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The effect of asphaltic support layers on slab track dynamics

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1 Abstract

2 The deployment of asphaltic support layers (ASLs) within railway track structures has the potential to increase
3 track bending stiffness, assist moisture runoff and provide a platform for track construction. These merits have
4 increased its usage within the rail industry, however the understanding of asphaltic track dynamics during train
5 loading remains limited. Therefore, the primary aim and novelty of this study is the development of new
6 knowledge into the dynamic behavior of concrete slab track systems enhanced with asphaltic underlays. To do
7 so, a numerical simulation approach is used, comprised of two sub-models: 1) a coupled multi-body
8 vehicle–track model, for the purpose of computing wheel/rail forces; and 2) a 3D dynamic finite element track-
9 ground model to simulate stress wave propagation in the sub-structure. The models are validated using both
10 analytical results and field tests, and then used to simulate slab track systems with ASL thicknesses of: 0, 0.05,
11 0.07, 0.10, and 0.15 m. First the dynamic response at locations both near and far from the track joints are
12 compared to quantify the asphaltic foundation stresses, deflections, accelerations and strains. It is found that stress
13 concentrations occur near the concrete base joints and are an important consideration for ASL design. Next,
14 asphalt concrete durability at 400 km/h line speed is explored considering seasonal temperature variations and it
15 is found that the expected cumulative damage meets serviceability requirements. Finally, the influence of
16 different asphaltic layer thicknesses on reaction modulus is discussed, concluding that the optimal thickness range,
17 considering plastic deformation and construction constraints, is between 0.07 m and 0.10 m.

18 **Keywords:** Asphalt concrete; Asphaltic slab track; Vehicle–track coupled dynamics; Dynamic finite element
19 method; Railway asphalt vibration; Railway trackbed

ASL	Asphalt support layer	C_p	Damping of fastener
AUR	Upper roadbed underneath asphalt support layer	C_{BT}	Tangential coefficients of spring damping
ALR	Lower roadbed in asphalt track foundation	C_n	Courant number
AASHO	American Association of State Highway Officials	C_1, C_2	Positive constants in WLF equation
ASTB	Asphaltic slab track bed	C_s	Shear-wave velocity
CRTS III	China Railway Track System III	C_p	Compressive wave velocity
CRTS II	China Railway Track System II	C_{pz}	Primary suspension damping
DOF	Degree of freedom	C_{sz}	Secondary suspension damping
FEM	Finite element method	EI_Y	Rail bending stiffness to the Y -axis
HSR	High-speed rail	E_A	Elastic modulus of asphalt concrete
LR	Lower roadbed in conventional slab tracks (base model)	$E_s I_s$	Slab bending stiffness
NMS	Mode numbers of $X_n(x)$ selected for slab modelling	$E_b I_b$	Concrete base bending stiffness
NM	Order number of rail vibration mode	E_r	Elastic modulus of rail
NMB	Mode numbers of $X_n(x)$ selected for concrete base modelling	F_{rs}	Fastener force
SBV	Schwellen MIT Bitumen Verguss	G_{wr}	Constant concerning the wheel/rail contact condition
VFA	Voids filled with asphalt	H_b	Height of concrete base
WLF	Williams–Landel–Ferry	H_s	Height of track slab
$[C_v]$	System matrices for damping of vehicle	I_r	Area moment of rail
$[K_v]$	System matrices for stiffness of vehicle	J_t	Bogie pitch moment of inertia
$[M_v]$	System matrices for mass of vehicle	J_c	Car body pitch moment of inertia
$\{P_{wr}\}$	System force vectors of wheel/rail interaction	K_{BT}	Tangential coefficients of spring stiffness
$\{Z_v\}$	System displacement vectors of vehicle	K_{BN}	Normal coefficients of spring stiffness
$[\varepsilon]$	Threshold elastic strain	K_{sz}	Secondary suspension stiffness
$T_n(t)$	Generalized coordinate concerning time t	K_{30}	Subgrade reaction modulus
$X_n(x)$	Mode functions of the slab with x coordinates	k_{sc}	Stiffness per unit length of self-compacting concrete
$q(t)$	Generalized coordinate for the vertical motion of rail	K_p	Stiffness of fastener
$\delta(x)$	Dirac function	K_{pz}	Primary suspension stiffness
$\delta Z_j(t)$	Compression at the j^{th} contact point solved by vertical track irregularity, and the wheel and rail displacements	L_t	Axle spacing
B_s	Width of track slab	l_s	Spacing of fasteners
B_b	Width of concrete base	m_r	Mass per unit length of rail
C_{BN}	Normal coefficients of spring damping	n_0	Total number of rail fasteners mounted on a single slab
L_{\min}	Distance between two adjacent nodes on a moving load's path.	x_{wj}	Position coordinates of the j^{th} wheelset
L_v	Vehicle length	x_i	Position coordinates of i^{th} fastener point
L_b	Length of concrete base	α_N	Normal correction coefficient
L_c	Bogie spacing	α_T	Tangential correction coefficient
L_s	Length of a single slab	β_n	Frequency coefficient corresponding to $X_n(x)$
m_s	Track slab mass	λ_{\min}	The smallest wave length
M_t	Bogie mass	ε_r	Tensile strain at the base of ASL
M_w	Wheelset mass	ε_z	Compression strain on the surface of

			roadbed
M_c	Total mass of Car body and dead weight tonnage	ε_{zu}	Compression strain on AUR surface
N_a	Number of load applications required by fatigue damage	ε_{zl}	Compression strain on ALR surface
N_s	Number of loadings with respect to allowable permanent deformation	C	Constant related to VFA
N_i	Number of bogie passages in each season	G	Shear modulus
N_{si}	Allowable number of bogie passages defined for each season related to the deformation-included damage	K	Bulk modulus of the structural layers
N_{ai}	Allowable number of bogie passages defined for each season related to the fatigue damage	N	Total number of rail fasteners
P_0	Design axle load	P	Wheel/rail contact force
R_w	Wheel rolling radius	R	Distance from the boundary to the vibration source
T_{ref}	Reference temperature	T	Temperature
a_T	Shift factor	c	Wave velocity in a medium
c_s	Damping of cement asphalt mortar	β	Correction factor
c_{sc}	Damping per unit length of self-compacting concrete	ρ	Mass density
c_f	Damping per unit length of soil foundation	Δt	Fixed time increment
k_f	Stiffness per unit length of soil foundation		
k_s	Stiffness of cement asphalt mortar layer		

22 1. Introduction

23 Ballastless tracks are commonly used on high-speed lines due to their perceived higher performance and
24 reduced maintenance requirements compared to ballasted tracks [1–4]. The subgrade serves as a structural
25 foundation; however, it can be negatively affected by environmental factors (e.g., rainfall and humidity),
26 potentially resulting in frost heave [5,6], mud pumping [7], and other defects [8,9]. Furthermore, the subgrade is
27 typically formed from unbound material with minimal bending stiffness to spread the induced traffic loads.

28 As a solution, placing asphaltic support layers (ASL) within ballasted and ballastless track systems has
29 recently emerged as an attractive method to reduce substructure deformation by increasing bending stiffness and
30 waterproofing [10–12]. For example, asphalt mixtures, with densely graded aggregates and low air voids (1–3%),
31 can help to provide isolation properties and resistance against water damage [13–15]. Further, during track
32 construction, asphaltic layers provide a high-quality foundation for the operation of heavy plant, thus improving
33 track-bed construction quality. Despite the potential benefits of ASLs, there has been limited scientific research
34 to optimize their usage for railways. Although asphaltic materials are commonplace and well understood in the
35 field of highways, the dynamic loads and lifespans expected in a railway environment are significantly different.
36 For example, highway asphalt pavements lifespans are 15 to 20 years, whereas high-speed rail (HSR)
37 infrastructure is designed to last much longer, up to 100 years [16]. Similarly, highways typically experience
38 relatively low moving load speeds, excited by wavelength ranges that differ significantly from HSR. Further,
39 expansion joints between adjacent track slabs or concrete bases leads to stress concentrations in the track
40 foundations.

41 Over the last half century, asphalt-based materials have been introduced into existing and newly developed
42 railway systems. The primary application of asphaltic layers has been for ballasted track. For ballasted railway
43 systems, an asphalt underlayment serves as an additional structural layer between the ballast and the
44 subgrade/subballast, with the aim of reducing subgrade deterioration under the combined effects of train loading

45 and water seepage [17]. Early adopters of asphalt included Austria, Italy, America, and Japan, with tests of the
46 technology beginning in the 1960's [18]. The related long-term field evaluations, simulation calculations and
47 recent laboratory tests [19] have shown it to have a positive benefit on trackbed performance.

48 In addition to ballasted track applications, the asphaltic slab track bed (ASTB) system involves laying
49 asphalt directly under a concrete slab track. Several European countries and Japan first introduced ASTB to
50 construct HSRs and urban transit lines [20]. Varieties of asphaltic slab track beds have been proposed [14],
51 including RA-slab on the Sanyo and Tohoku Shinkansen lines in Japan [21] and Schwellen mit BitumenVerguss
52 (SBV) systems in Germany [22]. However, the widespread application of RA-slabs has been delayed in Japan
53 due to challenges regarding settlement and durability [21]. Further, the SBV system in Germany was also
54 ultimately superseded by system with asphaltic trackbed laid directly below the sleepers [22]. Considering the
55 potential benefits, yet the practical challenges of developing asphaltic slab tracks, currently there is the
56 opportunity to develop new knowledge into the behavior of the technology to overcome the current challenges.

57 In recent years, dynamic models have been proposed to simulate the interaction of between railway vehicles,
58 track and subgrade [23–25]. When studying subgrade dynamics, the FEM has been proposed to investigate the
59 effect of nonlinear material properties, irregular geometric shapes and structural discontinuities [26]. These
60 include Hall [27] who developed a 3D finite element track-soil model using ABAQUS, which was subject to
61 moving point loads, and the related works have been done by Connolly et al. [28,29], Shih et al. [30,31], Powrie
62 et al. [32] and Varandas et al. [33]. Alternatively, El Kacimi et al. [34] proposed a Fortran based 3D FE code in
63 the time domain, while Galvín and Domínguez [35] proposed a frequency domain solution. Kouroussis et al. [36]
64 combined a vehicle-track coupled system with a full 3D ground model to predict vibrations induced by railway
65 transportation. Auersch [37] and O'Brien & Rizos [38] also coupled a multi-body vehicle system with a 3D
66 FEM-BEM model to evaluate dynamic responses caused by moving trains. Alternatively, 2.5D methods [39–
67 46] and the Thin-Layer Method (TLM) [47–51] have been proposed to take advantage of domain geometry

68 invariances and thus improve computational efficiency. Based on the aforementioned approaches, [52–54] used
69 the finite element method to analyse asphalt railway substructures.

70 Considering this previous research, two shortcomings are common: 1) Many railway asphalt studies
71 simplify the excitation as a moving point load, thus ignoring train-track interaction, and thus of random track
72 irregularities. This is an important contributor to track dynamics; 2) Most studies have focused on the underlying
73 behaviour of continuous asphalt track structures, thus ignoring the effect of stress concentrations at the
74 longitudinal expansion joints between track slabs and bases. Again, this is an important factor for consideration
75 because this is where the largest dynamic stresses will occur.

76 Therefore, this work proposes a modified approach that is well-suited to simulating asphaltic railway track,
77 including train-track interaction and expansion joints. It combines a vehicle–track coupled dynamics model with
78 a 3D track–subgrade FE model to simulate the dynamic behavior of ASL within a concrete slab track system. In
79 the coupled dynamic model, random track irregularities are employed as system excitations to calculate the
80 wheel/rail forces. In the FE model, a refined track–subgrade system is proposed, capable of accounting for non-
81 linear material properties, specific geometries and structural discontinuities. A generalized Maxwell model in the
82 form of Prony series is adopted for describing the viscoelasticity of asphalt concrete. The primary novelty of the
83 paper is the use of the model to investigate the dynamic behavior of asphaltic support layers in the presence of
84 train-track interaction and slab joints. To do so, a slab track system model with five different thickness of ASL is
85 simulated. Considering the longitudinal discontinuity of the slab track structure, the stresses, deflections,
86 accelerations, and strains of three joint-dependent locations are analysed. Finally, the durability of asphalt
87 concrete under long-term railway operations is examined, and the implications on ASL design are discussed.

88 **2. Modelling framework**

89 **2.1 Modelling overview**

90 Vehicle–track vibrations are primarily induced by the irregular surfaces of wheels and rails. To consider

91 this interaction, a vehicle–track coupled dynamic model is used to calculate the wheel/rail forces. After that, to
92 evaluate the dynamic response of varying subgrades under train passage especially at joint-dependent locations,
93 a track–subgrade model should be capable of accounting for nonlinear material properties, detailed track
94 geometry, structural and discontinuities. To do so, this paper proposes a hybrid modelling strategy solved in the
95 time domain to meet these objectives. The two main modelling steps are shown in Fig. 1:

96 **Vehicle–track sub-model:** Calculates the wheel/rail force using vertical vehicle–track coupled dynamics
97 theory. A detailed model is developed for vertical interactions between railway vehicles and tracks. The moving
98 vehicle is modelled as a multi-body system with ten DOF's; the track substructure is simulated as an infinite Euler
99 beam supported on an elastic foundation consisting of the three layers: rail, track slab, and concrete base. The
100 vehicle and the track subsystems are coupled through wheel/rail interaction described using Hertzian non-linear
101 elastic contact theory. Random track irregularities are employed as system excitations via a time–frequency
102 transformation technique.

103 **Track–ground sub-model:** Calculates the dynamic response of the track–subgrade system using FEM in
104 the time domain. A 3D FE model is developed considering nonlinear material properties, track geometry, and
105 structural discontinuities. It is used to compute the subgrade's dynamic stress, deflection, and acceleration.
106 Nonreflective boundary conditions facilitate wave propagation towards the far-field area and prevent outward
107 propagating waves from reflecting into the domain of interest. The wheel/rail forces calculated from
108 Vehicle–track sub–model are inputs for the 3D FE track–ground sub–model.

