Seismic response of CRTS II ballastless track-bridge system considering the damage of track structure

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Abstract
Based on the 16–32 m simply-supported beam bridge on Shanghai-Kunming high-speed railway, a collision model of CRTS II track system was established. The model considered the longitudinal, horizontal, as well as vertical nonlinear constraints among structural layers, and the Kelvin element was used to simulate the pounding effect of the gaps. The seismic response of the CRTS II track system was firstly analyzed and then the influences of different cases of damaged track plate, base plate, and mortar layer were discussed. The study revealed that: (1) The rail, track plate, and base plate all bear large longitudinal force and their stress envelope curves are all anti-symmetric. (2) The broken track plates result in a sharp decrease in track plate stress and an extreme increase in rail stress and base plate stress near the gap. The broken base plates result in a sharp decrease in base plate stress and an extreme increase in rail stress and track plate stress near the gap. (3) Both the pounding frequency and pounding force between broken slabs are relatively large and will decrease after some time. (4) The broken slabs near ends of the first bridge span greatly increase the pounding force of stoppers close to the abutment. (5) The gap width has a huge influence on the pounding force and times of stoppers and gaps. (6) The debonding of mortar layer has a great influence on the vertical displacement of rail, track plate, and the base plate.

Keywords
Bridge engineering, ballastless track, seismic response, damage cases of track structure, pounding effect

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Introduction

China Railways Track Structure (CRTS II) on bridge was developed by Max Bögl company,¹ and it has become a unique ballastless track in China. Compared with a longitudinal discontinuous track structure, CRTS II adopts the base plate instead of the rail as the main load-bearing component of the longitudinal force. And a sliding layer is set to minimize the interaction between the base plate and the bridge deck.² These characteristics enable CRTS II to lay continuously on the long-span bridge, as shown in Figure 1(a).

However, because CRTS II is continuous longitudinally, it is almost inevitable for the cracking of track plate and base plate and the debonding of mortar layer after a period of time,³⁻⁵ as shown in Figure 2.

Scholars over the world have carried out extensive research into the forces and deformation behavior of CRTS II track system under the influence of temperature and live loads. Wei et al.⁶ studied the mechanical properties and its effects on CRTS II under the action of temperature. Fang et al.⁷ analyzed the vertical vibration of CRTS II based on train-track element. Zeng et al.⁸ studied the effect of temperature changes on the relative displacement between mortar layer and track plate. Wang et al.⁹ studied the vertical displacement of the rail in the frequency domain and dynamic responses of CRTS II by transient dynamic analysis. Dai
et al.\textsuperscript{10} proposed an analytical algorithm to analyze the track-bridge interaction on long-span steel bridges under thermal action based on the bilinear resistance model. Zhao et al.\textsuperscript{11} studied the performance of the system under different damage cases of wide juncture in track plate. Chen et al.\textsuperscript{4} analyzed the forces and displacement behavior of rail, track plate, mortar, and pier under the influence of cracks in base plate. Wang et al.\textsuperscript{12} studied the influence of wide juncture in track plate and the debonding of mortar layer on the warping displacement and longitudinal force of track plate.

Earthquake will generally lead to a large superstructure displacement of the bridge, which may result in pounding, bearing unseating, or even span collapse.\textsuperscript{13} Because the CRTS II track system has various complicated components which have strong nonlinear characteristics, the nonlinear bridge-track interaction under earthquake action will be more complicated.\textsuperscript{14–17} Considering that CRTS II is only used in China and its application time is short, it is necessary to study the nonlinear collision characteristics of CRTS II track structure on the bridge under earthquake.\textsuperscript{18,19} The seismic response of CRTS II with broken plates and damaged mortar is still not clear and only few researches were conducted in that area.

Considering 16–32 m simply-supported bridges on Shanghai-Kunming high-speed railway for typical study, a spatial collision model of CRTS II track-bridge system was established. The bridge for the study is located in the valley and therefore the model considers the nonlinear constraints among the structural layers. Different cases of broken track plates, broken base plates and damaged mortar were set according to the mechanical features of track structures and the Kelvin element was used to simulate the collision effect at the gaps. The seismic response of the system with no broken slabs was analyzed. Besides, the forces and deformation behavior of key components under different cases of broken slabs were studied. The track system seismic response with the debonding of mortar layer was analyzed and the dynamic characteristics of the system with different gap widths were obtained.
Collision model for CRTS II ballastless track-simply-supported beam bridge system

From top to bottom, CRTS II track consists of rail, fastener, longitudinal consecutive track plate, CA mortar layer, longitudinal consecutive base plate, two fabrics, and one membrane structured sliding layer, stopper, anchor, and friction plate, etc., as shown in Figure 1(b). The rail end is fully constrained to simulate the seamless track, the track plate, and the base plate end are constrained, and the friction plate is set to simulate the contact characteristics between the base plate and the bridge deck.

