Studies on Train-Bridge Coupling System Considering Thermal Effect of Piers

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***Abstract*—In order to investigate the influence of the pier height change due to thermal effect on train riding performance for long span concrete continuous rigid frame bridge, a full scale model is established and its free vibration analysis is performed. Then the spatial vibration responses of train-bridge coupling system are analyzed considering two temperature conditions, e.g. normal condition and 20oC thermal difference. Consequently, the vertical and lateral stiffness of the bridge and the vehicle riding performance are verified according to the evaluation indices. The results show that: (1) For both calculation conditions, the longitudinal and transverse displacement of the bridge is within the limit as well as the acceleration, which means that the influence of the pier height change due to thermal effect is negligible; (2) Under the condition of 20oC thermal difference, with CRH2 and CRH3 high-speed train passing through, the indices for the vehicle, including derailment coefficient, wheel unloading rate and lateral wheel-rail force, are within the limit and similar to the results obtained under normal condition; (3) Under the condition of 20oC thermal difference, with CRH2 and CRH3 high-speed train passing through, the lateral and vertical Sperling Index is comparable with the results from normal condition, and all the results can meet the requirement, which means that the influence of the pier height change on vehicle riding comfort can be neglected; (4) The obtained results can be a useful reference in the dynamic design for similar railway bridge.**

***Index Terms*—thermal effect, track irregularities, train-bridge coupling, simulation analysis**

I. Introduction

Temperature change leads to linear expansion on adjacent piers and abutments of the bridge, referred as thermal effect, which will result in height change of the pier, especially for the high piers on long span concrete continuous rigid frame bridge with different pier height and large temperature variation. The height change of the pier will further lead to deformation of the bridge, which will influence the track irregularity. It is known that, track irregularity is an important exciting source for train-bridge coupling system.

The spatial vibration problem of the train-bridge system is prominent with high speed train passing through the bridge [1-2]. Nowadays, static study of concrete continuous rigid frame bridge is well developed and a considerable number of investigations have been conducted on the dynamic behavior of train-bridge coupling system. Nevertheless, the study of the influence of thermal effect induced pier height change on the dynamic response of the train-bridge coupling system is still limited, which should be taken into account in the dynamic design of the bridge.

In this paper, a high pier long span continuous rigid frame bridge is selected as the prototype bridge. The dynamic property of this bridge is analyzed firstly. Then, the pier height change under two different temperature conditions is employed to the track irregularity. Afterwards, the vibration performance of the train-bridge coupling system is analyzed with CRH2 and CRH3 high-speed train passing on, hence to investigate whether the vibration response of the bridge and the safety and riding comfort of the cars can meet the requirements.

II. Project Overview

The prototype bridge is a continuous rigid frame bridge with the span of (72+128+72)m with pre-stressed concrete box girder connected at the end, as illustrated in Fig. 1. The girder, which is a single box single cell box girder with variable cross section, as shown in Fig. 2, has a deck width of 12.2m, a total length of 273.4m and girder height of 5.4m and 9.8m at the middle of the span and central bearing point respectively. Two main piers, which are round hollow piers, have a height of 82m and 85m, with lateral width of 7.2m and longitudinal width of 8.0m on the top. The outer and inner slope of the pier is 40:1 and 20:1 for the top 60m and 50:1 and 5:1 lower than 60m in the lateral direction, and the slope is vertical in the longitudinal direction, as shown in Fig.3.

III. Vibration Model of Train-Bridge Coupling System

In the train-bridge coupling system, the train vehicle model is developed in ADAMS/RAIL software and the bridge is modeled in MSC.PATRAN software. Then, the bridge model is imported into ADAMS/RAIL with obtained modes, and afterwards the train-bridge coupling system is integrated through wheel-rail interaction and simulated by using modal analysis method.

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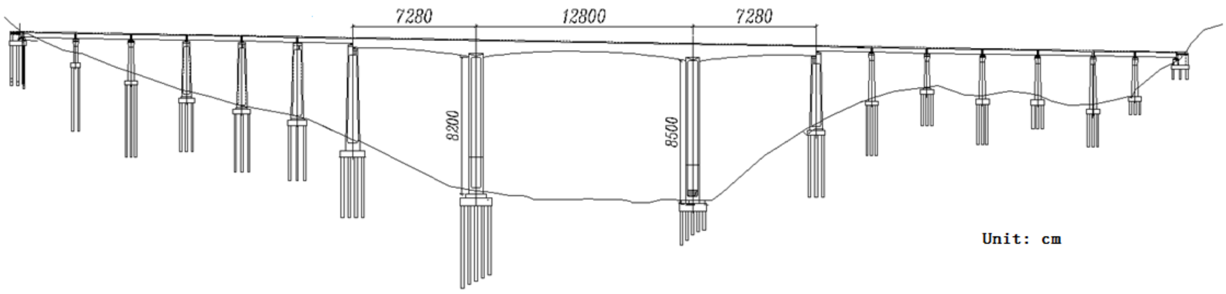

