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# Assessing the damaging effects of railway dynamic wheel loads on railway foundations

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- Assessing the Damaging Effects of Railway Dynamic Wheel Loads on Railway
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#### 36 ABSTRACT

Dynamic train wheel loads, which can be significantly greater than static loads, occur 37 due a variety of factors and unless they are properly considered in track structural 38 design, significant unplanned maintenance and premature track failure may result. 39 This is particularly so for traditional ballasted railways built on soft foundations 40 because although ballast lends itself to maintenance, it is often problematic and costly 41 42 to repair damaged foundations. To address this, a rigorous combined analytical-numerical approach is described to predict and characterize, for the first 43 time, the damage to which railway foundations can be subjected as a result of 44 45 dynamic loads. The approach marries a sophisticated three-dimensional dynamic model of the train-track system incorporating vertical track quality, foundation soil 46 distress models, statistical analysis methods and results of field investigation. 47 The resulting analyses demonstrate that the magnitudes and distributions of 48 dynamic loads are a function of train speed and track quality and that specific 49 locations experience significantly higher amounts of damage which can lead to a 50 variety of track faults. The approach is illustrated via a study of a heavy haul 51 railway line in China where the wheel loads and tonnage carried are set to increase 52

significantly. The study suggests that the thickness of the ballasted layer would need
 to increase by over 20% to prevent premature foundation failure provided that the
 track is maintained in good condition, and by significantly more should the track

- 56 condition be allowed to deteriorate.
- 57

58 Keywords: Railway, dynamic loads, foundations, design

#### 60 INTRODUCTION

The railway track is a structural system built to withstand the combined effects of traffic and the environment for a pre-determined period of time, so that railway vehicle operating and maintenance costs, passenger comfort and safety are kept within acceptable limits and the foundation is adequately protected. Dynamic train loads induced by track irregularities and vehicle characteristics can reduce significantly the life of the components of the structural system.

Although a number of international railway infrastructure operators have developed railway track structural design standards they do not adequately take into account the spatial fluctuating nature of dynamic loads [1]. By implication the use of these design standards may lead to the under design of the structural system, premature failure of track components and its foundation, unplanned maintenance, reduced safety and higher train operating costs.

To better understand the implications of dynamic loads on the railway system, considerable research has been undertaken to measure dynamic loads in the field and also to estimate, via laboratory analysis, their potential impacts on the deterioration of the railway structural system. Methods have also been proposed which if used provide a means for enabling the spectrum of dynamic train loads to be accounted for within railway track design, principally by Eisenman [2] and Stewart and O'Rourke [3].

The method suggested by Eisenman [2] is based on studies of measured dynamic loads and takes into account vehicle speed and track condition. For speeds of up to 60 km/h, Eisenman found that dynamic loads followed a Gaussian distribution with a mean value which was independent of the operating speed, *V*, but dependent on track condition,  $\varphi$ . At 60 km/h and above the dynamic forces were found to be a function of both vehicle speed track condition.

Stewart and O'Rourke's method [3] relies on field measurements of dynamic loads. 82 For the analysis of the substructure they assume that a single load application comprises of the two 83 84 axles of the trailing and two of the leading axles of a pair of coupled wagons. To calculate the distribution of the maximum loads. Stewart and O'Rourke assume that the maximum static train 85 load acts on the outer two axles of the configuration, whilst the inner two axles impart a high 86 dynamic load corresponding to a very low probability of occurrence (they suggest 0.01%) 87 determined from the field data. When the effects of fatigue loading on the foundation are to be 88 calculated, Stewart and O'Rourke suggest that a spectrum of loads should be determined from the 89 distribution of field measured loads. To achieve this, they propose that the frequency distribution 90 of measured loads should be divided into a number of bands of probability of occurrence (e.g. 0 -91 5%, 5 - 10% etc.) and that a representative load application for design is determined for each. 92

The studies by Eisenman [2] and Stewart and O'Rourke [3] are based on empiricism, rely on 93 field measurement, which can be time consuming and expensive, and they overlook a number of 94 important factors. Both methods assume that the distribution of loads in the foundation matches 95 that of the surface wheel load distribution. However, the mathematical relationship between the 96 applied surface wheel load and the resulting damage to the foundation is non-linear and therefore 97 the statistical distribution of component damage does not match that of the surface loads. Further. 98 the deterioration at any point along the track structure depends on the accumulated damage due to 99 each passing wheel load which can vary in magnitude depending on the proximity of a track 100 irregularity. Dynamic loads due to track irregularities are likely to be highest in the vicinity of a 101 particular track irregularity, where they are likely to occur repeatedly leading to increased rates of 102 103 track deterioration at these locations.