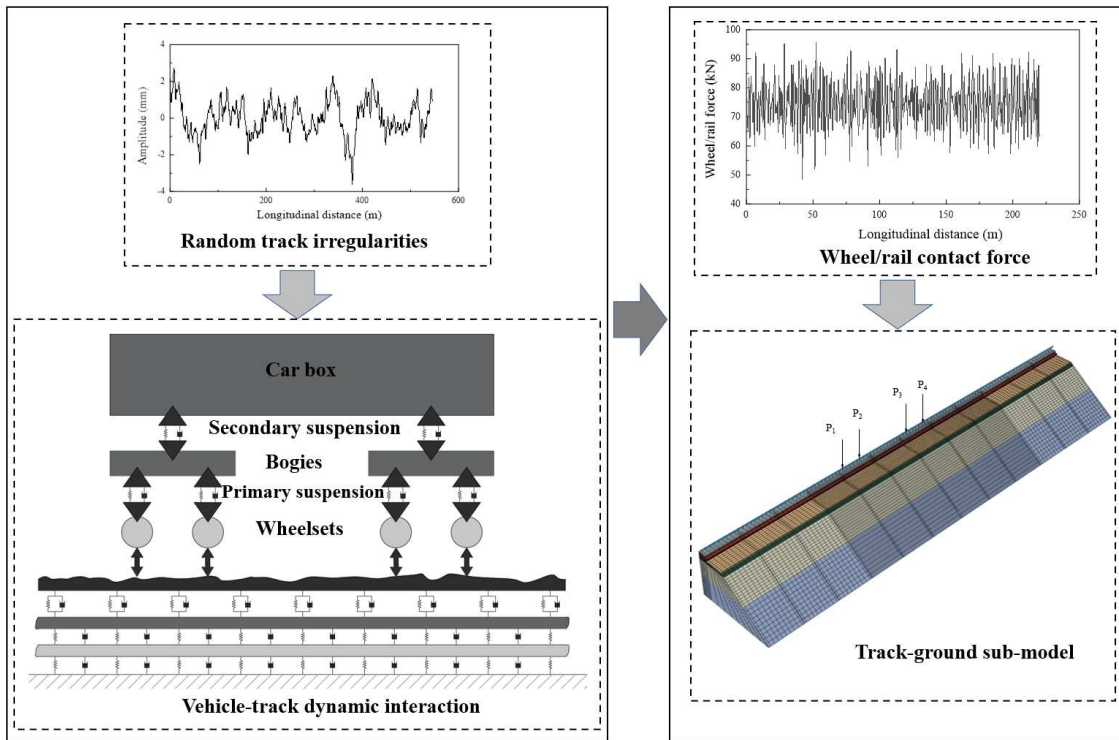


Figure 1. Model overview

2.2 Vertical vehicle-track coupled dynamics

2.2.1 Dynamic equations of motion

A vehicle-track vertically coupled dynamics model incorporating subgrade support is established to characterize the dynamic interactions between the vehicle, slab track structure, and foundation (subgrade), as shown in Fig. 2.

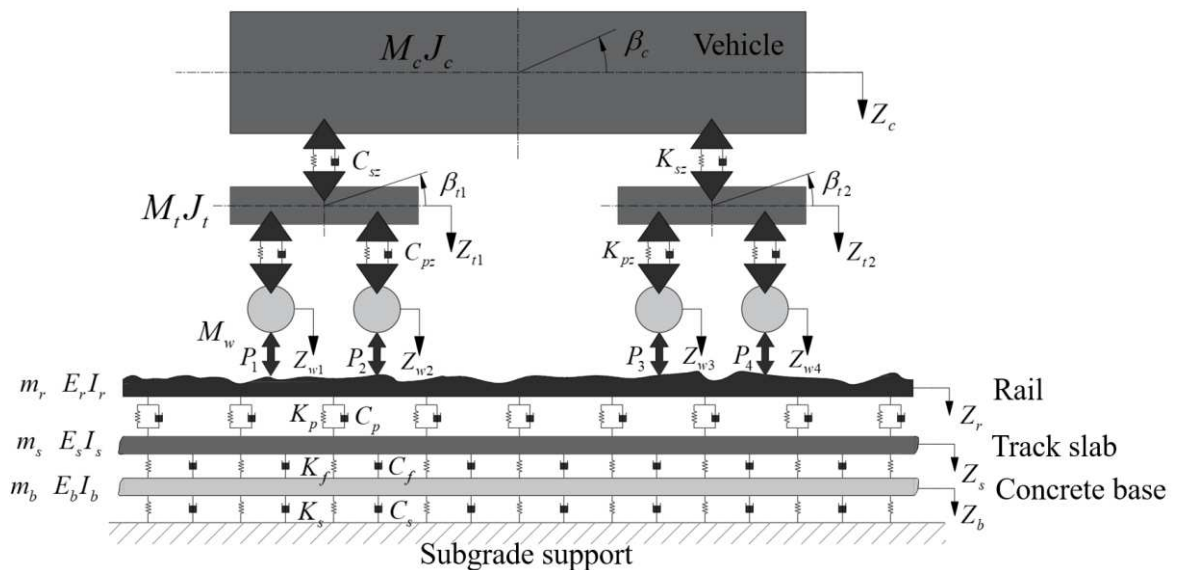


Figure 2. Vertical vehicle-track sub-model considering the subgrade

118 The vehicle consists of one car body, two bogie frames, and four wheelsets in a four-axle mass-spring-
 119 damper system. The primary and secondary suspensions characterized by spring-damping elements are
 120 employed to connect the structural components. The bogie frames and car body have two degrees of freedom
 121 (DOFs), which means vertical and pitch motions are considered. The wheelsets only have one DOF for vertical
 122 motion. The dynamic equation of the vehicle-track interaction model is derived in submatrix form as [23,55]:

$$123 \begin{bmatrix} \mathbf{M}_V & 0 \\ 0 & \mathbf{M}_T \end{bmatrix} \begin{Bmatrix} \ddot{\mathbf{X}}_V \\ \ddot{\mathbf{X}}_T \end{Bmatrix} + \begin{bmatrix} \mathbf{C}_V & 0 \\ 0 & \mathbf{C}_T \end{bmatrix} \begin{Bmatrix} \dot{\mathbf{X}}_V \\ \dot{\mathbf{X}}_T \end{Bmatrix} + \begin{bmatrix} \mathbf{K}_V & 0 \\ 0 & \mathbf{K}_T \end{bmatrix} \begin{Bmatrix} \mathbf{X}_V \\ \mathbf{X}_T \end{Bmatrix} = \begin{Bmatrix} \mathbf{F}_V \\ \mathbf{F}_T \end{Bmatrix} \quad (1)$$

124 where the subscripts 'V' and 'T' refer to the vehicle sub-system and track sub-system, respectively; $[\mathbf{M}]$, $[\mathbf{K}]$ and
 125 $[\mathbf{C}]$ are the system matrices for mass, stiffness, and damping, respectively; $\{\mathbf{F}\}$ and $\{\mathbf{X}\}$ are the system force and
 126 displacement sub-vectors, respectively. Nonlinear contact force is considered for the wheel/rail interactions.

127 Rail fasteners discretely support a finite-length Bernoulli-Euler beam that represents the rail. Following the
 128 Ritz method, the equation for dynamic rail motion can be expressed as [23,55]:

$$129 \begin{cases} \ddot{q}_k(t) + \frac{EI_Y}{m_r} \left(\frac{k\pi}{l} \right)^4 q_k(t) = -\sum_{i=1}^N F_{rsi}(t) Z_k(x_i) + \sum_{j=1}^4 P_j(t) Z_k(x_{wj}), k = 1 \sim NM \\ Z_r(x, t) = \sum_{k=1}^{NM} Z_k(x) q_k(t) \end{cases} \quad (2)$$

130 where the subscript 'k' refers to the fastener; $q(t)$ denotes the generalized coordinate for the vertical motion of the
 131 rail; EI_Y denotes the rail bending stiffness to the Y-axis; $Z_r(x, t)$ is the vibration displacement of rail; x_{wj} and x_i are
 132 the position coordinates of the j^{th} wheelset and i^{th} fastener point, respectively; P is the wheel/rail contact force; F_{rs}
 133 is the fastener force; N denotes the total number of rail fasteners; and NM represents the order number of the rail
 134 vibration mode.

135 The track slab is modelled as a simply supported Euler-Bernoulli beam on a concrete base, which is in turn
 136 supported by a nonlinear viscoelastic foundation. Following the Ritz method, the equations of dynamic motion
 137 for the track slab and concrete base are given as [23,55]:

$$\left\{ \begin{aligned}
\ddot{T}_{sn}(t) + \frac{E_s I_s L_s \beta_n^4}{m_s} T_{sn}(t) &= \sum_{j=1}^{n_0} \frac{F_{rsj}(t)}{m_s} X_n(x_j) - \frac{k_f}{m_s} [L_s T_{sn}(t) - \int_0^{L_s} X_{sn}(t) Z_b(x,t) dx] \\
&\quad - \frac{c_f}{m_s} [L_s \dot{T}_{sn}(t) - \int_0^{L_s} X_{sn}(t) \dot{Z}_b(x,t) dx], n = 1 \sim NMS \\
Z_b(x,t) &= \sum_{n=1}^{NMB} X_{bn}(x) T_{bn}(t)
\end{aligned} \right. \quad (3)$$

$$\left\{ \begin{aligned}
\ddot{T}_{bn}(t) + \frac{E_b I_b L_b \beta_n^4}{m_b} T_{bn}(t) &= \frac{k_f}{m_b} \left[\int_0^{L_b} X_{bn}(t) Z_s(x,t) dx - L_b T_{bn}(t) \right] + \frac{c_f}{m_b} \left[\int_0^{L_b} X_{bn}(t) \dot{Z}_s(x,t) dx - L_b \dot{T}_{bn}(t) \right] \\
&\quad - \frac{k_s}{m_b} L_b T_{sn}(t) - \frac{c_s}{m_b} L_b \dot{T}_{sn}(t), n = 1 \sim NMB \\
Z_s(x,t) &= \sum_{n=1}^{NMS} X_{sn}(x) T_{sn}(t)
\end{aligned} \right. \quad (4)$$

140 where $X_n(x)$ is the mode function of the slab with x coordinates; β_n is the frequency coefficient corresponding
141 to $X_n(x)$; $T_n(t)$ is the generalized coordinate concerning time t ; NMS and NMB are the mode numbers $X_n(x)$
142 for the slab and concrete base models respectively; $Z_s(x, t)$ and $Z_b(x, t)$ are the vibration displacement of track
143 slab and concrete base, respectively; $E_s I_s$ and $E_b I_b$ are the slab and base bending stiffness respectively; m_s and m_b
144 denote the track slab and concrete base mass respectively; k_f and c_f are the stiffness and damping of the cement
145 asphalt mortar layer between the track slab and base respectively; k_s and c_s are the stiffness and damping of
146 subgrade support respectively; L_s and L_b are the slab and base length respectively; and n_0 is the total number of
147 rail fasteners mounted on a single slab.

148 2.2.2 Vertical wheel/rail interaction

149 The vertical wheel/rail contact force determines the wheel/rail dynamic interaction. The derivation of the
150 Hertzian nonlinear contact theory leads to:

$$151 P_{wrj}(t) = \begin{cases} \left[\frac{1}{G_{wr}} \delta Z_j(t) \right]^{-3/2}, & \delta Z_j(t) > 0 \\ 0, & \delta Z_j(t) \leq 0 \end{cases} \quad (5)$$

152 where G_{wr} is the constant concerning the wheel/rail contact condition and $\delta Z_j(t)$ is the compression at the j^{th}
153 contact point, which is solved using the vertical track irregularity and the wheel and rail displacements.

154 2.2.3 Random track irregularities

155 Vehicle–track vibrations are primarily excited by the irregular surfaces of wheels and rails. Two main types
156 of geometric irregularities exist: 1) specific irregularities, such as the wheel flat, out-of-round wheel, dipped rail
157 joint, and rail corrugation, and 2) random irregularities, such as the roughness on the surfaces of wheels and rails,
158 which influence wheel–rail rolling noise and track geometry. Random track irregularities are inherent on all
159 railway lines and influence the dynamic performance of vehicle and track. This paper focuses on excitations due
160 to random track irregularities.

161 Random track irregularities can be described using power spectral density (PSD) as a function of spatial
162 frequency, of which there are various formulations. The formulation used in this work is based on TB/T 3352-
163 2014 (National Railway Administration of PRC), which divides the track into different classes to quantify track
164 unevenness [56]. To input the track spectrum into the vehicle–track system, the PSD function is transformed into
165 a rail geometry varying with the track’s longitudinal distance (or in the time domain) through a time–frequency
166 transformation technique [57].

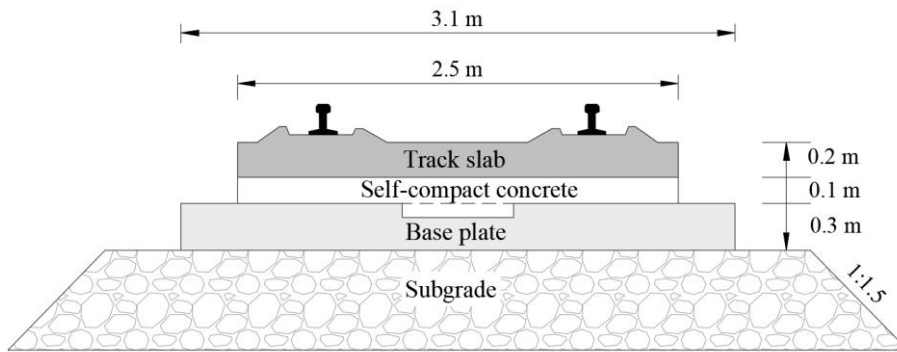
167 **2.3 FE Model**

168 A three-dimensional dynamic finite element model of a track structure was built using the commercial
169 software ABAQUS (version 2018) to perform comprehensive analyses of substructure mechanical behaviours
170 in a ballastless track system (CRTS III) under train loading. The train forces were calculated using the coupled
171 dynamics model in the previous computation step and then introduced into the FE model as an excitation.

