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Research Article

Mechanical Response Analysis of Double-Column High Piers Impacted by Rolling Stones under the Influence of Pier Height Differences

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Abstract

Bridges with double-column high piers are mostly used in mountain areas. This kind of bridge can adapt to complex landform. But the double-column high piers have weak anti-impact capability, which are easily broken under the impact of rolling stones. In order to enhance the performance of bridge piers in resisting against the impact of rolling stones, we conducted the simulation analysis of the mechanical response of double-column high piers. First, we used the explicit dynamic analysis software LS-DNYA to establish the impact model between rolling stones and double-column high piers. Second, we extracted the impact force curve of rolling stones, time history curve of reinforcement stress and change curve of joint displacement at the top of pier to analyze the mechanical response laws of bridge piers under the change of height difference of the pier. Results show that the maximum impact force of rolling stones is not significantly affected by the height difference of the pier. The maximum impact force of rolling stones is mainly influenced by the characteristics of the rolling stones. The height difference of pier has adverse effect on the stress of reinforcement. The maximum joint displacement at pier top is increased with the increase in the height difference of the pier, and the growth rate of maximum joint displacement at the top of piers firstly gradually increased and then declines step by step with the increase in the pier height difference in the direction of impact. The maximum joint displacement at pier top is firstly gradually increased and then reduced as the pier height difference is increased in the non-impact direction. The obtained conclusions provide a significant reference for the design of rolling stone impact resistance of bridges in mountainous areas.

Keywords: Double-column high pier; Rolling stone impact; LS-DYNA

1. Introduction

Various countries in the world have built expressways or railways to improve traffic conditions and develop transport networks in mountain areas in recent years. However, due to its special topography, there are many high mountains, gorges, river valleys, and deep ravines. Thus, bridges are built to connect traffic routes. The lower part of the bridge mostly adopted double-column high piers, and the heights of the piers are different. In practical design of bridge piers, the solid pier column design without consideration of pier height difference usually involves large excavation and workload filling and occupies excessive mountain resources. The design of double-column high piers considering the height difference can conserve building space and materials. Bridge with double-column high pier constructed in mountain areas considered the pier height difference can improve the construction efficiency and avoid excessive occupation and utilization of mountain resources, such as bridge (Fig.1) in Nan chuan District, Chongqing City, China. It completed the expected function of easing the traffic pressure. In addition, the double-column high piers can better adapt to the special terrain of mountainous mountains with high cliffs and steep canyons considering the design of a certain pier height difference. Due to these characteristics, the design of double-column high piers considering the height difference has been widely applied in the bridge construction in mountain areas in the world, such as Bai hua Bridge in China (Fig.2). However, this bridge pier has weak anti-impact capability. They can be easily broken under the impact of rolling stones in mountain areas. This shortcoming affected the safety of structures, personnel, and running vehicles along the traffic lines in mountain areas. With the increasing construction of bridges in mountainous areas in recent years, double-column high piers play an increasingly significant role in the traffic route network. The frequency of rocks hitting the bridge is gradually increasing, which further increases the impact on the bridge structure. Therefore, analyzing the mechanical response of bridge piers to the impact of rolling stones under varying pier height difference and formulating the corresponding anti-impact measures is necessary to ensure the safety of pier structures.

Double-column high piers are different from solid piers. Therefore, connecting beams will be built at appropriate positions at the height of the pier to increase the stability of the pier structure. The force characteristics of double-columns are quite different from those of general single-pillar piers or double-pillar piers without tie beams. [1]. When the double-column high piers are impacted by rolling stones, their mechanical properties and bearing capacities are changed. They suffered from damage and destruction, which seriously affected the normal service functions of bridges and the safety of traffic line [2]. For the bridge pier damage under impact, previous research focused on the

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bridge pier damage under the impact of ships or vehicles [3-8]. These bridge piers are kept away from mountainous areas and mostly located in cities, with relatively flat terrain and relatively low pier height. The impact mechanisms of ships and vehicles are different from that of rolling stones to some extent [9]. In comparison with other single-column piers or double-column bridge piers with small pier height in mountain areas, double-column high piers are featured with extremely great height and poor rigidity. Thus, they usually experience complicated nonlinear structural changes. The materials undergo great deformations under the impact of rolling stones [10]. The bridge piers undergo reinforcement deformation and concrete damage due to the impact of rolling stones, especially the impact of high-velocity rolling stones. This condition directly degraded the bearing capacity of bridge piers and changed the original stress state of structures above the bridge due to the transmission of impact stress wave transmission of rolling stones. There by affecting the safety of upper structures and the whole bridge [11]. Hence, the research results regarding the ship–pier impact and vehicle–pier impact should not be used in the anti-impact performance evaluation or formulation of anti-impact measures for already built double-column high piers or newly built double-column high piers. With the rapid development of finite element computer technology, software technologies have gradually become a powerful research means adopted by numerous scholars. LS-DYNA is mature and reliable explicit dynamic analysis software widely favored by researchers from all over the world. This software can be used to analyze a series of nonlinear dynamic problems, such as metal forming and manufacturing, rapid collision, and explosion. This software is applicable to the solving and analysis of heat transfer, fluid problem, and fluid–solid coupling problem. The accuracy and reliability of the software analysis results are verified through repeated tests [12-17].