In order to ensure the correctness of the modeling method in this study, the simulation of track/bridge interaction is compared with the calculation example in UIC Code 774-3, which confirms the accuracy of the calculation method. For the seismic response analysis of bridges, the existing mature calculation method is adopted, and the calculation results have good reference value.

The rail type and the fastener that were used in the model for the study are CHN60 and WJ-8 respectively. The longitudinal force-displacement relation is shown below.

\[
r_L = \begin{cases} 
12x & |x| \leq 2\text{mm} \\
24 & |x| > 2\text{mm}
\end{cases}
\]

In this equation, \( r_L \) is the longitudinal resistance of fasteners (unit: kN) and \( x \) is the relative displacement between rail and support rail bed (unit: mm).

The horizontal resistance of fasteners adopted experimental fitting results:

\[
r_H = \begin{cases} 
4.5y & |y| \leq 2\text{mm} \\
9 & |y| > 2\text{mm}
\end{cases}
\]

In this equation, \( r_H \) represents the horizontal resistance of fasteners (unit: kN) and \( y \) represents the relative displacement between rail and support rail bed (unit: mm).

The vertical stiffness of fastener depends on its stress state, vertical stiffness of bearing pad, and elastic strip. The vertical stiffness model is shown in Figure 3.

The dimension of the Track plate is 2.55 m (width) by 0.2 m (thickness) and its concrete grade is C55. The track plate is continuous on its longitudinal direction owing to the longitudinal steel connected by lock parts tensioning. The space between track plate and base plate was filled with CA mortar whose stiffness in longitudinal, horizontal and vertical direction was obtained from the literature.

The size of base plate was 2.95 m \( \times \) 0.19 m (width \( \times \) thickness) and the concrete strength was 30 MPa. It was also laid continuously in the longitudinal direction.

The sliding layer was used to separate the base plate from the beam element. The vertical compressive stress and horizontal friction were considered in the sliding layer, and the sliding friction coefficient was assumed to be 0.3. The uniform three-dimensional springs were used to simulate the sliding layer. The vertical were linear springs that only consider the compressive stress. The longitudinal and
horizontal were ideal elastic-plastic springs, and the elastic-plastic boundary point is 0.5 mm.

Through consolidation mechanism, the base plate was anchored on the deck in order to reduce its longitudinal displacement. The friction plates with a length of 50 m were arranged at both ends of the bridge, and the friction coefficient was 0.7. The Inverted T-shaped anchors were used to fixed the friction plates. In addition, the simulation of embankment section with a length of 250 m was taken as the boundary constraint of the model.

Stoppers were installed every 15 m within the bridge to restrain the vertical and horizontal deformation of base plate. Considering the tiny gap between post pouring stoppers and base plate (about 1 mm), stoppers and base plate were simulated by hook element with a maximum stiffness (1e8 kN/m).

This bridge adapts the 32 m double-line simply supported box beams in which the double line model was used to simulate the sliding support. The friction coefficient is 0.3 and the elastic-plastic deformation point 3 mm.

Considering the confinement of stirrups, the concrete was simulated by the Mander-Confined model and the moment-curvature characteristics of piers are shown in Figure 4. Thin-walled round hollow piers were used in this bridge. The highest pier in the V-shaped valley was 30 m and the concrete strength of the pier used was 40 MPa. Also, the nonlinear fiber beam elements were used to simulate the piers. The pier layout is shown in Figure 5(a).

The equivalent stiffness matrix with 6-DOF was used to simulate the interaction between pile and soil. Figure 5(b) shows the finite element model.

The Rayleigh damp was used for the track-bridge system, the damping ratio \( \eta \) was 0.05, and the damping coefficient \( \alpha \) and \( \beta \) could be calculated by formula (3):

\[
\alpha = 2\eta \frac{w_1 w_2}{w_1 + w_2}, \quad \beta = 2\eta \frac{1}{w_1 + w_2}
\]

In this equation, \( w_1 \) and \( w_2 \) are the former two-class nature frequency of the system.
The seismic fortification intensity was 8 and El-Centro wave along the bridge (longitudinal direction) was adopted. The spectral characteristic of seismic wave was reserved, and only the maximum peak acceleration was adjusted to 0.57g (rare occurrence earthquake).\(^2\) The duration of the seismic wave is the first 30 s including the ground motion peak.

