Wave propagation induced by moving loads in heavy haul railway embankment-bridge transition zone

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ABSTRACT

An elaborate three-dimensional numerical model was developed for the dynamic behavior analysis of a coupled nonlinear track-subgrade-transition zone-foundation on a heavy haul railway. The model was composed of rail, sleeper, fastener, latite ballast, capping layer, subballast, transition zone, pier, main girder, pile cap and foundation. All the parts were modelled with solid element except the fastener, which was simulated with spring and dashpot. The initial stress state in the foundation was generated, on this basis the construction of the substructure was modelled, then the placement of sleeper and rail was simulated followed by an implicit dynamic computation simulating the running of two bogies. It was found that the stress waves induced by the pressure movement propagated not only in forward and backward direction but also in lateral and vertical direction. The moving pressure had a limited influence range. It was observed that there existed intensive stress concentration in the vicinity of transition zone and pier due to the great impedance difference from other location. It was expected that the acceleration response in longitudinal direction had a gradual increase from the subgrade to the transition zone with the corresponding increase of stiffness along the railway, the acceleration reached its maximum near the transition zone-bridge border and had abrupt jump to a very lower value, and then kept almost the lower value along the railway. The findings in this investigation greatly help further study into the mechanism of long-term deformation and stabilization of transition zone in railway engineering.

Keywords: stress waves propagation; transition zone; moving load; heavy haul railway

1 INTRODUCTION

Heavy haul railway plays an important role in transporting heavy goods. The performance of the transition zone on the heavy haul railway is a key issue in design and maintenance. A reasonable design can be made only when the dynamic behavior of the transition zone is thoroughly understood. Many researches have been conducted in this aspects (Tian et al. 2014; Heydarinoghabi et al. 2017; Zhao et al. 2018), however, the models in existing literatures are oversimplified, which is far away from reality. Besides, the initial conditions and boundaries are not correctly dealt with. So it is urgent to conduct more detailed analyses to overcome the above-mentioned disadvantages.

2 MODEL DESCRIPTION

A detailed three-dimensional numerical model was developed for the dynamic response analysis of a coupled nonlinear track-substructure-foundation system on a ballasted heavy haul railway embankment-bridge transition zone. Unified 3D viscoelastic static-dynamic artificial boundaries were adopted to model the infinite foundation domain. Material nonlinearity, contact nonlinearity, and general geometric nonlinearity were simultaneously considered in the analysis. The initial stress state was generated in the foundation followed by the constructions of the substructure and track. The moving pressures imposed on the top of the rails, corresponding to an axle load of 30 tonne and travelling speed of 120 km/h, were then simulated, and the formation, development, and attenuation of the stress waves induced by the moving pressures were studied.

The complete model for the transition zone is composed of several parts, namely the embankment, transition zone, bridge and foundation, as demonstrated in Fig.1.

The detailed model consists of rail, sleeper, fastener, latite ballast, ballast, capping layer and foundation. The sleeper is evenly placed on the latite ballast with fixed spacing of 0.65 m. The fastener is modelled with spring and dashpot, the rest parts are modelled with solid element. The total number of nodes, element and degree of freedom is 1,709,800, 1,220,271 and 5,129,400, respectively.

The parameters used are listed in Table 1. The elastic constitutive model is selected for rail, sleeper, main girder, pier, abutment and pile cap, the Drucker-Prager model is adopted for the rest part of the model except the fastener, which is simulated by spring and dashpot.