Fig. 1. Changba Bridge

Fig. 2. Baihua Bridge

Many scientific researchers and working personnel have used LS-DYNA dynamic software to investigate the rolling stones–bridge pier collision problems in recent years. However, few studies have focused on double-column high piers. At the same time, the pier height difference has been rarely considered.

The typical double-column high piers in mountain areas in Nan chuan District, Chongqing City, China, were taken as the concrete study objects. On the basis of the existed research results, we use the explicit dynamic analysis software LS-DYNA to simulate the impact process of rolling stones for the double-column bridge piers considering the different pier height. We analyze the mechanical response of the double-column high piers under working conditions. We obtained the pier height-dependent change laws of maximum reinforcement stress and maximum displacement at pier top. The findings can serve as reference for the anti-impact (rolling stones) design of double-column high piers and reinforcement measure design in mountain areas.

2. State of art

The impact force of rolling stones is an important basis for the regional protective structural design [18]. The impact force of rolling stones is calculated by referring to the relevant empirical approaches as stipulated in the tunnel specifications in China's highway and railway industries, including the Tunnel Manual in the Railway Industry, the calculation formula for the impact force of rolling stones proposed by Yang[19], and its improved formula. Foreign specifications include semi-empirical and semi-theoretical algorithms, such as the calculation method for the impact force of rolling stones proposed by Japan Highway Public Corporation [20] and that recommended by Swiss Labiouse (1996)[21]. However, the factors considered by these formulas are incomplete. The impact force of rolling stones is affected by complicated factors. Thus, the impact force of rolling stones calculated by these formulas deviates from practical engineering to a certain degree. For instance, Ye compared the existing calculation formulas for the impact force of rolling stones [22] and pointed out the error between the impact force results of rolling stones calculated by the existing calculation formulas and practical engineering results. He found that the results calculated through the Japanese and Swiss formulas accord with the practical engineering by comparatively analyzing the different calculation methods. However, their scope of application is restricted by complex influencing factors. Many scholars have conducted numerous studies to obtain an accurate calculation formula for the impact force of rolling stones. Hou [23] explored the kinetic characteristics of rolling stones and the calculation methods for the impact force of rolling stones by considering the autorotation factor of rolling stones and applied them to the rolling stone − induced disaster analysis in practical engineering. However, accurately estimating the shape and size of rolling stones in practical situation is difficult. An assumption applicable to this formula is that the rolling stones are spherical, and the impact force result of rolling stones obtained using this formula is partially large. Richie [24] studied the motion rules of rolling stones. All of the abovementioned studies with respect to the impact force of rolling stones have attached importance on the influences of the characteristics of rolling stones, such as velocity, mass, impact position, and shape, on the impact force of rolling stones. The change laws of the impact force of rolling stones with the velocity, mass, and impact position, and the relational expressions of impact force with various factors are analyzed. However, the impact force of rolling stones is affected by the characteristics of the impacted structure, such as the structural form, strength, and material characteristics. Thus, the effect factors of the impact force of rolling stones are incomplete. Different structures present different responses to the impact of rolling stones due to the material and density. Thus, the influence borned by the structures from the impact of rolling stones cannot be ignored.

Therefore, directing at the deficiencies of the existing studies, Changba Bridge in Nanchuan District, Chongqing City, China, was taken as the concrete study object. The mechanical response of double-column bridge piers to the impact of rolling stones under varying pier height difference was studied on LS-DYNA software. The time history curve of the impact force of rolling stones, reinforcement stress – strain curve, and displacement change curve at pier top were extracted. The change laws of the maximum response value of reinforcement stress and the maximum response value of displacement at pier top with the pier height difference were analyzed and summarized.

The remainder of this study is organized as follows: Section 3 expounds the parameter determination of finite element modeling. Section 4 proposes the time history curve of the impact force of rolling stones, reinforcement stress–strain curve, and displacement change curve at pier top and analyzes the results. Section 5 summarizes the study and provides the relevant conclusions.

3. Methodology

3.1 Engineering background

The prototype simulated in this study was the double-column high piers of Changba Bridge located in Nanchuan District, Chongqing City. The bridge was an integral type bridge, where the left and right breadths were 12 m, the overall bridge width was 24 m, the numbers of left and right bridge spans were 5, and the bridge length was 220 m. The layout drawing of bridge elevation is shown in Fig.3, where No.1 and No.2 bridge piers of the bridge were hollow thin-walled piers with the pier height of 47 and 60 m, respectively. The No.3 bridge pier of the continuous beam bridge was a double-column high pier with a tie beam structure, whose pier height of 40 m, and the No.4 bridge pier of the continuous beam was a double-column pier, with the pier height of 14 m. The double-column high pier (No.3 bridge pier) with the height of 40 m was numerically simulated. The bridge pier had a circular cross section with the diameter of 2.3 m, and the height of beam was 2.2 m. The profile map of No.3 bridge pier is shown in Fig.4. The concrete strength grade of bridge piers was C50. The thickness of concrete protective cover was 50 mm. The reinforcement strength grade was HRB335. The diameters of longitudinal bar and stirrup of bridge piers were 28 and 12 mm, respectively. A 2 m reinforced section was arranged at the joint between bridge pier and bent cap and that between bridge pier and pile foundation, and the stirrup spacing was 10 cm. The No. double-column high pier with 40 m in height was selected for the simulation study.

