

# Analysing method for checking dynamic behaviour of steel railway bridges

*study of dynamics of railway bridges and applicability assessment of simplified dynamic analysing  
method.*

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# Chapter 1

## INTRODUCTION

### 1.1 Summary of topic

The lateral dynamic behaviour of steel railway bridges are minimally discussed in Eurocodes and designers lack knowledge of background of criteria proposed in the code. For example, there is one criterion in Eurocode requiring railway bridges should have a lateral natural frequency higher than 1.2Hz. However, this criterion is becoming more and more unsuitable because longer span provides lower natural frequencies. For bridges having span more than 100m, it is almost guaranteed that the first lateral natural frequency of the bridge fall below 1.2Hz, unable to meet the requirement of Eurocode.

Criteria on lateral dynamics of railway bridges are complicated if taking vehicle systems and interaction into account. Designers need a better knowledge on railway dynamics and a tool in calculating the lateral dynamic behaviour of the whole system. This tool needs to be simple to meet the engineering needs.

### 1.2 Motivation of the thesis

### 1.3 Objectives and research question

The main goal of this thesis is to discuss the sufficiency and reliability of current criteria regarding lateral dynamics of steel railway bridges that exist in Eurocode 1991-2. Original reports where criteria were proposed will be reviewed to reveal the background of them to Eurocode users.

Depending on the output of previous step, if the current requirements/criteria show lack of reliability/sufficiency of Eurocode 1991-2, further recommendation will be made for revising or amending current Eurocode.

This report also aims to providing knowledge of railway bridge dynamics to Eurocode users. The dynamics of railway bridge concerns cross-field knowledge, including intelligence of railway tracks/ vehicles/ bridge structures, etc. Some of them are essential to dynamics problem but unfamiliar to Eurocode users. Thus the report is willing to convey users the reason and importance of dynamic concerns of railway bridges.

### 1.4 Main steps

In order to provide a better tool for designers when they encounter lateral dynamic problems on steel railway bridges, following objectives are made:

1. Literature research of dynamic actions as well as their criteria on railway bridges, rails and train vehicles in order to give a better understanding of the background of the criteria which is unclear to the designers. Study the dynamic behaviour of these system respectively. Then discuss their effects when combined.
2. Find the original reports which proposed 1.2Hz criterion but assess the content of these reports.
3. Develop a method to check if a bridge is prone to encounter dynamic problems. The method should be simple. It should be compatible with FEM software and give suggestions for further bridge modification. However form of the method will depend on output of the previous research output.

## **1.5 Research methodology**

## **1.6 Outline of the report**

## Chapter 2

# LITERATURE REVIEW

Eurocode 1990 and Eurocode 1991-2 and their corresponding National Annex are primary codes to be fulfilled through out the whole process of conducting a railway bridge in Netherlands. It is of great importance to study dynamic effect on railway bridges due to increasing usage of public train service. What's more, as the need on capacity of railway service increase, high-speed train dynamic loading becomes a more general issue in the design of railway bridges. It is common knowledge that bridge structures loaded by high-speed trains have bigger chance of resonance, as well as of being required to be dynamic analysed in designing process.

Unfortunately in Chapter 6.4 of Eurocode NEN-EN 1991-2-2003, the description of various subjects is vague including general procedures of conducting a dynamic analyses and methods of additional dynamic analysing calculating, etc. The following paragraphs aim to summarize Chapter 6.4 of Eurocode 1991-2 [9], in order to give a better interpretation.

This literature research will be done by reviewing both physics knowledge and engineering standards.

### 2.1 Forced Vibrations under Harmonic Force

Assume there is a simple one degree-of-freedom mass-spring system and an external force is acting on it. The force is given as  $F(t) = F_0 \cos(\omega t)$ . In this case the equation of motion takes the form

$$m\ddot{x} + kx = F_0 \cos(\omega t) \quad (2.1)$$

The general solution can be written as

$$x(t) = A \cos(\omega_n t) + B \sin(\omega_n t) + \frac{F_0}{k} \frac{1}{1 - \omega^2/\omega_n^2} \cos(\omega t) \quad (2.2)$$

The unknown constants A and B depend on the initial conditions.

The steady-state solution is given as:

$$x_{\text{steady}} = X \cos(\omega t) = \frac{F_0}{k} \frac{1}{1 - \omega^2/\omega_n^2} \cos(\omega t) \quad (2.3)$$

The amplitude of vibrations of the mass-spring system is given by:

$$|X| = \left| \frac{F_0}{k} \frac{1}{1 - \omega^2/\omega_n^2} \right| \quad (2.4)$$

The amplitude-frequency dependencies is shown in 2.1

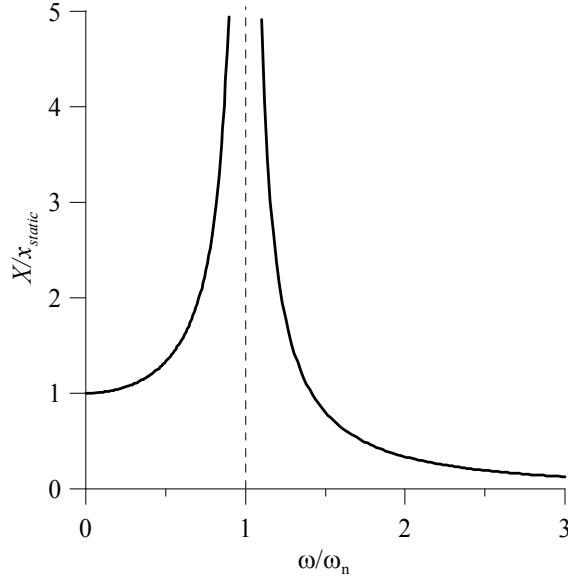


Figure 2.1: Amplitude-frequency characteristic. Extracted from [15, p. 2.2.2]

## 2.2 Introduction to the dynamics of railway bridges

[11] Dynamics of railway bridges is concerned with the study of deflections and stresses in railway bridges. The loads are represented by the moving wheel and axle forces, by means of which railway vehicles transmit their load and inertia actions to railway bridges. A survey of the dynamic effects of vehicles on railway bridges is given in Figure 2.2

### 2.2.1 Factors influencing dynamic behaviour

As stated in [9, p. 6.4.2] there are 11 factors influencing dynamic behaviour of a railway bridge. The principal factors which influence dynamic behaviour are:

- the speed of traffic across the bridge
- the span L of the element and the influence line length for deflection of the element being considered
- the mass of the structure
- the natural frequencies of the whole structure and relevant elements of the structure and the associated mode shapes (eigenforms) along the line of the track
- the number of axles, axle loads and the spacing of axles
- the damping of the structure
- vertical irregularities in the track
- the unsprung/sprung mass and suspension characteristics of the vehicle
- the presence of regularly spaced supports of the deck slab and/or track (cross girders, sleepers etc.)
- vehicle imperfections (wheel flats, out of round wheels, suspension defects etc.)
- the dynamic characteristics of the track (ballast, sleepers, track components etc.)

Other factors may include:

1. The track number of the bridge and their alignment.
2. Multiple trains running on bridge simultaneously.
3. Track alignment

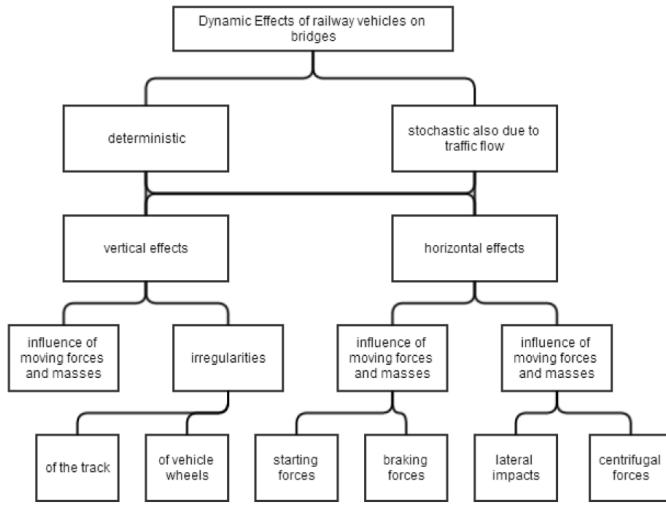


Figure 2.2: Dynamic effects of railway vehicles on bridges. Extracted from [11, p. 1.1]

## 2.3 Vertical Dynamic effects

As stated in EN 1991-2[9], the static stress and deformations (and associated bridge deck acceleration) induced in a bridge are increased and decreased under the effects of moving traffic by the following:

- the rapid rate of loading due to the speed of traffic crossing the structure and the inertial response (impact) of the structure,
- the passage of successive loads with approximately uniform spacing which can excite the structure and under certain circumstances create resonance (where the frequency excitation (or a multiple thereof) matches a natural frequency of the structure (or a multiple thereof), there is a possibility that the vibrations caused by successive axles running onto the structure will be excessive),
- variations in wheel loads resulting from track or vehicle imperfections (including wheel irregularities).

For determining the effects (stresses, deflections, bridge deck acceleration etc.) of rail traffic actions the above effects shall be taken into account.

### 2.3.1 Train Actions

See section 2.9

### 2.3.2 Wind Actions

The nature of the wind load is dynamic. This means that its magnitude varies with respect to time and space.

According to [16]: The limitations behind the applications of the EN-1991-1-4, Eurocode1, actions on structures-general actions-wind load-part 1-4, lead the structural designers to a great confusion. This may be due to the fact that, EC1 provides only the guidance for the bridges whose fundamental mode of vibrations have constant sign (e.g. simply supported structures) or a simple linear sign (e.g. cantilever structures) and these modes are the governing mode of vibrations of the structure; it analyzes only the along-wind response of the structure and not the cross wind response

and the simplified methods recommended in this code are covering only the structures with simple geometrical configurations.

## 2.4 Horizontal transverse dynamic effects

There's only one criterion in the Eurocodes mentioned that the bridge's first lateral natural frequency should no lower than 1.2 Hz.

However, as more and more long-span bridges are built nowadays, this requirement is not valid for more bridges. This is because, in general, the lateral natural frequency of a bridge decreases when span increases. For bridges with span longer than 100m, there's few bridge can have a lateral frequency higher than 1.2Hz, according to senior engineers' designing experience.

So it is vital to discuss horizontal dynamic effects for the sake of longer span bridges. In additional, study the requirements for horizontal vibration of railway bridges to make the results of dynamic analysis usable.

### 2.4.1 Sources induce transverse dynamic reactions

According to [18][11][9], following sources are identified:

- Horizontal track irregularities
- Sinusoidal motion of conical wheels along cylindrical rail heads
- Centrifugal forces on curved tracks
- Train switches

### 2.4.2 Horizontal vibration of a beam

Fryba[11] described the first two sources in mathematical terms: Consider a simply supported beam loaded by moving train loading, horizontal vibrations of the beam in a transverse direction  $w(x, t)$  are generated by lateral random forces  $H_n(t)$  due to random irregularities and sinusoidal motion. The differential equation for the deformation is:

$$EI_y w^{IV}(w, t) + \mu \ddot{w}(x, t) = \sum_{n=1}^N \varepsilon_n \delta(x + d_n - ct) H_n(t) \quad (2.5)$$

where  $H_n(t)$  stands for forces due to random irregularities and sinusoidal motion.  $H_n(t)$  is of a typically random character and can be replaced by horizontal transverse forces with zero mean values. See Figure. 2.3 for example.

[11, Table 9.1] gives sample of natural frequencies of steel truss bridges with open deck. Shown as follow Table 2.1. Please note that truss bridges have high stiffness.

#### 2.4.2.1 Centrifugal forces

In [9, p. 6.5.1] specifies following principles about centrifugal forces act on railway bridges:

Where the track on a bridge is curved over the whole or part of the length of the bridge, the centrifugal force and the track cant shall be taken into account.

The centrifugal forces should be taken to act outwards in a horizontal direction at a height of 1.80m above the running surface (see [9, Figure 1.1]). For some traffic types, e.g. double stacked containers, an increased value of  $h_t$  should be specified.

The centrifugal force shall always be combined with the vertical load. The centrifugal force shall not be multiplied by the dynamic factor  $\Phi_1$  or  $\Phi_3$ .

The characteristic value of the centrifugal force shall be determined according to the following equations:

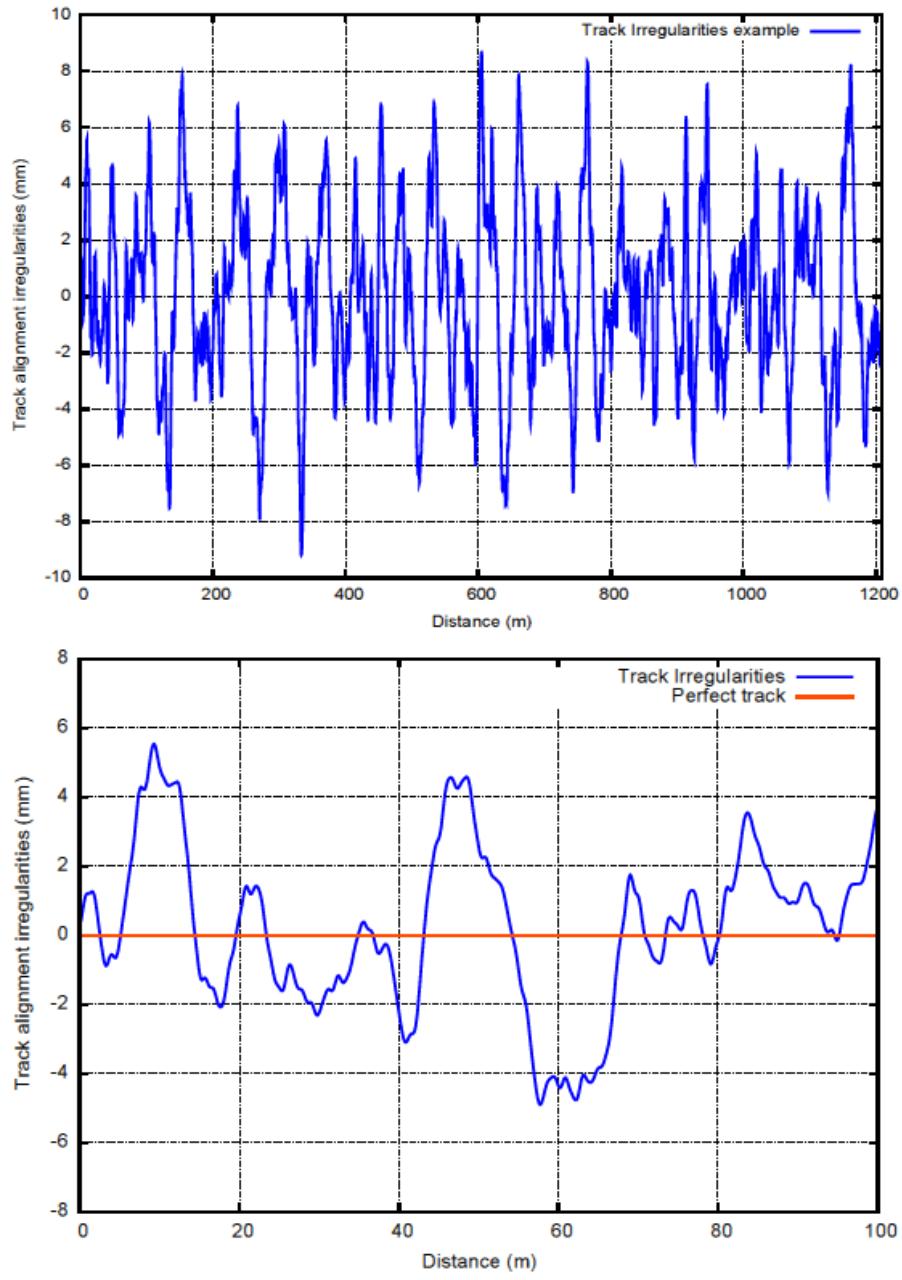


Figure 2.3: Example of a track lateral alignment irregularities profile for a track with low irregularities in a total length of 1209m. Extracted from [18]

Vibration Type	Symbol	j	Bridge with span	
			$l = 25.85m$	$l = 48.4m$
Vertical	$f_j$	1	8.7	5.4
		2	34.7	21.5
		3	78.1	48.3
Horizontal	$f_{hj}$	1	15.3	4.7
		2	61.1	19.0
		3	137.5	42.7
Horizontal	$f_{yj}$	1	14.8	4.7
		2	53.9	18.1
		3	107.4	38.8
Torsional	$f_{\varphi j}$	1	14.7	4.6
		2	52.5	17.5
		3	101.7	36.4
Torsional	$f_{\xi j}$	1	35.7	19.2
		2	71.3	38.4
		3	107.0	57.7
Torsional	$f'_{\varphi j}$	1	34.9	16.7
		2	73.5	35.4
		3	117.4	56.9
Torsional	$f'_{\varphi j}$	1	36.5	19.7
		2	77.6	41.9
		3	126.4	67.5

Table 2.1: Spatial vibrations of steel truss bridges with open bridge deck

$$Q_t k = \frac{v^2}{g \cdot r} (f \cdot Q_{vk}) = \frac{V^2}{127r} (f \cdot Q_{vk}) \quad (2.6)$$

$$q_{tk} = \frac{v^2}{g \cdot r} (f \cdot q_{vk}) = \frac{V^2}{127r} (f \cdot q_{vk}) \quad (2.7)$$

where:

- $Q_{tk}, q_{tk}$  Characteristic values of the centrifugal forces
- $Q_{vk}, q_{vk}$  Characteristic values of the vertical loads specified in [9, p. 6.3]
- $f$  Reduction factor, see below
- $v$  Maximum speed in accordance with [9, 6.5.1(5)][m/s]
- $V$  Maximum speed in accordance with [9, 6.5.1(5)][km/h]
- $g$  Acceleration due to gravity [9.81m/s<sup>2</sup>]
- $r$  Radius of curvature [m].