Advances in computer modeling capability are enabling accurate and complex numerical models of the railway train-track system to be built. These if used carefully can better help to understand railway track system performance under dynamic loads in a variety of operating conditions, thereby reducing the need for potentially time consuming and costly field and laboratory trials. Numerical models have been developed to investigate a variety of track related issues

- including those associated with the transition between stiff track structures and less stiff railway
- track [4-7], ground vibration [8, 9], seismic analysis [10], critical velocity [11] and the integrity of track components under dynamic loads [12-15].

However, very few numerical studies have been undertaken to investigate the relationships 112 between dynamic wheel loads, track structural design and railway foundation deterioration along a 113 section of railway track. To address this, this paper establishes a rational analytical-numerical 114 procedure which enables dynamic wheel loads to be properly accounted for in the structural design 115 of railway track, thereby preventing premature failure and unplanned maintenance. The procedure 116 builds on that suggested for the analysis of highway distress [16]. It utilizes a three dimensional 117 dynamic finite element model (FEM) of the railway train-track system incorporating track quality, 118 foundation structural distress models and statistical analysis methods. The procedure is 119 demonstrated via a case study of the Shuanghuang coal route in China. 120

122 THEORETICAL FRAMEWORK

123 The approach proposed consists of the following elements:

- i) Structural distress models of the track foundation to determine the values of the critical
   stresses, strains and deflections in the materials which comprise the substructure as a
   function of the magnitude and number of load applications
- ii) A 3-D dynamic FEM of the railway train-track system, incorporating a model of track
   quality variability, to enable stresses, strains and deflections to be computed as a
   function of dynamic train loads at specific locations in the track structural system (see
   Figure 1).
- 132133 Structural distress models
- 134 Railway Foundation Failure Mechanisms

The track foundation becomes progressively damaged through the cumulative effects of traffic
induced repetitions of stresses and strains. For fine-grained subgrade soils the resulting damage

137 can manifest as progressive shear failure and / or an excessive rate of settlement [17].

138 Progressive shear failure occurs where cyclic stresses are sufficiently high and are applied for

long enough to cause material to be sheared and remolded. An excessive rate of settlement

140 occurs through plastic deformation of the subgrade and may cause a ballast pocket to form. For

shear failure, the design problem can be considered to be putting a limiting value on the plastic

strain, whereas for an excessive rate of settlement the design problem is to limit the amount of cumulative plastic deformation [17].

144

121

124

### 145 *Distress models*

146 For fine-grained subgrade soils, it is recognized that plastic strain is a function of the number of

- 147 loading cycles, *N*, soil stress history and drainage conditions. Models to predict plastic strain,  $\varepsilon_p$ , 148 in fine grained materials are typically of the following form [18]:
- 149

150

- $\varepsilon_{p} = CN^{b}$
- 151
- 152 Where C is a constant related to the material properties.
- 153

(1)

To take into account soil physical state and type a modified version of Equation 2 has been suggested [17]:

156

157 
$$\varepsilon_{p} = a \left(\frac{\sigma_{d}}{\sigma_{s}}\right)^{m} N^{b}$$
(2)

158

159 Where *a*, *b* and *m* are material parameters determined from experiment,  $\sigma_d$  is the deviator stress 160 and  $\sigma_s$  is the soil static strength.

161

162 Noting that the permanent deformation,  $\rho$ , can be written as

163 
$$\rho = \int_{a}^{b} \varepsilon_{p} ds$$
(3)

164 where *T* is the thickness of the foundation.