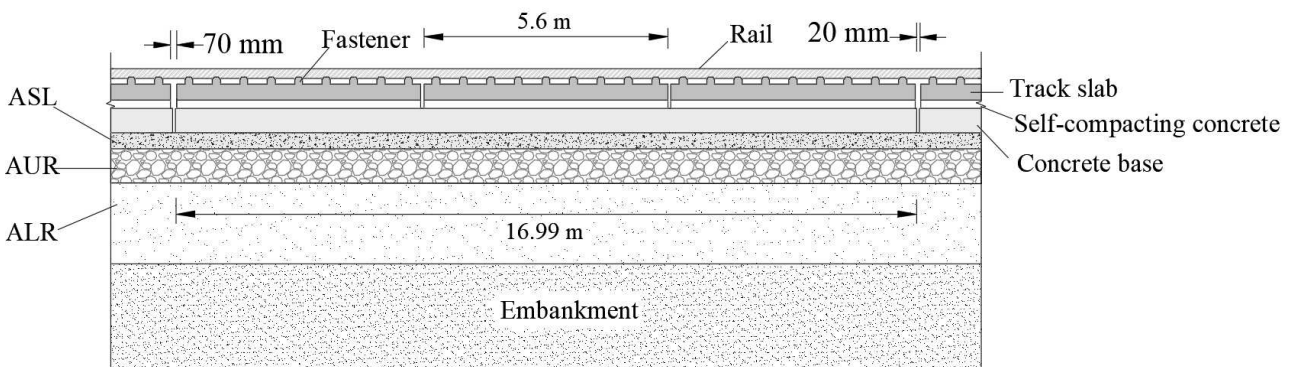
172 2.3.1 General description

173 The ballastless track under consideration was the CRTS III slab track system [16], as shown in Fig. 3. It
174 comprised UIC60 rail, WJ-8B fasteners, P5600 track slab, self-compacting concrete, concrete base, and subgrade.
175 The steel rail is modelled as a 2-node linear beam and the fasteners were deployed at a spacing of 0.63 m [58].
176 The standard CRTS III P5600 track slab measured 5.6 m in length, 2.5 m in width, and 0.2 m in thickness. The

177 self-compacting concrete layer has the same dimensions as the track slab and is 0.1 meters thick. The concrete
 178 base dimensions are 17.0 m long, 3.1 m wide, and 0.3 m thick. The ballastless track system has expansion joints
 179 between the slab and concrete base, and the expansion joints are 70 mm wide for the slab and 20 mm wide for
 180 the base. As shown in Fig. 3b, a single concrete base supports three track slabs. From the top down, the asphaltic
 181 track foundation consists of the ASL, asphaltic upper roadbed (AUR), asphaltic lower roadbed (ALR), and
 182 embankment. Note that the total thickness of ASL and AUR is 0.4 m, while the UR thickness is 0.4 m in
 183 conventional track foundations [16]. The thicknesses of the ALR and embankment are 2.3 m and 3.0 m,
 184 respectively. The subgrade slope gradient was 1:1.5.



(a)



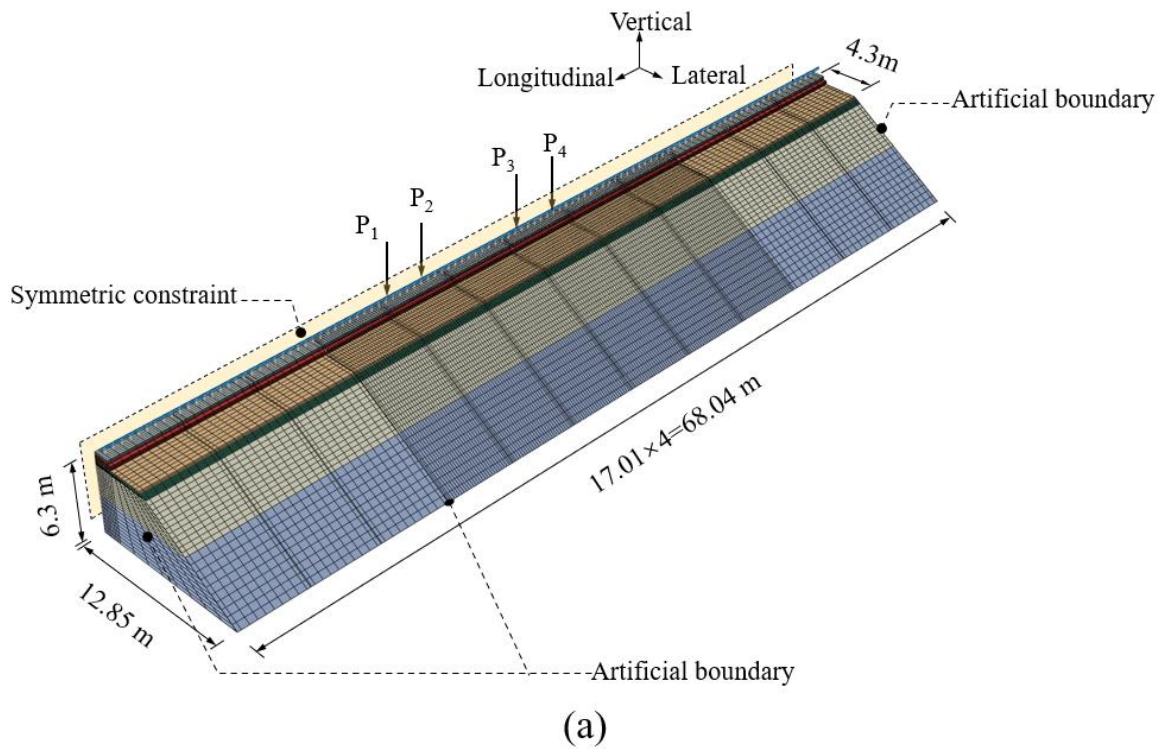
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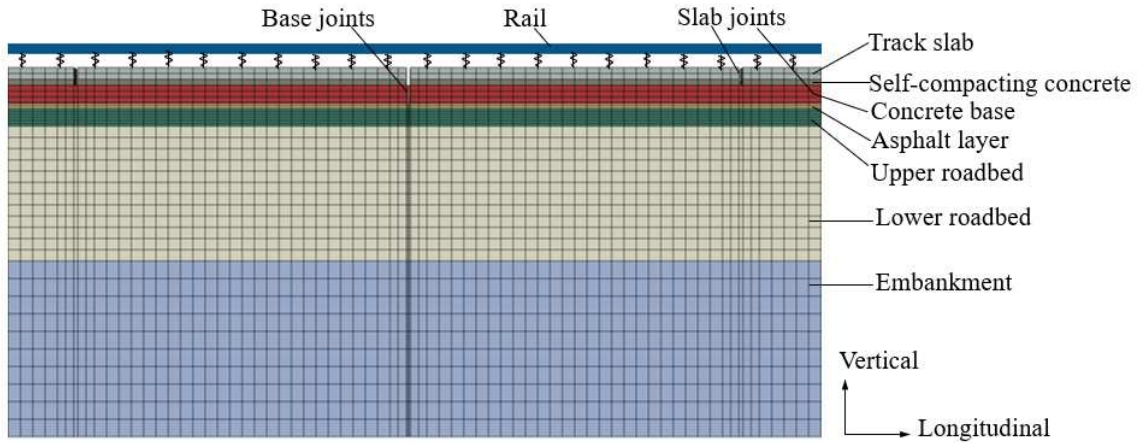
Figure 3. Components of the CRTS III ballastless slab track with expansion joints: (a) cross-section; (b) side

view

186
 187 The developed FE model (Fig. 4) shows four concrete bases totaling 68.04 m. The framework improves
 188 computational efficiency by taking the symmetrical half of the slab track system. CARTESIAN connectors
 189
 190

191 simulate fasteners with a vertical dynamic stiffness of 40 kN/mm and damping of $50 \text{ kN}\cdot\text{s}\cdot\text{m}^{-1}$. The track slab
192 and self-compacting concrete layer are merged into a composite structure by deploying a reinforcing mesh and
193 two rows of door-type modules [59]. A tie constraint is adopted between the track slab and self-compacting
194 concrete layers. In this instance, the tie constraint means that no relative displacement occurs between the two
195 contact surfaces. The self-compacting concrete layer and concrete base are in contact with a coefficient of friction
196 of 0.7 [60]. Because of the low roadbed stiffness and small surface dynamic deformation, a tie constraint is
197 applied to the concrete base and formation. Non-reflective boundary conditions are employed to facilitate wave
198 propagation towards the far-field and prevent the outward propagating waves from reflecting into the
199 computational domain [61].





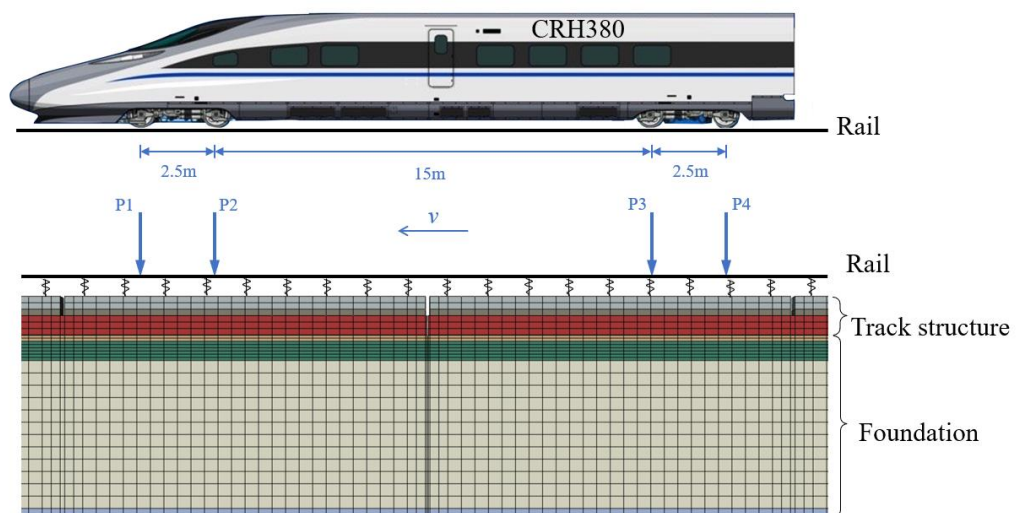
(b)

201

202 Figure 4. Three-dimensional FE model of the track–subgrade system: (a) general view; (b) elevation view

203 **2.3.2 Moving traffic loads**

204 A double-axle load pattern is used in the loading phase to ensure consistency between the two modelling
 205 techniques (Fig. 5). Similar to the coupled dynamics model, the passage of one car is considered in the dynamic
 206 FE analysis, corresponding to two bogies and four axles. The wheel/rail contact forces derived from the previous
 207 coupled dynamics steps are automatically submitted to a user subroutine DLOAD in ABAQUS, which exerts
 208 the concentrated moving force on the rail top based on its spatiotemporal distributions.

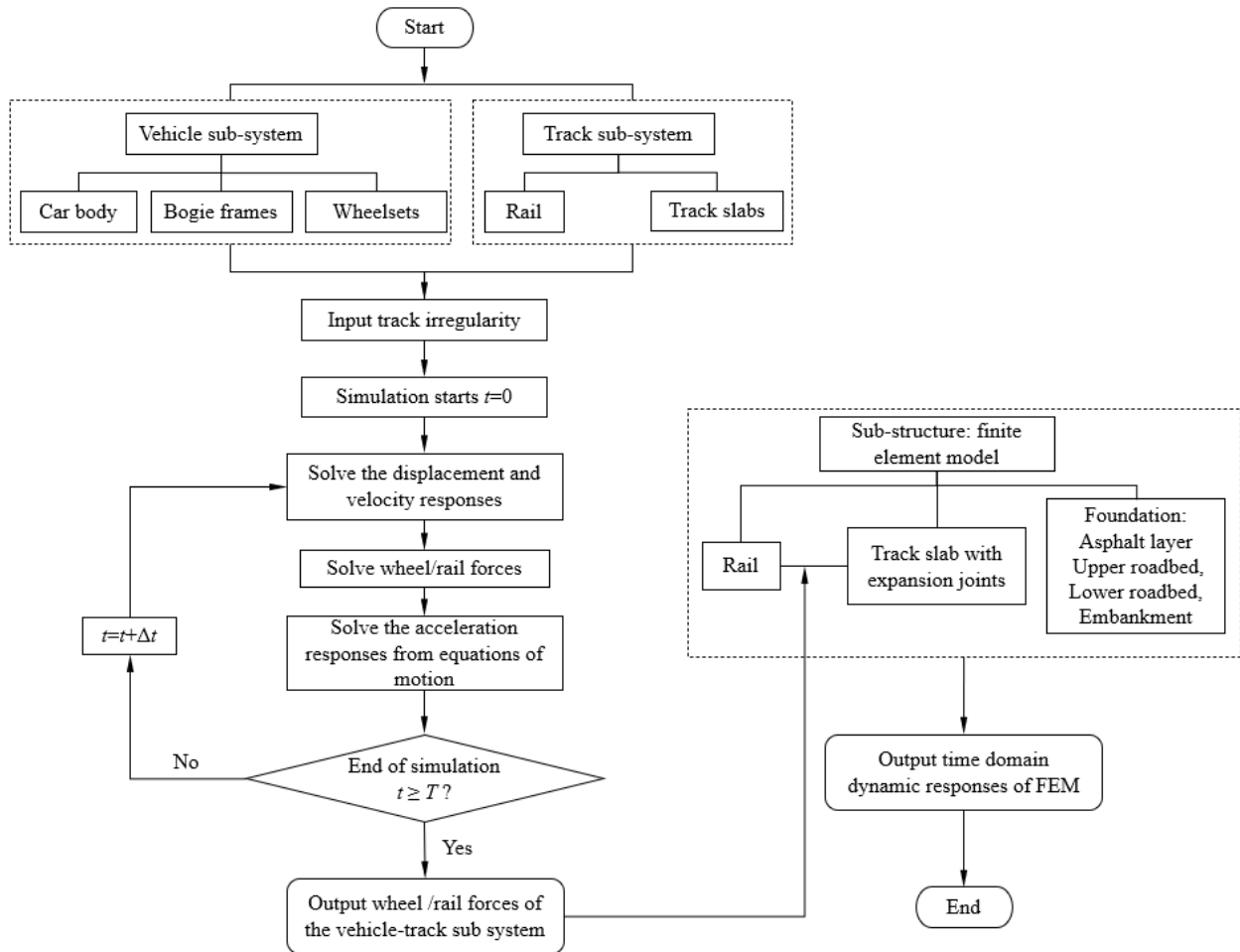


209

210 Figure 5. Configuration of moving train loads as inputs in FE analysis

211 The implementation steps of the dynamic simulation model are displayed in Fig. 6, where the two

212 subsystems are sequentially coupled. The vehicle-track coupled model is used as a first step: it provides the time
 213 history of the wheel/rail forces exerted on the rail, which is, in turn, used as inputs in a 3D FE model of the track
 214 foundation. The complete process is performed in the time domain.



215

216

Figure 6. Flowchart for the two-step modelling framework

217 3. Validation

218 The following describes two validations used to assess model accuracy. First, the dynamic stress on the
 219 subgrade surface of a base model (0 cm thick ASL) is validated by comparing the calculation results against that
 220 from Green's function. Then, the deflection and acceleration on the ASL surface are validated using measurement
 221 data from in-situ tests performed on the Zhengzhou–Xuzhou line in China.

222 3.1 Validation 1: Analytical solution without ASL

223 The two-step modelling framework was validated by comparing the calculation results against those
 224 computed using an alternative Green's function solution approach [62,63]. This alternative approach also divided
 225 the vehicle-track-subgrade system into two subsystems: the vehicle-rail subsystem and the slab-subgrade
 226 subsystem, which were coupled through fastener connections.