For Load Model 71 (and where required Load Model SW/0) the reduction factor  $f$  is given by:

$$f = [1 - \frac{V - 120}{1000} (\frac{814}{V} + 1.75) (1 - \sqrt{\frac{2.88}{L_f}})] \quad (2.8)$$

## 2.5 Torsional vibration

According to [11, p. 9.1.3], the horizontal lateral forces of railway vehicles act on the rail top level,i.e. outside the cross section centroid of the bridge in the majority of cases. Let the difference of elevations

be  $h$ . Consequently, they affect the bridge by twisting moments  $hH_n(t)$ . The differential equation of a beam due to simple torsion is

$$-GI_\xi\xi''(x,t) + \mu\ddot{\xi}(x,t) = \sum_{n=1}^N \varepsilon_n \delta(x + d_n - ct)hH_n(t) \quad (2.9)$$

where  $\xi(x,t)$  is the rotation about the longitudinal beam axis  $x$ ,  $G$  is the modulus of elasticity in shear,  $GI_\xi$  is the moment of torsional rigidity per unit length,  $\mu_\xi$  is the mass polar moment of inertia with regard to axis  $x$  per unit length.

## 2.6 Theoretical bridge models

According to [11, Chapter.2], theoretical models of railway bridges are of two types

- with continuously distributed mass
- with mass concentrated in material points(lumped mass)
- their combinations

### 2.6.1 Mass beams

The most common simplified model for bridge is a simply supported Euler-Bernoulli beam model(see Figure.2.4). The equation of motion of the beam expresses the equilibrium of forces per unit length:

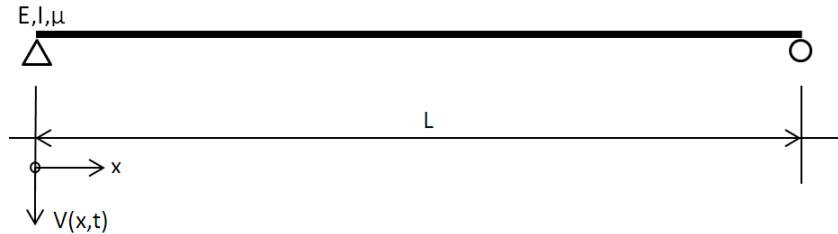


Figure 2.4: Mass beam model of span  $L$

$$EI \frac{\partial^4 v(x,t)}{\partial x^4} + \mu \frac{\partial^2 v(x,t)}{\partial t^2} + 2\mu\omega_b \frac{\partial v(x,t)}{\partial t} = f(x,t) \quad (2.10)$$

where:

1.  $v(x,t)$ :vertical deflection of the beam at the point  $x$  and at time  $t$
2.  $E$ :modulus of elasticity of the beam
3.  $I$ :moment of inertia of beam cross section
4.  $\mu$ :mass per unit length of the beam
5.  $\omega_b$ :circular frequency of viscous damping
6.  $f(x,t)$ :load at point  $x$  and time  $t$  per unit length of the beam

### 2.6.2 Continuous beam

The continuous beam model is suitable for multi-span bridges in general. The equation of motion is the same as simple beam model in Equation2.10.

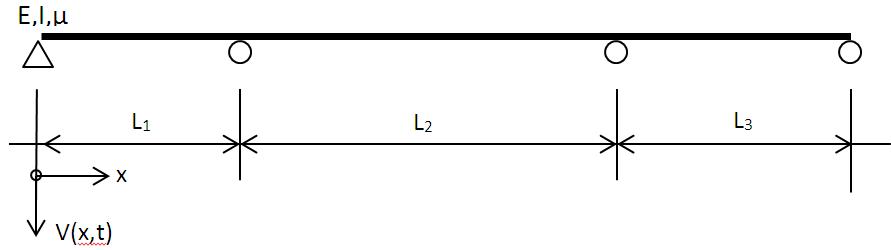


Figure 2.5: Continuous beam model

### 2.6.3 Complex systems

#### 2.6.3.1 Trusses

#### 2.6.3.2 Frames

#### 2.6.3.3 Curved bars

### 2.6.4 High strength steel bridges

The advantage of high strength steel bridges are

1. High quality material
2. Speed of construction
3. Versatility
4. Modification and repair
5. Recycling
6. Durability
7. Aesthetics

[14]: Use of high-strength steels for bridge construction in Japan dates back to the 1960s. Several hundred bridges been constructed using 500MPa and 600MPa yield strength steel, and steel with a normal yield strength of 800 MPa has also been used on several projects. In Europe, a variety of high-strength steels with yield strength from 460MPa to 690MPa are available for bridge applications. European structural steel standard EN 10025: 2004 grade S460ML, which has a nominal yield strength of 460MPa, can be welded at room temperature for plate thickness up to 90mm and has a specified minimum Charpy V-notch(CVN) every of 27 J at -50°C.

## 2.7 Modelling of railway vehicles

According to Newton's law, 2 basic load effects are produced by moving train: vertical forces due to vehicle weight, and inertia effects caused by vehicle acceleration. The loads on a railway bridge are very complex problems thus in engineering practice, loads are often simplified. But, the simplification depends on the purpose of the analysis.

### 2.7.1 Moving vertical forces model

If the inertia effects are neglected, loads of the moving trains can be modelled as moving vertical forces. For example, load diagram for type TALGO trains is shown in Figure.2.6 proposed by [17].

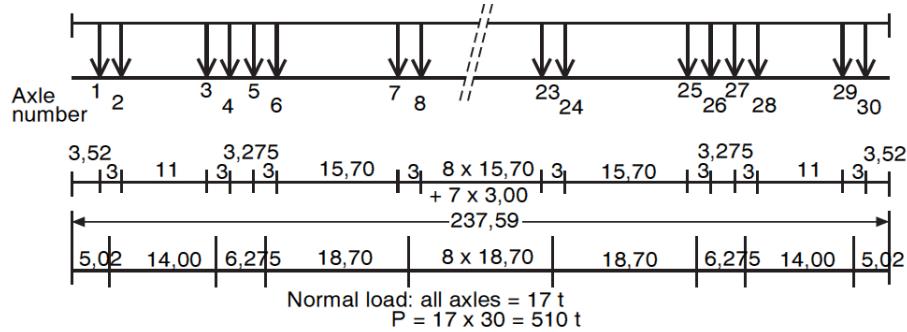


Figure 2.6: Moving vertical force model for TALGO trains

## 2.7.2 Advanced models

Nowadays more and more models have been proposed to meet different requirements of railway bridge dynamic analysis. The complexity of these models differs from each other but they are all more complicated than moving vertical forces models. For example, vehicle-bridge interaction model takes vehicle suspension system into account, which gives an alternative for discovering resonance effects between bridge and the vehicle suspension systems.

See Figure 2.7 for an example of advanced model.

## 2.7.3 Models proposed in Eurocodes

See Chapter 2.9.4

## 2.8 Track model

Proposed in [17, A.6.1.3], the track is represented by Timoshenko beam elements for the rails and takes account of the rail/sleeper fastening characteristics as well as the ballast(if one exists).

*"A sleeper is generally represented by two beam elements, with two covering the rail and one used for the deck. Sleepers and ballast are modelled as concentrated masses. They are linked to the nodes of the rail and the bridge by a parallel spring and damper system. The track can be modelled to any length on both side of the bridges, where the stiffening effect of the bridge has to be taken into account. The effects of track distribution are not considered. Each vehicle is able to absorb the kinetic energy of the bridge and it is for this reason that, at resonance, the deflections and accelerations of the bridge obtained with this model are lower than those obtained with a live load diagram."*

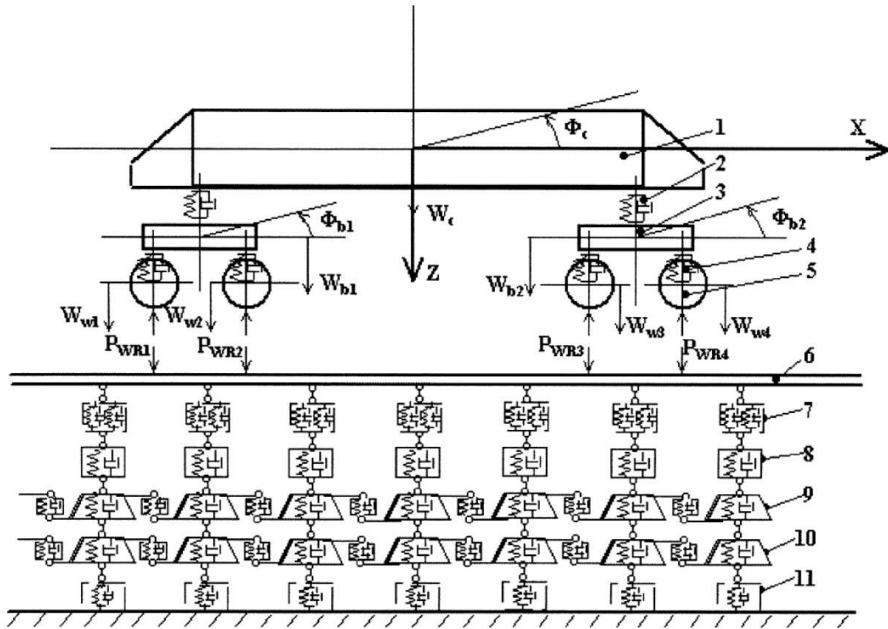
The most complete model for analysing train/track/bridge interaction is shown in Figure

## 2.9 General design principles and procedures concerning railway bridge dynamics proposed by Eurocodes

### 2.9.1 Requirements for railway bridge verification

[12] propose following requirements

1. Checks on bridge deformations shall be performed for traffic safety purposes for the following items:
  - vertical accelerations of the deck
  - vertical deflection of the deck throughout each span
  - unrestrained uplift at the bearings(to avoid premature bearing failure)



- 1 – Wagon Body ( $M_c$  – mass,  $J_c$  – inertial moment),
- 2 – Secondary Suspension ( $K_{sc}$  – stiffness coefficient,  $C_{sc}$  – damping coefficient),
- 3 – Bogie ( $M_b$  – mass,  $J_b$  – inertial moment),
- 4 – Primary Suspension ( $K_{pr}$  – stiffness coefficient,  $C_{pr}$  – damping coefficient),
- 5 – Wheelset ( $M_w$  – mass),
- 6 – Rail (Timoshenko Beam)
- 7 – Fastener and Pad ( $K_f$ ,  $K_p$  – stiffness coefficients,  $C_f$ ,  $C_p$  – damping coefficients),
- 8 – Sleeper ( $M_s$  – mass,  $K_{sl}$  – stiffness coefficient,  $C_{sl}$  – damping coefficient),
- 9 – Ballast ( $M_{bl}$  – mass,  $K_{bl}$  – stiffness coefficient,  $C_{bl}$  – damping coefficient),
- 10 – Subballast ( $M_{sb}$  – mass,  $K_{sb}$  – stiffness coefficient,  $C_{sb}$  – damping coefficient),
- 11 – Subgrade ( $K_{sg}$  – stiffness coefficient,  $C_{sg}$  – damping coefficient).

Figure 2.7: A dynamic model for the vertical interaction of the rail track and wagon system. Proposed in [19]

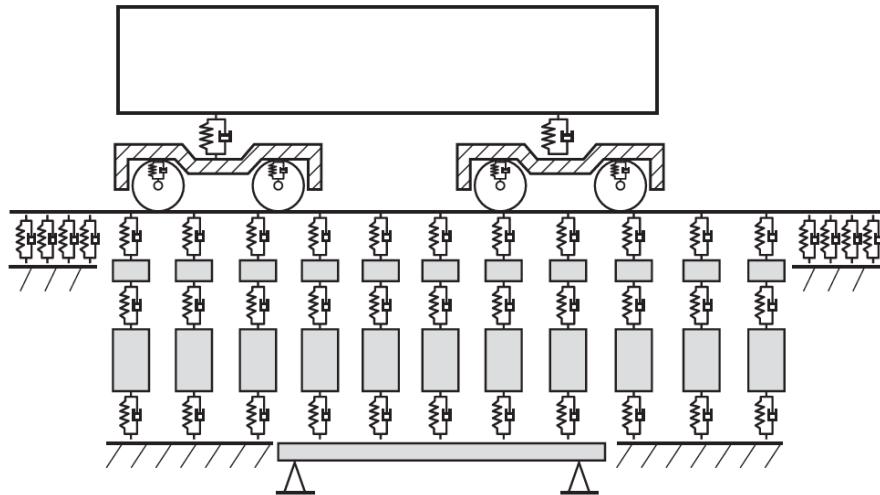


Figure 2.8: Diagram of the dynamic train-track-bridge model. Extracted from [17, Fig. 15]

- vertical deflection of the end of the deck beyond bearings(to avoid destabilising the track, limit uplift forces on rail fastening systems and limit additional rail stresses)
  - twist of the deck measured along the centre line of each track on the approaches to a bridge and across a bridge(to minimise the risk of train derailment)
  - rotation of the ends of each deck about a transverse axis or the relative total rotation between adjacent deck ends(to limit additional rail stresses, limit uplift forces on rail fastening systems and limit angular discontinuity at expansion devices and switch blades)
  - longitudinal displacement of the end of the upper surface of the deck due to longitudinal displacement and rotation of the deck end(to limit additional rail stresses and minimise disturbance to track ballast and adjacent track formation)
  - horizontal transverse deflection(to ensure acceptable horizontal track radii)
  - horizontal rotation of a deck about a vertical axis at ends of a deck (to ensure acceptable horizontal track geometry and passenger comfort)
  - limits on the first natural frequency of lateral vibration of the span to avoid the occurrence of resonance between the lateral motion of vehicles on their suspension and the bridge
2. Checks on bridge deformations should be performed for passenger comfort, i.e. vertical deflection of the deck to limit coach body acceleration in accordance with A2.4.4.3[12]
  3. The limits given in A2.4.4.2 and A2.4.4.3[12] take into account the mitigating effects of track maintenance (for example to overcome the effects of the settlement of foundations, creep, etc.)

### 2.9.2 Conceptual check

The conceptual check is to help designers avoid unsafe designs in conceptual stage. Once the bridge type and rough geometry is sketched, designers can easily know whether the bridge would have dynamic problem in the future.

For example, in [1, p. cl.8.7.4], it is stated that if the bridge meets criteria 2.11 , no dynamic analysis is necessary.

$$V > 200 \text{ km/h} \quad \delta_{dyn} \leq \text{ value given by the dynamic study, but } \delta_{stat} \leq L/2600 \quad (2.11)$$

On the other hand, no other conceptual check criterion is given for train speed under 200km/h.

Since there is higher possibility that, for bridges with span larger than 100m, they will have resonance with the normal trains.

### 2.9.3 Logic diagram

This logic diagram is used to determine whether a dynamic analysis is required, as shown in Figure 2.9a, is represented in [9] Chapter 6.4 Dynamic Effects, where  $V[Km/h]$  is the Maximum Line Speed at the site,  $L[m]$ , the span length,  $n_0[Hz]$ , the first natural frequency under permanent loads,  $n_T$ , the first natural torsional frequency for the same load,  $v[m/s]$  the maximum nominal speed and finally  $(v/n_0)_{lim}$ , as given in Annex F of EN1991-2. The frequency first of vibration,  $n_0$ , must be within the limits established in figure 2.9b.

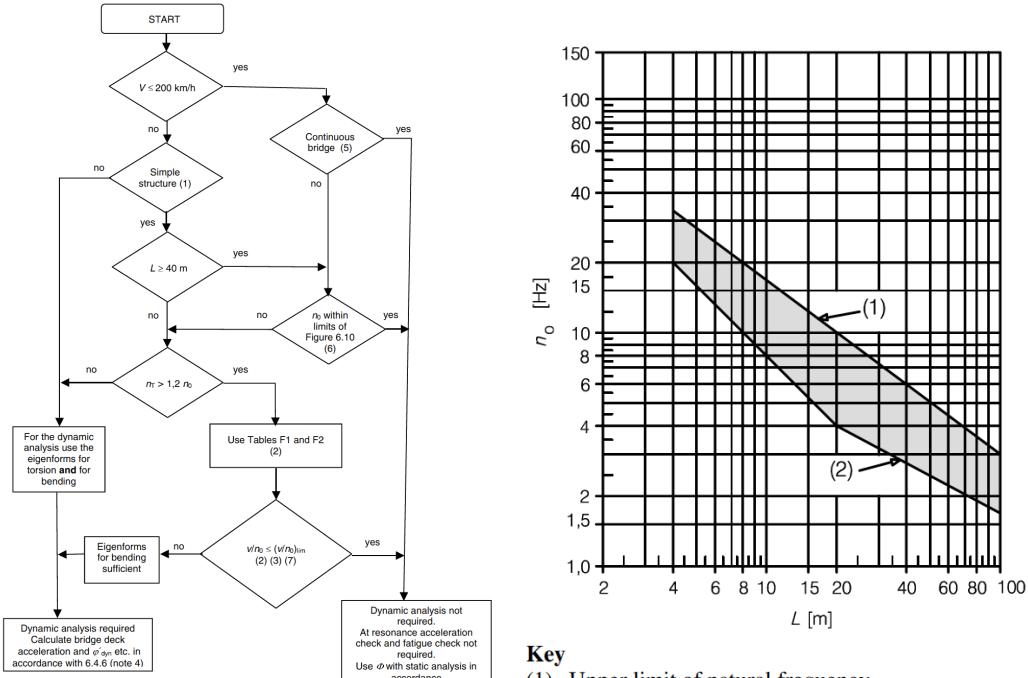
Checking through the logic diagram is regarded as first step is because designers can try to avoid designs that will be required to be dynamic analysed in the very beginning of the designing phase.