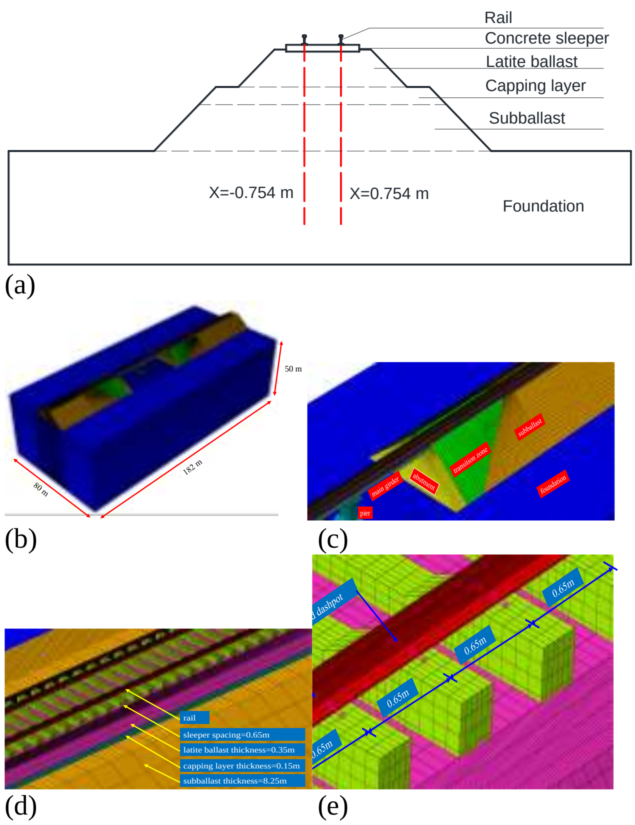


Fig. 1. Numerical model. (a) cross section of subgrade; (b) mesh for the complete model; (c) transition zone details; (d) track and subgrade details; (e) rail and sleeper details.

Table 1. Parameters used and constitutive models.

Name / Value	Rail	Sleeper	Latite ballast	Capping layer	Subballast
Density ($kN\cdot m^{-3}$)	78.0	24.0	15.3	18.0	18.0
Elastic modulus (GPa)	206.0	30	0.2	0.14	0.11
Poisson's ratio	0.3	0.1	0.3	0.35	0.26
Cohesion (kPa)	-	-	0.0	1.0	12.0
Angle of internal friction (°)	-	-	45.0	35.0	26.0
Damping ratio	0.1	0.03	0.04	0.04	0.035

To be continued

Name / Value	Transition	Abutment	Main girder	Pier
Density ($kN\cdot m^{-3}$)	19.0	24.0	24.5	24.0
Elastic modulus (GPa)	1.2	30.0	35.0	30.0
Poisson's ratio	0.25	-	–	–
Cohesion (kPa)	11.0	-	–	–
Angle of internal friction (°)	24.0	-	-	-
Damping ratio	0.05	0.03	0.03	0.03

To be continued

Name / Value	Pile cap	Foundation	Fastener	
			Spring	Dashpot
Density ($kN\cdot m^{-3}$)	23.5	20.0	600 kN/mm	60 kN·s/m
Elastic modulus (GPa)	25.0	0.06		
Poisson's ratio	0.1	0.26		
Cohesion (kPa)		6.0		
Angle of internal friction (°)		28.0		
Damping ratio	0.025	0.075		

The dynamic wheel-rail interaction results in contact forces including the pressure (vertical direction) and the shear stresses (lateral and longitudinal direction) on top of the rail. The mechanism of the contact forces is quite complex. For clarity and simplicity, only the pressure is considered in this analysis and is applied to the rail top in this study.

Emphasis is put on the responses induced by the second bogie of front carriage and the first bogie of the rear carriage, as depicted in Fig. 2. The amplitude of the pressure is determined by wheel-rail contact area and axle load, in this analysis, the contact area is assumed to be 320 mm^2 in term of previous research (Wang et al. 2013; Wei et al. 2015), and the axle load is 30 tonne, so the amplitude of pressure is calculated as 468.75 MPa. The operation speed of the train is set to 120 km/h.

The simulation steps include the generation of initial stress in the foundation, the construction of the subgrade and track system followed by implicit dynamic computation modeling the running of the two bogies of heavy haul train. The unified three-dimensional viscoelastic static-dynamic artificial boundaries are introduced to model the infinite domain (Liu et al. 2005; Xue and Zhang 2014; Xue and Zhang 2016; Xue and Zhang 2018).