227 To perform the comparison, the CRH2 Electric Multiple Unit running at 300 km/h was considered
 228 (Appendix A). Table 1 lists the primary parameters for the ballastless track system. The total length of the rail
 229 infrastructure model is 306.18 m, and the vehicle runs a maximum distance of 200 m. The first 263 modes of
 230 vibration are used for the rail, track slab, and base, resulting in 789 DOFs in the track model.

231 Table 1. Calculation parameters for the CRTS III slab track

Components	Parameters	Symbols	Units	Values
Rail	Elastic modulus	E_r	$\text{N}\cdot\text{m}^{-2}$	2.1×10^{11}
	Area moment	I_r	m^4	3.09×10^{-5}
	Mass per unit length	m_r	$\text{Kg}\cdot\text{m}^{-1}$	60.8
Fastener	Stiffness	K_p	$\text{N}\cdot\text{m}^{-1}$	2.5×10^7
	Damping	C_p	$\text{N}\cdot\text{s}\cdot\text{m}^{-1}$	3.625×10^4
	Spacing	l_s	m	0.63
Track slab	Length	L_s	m	5.6
	Width	B_s	m	2.5
	Height	H_s	m	0.3
	Density	ρ_s	$\text{Kg}\cdot\text{m}^{-3}$	2,600
Concrete base	Length	L_b	m	16.99
	Width	B_b	m	3.1
	Height	H_b	m	0.3
	Density	ρ_b	$\text{Kg}\cdot\text{m}^{-3}$	2,500
Self-compacting concrete	Stiffness per unit length	k_{sc}	$\text{N}\cdot\text{m}^{-2}$	1.25×10^9
	Damping per unit length	c_{sc}	$\text{N}\cdot\text{s}\cdot\text{m}^{-2}$	8.3×10^4
Subgrade	Stiffness per unit length	k_f	$\text{N}\cdot\text{m}^{-2}$	1.7×10^8
	Damping per unit length	c_f	$\text{N}\cdot\text{s}\cdot\text{m}^{-2}$	1.5×10^5

232 According to the power spectral density (PSD) of track irregularities of HSRs specified in TB/T 3352-2014
 233 [56], the PSD function of the ballastless track spectrum of HSRs is piecewise fitted with a power function:

234

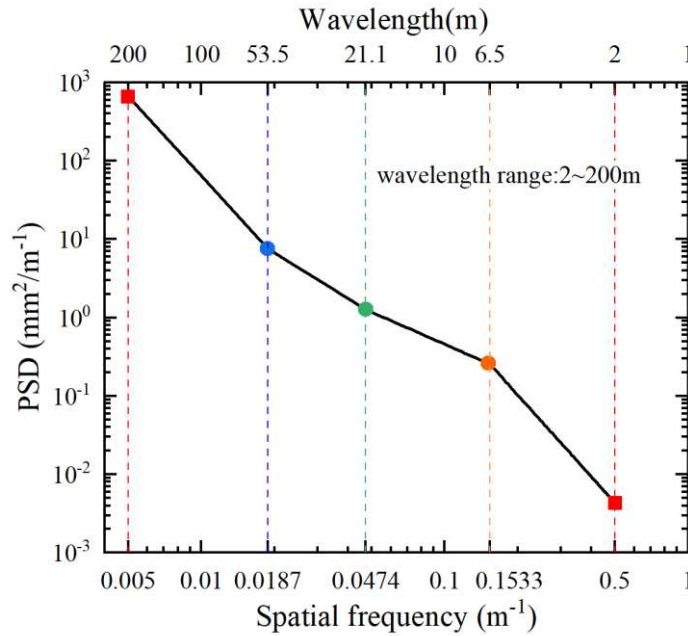
$$S(f) = \frac{A}{f^k} \tag{11}$$

235 where $S(f)$ is the PSD of track irregularities, as shown in Fig. 7; f is the spatial frequency; and A and k are the fit
 236 coefficients, as shown in Table 2.

237

Table 2. Values of A and k for the vertical ballastless track spectrum

Type	A	k
The first segment	1.0544×10^{-5}	3.3891
The second segment	3.5588×10^{-3}	1.9271
The third segment	1.9784×10^{-2}	1.3643
The fourth segment	3.9488×10^{-4}	3.4516



238

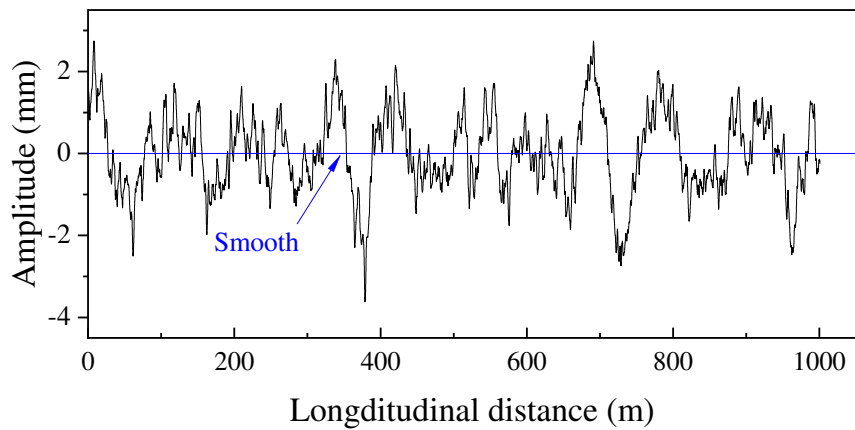
239

Figure 7. PSD of vertical ballastless track irregularities

240 The samples of time-domain track irregularities are generated using the inverse Fourier transform method.

241 The wavelength range of the simulation is from 1 to 200 m, and Fig. 8 displays the profile of the generated

242 moderate track irregularities.



243

244

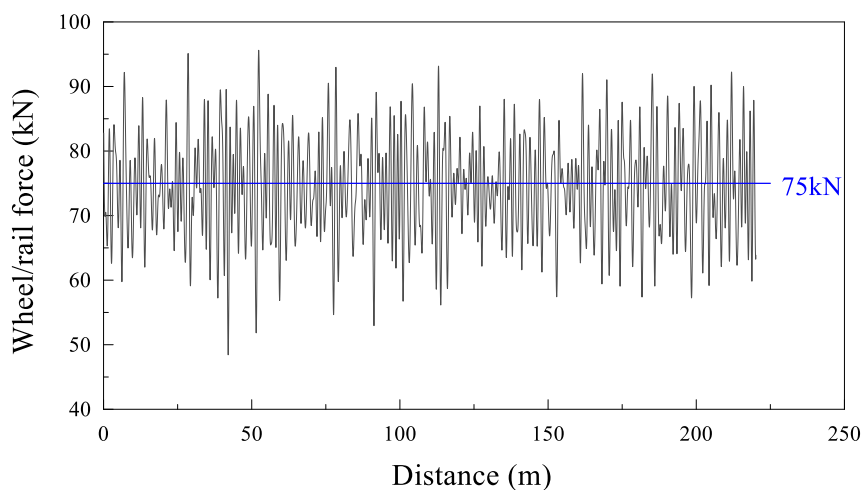
Figure 8. Profile of random track irregularities

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The dynamic responses of the slab track structure are obtained based on the vehicle-track vertically coupled dynamics model with random vertical track irregularities. At a speed of 300 km/h, the calculated wheel/rail contact forces considering track irregularity are shown in Fig. 9.



248

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Figure 9. Longitudinal distribution of the wheel/rail contact force (static wheel load = 75 kN, running speed = 300 km/h)

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Table 3 summarizes the material properties of the track components and formation layers, where the fills are classified per TB10621-2014 [16]. The constitutive relations of the rail, track slab, self-compacting concrete, and concrete base are described using linear elastic models. According to TB10621-2014, the subgrade reaction modulus (K_{30}) values for each layer are 190, 150, and 130 MPa/m. A linear elastic-perfectly plastic model describes the subgrade soil with the Mohr-Coulomb failure criterion, and its elastic modulus is determined per

256 the reported methods [64,65].

257 Table 3. Material properties of the FE track–subgrade model

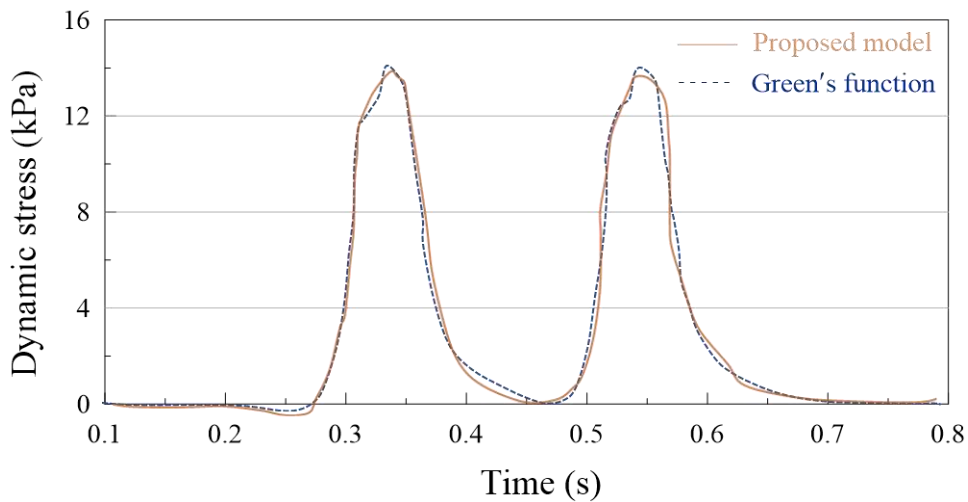
Component	Material	Bulk Density (kg/m ³)	Modulus (MPa)	Poisson's ratio	Damping ratio	Friction angle (°)	Cohesion (kPa)
Rail	Steel	7,830	210,000	0.300	0.01	/	/
Track slab	C60 concrete	2,600	36,000	0.167	0.03	/	/
Self-compacting concrete	C40 concrete	2,500	32,500	0.167	0.03	/	/
Concrete base	C40 concrete	2,500	32,500	0.167	0.03	/	/
Upper roadbed	Graded gravel	2,100	228.9	0.300	0.08	28	32
Lower roadbed	Class A/B fill (coarse)	2,050	186.0	0.350	0.07	25	26
Embankment	Class A/B/C fill	2,000	163.5	0.400	0.10	22	25

258 As shown in Fig. 10, the time histories of the monitored surface stress by the two approaches are comparable.

259 The monitoring point on the roadbed surface shows peak dynamic stresses of 13.8 kPa and 14.1 kPa for the

260 proposed model and Green's function, respectively. Their peak stress difference is less than 2.2%, confirming the

261 model accuracy in such circumstances.



262

263 Figure 10. Comparison of dynamic stresses from the proposed model and Green's function

264 3.2 Validation 2: ASL field tests

265 After performing a numerical validation, a validation using field tests was performed. To do so

266 measurement data was recorded on a test section of the Zhengzhou–Xuzhou railway line (CRTS III). ASL was

267 experimentally applied and has been in operation since 2016. The slab track structure incorporates a 0.1 m thick

268 ASL overlying the upper roadbed [54]. Fig. 11 shows the photos taken from the test section. In the construction
269 process, a monitoring system consisting of sensors, power supply systems, and a data acquisition unit were
270 installed in the test section to monitor the dynamic performance of ASL during operation.



(a)



(b)

271
272 Figure 11. Site photos of the instrumented rail line [66]: (a) asphaltic support layer; (b) integrated monitoring
273 systems near the shoulder

274 Regarding numerical simulation, the track response due to the passage of a CRH380 train was considered,
275 running at 350 km/h, and with an axle load of 170 kN (Appendix A). The primary parameters for the CRTS III
276 track system are the same as in Table 1 with track irregularity.

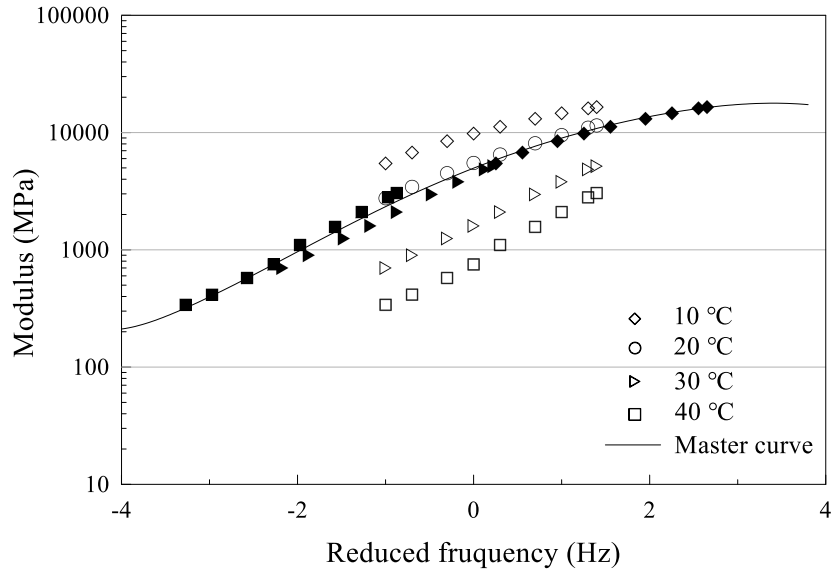
277 Asphalt concrete is a viscoelastic material whose dynamic behaviour is influenced by temperature and
278 loading frequency [67,68]. Therefore, the asphalt concrete model is carefully described in this study, given its
279 constitutive relation to the dynamic FE analysis. The viscoelasticity of asphalt concrete is characterized using the
280 generalized Maxwell model in the form of the Prony series [69], whose coefficients were determined by
281 extracting and testing in-situ samples:

282 1) Cores of asphalt concrete were drilled from test sections on the test railway line and subjected to dynamic
283 modulus testing using universal testing apparatus;

284 2) The master curve of the dynamic modulus was fitted using the Williams–Landel–Ferry Equation [70]
285 via the time–temperature superposition principle (Fig. 12);

286 3) The relaxation modulus was determined from the fitted master curves [71]. The least squares method

287 was then used to calculate the Prony series coefficients [69].



288

289

Figure 12. Master curve of asphalt concrete dynamic modulus [72]

290

Table 4 lists the coefficients of an eleventh-order Prony series for characterizing the constitutive relation of

291

the asphalt concrete. These coefficients were directly imported into ABAQUS to simulate the dynamic behaviour

292

of asphalt concrete. The fitting equation introduces a shift factor (a_T) to relate strains at different temperatures.

293

The shift factor is calculated by:

294

$$\text{Log } a_T = \frac{-C_1(T-T_{\text{ref}})}{C_2+(T-T_{\text{ref}})} \quad (6)$$

295

where log is the decadic logarithm, T_{ref} is the reference temperature, and C_1 and C_2 are positive constants

296

dependent upon the T_{ref} and material properties.