3.2 Finite element modeling

In the finite element simulation of double-column high piers under the impact of rolling stones in mountain areas, solid 164 solid elements were selected as the concrete elements. We selected the Holmquist-Johnson-Cook model [31-36] to study the impact problem as the material constitutive model. The reinforcement elements were LINK160 elements. The material model was bilinear kinematic hardening elastic-plastic model, Solid164 solid elements were selected as the rolling stone elements, and the material constitutive model was a rigid model. The material parameters of concrete, reinforcement, and rolling stones are follows:

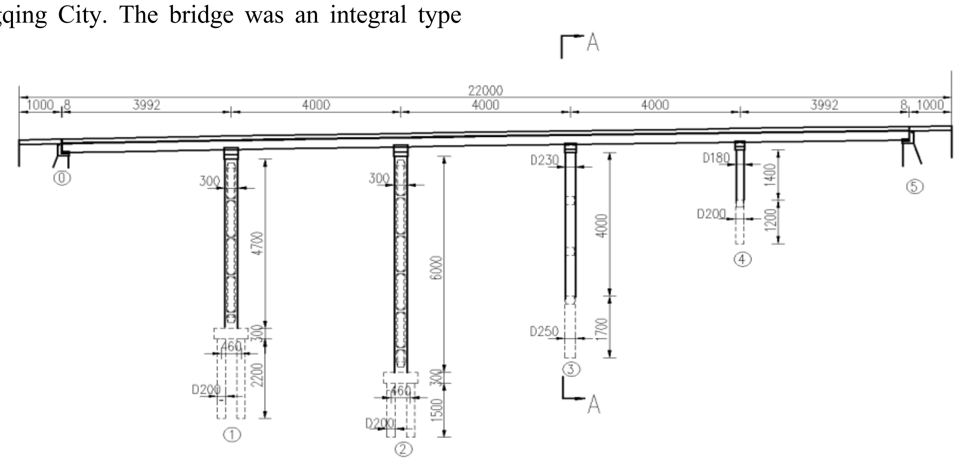


Fig.3. Layout Drawing of Bridge Elevation (unit: cm)

Table 1. HJC material constitutive parameters of double-column high pier concrete

ρ_0 / g•mm^{-3}	G/GPa	$f^{'}$ / MPa	A	B
0.0025	19.33	58	0.79	1.6
C	N	S_{max}	D1	D2
0.007	0.61	7.0	0.04	1
$\varepsilon_{f\,min}$	T / MPa	P_{crush} / MPa	μ_{crush}	P_{lock} / GPa
0.01	5.8	12	0.829e-3	0.8
μ_{lock}	K_1 / MPa	K_2 / MPa	K_3 / MPa	$\dot{\varepsilon}_0$
0.1	85	-171	208	1

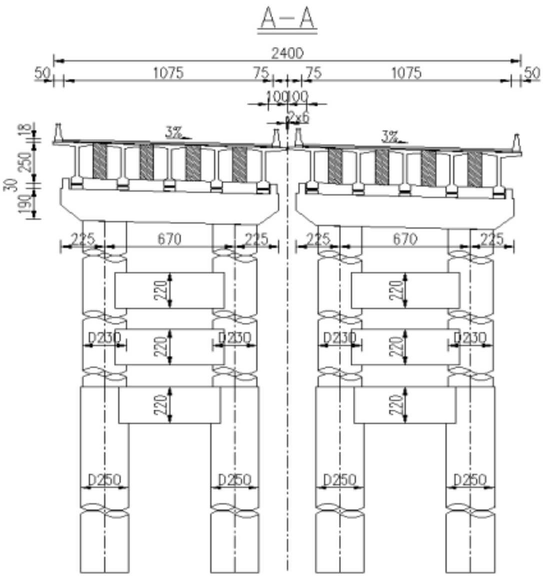


Fig.4. Profile Map of NO.3 Bridge Pier (unit: cm)