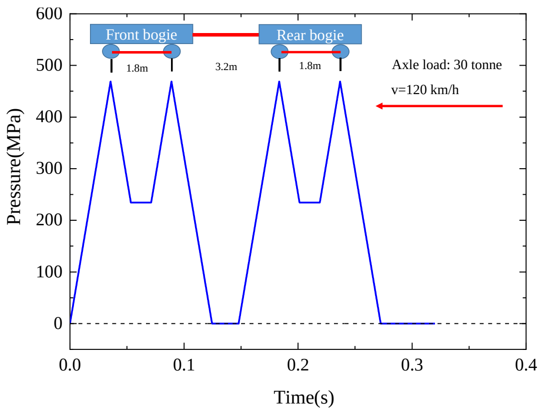
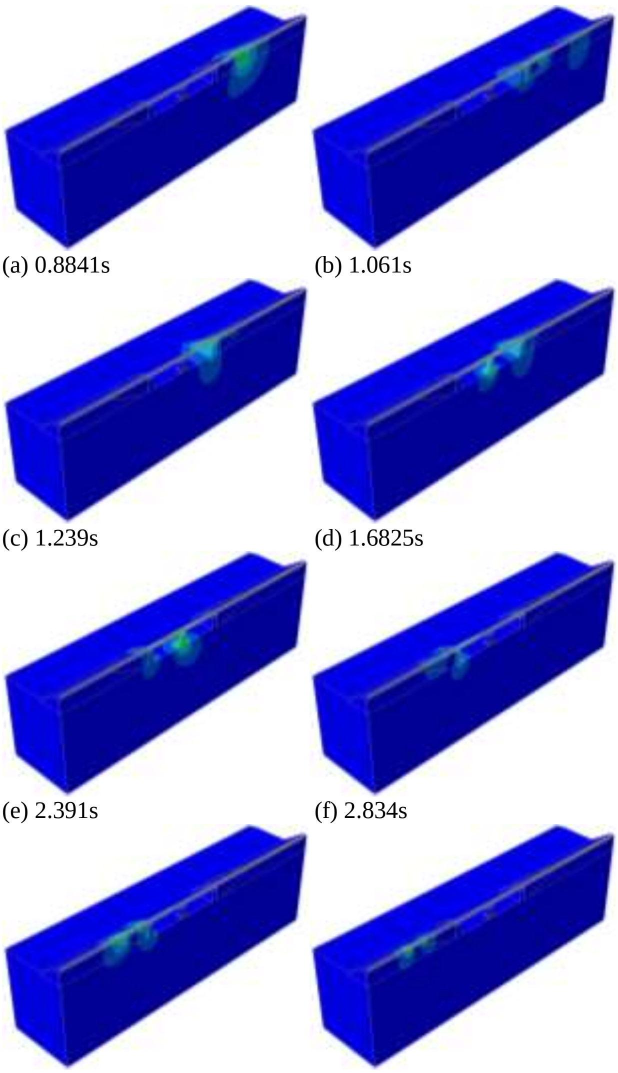


Fig. 2. Pressure imposed on the top of the rail.



(a) 0.8841s (b) 1.061s

(c) 1.239s (d) 1.6825s

(e) 2.391s (f) 2.834s

(g) 3.100s (h) 3.543s

Fig. 3. Distribution of dynamic stress field at different instants.

3 RESULTS AND ANALYSES

In this simulation, the dynamic responses including displacement response, acceleration response, stress response, et al. were obtained. Due to the page limitation, only a small part of the results is presented in this paper for analysis.

3.1 Stress wave propagation analyses

The propagation of stress wave induced by moving pressure can be directly investigated by cutting the model in longitudinal direction using vertical plane at X=0.754 m shown in Fig. 1. The dynamic stress fields at different instants are plotted in Fig. 3.