297

Table 4. Prony series coefficients for the asphalt concrete ($T_{\text{ref}} = 20\text{ }^\circ\text{C}$) [72,73]

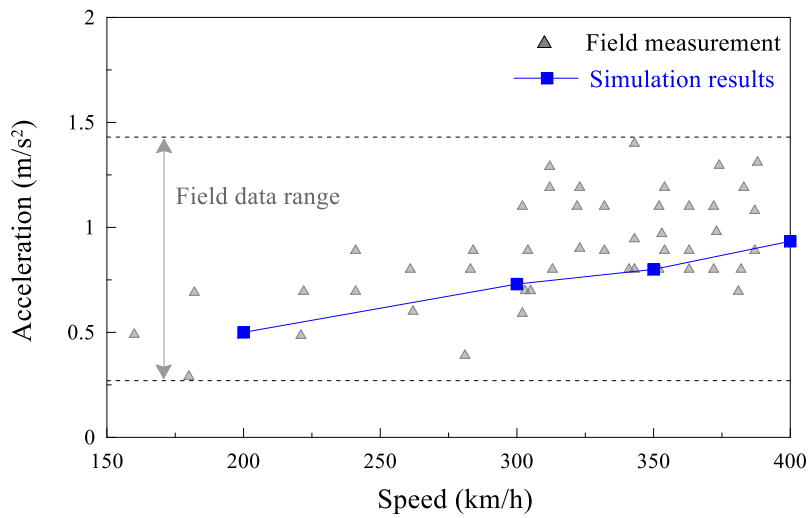
Serial number i	Relaxation time τ_i (s)	Prony series g_i	Serial number i	Relaxation time τ_i (s)	Prony series g_i
1	0.000 001	0.031 92	7	1	0.146 179
2	0.000 01	0.055 066	8	10	0.068 084
3	0.000 1	0.094 570	9	100	0.025 889
4	0.001	0.148 209	10	1,000	0.008 865
5	0.01	0.203 485	11	10,000	0.005 084
6	0.1	0.212 089			
C_1			11.7		
C_2			91.2		

298

Fig. 13 compares the field observations with simulation results of train-induced accelerations and

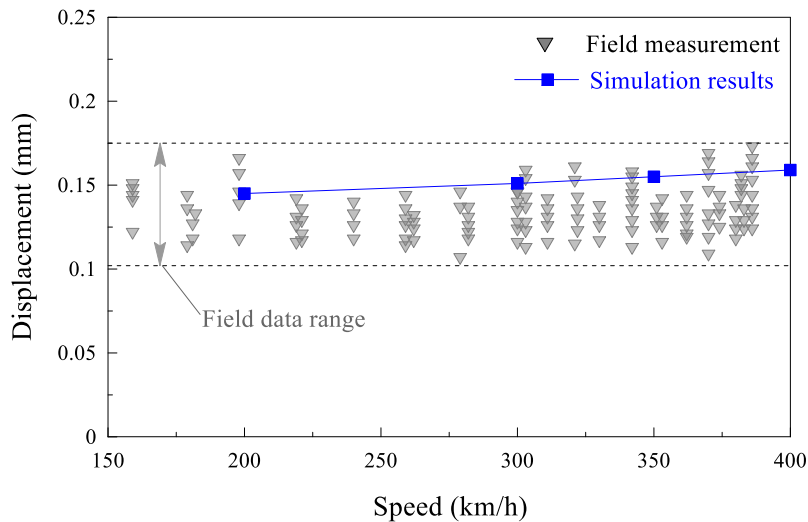
299

displacements at the surface of the ASL.



(a)

300



(b)

301

Figure 13. Field measurements and simulation results on top of ASL with varying operating speeds, (a)

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accelerations, and (b) displacements

303

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A relatively linear best-fit relationship can be observed in Fig. 13 between the acceleration levels and the

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running speed. In contrast, the speed has limited influence on the displacement of the upper ASL surface. Across

306

the speed levels, the measured displacements are scattered, ranging from 0.10 mm to 0.17 mm; while the

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variations in simulated values with operating speed are less significant, ranging from 0.115 mm to 0.125 mm.

308

The simulated values always fall within the range of in-situ measurements, suggesting that the simulation is

309 consistent with field observations. Considering the simulation models' performances in the two scenarios, it is
310 possible to conclude that the two-step modelling framework is practical for studying the dynamic responses of
311 ASL in slab track systems.

312 4. Results and analysis

313 Based on the validated two-step modelling approach, the dynamic behaviour of ASL's under moving train
314 loads is simulated and assessed to reveal the material's role in the vibration and mechanical responses of slab
315 track systems. Three representative positions (A, B, and C) are selected to evaluate the longitudinal structural
316 discontinuities of slab tracks, as shown in Fig. 14. Considering a CRH380 vehicle running on the CRTS III slab
317 track system, the model is developed with different ASL thicknesses: 0, 0.05, 0.07, 0.10, and 0.15 m. The ambient
318 temperature is set to 20 °C, and a train speed of 400 km/h and axle load of 170 kN are considered for all models.
319 To understand ASL behaviour, dynamic stresses and displacements are considered because they are critical
320 design indexes for the subgrade. Accelerations are studied because they are related to ride comfort, while tensile
321 and vertical strains are studied due to their relationship with asphalt concrete durability.

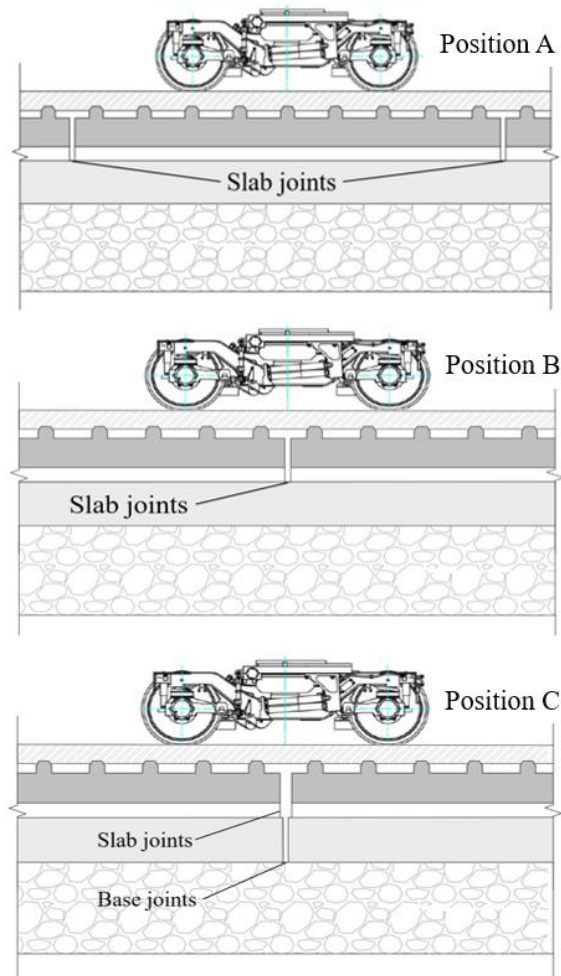


Figure 14. Three representative slab joint positions

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324 4.1 Dynamic stresses

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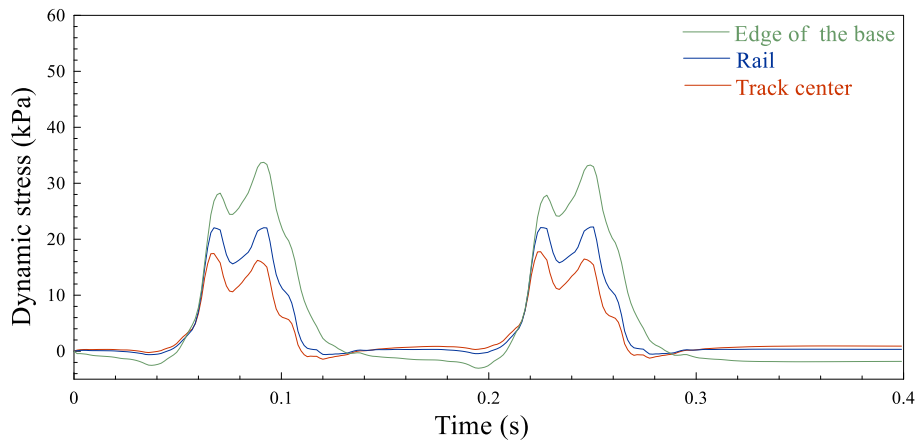
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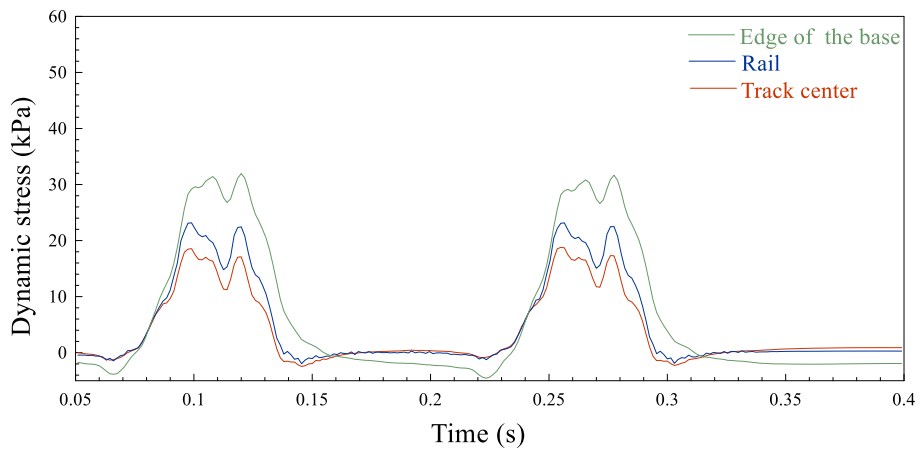
333

The ASL and base models exhibit a similar transverse stress distribution pattern, regardless whether the double-axle load is imposed on continuous (position A) or discontinuous (position B and position C) locations. From the symmetrical axis of structures to the slab edge, the train-induced stress is uniformly distributed in the transverse direction, consistent with field observations and previous findings [74,75], while a stress concentration occurs at the edges of concrete base. According to the approach in Ref. [76], the mean stress in each section can be calculated by the effective area and stress distribution on it. Taking a 0.05 m thick ASL as an example, Fig. 15 shows the dynamic stress histories at points A, B, and C on the ASL surface, while the stresses on the AUR surface are displayed in Fig. 16. Due to their similar pattern, only the average dynamics stress across the three lateral locations are shown in Fig. 16.



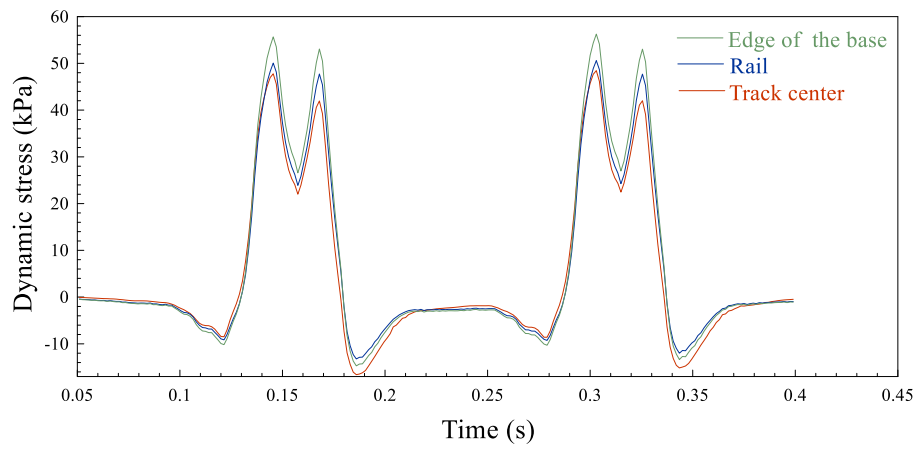
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(a)



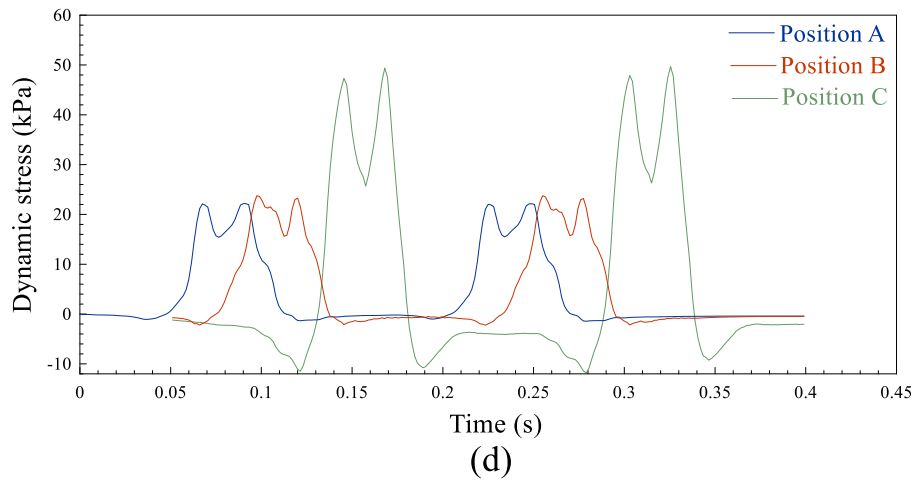
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(b)



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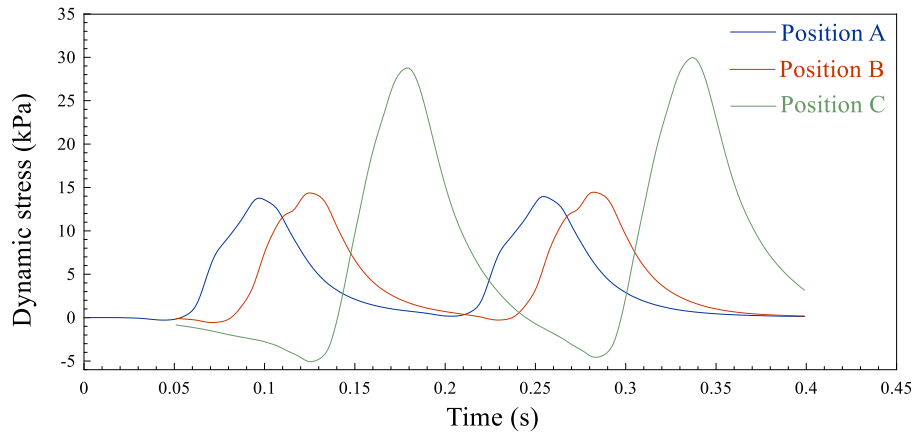
(c)



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Figure 15. Stress time histories on the upper surface of ASL: (a) position A; (b) position B; (c) position C;
 339 (d) averaged dynamic stress across three lateral locations (ASL thickness = 0.05 m)



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Figure 16. Stress time histories on the surface of the upper roadbed (ASL thickness = 0.05 m)

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Comparing the stress response at the ASL and AUR surfaces, the variations in stress levels across different loading points have a similar pattern. The induced stress peaks are largest at point C, followed by points B and A. Note that the stress difference between points A and B is minimal, while the stress peaks at point C are substantially higher than at the other locations. The presence of expansion joints reduces the track structure stiffness, resulting in a stress concentration below the longitudinal discontinuities. The expansion joints between adjacent concrete bases contribute more to the surface stress than those between track slabs.