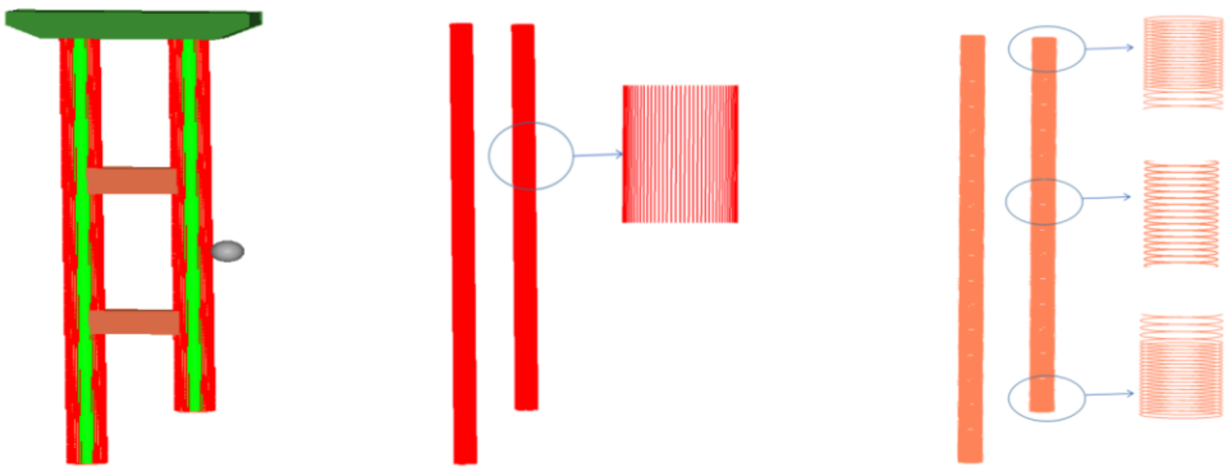
Table 2. Material parameters of reinforcement constitutive model

Project	ρ_0 / g•mm^{-3}	E/GPa	υ	σ_0/MPa	E_t/MPa	C/S^{-1}	**P**	f_s	β
reinforced	0.0078	206	0.3	335	1.2	40	5	0.2	0

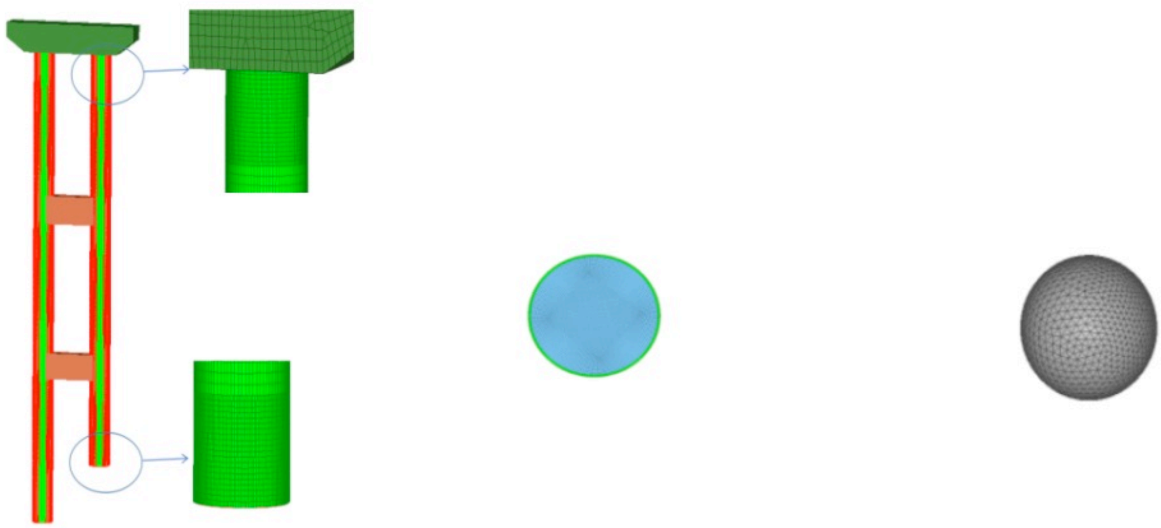
Table 3. Rolling stone material parameters

Project	ρ_0 / g•cm^{-3}	E_s/GPa	υ
Rolling stone	2.3	2.63	0.22

In the modeling process, the pier concrete model was established in three separate parts: concrete protective cover (25 mm) in outer ring, concrete protective cover (25 mm) in inner ring, and concrete column (radius: 1.15 m) in inner ring, to display the concrete damage conveniently. The mesh size of the circular cross section of bridge pier was determined as 100 mm in accordance with the spacing of longitudinal reinforcement to facilitate the establishment of longitudinal bars in the mesh generation for the cross section of pier concrete. In the mesh generation for the elevation of pier concrete, local mesh refinement was performed within the 2 m range of pier bottom and top, and the mesh size was 100 mm. No mesh refinement was conducted within the 2 m pier concrete zones at the bottom and top of bridge piers. Thus, the meshes were enlarged to 200 mm to facilitate the follow-up stirrup establishment in the refinement zone and reduce the model calculation time. The size of pier concrete was identical with the reinforcement size. The finite element models of rolling stones and bridge piers after the mesh generation are shown in Fig.5. The simulation calculation time was set as t = 0.1 s.



(a) Physical models of rolling stones and bridge piers (b) Longitudinal bars (c) Stirrups



(d) Pier concrete (e) Rolling stone (f) Rolling stone
Fig. 5. Finite Element Models of Rolling Stones and Bridge Piers

3.3 Model working conditions
The simulation was conducted considering the height differences in the impact direction and those differences are listed in Table .4.

Table 4. Working conditions of different pier height

Working condition	Mass(t)	Speed(m/s)	Impact position	Pier structure form	Pier height difference
Operating condition 1 (basic operating condition)	5	10	Middle	Double-column high piers	0
Operating condition 17–25	5	10	Middle	Double-column high piers	Impact direction difference 1–9 m
Operating condition 26–34	5	10	Middle	Double-column high piers	Non-impact direction difference 1–9 m

4. Result Analysis and Discussion

4.1 Impact force analysis of rolling stone
The time history cure of the impact force borne by the double-column high piers from the rolling stones under the basic working conditions was extracted, which is shown in Fig.6.