It was found from Fig. 3 that the pressure movement agitated stress waves in infrastructure. The stress wave propagates not only in the direction of load movement (longitudinal direction) but also in transverse and vertical direction, and even in backward direction. The moving pressures had different influence range at different instants, but the affected area is quite limited.
It could be seen that the wave fields in the vicinity of the transition zone and the pier were much stronger than other locations when the transition zone and pier were in the influence range of the pressures. There existed intensive stress concentration near the transition zone and the pier due to their huge difference of wave impedance.

3.2 Longitudinal responses

The stiffness of subgrade, transition zone and bridge is different, the variation of stiffness along railway line is schematically demonstrated in Fig. 4, the stiffness has abrupt change at the border of the transition zone and the bridge due to the great difference of their materials. The longitudinal distribution of vertical acceleration on capping layer top just below the rail (see Fig. 1) is depicted is Fig. 5.

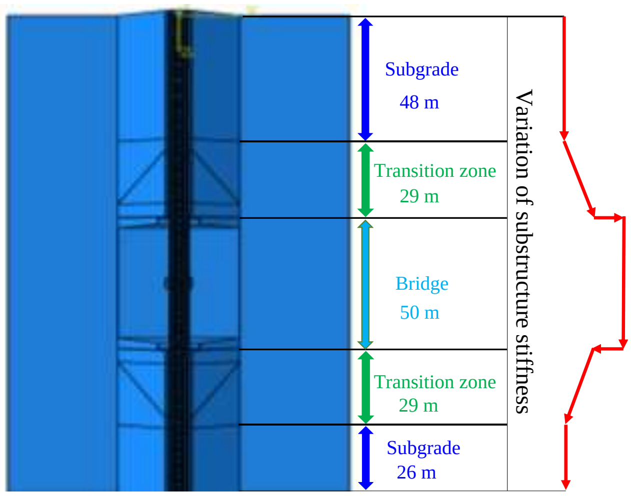


Fig. 4. Variation of substructure stiffness along railway line

It could be clearly seen that the vertical acceleration gradually increased with the increase of stiffness at transition zone, it had drastically change near transition zone-bridge area, and then decayed quickly to very low value. The variation of acceleration in longitudinal direction coincided well with the variation of substructure stiffness.
The drastic change of acceleration response could result negative effect on ride comfort and deteriorate the performance during the railway service period, suitable

and practical measures should be taken to deal with this problem.

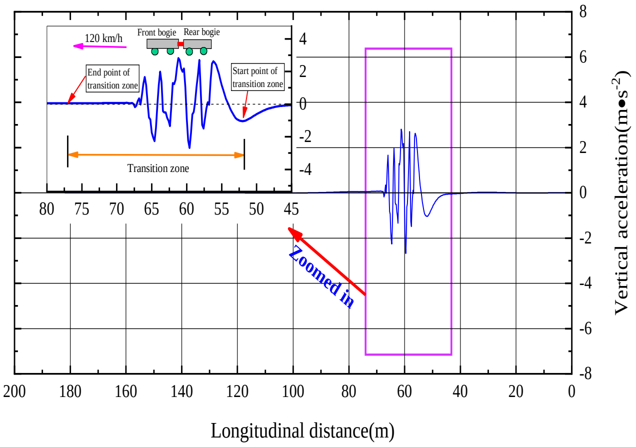


Fig. 5. Vertical acceleration response at 0.7069 s on capping layer top along railway line

4 CONCLUSION

A detailed three-dimensional model was developed for the dynamic behavior analysis of a coupled nonlinear track-subgrade-foundation on a heavy haul railway. The analyses resulted in the following conclusions:

(1) The movement of each pressure can agitate stress wave propagating in longitudinal, lateral and vertical directions simultaneously. Each pressure has an influence range, but the affected area is limited.

(2) Intensive stress concentration occurs near the transition zone and pier due to the great impedance difference.

(3) The longitudinal response of vertical acceleration varies in accordance with the change of substructure stiffness, abrupt jump is observed at the border of transition zone and bridge.

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