The upper limit (1) is defined as

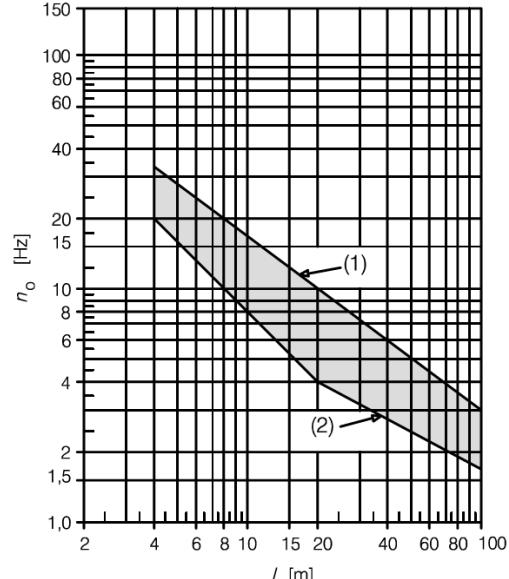
$$n_0 = 94,76L^{-0,748} \quad (2.12)$$

and the lower limit (2) as:

$$n_0 = \begin{cases} 80/L & \text{for } 4m \leq L \leq 20m \\ 94,76L^{-0,748} & \text{for } 20m \leq L \leq 100m \end{cases} \quad (2.13)$$



(a) Flow chart for determining whether a dynamic analysis is required. Extracted from EN1991-2[9]



(b) Limits for bridge natural frequencies,  $n_0[Hz]$ , as a function  $L[m]$ . Extracted from EN1991-2[9].

Figure 2.9: Logic diagram for determining whether dynamic analyses are necessary, extracted from [9, p. 6.4.4]

## 2.9.4 Train models

According to NEN 1991-2[9] and Designers Guide[1] Rail traffic actions are defined by means of load models. Four models of railway loading are given:

- LM71 and LM SW/0(for continuous bridges) to represent normal rail traffic on mainline railways (passenger and heavy freight traffic)
- SW/2 to represent abnormal loads or wagons
- LM 'unloaded train' to represent the effect of an unloaded train
- LM HSLM (comprising HSLM-A and HSLM-B) to represent the loading from passenger trains at speeds exceeding 200km/h.

### 2.9.4.1 Load Model 71

LM71 represents the static effect of vertical loading due to normal rail traffic.

The load arrangement and the characteristic values for vertical loads have to be taken as shown in Figure. 2.10

The actions listed below, associated with LM71, have to be multiplied by factor  $\alpha$ :

- equivalent vertical loading for earthworks and earth pressure effects
- centrifugal forces
- nosing force (multiplied by  $\alpha$  for  $\alpha \geq 1$  only)
- traction and braking forces
- derailment actions for accidental design situations
- Load Model SW/0 for continuous span bridges

*For international lines, it is recommended that a value of  $\alpha \geq 1.0$  is adopted. But this freedom of choice of the factor  $\alpha$  could lead to a non-uniform railway network in Europe. Therefore in UIC Code 702<sup>4</sup>  $\alpha = 1.33$  is generally recommended for all new bridges constructed for the international freight network, but not compulsory.*

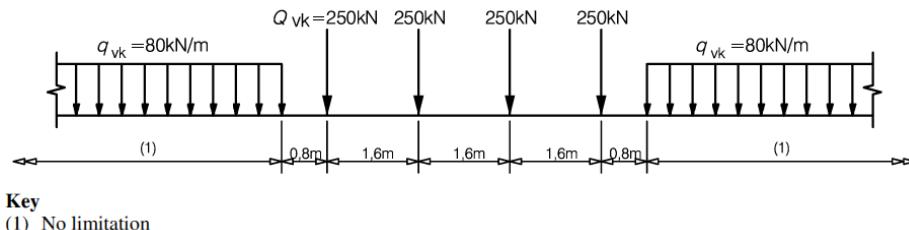


Figure 2.10: Load Model 71 and characteristic values for vertical loads. Extracted from EN1991-2[9]

### 2.9.4.2 Load Models SW/0 and SW/2

Load Models SW/0 represents the static effect of vertical loading due to normal rail traffic on continuous beams.

Load Model SW/2 represents the static effect of vertical loading due to heavy abnormal rail traffic.

The load arrangement is as shown in Figure. 2.11, with the characteristic values of the vertical loads according to Table.2.2

### 2.9.4.3 Load Model 'unloaded train'

From some specific verification purposes a specific load model is used, called 'unloaded train'. The Load Model 'unloaded train' consists of a vertical uniformly distributed load with a characteristic value of 10.0kN/m.

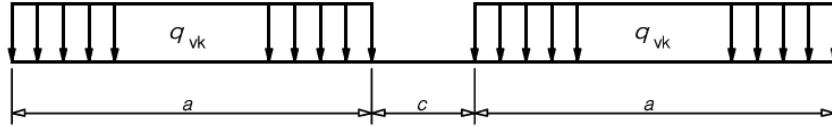


Figure 2.11: Load Models SW/0 and SW/2.. Extracted from EN1991-2[9]

Load model	$q_{vk}(kN/m)$	$a(m)$	$c(m)$
SW/0	133	15.0	5.3
SW/2	150	25.0	7.0

Table 2.2: Characteristic values for vertical loads for Load Models SW/0 and SW/2

#### 2.9.4.4 Load Models SHLM

Load Models HSLM comprises two separate universal *high-speed* trains with variable coach lengths. In order to ensure that they deliver dynamic behaviour with regards to current and future train traffic, bridges should be calculated using the Universal Dynamic Train(HSLM) consisting of HSLM-A and/or HSLM-B. These are defined as follows:

- For the definition of train HSLM-A, a set of ten reference trains A1 to A10: see Figure. 2.12 and Table. 2.3 below.
- For the definition of train HSLM-B: See Figure. 2.13 and ‘2.14 below

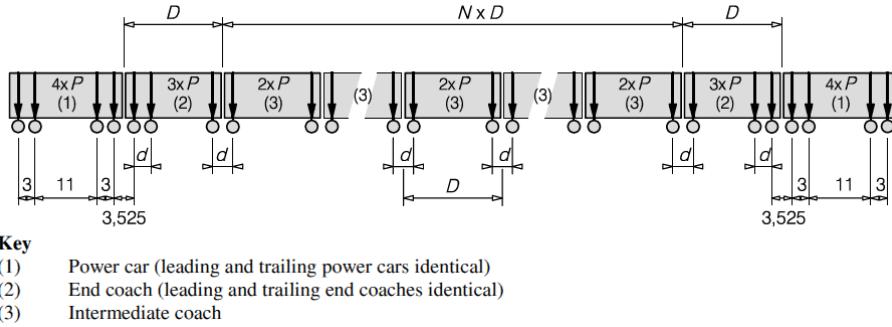


Figure 2.12: Diagram of Universal Dynamic Train HSLM-A. Extracted from EN 1991-2[9]

This Load Model comprises  $N$  number of point forces of 170kN at regular spacing  $d(m)$  (2.13) where  $N$  and  $d$  are defined in Figure. 2.14

Table. 2.4 illustrates how HSLM-A and HSLM-B are applied and indicates the train to be used for dynamic bridge calculations.

#### 2.9.5 Dynamic factor

The dynamic factor  $\Phi$  takes account of the dynamic magnification of stress and vibration effects in the structure but does not take account of resonance effects.

Generally the dynamic factor  $\Phi$  is taken as either  $\Phi_2$  or  $\Phi_3$  according to the quality of track maintenance as follows:

- (a) For Carefully maintained track:

$$\Phi_2 = \frac{1.44}{\sqrt{L_\Phi} - 0.2} + 0.82 \quad \text{with } 1.00 \leq \Phi_2 \leq 1.67 \quad (2.14)$$

Universal train	Number of intermediate coaches, $N$	Coach length $D(m)$	Bogie axle spacing $d(m)$	Point force $P(kN)$
A1	18	18	2.0	170
A2	17	19	3.5	200
A3	16	20	2.0	180
A4	15	21	3.0	190
A5	14	22	2.0	170
A6	13	23	2.0	180
A7	13	24	2.0	190
A8	12	25	2.5	190
A9	11	26	2.0	210
A10	11	27	2.0	210

Table 2.3: HSLS-A, definition of ten trains. Extracted from EN1991-2[9]

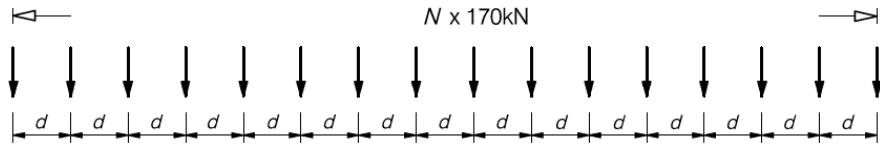


Figure 6.13 - HSLS-B

Figure 2.13: Diagram of Universal Dynamic Train HSLS-B. Extracted from EN 1991-2[9]

(b) For track with standard maintenance:

$$\Phi_3 = \frac{2.16}{\sqrt{L_\Phi} - 0.2} + 0.73 \quad \text{with } 1.00 \leq \Phi_3 \leq 2.0 \quad (2.15)$$

where  $L_\Phi$  is the ‘determinant’ length (length associated with  $\Phi$ ) in metres as defined in Table 6.2, EN1991-2[9].

## 2.9.6 Static analysis

A static analysis shall be carried out with the load models defined in Section Load Models(LM71 and where required Load Models SW/0 and SW/2). The result shall be multiplied by the dynamic factor  $\Phi$  (and if required multiplied by  $\alpha$ )

## 2.9.7 Bridge parameters

In designers’ guide[1], bridge parameters on dynamic effects are discussed.

### 2.9.7.1 Structural damping

Structural damping is a key parameter in dynamic analysis. The magnitude of the vibrations depends heavily on structural damping, especially in proximity to resonance.

### 2.9.7.2 Mass of the bridge

Maximum dynamic effects occur at resonance peaks, where a multiple of the load frequency coincides with the natural frequency of the structure. Underrating the mass will lead to over-estimation of the natural frequency of the structure and of the speed at which resonance occurs.

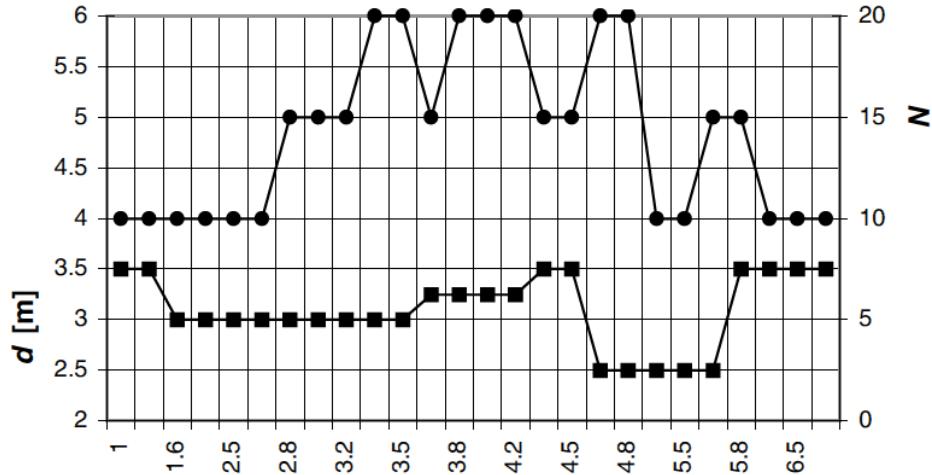


Figure 2.14: Universal Dynamic Train HSLSM-B. Extracted from EN 1991-2[9]

Structural configuration	Span	
	$L < 7m$	$L \geq 7m$
Simply supported span	HSLM-B	HSLM-A
Continuous structure or Complex Structure	Trains A1 to A10 inclusive	Trains A1 to A10 inclusive

Table 2.4: Application of HSLM-A and HSLM-B. Data extracted from EN 1991-2[9]

### 2.9.7.3 Stiffness of the bridge

Maximum dynamic load effects are likely to occur at resonant peaks when a multiple of the frequency of loading and a natural frequency of the structure coincide. Any overestimation of bridge stiffness will overestimate the natural frequency of the structure and speed at which resonance occurs; it provides conservative results.

### 2.9.8 Dynamic analysis

EN 1991-2[9] doesn't specify the method of dynamic analysing, but UIC Leaflet 776<sup>2</sup>[17] indicates that for normal bridges there are 4 methods of analysing(an approximate method and three simplified method). However, 776<sup>2</sup>[17] also indicates for special structures (bridges with long spans such as bowstring bridges), have to be solved using generic finite element (FEM) programs.

Analysing using FEM programs can be very time and money consuming providing a complicated structure. Fortunately UIC Leaflet 776<sup>2</sup>[17] is an optional reference so FEM analysing is not required in EN 1991-2 [9]. Thus finding a simplified way of analysing complicated bridge structures is vital. The development of dynamic analysing methods will be discussed in Chapter.??

### 2.9.9 Verification of the Limit States

[9, p. 6.4.6.5] proposes following principles to be followed during design:

To ensure traffic safety:

1. The verification of maximum peak deck acceleration shall be regarded as a traffic safety requirement checked at the serviceability limit state for the prevention of track instability
2. The dynamic enhancement of load effects shall be allowed for by multiplying the static loading by the dynamic factor  $\Phi$  defined in [9, p. 6.4.5]. If a dynamic analysis is necessary, the results of the dynamic analysis shall be compared with the results of the static analysis enhanced by

$\Phi$  (and if required multiplied by  $\alpha$  in accordance with [9, p. 6.3.2]) and the most unfavourable load effects shall be used for the bridge design.

3. If a dynamic analysis is necessary, a check shall be carried out according to [9, p. 6.4.6.6] to establish whether the additional fatigue loading at high speeds and at resonance is covered by consideration of the stresses due to load effects from  $\Phi \times LM71$  (and if required  $\Phi \times LoadModelSW/0$  for continuous structures and classified vertical load in accordance with [9, 6.3.2(3)] where required). The most adverse fatigue loading shall be used in the design.

#### 2.9.9.1 Ultimate limit states

#### 2.9.9.2 Serviceability limit states - traffic safety

**Vertical accelerations of the deck** The maximum peak values for bridge deck accelerations, calculated along each track, shall not exceed the following design values:

1.  $3.5m/s^2$  for ballasted track
2.  $5m/s^2$  for direct fastened track with tracks and structural elements designed for high speed traffic

Also, the range of frequencies to take into account in the determination of the dynamic response in terms of accelerations, shall not exceed the maximum of the following values:

1.  $30Hz$
2.  $1.5 \times n_0$
3. the frequency of the third mode of vibration of the member in study

**Deck twist** The twist of the bridge shall be calculated taking into account the characteristic values of Load Model 71 as well as SW/0 or SW/2 as appropriated multiplied by  $\phi$  and  $\alpha$  and Load Model HSLSM including centrifugal effects, all in accordance with EN1991-2, 6.

Twist shall be checked on the approach to the bridge, across the bridge and for the departure from the bridge (see A2.4.4.1(2)P).

The maximum twist  $t$  [mm/3m] of a track gauge  $s$  [m] of 1.435m measured over a length of 3m(Figure2.15) should not exceed the values given in Table:2.5

Speed range $V$ (km/h)	Maximum twist $t$ (mm/3m)
$v \leq 120$	$t \leq t_1$
$120 \leq v \leq 200$	$t \leq t_2$
$v > 200$	$t \leq t_3$

Note The values for  $t$  may be defined in the National Annex.

The recommended values for the set of  $t$  are:

$$t_1 = 4.5$$

$$t_2 = 3.0$$

$$t_3 = 1.5$$

Values for a track with a different gauge may be defined in the National Annex

Table 2.5: Limiting Values of deck twist

**Vertical deformations** For all the structures configurations, loaded with the classified characteristic vertical LM 71, or with the models SW/0 and SW/2 if required, the maximum total vertical deflection measured along any track due to railway traffic actions should not exceed  $L/600$ . The angular rotations at the end of decks, represented in Figure.2.16 , in the vicinity of expansion devices, switches and crossings, should be verified.

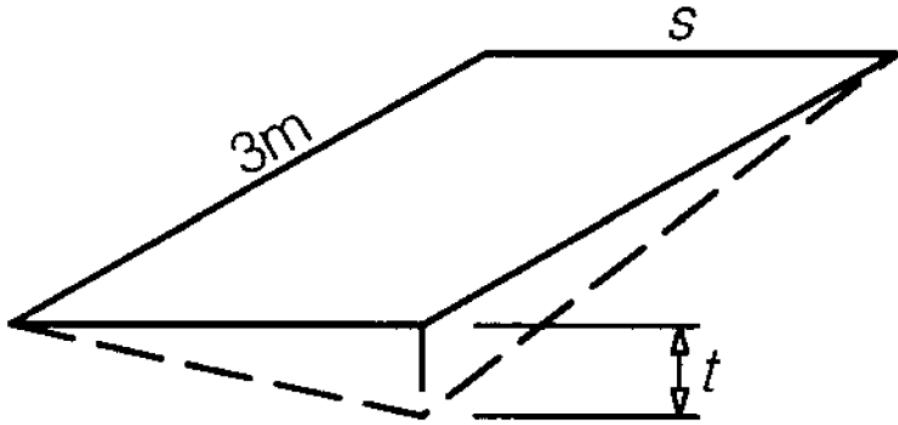


Figure 2.15: Definition of deck twist. Extracted from [7, Figure A2.1]

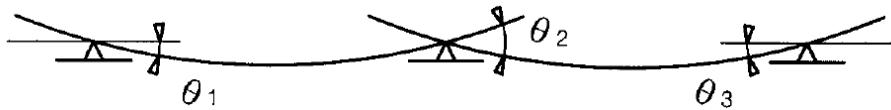


Figure 2.16: Definition of angular rotations at the end of the decks. Extracted from [7, Figure A2.2]

**Transverse deformations and vibrations** [7, A2.4.4.2.4] proposed that transverse deformation and vibration of the deck shall be checked for characteristic combinations of Load Model 71 and SW/0 as appropriate multiplied by the dynamic factor  $\phi$  and  $\alpha$  (or real train with the relevant dynamic factor if appropriate), wind loads, nosing force, centrifugal forces in accordance with [9, p. 6] and the effect of a transverse temperature differential across the bridge.