The Kelvin element with linear spring and damper in parallel was used to simulate the seismic pounding effect, which can only transfer normal pressure without considering mutual penetration between contact surfaces. The relationship between the stiffness \( K \) and element displacement at \( i \) and \( j \) can be expressed as:

\[
K = \begin{cases} 
0 & u_i - u_j < \Delta G \\
\frac{k}{C} & u_i - u_j \geq \Delta G 
\end{cases}
\]  

\( \quad (4) \)

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**Figure 4.** Moment-curvature of the piers.

**Figure 5.** Finite element model: (a) diagram of the valley and (b) collision model of CRTS II track system.
The $\Delta G$ was the initial spacing, and the spacing width between the broken plates was considered. $k$ is the pounding stiffness, and the axial stiffness of the beam was considered. The energy loss during pounding was expressed by the damping $c$:  
\[
\zeta = 2\sqrt{\frac{km_1m_2}{m_1+m_2}}, \quad \zeta = \frac{-\ln r}{\sqrt{(\ln r)^2 + \pi^2}}
\]  
$\zeta$ was the damping ratio. For concrete, the restitution coefficient $r$ is 0.65. $m_1$ and $m_2$ are the beam mass at both ends of the pounding element respectively.

**Seismic response of CRTS II track system**

Stress and deformation behavior of the system under the El-Centro wave ($0.57g$) are shown in Figure 6. It can be seen from Figure 6 that the envelope curves of rail stress, track plate stress and base plate stress are anti-symmetric. In view of the nonlinear constraints between the bridges and the track, the continuous track structure (rail, track plates, and base plates) bears great longitudinal force under earthquake action, and its dynamic interaction is extremely complex. The stress envelope diagram generally presents an anti-symmetric double diamond shape, which is similar to previous research results.  

The maximum values of rail stress and track plate stress which occurred at pier 3 and pier 15 are 120.4 and 21.9 MPa. The peak values of base plate stress appear at the fixed supports of the bridge and its maximum value is 20.9 MPa which occurred at the anchor.

The maximum longitudinal deformation of fasteners occurred at the anchor (1.8 mm). During the earthquake, there is a great pounding force between base plates and stoppers because of the large vertical displacement in base plates. The maximum value is 31.7 kN which occurred at the stopper near the abutment.

**Seismic response of the system with broken track plates**

**Broken track plates in different locations**

The dynamic response of different cases under the action of El-Centro wave are shown in Figure 7. Case 1 represents the track plates near abutment; case 2 represents the movable support at the first bridge span and case 3 represents the anchor. Case 4 represents no crack case as a comparison.

As shown in Figure 7, the broken track plates near the abutment and movable support at the first bridge span have a great influence on local rail stress, track plate stress, and base plate stress. In comparison with case 4, the maximum local rail stresses are 141.6 MPa (case 1) and 153.9 MPa (case 2), which increase by 33.3% and 37.3%. The maximum local base plate stresses are 26.6 MPa (case 1) and 25.4 MPa (case 2) with an increased rate of 61.2% and 44.3%. The local tensile
Figure 6. Stress and deformation features of the system: (a) longitudinal force of track structures, (b) longitudinal deformation of fasteners, and (c) vertical pounding force of stoppers.
Figure 7. Seismic response of CRTS II track system with broken track plates in different locations: (a) rail stress envelope, (b) track plate stress envelope, (c) base plate stress envelope, (d) time history of pounding force between broken track plates, (e) stoppers vertical pounding force, (f) time history of stopper pounding force near pier 2, (g) longitudinal deformation of fasteners, and (h) longitudinal deformation of mortar layer.
stresses in track plates decrease rapidly with a maximum value less than 3 MPa. The longitudinal force of the base plate is transmitted to the embankment by the anchor. Consequently, the broken track plate near anchor has little influence on local rail stress and track plate stress while there is significant increase in the tensile stress of base plate near the gap with a maximum value up to 29.4 MPa.

Under the case of broken track plate near movable support at the first bridge span, there is a frequent pounding phenomenon near the gap and the maximum pounding force nearby is 4816.3 kN. In the other two cases, the pounding frequency as well as the force are small. The pounding time and pounding force will decrease to a great extent with time.