165 166 then:

167 
$$\rho = \int_0^T A\left(\frac{\sigma_d}{\sigma_s}\right)^m N^b$$
(4)

168

#### 169 *Phenomological theory of cumulative damage*

The phenomological theory of cumulative damage was advanced by Miner [19] to predict the fatigue life of materials subjected to fluctuating stress amplitudes. The theory states that the cumulative damage D, is the linear summation of damages,  $D_i$ , due to  $N_i$  applications at stress or strain level *i*:

174

175 
$$D = \sum_{i=1}^{r} D_i = \sum_{i=1}^{r} \frac{N_i}{N_{f_i}}$$
 (5)

176

177 Where  $N_{fi}$  is the number of applications to failure at stress, or strain, level *i*. Failure occurs when 178 D = 1.

Using Equations 3 to 6, it is possible to estimate the proportion of the total life used at a location kon the railway track due to a single load application as follows:

182 For shear failure:

181

184 
$$D_{sf(k)} = \sum_{j=1}^{N} \frac{1}{\left(\frac{\varepsilon_{Psf}}{a}\right)^{\frac{1}{b}} \left(\frac{\sigma_s}{\sigma_d}\right)^{\frac{m}{b}}}$$
(6)

185

187

186 where  $\varepsilon_{psf}$  is the plastic strain at failure.

188 For plastic settlement:

190 
$$D_{pf(k)} = \sum_{j=1}^{N} \frac{1}{\left(\frac{\rho_{sf}}{A}\right)^{\frac{1}{b}} \int_{0}^{T} \left(\frac{\sigma_{s}}{\sigma_{d}}\right)^{\frac{m}{b}} dt}$$
 (7)

- 191 where  $\rho_{sf}$ , is the amount of plastic deformation at failure.
- 192 For the application given below, the value of the parameters used in equations 6 and 7 are shown in
- 193 Table 1, together with a description of how they were obtained.

# 194195 APPLICATION

As part of China's solution to its current shortage of transport capacity it is increasing utilization on many of its heavy haul railway lines. One such line is the 588km long Shuanghuang railway, an important route in China's coal corridor, which runs from Shenchi in Shanxi province to the Huanghua port in Hebei province. The line currently carries between 30-40 million tonnes of coal annually at speeds of up to 75km/h. In order to satisfy predicted greater coal output the amount carried on the line is set to increase to 600 million tonnes/yr with an increase in train wheel loads from 125kN to 150kN [20].

An extensive research project was undertaken to investigate the implications of the proposed increases in tonnage and loads on safety, track component damage, track structural design and maintenance. As part of the project a plain line (tangent) section of an embankment section was instrumented with accelerometers during its construction to measure train-induced accelerations in the rail, ties and the embankment at various locations [4]. The work presented here examines, using the suggested analytical-numerical procedure, the implications of the proposed increases in traffic on track structural design and maintenance.

### 210211 Numerical Model

A dynamic train-track FEM representing a train of wagons running on a plain line 100 m section
of the embankment, incorporating a means to modify track quality, was built using the ABAQUS
Explicit<sup>™</sup> software. The model configuration of the model, consisting of 124, 357 elements and
176, 268 nodes, is shown in Figure 1Figure 1. The values of the material properties for each
component of the model are given in Table 1 Material properties for modelling purposesThe track
structure and vehicle models, the method used to incorporate track quality within the model and
the model's validation are briefly described below.

219

#### 220 Track structure

The rails, ties, ballast and embankment were represented using solid eight-node elements with the material properties given in Table 1. The track substructure was modelled as a layer of ballast underlain by three discrete layers of sand and gravel, engineering fill and silt sand respectively to represent the embankment's construction according to Chinese design standards [21].



#### Figure 1 FEM representation of railway track system

- 239 Track quality
- 240 Three vertical track profiles representing good, fair and poor track quality according to the US
- 241 Federal Railroad Administration classification system were represented within the FEM. The
- vertical profiles were characterized within the FEM using a one-sided power spectral density
- 243 (PSD) function,  $S_{y}(\Omega)$ , suggested by Fries and Coffey [24]:
- 244

245 
$$S_{v}(\Omega) = \frac{kA_{v}\Omega_{c}^{2}}{\Omega^{2}(\Omega^{2} + \Omega_{c}^{2})}$$
(8)

246

where  $\Omega$  is the spatial frequency of the track irregularity (Hz),  $A_v$  is the roughness coefficient (cm<sup>2</sup>.rad/m) and *k* is the safety coefficient. The values of the coefficients in Equation 8 which were used to represent the three track quality states are given in Table 1. The same profile was used for both rails.