The transverse deflection  $\delta_h$  at the top of the deck should be limited to ensure:

1. a horizontal angle of rotation of the end of a deck about a vertical axis not greater than the values given in Table. 2.6 , or
2. the change of radius of the track across a deck is not greater than the values in Table. 2.6 , or
3. at the end of a deck the differential transverse deflection between the deck and adjacent track formation or between adjacent decks does not exceed the specified value

The first natural frequency of lateral vibration of a span should not be less than  $f_{h0}$ . The value for  $f_{h0}$  may be defined in the National Annex. The recommended value is:  $f_{h0} = 1.2\text{Hz}$

**Longitudinal displacements** For rails on the bridge and on the adjacent abutment, the permissible additional rail stresses due to the combined response of the structure and the track, due to variable actions, should be limited to the following design values:

1. Compression:  $72\text{KN/mm}^2$
2. Tension:  $92\text{KN/mm}^2$

When determining the combined response of track and structure to traction and braking forces, these forces should not be applied on the adjacent embankment unless a complete analysis is carried out considering the approach, passage over and departure from the bridge of rail traffic on the adjacent embankments to evaluate the most adverse load effects.

For the determination of load effects in the combined track/structure system a model based upon Figure 2.17 may be used where the longitudinal load/displacement behaviour of the track or rail supports may be represented by the relationship shown in Figure 2.18.

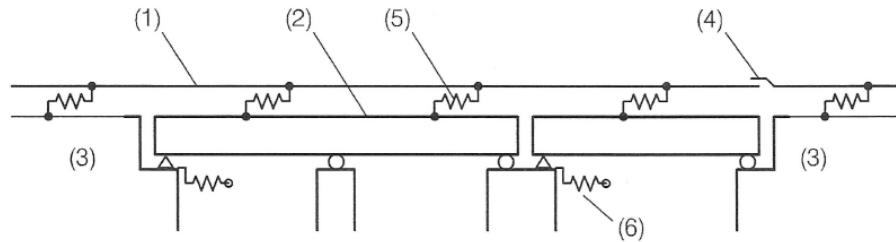
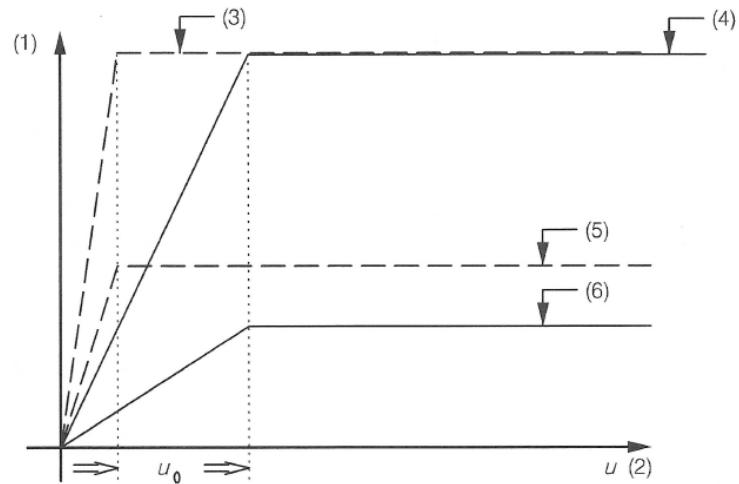


Figure 2.17: Model of a track/structure system. Extracted from [9, Figure 6.19]



#### Key

- (1) Longitudinal shear force in the track per unit length
- (2) Displacement of the rail relative to the top of the supporting deck
- (3) Resistance of the rail in sleeper (loaded track)  
(frozen ballast or track without ballast with conventional fastenings)
- (4) Resistance of sleeper in ballast (loaded track)
- (5) Resistance of the rail in sleeper (unloaded track)  
(frozen ballast or track without ballast with conventional fastenings)
- (6) Resistance of sleeper in ballast (unloaded track)

Figure 2.18: Variation of longitudinal shear force with longitudinal track displacement for one track. Extracted from [9, Figure 6.20]

Speed range V(km/h)	Maximum horizontal rotation(radian)	Maximum change of radius of curvature	
		Single deck	Multi-deck bridge
$V \leq 120$	$\alpha_1$	$r_1$	$r_4$
$120 \leq V \leq 200$	$\alpha_2$	$r_2$	$r_5$
$V > 200$	$\alpha_3$	$r_3$	$r_6$

NOTE 1 The change of the radius of curvature may be determined using:

$$r = \frac{L^2}{8\delta_h}$$

NOTE 2 The transverse deformation includes the deformation of the bridge deck and the substructure(including piers, piles and foundations).

NOTE 3 The values for the set of  $\alpha_i$  and  $r_i$  may be defined in the National Annex. The recommended values are:

$$\begin{aligned}\alpha_1 &= 0.0035; \alpha_2 = 0.0020; \alpha_3 = 0.0015; \\ r_1 &= 1700; r_2 = 6000; r_3 = 14000; \\ r_4 &= 3500; r_5 = 9500; r_6 = 17500\end{aligned}$$

Table 2.6: Maximum horizontal rotation and maximum change of radius of curvature

Model in Figure2.17 is very important to evaluate the security of the track structure and not the structural security. High track deformations can lead to unfavourable effects for the structure and for vehicles when these are crossing the bridge.

### 2.9.9.3 Serviceability limit states - passenger comfort

For these type of verifications [7] defines the limiting values for the maximum vertical deflection for passenger comfort, as following:

1. Comfort criteria
  2. Deflection criteria for checking passenger comfort
  3. Requirements for a dynamic vehicle/bridge interaction analysis for checking passenger comfort
- Passenger comfort depends on the vertical accelerations,  $b_v$ , inside the coach. These levels of comfort limiting values for the vertical accelerations are presented in Table.2.7

Very good	$1.0m/s^2$
Good	$1.3m/s^2$
Acceptable	$2.0m/s^2$

Table 2.7: Recommended accelerations values to ensure the respective levels of comfort. Extracted from [7, Table A 2.9].

In order to limit vertical vehicle acceleration, being the limits defined in Table2.7,vertical displacements should be less than the maximum permissible vertical deflection,  $\delta$ , obtained from Figure2.19. These values are expressed in function of the span length  $L[m]$ , and train speed  $V[km/h]$ , which is valid only for railway bridges with three or more successive simply supported spans. Alternatively these accelerations can be determined considering the vehicle-structure interaction dynamic analysis.

Additionally, the limiting values of  $L/\delta$ , defined in Figure2.19 are given for  $b_v = 1.0m/s^2$ .

Vertical deflections should be determined with the LM 71 model multiplied by the factor  $\phi$  and adopting  $\alpha = 1$ , being only one track loaded for the case of bridges with two or more tracks.

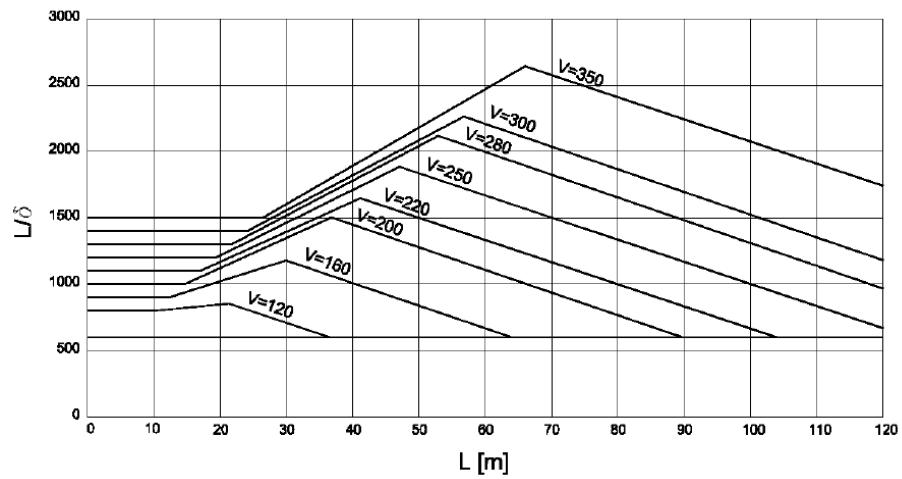


Figure 2.19: Maximum permissible vertical deflection  $\delta$  for railway bridges with 3 or more successive simply supported spans corresponding to a permissible vertical acceleration of  $b_v = 1m/s^2$  in a coach for speed  $V$ [km/h]. Extracted from [7, Figure A2.3]

# Chapter 3

## Train vehicles layouts and geometry

### 3.1 Locomotives

#### 3.1.1 4-axle locomotives

Generally, the relevant parameters for categorisation of 4-axle locomotives are axle load P (18 t to 22,5 t) and the bogie axle spacing (2,2 m to 3,4 m).

Typically the mass per unit length is less than 6,4 t/m and the distance from the end axle to the end of the nearest coupling plane is greater than 1,9 m

#### 3.1.2 6-axle locomotives

Generally, the relevant parameters for categorisation of 6-axle locomotives are:

- the maximum axle load P (18 t to 22 t) in combination with;
- the distance between axles within a bogie (1,80 m to 2,25 m).

Typically, the mass per unit length (p) is less than 6,4 t/m and the distance from end axle to the end of the nearest coupling plane (a) is greater than 2,1 m.

### 3.2 Passenger carriages

### 3.3 Wheelset and track dimensions

Generally the track gauge is used as a distance measured between the two rails, more specifically the distance between the inside of the railheads measured 14mm below the surface of the rail. By choosing 14 mm the measurement is less influenced by lipping or lateral wear on the rail head and by the radius  $r = 13$  mm of the rail head face. On normal track the gauge is  $1435_{-3}^{+10}$  mm with a maximum gradient of 1:3000. For new track, however, NS apply the following standards:

1. Mean gauge per 200 m:  $1435_{-1}^{+10}$  mm
2. Standard deviation within a 200 m section less than 1 mm

### 3.4 Conicity and Equivalent Conicity of Wheels

Originally conical tire profiles with an inclination of 1:20 were used. Since a centrally applied load on the railhead is desired, a rail inclination of 1:20, as shown in Figure 2.1, was also selected;

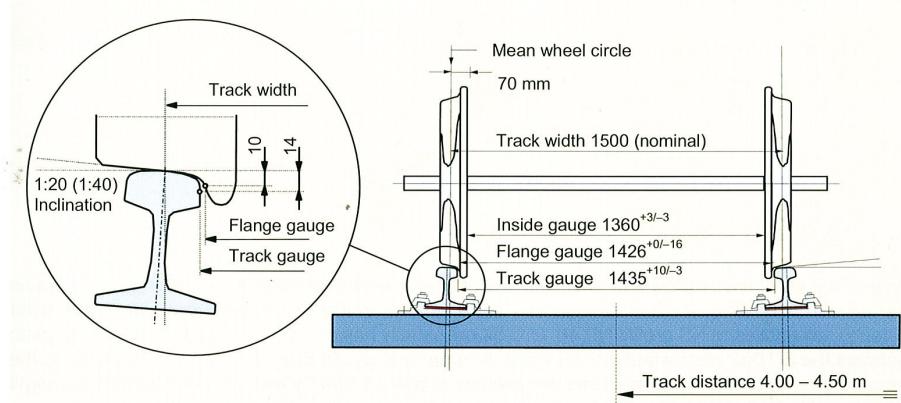


Figure 3.1: Wheelset and track dimensions for straight normal gauge track. Extracted from [8, p.17]

this for instance still applies to NS profile NP 46. UIC 54 rail usually has an inclination of 1:40. This inclination matches the S 1002 worn wheel profile which is in general use in Europe. During manufacturing the tires are given a profile which matches the average shape caused by wear. In contrast to the straight conical profile this has a hollow form.

It is clear that regarding a worn profile the conicity depends on the actual shape of the rail head and tire, including any wear, track gauge, and rail inclination. Likewise, elastic deformation of the wheelset and rail fastenings plays a role.

Generally, the effective or equivalent conicity is defined as:

$$\gamma_e = \frac{\Delta r}{2y} = \frac{r_1 - r_2}{2y}$$

Here  $r_1 - r_2$  is the instantaneous difference in rolling radius of the wheel treads; generally speaking this is a non-linear function of the lateral displacement  $y$  of the wheelset with respect to the central position. The difference between conical and worn profiles is given in Figure 3.2. To enable numerical comparisons  $\gamma_e$  is determined at a certain lateral displacement  $y = \bar{y}$ .

With a conical profile the conicity is constant and above equation becomes:

$$\gamma_e = \frac{\Delta r}{2y} = \frac{(r + \gamma y) - (r - \gamma y)}{2y} = \gamma$$

### 3.5 Worn wheel profiles

A perfectly conical wheel profile is unstable as far as its shape is concerned, but will take on a shape that is stable as the effect of wear.

Practical research has shown that over a period of time wheel profiles stabilise with wear at an equivalent conicity of 0.2 to 0.3. With regards to running stability, the equivalent conicity must remain below 0.4 and to ensure the centering effect it must be greater than 0.1.

### 3.6 Trains in Netherlands

Passenger trains now in service include following models:

1. The DD-AR (Dubbeldeksaggloregiomaterieel)

EMUs were delivered as DDM-2/3 resembling the bilevel rail cars series DDM-1 from 1985 and operates in fixed formations of 3 or 4 coaches. 4 car trains use a class 1700 locomotive

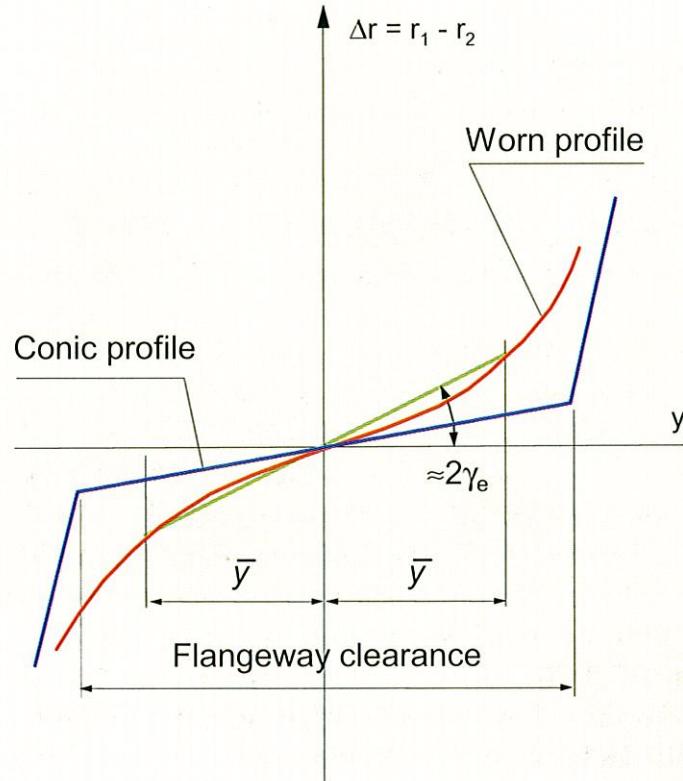


Figure 3.2:  $y - \Delta r$  curves. Difference between conical and worn wheel profiles. Extracted from [8, p. 2.4]

for traction, 3 car trains use an mDDM motorcar, which resembles a DD-AR driving trailer but has electric motors and a single passenger deck on top; the level of this deck is higher than that of a regular single deck rail car, but lower than the upper deck of the other coaches. Three types of coaches are available: Bv (second class), ABv (first and second class) and Bvk (second class driving trailer). The DDM-2/3 series are being modernised from 2010/2013 and after modernisation the series was renamed as NID (Nieuwe Intercity Dubbeldekker).

## 2. The VIRM (Verlengd Interregiomaterieel)

also called Regiorunner was partially rebuilt from trainsets DD-IRM (Dubbeldeks Interregiomaterieel). DD-IRM was delivered in 3- and 4-car trainsets. 3-car trainsets got one extra coach, 4-car trainsets got two extra coaches. Also, new 4- and 6-car trainsets were built. Thus, a train consists of one or more combinations of 4 or 6 double deck coaches; each combination (multiple unit) has electric motors. More than three hundred coaches are currently operative in the Netherlands.

## 3. The Koploper (ICM) (Intercitymaterieel)

is a 3- or 4-car multiple unit that when coupled with another one, allows passengers to walk through (the name Koploper being a play on words literally "head walker", but in actual use meaning "front runner"). The Dutch Railway Company decided to close the heads permanently on 31 October 2005 because the mechanism broke down too often. A scheduled modernisation of around 7 million euro will see the ICM fleet updated. The renovated ICM trains provide 13% more seats (reducing the leg room to uncomfortable small for the long haul journeys they serve in 2nd class, which is further aggravated by a waste bin that is placed on the backsides of

the seats in front), have a new interior, a bathroom accessible by wheelchairs, airconditioning as well as upgrades to the engine and connection systems. The head doors are removed. Also, these (renovated) trains are the first trains in the NS fleet equipped with OBIS. OBIS provides a (free) WiFi-connection on board, along with in-train journey information provided through screens and (automated) vocal announcements through the trains speakers. This journey information provides the actual status, and thus is always up-to-date to the actual situation this trip, and the stations is passes.