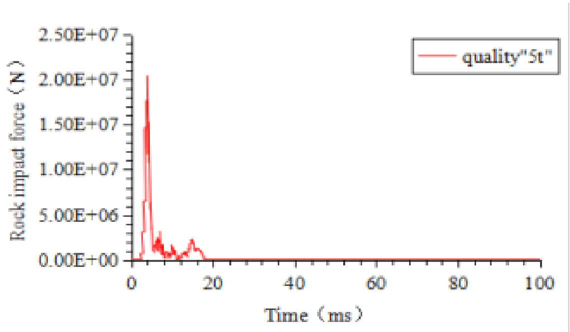
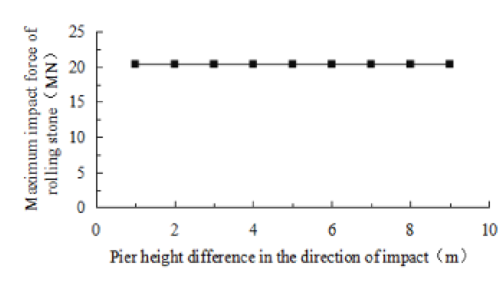


Fig. 6. Time History Curve of Impact Force under Basic Working Conditions

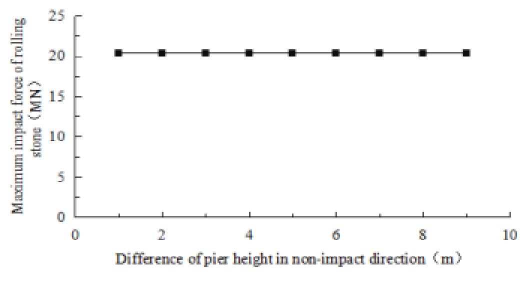
The impact force of rolling stones reached the maximum value of 20.37 MN at t=3 ms. After the maximum value, the impact force reduced to a small nonzero value at t=4 ms and then presented an oscillating variation trend with the compression of rolling stones and pier concrete. When the rolling stones no longer contacted the concrete, the impact force became zero, that is, the impact force of rolling stones was 0 MN after t=10 ms in the time history curve chart. The reason to the impact force did not rapidly decline to 0 after reaching the maximum value was the interaction between the rolling stones and bridge pier concrete. Specifically, the motion acceleration and velocity direction of rolling stones faced the impact direction before the rolling stones impacted the pier concrete. The bridge piers generated a reactive force under the impact of rolling stones when the rolling stones impacted the pier concrete due to the impact deformation resistance of the pier concrete itself. Thus, the motion acceleration of rolling stones faced the reverse direction of impact direction. However, the rolling stones moved toward the impact direction instantaneously when bearing the reactive force of bridge piers due to the inertia. The kinematic velocity was rapidly reduced. When the impact force of rolling stones exceeded the concrete bearing capacity, the concrete was damaged and fell off, leading to the change in the reactive force generated to the rolling stones and affecting the kinematic velocity of rolling stones. Hence, the impact force of rolling stones declined from the maximum value to a small nonzero value, as shown in the time history curve of impact force. Subsequently, it was changed in an oscillating manner with the compression of rolling stones–bridge pier concrete until the rolling stones no longer contacted the concrete, and the impact force became zero.

Above analysis indicates that the impact force reaches the maximum immediately after the concrete of double column high pier structure is impacted by rolling stones. The impact force rapidly declined to a small nonzero value after reaching the maximum value, and then presented an oscillating change with the compression of rolling stones and concrete due to the interaction between rolling stones and pier concrete, where the impact force was smaller than that at the moment of impact. When the rolling stones no longer contacted the pier concrete, the impact force became 0. Therefore, focusing on the maximum impact force of rolling stones is necessary.

The maximum attack force values of rolling stones under different pier height differences were extracted, which is shown in Fig.7. The maximum impact force of rolling stones was mostly unaffected by the pier height difference.



(a)

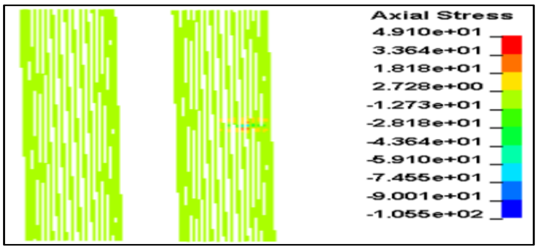


(b)

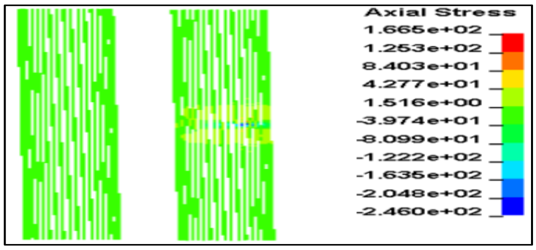
Fig.7. Maximum Impact Forces of Rolling Stones under Different Pier Height Differences

