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Numerical Modelling on Seismic Responses of High-Speed Railway Track-Bridge Systems Using Combined Curved Steel Dampers with Novel Porous Energy-Absorbing Materials

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ABSTRACT: High-speed railway bridges are critical infrastructure in seismically active regions; however, their complex track-bridge interaction poses significant challenges to conventional seismic protection systems. To mitigate seismic damage risks and enhance post-earthquake recoverability of high-speed railway bridges, this paper proposes a combined curved steel damper (CCSD) that incorporates a novel porous energy-absorbing material, which achieves stable energy dissipation through bending deformation. The hysteretic energy dissipation characteristics of the damper were investigated through refined finite element simulations. The results indicate that the damper exhibits stable energy dissipation efficiency under cyclic loading, with plump, continuous hysteresis loops and no significant pinching effect. Furthermore, a numerical model of the high-speed railway track-bridge system (HSRTBS) integrated with the damper was developed to quantify its control effect on the seismic response of the system. The results demonstrate that the damper significantly reduces the seismic response of most components: the average reduction in the peak response of bearings exceeds 40%, the reductions for components such as the girder and rail exceed 20%, and the reduction for fasteners also surpasses 10%. Although the peak response of the piers increases slightly, the damage remains at a slight level, which is within the acceptable engineering range. In summary, the damper proposed in this study can effectively control the seismic response of high-speed railway track-bridge systems, providing a reference for the seismic design and resilience enhancement of high-speed railway bridges in high-intensity seismic zones.

KEYWORDS: Porous energy-absorbing material; combined curved steel damper; high-speed railway track-bridge systems; numerical model; seismic response control

1 Introduction

China is located in an earthquake-prone region, where earthquakes, as a highly destructive natural disaster, occur periodically and pose a significant threat to lifeline engineering such as urban and railway networks [1,2]. When strong earthquakes strike suddenly, they often cause severe damage to infrastructure including buildings, high-speed railways, and bridges, endangering structural safety and resulting in major casualties and economic losses [3–5]. Therefore, enhancing the seismic resilience of engineering structures has become a central issue in the field of architectural and bridge seismic engineering.

Traditional isolation bearings (such as elastomeric bearings and friction pendulum bearings) often face a trade-off between displacement control and durability. Although elastomeric bearings provide high vertical stiffness and can accommodate girder rotations, their materials are prone to aging, which may lead to stiffness degradation and insufficient energy dissipation over long-term use [6,7]. Friction pendulum bearings exhibit better durability and can withstand large vertical loads; however, under near-fault ground motions, they may increase girder displacement by 30%–50%, thereby aggravating track geometric errors [8]. In contrast, installing dampers can more effectively absorb seismic energy and suppress structural response, making them a preferred strategy for enhancing the seismic performance of engineering structures [9,10].

Metallic dampers, with their clear force-transfer paths, stable hysteretic performance, and strong environmental adaptability, are widely used in earthquake engineering to suppress structural seismic responses [11,12]. Currently, substantial research outcomes have been accumulated for this type of damper. Xie et al. [13] developed a simplified analytical framework to simulate the hysteretic response of bidirectional U-shaped steel dampers, with its effectiveness and applicability being confirmed. Zhai et al. [14] put forward an innovative S-shaped steel plate damper; monotonic and cyclic loading tests demonstrated its stable hysteresis, favorable energy dissipation, and large-deformation ductility. Parametric analysis indicated that the height-to-thickness ratio has a critical influence on stiffness and ductility, leading to the development of a practical design formula. Along another research direction, Nouri et al. [15] proposed an arc-and-ring yielding damper featuring a central ring with four surrounding arcs. This damper demonstrated pinch-free stable hysteresis and high energy dissipation. By combining multivariate nonlinear fitting with an artificial neural network, the researchers derived closed-form relationships between geometric parameters and key performance indicators (yield force, maximum force, yield displacement, and dissipated energy), enabling straightforward sizing in practice. Marioni [16] conducted systematic work on circular steel dampers with varying heights, showing that such dampers exhibit high yield ratios and stable damping characteristics. Building on this, Wu and Tong [17] further optimized the cross-section height, replaced mild steel with dual-grade high-strength steel, designed an arc-shaped steel damper, and established the corresponding mechanical model. More recently, Nouri et al. [18] developed a multiple bean damper featuring cellular bean-shaped fuses encircling an inner shaft, where the flexural and axial deformations of the beans serve as the primary energy-dissipation mechanism; full-scale cyclic tests and parametric finite element studies revealed that bean thickness and height are the dominant parameters, and analytical formulas for yield force and elastic stiffness were derived, supporting displacement-based seismic design.

Porous metal is a novel material that has emerged in recent years, containing numerous internal pores, which grants it unique properties while retaining fundamental metallic characteristics [19,20]. This type of material not only exhibits excellent specific strength, impact resistance, and energy absorption per unit volume, but also demonstrates favorable breathability, sound absorption, and thermal insulation properties [21]. In terms of mechanical performance, Zhao et al. [22] investigated the mechanical behavior of PM-35 porous metal under monotonic and cyclic loading and applied it as a core material in buckling-restrained braces. The results indicate that PM-35 shows significant differences in tensile and compressive performance, with notable plastic deformation capability; its yield strength is relatively low, approximately half that of Q195 steel, while its plastic deformation amount is large, reaching five times that of Q195 steel. Xue et al. [23] conducted quasi-static compression and dynamic impact tests on porous iron with porosity levels ranging from 10% to 30%, finding that it exhibits bilinear mechanical responses under impact loading, with yield strength, load-bearing capacity, and energy absorption performance varying with strain rate. The yield strength ranges from 30 to 400 MPa. Similarly, Iqbal et al. [24] examined the mechanical and energy-absorbing properties of porous bronze under impact loading. The experiments showed that its force-displacement relationship conforms to a bilinear model, with energy absorption being insensitive to strain rate, and compressive strength ranging approximately from 60 to 160 MPa.

Based on the analysis above, this research integrates the advantages of porous metal energy-absorbing materials to propose a combined curved steel damper that incorporates this novel material. First, a numerical analysis model of the damper is established to comprehensively analyze its hysteretic performance. Furthermore, the damper is integrated into a HSRTBS, and through nonlinear time-history response analysis, the influence of the CCSD on the seismic performance of each key component in the HSRTBS is systematically investigated.

2 The CCSD with Novel Porous Energy-Absorbing Material

2.1 Novel Porous Energy-Absorbing Material

PM-35 porous metal filler is a type of porous metallic material fabricated by Sintokogin, Ltd. It is produced by compacting and high-temperature sintering fine spherical stainless steel powder [22]. The stress-strain curve obtained from quasi-static tests of the material is shown in Fig. 1. According to the characteristics of the curve, PM-35 porous metal exhibits no distinct yield plateau. Given that the stress-strain curve of PM-35 steel does not exhibit a distinct yield plateau, the stress corresponding to 0.2% plastic strain was taken as the yield strength. It should be noted that the elastic modulus was calculated from the slope of the elastic portion of the monotonic curve (in the range of 1/3 to 2/3 of the compressive yield stress), and mechanical parameters such as yield strength are listed in Table 1. Further analysis of the material properties indicates that its mechanical indicators, including compressive ultimate load and yield strength, show low sensitivity on loading rate and porosity [22].

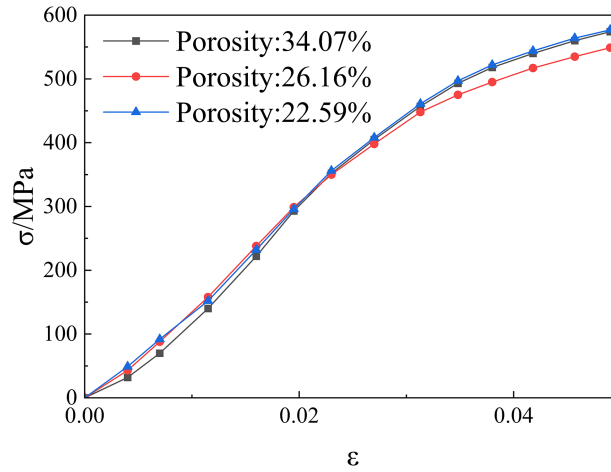


Figure 1: Stress-strain curve of PM-35 porous metal.

Table 1: Mechanical properties of PM-35 porous metal.