251

252

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### Table 1 Material properties for modelling purposes

Component	l	Property	Value	Note
	Mass of	of car body (Kg 10 <sup>3</sup> )	91.4	Two UIC Class T0AB freight wagons were modelled. The associated FEM parameters
	Inertia	of car body (Kg m <sup>2</sup> )	1.33×10 <sup>5</sup>	required for the FEM are as suggested by [22].
	Mass of	of bogie (Kg)	496	
	Inertia	of bogie (Kg m <sup>2</sup> )	190.	
Freight Wagon	Mass of	of wheel (Kg)	1257	
	Primar	y suspension stiffness (MN/m)	13	
	Primar	y suspension damping (Ns/m)	3×10 <sup>5</sup>	
	Second	dary suspension stiffness (MN/m)	4.4	
	Second	dary suspension damping (Ns/m)	4×10 <sup>3</sup>	
	Young	's modulus, GPa	210	Parameters for the FEM chosen to match the type of rail installed in situ
Rail	Poisso	n's ratio	0.3	
	Densit	y kg/m <sup>3</sup>	7830	
<b>T</b> '.	Young	's modulus, GPa	35	From Chinese railway design standards: TB10001: 2005: [21]
11e	Poisso	n's ratio	0.22	
	Densit	y kg/m <sup>3</sup>	2600	
	Vertica	ll stiffness, kN/mm	78	Values taken from those suggested by [9]`for the analysis of a heavy haul line in China built to
	Vertica	ll damping kN.s/m	50	similar standards
Fastener	Horizo	ontal stiffness, kN/mm	45	
	Horizo	ontal damping ,kN.s/m	60	
Resilient		ent modulus, MPa	180	From Chinese railway design standards: TB10001: 2005: [21]. Note the ballast was assumed
Ballast (0.5m t	hick) Poisso	n's ratio	0.27	to be clean (i.e not fouled)
	Densit	y kg/m <sup>3</sup>	1650	
	Poor (FRA4)	$\Omega_c (\mathrm{cm}^2.\mathrm{rad/m})$	0.8245	Track quality is typically represented by a one-sided power spectral density (PSD) function in
1 1	(max speed 96	$A_v$ (cm <sup>2</sup> .rad/m)	0.5376	numerical models of the track system. For this study the PSD suggested by Fries and Coffee
rack quality*	<u>km/h)</u>	k	0.25	[23] was selected. The values of the coefficients $\Omega_c$ , $A_v$ and k selected for each track quality
		Amplitude (mm)	30~40	state are those suggested by Fries and Coffey [23] to represent good, fair and poor track

	<u>Moderate</u>	$\Omega_c (\mathrm{cm}^2.\mathrm{rad/m})$	0.8245	quality according to the US Federal Railroad Administration classification system.
	(FRA5) (max	$A_{\nu}$ (cm <sup>2</sup> .rad/m)	0.2095	
	<u>speed 128</u>	k	0.25	
	<u>km/h)</u>	Amplitude (mm)	10~15	
		$\Omega_c (\mathrm{cm}^2.\mathrm{rad/m})$	0.8245	
	Good (FRA6)	$A_{\nu}$ (cm <sup>2</sup> .rad/m)	0.0339	
	(max speed	k	0.25	
	<u>176 km/h)</u>	Amplitude (mm)	5~6	
		Resilient modulus MPa	180	Initial modulus values determined from plate loading tests conducted in-situ on the 100 m
	Parameters for	Poisson's ratio	0.3	used to obtain the final resilient modulus values by successively modifying the resilient
	FEM	Density kg/m <sup>3</sup>	2300	modulus values in each of the three layers until the accelerations given by the model matched
Layer 1: Sand gravel				neid measurements [2+].
(0.7 m thick)		a	0.52	Determined from plastic deformation laboratory tests on material taken from the 100m section
	Parameters for	b	0.15	material in layer 1 cannot be considered to be fine-grained, it was found that its permanent
	distress model	m	1.49	deformation characteristics could be modelled using an equation of the form given by
		σ <sub>s</sub> (KPa)	350	Equation / [25]
	D	Resilient modulus, MPa	130	As for layer 1 [24].
	Parameters for	Poisson's ratio	0.3	
Layer 2: Class A engineering fill (2.3	FEM	Density kg/m <sup>3</sup>	2100	
		a	0.85	As for layer 1 [25]
m thick)	Parameters for	b	0.14	
	Distress model	m	1.49	
		$\sigma_{s,k}Pa$	200	
		Resilient modulus, MPa	50	As for layers 1 and 2 [24].
Layer 3: Silty sand (3	Parameters for	Poisson's ratio, v	0.25	
m thick)	FEM	Density kg/m <sup>3</sup>	1800	