Figure 1. Layout of the bridge

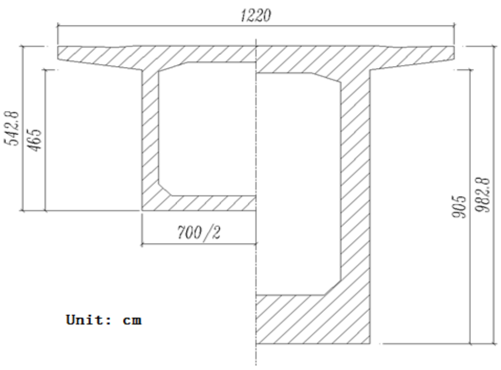

Figure 2. Cross section at mid-span and supporting point.

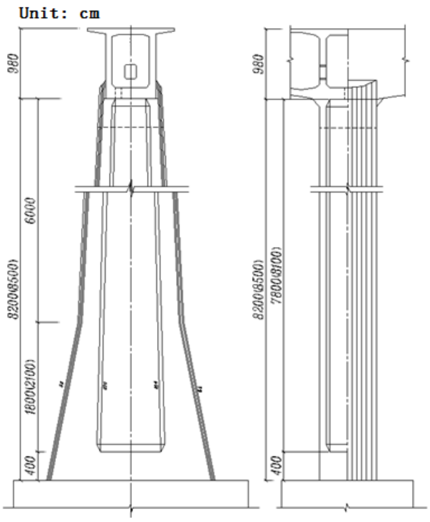

Figure 3. Pier details, lateral (left) and longitudinal (right).

A. Train Model

In this study, the train model is composed of several locomotives and vehicles. Each locomotives or vehicle consists of a car body, two bogies, four wheel-sets, and the spring and damping connections between the three components. The vehicle body has five degrees-of-freedom (dofs), e.g. lateral displacement, roll displacement, yaw displacement, vertical displacement and pitch displacement, as shown in Fig. 4. Three dofs are considered for the wheel with lateral displacement, roll displacement and vertical displacement. Thus, the idealized model for each vehicle with 2-bogies and 4-axles can be modeled by a 27-dof dynamic system. Train formation is listed in Table I.

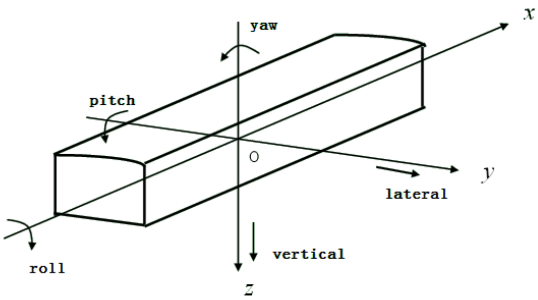

Figure 4. Five degrees of freedom for car body.

TABLE I. TRAIN FORMATION

Train type	Classification	Train speed (km/h)	Track irregularity
CRH2 train	4×(3 motor car+1 trailer)	200、250	German low-interference track irregularity
CRH3 train	4×(2 motor car+1 trailer)	250、300	

B. Bridge Model

A 3D finite element model of the bridge is established in original scale using beam element for the girder, pier and pile foundation, with 1270 nodes and 1178 elements in total, as shown in Fig. 5. The bearing is simulated by master-slave constraint, and the elastic modulus and Poisson ratio are chosen according to Chinese railway bridge code. The secondary dead load is converted into equivalent mass weight and then assigned to the corresponding bridge elements.

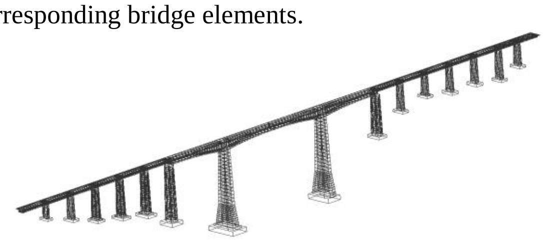

Figure 5. Finite element model of the bridge.