Pore Size (μm)	Weight (g)	Volume (mm ³)	Density (g/cm ³)	Elastic Modulus (MPa)	Yield Strength (MPa)
7	116.4	20,347.2	5.72	16,281	32.5
25	116.1	20,347.2	5.70	15,950	31.9
35	117.1	20,347.2	5.75	15,580	31.1

2.2 Configuration of the CCSD

Metal dampers are widely used in structural vibration control due to the stable hysteretic energy dissipation properties of metallic materials. As shown in Fig. 2, the novel combined curved steel damper adopted in this paper mainly consists of an upper connecting plate, a lower connecting plate, and intermediate metal plates. The intermediate metal plate is made of PM-35 porous metal and comprises U-shaped plates symmetrically arranged at both ends and X-shaped plates arranged in parallel. Under lateral cyclic loading, the end plates can yield synchronously. The U-shaped metal plates on both sides serve multiple functions: on the one hand, they act as the primary energy-dissipating elements, absorbing seismic energy through plastic deformation to provide vibration control; on the other hand, they also serve as connection nodes, providing necessary structural constraints.

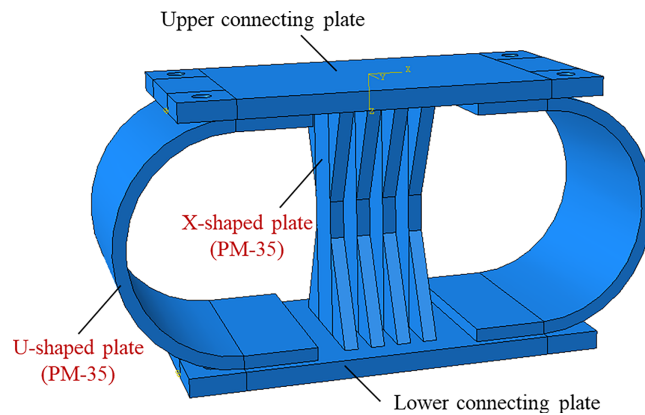


Figure 2: Configuration of the CCSD.

The main structural parameters of the CCSD are as follows: the arc radius of the U-shaped plate is 116 mm, the plate width is 200 mm, the total length is 210 mm, and the plate thickness is 14 mm; the outer end steel plate measures 460 mm × 220 mm × 20 mm; the longest side length of the X-shaped plate is 160 mm, the middle short side length is 40 mm, and the plate width is 260 mm. The components are primarily connected by welding.

2.3 Numerical Modeling and Mechanical Performance of the CCSD

To further investigate the mechanical performance of the CCSD, a finite element model was developed in ABAQUS [25–27]. During modeling, solid parts of the upper connecting plate, lower connecting plate, and intermediate metal plate were first created based on actual geometric dimensions, and then assembled via the “Assembly” module to form a complete three-dimensional model. A bilinear kinematic hardening model was adopted for the material constitutive behavior: the U-shaped and X-shaped metal plates used the aforementioned PM-35 porous metal material with a poisson’s ratio of 0.23; the outer steel plates were made of Q345 steel with an elastic modulus of 210 GPa, a poisson’s ratio of 0.3, and a yield strength of 345 MPa.

Mesh quality directly affects computational accuracy and convergence [28,29]. An adaptive refinement strategy was employed for the main energy-dissipating components (U-shaped and X-shaped plates) based on preliminary mesh analysis. The C3D8R element type was adopted, with a base mesh size of 10 mm. The mesh was locally refined to 5 mm in critical regions, including the curved bending segment of the U-shaped plate, the central intersection area of the X-shaped plate, and the four corner root connections to the flange. The connecting plates were discretized using a structured mesh with a size of 10 mm. The resulting mesh is

shown in Fig. 3. Specifically, the discrete model illustrated in Fig. 3 comprises a total of 34,060 nodes and 20,552 elements. The mesh quantities for each component are presented in Table 2.

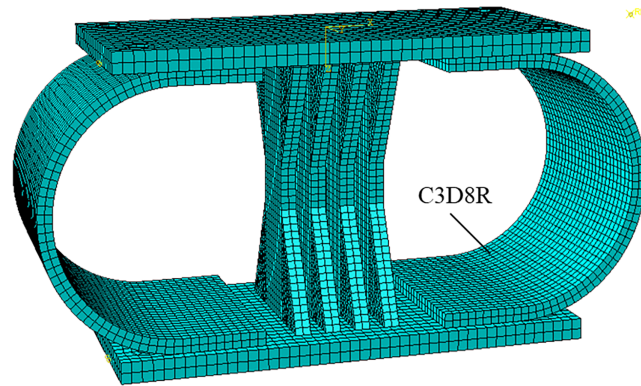


Figure 3: Meshing of the CCSD.

Table 2: Mesh quantities for each component.

	The Upper Plate	U-Shaped Plates	X-Shaped Plates	The Lower Plate
Nodes	1277	8550	22,956	1277
Elements	640	5184	14,088	640

In the contact settings, the “Tie” constraint was used to simulate welded connections, aiming to achieve a smoother load-displacement response curve [30]. The remaining contact surfaces were defined with “General contact” [10], where the contact properties include both tangential and normal components: the tangential behavior employed penalty-based contact with a friction coefficient of 0.15, and the normal behavior was set as hard contact to prevent interpenetration between components. Furthermore, a reference point RP-1 was defined on the top surface of the upper connecting plate, and a coupling constraint was applied to couple all degrees of freedom in this region to RP-1, as shown in Fig. 4.

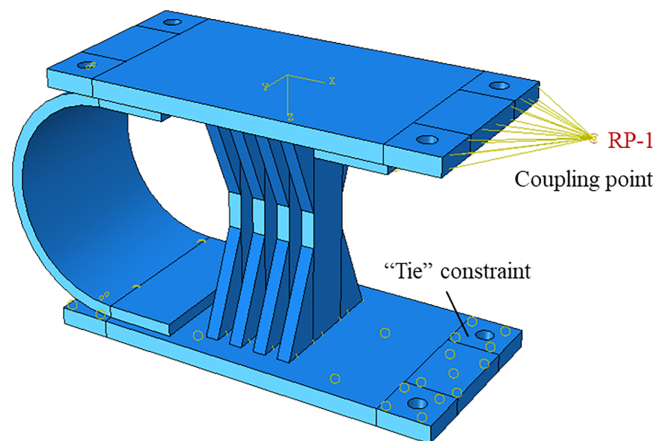


Figure 4: Contact settings of the CCSD.

In the finite element analysis, a displacement-controlled loading method was adopted. All degrees of freedom of the lower connecting plate of the damper were constrained, while only the translational degree of freedom (DOF) in the horizontal direction was retained for the upper connecting plate, with constraints applied in all other directions. The horizontal displacement load was applied through the reference point RP-1, and the specific loading protocol and boundary condition are illustrated in Figs. 5 and 6.

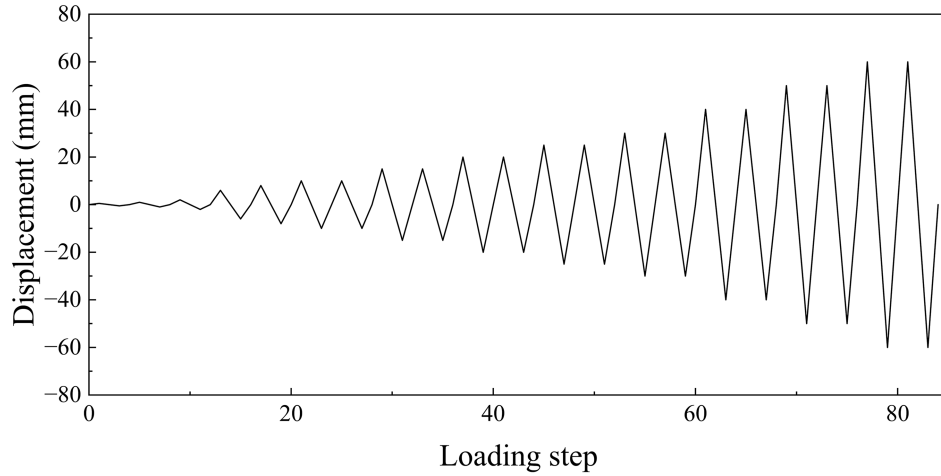


Figure 5: Loading protocol.