	a	0.64	As for layes 1 and 2 [25]
Parameters for	b	0.1	
Distress model	m	1.16	
	$\sigma_{s}$ , kPa	100	

\*Note that the maximum permitted amplitudes of track quality deviations on main-line heavy haul railway track in many countries is limited to between 6 – 10 mm.

#### 256 Vehicle Model

In accordance with analytical railway foundation design convention, the railway freight vehicle 257 was represented by the leading and trailing bogies of two coupled wagons [3]. The coupled 258 wagons were modelled using a multibody system consisting of a car body, bolster, frame and 259 wheelset (Figure 1). The primary suspension system, connecting the wheels and the frame, and 260 the secondary system were modelled using a series of linear springs and viscous dashpots. The 261 rail-wheel interaction in the normal direction was modelled as a Hertzian contact (where 262 separation is allowed resulting in a zero contact force), since it is widely used in FE analyses to 263 represent the contact between spherical objects and deformable surfaces (such as the wheel and 264 the rail respectively in railway applications [9]). Hertzian contact assumes that the contact 265 surface between the wheel and rail increases as the deformation increases. The normal contact 266 force P(t) can be determined as follows [9]: 267

269 
$$P(t) = \left[\frac{1}{G}\Delta Z(t)\right]^{\frac{3}{2}}$$
270 (9)

where  $\Delta Z(t)$  is the elastic compression between the rail and wheel (m). The contact constant G is given by  $G = 3.86R^{-0.115} \times 10^{-8} \text{ m/N}^{3/2}$  where *R* is the wheel radius. The wheel-rail creep force,  $\tau_{\text{crit}}$ , is given by:

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278

268

$$275 \quad \tau_{crit} = \mu P \tag{10}$$

where  $\mu$  is the coefficient of friction between the rail and the wheel.

To quantify the effects of fluctuating dynamic wheel loads, the maximum stresses due to the passage of a train were computed at discrete sections along the track sufficiently short in length to enable the peak stresses to be determined. The principle of superposition was used to calculate the effect of a train of wagons. It was found that superimposing the loads in this way increased the stresses by less than 5% compared to calculating the stresses due only to the two coupled wagons.

285

#### 286 *Model validation*

To provide confidence in the outputs of the developed FEM, computed and field measured and 287 accelerations and vibrations were compared at various positions in the track structure for a train 288 travelling at 71 km/h [4]. Since the embankment section has been newly constructed the track 289 quality in the FEM was considered to be perfectly smooth. Table 3 shows that the computed and 290 measured values are generally in good agreement, albeit that the computed values are slightly 291 higher. A reason for the slightly higher computed values could be attributed to the heavy rainfall 292 which occurred just before the vibration and acceleration measurements, but after the field tests 293 which were carried out to determine the properties of the materials in the embankment (see Table 294 295 1). 296

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	Accelerations (ma	ax) (g)	Vibration (dB)				
	Computed	Field	Computed	Field			
Rail	40.00	23-50	155.0	153.7			
Ties	10.00	3-11	147.1	138.0			
Foundation surface	1.0	0.3-0.8	109.6	102.5			
Foundation at 2m depth	0.40	0.4-0.6	105.0	100.5			

#### **Table 2:** Comparison of modelled and field measured accelerations and vibrations

#### 300

301 ANALYSIS

Using the FEM and distress models described above, the maximum wheel force time histories and 302 maximum stress time histories were calculated at discrete locations along and within the modelled 303 embankment for a train of wagons with wheel loads of 125kN and 150kN travelling at speeds 304 between 50 and 250km/h. Since the highest peak dynamic forces, and therefore the initiation of 305 defects are likely to occur in the vicinity of specific track quality irregularities which cause large 306 dynamic loads, damage was related to the area of track which becomes significantly damaged 307 Track which is significantly damaged was assumed to be that subject to loads greater than 308 [16]. the 95<sup>th</sup>, 98<sup>th</sup> and 99<sup>th</sup> percentile values. 309

