. Conclusions

This research conducted an elastoplastic dynamic analysis of both mortise-and-tenon joint piers and monolithic piers using a 30-ton heavy-duty truck. It comparatively analyzed their dynamic responses and damage patterns under impacts at different speeds and examined the design parameters that affected their crashworthiness. The specific conclusions are as follows:

1. The plastic damage level of prefabricated piers is similar to that of monolithic piers. However, due to differences in structural forms, their failure modes differ. When impact speeds were low, monolithic piers primarily experienced bending failure of the pier column. As impact speeds increased, the failure mode transitioned to oblique shear failure that propagated from the impact surface to the pier base. Prefabricated piers, on the other hand, have joints, resulting in weaker integrity compared to cast-in-place piers. Their failure mode manifested as the opening of joints at the base of the impact surface and localized concrete crushing at the back impact surface. As the duration of impact increased, the damage area gradually expanded from the column foot to the column mid-line. Upon impact, the vehicle's kinetic energy rapidly converted into strain energy and crack propagation energy in the pier, displaying significant nonlinear characteristics. Monolithic piers exhibited severe damage at the contact surface and cross-sectional transition areas, while prefabricated piers showed notable damage at the contact surfaces and joints.
2. Compared to monolithic piers, the peak impact force of prefabricated piers was not significantly different. However, their displacement response gradually decreased, indicating that the mortise-and-tenon joint provided a considerable horizontal restraint for the pier column. The lateral force generated by the impact induces immense shear stress within the pier. Internal stirrups effectively resisted this shear force, preventing the pier from disintegrating due to shear failure. Although the UHPC-lapped reinforcement connection provides prefabricated piers with strong lateral resistance, the insufficient integrity at the joints and the weaker shared load-bearing capacity of the reinforcement compared to monolithic piers resulted in excessive stress on local stirrups. Nevertheless, due to insufficient integrity at the joint and weaker shared load-bearing capacity of steel reinforcement compared to monolithic piers, local stirrups may experience excessive stress.
3. The mortise-and-tenon joint depth is a crucial factor affecting the crashworthiness of prefabricated piers. An insufficient insertion depth affected the connection between the pier column and the pile cap, leading to excessive local stress concentrations in the dowel, grouting material, and pile cap recess, resulting in severe damage. As the insertion depth increased, the displacement response and energy dissipation capacity showed an increasing trend, while damage near the joint was significantly reduced. However, a large insertion depth may cause the pier column to exhibit characteristics similar to an elastic foundation beam, reducing the impact force response.
4. Compared to the diameter of the overlapping reinforcement, the length of the overlapping segment had a more significant impact on the crashworthiness of piers. When the length of the overlapping segment was too short, it significantly increased the failure probability of the overlapping connection between the embedded reinforcement in the pier column and the pile cap, deepening the damage to the pier column and pile cap. Within the range of 5d to 12.5d for the length of the overlapping segment, the peak impact force of the pier column positively correlated with the length of the overlapping segment. Simultaneously, it reduced damage to the grouting material and pile cap, indicating that a reasonable length of the overlapping segment effectively protected the connection at the joint.
5. The diameter of the overlapping reinforcement and the elastic modulus of the grouting material had a significant impact on the stiffness of the recess and grouting material. Increasing the diameter and elastic modulus enhanced the restraint effect of the pier base on the tenon joint, resulting in increased deformation of the pier body. Bending deformation acted as a buffer against the impact force, thus reducing the peak impact force. Appropriately increasing the diameter of the overlapping reinforcement controlled excessive local damage in the recess, while changing the elastic modulus had minimal impact on improving pier damage.

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