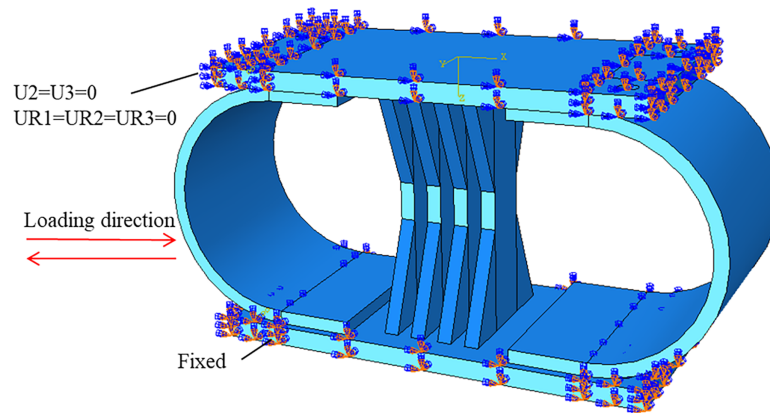


Figure 6: Boundary condition.

The stress contour of the damper under cyclic loading is shown in Fig. 7. It can be observed that the plastic deformation and energy dissipation of the CCSD are primarily concentrated in the X-shaped plate and the U-shaped metal plates. During the initial loading stage, the arc region of the U-shaped plates yields first. As both components are made of the same material, the X-shaped plate rapidly participates in energy dissipation afterward, without exhibiting noticeable staged yielding behavior. In the mid-loading stage, plastic deformation of the U-shaped plates remains concentrated in the arc regions, while the X-shaped plate develops plastic deformation mainly on its upper and lower sides; its central portion remains elastic due to the controlled minimum width in the design. As shown in Fig. 8, the hysteresis curve exhibits a plump, spindle-shaped profile, continuous and complete without pinching or slip phenomena, indicating that the damper maintains consistently absorbs and dissipates energy in a stable manner even under large

deformations, thus meeting the fundamental criteria for energy dissipation and seismic control. Moreover, the damper exhibits a high initial stiffness, and although a slight reduction in stiffness occurs in later cycles—reflected by a marginally smaller slope compared to the first cycle—the loop morphology remains steady with no notable pinching. The forward and reverse loading-unloading paths nearly overlap, indicating a highly symmetrical mechanical response. This symmetry reflects consistent performance and stable behavior of the damper under both tensile and compressive loading conditions. In summary, the damper exhibits desirable stiffness and energy dissipation characteristics, enabling it to effectively achieve the intended vibration reduction objectives.

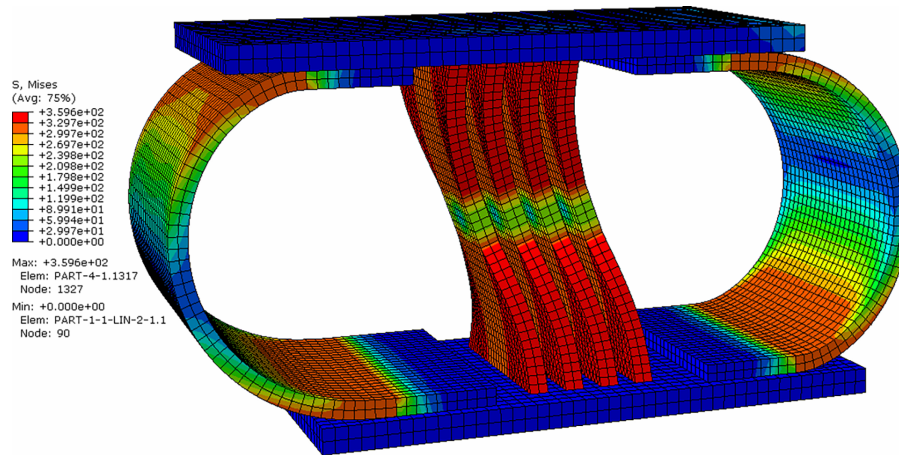


Figure 7: Results of finite element simulation.

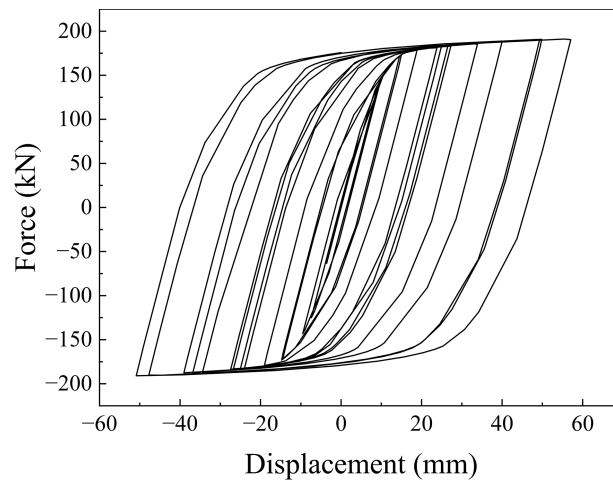


Figure 8: Hysteresis curve of the CCSD.

3 High-Speed Railway Track-Bridge System with CCSDs

3.1 Numerical Model of HSRTBS

The track-bridge system of high-speed railways constitutes a complex coupled assembly that integrates the main bridge structure with the ballastless track, which exhibits significant differences from conventional highway bridges [31]. The interaction between the superstructure track and the bridge increases the

complexity of vibration transmission. Damage to a single component in the system may trigger cascading risks, thereby endangering the overall structural integrity and operational safety. Hence, in assessing the seismic resilience of such bridges, the role and influence of the track structure must be fully considered [32].

This research focuses on a five-span, equal-height, simply-supported high-speed railway girder bridge fitted with the Chinese railway track system (CRTS) II slab ballastless track, analyzing the influence of installed CCSD dampers on the seismic response of the system. The model overview is shown in Fig. 9. The superstructure consists of a 32.6 m standard-span prestressed concrete box girder, and the substructure comprises solid circular-ended piers with a height of 14 m, constructed with C35 concrete. Longitudinal main reinforcement uses HRB400 steel. Pot rubber bearings are employed, supported by bearing seats made of C50 concrete, with a single seat length of 2.5 m. Four types of bearings are arranged at the top of the piers: bi-directional moveable bearings, laterally moveable bearings, longitudinally moveable bearings, and fixed bearings. The specific layout is shown in Fig. 9. The friction plate is constructed with C30 concrete, with a total length of 50 m and cross-section dimensions of $9.0 \text{ m} \times 0.4 \text{ m}$. A hydraulic bearing layer (HBL) is provided above the subgrade to support the superstructure track system. This layer has a trapezoidal cross-section measuring 2.95 m at the top, 3.25 m at the bottom, and 0.3 m in height.