#### 4. The Sprinter (SGM, Stads Gewestelijk Materieel)

is a two or three car electric, used on small distances. They are named Sprinter because they're able to accelerate and brake quite fast, making them very suitable for 'stoptrein' services. They were also specifically designed for urban environments where they run commuter services. As a result, they are most commonly found in the Randstad area. The initial idea was that the Sprinter would provide somewhat of a subway/metro service but this plan failed as the cities of Amsterdam and Rotterdam continued to construct their own rapid transit systems. Nevertheless, in the densely populated Randstad, the Sprinters remain popular. Two car versions were revised and renamed to Citypendel. All Sprinters are now refurbished into the new white/yellow/dark blue livery.

All of passenger train coaches have a wheel diameter of 920 mm.

Locomotives of freight trains have wheel diameter of 1000 mm.

# Chapter 4

## Parametric Study

### 4.1 Effects investigated in Parametric Study

Effects investigated in this report will be the same effects investigated in DT 329, which is described in Sec.???. However, according to the statement in Sec.2.3[Summary of results] in the same report,

Even when the axle repeat frequency matches the first lateral bending mode of each span, there is no evidence that the resonant behaviour of the span and train has any effect on subsequent spans, since the resonant effects do not appear to grow from span to span.

the third investigated resonance effect 'coincidence between the length of the span and the kinematic wavelength of the trailing vehicles' is neglected in this thesis.

### 4.2 Speed range of dynamic consideration

The thesis will focus on normal speed trains because IV-Groep is only interested in normal speed train lines, which its projects are being built for. Higher boundary is set according to maximum speed allowed on Dutch railways, while lower boundary is set according to an estimation. The reason for a lower boundary speed is that dynamics issues for railway infrastructures increase with respect to vehicle speed, which means generally less concern is needed when the speed of train is low. It's certain that there exists a threshold of speed for every type of train that dynamic behaviour of them start to be a problem to concern but this threshold of speed varies from different scenarios. Till now no solid research can give a value to this threshold of speed so this is why estimation of lowest speed is adopted.

My assumption of lower boundary of speed for consideration is 120 km/h. This is still very conservative because according to logic diagram Eurocode 1991-2[9], dynamic check is always needed when  $V \geq 200\text{km/h}$  for vertical direction. Under same speed, the kinematic energy passed to bridge by running vehicle in vertical direction is apparently higher than that in lateral direction on a straight track. So it is believed that in lateral direction, the threshold will be even higher than 200 km/h according to Eurocode 1991-2. Value 120 km/h is taken according to [10, Appendix F]: *Speeds which do not require dynamic compatibility checks* where 120 km/h is the lowest speed can be found in the table, excluding special vehicle. This table is attached in this thesis in Appendix.C.

Upper boundary for consideration:

Normal trains running on Dutch railway has a speed limit of 160 km/h so the upper boundary for speed is set to 160 km/h, which is also 44.44 m/s.

$$V_{max} = 44.44\text{m/s}$$

Lower boundary for consideration:

$$V_{min} = 33.33m/s$$

However, it is strongly advised that future research investigate the lowest speed threshold for dynamic for consideration for train vehicles in the Netherlands due to the fact that the lower boundary used in this thesis is an assumption.

### 4.3 Equivalent conicity used in this study

According to [8, Section.2.6],

Practical research has shown that over a period of time wheel profiles stabilise with wear at an equivalent conicity of 0.2 to 0.3. With regards to running stability, the equivalent conicity must remain below 0.4 and to ensure the centering effect it must be greater than 0.1.

conicity range will be 0.2 to 0.3.

It is suggested that vehicle maintenance sector ensure wheels of train wheels stay in the safe zone of conicity.

### 4.4 Parametric Study on Frequency of Klingel movement

Klingel movement is proposed by Klingel which can well predict the moving trend of a single wheelset on a straight railway track. However, the kinematic movement of a certain wheelset assembled into a running train is different from the movement of a single free wheelset. This is due to multiple bodies interact with each other, introducing more complicated mechanism in wheel/rail interaction.

This parametric study focuses on Klingel movement of a single wheelset. First part of the parametric study will try to discuss the relationship of Kiingle frequency of a wheelset and kinematic movement frequency of a whole train. Second part of the parametric study will use realistic data of Dutch railway/vehilces to assess the frequency bandwidth of Dutch native trains.

Parametric to be studied:

Speed of train, radius of the wheel and conicity of the wheel.

Gauge distance is fixed to 1435mm according to UIC standard.

Frequency is linear to speed if other parameters are fixed.

### 4.5 Comparison between Klingle movement and train kinematic movement studied in D181 DT329

By comparing the result from above parametric study and kinematic wavelength obtained by D181, presented as table.C2 in original report, parametric study results show close prediction for kinematic wavelength of freight train locomotive/coach/wagon. It's because freight train suspension system is simpler and stiffer compared to passenger train's, making the behaviour of train acts more similar to the behaviour of a single wheelset of bigger mass. However, results of wavelength of single wheelset is 33% shorter than kinematic wavelength of train because suspension system of passenger coaches are much more sophisticated. The wavelength of passenger coach is highly related to the characteristics of its suspension systems. These data is often difficult to obtain.

The train parameter used in this part of parametric study is attached in the Appendix.D.

Table 4.1: Add caption

	Speed	Gauge	Base wheel distance	Radius	Conicity
<b>BR CLASS 56 LOCOMOTIVE</b>	1.0000	1435.0000	1500.0000	290.0000	0.0500
<b>FS E444 LOCOMOTIVE</b>	1.0000	1435.0000	1500.0000	550.0000	0.0500
<b>FS ETR500 LOCOMOTIVE</b>	1.0000	1435.0000	1500.0000	550.0000	0.0500
<b>UIC FREIGHT WAGON (LADEN)</b>	1.0000	1435.0000	1500.0000	460.0000	0.0500
<b>FS ETR500 COACH</b>	1.0000	1435.0000	1500.0000	440.0000	0.0500
<b>UIC COACH</b>	1.0000	1435.0000	1500.0000	445.0000	0.0500
	1.0000	1435.0000	1500.0000	500.0000	0.0250
	1.0000	1435.0000	1500.0000	500.0000	0.2000
	1.0000	1435.0000	1500.0000	500.0000	0.3000
	1.0000	1435.0000	1500.0000	460.0000	0.0250
	1.0000	1435.0000	1500.0000	460.0000	0.2000
	1.0000	1435.0000	1500.0000	460.0000	0.3000

## 4.6 Assess of frequency bandwidth based on realistic data of Dutch Rail/Vehicle

The wavelength of passenger coach is highly related to the characteristics of its suspension systems. These data is often difficult to obtain. To establish an easy approach, wavelength of passenger train calculated in this section will be multiplied by an amplification factor of 1.5. This value is obtain by train wavelength/wheelset wavelength ratio in previous parametric study. Please note this factor is an estimation. However, wavelength of freight train is not modified due to the conclusion that freight train's suspension system is stiff enough for the Kingle movement of a single wheelset to describe the kinematic movement of a whole freight train.

However, future research is highly recommended to be conducted to study the kinematic wavelength of complete vehicles in the Netherlands, using realistic data of their suspension systems.

Klingel's formula: Klingel has done experiments and has given that the wavelength of a single wheelset:

$$\lambda_0 = 2\pi \sqrt{\frac{rG}{2\gamma}}$$

where:

G = Dynamic Gauge

r = Dynamic Wheels Radius

g = Conicity

For 2 wheelsets connected by a bogie:

$$\lambda = \lambda_0 \sqrt{1 + \left(\frac{I}{G}\right)^2}$$

where:

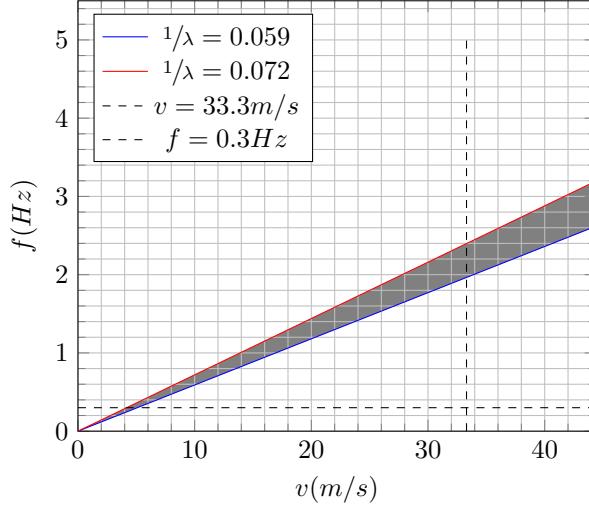
I = Rigid wheel base

Thus range of  $\lambda$  is obtained by inputting data discussed in previous sections of this chapter.

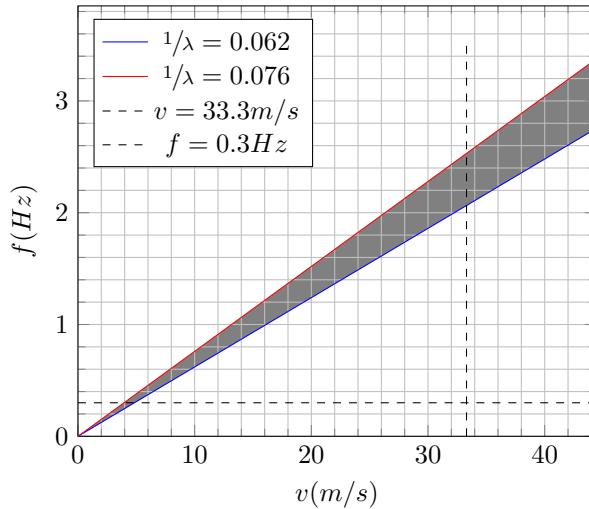
Following plots are generated according to linear relationship between frequency and speed:

$$f = v \frac{1}{\lambda}$$

Lateral frequency of freight train with respect to speed



Lateral frequency of passenger train with respect to speed



## 4.7 Parametric study on frequency caused by axle repeat pattern

Following wavelength of axle repeat pattern is obtained by extracting MU standards in [10]. Detailed information can be found in Appendix.D

Range of wavelength(m) of first repeat pattern mode:

$$\lambda_{1stRepeat} \in (9.2, 9.8) \cup (12.8, 13.5) \cup (14.9, 16) \cup (17, 17.5) \cup (18.7, 19.5) \cup (23.8, 27.5)$$

Range of wavelength(m) of second repeat pattern mode:

Table 4.2: Wavelength of axle repeat pattern(m)

Type	L_coa min	L_coa max	2*L_coa min	2*L_coa max
<b>CB_1</b>	23.8	25.3	47.6	50.6
<b>CB_2</b>	25.3	27.5	50.6	55
<b>AB_1</b>	14.9	16	29.8	32
<b>AB_2</b>	18.8	19.5	37.6	39
<b>AB_3</b>	17	17.5	34	35
<b>AB_4</b>	18.7	19.2	37.4	38.4
<b>SA_1</b>	9.2	9.8	18.4	19.6
<b>SA_2</b>	12.8	13.5	25.6	27

$$\lambda_{2ndRepeat} \in (18.4, 19.6) \cup (25.6, 27) \cup (29.8, 32) \cup (34, 35) \cup (37.4, 39) \cup (37.6, 55)$$

Range of frequency(Hz) yielded by dividing 1m/s with wavelength of first repeat pattern mode:

$$\frac{1}{\lambda_{1stRepeat}} \in (0.036, 0.042) \cup (0.051, 0.053) \cup (0.057, 0.059) \cup (0.063, 0.067) \cup (0.074, 0.078) \cup (0.102, 0.109)$$

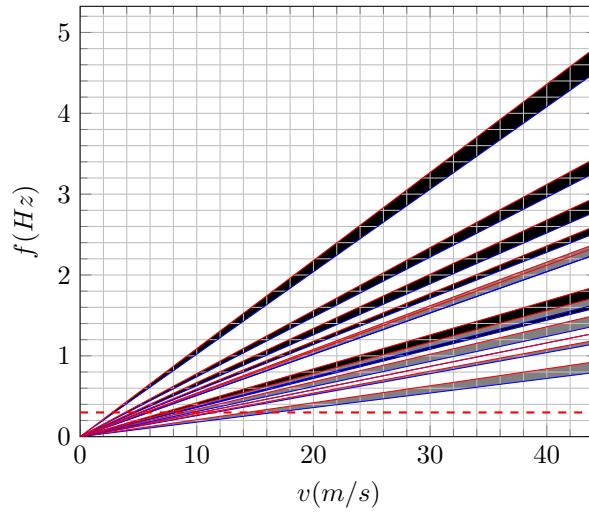
Range of frequency(Hz) yielded by dividing 1m/s with wavelength of second repeat pattern mode:

$$\frac{1}{\lambda_{2ndRepeat}} \in (0.018, 0.021) \cup (0.026, 0.027) \cup (0.029, 0.029) \cup (0.031, 0.034) \cup (0.037, 0.039) \cup (0.051, 0.054)$$

Following plot is generated according to linear relationship between frequency and speed:

$$f = v \frac{1}{\lambda_{repeat}}$$

Two repeat pattern frequencies with respect to speed



## 4.8 Conclusion of parametric study

This parametric study illustrates a tool of obtaining danger zone for train speed for a specific bridge first lateral natural frequency.

For example, if a bridge has a first lateral natural frequency of 0.3Hz:

1. To avoid resonance between freight train and bridge caused by kinematic frequency coincidence, speed of train should be regulated above 4m/s
2. To avoid resonance between passenger train and bridge caused by kinematic frequency coincidence, speed of train should be regulated above 5m/s
3. To avoid resonance caused by axle repeat pattern, speed of train should be regulated above 16m/s

As a conclusion, if the speed of train is regulated above 16m/s, there won't be resonance between bridge and train.

## Chapter 5

# Recommendations on improvement on Eurocode

### 5.1 Another criterion proposed by J.J.Reber

A SNCB bridge, driver complains  
To be used in combination with Sijian Deng's criterion.

### 5.2 Misc

Define eigen frequency and natural frequency  
Define how-to-do and what-to-do a dynamic analysis

### 5.3 Improve criteria for lateral bridge dynamics

Following sections provides several orientations for improving criteria for lateral bridge dynamics in terms of safety and serviceability of running stock.

#### 5.3.1 Requirements for traffic safety(horizontal)

Requirements other than bridge first lateral frequency higher than 1.2Hz. Since there's no further requirements mentioned by Eurocode, following requirements are gathered from other European codes, eg. British standards, UIC leaflet, etc.

- Requirements regarding traffic safety for vehicles
  - (a) Guiding Force: [2] , [6] and[5] propose safty limitations against railway vehicle overturning. From[6] the maximum guiding force for a vehicle with a load per axle of 170kN(AVE) is 66kN per axle and 48kN per axle for a vehicle with a load per axle of 112kN(ICE2). For the R1 freight wagon(load per axle of 245kN), the maximum guiding force per axle is 78kN.
  - (b) Maximum lateral acceleration of the railway vehicle: proposed by [21]
- Requirements regarding safety for bridge
  - [12] A2.4.4.1(2): Horizontal transverse deflection(to ensure acceptable horizontal track radii) and horizontal rotation of a deck about a vertical axis at ends of a deck(to ensure acceptable acceptable horizontal track geometry and passenger comfort)

### 5.3.2 Requirements for traffic safety on derailment: Railway vehicle derailment mechanism and safety criteria

Derailment mechanisms

1. vehicle resonant response
2. lateral instability
3. vehicle overturning
4. vertical wheel unloading
5. flange climb
6. rail roll-over
7. track panel shift
8. longitudinal train forces

The four types of derailment: flange climb derailment, derailment caused by gauge widening and rail roll-over, derailment caused by track panel shift, derailment cause by vehicle lateral instability have a common cause of high lateral force at the wheel-rail interface. According to [13, Chapter 8, IV] any conditions that lead to high lateral forces or lead to lower the ability of the system to sustain the force should be corrected.

#### 5.3.2.1 Flange climb derailment

Wheel flange climb derailments are caused by wheels climbing onto the top of the railhead then further running over the rail. Wheel climb derailments generally occur in situations where the wheel experiences a high lateral force combined with circumstances where the vertical force is reduced on the flanging wheel. The high lateral force is usually induced by a large wheelset angle-of-attack. The vertical force on the flanging wheel can be reduced significantly on bogies having poor vertical wheel load equalisations, such as when negotiating rough track, large track twist, or when the car is experiencing roll resonances.

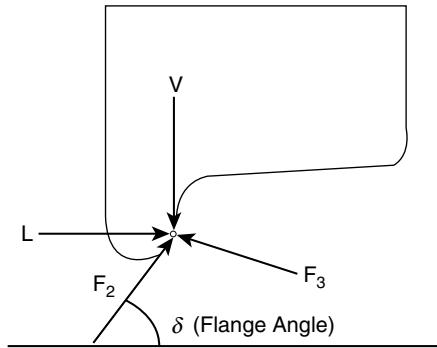


Figure 5.1: Forces at flange contact location. Extracted from [13, Figure8.4]

The criterion L/V ratio can be expressed as:

$$\frac{L}{V} = \frac{\tan \delta - \frac{F_2}{F_3}}{1 + \frac{F_2}{F_3} \tan \delta} \quad (5.1)$$

Nadal's famous L/V ratio limiting criterion, given by Equation.5.2, was proposed for the saturated condition  $F_2/F_3 = \mu$

$$\frac{L}{V} = \frac{\tan \delta - \mu}{1 + \mu \tan \delta} \quad (5.2)$$

### 5.3.2.2 Derailment caused by gauge widening and rail rollover

Derailments caused by gauge widening usually involve a combination of wide gauges and large lateral rail deflections (rail roll), as shown in Figure 5.2. Large lateral forces from the wheels act to spread the rails in curves. Both rails may experience significant lateral translation and/or railhead roll, which often cause the nonflanging wheel to drop between rails.