C. Solutions to the Vibration Response of Train-Bridge Coupling System

The vibration equations of the bridge system and the vehicle system are integrated into a coupling system based on the equilibrium of the forces and the deformation compatibility, illustrated in Fig. 6. The dynamic characteristics of the bridge and the vehicles are obtained in the time domain by an iterative procedure at each time step based on equations of motion of vehicle and bridge system. The interaction between the train and bridge can be calculated through the compatibility condition of the displacement and equilibrium equations of the wheel/rail forces.

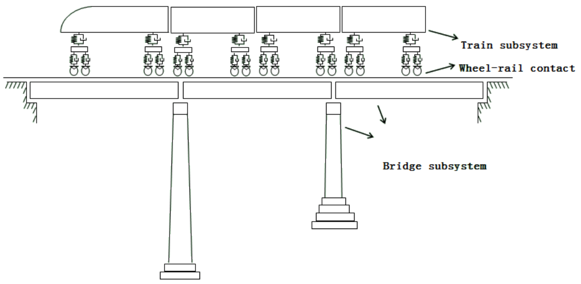

Figure 6. Train-bridge interactive dynamic model.

Hertzian contact theory is utilized for wheel-rail interaction. The vertical wheel-set forces comprise of axle weight, inertial force, and the forces from connected vertical springs and dampers. And, in the lateral and longitudinal direction, there exists creep force due to wheel-rail interaction, which can be simulated on the basis of Kalker contact creep theory [3-4].

IV. Indices for Safety and Riding Comfort of the Vehicle and the Threshold for Bridge Vibration Response

Presently, several parameters are used to evaluate the safety of the train vehicle, such as derailment coefficient and wheel unloading rate. Plus, vehicle acceleration and Sperling index are usually employed to evaluate the riding comfort of the running railway cars. In this paper, the evaluation indices for the safety and riding comfort of the railway cars and the threshold for the vibration response of the bridge are illustrated in Table Ⅱ, according to the criteria adopted in previous speed raising experiments [5-7].

TABLE II. Indices for the Safety and Riding Comfort of the Railway Cars and the Limit for the Vibration Response of the Bridge

Evaluation index	Parameter	Criteria
Safety index	Derailment coefficient	≤0.8
	wheel unloading rate	≤0.6
Riding comfort index	Acceleration of the car body	vertical≤0.25g；lateral≤0.2g（intermediate speed≤200km/h）
		vertical≤0.13g；lateral≤0.1g（high speed≥200km/h）
	Sperling comfort index	Excellent <2.5
		Good 2.50～2.75
		Qualified 2.75～3.0
Threshold for vibration response of the bridge	Vertical acceleration	0.35g=3.5m/s^2（half range、ballasted track）
		0.50g=5.0m/s^2（half range、ballastless track）
	Lateral acceleration	0.14g=1.4m/s^2（half range）

V. Characteristics of Bridge Free Vibration

Modal analysis is used to solve the dynamic response of the bridge. Owing to the fact that the dynamic response is dominantly influenced by its several lowest modes, this approach has a great advantage that an adequate estimation of the dynamic response can be obtained by considering only a few modes of vibration, especially for complex structure with hundreds of degrees-of-freedom. The results of free vibration of this bridge are presented in Table III.

TABLE III. Results of Free Vibration Analysis for Continuous Rigid Frame Bridge with the Span (72+128+72)m

Number	Frequency/Hz	Mode type
1	0.538	Longitudinal drift
2	0.683	1st order symmetric transverse bending
3	0.837	1st order anti-symmetric transverse bending
4	1.590	1st order symmetric vertical bending
5	2.361	1st order anti-symmetric vertical bending

It is presented in the table that, when the horizontal restraint is released at the two ends of the bearings in rigid frame bridge, allowing the longitudinal movement, the fundamental mode is longitudinal drift, and the 1st order transverse and vertical bending frequency is 0.683Hz and 1.590Hz respectively, which shows that the vertical stiffness is larger than the lateral one for this bridge.

VI. Simulation Method for the Pier Height Change Due to Thermal Effect

Although the deformation of the pier and abutment caused by thermal effect is negligible regarding to the span of the bridge, it is rather important for the track since it will lead to the vertical displacement of the girder and hence the track will deform consequently. So, the irregularity indirectly induced by thermal effect is considered as one kind of track irregularity, and it can be classified as long-wave irregularity which will have a significant influence on high-speed railway with high demanding of the riding comfort. Therefore, an integrated irregularity curve is introduced in this paper, combining the irregularity from the deformation of the pier due to thermal effect with the original track vertical irregularity, to analyze the spatial vibration response of the bridge subjected to vehicle dead load. Under the circumstance of temperature variation of 20^{0}C, the vertical deformation of each pier is listed in Table IV, noted that Pier 6~9 are the piers in rigid frame bridge. It shows that the vertical deformation of the pier is increasing with the height increasing.