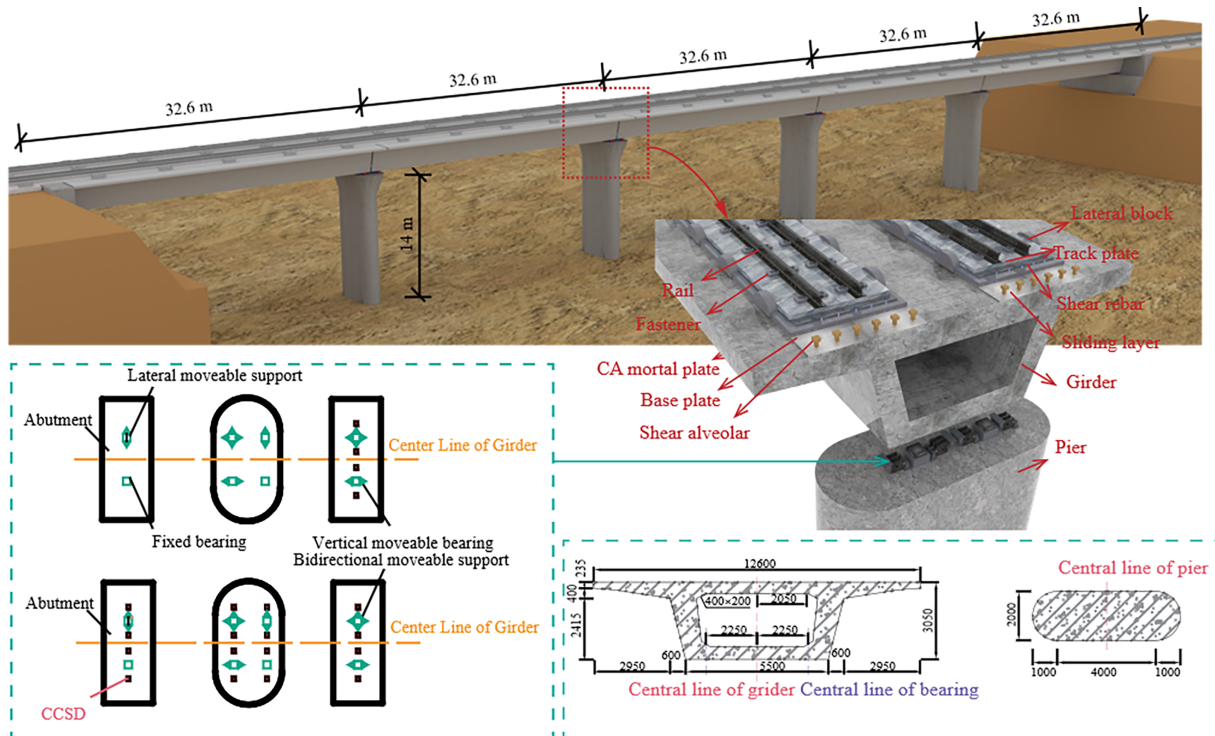


Figure 9: General representation of the high-speed railway track-bridge system.

The CRTS II slab ballastless track system comprises fasteners, rails, track plates, cement asphalt (CA) mortar layer, sliding layer, base plate, shear alveolars, shear rebars, lateral blocks, and end anchors. Fig. 10 presents a schematic of the partial cross-section of this track structure. CHN60 rails are employed, paired with WJ-8 fasteners. Both the track plate and the base plate measure 2.55 m in width, with respective thicknesses of 0.20 and 0.19 m. Between them lies a 30 mm-thick CA mortar layer, while a 60 mm-thick sliding layer is placed beneath the base plate. Above the fixed bearings, shear alveolars are positioned to transfer longitudinal forces between the girder and the track. At the girder ends and within the end anchor

zones, shear rebars are anchored into both the track plate and the base plate. Lateral blocks are placed along the longitudinal direction of the girder with alternating spacings of 5.74 and 8.0 m. An inverted T-shaped end anchor is provided at the starting point of the transition zone between the bridge and the subgrade; its top is connected to the friction plate, and its bottom is rigidly connected to the base plate.

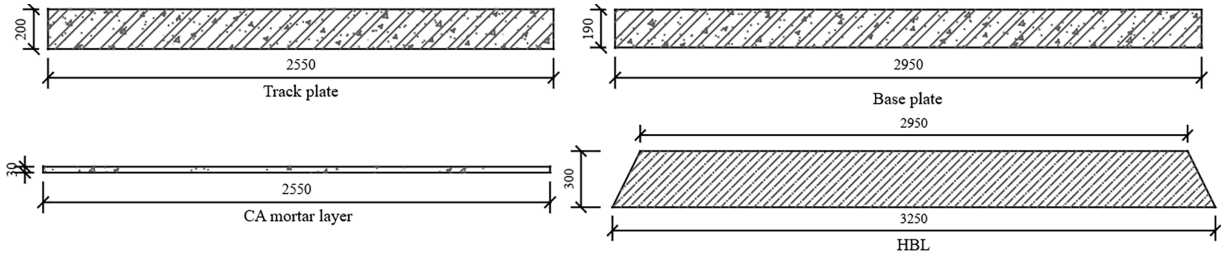


Figure 10: Schematic of the track structure cross-section.

The basic principle of track-bridge system interaction can be elaborated through the differential equations governing track-bridge interaction. As shown in Fig. 11, taking a micro-segment of length dx for analysis, and based on the force equilibrium conditions and deformation compatibility conditions of the rail, base slab, track slab, and bridge, the following equation is obtained:

$$dP_r + qdx - P_{rs}(z_{rs})dx = 0 \quad (1)$$

$$dP_s + p_{rs}(z_{rs})dx - p_{sd}(z_{sd})dx = 0 \quad (2)$$

$$dP_d + p_{sd}(z_{sd})dx - p_{db}(z_{db})dx = 0 \quad (3)$$

$$dP_b + p_{db}(z_{db})dx = 0 \quad (4)$$

$$\frac{du_r}{dx} = \frac{P_r}{E_r A_r} \quad (5)$$

$$\frac{du_s}{dx} = \frac{P_s}{E_s A_s} \quad (6)$$

$$\frac{du_d}{dx} = \frac{P_d}{E_d A_d} \quad (7)$$

$$\frac{du_b}{dx} = \frac{P_b}{E_b A_b} \quad (8)$$

where: P_r , P_s , P_d and P_b are the forces at the left ends of the rail, track plate, base plate, and bridge, respectively. The external force applied to the rail is represented by q . Meanwhile, p_{rs} , p_{sd} and p_{db} stand for the longitudinal resistances between the rail and track plate, between the track plate and base plate, and between the base plate and bridge. u_b , u_d , u_s and u_r are the longitudinal displacements of the bridge, base plate, track plate, and rail, respectively. Similarly, z_{rs} , z_{sd} and z_{db} represent the relative displacements between the rail and track plate, between the track plate and base plate, and between the base plate and bridge, respectively. The elastic moduli of the rail, track plate, base plate, and bridge are denoted by E_r , E_s , E_d and E_b , respectively, while A_r , A_s , A_d and A_b are their corresponding cross-sectional areas.

Solving the above equations simultaneously yields:

$$\frac{d^2 z_{rs}}{dx^2} = \frac{1}{E_r F_r} [p_{rs}(z_{rs}) - q] - \frac{d^2 u_s}{dx^2} \quad (9)$$

$$\frac{d^2 z_{sd}}{dx^2} = \frac{1}{E_r F_r} [p_{sd}(z_{sd}) - p_{rs}(z_{rs})] - \frac{d^2 u_d}{dx^2} \quad (10)$$

$$\frac{d^2 z_{db}}{dx^2} = \frac{1}{E_r F_r} [p_{db}(z_{db}) - p_{sd}(z_{sd})] - \frac{d^2 u_b}{dx^2} \quad (11)$$

Considering the geometric constraint condition, in the fixed zone, the sum of the displacements of the rail, base plate, and track plate should be zero, thus we obtain:

$$\int du_r = \int P_r dx / E_r A_r = 0 \quad (12)$$

$$\int du_s = \int P_s dx / E_s A_s = 0 \quad (13)$$

$$\int du_d = \int P_d dx / E_d A_d = 0 \quad (14)$$

According to different types of longitudinal forces, the aforementioned differential equations have different solution methods and applicable ranges. After considering the displacement of piers and abutments as well as the nonlinearity of the longitudinal interlayer resistances between layers, the differential equations form a nonlinear system that requires numerical methods for solution. To facilitate the solution, the finite element method is adopted to discretize each structural layer into elements.

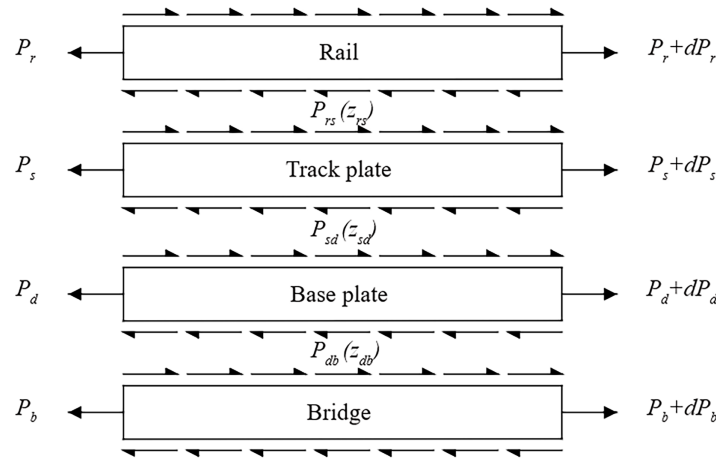


Figure 11: Interaction mechanism of the track-bridge system.