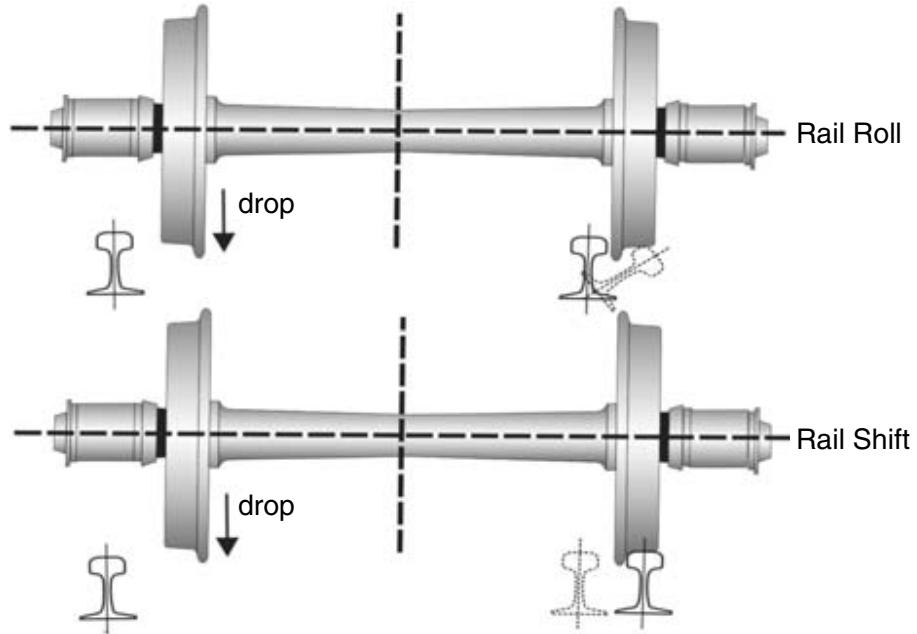


Figure 5.2: Gauge widening derailment. Extracted from [13, Figure 8.18]

**AAR Chapter XI rail roll criterion** The AAR Chapter XI rail roll criterion is established by using the L/V ratio. The roll moment about the pivot point is given by,

$$M = Vd - Lh \quad (5.3)$$

under an equilibrium condition, just before the rail starts to roll,  $M$  approaches to zero, then,

$$\frac{L}{V} = \frac{d}{h} \quad (5.4)$$

This L/V ratio is considered as the critical value to evaluate the risk of rail roll. When the L/V ratio is larger than the ratio of  $d/h$ , the risk of rail roll becomes high. The critical L/V ratio for rail roll can vary from above 0.6 for contact at the gauge side to approximately 0.2 when the contact position is at the far-field side based on the dimension of the rails. This is because the distance  $d$  is reduced. Note that this L/V ratio is calculated assuming that neither the rail fasteners nor the torsional stiffness of the rail section provide any restraint.

### 5.3.2.3 Derailment caused by track panel shift

Track panel shift is the cumulative lateral displacement of the track panel, including rails, tie plates and ties, over the ballast, as shown in Figure 5.3. A small shift of these components may not immediately cause the loss of guidance to bogies. However, as the situation gradually degrades to

a certain level, wheels could lose guidance and drop to the ground at some speed. The derailments caused by track panel usually result in one wheel falling between the rails and the other falling outside of the track.

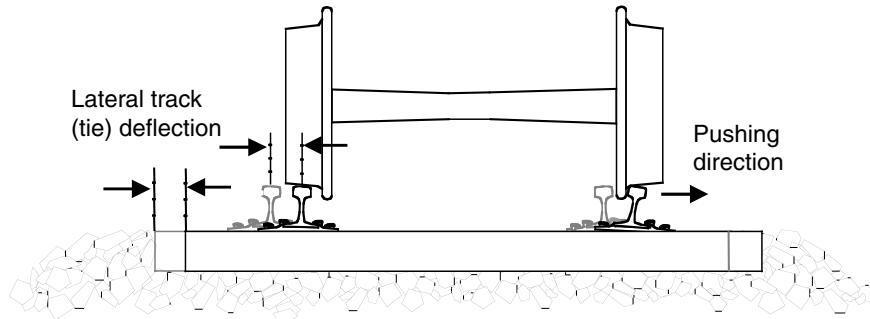


Figure 5.3: Lateral track panel shift. Extracted from [13, Figure8.27]

**Panel shift criterion** Researched by the French National Railways suggested that the limiting lateral axle load can be defined in a general expression for preventing excessive track panel shift:

$$L_c = aV + b \quad (5.5)$$

where  $L_c$  is the critical lateral load and  $V$  is the vertical axle load. [13, Table 8.2] lists two groups of suggested values of  $a$  and  $b$ . It is possible that different values for  $a$  and  $b$  can be specified in different areas.

#### 5.3.2.4 Derailment caused by vehicle lateral instability

On tangent track, the wheelset generally oscillates around the track centre due to any vehicle and track irregularities, as shown in Figure 5.4. This movement occurs because vehicle and track are never absolutely smooth and symmetric. This self-centring capability of a wheelset is induced by the coned shape of the wheel tread. However, as speed is increased, if the whelset conicity is high, the lateral movement of wheelset, as well as the associated bogie and car body motion, can cause oscillations with large amplitude and a well-defined wavelength. The lateral movements are limited only by the contact of the wheel flanges with the rail. This vehicle dynamic response is also termed as vehicle hunting, and can produce high lateral forces to damage track to cause derailments.

Derailment cause by vehicle hunting can have derailment mechanisms of all four types discussed in the previous sections. The high lateral force induced from hunting may cause wheel flange climbing on the rail, gauge widening, rail roll-over, track panel shift, or combinations of these. The safety concerns for this type of derailment, usually occurring at higher speeds, make it an important area of study.

Hunting predominantly occurs in empty or lightweight vehicles. The critical hunting speed is highly dependent on the vehicle/track characteristics. Investigation of the critical speed for such a system with nonlinearities is to examine the vehicle response to a disturbance using a numerical solution of the equations of motion.

#### 5.3.3 Requirements for traffic serviceability(horizontal)

The criteria Comfort Indexes for assessing ride comfort in railway vehicles proposed in [20]. This standard describes a methodology for assessing ride comfort as a function of longitudinal, vertical and transverse accelerations.

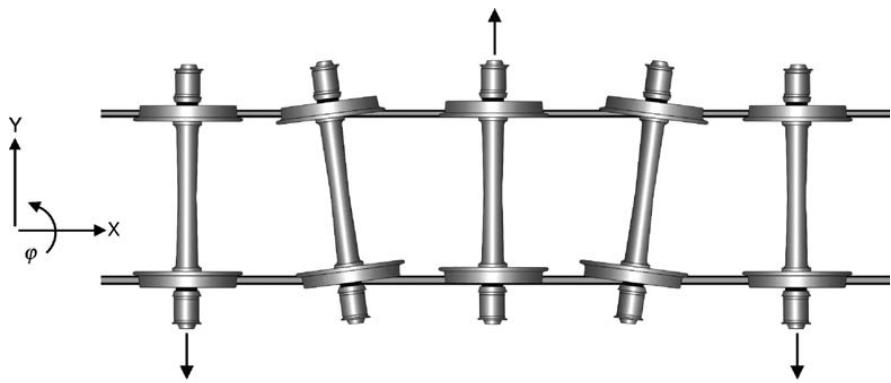


Figure 5.4: Wheelset oscillates around the track centre. Extracted from [13, Figure8.28]

Comfort Index indicates the percentage of passengers experiencing discomfort in a specific situation. These indexes can be computed via empiric formula given in the standard, which depend on variables such as lateral acceleration, rate of change of acceleration and rolling velocity. All these values are filtered with a moving average filter that eliminates small wavelength components. Using this methodology for the computed worst-case situations, the comfort indexes have been found excellent, therefore no passenger should feel uncomfortable.

## Chapter 6

## Conclusion

# Appendices

## Appendix A

# Investigation of report series created by D181 Committee Group

D181 Committee Group is created by UIC, in order to investigate Lateral Forces on Railway Bridges. Some of the proposed criteria in reports created by this committee group are adopted by Eurocode Committee to created Eurocode 1991-2.

### A.1 Structure of report series

Reports involved in the series are listed below in the order of publishing time:

1. RP 1: Summaries of national standards and literature survey
2. RP 2: Submitted programs and example of application
3. RP 3: Dynamic measurements on the steel bridge over the Brenta river on the MilanVenice line at 234 + 0.963 km
4. RP 4: Dynamic measurements on steel bridges over the Vh river by Sala on the MarcheggSzob line at 117 748 km
5. DT 312: Etude de l'influence de la fréquence du filtre sur les valeurs mesurées des forces verticales et latérales sur les rails
6. RP 5: Dynamic measurements on the metal arched bridge on PKP
7. DT 313: Analyse des déformations latérales d'un pont souple (cas du PONT de LIXHE) Ligne SNCB de TONGRESMONTZEN par J.J. REBER SBB Bau GD
8. DT 329: Parametric study Part 1: Parametric study Initial phase (September 1994) Part 2: Parametric study Phase 2 (February 1995) Authors: L.T. James and G.A. Scott
9. RP 6: Final Report

In this thesis DT 329 and RP 6 are obtained and studied, but other reports in English version are not available to the researcher.

### A.2 Items of interests in report series

1. The proposed 1.2 Hz Criterion and its background. It is discussed in RP6.
2. Resonance mechanisms studied. They are discussed in DT329.
3. Lateral forces on the bridges

### A.3 Resonance mechanisms studied in DT329

### A.3.1 Methodology of Parametric Research DT329

The research was conducted in two phases.

For the initial phase, the dynamic lateral response to the passage of different train types of various theoretical bridge models are to be examined using VAMPIRE. The method of modelling bridge behaviour adopted is the Theory of Normal Modes. Each train is modelled as a series of masses interconnected by suspension components of known characteristic. Time-step integrations are then performed to simulate the passage of a train over the bridge model along a track sample, which extends beyond the bridge.

Comparisons of measured bridge responses with VAMPIRE simulations of the bridges and trains involved were the subject of earlier studies for ERRI Committee D 181, the results being documented in reports RP 3, RP 4, and RP 5 of the Committee. Each vibration model was derived from finite element analysis of the bridge structure.

The committee have selected eleven sets of bridge parameters to be subjected to a selection of lateral load conditions. Lateral loading is provided by three train types, each running at two speeds within their respective operational ranges, on track appropriate to the train type. The combinations selected for this phase are tabulated in Appendix 1. Since the bridge design parameters are necessarily general, finite element techniques for producing bridge vibration models are inappropriate; general transverse beam vibration theory is used instead.

For the secondary phase, as part of ERRI(UIC) investigation of Lateral Forces on Railway Bridges, the ERRI D181 Committee have commissioned British Rail Research (BRR) to carry out a parametric study(DT329) using their VAMPIRE dynamics software.

The study has investigated the influence of key bridge parameters and of different train types on the lateral deflections and lateral forces exerted on the bridge. The study has been carried out in two phases.

The first phase examined 52 combinations of bridge parameters and train configurations. This study showed that some of the input parameters had little influence on the dynamic behaviour of the bridge and that others had a significant influence. There were also some parameters which appeared to show resonant effects at particular values. The results were presented in the initial report (Reference [1], part 1 of this ERRI D 181/DT 329).

From the results of the first phase of the study, the ERRI DI81 Committee were able to select the cases for the second part in order to concentrate on those parameters which had the largest influence on the behaviour of the bridge. The cases to be considered were divided into three categories. These investigated the influence of multiple span bridges (viaducts), the influence of track quality and the influence of stiffness/span/frequency on the resonant behaviour of the bridge.

It is noted that all studies were done using VAMPIRE software, which is a computer program aiming to simulate running train vehicle behaviour. The reliability of simulation has been discussed and confirmed in RP2: Submitted programs and examples of application. However, this thesis doesn't have any insight into this report since it's not available to the researcher.

Focus is placed on secondary phase of study of DT329, since it consists research on resonance behaviour of the bridge.

### A.3.2 Model of bridge

The bridge cases were modelled by assuming the bridges to behave as simply supported uniform beams. Transverse beam theory was then used to determine the frequencies and mode shapes of vibration for a given combination of span, mass per unit length and flexural rigidity. The modal information for the bridge was then used in a 'Normal Modes' analysis of the bridge.

For each case, all lateral modes of vibration up to and including 20 Hz were used. In order to prevent this artificially over-simplifying the model, if fewer than five modes were 20 Hz or less, all of the first five were used.

### A.3.3 Bridge parameters

The spans to be considered are: 20 m, 33 m, 54 m, 90 m and 120 m. The flexibilities, defined as deflection of mid span over span length due to a static point load of 100 kN at mid span, are: 114000, 1/10000, and 1120000. The mass per unit lengths required are: 2 tonnes/m, 6 tonnes/m, and 10 tonnes/m.

### A.3.4 Vehicle parameters

Three train types are considered: a typical freight train, a typical standard passenger train, and a typical high speed passenger train. Appendix.B details the parameters used to construct each model. In general, each model consists of a locomotive and a number of identical vehicles appropriate to the train type. The total number of axles in each train is 24. Although effects on the train are only examined on the first vehicle of each type, extra vehicles are added to the train to see what cumulative effects occur to the bridge.

The freight train consists of a British Railways Class 56 locomotive and nine UIC wagons. This has a total length of 131.56 m, which assumes a nominal vehicle coupling distance of 4 m. Runs at 60 km/h and 100 km/h are required.

The standard passenger train consists of an E444 locomotive and five UIC coaches. This has a total length of 143.8 m. It is based on one of two train models used as part of the study of the FS Bridge discussed in report RP 3 of the Committee, differing only by the addition of three extra coaches. This is required to run at 160 km/h and 200 km/h.

The high speed passenger train consists of an ETR500 locomotive and five ETR500 coaches, having a total length of 145.8 m. It is based on the other FS bridge study train model mentioned above, differing from the original by an additional three ETR500 coaches. It is required to run at 300 km/h and 350 km/h.

### A.3.5 Track

For initial study phase, the track samples used were consistent with each train type. PSD plots of each are shown in Figures B.13 to B.15. Sample TRACKFRT.DAT was used for all analysis runs for the freight train. This is measured data from a typical BR freight line. Sample TRACKPKN.DAT was used for the standard passenger train analysis runs. This is measured data from a part of the BR East Coast main line. Sample TRACKPH.DAT was used for high speed passenger train analysis runs. This is measured data from a typical DB high speed line.

Samples of 500 m were chosen so that there would be 100 m before the bridge and at least 100 m after the bridge for all combinations of span and train length. The initial 100 m is required to check vehicle behaviour on the track irregularity alone, and the portion after the train has left the bridge is required to check that the bridge vibrations decay.

For secondary study phase, the track data used to excite the mathematical models was taken from the British Rail Research library of measured track data. For the viaduct and resonance investigations, the track files used were the same as those used in the first part of the study. For the investigation of the influence of track quality, additional track data was used so as to give the widest possible range of realistic track qualities.

### A.3.6 Contact data

For each run the same contact data was used, consisting of rails inclined at 1:20, and wheel profiles of conicity of 0.05 (based on standard British Rail 113A rails and PI wheel profiles). The coefficient of friction applied was 0.3.

### A.3.7 Summary of modelling procedure

The modelling of simulation for most of the research purposes of both phases generally used a routine where bridge module and same train-type-coupled track irregularities, vehicle parameters were adopted. That means the parameters of track and vehicle are exactly the same among all simulations are the same if not specially noted. This frequently used simulation set-up will be called routine in the following paragraphs. See previous sections for detailed information about routine.

The routine is used in following researches:

1. Phase I: All research on bridge parameters
2. Phase II: Viaduct study
3. Phase II: Resonant study

Track quality study in secondary phase which used different simulation set-ups. The following trains:

Freight 60 km/h

Freight 100 km/h

Freight 120 km/h

Passenger 200 km/h

High Speed 350 km/h

were run, each with 3 different conicity values (0.05, 0.2 and 0.4) over 14 sections of measured track data each of length 1 km.

### A.3.8 Data produced

For every analysis run the following results were obtained at intervals of 0.01 seconds.

BRIDGE DATA:

Lateral displacement at mid span Lateral acceleration at mid span

VEHICLE LATERAL ACCELERATION DATA:

Loco body at leading pivot

Leading coach/wagon body at leading pivot/axle Loco leading bogie

Leading coach/wagon leading bogie/axle

TOTAL LATERAL FORCE DATA:

Loco leading bogie

Leading coach/wagon leading bogie/axle

LATERAL FORCES ON INDIVIDUAL WHEELS

Leading coach/wagon, first axle, left and right wheels

Leading coach/wagon, second axle, left and right wheels

Loco, first axle, left and right wheels

Loco, second axle, left and right wheels

In addition, for freight train runs, since the locomotive has two bogies of three axles, the forces on the individual wheels of the third axle were also produced.

Peak values for each of the outputs produced for the required ranges were obtained. For bridge outputs, peak values were taken for the period where any part of the train was on the bridge. For loco and leading coach/wagon outputs, peak values were taken whilst the vehicle in question was in contact with the bridge.

Peak values for each output were then read into a spread sheet where they could be compared more easily to check for emerging trends. The spread sheet has been partially automated to produce

graphs of a single output for each train type for a single varying bridge parameter, for given values of the other bridge parameters. Figures 4 to 30([3]) show typical plots which have been produced in this manner.