310

#### 311 Wheel forces

Figure 2 shows the distribution of the magnitude of the maximum wheel forces at discrete

sections along the embankment for a train of wagons with nominal wheel loads of 125kN

travelling at 75km/h with the three track quality profiles. The distribution results from a

combination of the variability in track quality and the presence of ties which cause the stiffness of

the track system to vary along the track. The variability apparent in the distribution of loads

reduces as the track quality improves, corroborating Eisenman's experimental findings [2].

318



Figure 2 Dynamic rail force distribution at a speed of 75 km/h

To demonstrate the effect of speed and track quality on the magnitude of the maximum 321 wheel loads, Figure 3 shows the 95<sup>th</sup>, 98<sup>th</sup> and 99<sup>th</sup> wheel forces (normalized by the static wheel 322 load) as a function of vehicle speed and track condition. As might be expected, the dynamic 323 wheel load increases with both train speed and deteriorated track. For example, when the track 324 quality is in good condition and for train speeds of 50km/h, 2% of the track experiences dynamic 325 wheel loads of between 15%-20% greater than the static wheel load. For speeds of 250km/h the 326 dynamic forces are between 35%-45% greater than for the static case. When the track quality is in 327 poor condition the corresponding load increases are 44%-52% and 125%-205% respectively. 328

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#### Figure 3 Dynamic wheel force vs. speed

Figure 3 shows a local maximum dynamic load for all three track conditions at a vehicle 333 speed of approximately 100km/h. For perfectly smooth track the critical speed is that which 334 results in a wheel encountering a tie at a frequency which matches the resonant frequency of the 335 track structural system [11]. This frequency is known as the tie-passing frequency. When this 336 phenomenon occurs, the response of the track to successive loading cycles are in-phase resulting 337 in an amplified track response. For imperfect track, Figure 3 shows that the magnitude of the 338 amplified response is also related to the quality of the track and therefore the magnitude of the 339 dynamic loads. The effect of resonance is particularly apparent for the poorest track quality 340 (FRA4) when the amplitudes of the most extreme dynamic loads caused by the presence of large 341 track irregularities are sufficient for the resonance to be apparent. The resonant frequency can 342 vary from 30Hz to 2000Hz depending on a number of factors including [26]: 343

344
345 1. Vehicle characteristics:
346 a. Wheels, bogie and wagon spacing
347 b. Sprung and un-sprung mass, primary and secondary stiffness and damping
348 2. Track properties:
349 a. Track stiffness and damping of the different track components.

- b. The mass of the track structure, i.e. ballast, sub-ballast and subgrade.
- c. Tie spacing

350

351 352

### 353 **Permanent strain and settlement in the embankment**

The proportion of the amount of the life used by a single passage of the coupled wagon system in the three layers of the embankment was determined using equations 6 and 7 respectively. In accordance with the literature, permanent strain of 2% and 25mm of settlement in the embankment were taken to indicate failure according to the two measures of damage [27]. For a train travelling at 75km/h, the distribution in the Class A Engineering Fill layer of the embankment (i.e. the second layer) of fatigue life usage according to the strain and settlement criteria is shown in Figure 4.

361 In general it can be seen from Figure 4 that the variability and magnitude of the damage increases as the track quality decreases. It is also apparent that some sections along the 362 embankment are subject to much greater damage than others (i.e. those sections which are in the 363 vicinity of a particular irregularity). For example, for the second layer of the embankment when 364 the track is in good condition the computed 99<sup>th</sup> percentile value of strain damage is 365 approximately 1.7 times greater than the computed median value and 2.3 times greater than the 366 median value when the track is in poor condition. Similarly, for the settlement criterion the 99<sup>th</sup> 367 percentile value is approximately 1.4 times greater than the median value when the track is in 368 good condition and 2.8 times greater when the track is in poor condition. 369