4.2 Time history analysis of reinforcement stress

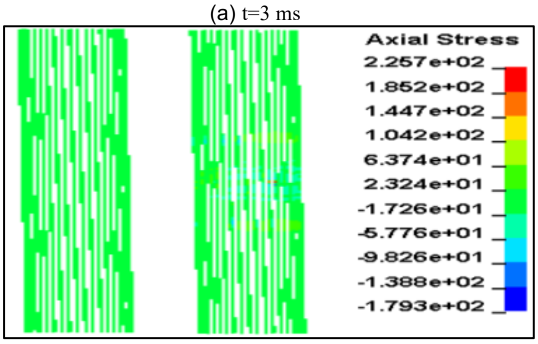
The reinforcement stress distribution in the double-column high pier under the impact of rolling stones can reflect the part of pier reinforcement susceptible to the impact of rolling stones. The stress nephograms of longitudinal bar and stirrup in the bridge pier under the basic working conditions at typical time are shown in Fig.8 and Fig.9.



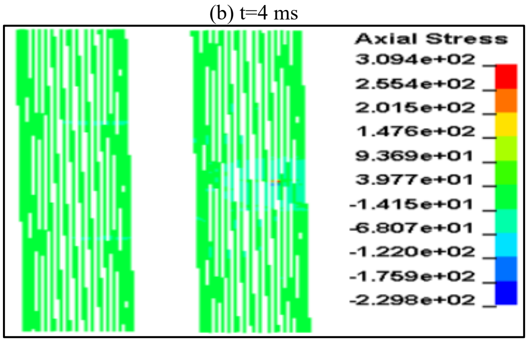
(a) t=3 ms



(b) t=4 ms

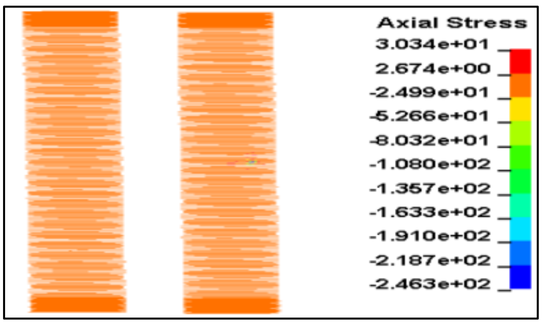


(c)t=7 ms

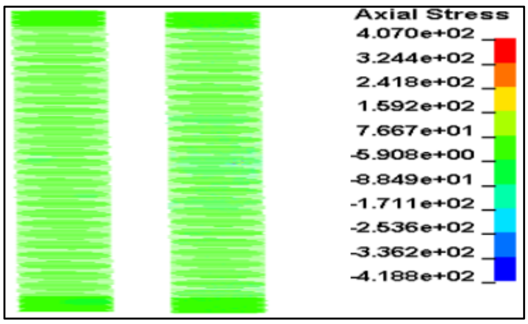


(d) t=10 ms

Fig.8. Stress Nephogram of Longitudinal Bars in Bridge Pier under Basic Working Conditions



(a)t=3 ms



(b) t=15 ms

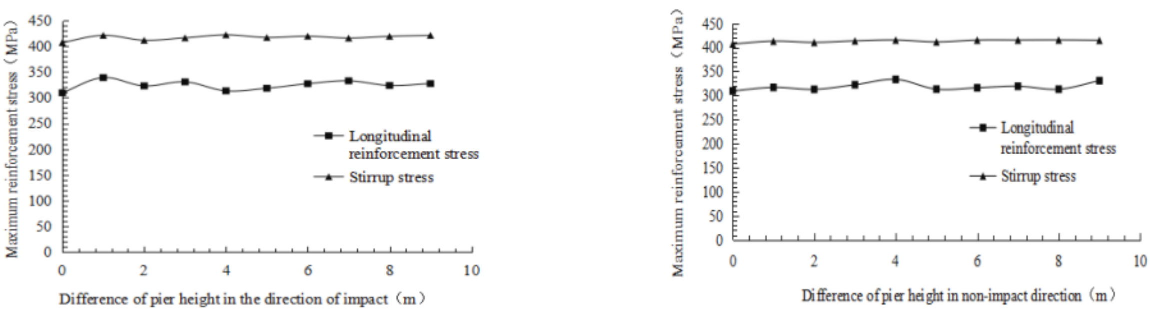
Fig.9. Stress Nephogram of Stirrups in Bridge Pier under Basic Working Conditions

As shown in Fig.8, the longitudinal bar stress at the impact part of bridge pier changed and experienced the stress concentration phenomenon at t=3 ms. The maximum longitudinal stress appeared at the impact part with the maximum stress value of 49.1 MPa, and the longitudinal bars in the bridge pier did not reach the yield strength, being under the elastic phase. The longitudinal wave stress at the impact part started diffusing around due to the stress wave transmission. At t=4 ms, the stress concentration phenomenon appeared at the back, upper edge, and lower edge of the impact part of longitudinal bars in the bridge pier. At t=10 ms, the longitudinal bars reached the maximum stress of

309.368 MPa, which was extremely close to the yield strength of longitudinal bars. However, this condition still indicated that the longitudinal bars were in the elastic phase. As shown in Fig.9, at t=3 ms, the stirrup stress at the impact part of bridge pier changed and concentrated, and the maximum stirrup stress (30.34 MPa) appeared at the impact part. With the passing of time, the stirrup stress at the impact part continuously increased. At t=15 ms, the stirrup stress reached the maximum value of 406.977 MPa at the impact part, exceeding the yield strength of stirrups, which were no longer in the elastic phase.