### A.3.9 Resonance caused by axle repeat pattern

Axle repeat patterns are wavelength phenomena - regardless of vehicle speed, the repeat length is constant. However, since frequency is speed divided by wavelength, the frequency of the axle repeat patterns vary with train speed. A table of axle repeat pattern lengths, and typical frequencies arising from train speed are given in Figure.B.1

By running train in different speeds shows resonance is possible between train and bridge if the axle passing frequency coincides with the first lateral bridge mode, the effect occurring in bridge lateral displacement over a limited frequency range around the resonance frequency.

However, the speed on theory which should yield resonance effect may be different from the speed that actually triggered

### A.3.10 Resonance caused by kinematic movement of trains

Kinematic wavelength also gives rise to frequencies which vary with speed for the same reason. For first lateral bending mode coincidence with kinematic frequency, the kinematic wavelength of each train type had to be established, by running each train at a range of typical operating speeds over a discrete lateral irregularity, and examining the frequency content of the lateral wheel motion. The resulting wavelength ranges are tabulated in Table.B.2.

The most likely possible resonance in the initial studies to be of this type was between the passenger train at 200 km/h (55.556 m/s) on passenger track and BR PI wheel profiles, and a span of 54 m, stiffness 1/10000, mass/length of 6 tonnes/m. This combination was examined by varying the speed between 55.556 - 64.6 m/s over the span, and by varying the stiffness of the span between 117000 and 1112000 running the train at 55.556 m/s. Another combination was examined - the ETR500 train running between 65 - 80 m/s on high speed track and BR PI wheel profiles, for a span of 38 m, stiffness 1110000, and mass/length 10 tonne/m; the span in this case was chosen to coincide with the kinematic wavelength of the coaches.

Coincidence of vehicle kinematic frequency with bridge first lateral bending mode may cause resonance to occur over a broad range of frequencies to a less pronounced effect than coincidence of axle passing frequencies. Evidence of coincidence of kinematic wavelength with length of span has been found in the lateral acceleration of bridges, but was not demonstrated in the lateral bridge displacement in the cases examined. For short kinematic wavelengths, this effect could not be seen, possibly because of lack of time for the bridge to respond.

### A.3.11 Apparent shift in resonance frequency

It is frequently observed in the output of both resonance effects that apparent resonance happens at some frequencies higher than frequencies calculated on theory. This is explained in following quote on [3, Page 13, Secondary Phase]

Although the peak mid span displacement was expected to occur at 28.5 mis, it can be seen that for the runs with just the coaches that the peak occurs at about 32 m/s. This is confirmed to be a resonance-type effect rather than a discrete event in the time histories of the runs, a selection of which are shown in Figure C6(Original report), of which an extract of 100-285m follows as Figure C7(Original report). This speed is mid way between axle passing frequency coinciding with first bending mode of the bridge, and kinematic frequency coinciding with the first bending mode. So, the apparent shift in resonant frequency may be due to a combination of these effects (see discussion of kinematic

frequency resonance results). However, an alternative explanation may be that as track forces generally increase with speed, the deflection of the span would be expected to increase. If this effect continued through a resonant band, the peak displacement would appear greater at a speed slightly above that calculated for resonance, as sketched in Figure A.1.

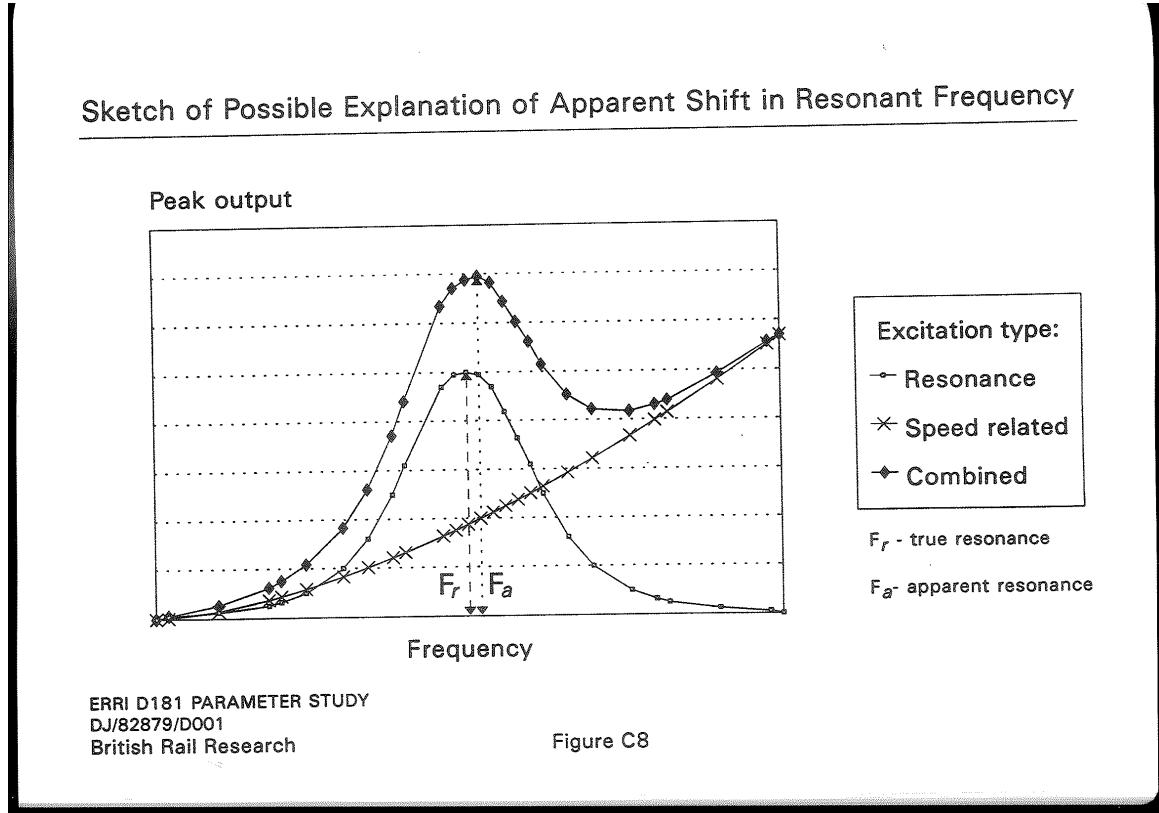


Figure A.1: Sketch of Possible Explanation for Apparent Shift in Resonance Frequency. Extract from [3, Appendix 2]

What's more, track irregularities profile in normal analysis runs may also contribute to resonance shift. This is due to the wavelength on theory is obtained by an analysis run adopting a discrete irregularity, which is different from those analysis runs yielding apparent resonance shifts. And theory resonance frequency is calculated on the wavelength output of this specific run.

Both explanations indicate that apparent resonance frequency can hardly be predicted. Apparent frequency may even shift into domain lower than theory frequency.

#### A.4 Debate on proposed 1.2Hz Criterion in RP6

Evidence of [4] is the origin of [9, A.2.4.4.2.4(3)] is found in [4, p4.2: Lateral Frequencies]:

In order to avoid the phenomena of lateral resonance in vehicles, the first natural frequency of lateral vibration of the span  $f_{lt}$  such that:

$$f_{lt} \geq 1.2\text{Hz} \quad (\text{A.1})$$

The statement exactly coincides with criterion A.2.4.4.2.4(3) in Eurocode 1991. It is sufficient to acknowledge D181 RP6 as the origin of criterion A.2.4.4.2.4(3) because this report is created by UIC.

The value of frequency limit, 1.2Hz is explained in [4, p3.2: Criterion 2]:

To avoid the occurrence of resonance in the lateral motion of the vehicles due to the lateral motion of the bridge, a limit value lower than the first natural frequency  $f_{1t}$  of the lateral vibration of the span studied should be fixed. The natural frequency for lateral movements is between 0.5 and 0.7 Hz for coaches and between 0.7 and 1 Hz for locomotives. We therefore propose a safety margin  $F_{lt} \geq 1.2\text{Hz}$

The original author of report RP6 Graham Scott was contacted to reveal the background of 'natural frequency for lateral movements'. Mr.Scott is still in charge of the development of software VAMPIRE and he's still active in the field. Unfortunately he was unable to remember what did 'natural frequency for lateral movements' stand for in previous quotes since it was written nearly 20 years ago. He passed me to his colleague Alan Minnis for further questions. Mr.Minnis stated following:

Looking at the values I think they would refer to typical rigid body modes of a vehicle. These are independent of speed and a typical passenger coach with air suspension will have a lower sway frequency of around 0.6Hz which is within 0.5-0.7Hz. Locomotives tend to have a slightly stiffer suspension hence the slightly higher frequency range.

Mr.Minnis statements, combined with results yielded in supporting parametric report DT329 proves 1.2Hz criterion isn't sufficient in avoiding the occurrence of resonance between train and bridge structure, nor it's sufficient in providing safety to the bridge. The reason is listed below:

1. There is lots of evidence can be found in report DT329, showing resonance can happen on a bridge with a first lateral natural frequency even higher than 1.2Hz, which is self-conflicting with 1.2Hz criterion. For example, [3, Page 14,Phase II] shows resonance occurs on 1.71Hz:

The first lateral bending mode of this bridge is at 1. 71 Hz. The kinematic wavelength of the passenger coaches is around 34-38 m, giving a kinematic frequency range of 1.46 - 1.63Hz. Speeds of 58.14 m/s(1.53-1.71 Hz)and 64.6m/s(1.7-1.9Hz)were also done. The mid span lateral displacement for each of the time histories are shown in Figure C12(Original report). The slowest speed appears to show the greatest resonance.

2. The resonance between rigid body mode of train and first lateral vibration mode of the bridge has never been discussed in all D181 report series. No research proofed this kind of resonance can happen in real life scenario.

## A.5 Lateral forces on railway bridges

It is concluded in initial phase of the study that presence of the bridge doesn't influence the track forces and track quality is a major factor in determining the lateral forces generated by a particular train on [3, Page 7, Secondary Phase]

From the initial study [1], it was concluded that the track quality on a bridge is a major factor in determining the lateral forces generated by a particular train. The D181 Committee therefore asked BRR to determine the peak track forces generated over a wide range of track qualities.

The length of a bridge is small compared to the overall length of a railway track and so track quality on a single bridge may not be representative of that on other bridges on the same route. However the initial study concluded that, in general, the lateral track forces are not influenced by the presence of the bridge.

The resonance effects mentioned in the previous sections were only observed in deflection and acceleration domain due to the reason that the presence of the bridge doesn't influence the track forces.

The output of track forces from track quality study was used to create a loading model for proposal in RP6. The lateral forces are given as characteristic values  $F_m + 3s$ ,  $s$  being the standard deviation. Lateral forces from vehicle/bridge interaction as a result of hunting are presented in [4, Page 33]. To illustrate, a sample of load model is presented in Figure A.2. See [4, Page 33] for detailed usage of each load model.

## A.6 Summary of D181 report series

The report series managed to create load models for lateral dynamics railway effects. It is worthy to note that the lateral force on the track wasn't influenced by the presence of the bridge. The major influencing parameter for lateral force is track quality and conicity of the wheel profile.

The resonance phenomenon was successfully reproduced and observed, though its only visible in deflection and acceleration domain. A basic characteristic of resonance, regardless of resonance type, is apparent resonance frequency will shift from resonance frequency calculated on theory. The shift is unpredictable in the sense of both direction and magnitude. The effect related to speed start to creep in when speed is higher, making the effect of resonance less pronounced in higher speed(Figure A.1).

Please note that every bridge will always have resonance with running train because axle repeat pattern and kinematic movement are both wavelength phenomenon, which means there is always a speed of train yielding a vibration frequency coincides with the first lateral natural frequency of bridge. However, the effect of resonance happening on long-span bridge is usually unpronounced since the speed of the train is low as 2.5m/s to 14m/s when resonance occurs.

Some of the conclusion and proposed criteria in RP6 were adopted in Eurocode 1991-2. It is proofed that 1.2Hz criterion was adopted without amending. However, most probably load cases created by RP6 was amended before being adopted into [9, A6.5.2], since there exists slight difference.

The 1.2Hz criterion was under debate and proofed unreliable in fulfilling its original intention, avoiding occurrence of resonance. There is no research in D181 report series supporting this criterion, nor there exists literature behind the natural frequency of vehicles. This criterion ignored the possibility of future bridge designs with long span would certainly have a natural frequency lower than 1.2 Hz. It is advised that Eurocode review this criterion and revise it.

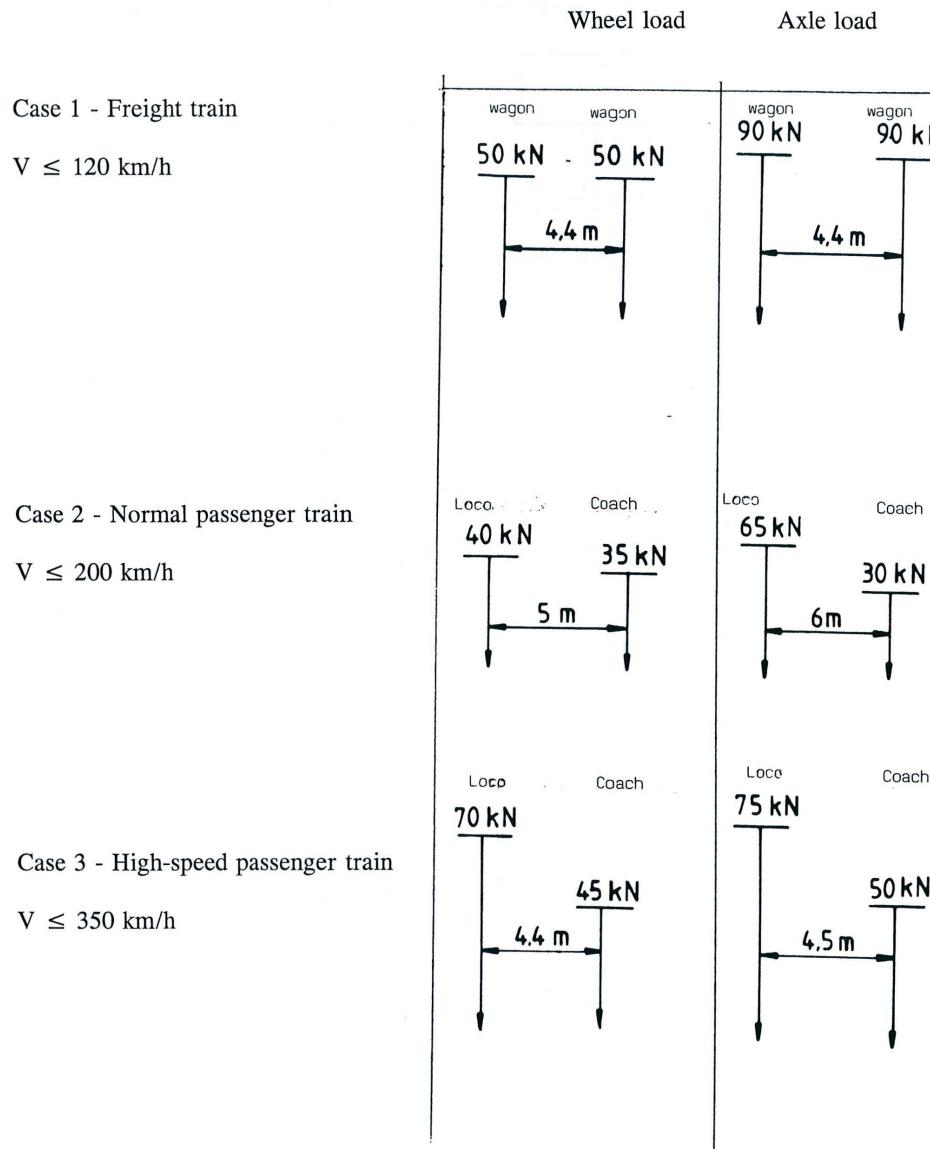


Figure A.2: LATERAL WHEEL AND AXLE FORCES FOR BRIDGES. Extract from [4, Fig 3.1]

## Appendix B

# Plots and diagrams used in D181 DT 329

Freight train: Principle axle repeat patterns	dist m	Speed	
		60 km/h	100 km/h
wagon n axle 2 - wagon n+1 axle 1	4.00	4.17	6.94
wagon wheelbase	9.00	1.85	3.09
wagon n axle m - wagon n+1 axle m	13.0	1.28	2.14
wagon n axle m - wagon n+2 axle m	26.0	0.64	1.07
Passenger train: Principle axle repeat patterns	dist m	Speed	
		160 km/h	200 km/h
coach n axle 1 - 2, and coach n axle 3 - 4	2.56	17.36	21.70
coach n axle m - coach n+1 axle m	26.4	1.68	2.10
coach n axle m - coach n+2 axle m	52.8	0.84	1.05
ETR 500 train: Principle axle repeat patterns	dist m	Speed	
		300 km/h	350 km/h
coach n axle 1 - 2 and coach n axle 3 - 4	3.0	27.78	32.41
coach n axle m - coach n+1 axle m	26.1	3.19	3.72
coach n axle m - coach n+2 axle m	52.2	1.60	1.86
coach n axle m - coach n+3 axle m	69.3	1.20	1.40

Table B.1: Axle repeat patterns and typical frequencies. Extracted from [3, Appendix C]

Kinematic wavelength, m	Freight train	Passenger train	ETR500 train
Locomotive	39 - 45	32 - 38	39 - 45
Coach/wagon	24 - 39	34 - 38	36 - 40

Table B.2: Kinematic wavelength ranges per vehicle, with BR P1 profiles. Extracted from [3, Appendix C]