The stopper is a key component in keeping the vertical stability of base plate. There is a great pounding effect between base plate and stopper under earthquake action. The broken track plate near anchor has little influence on the vertical pounding force and frequency of stoppers because it is far away from stoppers and the longitudinal force of base plate is transmitted to the subgrade. However, the stoppers bear large pounding force in case 1 and case 2 with a maximum value of 67.6 and 88.3 kN.

Because fasteners and mortar layer are directly connected with track plate, broken track plate has a great influence on their longitudinal deformation. Under the case of no broken slabs, their maximum values of longitudinal deformation occurred near the anchor. The values of the deformations are 2 mm (fasteners) and 3 mm (mortar layer). However, the maximum values of deformation was determined at the gaps under the cases of broken slabs. The maximum values of fasteners deformation are 7.4 mm (case 1), 9.4 mm (case 2), and 6.8 mm (case 3) and for the mortar layer, the deformations are 7.1 mm (case 1), 7.4 mm (case 2), and 8.2 mm (case 3).

Because of the strong energy absorption of the sliding layer and the large longitudinal stiffness of anchor and pier, the influence of local broken track plates on the force and deformation on anchor, pier, and sliding layer is very small.

Broken track plate with different gap widths

The influence of different gap widths (1, 2, 5, and 10 mm) was analyzed by considering the broken track plate near the movable support at the first bridge span. The results of the study are shown in Figure 8.

From Figure 8 it can be seen that the gap width on the track plate has little influence on the tensile stresses of rail, track plate, and base plate. However, the compressive stresses of rail and base plate increase slightly as the gap width increases. The compressive stresses of rail near the gap are 89.2 MPa (1 mm) and 106.5 MPa (10 mm). The compressive stresses of base plate near the gap are 14.8 MPa (1 mm) and 17.0 MPa (10 mm).

The result also shows that the smaller the gap width, the higher the pounding frequency and the larger the pounding force. The maximum pounding forces are
Figure 8. Dynamic response of the system with different gap widths in broken track plate: (a) rail stress envelope, (b) track plate stress envelope, (c) base plate stress envelope, (d) time history of pounding force between broken track plates, (e) vertical pounding force of stoppers, and (f) time history of stopper pounding force near pier 2.
5739.8 kN (1 mm), 4816.3 kN (2 mm), and 2707.6 kN (5 mm). However, there is nearly no collision effect when the gap width is 10 mm.

The different gap widths in track plate have a great influence on the vertical collision force and frequency of stoppers. The wider the gap width, the larger the vertical displacement of the base plate nearby. Consequently, the vertical pounding force of local stoppers is large. The stopper near the abutment bears larger pounding force in which the maximum values are 132.0 kN (10 mm), 113.0 kN (5 mm), 88.3 kN (2 mm), and 79.0 kN (1 mm).

There is no significant difference among the force and deformation curves of anchor, pier bottom, fasteners, mortar layer, and sliding layer under different gap widths. It is obvious that the gap width has little influence on them.

Seismic response of the system with broken base plates

Broken base plates in different locations

Assuming that the base plates near abutment (case 1), movable support at the first bridge span (case 2), and anchor (case 3) are damaged under the action of El-Centro wave (0.57g), the dynamic response of the three cases are shown in Figure 9. In which case 4 represents no crack case as a comparison.

As shown in Figure 9, the stress curves of rail and track plate are smooth when there is no broken slab. On the other hand, peak values appear near the gaps under the broken cases. The peak values of rail stress are 144.2 MPa (case 1), 191.9 MPa (case 2), and 97.6 MPa (case 3), increasing by 35%–68% compared with case 4. The peak values of track plate stress are 33.4 MPa (case 1), 49.0 MPa (case 2), and 28.0 MPa (case 3), increasing by 76%–145% compared with case 4. Under the three cases, the stresses of base plates near the gap decrease sharply with a maximum value less than 4 MPa.

Under the cases of broken base plates near abutment and anchor, the maximum longitudinal force of anchors is about 16,350 kN. The strong energy absorption of the sliding layer reduces the influence of case 2 on anchor. The maximum longitudinal force of anchor is 14,934 kN.

The displacement of the broken base plate near anchor is small owing to the constraints of the anchor. Consequently, the pounding frequency was high and the pounding force increase significantly near the gap compared with the other two cases. The maximum pounding force is 6822.5 kN in this case while it’s 4500 kN for the other two cases. It is worthy to note that the pounding times and pounding force will decrease to a great extent after a period of time.