As shown in Fig. 12, a coupled numerical model integrating the HSRTBS with the CCSDs was developed in this study. Elastic beam-column elements with a uniform length of 0.65 m were adopted to simulate the primary load-bearing components. These components include the girder, rails, track plate, base plate, friction plate, and hydraulic bearing layer (HBL). Their material constitutive relationships are summarized in Table 3 [30], and they are assumed to remain essentially elastic under the design-basis earthquake. The connection components were all represented by zero-length elements with mechanical parameters listed in Table 4 [30]. These components include fixed bearings, moveable bearing, CA mortar layer, sliding layer, shear alveolars, shear rebars, fasteners, and lateral blocks. Nonlinear beam-column elements based on fiber sections were employed for the piers. In the pier model, Steel02 was used for longitudinal reinforcement, while Concrete01 and Concrete04 were assigned to the cover concrete and core concrete, respectively. The parameters of the concrete materials are listed in Table 5. For simplicity, the pier bases were assumed to be

fixed, and pile-soil interaction was excluded from the current model. The CCSD was also modeled as a zero-length element, with its constitutive behavior defined by the Steel02 material model. This enables accurate simulation of strength degradation, stiffness deterioration, and hysteretic energy dissipation.

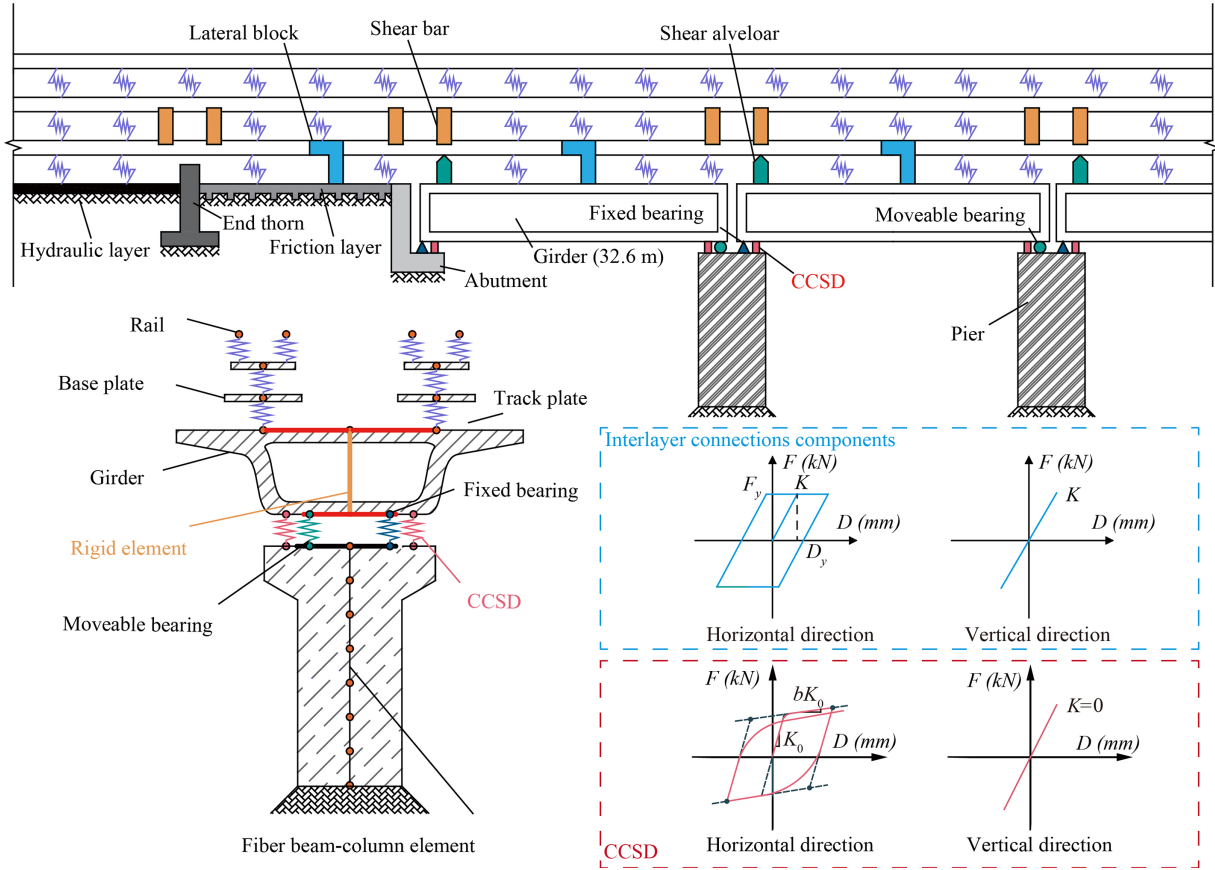


Figure 12: High-speed railway track-bridge system finite element model.

Table 3: Material parameters for system elastic elements.

Component	Material	Poisson Ratio	Density (kg/m^3)	Elastic Modulus (MPa)
Rail	Steel (Q235)	0.3	7850	2.06×10^5
HBL	Hydraulic material	0.2	2500	1.80×10^3
Track plate	Concrete (C55)	0.2	2500	3.65×10^4
Friction plate	Concrete (C30)	0.2	2500	3.25×10^4
Base plate	Concrete (C30)	0.2	2500	3.25×10^4
Girder	Concrete (C50)	0.2	2500	3.55×10^4

Table 4: Material parameters for system zero-length elements.

Component	Longitudinal and Transverse				Vertical Direction
	F_l (kN)	D_l (mm)	F_t (kN)	D_t (mm)	K_v (kN/mm)
Fixed bearing	1000	2	1000	2	2×10^6
Moveable bearing	100	2	100	2	2×10^6
Sliding layer	14	0.5	14	0.5	/
CA mortar layer	42	0.5	42	0.5	2×10^4
Shear alveolar	1465	0.12	1465	0.12	2×10^4
Shear rebar	23	0.08	23	0.08	2×10^3
Fastener	15	2	60	0.5	2×10^4
Lateral block	0	0	453	2	1.4×10^6

Table 5: Material parameters for concrete of the pier.

	$f_{ck}/(\text{MPa})$	ϵ_{c0}	$f_{cu}/(\text{MPa})$	ϵ_{cu}
Concrete01	28.00	0.002	5.60	0.004
Concrete04	29.70	0.0026	5.94	0.012

Note: f_{ck} is the compressive strength; ϵ_{c0} denotes the peak compressive strain; f_{cu} is the ultimate compressive stress; ϵ_{cu} is the ultimate compressive strain.

The accuracy of the numerical model was validated by comparing the simulated pier hysteretic response against low-cycle fatigue test data reported in reference [33]. As shown in Fig. 13a, a close agreement between the numerical and experimental curves is observed, even when material variability and measurement uncertainties are considered. Furthermore, the overall system model was validated using shaking table test results from existing studies on multi-span simply supported girder bridges equipped with CRTS II ballastless track systems [32]. For all key mechanical responses, the relative deviations between the simulated outputs and experimental measurements remain below 20%. Fig. 13 also presents a comparison between the hysteretic behavior predicted by the Steel02 model for the CCSD and that obtained from the refined Abaqus simulation. It is evident that the Steel02 model can capture the mechanical response of the CCSD well. The energy dissipation comparison between the Steel02 and Abaqus models for the CCSD is shown in Table 6. The total cumulative energy error under cyclic loading is 8.12%, meeting the engineering tolerance for seismic device modeling. The above validation work demonstrates that the established finite element model of the high-speed railway track-bridge system exhibits good reliability and computational accuracy.

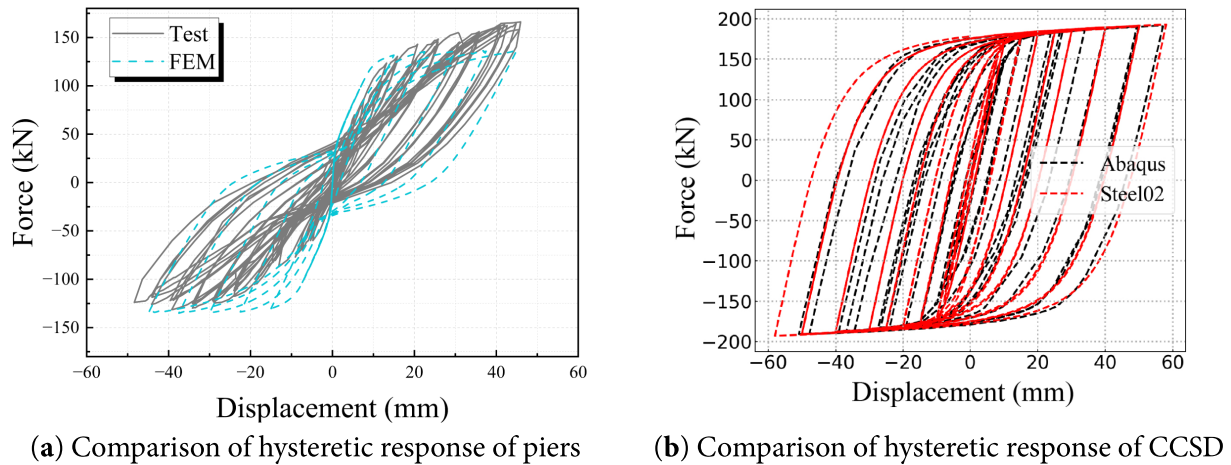


Figure 13: Validation of pier and CCSD models.