348

349

The calculated stress peaks on ASL, AUR and ALR, accounting for the layer thickness and loading positions, are listed in Table 5.

350

Table 5. Dynamic stress peaks on the surface of ASL, AUR, and ALR

Thickness of ASL (m)	Stress on surface of ASL (kPa)			Stress on surface of AUR (kPa)			Stress on surface of ALR (kPa)		
	Position A	Position B	Position C	Position A	Position B	Position C	Position A	Position B	Position C
0	/	/	/	16.14	17.05	33.40	13.90	14.01	18.42
0.05	22.17	23.75	49.67	13.94	14.48	28.48	13.30	13.36	17.36
0.07	24.25	25.66	54.91	13.82	14.29	26.63	13.09	13.12	17.14
0.10	28.62	29.02	63.86	13.52	13.89	23.98	12.77	12.76	16.79
0.15	30.74	33.98	70.12	12.51	12.70	21.98	12.15	12.11	15.92

351

Increasing the thickness of ASL decreases the corresponding stress peak on the surface of AUR and ALR.

352

While the peak stress levels on the surface of the subgrade with ASL are higher than those of the base model (no

353

ASL), the peak stress levels increase with the increasing thickness of ASL. As the modulus of asphalt concrete

354

is higher than that of graded gravel, the stiffness of the ASL is greater than that of upper roadbed, leading to

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higher stress on the surface of the subgrade. In the same way, the bending stiffness of ASL increases with

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increasing ASL thickness, leading to higher stress levels.

357

4.2 Deflections of roadbed surface

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Fig. 17 shows the dynamic displacement time histories on top of a 0.05 m thick ASL at the three different

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monitoring positions. The minimum displacement occurs at position A, slightly lower than the peak at position

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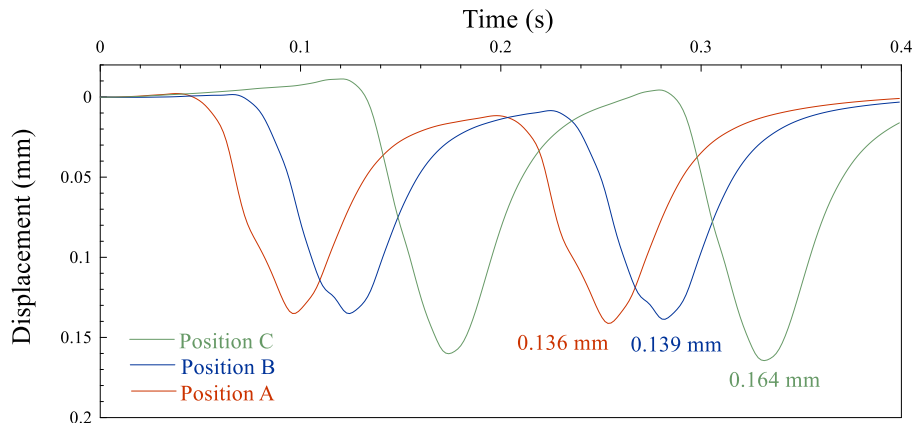
B. In contrast, the maximum displacement occurs at position C, which is significantly higher than the former two.

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The displacement patterns are comparable to those observed for the dynamic stress time histories across different

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positions.



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Figure 17. Deformation time histories of three longitudinal positions (ASL thickness = 0.05 m)

365 The peak dynamic displacement on the ASL surface against the ASL thickness is provided in Table 6.

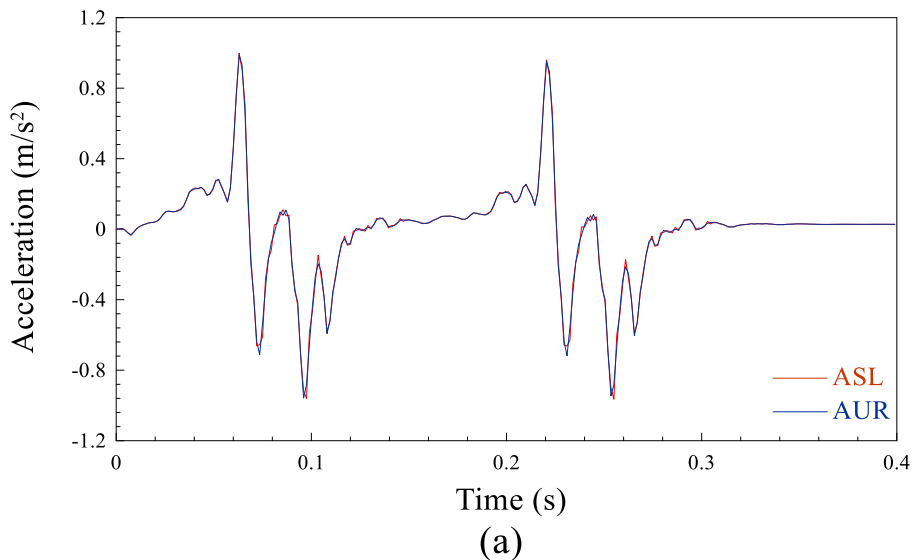
366 Table 6. Dynamic displacement on the surface of ASL

Thickness of ASL (m)	Displacement on surface of ASL (mm)		
	Position A	Position B	Position C
0	0.139	0.142	0.173
0.05	0.136	0.139	0.164
0.07	0.134	0.136	0.162
0.10	0.132	0.133	0.159
0.15	0.124	0.125	0.149

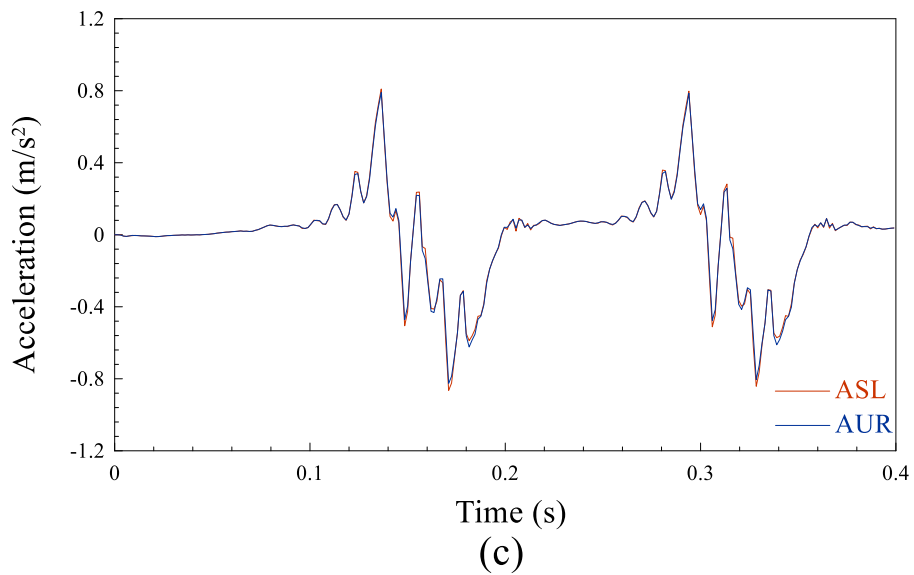
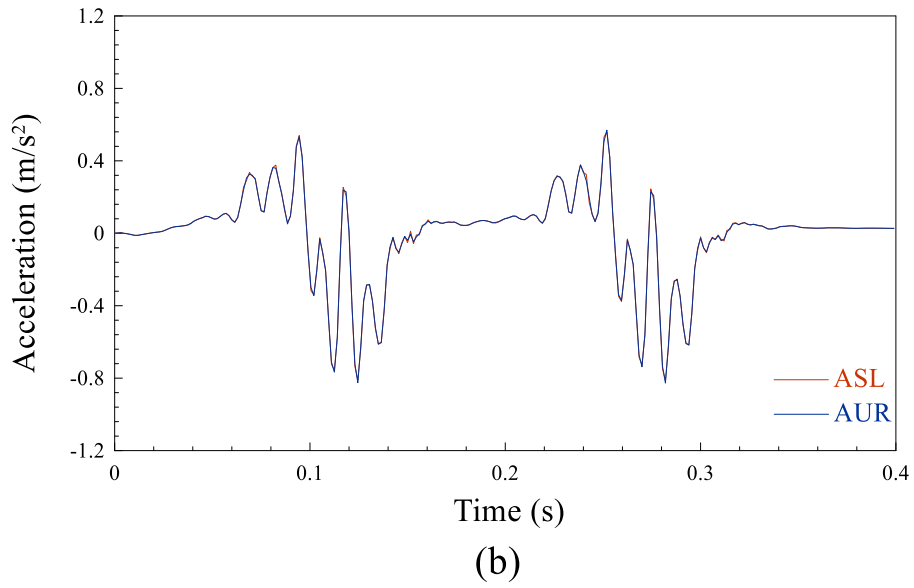
367 The displacement peak occurs at position C for a specific ASL thickness, and an insignificant difference is
368 observed between positions A and B. Under longitudinal discontinuities, the calculated maximum displacements
369 satisfy the specification requirements (0.22 m in slab tracks) suggested in [16]. As the thickness of ASL increases,
370 all displacement peaks across different longitudinal locations decrease.

371 4.3 Accelerations

372 Fig. 18 plots the acceleration time histories on the surface of the 0.05 m thick ASL and AUR, in which the
373 acceleration responses are compared between three positions. At each specific longitudinal location, the
374 acceleration responses (magnitude and phase) on the two structural layers of the asphaltic track system are similar.
375 Considering the different locations, the maximum acceleration magnitude occurs at position A, followed by
376 positions B and C. The maximum acceleration amplitude is observed in the continuous part of the structure
377 (position A).



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Figure 18. Acceleration time histories of ASL and AUR underneath three longitudinal positions: (a) position A, (b) position B, (c) position C (ASL thickness = 0.05 m)

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The relationship between vertical acceleration amplitudes and ASL thickness are presented in Table 7, considering the longitudinal positions and base model. In asphaltic track systems, the trends at three positions are similar and manifest as acceleration amplitude decreasing with increasing ASL thickness. The introduction of ASL decreases the acceleration amplitude on the roadbed surface compared to the base model, which demonstrates the vibration damping effect of the reinforced track structure.

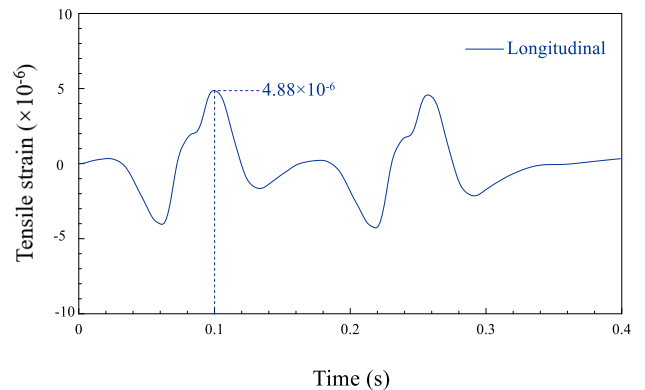
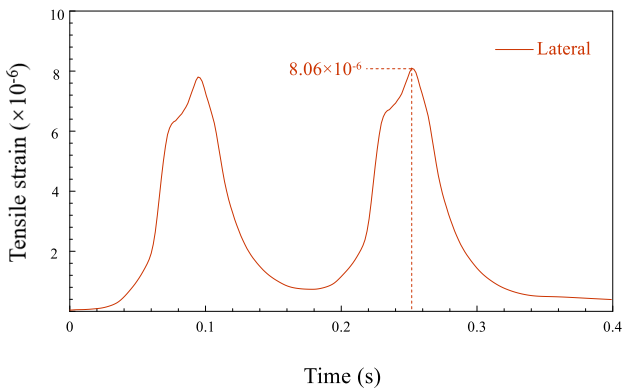
Table 7. Acceleration on the surface of ASL

Thickness of ASL (m)	Acceleration on surface of ASL (m/s ²)
----------------------	----------------------------------------------------

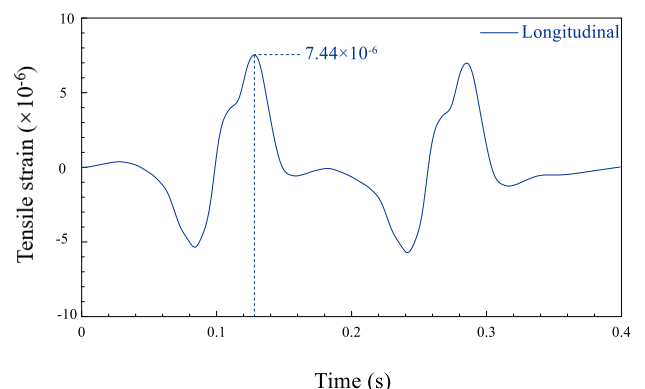
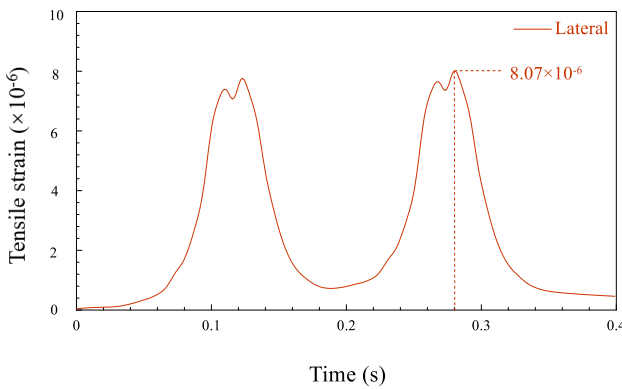
	Position A	Position B	Position C
0	1.025	0.882	0.816
0.05	0.960	0.840	0.729
0.07	0.955	0.833	0.723
0.10	0.938	0.818	0.702
0.15	0.895	0.785	0.644

389 **4.4 Tensile and vertical strains within ASL**

390 The longitudinal and transverse tensile strains at the bottom of the 0.05 m thick ASL are displayed in Fig.
 391 19. The lateral tensile strain remains positive during the loading–unloading phase, indicating that the asphalt
 392 concrete is constantly subjected to tensile loads in the lateral direction as one vehicle travels through. During the
 393 passage of the double-axle load, the longitudinal tensile strain varies from a negative value to a positive value.
 394 Unlike the lateral response, the bottom of asphalt concrete experiences compression and tension; each strain peak
 395 identified from the time history curves corresponds to the centre of one bogie (x-axis: time domain to the spatial
 396 location).