Assuming the train and track operating conditions remain unchanged the same sections of 370 track will experience these greater amounts of damage for every load cycle over the life of the 371 The resulting localized settlement is likely to cause increased dynamic loads in the same 372 track. 373 vicinity thus accelerating further the accrued damage. An example is the occurrence of localized failures, such as wet spots which can become apparent on railways built on soft foundations. Wet 374 spots are caused by the upward migration of fines into the ballast under dynamic loads leading to 375 ballast fouling and poorly performing ballast. The resulting non-homogenous railway track 376 stiffness leads to worsening track quality which will increase further the magnitude of dynamic. 377

Constant tie spacing can result in resonance as mentioned above and therefore varying the 378 spacing between ties could be a means of reducing localized track deterioration. In practice 379 however, this is impractical for many railway infrastructure operators who use automated tie 380 relaying systems and tamping machines. In the UK, for example, the practice is only employed 381 at problematic sites where tie spacing is varied manually by +/-5%. The approach advocated 382 here can also be used to investigate alternative approaches to avoid resonance, such as 383 determining permissible ranges of speeds at which specific types of vehicle can travel on 384 particular sections of track. 385

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# Figure 4 Distribution of fatigue life usage (damage) in second layer of embankment at a speed of 75 km/h

The 95<sup>th</sup>, 98<sup>th</sup>, 99<sup>th</sup> and 100<sup>th</sup> percentile computed fatigue life usage for a single passage of the coupled wagon system as a function of speed and track condition, are shown in Figure 5 and Figure 6 according to the strain and settlement measures of damage. In both figures the values have been normalized by the fatigue life by the application of a single static load. The resonance effect is evident at vehicle speeds of approximately 100km/h.

As may be expected the damage in the second layer of the embankment increases with 397 both train speed and reduced track condition. For example, when the track quality is in good 398 condition (FRA6) and for train speeds of 50km/h, 5% of the area of second layer of the 399 embankment is subject to between 3-4 times the plastic strain than would be caused by a static 400 train load. For speeds of 100km/h, 5% of the area of the second layer is subject to levels of 401 402 plastic strain which are between 6-8.5 times greater than caused by a static load. For track in poor condition (FRA4) the corresponding computed increases in plastic strain are between 14-20 403 (50 km/h) and 300-450 (100 km/h) times respectively. Similar results can be observed for the 404 permanent settlement measure of damage. 405

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#### Figure 5 Plastic strain vs. speed (second layer of the embankment)



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#### Figure 6 Total settlement vs. speed (second layer of the embankment)

#### 413 APPLICATION

To demonstrate the implications of the proposed changes to the Shuanghuang line an analysis was carried out to compare the number of load cycles which each layer in the embankment could further undergo before failure. This analysis considered the plastic strain criterion under the following regimes:

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4191. Existing regime (i.e. 40MGT/yr for trains with wheel loads of 125kN for another 90 years.420This is equivalent to  $\frac{40 \times 10^6 \times 10^3 \times 90}{125/10 \times 8} = 3.6 \times 10^{10}$  load cycles (assuming 1 load cycle in421the subgrade comprises of the leading and trailing axle axles of two adjacent wagons.)4222. The proposed regime for the remaining 90 years of the life of the track (i.e. 600MGT/yr423with wheel loads of 150kN which is equivalent to  $\frac{600 \times 10^6 \times 10^3 \times 90}{150/10 \times 8} = 4.5 \times 10^{11}$  load424cycles).

For the two regimes, the ballast was considered to be clean throughout the analysis. The 425 fractions of the remaining number of cycles to failure of each layer was calculated using Equation 426 6 and are shown in Table 3 normalized by the desired number of loading cycles. Under current 427 operating conditions, Table 3 shows that for all track speeds considered, except for 150km/h, 428 when the track quality is in a poor or fair condition the material in the second layer of the 429 430 embankment would fail prematurely. From Table 3 it can also be seen that for the proposed heavier wheel load regime the second layer would fail prematurely no matter the train speed or 431 track condition. The upper layer of the embankment would also fail prematurely if the condition 432 of the track was maintained to anything other than a good condition, except for the case where the 433 train speed is limited to 75km/h. The effect of travelling at the critical speed on the reduction in 434 the life of the material in the embankment is also evident for the heavier wheel load. 435

The consequence of ballast maintenance on track life is evident by comparing the remaining life under the three track conditions modeled. For example, the second layer will last between 4-8 times longer, depending on the speed of the train, if the track is in a good condition (FRA6) compared to a fair condition (FRA5).