On the basis of the above analysis, the maximum stresses of longitudinal bars and stirrups appeared at the impact part under the impact of rolling stones, that is, the impact part of reinforcements was influenced to the greatest extent. Therefore, the emphasis should be laid on the maximum reinforcement stress in this study.

The variation diagram of maximum reinforcement stress in bridge piers under different pier height differences is shown in Fig.10. Table.5 and Table.6 present the maximum reinforcement stress values in bridge piers under different pier height differences.



(a) (b)

Fig.10. Variation Diagram of Reinforcement Stress under Different Pier Height Differences

Table 5. Bridge pier reinforcement stress table under the impact direction of pier height difference condition

Height of pier The name of the	Bridge pier reinforcement stress (MPa)	
	Longitudinal reinforcement stress	**Stirrup stress**
0	309.368	406.977
1	338.809	421.036
2	322.751	411.511
3	330.532	416.183
4	313.129	422
5	318.16	416.872
6	327.01	419.413
7	332.637	415.684
8	323.563	419.196
9	327.224	421.036

Table 6. Bridge pier reinforcement stress under the condition of constant height difference in nonimpact direction

Height of pier The name of the	Bridge pier reinforcement stress(MPa)	
	Longitudinal reinforcement stress	**Stirrup stress**
0	309.368	406.977
1	316.768	412.907
2	312.702	410.337
3	322.263	413.517
4	333.361	415.282
5	313.141	411.329
6	315.937	415.123
7	319.138	415.258
8	312.998	415.434
9	330.725	414.766

As shown in Fig. 10, Table.5 and 6, the longitudinal reinforcement stress of bridge pier was 309.368 MPa, and the stirrup stress was 406.977 MPa under the pier height difference of 0 m. Under the pier height difference of 1 m in the impact direction, the longitudinal reinforcement and stirrup stresses in the bridge pier were 338.809 and 421.036 MPa, respectively. Those in the bridge pier under the pier height difference of 1 m in the nonimpact direction were 316.768 and 412.907 MPa, respectively. Therefore, the pier height difference generated an adverse influence on the reinforcement stress. Under the pier height difference in the impact direction, the longitudinal reinforcement and stirrup stresses presented irregular changes with the increase in the pier height difference. When the pier height difference was 1 m in the impact direction, the longitudinal reinforcement and stirrup stresses in the bridge pier reached the maximum values of 338.809 and 421.036 MPa, respectively. When the pier height difference was 4 m in the impact direction, the increase amplitudes of longitudinal reinforcement and stirrup stresses were the smallest relative to those under the pier height difference of 0 m. That is, this pier height difference influenced the reinforcement stress to the minimum extent. Under the pier height difference in the nonimpact direction, the longitudinal reinforcement and stirrup stresses presented irregular changes with the increase in the pier height difference. Specifically, the longitudinal reinforcement and stirrup stresses in the bridge pier reached the maximum values of 333.361 m and 415.282 MPa, respectively, when the pier height difference was 4 m in the nonimpact direction. Under the pier height difference of 2 m in the nonimpact direction, the longitudinal reinforcement and stirrup stresses in the bridge pier increased to the minimum extent relative to those under the pier height difference of 0, that is, the influencing degree generated on the reinforcement stress was the slightest.

The impact degree of rolling stones on the pier reinforcement was high when the height difference of double-column high piers was singular. Thus, the pier height difference should be kept at 0 as much as possible in the design. Limited by the geological conditions, the impact degree was mild under an even height difference value between double-column high piers.

4.3 Joint displacement analysis at pier top

The time history curve of joint (1097277 joint) displacement at pier top and that of joint (1250519 joint) displacement at the impact part under the pier height differences in the impact direction and nonimpact direction are shown in Fig.11 and Fig.12, respectively, to study the influence of pier height difference on the joint displacement at critical parts.

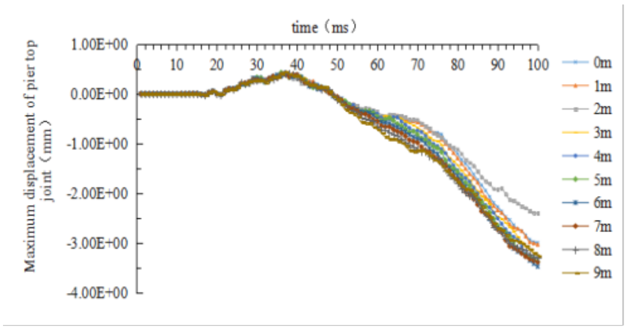


Fig.11. Time History Curves of Joint (1097277 Joint) at Pier Top under the Height Pier Differences in the Impact Direction