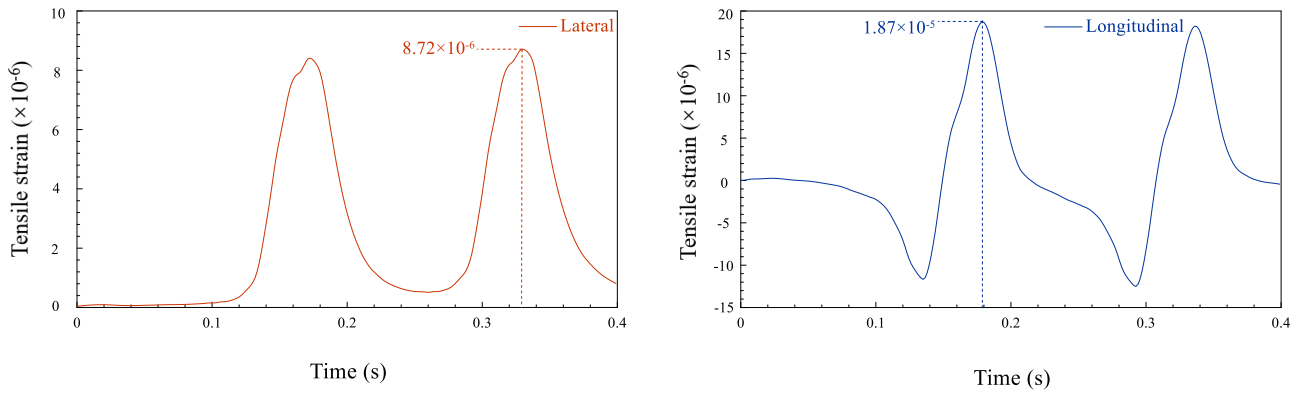


(a)



(b)

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(c)

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Figure 19. Lateral and longitudinal tensile strain at the bottom of ASL: (a) position A, (b) position B, (c) position C (ASL thickness = 0.05 m)

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Table 8 demonstrates the variation in lateral and longitudinal tensile strains with ASL thickness, in which the strain responses between the three locations are compared. Under the same thickness, lateral and longitudinal tensile strains are at a minimum at position A, while they are at a maximum at position C. The expansion joints located between the concrete bases result in a marked increase in the tensile strains at the ASL base. At position C, the longitudinal tensile strain is significantly higher than the lateral tensile strain. When assessing the impact of layer thickness on the strain response, one observes that the tensile strain decreases with increasing ASL thickness in the lateral direction. At the same time, it increases with an increasing layer thickness in the longitudinal direction. The thickness effect is more significant on the strain response at position C than at positions A and B. Since the longitudinal tensile strain is critical to ASL fatigue life (serviceability), increasing the ASL thickness is not conducive to asphalt durability.

Table 8. Tensile strains on the surface on the bottom of ASL

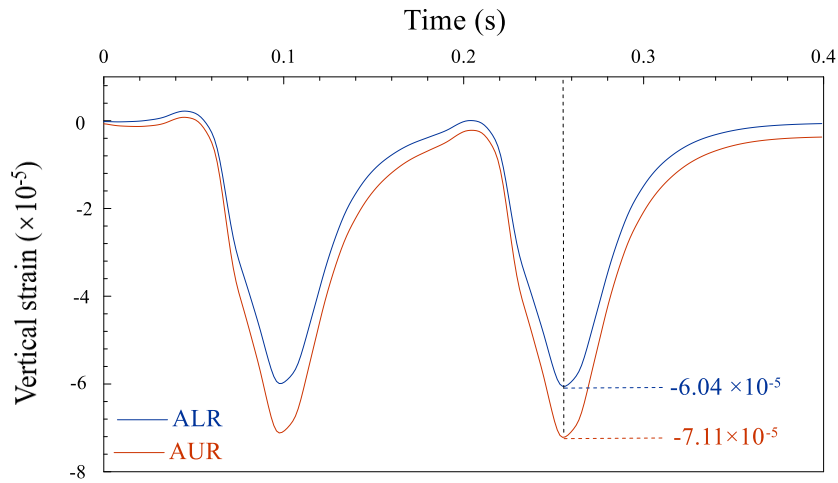
Thickness of ASL (m)	Lateral tensile strains on surface of ASL			Longitudinal tensile strains on surface of ASL		
	Position A	Position B	Position C	Position A	Position B	Position C
0.05	8.06×10^{-6}	8.07×10^{-6}	8.72×10^{-6}	4.88×10^{-6}	7.44×10^{-6}	1.87×10^{-5}
0.07	7.81×10^{-6}	7.82×10^{-6}	8.43×10^{-6}	5.11×10^{-6}	7.57×10^{-6}	2.14×10^{-5}
0.10	7.49×10^{-6}	7.52×10^{-6}	8.2×10^{-6}	5.38×10^{-6}	7.64×10^{-6}	2.32×10^{-5}
0.15	7.02×10^{-6}	7.06×10^{-6}	7.83×10^{-6}	6.11×10^{-6}	7.76×10^{-6}	2.57×10^{-5}

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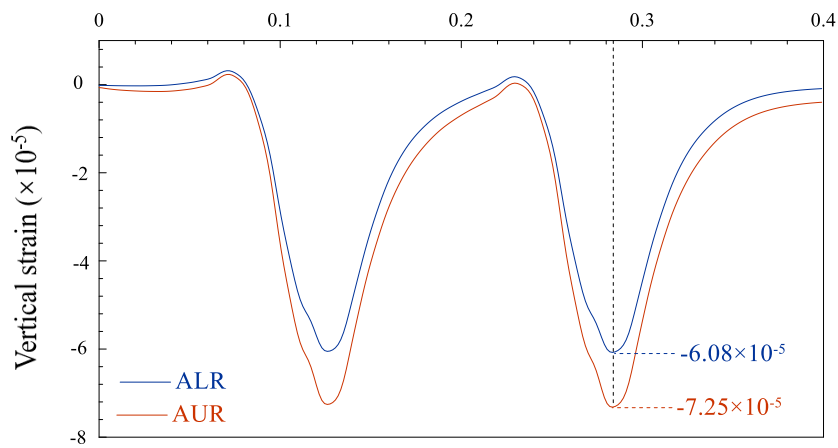
For an ASL thickness of 0.05 m, Fig. 20 shows the time histories of vertical strains on top of AUR and ALR, in which the responses between the three positions are compared. Table 9 compares the vertical strain on

415 top of AUR and ALR under various ASL thicknesses. The maximum vertical strain occurs at position C. On top
416 of AUR and ALR, the vertical strains decrease with thicker ASL, suggesting that the use of ASL reduces the
417 strain levels in the roadbeds, especially in the upper layer.



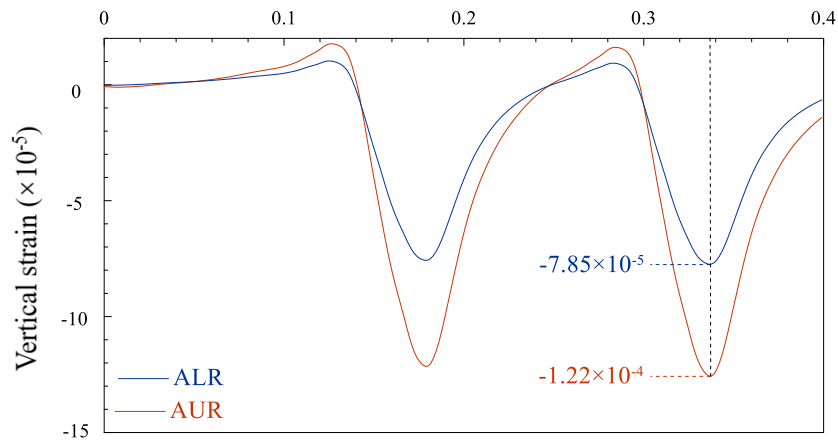
418

(a)



419

(b)



420

(c)

Figure 20. Vertical strain on the surface of the upper and lower roadbeds: (a) position A, (b) position B, (c) position C (ASL thickness = 0.05 m)

Table 9. Vertical strains on the surface of AUR and ALR

Thickness of ASL (m)	Vertical strains on surface of AUR			Vertical strains on surface of ALR		
	Position A	Position B	Position C	Position A	Position B	Position C
0	-9.38×10^{-5}	-9.66×10^{-5}	-1.72×10^{-4}	-6.20×10^{-5}	-6.24×10^{-5}	-7.95×10^{-5}
0.05	-7.11×10^{-5}	-7.25×10^{-5}	-1.22×10^{-4}	-6.04×10^{-5}	-6.08×10^{-5}	-7.85×10^{-5}
0.07	-6.63×10^{-5}	-6.75×10^{-5}	-1.12×10^{-4}	-6.02×10^{-5}	-6.05×10^{-5}	-7.84×10^{-5}
0.10	-6.09×10^{-5}	-6.17×10^{-5}	-9.86×10^{-5}	-5.93×10^{-5}	-5.93×10^{-5}	-7.74×10^{-5}
0.15	-5.43×10^{-5}	-5.48×10^{-5}	-9.14×10^{-5}	-5.55×10^{-5}	-5.57×10^{-5}	-7.26×10^{-5}

5. Design considerations for ASL

5.1 Asphalt concrete durability

According to the design method proposed by the Asphalt Institute [77], the horizontal tensile strain (ϵ_t) at the base of the ASL and the compressional strain (ϵ_z) on the surface of the gravel roadbed (AUR) are indicators for evaluating the asphalt's fatigue cracking potential and the capacity to resist permanent deformation.

Although both indicators are commonly used in asphalt pavement design, the stress paths induced under moving train loads differ from highway environments. For example, considering highway structures, the asphalt pavement carries the tire load directly, while for railways the train load is distributed through the rails, sleeper, and track superstructure before reaching the ASL surface. Therefore, the multilayered elastic theory commonly used in pavement design does not apply for calculating the tensile strains at the surface of the ASL, or the compressional strains at the surface of the railway AUR. To overcome this, this study obtains the strain level of ASL under moving train loads through FE track-ground modelling, based on which fatigue cracking and permanent deformation are checked. It should be noted that asphalt trackbed mix design is typically similar to that for highway asphalt pavements. Therefore although the strain parameters of ASL are calculated by the FE method, a highway pavement approach is used for evaluating the durability of ASL.

The primary goal of examining asphalt fatigue is to ensure the number of loadings during its service life doesn't exceeded the allowable number of loadings concerning fatigue damage, which is governed by the tensile

441 strain at the base of ASL. An empirical formula is proposed [77] for the fatigue testing of asphalt concrete:

442
$$N_a = 18.4 \times C \times 6.167 \times 10^{-5} \varepsilon_r^{-3.291} E_A^{-0.854} \quad (8)$$

443 where N_a is the number of load applications required by fatigue damage; ε_r is the maximum tensile strain at the
444 base of ASL; E_A is the elastic modulus of asphalt concrete (MPa); and C is a constant related to the voids filled
445 with asphalt (VFA), determined by:

446
$$C = 10^{4.84(\text{VFA}-0.6875)} \quad (9)$$

447 The value of VFA can be prescribed using the Marshall mix design method: 65% - 75% for heavy traffic
448 (equivalent number of standard axles $> 10^6$), 65% - 78% for medium traffic (equivalent number of standard axles
449 between 10^4 and 10^6), and 70% - 80% for light traffic (equivalent number of standard axles $< 10^4$).

450 Local deformation governs the structural deformation of ASL, meaning vertical strains should be controlled
451 within a permissible range to prevent cracks developing. According to the experimental data provided by the
452 American Association of State Highway Officials (AASHO), the number of loadings concerning allowable
453 permanent deformation (N_s) is calculated by:

454
$$N_s = 1.365 \times 10^{-9} \varepsilon_z^{-4.477} \quad (10)$$

455 where ε_z is the compressional strain on top of AUR. ASL of rail infrastructure is underlain by AUR, comparable
456 to the base and subbase layers of a road pavement structure. Note that because the surface layer of an asphalt
457 pavement carries vehicle loads directly, the strength of base and subbase materials is generally higher than that
458 of AUR and ALR. To control the permanent deformation of ASL, it is recommended that two indicators are
459 used: the compressional strain on the AUR surface (ε_{z1}) and the compression strain on the ALR surface (ε_{z1}).

460 The elastic modulus of asphalt concrete is influenced by ambient temperature, meaning the degree of fatigue
461 damage can be estimated by summing it over a desired number of seasons. As an illustration, the ambient
462 temperature is assumed as 20 °C for spring and autumn, 5 °C for winter, and 40 °C for summer to set the thermal
463 conditions in FE submodels. To satisfy the durability requirements, the cumulative degree of fatigue damage and

464 degree of deformation-induced damage over the design service life should be less than 1.0. The values of m_a and
465 m_s are obtained by:

$$466 \quad m_a = \sum_{i=1}^4 m_{ai} = \sum_{i=1}^4 \frac{N_i}{N_{ai}} \quad (11a)$$

$$467 \quad m_s = \sum_{i=1}^4 m_{si} = \sum_{i=1}^4 \frac{N_i}{N_{si}} \quad (11b)$$

468 where m_a and m_{ai} are the degrees of fatigue damage per year and season respectively; m_s and m_{si} are the degree of
469 deformation-induced damage per year and season respectively; subscript $i = 1, 2, 3,$ and $4,$ corresponding to four
470 seasons in one year; N_i is the number of bogie passages in each season; and N_{ai} and N_{si} are the allowable numbers
471 of bogie passages defined for each season, related to the fatigue and deformation-induced damage respectively.
472 N_{ai} and N_{si} are fixed values if the design service life has been determined. The calculated durability indexes of
473 asphalt material for different ASL thicknesses (0.05, 0.07, 0.10, 0.15 m) are provided in Appendix B.

474 Assuming a 100-year service life for slab tracks and an annual loading of 1 million cycles, the cumulative
475 fatigue and deformation-induced damage for all ASL thicknesses is less than 1.0. This indicates that the long-
476 term durability of ASL satisfies serviceability requirements. For the increased thicknesses, the stresses on the
477 AUR and ALR surfaces decrease, which is beneficial to the deformation-related durability (cumulative degree
478 decreases from 0.319 to 0.123). Alternatively, the tensile strain at the base of the ASL increases with increasing
479 thickness, resulting in elevated fatigue risk (cumulative degree increases from 0.196 to 0.622).