Table 6: Comparison of energy dissipation of CCSD.

Cycle	Cyclic Energy Dissipation			Total Cumulative Energy		
	Abaqus	Steel02	Error (%)	Abaqus	Steel02	Cumulative Error (%)
1	0.001	0.0073	-633.45	0.001	0.0073	-633.45
2	0.0213	0.0299	-37.54	0.0223	0.0372	-64.22
3	0.0912	0.3113	-241.8	0.1135	0.3484	-206.97
4	0.3892	0.4093	-5.16	0.5027	0.7578	-50.75
5	0.6871	0.4653	33.28	1.1898	1.2231	-2.80
6	0.6126	0.7459	-21.77	1.8024	1.969	-9.24
7	2.2429	1.659	26.03	4.0453	3.628	10.32
8	2.5649	3.5696	-39.17	6.6102	7.1976	-8.89
9	4.4895	4.4722	0.38	11.0997	11.6698	-5.14
10	7.6319	6.6667	12.65	18.7315	18.3365	2.11
11	8.8519	7.519	15.06	27.5834	25.8555	6.26
12	10.3241	9.8429	4.66	37.9076	35.6984	5.83
13	10.2674	10.6949	-4.16	48.175	46.3933	3.70
14	15.0233	13.1099	12.74	63.1983	59.5032	5.85
15	18.2235	14.9065	18.2	81.4218	74.4097	8.61
16	23.2563	19.9258	14.32	104.678	94.3355	9.88
17	23.7288	21.7147	8.49	128.4068	116.0502	9.62
18	27.1902	26.9118	1.02	155.597	142.962	8.12

3.2 Selection of Ground Motion Records

Nonlinear dynamic time-history analysis of structures relies critically on ground motion records. Three principal strategies are currently adopted for selecting such records: the seismological parameter-based approach, the target spectrum-based method, and the most unfavorable earthquake method. Among these, the seismological parameter-based method is mainly employed to examine the effects of target

earthquakes without accounting for structural characteristics. Commonly used seismological parameters include magnitude, epicentral distance, and site conditions, and multiple parameters can be combined to narrow down the selection range. The bridge site is classified as Seismic Fortification Intensity VII with Site Class I. Based on this, 40 representative ground motion records were chosen from the Los Angeles SAC project [34–37], including 21 near-fault non-pulse-like records and 19 near-fault pulse-like records. Fig. 14 displays the acceleration response spectra of these selected records, and Table 7 summarizes the corresponding information. The peak ground acceleration of the selected records ranges from 0.36 to 1.03 g, with magnitudes varying between 5.9 to 7.1. These ground motions have been extensively adopted for seismic performance evaluation of engineering structures both in China and internationally. In the present analysis, the ground motion input is assumed to be identical at all supports, and the excitation is applied in the transverse direction of the bridge [30].

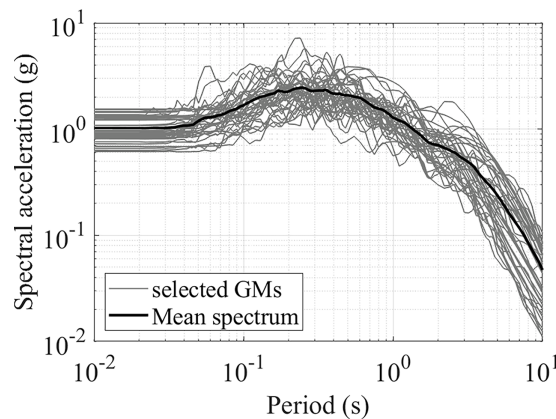


Figure 14: Ground motion response spectrum.

Table 7: Selected ground motion information.

Record	Earthquake Name	Station	Year	Magnitude	PGA	Duration
1	Imperial Valley-02	El Centro Array #9	1940	7.0	0.4432	40.000
2	Imperial Valley-06	Agrarias	1979	6.5	0.78411	28.360
3	Imperial Valley-06	Brawley Airport	1979	6.5	0.48525	37.820
4	Imperial Valley-06	Chihuahua	1979	6.5	0.59159	40.000
5	Imperial Valley-06	EC Meloland Overpass FF	1979	6.5	0.42457	36.975
6	Imperial Valley-06	El Centro Array #10	1979	6.5	0.86109	39.035
7	Imperial Valley-06	El Centro Array #11	1979	6.5	0.4742	39.010
8	Imperial Valley-06	El Centro Array #12	1979	6.5	0.59475	39.545
9	Imperial Valley-06	El Centro Array #3	1979	6.5	0.5203	39.000
10	Imperial Valley-06	El Centro Array #4	1979	6.5	0.51317	39.285
11	Imperial Valley-06	El Centro Array #5	1979	6.5	0.54	39.035
12	Imperial Valley-06	El Centro Array #6	1979	6.5	0.78947	37.560
13	Imperial Valley-06	El Centro Array #8	1979	6.5	0.59552	38.960
14	Imperial Valley-06	El Centro Differential Array	1979	6.5	0.59551	37.745
15	Imperial Valley-06	Holtville Post Office	1979	6.5	0.73686	39.330
16	Imperial Valley-06	Parachute Test Site	1979	6.5	0.44715	39.985
17	Victoria, Mexico	Chihuahua	1980	6.3	0.35994	26.920
18	Westmorland	Parachute Test Site	1981	5.9	0.37021	40.000

(Continued)

Table 7 (continued)

Record	Earthquake Name	Station	Year	Magnitude	PGA	Duration
19	Chalfant Valley-02	Bishop-LADWP South St	1986	6.2	0.84287	39.980
20	Superstition Hills-01	Wildlife Liquef. Array	1987	6.2	0.58765	29.805
21	Superstition Hills-02	El Centro Imp. Co. Cent	1987	6.5	0.56735	40.000
22	Superstition Hills-02	Westmorland Fire Sta	1987	6.5	0.47986	40.000
23	Loma Prieta	Gilroy Array #4	1989	6.9	1.0324	39.945
24	Loma Prieta	Saratoga-Aloha Ave	1989	6.9	0.7798	39.950
25	Erzican, Turkey	Erzincan	1992	6.7	0.63338	39.955
26	Northridge-01	Arleta-Nordhoff Fire Sta	1994	6.7	0.49894	40.000
27	Northridge-01	Canoga Park-Topanga Can	1994	6.7	0.54151	24.990
28	Northridge-01	Jensen Filter Plant	1994	6.7	0.48722	28.615
29	Northridge-01	N Hollywood-Coldwater Can	1994	6.7	0.75402	47.770
30	Northridge-01	Newhall-Fire Sta	1994	6.7	0.77168	21.920
31	Northridge-01	Pacoima Kagel Canyon	1994	6.7	0.72185	40.000
32	Northridge-01	Rinaldi Receiving Sta	1994	6.7	0.72737	19.910
33	Northridge-01	Sun Valley-Roscoe Blvd	1994	6.7	0.55419	30.280
34	Northridge-01	Sylmar-Converter Sta East	1994	6.7	0.71441	40.000
35	Kobe, Japan	Shin-Osaka	1995	6.9	0.88618	55.900
36	Duzce, Turkey	Bolu	1999	7.1	0.38807	25.885
37	Duzce, Turkey	Duzce	1999	7.1	0.43036	39.010
38	Chi-Chi, Taiwan-03	CHY024	1999	6.2	0.58395	51.085
39	Chi-Chi, Taiwan-04	CHY074	1999	6.2	0.51348	112.045
40	Chi-Chi, Taiwan-04	CHY080	1999	6.2	0.57081	98.005

Note: PGA is the peak ground acceleration.