A further analysis determined the amount of additional granular material 440 (ballast/sub-ballast) required to reduce the deviator stress in the Engineering Fill layer so that 99% 441 of the track would not exceed the allowable fatigue life values. The resulting thicknesses are 442 given in Table 3 and demonstrate that significant additional amounts of granular material would 443 be required under the existing regime for all speeds if the track is not maintained in a good 444 Should the proposed changes to the capacity of the line take place, then the study condition. 445 suggests that an additional thickness of at least 110mm of the granular layer would be required. 446 provided that the track is maintained in good condition. 447

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	Sand / gravel layer							Engineering fill layer						Silty sand layer					Additional granular layer (mm)					
	FR	A4	FR	A5	FR	A6	FR	A4	FR	A5	FR	A6	FR	A4	FR	A5	FR	A6	FR	A4	FR	A5	FR	A6
Load (KN)	125	150	125	150	125	150	125	150	125	150	125	150	125	150	125	150	125	150	125	150	125	150	125	150
(4m B) 75	8.58	0.66	41.2	1.87	123	5.06	0.13	0.009	0.64	0.03	2.29	0.12	2480	32.1	12500	114	45300	419	105	280	20	185	0	110
100	2.43	0.19	13.8	0.63	81.25	3.33	0.02	0.002	0.22	0.01	1.61	0.08	368	4.78	3700	33.8	27000	251	220	420	75	260	0	130
125	5.55	0.21	29.3	0.69	113	3.56	0.06	0.003	0.55	0.01	2.03	0.06	263	4.02	2420	18.8	8900	88.2	160	375	25	250	0	150
150	13.68	0.24	66.5	0.76	160	3.80	0.18	0.004	1.52	0.01	2.58	0.04	191	3.38	1570	10.3	2680	30.9	90	335	0	245	0	175
175	8.33	0.15	25.8	0.30	97.25	2.12	0.12	0.003	0.54	0.00	1.38	0.02	92	1.63	398	2.62	1020	13.4	110	365	26	320	0	205

450 <b>Table 3.</b> Remaining number of cycles to fature (according to plastic strain criteria) and additional barast unexitess requirement	450	Table 3: Remaining number of c	cycles to failure (according t	o plastic strain criteria) and a	additional ballast thickness requiremen
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#### **CONCLUSIONS** 452

A novel rigorous analytical-numerical approach has been provided to take into account the spatial 453 fluctuating nature of dynamic wheel loads within railway track structural design methods. 454 Such an approach helps to ensure the adequate design of the structural system and thus facilitates the safe 455 operation of the railway track, prevents premature track failure and therefore unplanned 456

- maintenance, and reduces train operating costs. 457
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- 459 A number of conclusions may be drawn from the study as follows:
- 460
- 1. The magnitude of dynamic loads is a function of the train speed, axle load and track 461 quality and specific locations along the track, corresponding to areas of poorer track 462 quality, experience significantly higher dynamic loads. A natural frequency of vibration 463 of the track structure was identified which corresponds to the tie passing frequency and is 464 a function of the magnitude and wavelength of track irregularities. 465
- 2. The importance of ensuring good track quality is evident. Increased dynamic loads 466 resulting from poor track quality can lead to localized increased rates of foundation 467 deterioration and may lead to other types of track failure. This can cause a cycle of 468 worsening track quality which in turn increases the localized dynamic loads. 469
- 3. The case study analysis of the Shuanghuang line, although relatively simplistic in that it 470 assumed amongst other things constant train speed and track quality condition over time, 471 showed that, provided that the track is maintained in good condition, an additional 20% of 472 granular layer material would be required to prevent premature embankment failure. 473
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475 A number of other causes of dynamic loads could be considered within an enhanced version of the model, thereby further increasing the accuracy of any analysis. These include out of round wheels, 476 rail irregularities, fouled ballast and hanging ties. Furthermore it is recognized that the approach 477 advocated requires the use of a number of parameters associated with the FEM and the deterioration 478 models which need to be selected carefully for the conditions at hand. 479 480

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