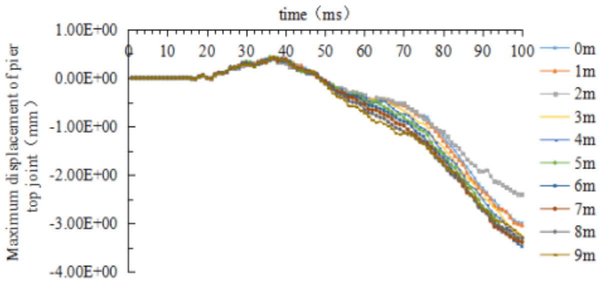


Fig. 12. Time History Curves of Joint (1097277 Joint) at Pier Top under the Height Pier Differences in the Nonimpact Direction

As shown in Fig.11 and Fig.12,the variation trends of joint displacement at pier top were relatively close under different pier height differences. These trends were increasing to the maximum displacement values first toward the reverse direction of the impact direction and then continuously increasing toward the same direction as the impact direction until reaching the maximum displacement values.

As shown in Table.7, with the increase in the pier height difference in the impact direction, the maximum joint displacements at pier top were 3.0347, 3.03764, 3.1133, 3.2271, 3.2761, 3.4414, 3.5300, 3.5974, 3.5847, and 3.3885 mm. Specifically, the maximum joint displacement at pier top increased with the increase in the pier height difference in the impact direction. The pier top growth rate gradually increased first and then declined with the increase in the pier height difference. With the increase in the pier height difference in the nonimpact direction, the maximum joint displacements at pier top were 3.0347, 3.111, 3.281, 3.331, 3.481, 3.507, 3.419, 3.3289, −3.286, and 3.035 mm. Thus, the maximum joint displacement at pier top gradually enlarged first and then reduced with the increase in the pier height difference in the nonimpact direction.

Table 7. Maximum displacement of pier top nodes under different pier height differences (unit: mm)

Height of pier location	Impact direction		Nonimpact direction	
	Y forward,	Y negative	Y forward,	Y negative
0 m	0.396	−3.0347	0.396	−3.0347
1 m	0.3908	−3.0764	0.397	−3.111
2 m	0.4072	−3.1133	0.422	−3.281
3 m	0.4131	−3.2271	0.411	−3.331
4 m	0.4214	−3.2761	0.434	−3.481
5 m	0.4094	−3.4414	0.431	−3.507
6 m	0.4124	−3.5300	0.395	−3.419
7 m	0.4158	−3.5974	0.402	−3.3289
8 m	0.4004	−3.5847	0.408	−3.286
9 m	0.3933	−3.3885	0.456	−3.035

5. Conclusions

The stress changes in bridge pier structure and displacement changes at critical parts under the impact of rolling stones were analyzed through the finite element simulation to improve the anti-impact (rolling stones) performance of double-column high pier and ensure the safety of traffic line in mountain areas. The dynamic response of the pier structure under the impact of rolling stones at different positions and different initial velocities was compared and analyzed. The factors leading to the change in the dynamic response of bridge pier were expounded, and the following conclusions were mainly obtained.

(1) In the impact process of rolling stones on the double-column high piers, the impact force of rolling stones reached the maximum value immediately after impacting the bridge pier concrete. The impact force rapidly declined to a small nonzero value after reaching the maximum value and then presented an oscillation variation trend with the compression of rolling stones and concrete due to the interaction between the rolling stones and bridge pier concrete. The impact force of rolling stones appearing in the oscillating variation phase was always smaller than that at the moment of impact. When the rolling stones no longer contacted the bridge pier concrete, the impact force became zero. Hence, focusing on the maximum impact force of rolling stones is necessary.

(2) Under the impact of rolling stones, the maximum stress values of longitudinal reinforcement and stirrups in the bridge pier appeared at the impact part, that is, the impact part of bridge pier reinforcement was influenced to

the greatest extent. Therefore, the emphasis should be laid on the maximum reinforcement stress in this study.

(3) The bridge pier height difference generated an adverse effect on the reinforcement stress. With different pier height differences, the variation trends of joint displacements at pier top were relatively approximate. These trends were increasing to the maximum displacement toward the inverse direction of the impact direction and then continuously increasing toward the same direction as the impact direction until reaching the maximum value.

(4) With the increasing in the pier height difference in the impact direction, the maximum joint displacement at pier top enlarged, and its growth rate gradually elevated first and then lowered. The maximum joint displacement at pier top increased first and then reduced with the increase in the pier height difference in the nonimpact direction.

The impact born by the double-column high piers from rolling stones was simulated under varying pier height difference. The stress changes in bridge piers and reinforcement were observed extremely well through the simulation, which was an easy operation, thereby laying a foundation for further analyzing the dynamic response of bridge piers to the impact of rolling stones under complicated working conditions. However, the material of rolling stones used in this study was a rigid model. Although the calculation result had certain safety, the dynamic response calculation value of pier was larger than that of actual project. Further studying the selection of material model for rolling stones and the value selection of material parameters are necessary. Therefore, the working conditions of rolling stones will be enriched in the follow-up study to contribute to the deep understanding of the mechanical response of double-column high bridge piers to the impact of rolling stones in mountain areas.

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