4 Simulation Results of Seismic Response for the HSRTBS

In structural safety assessment, the peak response of the structure under strong ground motions is typically adopted as a key indicator for evaluating whether the structure exceeds its limit states [30]. To simplify the assessment process, this study uniformly uses the average peak response under 40 ground motion records as the indicator for evaluating structural safety. Two cases are considered in the analysis: BTS, representing the conventional high-speed railway track-bridge system, and BTS-C, representing the energy-dissipated system equipped with the CCSD.

As key supporting components, the seismic response of bearings directly affects the safety of the bridge superstructure. Fig. 15a presents the peak displacement response of the fixed bearings. After installing the CCSD (BTS-C), the average peak displacement of the fixed bearings was reduced by 43.55% compared with the uncontrolled case (BTS), with a maximum reduction of 50.54%. Taking the fixed bearing at Pier 3 as an example, its maximum displacement decreased from 106.3 to 67.1 mm, a reduction of 36.88%. The improvement in moveable bearings is even more pronounced, as shown in Fig. 15b. Their average response decreased by 43.81%, with a maximum reduction of 51.22%. This percentage difference is primarily attributable to the distinct initial displacement compositions of the two bearing types. Furthermore, the friction mechanism of the movable bearings forms a parallel energy dissipation path together with the hysteretic energy dissipation of CCSD. However, this does not imply that the movable bearings have 'better compatibility'. In reality, fixed bearings, owing to their full longitudinal constraint, sustain larger seismic forces and possess a lower displacement threshold, making them the more vulnerable weak link in the system. Overall, CCSD provides effective protection for both types of bearings.

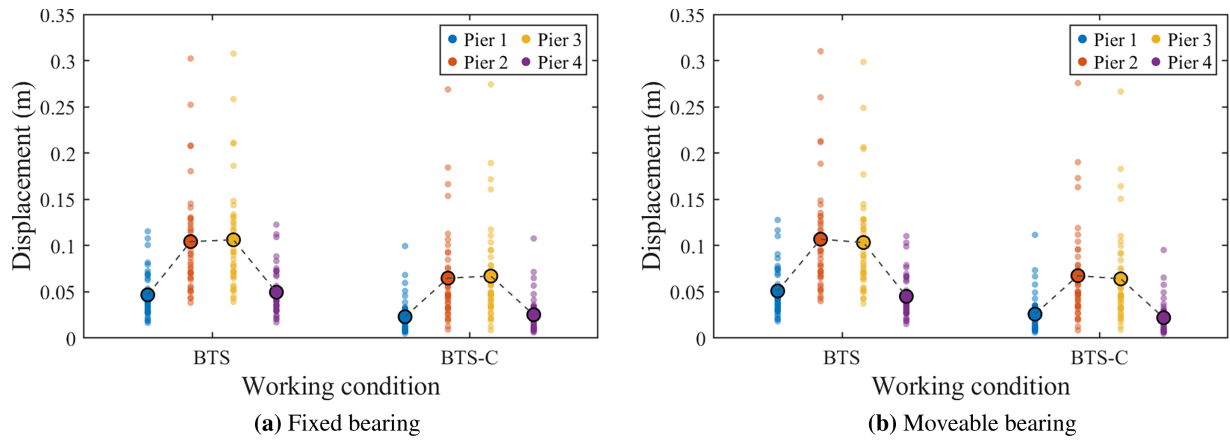


Figure 15: The deformation of the bearings components.

Fasteners serve as critical connections between the track and the bridge, and changes in their response reflect adjustments in the girder–rail interaction. The variation in peak fastener response within the high-speed railway track-bridge system is illustrated in Fig. 16: at the bridge-subgrade transition zone, the fastener response decreased from 0.489 to 0.438 mm, a reduction of 10.43%. Comparing the curves for the BTS and BTS-C cases reveals that the response changes are more pronounced within the bridge span (100–400 m), while the variation in the subgrade sections (<100 m, >400 m) is relatively small. The transition zone is a region where abrupt changes in vertical/longitudinal stiffness occur between the bridge deck and the embankment. The fastener forces here are predominantly governed by the differential displacement between the bridge end and the adjacent subgrade—a localized effect that is only indirectly influenced by the overall seismic response of the bridge. Although CCSD reduces the global displacement of the bridge, the relative displacement jump across the transition interface is driven by the independent movement of the subgrade combined with the continuity constraint of the rail, which cannot be fully controlled by CCSD alone. This spatial distribution characteristic indicates that the CCSD primarily reduces the shear demand on the fasteners by suppressing the overall vibration of the bridge, but has a limited effect on optimizing local constraints at the transition zone. Therefore, to comprehensively enhance the system performance, local strengthening measures such as end anchorage or adding restraining blocks are necessary. In addition, other strategies may be considered, including the installation of rail expansion devices at the bridge-subgrade interface, designing a gradual stiffness transition, or optimizing fastener stiffness.

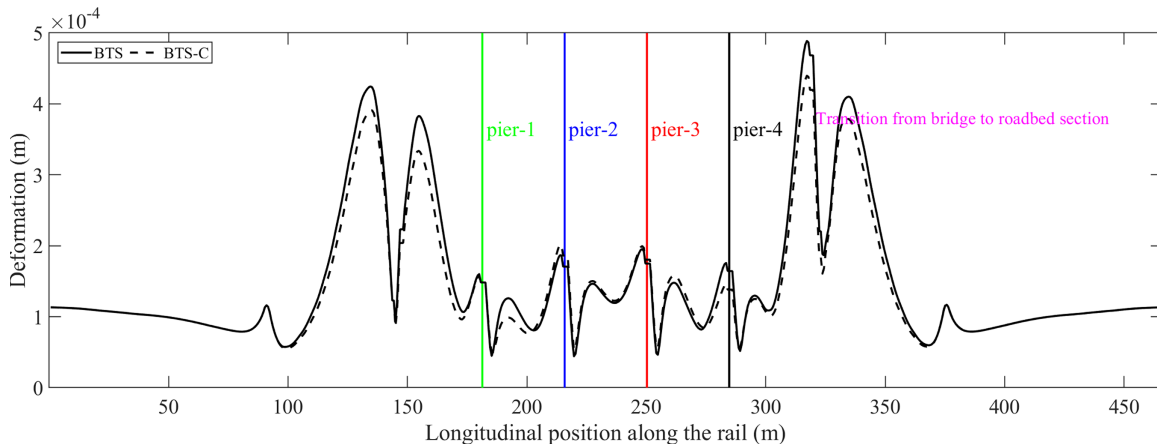


Figure 16: The deformation of the fastener.

Fig. 17 shows the variation of the main girder's seismic peak response. As the main load-bearing component, the mid-span region of the main girder is a vulnerable part of the structure under seismic action. After the installation of the CCSD, its peak displacement decreased from 120.7 to 96.3 mm, a reduction of 20.22%. The results indicate that the CCSD effectively suppresses the bending vibration mode of the main girder by dissipating seismic energy, thereby providing protection to the superstructure.

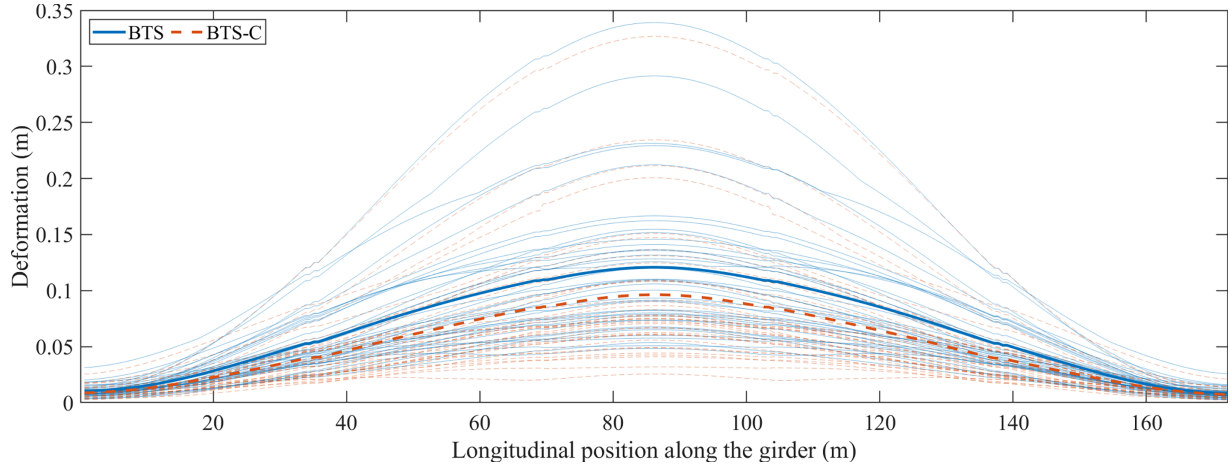


Figure 17: The deformation of the girder.