TABLE IV. VERTICAL DEFORMATION OF THE PIER DUE TO THERMAL EFFECT

Pier No.	Height (m)	Vertical deformation (mm)	Adjacent deformation difference (mm)
0 (abutment)	5	1	1.8
1	14	2.8	2
2	24	4.8	1.8
3	33	6.6	1.2
4	39	7.8	1.2
5	45	9	3.2
6	61	12.2	4.2
7	82	16.4	0.6
8	85	17	-8.2
9	44	8.8	-3.1
10	28.5	5.7	-1.7
11	20	4	1.2
12	26	5.2	0
13	26	5.2	1
14	31	6.2	-1.6
15	23	4.6	-3.2
16 (abutment)	7	1.4	

Two conditions are considered, e.g. normal service state and state considering temperature difference of 200C, from which the vertical deformation of the pier is compared afterwards.

The deformation of the pier is linearly interpolated into the distance starting from the point 20m before the bridge span to the place 20m after the end bottom of the bridge, and then added to the original track irregularity in the left and right rail. The combined track irregularity curve is shown in Fig. 7-8.

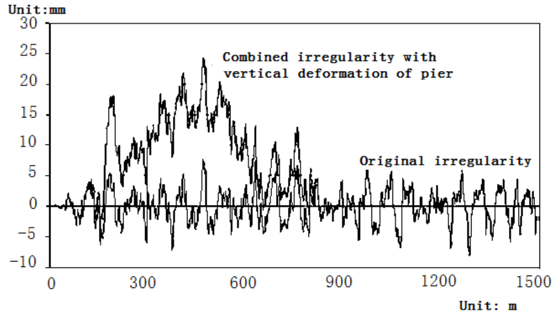

Figure 7. Irregularity curve for left track

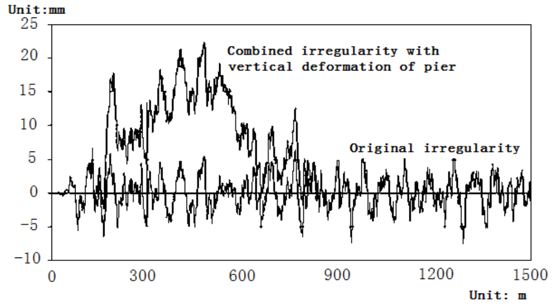

Figure 8. Irregularity curve for right track

VII. SOLUTIONS AND EVALUATION

A. *Investigation of the Thermal Effect on Bridge Response*

The results of the dynamic displacement and acceleration for the bridge at mid-span under the conditions aforementioned are illustrated in Tables V and VI.

TABLE V. MAXIMUM DISPLACEMENTS AT MID-SPAN FOR EACH SPAN (MM)

Span	Condition	CRH2/(km/h)				CRH3/(km/h)			
		200		250		250		300	
		vertical	lateral	vertical	lateral	vertical	lateral	vertical	lateral
Left span	normal	1.153	0.585	1.147	0.442	1.216	0.314	1.415	0.465
	Thermal effect	1.153	0.596	1.147	0.443	1.216	0.309	1.416	0.466
Middle span	normal	3.643	0.569	3.653	0.421	3.685	0.313	4.385	0.346
	Thermal effect	3.642	0.565	3.654	0.436	3.688	0.284	4.390	0.342
Right span	normal	1.245	0.524	1.145	0.514	1.149	0.449	1.503	0.425
	Thermal effect	1.243	0.536	1.144	0.480	1.147	0.339	1.503	0.417

TABLE VI. MAXIMUM ACCELERATION AT MID-SPAN FOR EACH SPAN (M/S^2)

Span	Condition	CRH2/(km/h)				CRH3/(km/h)			
		200		250		250		300	
		vertical	lateral	vertical	lateral	vertical	lateral	vertical	lateral
Left span	normal	0.037	0.008	0.048	0.027	0.100	0.021	0.066	0.021
	Thermal effect	0.035	0.009	0.048	0.025	0.100	0.022	0.066	0.023
Middle span	normal	0.049	0.009	0.123	0.027	0.063	0.009	0.119	0.030
	Thermal effect	0.049	0.010	0.121	0.025	0.064	0.009	0.120	0.031
Right span	normal	0.042	0.010	0.078	0.026	0.079	0.010	0.081	0.040
	Thermal effect	0.042	0.010	0.078	0.027	0.078	0.010	0.083	0.039