During the earthquake, there is a frequent collision phenomenon between stopper and base plate which are directly connected with each other. Compared with no crack case, it is obvious that the broken base plate near the movable support at the first bridge span has a great influence on the vertical pounding force of the stopper near abutment. The value is 80.2 kN, which increased by 153.0%. In the other two cases, the influence is small because the broken base plates are away from the stoppers.
Figure 9. Seismic response of the system with broken base plates in different locations: (a) rail stress envelope, (b) track plate stress envelope, (c) base plate stress envelope, (d) longitudinal force of anchors, (e) time history of pounding force between broken base plates, (f) vertical pounding force of stoppers, (g) time history of stopper pounding force near pier 2, (h) longitudinal deformation of fasteners, and (i) longitudinal deformation of mortar layer.
Under the three cases, the longitudinal deformation of fasteners near the gap increases slightly with a maximum value of less than 2 mm. The longitudinal deformation of mortar layer increases tremendously near the gap. The maximum values are 6.9 mm (case 1), 7.8 mm (case 2), and 10.0 mm (case 3).

Local broken base plates have little influence on the deformation of the sliding layer and the force at the bottom of the pier.

**Broken base plate with different gap widths**

The broken base plate near the movable support at the first bridge span was used to evaluate the influence of different gap widths (1, 2, 5, and 10 mm).

The results obtained from the analysis as shown in Figure 10 show that the base plate with different gap widths has little influence on the tensile stresses of local rail, track plate, and base plate. The local compressive stresses of rail, track plate and longitudinal deformation of fasteners increase slightly with an increase in gap width. The maximum compressive stresses of rail are 91.4 MPa for 1 mm gap width and 138.0 MPa for 10 mm gap width. For the track plate, the maximum compressive stresses are 19.6 MPa for 1 mm gap width and 35.7 MPa for 10 mm gap width. The maximum longitudinal deformations of fasteners are 1.7 mm (1 mm) and 2.2 mm (10 mm).

The collision frequency and force between broken slabs increase with decrease in gap width. The maximum pounding force are 5576.7 kN (1 mm), 4585.2 kN (2 mm), and 1951.5 kN (5 mm). There is almost no collision phenomenon when the width is 10 mm.

Different gap widths in base plate have great influence on the vertical collision force and frequency of stoppers. The wider the gap width, the larger the vertical displacement of the base plate nearby. Consequently, the vertical pounding force of local stoppers become larger. The stopper near the abutment bears larger pounding force. The maximum values are 98.7 MPa (10 mm), 94.1 MPa (5 mm), 80.2 MPa (2 mm), and 73.6 MPa (1 mm).

The base plate with different gap widths has little influence on the force and deformation behavior of the pier, anchor, sliding layer etc.

**The force and deformation behavior of the system with the debonding of mortar layer**

**Broken mortar layer in different locations**

According to the field observation, the debonding phenomenon of mortar layer is likely to occur at the beam-end and beam crevice. Therefore, the debonding of mortar layer in this work is assumed to happen at the following three locations; the left beam-end (case 1), the beam crevice between the second and third span (case 2), and the right beam-end (case 3). Taking the case of no damaged mortar layer (case 4) as a comparison, the force and deformation behavior of key components with broken mortar layer in different locations was analyzed. Figure 11 below, shows
Figure 10. Dynamic response of the system with different gap widths in broken base plate. (a) rail stress envelope, (b) track plate stress envelope, (c) base plate stress envelope, (d) time history of pounding force between broken base plates, (e) vertical pounding force of stoppers, (f) time history of stopper pounding force near pier 2, and (g) longitudinal deformation of fasteners.
that the influence of debonding of mortar layer on the longitudinal force of rail, track plate, and base plate is small.

It can be seen from Figure 11 that when the debonding of mortar layer appears at the left beam-end, the vertical displacement of rail nearby increases compared with the case of no damaged mortar layer. The maximum value increased is 3.3 mm at a rate of 14.7%. When the debonding of mortar layer happen at beam crevice between the second and third span, the upward displacement of rail increases to a certain extent. The maximum value increase is 1.4 mm with an increase rate of

**Figure 11.** Partial enlarged drawing of the vertical displacement of rail: (a) the vertical displacement of rail at the left beam-end, (b) the upward displacement of rail at the beam crevice between the second and third span, and (c) the downward displacement of rail at the beam crevice between the second and third span.
28.6%. The downward displacement decreases by 2.8 mm at rate of 9.8%. When the damaged mortar layer appears at the right beam-end, the vertical displacement of the rail undergoes a minor change.