Fig. 18 illustrates the variation in the peak response of the rail. It can be observed that after the installation of the CCSD, the peak displacement at the mid-span of the rail decreased from 120.8 to 96.4 mm, representing a reduction of 20.20%. This indicates that the CCSD dissipates seismic energy through plastic deformation, thereby reducing the risk of local stress concentration in the rail.

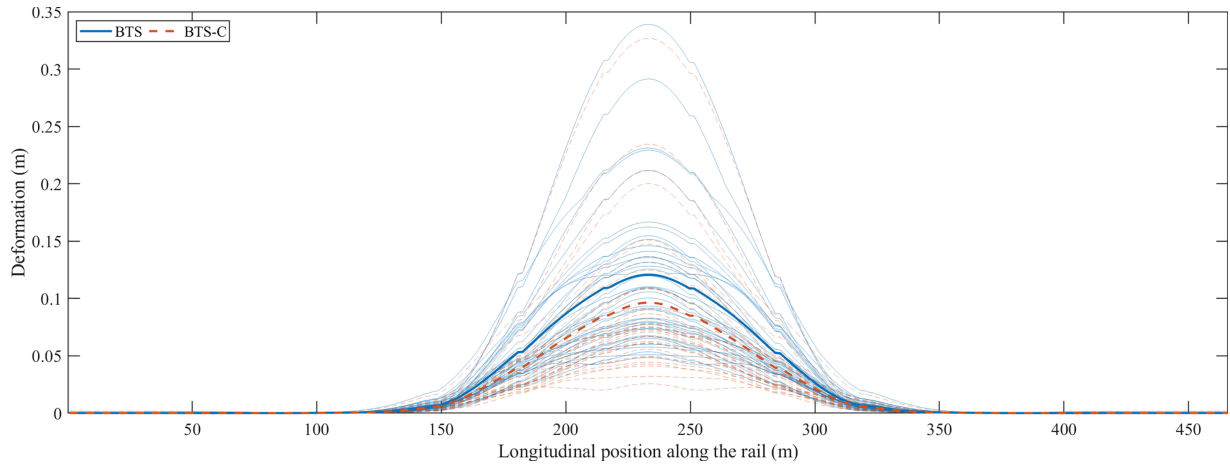


Figure 18: The deformation of the rail.

Fig. 19a show the variation in the peak response of the piers. After the installation of the CCSD, the average peak response of the piers increased by 29.86%, with a maximum increase of 38.80%. The pier drift ratio is adopted as the quantitative damage indicator, and Table 8 presents the corresponding threshold values for each damage level. Based on these criteria, although the pier response increased, the damage state remained within an acceptable range and did not reach the moderate damage level. From the perspective of

the overall system performance, the CCSD can still effectively enhance the seismic resilience of the HSRTBS. The amplification of pier response is attributed to the fact that the CCSD increases the overall stiffness of the vertical load-bearing system formed by the piers and bearings, making its mechanical behavior more elastic, which consequently leads to an increase in the bending moments and shear forces acting on the piers [30].

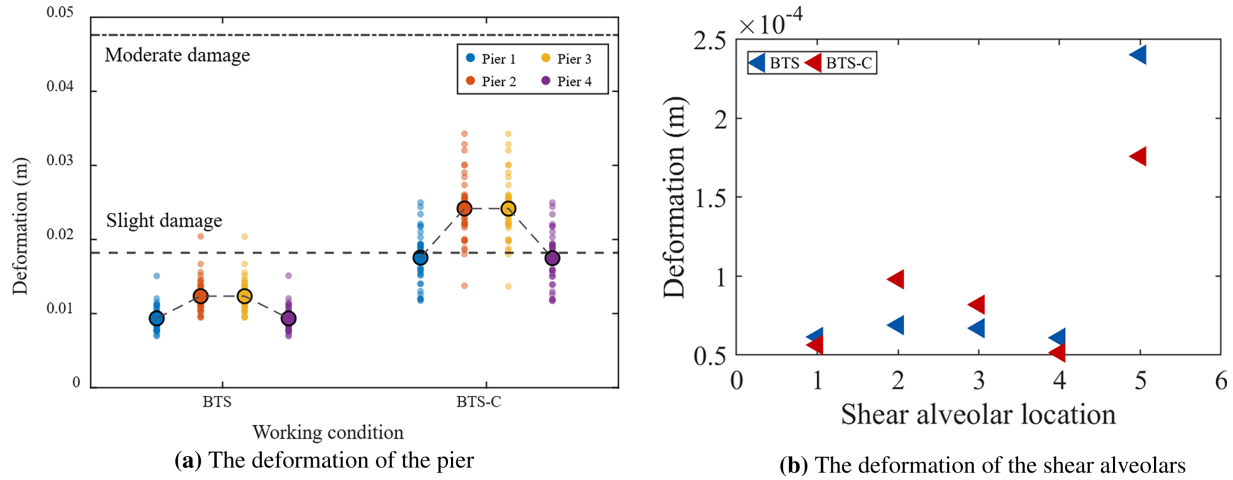


Figure 19: The deformation of the pier and shear alveolars.

Table 8: Damage indicators for piers [30].

Component	Damage Indicator	Slight	Moderate	Severe	Complete	Unit
Pier	Displacement/Height	0.13	0.34	0.95	2.05	%

Fig. 19b illustrates the spatial distribution characteristics of the peak response of the shear alveolars. The responses at different locations show non-uniform changes: the deformation decreases at Positions 1, 4, and 5, while it increases slightly at Positions 2 and 3. This discrepancy may be related to the local stress redistribution induced by the damper arrangement, and further parameter optimization is needed to achieve a balanced restraining effect. It is worth noting that the dynamic response of the shear alveolar at Position 5, located in the bridge-subgrade transition zone, is relatively significant. Controlling the response at this location with the CCSD is crucial for maintaining the structural integrity of the bridge-track system, and therefore this research focuses on it as a key analysis area. The data show that the peak response at Position 5 decreases from 0.24 to 0.17 mm, a reduction of 29.17%, effectively lowering the interfacial shear stress and contributing to the preservation of track geometry under seismic action.

5 Conclusions

This study establishes a numerical model of a combined curved steel damper incorporating a novel porous energy-absorbing material and analyzes its hysteretic energy dissipation characteristics. The damper is then incorporated into a high-speed railway track-bridge system model, and its capability to enhance the system's seismic performance is systematically assessed via nonlinear time-history analysis. The key findings are summarized as follows:

- (1) A numerical analysis model of the combined curved steel damper using porous metal energy-absorbing material was developed, and its hysteresis curve was obtained through cyclic loading simulation. The

curve exhibits a plump, symmetric spindle shape, is continuous and complete without pinching, and demonstrates excellent energy dissipation efficiency and stable stiffness characteristics, verifying the device's reliable capability to achieve the intended vibration reduction objectives.

- (2) A finite element model was developed for the high-speed railway track-bridge system incorporating the CCSD, and the device's contribution to improving the system's seismic performance was investigated in a systematic manner. The results show that installing the CCSD significantly improves the seismic response of most key components: the average reductions in peak response for fixed bearings and moveable bearings reach 43.55% and 43.81%, respectively; the responses of the main girder, rail, and shear alveolars are also effectively controlled, with reductions of 20.22%, 20.20%, and 29.17%, respectively; the peak response of fasteners decreases by 10.43%, indicating improvement as well. Although the peak response of the piers increases, the damage remains at a slight level, which is within the acceptable engineering range. In summary, the CCSD can significantly enhance the overall seismic performance of the system.

It should be noted, however, that soil-structure interaction was not considered in this study, and the fatigue failure behavior of the PM-35 porous metal has yet to be thoroughly investigated. Future investigations should aim to address these limitations, with special attention given to the effects of parameter uncertainty and the necessity of thoroughly validating the model's performance under diverse boundary conditions.

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