480 **5.2 Roadbed design**

481 It's good practice that cumulative plastic deformation should not develop in the slab track roadbeds of
482 railway lines [78], and the strains should not exceed the threshold defined in specifications [16]. In the case of
483 upper and lower roadbeds, the relationship between the elastic threshold ($[\varepsilon]$) for coarse fills and the K_{30} (MPa/m)
484 value can be expressed as [65]:

$$485 \quad [\varepsilon] = 0.28K_{30} + 107 \quad (12)$$

486 where K_{30} is modulus of subgrade reaction (MPa/m); ε is strain for coarse fills (10^{-6}); $[\varepsilon]$ means the elastic

487 threshold (10^{-6}) for coarse fills corresponds to a certain value of K_{30} . According to the K_{30} plate load test, the
 488 elastic modulus of coarse fills can be obtained by [78,79]:

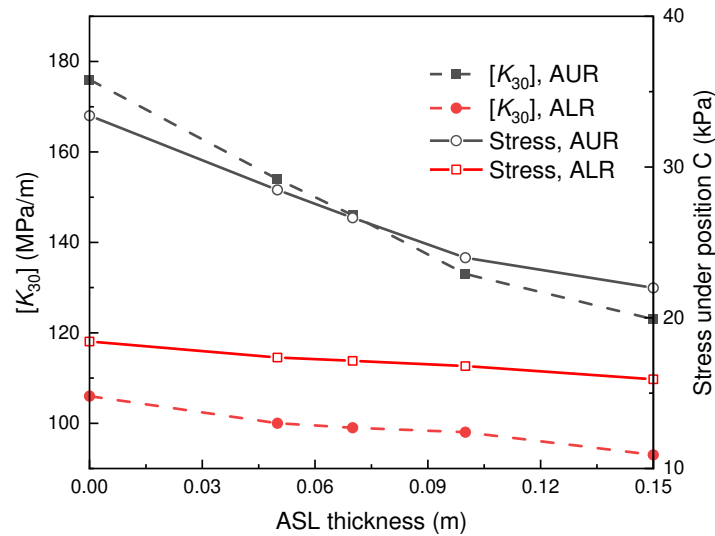
$$489 \quad E = 0.225K_{30}\beta \quad (13)$$

490 where β is the correction factor (m), which can be determined using established approaches [78,79]. Given a
 491 specific value of K_{30} , the threshold strain and stress can be calculated for coarse fills, and the results are
 492 summarized in Table 10.

493 Table 10. Threshold stress of coarse fills in roadbeds varying with K_{30} values

K_{30} (MPa/m)	Elastic modulus (MPa)	Threshold strain ($\times 10^{-6}$)	Threshold stress (kPa)
190	228.72	160.2	36.64
180	218.23	157.4	34.35
170	207.59	154.6	32.09
160	196.80	151.8	29.87
150	185.85	149	27.69
140	174.73	146.2	25.55
130	163.46	143.4	23.44
120	152.01	140.6	21.37
110	140.39	137.8	19.35
100	128.60	135	17.36
90	116.62	132.2	15.42

494 Based on the simulation results of AUR and ALR, the worst-case scenario of additional stress at different
 495 positions is likely to be directly below the expansion joints between adjacent concrete bases (position C).
 496 Combining the relationship between threshold stress and K_{30} (Table 10) with the ASL thickness–stress relation
 497 (Table 5, position C), it is possible to back-calculate the threshold [K_{30}], with varying ASL thickness, as shown
 498 in Fig. 21 (linear interpolation is used between discrete data points).



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Figure 21. Effect of ASL thickness on stress level and back-calculated $[K_{30}]$ of AUR and ALR.

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Fig. 21 indicates that increasing the ASL thickness leads to a decrease in the design value of K_{30} for AUR and ALR, and the decrease is more prominent for AUR. In conventional slab tracks, $[K_{30}]$ is approximately 176 MPa/m in the upper roadbed and 106 MPa/m in the lower roadbed. As per [16], the design K_{30} value should be larger than 190 MPa/m in upper roadbed and 130–150 MPa in LR. If we consider a reduction factor of 0.75 for fills under immersed conditions (worst-case scenario), the actual K_{30} may decrease to 142.5 MPa < 176 MPa/m in upper roadbed [80]. The trial calculations in Fig. 21 reveal that a minimum ASL thickness of 0.07 m is necessary for preventing cumulative plastic deformation in the AUR if immersed conditions are involved. In addition, asphalt concrete construction techniques typically require the paving thickness of single layers to be less than 0.1 m. If the ASL thickness exceeds 0.1 m, double-layer paving technology is usually used, resulting in a substantial increase in construction costs [72]. Therefore the optimum ASL thickness is likely to lie within 0.07–0.10 m for slab tracks.

512

6. Conclusions

513

514

The application of ASL for slab track systems was studied using a two-step modelling approach validated using analytical outcomes and field measurements. To investigate slab track dynamics with ASL, track system

515 models with different ASL thicknesses were developed and a conventional slab track was used as a base model.
516 The stresses, deflections, accelerations and strains within both continuous and discontinuous track structures were
517 analysed. Then the durability of asphalt concrete under seasonal temperature variations was examined and the
518 optimal design thickness for ASL discussed.

519 It was found that the application of ASL within slab track systems can improve the mechanical performance
520 of the supporting foundation. Increasing asphaltic thickness decreases the corresponding stress, deflection,
521 acceleration, and strain peaks on the surface of the AUR and ALR. While the dynamic stresses on the surface of
522 ASL and longitudinal tensile strain at the bottom of ASL increase with the increasing ASL thickness, which is
523 not conducive to the mechanical performance of ASL itself. The reduced track bending stiffness at expansion
524 joints causes stress concentrations below the longitudinal discontinuities, resulting in stress, deflection, and strain
525 peaks significantly higher than that at locations positions further away from the joints.

526 ASLs meet the durability requirements for a 100-year service life considering the thickness range under test:
527 0.05–0.15 m. As the thickness increases, the maximum compressional strains in the upper and lower roadbeds
528 decrease, while the longitudinal tensile strain at the base of the ASL increases. Considering plastic deformation
529 and construction constraints, ASL thicknesses between 0.07 m and 0.10 m appear suitable for slab tracks. Further
530 investigation of ASL assessments should incorporate the contribution of thermal regimes through coupled
531 thermomechanical simulations and advanced asphalt concrete constitutive models.

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536 **Declaration of Competing Interests**

537 None.

538 **Appendix A**

539 The calculation parameters for the CRH2 vehicle are provided in Table A1.

540 Table A1. Calculation parameters for the CRH2 vehicle

Parameters	Symbols	Units	Values
Car body mass	M_c	kg	45,600
Car body pitch moment of inertia	J_c	kg·m ²	2.231×10 ⁶
Bogie mass	M_t	kg	3,200
Bogie pitch moment of inertia	J_t	kg·m ²	1,752
Wheelset mass	M_w	kg	2,000
Primary suspension stiffness	K_{pz}	N·m ⁻¹	2.352×10 ⁶
Primary suspension damping	C_{pz}	N·s·m ⁻¹	3.92×10 ⁴
Secondary suspension stiffness	K_{sz}	N·m ⁻¹	1.982×10 ⁶
Secondary suspension damping	C_{sz}	N·s·m ⁻¹	1.96×10 ⁴
Bogie spacing	L_c	m	17.5
Wheel distance	L_t	m	2.5
Wheel rolling radius	R_w	m	0.43

541 The calculation parameters for the CRH380 vehicle are provided in Table A2.

542 Table A2. Calculation parameters for the CRH380 vehicle

Parameters	Symbols	Units	Values
Car body mass + dwt	M_c	kg	34,934+8,000
Car body pitch moment of inertia	J_c	kg·m ²	1.7118×10 ⁶
Bogie mass	M_t	kg	3,300
Bogie pitch moment of inertia	J_t	kg·m ²	1,807
Wheelset mass	M_w	kg	1,780
Primary suspension stiffness	K_{pz}	N·m ⁻¹	1.176×10 ⁵
Primary suspension damping	C_{pz}	N·s·m ⁻¹	1.0×10 ⁴
Secondary suspension stiffness	K_{sz}	N·m ⁻¹	2.4×10 ⁵
Secondary suspension damping	C_{sz}	N·s·m ⁻¹	2.0×10 ⁴
Bogie spacing	L_c	m	17.5
Wheel distance	L_t	m	2.5
Design axle load	P_0	kN	170
Vehicle length	L_v	m	25
Wheel rolling radius	R_w	m	0.43

543 **Appendix B**

544 The calculation results of the durability check for asphalt material under four ASL thicknesses are provided
545 in Tables B1, B2, B3, and B4, where four seasons are differentiated. ε_{r1} and ε_{r2} are the lateral and longitudinal
546 tensile strain at the base of ASL, respectively; ε_r is the horizontal tensile strain, which equals $\max\{\varepsilon_{r1}, \varepsilon_{r2}\}$; N_{ai} is
547 the allowable number of bogie passages defined for each season related to fatigue damage; m_{ai} is the degree of
548 fatigue damage per season; ε_{zu} and ε_{zl} are the compression strain on the AUR surface and on the ALR surface,
549 respectively; ε_z is the maximum of ε_{zu} and ε_{zl} ; N_{si} is the allowable number of bogie passages defined for each
550 season related to deformation-included damage; m_{ai} and m_{si} are the degree of fatigue deformation-induced
551 damage per season, respectively; m_a and m_s are the degree of fatigue damage and deformation-induced damage
552 per year, respectively.

553 Table B1. Essential parameters for durability with ASL thickness of 0.05 m

	Spring	Summer	Autumn	Winter
ε_{r1}	8.72×10^{-6}	1.33×10^{-5}	8.72×10^{-6}	8.40×10^{-6}
ε_{r2}	1.87×10^{-5}	1.91×10^{-5}	1.87×10^{-5}	1.85×10^{-5}
ε_r	1.87×10^{-5}	1.91×10^{-5}	1.87×10^{-5}	1.85×10^{-5}
N_{ai}	5.16×10^8	4.84×10^8	5.16×10^8	5.32×10^8
m_{ai}	4.85×10^{-4}	5.17×10^{-4}	4.85×10^{-4}	4.70×10^{-4}
ε_{zu}	1.22×10^{-4}	1.54×10^{-4}	1.22×10^{-4}	1.21×10^{-4}
ε_{zl}	7.85×10^{-5}	8.74×10^{-5}	7.85×10^{-5}	7.86×10^{-5}
ε_z	1.22×10^{-4}	1.54×10^{-4}	1.22×10^{-4}	1.21×10^{-4}
N_{si}	4.52×10^8	1.60×10^8	4.52×10^8	4.78×10^8
m_{si}	5.53×10^{-4}	1.56×10^{-3}	5.53×10^{-4}	5.23×10^{-4}
m_a	0.00196			
m_s	0.00319			

554 Table B2. Essential parameters for durability with ASL thickness of 0.07 m

	Spring	Summer	Autumn	Winter
ε_{r1}	8.43×10^{-6}	1.31×10^{-5}	8.43×10^{-6}	8.04×10^{-6}
ε_{r2}	2.14×10^{-5}	2.28×10^{-5}	2.14×10^{-5}	2.12×10^{-5}
ε_r	2.14×10^{-5}	2.28×10^{-5}	2.14×10^{-5}	2.12×10^{-5}
N_{ai}	3.32×10^8	2.69×10^8	3.32×10^8	3.45×10^8
m_{ai}	7.53×10^{-4}	9.30×10^{-4}	7.53×10^{-4}	7.25×10^{-4}
ε_{zu}	1.12×10^{-4}	1.46×10^{-4}	1.12×10^{-4}	1.11×10^{-4}
ε_{zl}	7.84×10^{-5}	8.36×10^{-5}	7.84×10^{-5}	7.81×10^{-5}
ε_z	1.12×10^{-4}	1.46×10^{-4}	1.12×10^{-4}	1.11×10^{-4}
N_{si}	6.53×10^8	2.01×10^8	6.53×10^8	6.92×10^8
m_{si}	3.83×10^{-4}	1.24×10^{-4}	3.83×10^{-4}	3.61×10^{-4}
m_a	0.00316			
m_s	0.00237			

Table B3. Essential parameters for durability with ASL thickness of 0.10 m

	Spring	Summer	Autumn	Winter
ε_{r1}	8.20×10^{-6}	1.24×10^{-5}	8.20×10^{-6}	7.73×10^{-6}
ε_{r2}	2.32×10^{-5}	2.82×10^{-5}	2.32×10^{-5}	2.28×10^{-5}
ε_r	2.32×10^{-5}	2.82×10^{-5}	2.32×10^{-5}	2.28×10^{-5}
N_{ai}	2.56×10^8	1.34×10^8	2.56×10^8	2.70×10^8
m_{ai}	9.77×10^{-4}	1.87×10^{-3}	9.77×10^{-4}	9.27×10^{-4}
ε_{zu}	9.86×10^{-5}	1.33×10^{-4}	9.86×10^{-5}	9.71×10^{-5}
ε_{zl}	7.74×10^{-5}	8.12×10^{-5}	7.74×10^{-5}	7.69×10^{-5}
ε_z	9.86×10^{-5}	1.33×10^{-4}	9.86×10^{-5}	9.71×10^{-5}
N_{si}	1.18×10^9	3.04×10^8	1.18×10^9	1.26×10^9
m_{si}	2.12×10^{-4}	8.22×10^{-4}	2.12×10^{-4}	1.99×10^{-4}
m_a	0.00475			
m_s	0.00145			

Table B4. Essential parameters for durability with ASL thickness of 0.15 m

	Spring	Summer	Autumn	Winter
ε_{r1}	7.827×10^{-6}	1.15×10^{-5}	7.827×10^{-6}	7.27×10^{-6}
ε_{r2}	2.57×10^{-5}	3.17×10^{-5}	2.57×10^{-5}	2.12×10^{-5}
ε_r	2.57×10^{-5}	3.17×10^{-5}	2.57×10^{-5}	2.12×10^{-5}
N_{ai}	1.81×10^8	9.14×10^7	1.81×10^8	3.43×10^8
m_{ai}	1.38×10^{-3}	2.73×10^{-3}	1.38×10^{-3}	7.30×10^{-4}
ε_{zu}	9.14×10^{-5}	1.32×10^{-4}	9.14×10^{-5}	8.96×10^{-5}
ε_{zl}	7.26×10^{-5}	7.92×10^{-5}	7.26×10^{-5}	7.15×10^{-5}
ε_z	9.14×10^{-5}	1.32×10^{-4}	9.14×10^{-5}	8.96×10^{-5}
N_{si}	1.65×10^9	3.19×10^8	1.65×10^9	1.81×10^9
m_{si}	1.51×10^{-4}	7.85×10^{-4}	1.51×10^{-4}	1.38×10^{-4}
m_a	0.00622			
m_s	0.00123			

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