Under the first three cases, the vertical displacement envelope of rail, track plate, and base plate is almost the same. Their change regularity and variation degree have little different from each other.

**The coupling effect of broken track plate and mortar layer**

It is a common case that the wide juncture at beam-end is damaged with the debonding of mortar layer due to the repeated warping deformation under temperature loads. Broken track plate together with the debonding of the mortar layer nearby was assumed to occur at the abutment (case 1), the movable support at the first bridge span (case 2) and the anchor (case 3). The force and deformation behavior of track system are shown in Figure 12.

As shown in Figure 12, the rail stress increases under the coupling effect of broken track plate and mortar layer near the abutment compared with the damage case of only track plate. The maximum value increase by 14.8 MPa with an increase rate of 10.9%. The maximum vertical displacement of rail decreases from 74.3 to 67.7 mm, which decreased by 6.6 mm. Under the coupling effect of broken track plate and mortar layer near the movable support at the first bridge span, the tensile stress of rail nearby decreased by 13.8 MPa and the compressive stress increased by 6 MPa. Also, the upward displacement of rail increased by 7 mm and the downward displacement increased by 3.7 mm.

The force on the track plate further decreases (close to 0) and that of base plate increased slightly under the first two cases. The force and deformation of the rail, track plate, and the base plate structure when both the track slab and the mortar layer near anchor are damaged at the same time are not different from that under the damage case of only track plate near anchor.

The vertical displacement envelope of track plate and base plate is similar to that of rail with the change of vertical displacement at relevant position slightly different. Compared with the damage case when only track plate is considered, the vertical displacement of track plate and base plate nearby decreases from 71.5 to 64.9 mm, which reduced by 6.6 mm under the coupling effect of damaged track plate and mortar layer near the abutment. When broken track plate and mortar layer appear near the movable support at the first bridge span, the vertical displacement of track plate and base plate witnesses a maximum value at the movable support. The values are 97.3 and 96.2 mm with an increase of 2.4 and 2 mm. The maximum amplification nearby is 5.7 and 4 mm.

**Conclusions**

The 16–32 m simply-supported beam bridge on Shanghai-Kunming high-speed railway was considered for this study in which Kelvin element was used to simulate
Figure 12. The force and deformation behavior of rail: (a) the envelope of rail stress (case 1), (b) the envelope of rail stress (case 2), (c) the vertical displacement of rail (case 1), and (d) the vertical displacement of rail (case 2).
the pounding effect. A collision model of CRTS II track system which considered the nonlinear constraints between the track layers was established. The seismic response of the system was analyzed and the forces and deformation behavior of key components under different cases of broken slabs were studied. The dynamic response of the system with different gap widths was obtained. The seismic response with the debonding of mortar layer of the system was analyzed. Findings from the results shows that:

1. During the earthquake, there is a large longitudinal force in the rail, track plate, and base plate and their stress envelope curves are all anti-symmetric.
2. The local base plate stresses increase by 28.4%–71% with broken track plates. The broken track plate near anchor has little influence on rail and track plate stress. However, the local rail stresses increased by 33.3%–37.3% and the track plate stresses decreased abruptly for the two other cases. Under the case of broken base plates, the local base plate stresses decrease rapidly, the local rail stresses increased by 35%–68% and track plate stresses by 76%–145%.
3. Under the condition of broken track plates, there is a frequent pounding effect. In broken track plate near the movable support at the first bridge span and broken base plate near the anchor, the pounding frequency is high, and the corresponding maximum pounding forces are 4816.3 and 6822.5 kN. Moreover, both the collision times and force will decrease to a great extent with time.
4. Stopper is the key component in keeping the vertical stability of base plate. There is great pounding effect between base plate and stopper under earthquake action. The broken slabs near both ends of the first bridge span will contribute to great pounding force in stoppers near the abutment.
5. When the gap width increase, there is a slight increase in the compressive stress of track structures. The gap width has huge influence on the pounding force and times of stoppers as well as gaps. The wider the gap, the larger the pounding force of stoppers nearby with a maximum value near the abutment. Besides, the pounding frequency and force of gaps increase with the decrease in gap width.
6. Under the coupling effect of broken track plate and mortar layer near the abutment or movable support at the first bridge span, the longitudinal force of the rail as well as the vertical displacement of the rail, track plate, and base plate changed significantly.

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