TABLE VII. VEHICLE RESPONSES

Parameter	Condition	CRH2/(km/h)				CRH3/(km/h)			
		200		250		250		300	
		Motor car	Trailer	Motor car	Trailer	Motor car	Trailer	Motor car	Trailer
Derailment coefficient （Q/P）	Normal	0.158	0.164	0.190	0.272	0.121	0.122	0.295	0.305
	Thermal effect	0.154	0.164	0.182	0.264	0.176	0.167	0.278	0.325
wheel unloading rate	Normal	0.286	0.327	0.380	0.446	0.295	0.300	0.385	0.418
	Thermal effect	0.295	0.335	0.384	0.451	0.307	0.320	0.382	0.430
Lateral axle forces (kN)	Normal	16.379	15.570	18.396	16.954	15.051	14.268	20.196	22.043
	Thermal effect	16.388	15.740	17.476	16.793	17.578	16.902	20.471	22.748
Vertical acceleration of car body (m/s^2)	Normal	0.765	0.703	0.872	0.900	0.767	0.688	0.959	0.926
	Thermal effect	0.789	0.712	0.904	0.909	0.785	0.706	0.962	0.937
Lateral acceleration of car body(m/s^2)	Normal	0.642	0.736	0.840	0.896	0.509	0.573	0.686	0.716
	Thermal effect	0.629	0.750	0.798	0.931	0.559	0.608	0.719	0.749
Vertical Sperling index	Normal	2.235	2.104	2.283	2.209	2.030	2.104	2.381	2.260
	Thermal effect	2.245	2.112	2.291	2.216	2.190	2.207	2.392	2.278
Lateral Sperling index	Normal	2.190	2.370	2.606	2.660	2.265	2.246	2.355	2.395
	Thermal effect	2.220	2.401	2.595	2.669	2.270	2.287	2.370	2.400

It can be seen that, in Table Ⅴ, when CRH2 train is passing through, the maximum vertical displacements are 3.653mm and 3.654mm for the aforementioned conditions, and the maximum lateral displacements are 0.585mm and 0.596mm, while for CRH3 high-speed train, the corresponding values are 4.385mm and 4.390mm in vertical direction, and 0.465mm and 0.466mm in lateral direction. Similarly, as for the acceleration, shown in Table Ⅵ, the maximum vertical acceleration are 0.123 and 0.121m/s^2 under two conditions when CRH2 train is passing through, and the maximum lateral acceleration are 0.027m/s^2, while for CRH3 high-speed train, the corresponding values are 0.119 and 0.120m/s^2 in vertical direction, and 0.040 and 0.039m/s^2 in the lateral direction. Both the vertical displacement and acceleration is within the limit. It can be concluded that the bridge vibration is highly depending on the train formation and the dynamic property of the vehicle in term of the displacement and acceleration.

It can also be noted that the pier height change due to the thermal effect is negligible for bridge response.

B. *Thermal Effect on Vehicle Response*

The response of the motor car and trailer of CRH2 and CRH3 high-speed train under two conditions are presented in Table VII.

In Table VII, for both types of the train, the results considering thermal effect are similar to the one under normal circumstance, and all the values can meet the requirement. In additional, the acceleration of car body and the Sperling Index in both directions are within the limit, especially the Sperling index being excellent or good eventually.

Moreover, the amplitude of the pier movement is about 16.4mm and the maximum difference of the evaluation of the piers is 8.2mm. However, since it varies gradually and evenly along the bridge, so called local settlement, the height change can be regarded as long-wave irregularity along the total span of the bridge, 680m including the approach bridge. So, despite of the change of pier height, this long-wave irregularity with small amplitude can be neglected in terms of the riding safety, and it has little influence on the riding comfort evaluated by Sperling index.

VIII. CONCLUSIONS

(1) For both calculation conditions, the longitudinal and transverse displacement of the bridge is within the limit as well as the acceleration, which means that the influence of the pier height change due to thermal effect is negligible;

(2) Under the condition of 20oC thermal difference, with CRH2 and CRH3 high-speed train passing through, the indices for the vehicle, including derailment coefficient, wheel unloading rate and lateral wheel-rail interaction, are within the limit and similar to the results obtained under normal condition;

(3) Under the condition of 20oC thermal difference, with CRH2 and CRH3 high-speed train passing through, the lateral and vertical Sperling Index is comparable with the results under normal condition, and all the results can meet the requirement, which means that the influence of the pier height change on vehicle riding comfort can be neglected;

(4) The obtained results can be a useful reference in the dynamic design for similar railway bridge.

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