## VEHICLE MODEL PARAMETER LISTS

VEHICLE TITLE      BR CLASS 56 LOCOMOTIVE

## MASSES &amp; INERTIAS

Number of bogies	2
Number of axles (per bogie)	3
Body mass	81.2 Mg
Body roll inertia	107.0 Mgm <sup>2</sup>
Body pitch inertia	1400.0 Mgm <sup>2</sup>
Body yaw inertia	1400.0 Mgm <sup>2</sup>
Bogie mass	5.6 Mg
Bogie roll inertia	5.0 Mgm <sup>2</sup>
Bogie pitch inertia	21.6 Mgm <sup>2</sup>
Bogie yaw inertia	21.6 Mgm <sup>2</sup>
Wheelset mass	2.2 Mg
Wheelset roll and yaw inertia	2.7 Mgm <sup>2</sup>

## DIMENSIONS

Semi pivot spacing	5.19 m
Semi wheelbase	2.09 m
Wheel radius	0.57 m
Body centre of gravity height above rail level	1.85 m
Bogie centre of gravity height above rail level	0.86 m

## PRIMARY SUSPENSION

Lateral stiffness (per axle)	0.1 MN/m
Vertical stiffness (per axle)	2.63 MN/m
Yaw stiffness (per axle)	29.0 MNm/r
Lateral damper rate (per axle)	- MNs/m
Vertical damper rate (per axle)	0.05 MNs/m
Vertical friction breakout (per axle)	- KN
Height above rail level of lateral springs	0.67 m
Lateral semi spacing of vertical springs	1.035 m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	1.035 m
Lateral semi spacing of vertical friction	- m

Figure B.1: BR CLASS 56 LOCOMOTIVE. Extract from [3, Appendix 2]

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Appendix 2

**SECONDARY SUSPENSION**

Lateral stiffness (per bogie)	0.77 MN/m
Vertical stiffness (per bogie)	2.72 MN/m
Roll bar stiffness (per bogie)	- MNm/r
Yaw stiffness (per bogie)	1.22 MNm/r
Lateral damper rate (per bogie)	0.084 MNs/m
Vertical damper rate (per bogie)	0.15 MNs/m
Yaw damper rate (per bogie)	0.055 MNms/r
Height above rail level of lateral springs	1.31 m
Lateral semi spacing of vertical springs	1.062 m
Height above rail level of lateral dampers	1.06 m
Lateral semi spacing of vertical dampers	1.062 m

Figure B.2: BR CLASS 56 LOCOMOTIVE. Extract from [3, Appendix 2]

## Appendix 2

### VEHICLE TITLE            UIC FREIGHT WAGON (LADEN)

#### MASSES & INERTIAS

Number of bogies	-
Number of axles (per wagon)	2
Body mass	41.0 Mg
Body roll inertia	35.0 Mgm <sup>2</sup>
Body pitch inertia	500.0 Mgm <sup>2</sup>
Body yaw inertia	500.0 Mgm <sup>2</sup>
Bogie mass	- Mg
Bogie roll inertia	- Mgm <sup>2</sup>
Bogie pitch inertia	- Mgm <sup>2</sup>
Bogie yaw inertia	- Mgm <sup>2</sup>
Wheelset mass	2.0 Mg
Wheelset roll and yaw inertia	1.7 Mgm <sup>2</sup>

#### DIMENSIONS

Semi pivot spacing	- m
Semi wheelbase	4.5 m
Wheel radius	0.46 m
Body centre of gravity height above rail level	1.5 m
Bogie centre of gravity height above rail level	- m

#### PRIMARY SUSPENSION

Lateral stiffness (per axle)	1.5 MN/m
Vertical stiffness (per axle)	2.6 MN/m
Yaw stiffness (per axle)	10.0 MNm/r
Lateral damper rate (per axle)	0.034 MNs/m
Vertical damper rate (per axle)	- MNs/m
Vertical friction breakout (per axle)	3.0 KN
Height above rail level of lateral springs	0.46 m
Lateral semi spacing of vertical springs	1.0 m
Height above rail level of lateral dampers	0.46 m
Lateral semi spacing of vertical dampers	- m
Lateral semi spacing of vertical friction	1.0 m

Figure B.3: UIC FREIGHT WAGON (LADEN). Extract from [3, Appendix 2]

**SECONDARY SUSPENSION**

Lateral stiffness (per bogie)	- MN/m
Vertical stiffness (per bogie)	- MN/m
Roll bar stiffness (per bogie)	- MNm/r
Yaw stiffness (per bogie)	- MNm/r
Lateral damper rate (per bogie)	- MNs/m
Vertical damper rate (per bogie)	- MNs/m
Yaw damper rate (per bogie)	- MNms/r
Height above rail level of lateral springs	- m
Lateral semi spacing of vertical springs	- m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	- m

Figure B.4: UIC FREIGHT WAGON (LADEN). Extract from [3, Appendix 2]

Appendix 2

**VEHICLE TITLE            FS ETR500 LOCOMOTIVE**

**MASSES & INERTIAS**

Number of bogies	2
Number of axles (per bogie)	2
Body mass	55.98 Mg
Body roll inertia	53.366 Mgm <sup>2</sup>
Body pitch inertia	1643.0 Mgm <sup>2</sup>
Body yaw inertia	1630.0 Mgm <sup>2</sup>
Bogie mass	3.896 Mg
Bogie roll inertia	3.115 Mgm <sup>2</sup>
Bogie pitch inertia	5.843 Mgm <sup>2</sup>
Bogie yaw inertia	8.107 Mgm <sup>2</sup>
Wheelset mass	2.059 Mg
Wheelset roll and yaw inertia	1.164 Mgm <sup>2</sup>

**DIMENSIONS**

Semi pivot spacing	6.0 m
Semi wheelbase	1.5 m
Wheel radius	0.55 m
Body centre of gravity height above rail level	1.65 m
Bogie centre of gravity height above rail level	0.64 m

**PRIMARY SUSPENSION**

Lateral stiffness (per axle)	12.0 MN/m
Vertical stiffness (per axle)	3.55 MN/m
Yaw stiffness (per axle)	15.4 MNm/r
Lateral damper rate (per axle)	- MNs/m
Vertical damper rate (per axle)	0.3 MNs/m
Vertical friction breakout (per axle)	- KN
Height above rail level of lateral springs	0.55 m
Lateral semi spacing of vertical springs	1.03 m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	1.2 m
Lateral semi spacing of vertical friction	- m

Figure B.5: FS ETR500 LOCOMOTIVE. Extract from [3, Appendix 2]

Appendix 2

**SECONDARY SUSPENSION**

Lateral stiffness (per bogie)	0.584 MN/m
Vertical stiffness (per bogie)	1.888 MN/m
Roll bar stiffness (per bogie)	1.0 MNm/r
Yaw stiffness (per bogie)	- MNm/r
Lateral damper rate (per bogie)	0.037 MNs/m
Vertical damper rate (per bogie)	0.145 MNs/m
Yaw damper rate (per bogie)	0.938 MNms/r
Height above rail level of lateral springs	0.95 m
Lateral semi spacing of vertical springs	1.03 m
Height above rail level of lateral dampers	0.73 m
Lateral semi spacing of vertical dampers	1.05 m

Figure B.6: FS ETR500 LOCOMOTIVE. Extract from [3, Appendix 2]

Appendix 2

**VEHICLE TITLE            FS ETR500 COACH**

**MASSES & INERTIAS**

Number of bogies	2
Number of axles (per bogie)	2
Body mass	34.23 Mg
Body roll inertia	54.63 Mgm <sup>2</sup>
Body pitch inertia	1821.0 Mgm <sup>2</sup>
Body yaw inertia	1760.0 Mgm <sup>2</sup>
Bogie mass	2.76 Mg
Bogie roll inertia	2.034 Mgm <sup>2</sup>
Bogie pitch inertia	2.504 Mgm <sup>2</sup>
Bogie yaw inertia	4.071 Mgm <sup>2</sup>
Wheelset mass	1.58 Mg
Wheelset roll and yaw inertia	0.753 Mgm <sup>2</sup>

**DIMENSIONS**

Semi pivot spacing	9.5 m
Semi wheelbase	1.5 m
Wheel radius	0.44 m
Body centre of gravity height above rail level	1.5 m
Bogie centre of gravity height above rail level	0.68 m

**PRIMARY SUSPENSION**

Lateral stiffness (per axle)	4.35 MN/m
Vertical stiffness (per axle)	1.61 MN/m
Yaw stiffness (per axle)	14.0 MNm/r
Lateral damper rate (per axle)	- MNs/m
Vertical damper rate (per axle)	0.015 MNs/m
Vertical friction breakout (per axle)	- KN
Height above rail level of lateral springs	0.44 m
Lateral semi spacing of vertical springs	0.96 m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	0.96 m
Lateral semi spacing of vertical friction	- m

Figure B.7: FS ETR500 COACH. Extract from [3, Appendix 2]

Appendix 2

**SECONDARY SUSPENSION**

Lateral stiffness (per bogie)	0.256 MN/m
Vertical stiffness (per bogie)	0.722 MN/m
Roll bar stiffness (per bogie)	1.0 MNm/r
Yaw stiffness (per bogie)	- MNm/r
Lateral damper rate (per bogie)	0.04 MNs/m
Vertical damper rate (per bogie)	0.065 MNs/m
Yaw damper rate (per bogie)	0.70 MNms/r
Height above rail level of lateral springs	0.8 m
Lateral semi spacing of vertical springs	0.96 m
Height above rail level of lateral dampers	0.8 m
Lateral semi spacing of vertical dampers	1.2 m

Figure B.8: FS ETR500 COACH. Extract from [3, Appendix 2]

Appendix 2

**VEHICLE TITLE                    FS E444 LOCOMOTIVE**

**MASSES & INERTIAS**

Number of bogies	2
Number of axles (per bogie)	2
Body mass	64.6 Mg
Body roll inertia	53.366 Mgm <sup>2</sup>
Body pitch inertia	1643.0 Mgm <sup>2</sup>
Body yaw inertia	1630.0 Mgm <sup>2</sup>
Bogie mass	4.0 Mg
Bogie roll inertia	3.115 Mgm <sup>2</sup>
Bogie pitch inertia	5.843 Mgm <sup>2</sup>
Bogie yaw inertia	8.107 Mgm <sup>2</sup>
Wheelset mass	2.1 Mg
Wheelset roll and yaw inertia	1.164 Mgm <sup>2</sup>

**DIMENSIONS**

Semi pivot spacing	4.5 m
Semi wheelbase	1.3 m
Wheel radius	0.55 m
Body centre of gravity height above rail level	1.65 m
Bogie centre of gravity height above rail level	0.64 m

**PRIMARY SUSPENSION**

Lateral stiffness (per axle)	12.0 MN/m
Vertical stiffness (per axle)	4.0 MN/m
Yaw stiffness (per axle)	15.4 MN/m
Lateral damper rate (per axle)	- MNs/m
Vertical damper rate (per axle)	0.03 MNs/m
Vertical friction breakout (per axle)	- KN
Height above rail level of lateral springs	0.55 m
Lateral semi spacing of vertical springs	1.03 m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	1.2 m
Lateral semi spacing of vertical friction	- m

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Figure B.9: FS E444 LOCOMOTIVE. Extract from [3, Appendix 2]

## Appendix 2

### SECONDARY SUSPENSION

Lateral stiffness (per bogie)	1.0 MN/m
Vertical stiffness (per bogie)	2.1 N/m
Roll bar stiffness (per bogie)	1.0 Nm/r
Yaw stiffness (per bogie)	- Nm/r
Lateral damper rate (per bogie)	0.037 MNs/m
Vertical damper rate (per bogie)	0.145 MNs/m
Yaw damper rate (per bogie)	0.938 MNms/r
Height above rail level of lateral springs	0.95 m
Lateral semi spacing of vertical springs	1.03 m
Height above rail level of lateral dampers	0.73 m
Lateral semi spacing of vertical dampers	1.20 m

Figure B.10: FS E444 LOCOMOTIVE. Extract from [3, Appendix 2]

Appendix 2

**VEHICLE TITLE            UIC COACH**

**MASSES & INERTIAS**

Number of bogies	2
Number of axles (per bogie)	2
Body mass	32.0 Mg
Body roll inertia	56.8 Mgm <sup>2</sup>
Body pitch inertia	1970.0 Mgm <sup>2</sup>
Body yaw inertia	1970.0 Mgm <sup>2</sup>
Bogie mass	2.615 Mg
Bogie roll inertia	1.722 Mgm <sup>2</sup>
Bogie pitch inertia	1.476 Mgm <sup>2</sup>
Bogie yaw inertia	3.067 Mgm <sup>2</sup>
Wheelset mass	1.70 Mg
Wheelset roll and yaw inertia	1.30 Mgm <sup>2</sup>

**DIMENSIONS**

Semi pivot spacing	9.5 m
Semi wheelbase	1.28 m
Wheel radius	0.445 m
Body centre of gravity height above rail level	1.503 m
Bogie centre of gravity height above rail level	0.68 m

**PRIMARY SUSPENSION**

Lateral stiffness (per axle)	6.4 MN/m
Vertical stiffness (per axle)	1.46 MN/m
Yaw stiffness (per axle)	60.0 MNm/r
	- MNs/m
Lateral damper rate (per axle)	0.005 MNs/m
Vertical damper rate (per axle)	- KN
Vertical friction breakout (per axle)	- m
	- m
Height above rail level of lateral springs	0.445 m
Lateral semi spacing of vertical springs	1.0 m
Height above rail level of lateral dampers	- m
Lateral semi spacing of vertical dampers	1.0 m
Lateral semi spacing of vertical friction	- m

Figure B.11: UIC COACH. Extract from [3, Appendix 2]

Appendix 2

**SECONDARY SUSPENSION**

Lateral stiffness (per bogie)	0.32 MN/m
Vertical stiffness (per bogie)	0.86 MN/m
Roll bar stiffness (per bogie)	0.94 MNm/r
Yaw stiffness (per bogie)	- MNm/r
Lateral damper rate (per bogie)	0.059 MNs/m
Vertical damper rate (per bogie)	0.074 MNs/m
Yaw damper rate (per bogie)	0.591 MNms/r
Height above rail level of lateral springs	0.825 m
Lateral semi spacing of vertical springs	1.0 m
Height above rail level of lateral dampers	0.825 m
Lateral semi spacing of vertical dampers	1.3 m

Figure B.12: UIC COACH. Extract from [3, Appendix 2]

Freight track: Trackfrt. dat

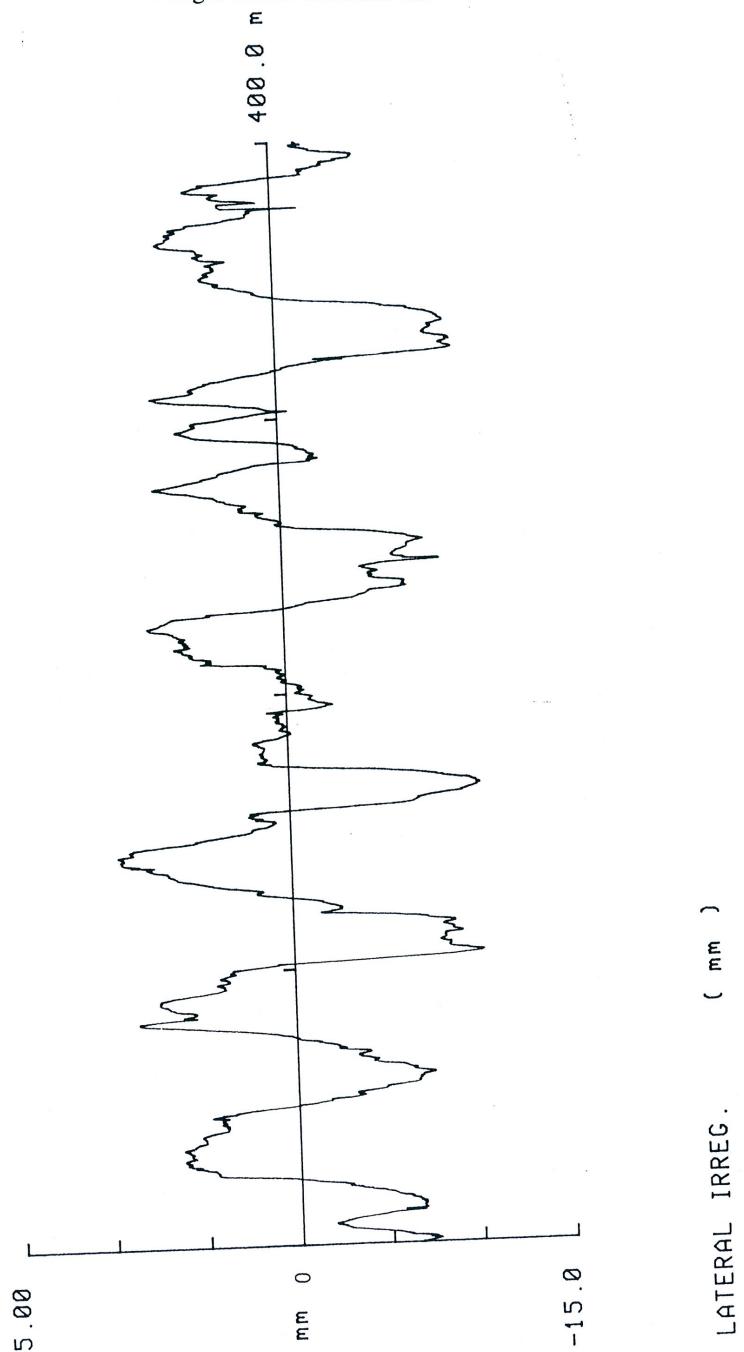
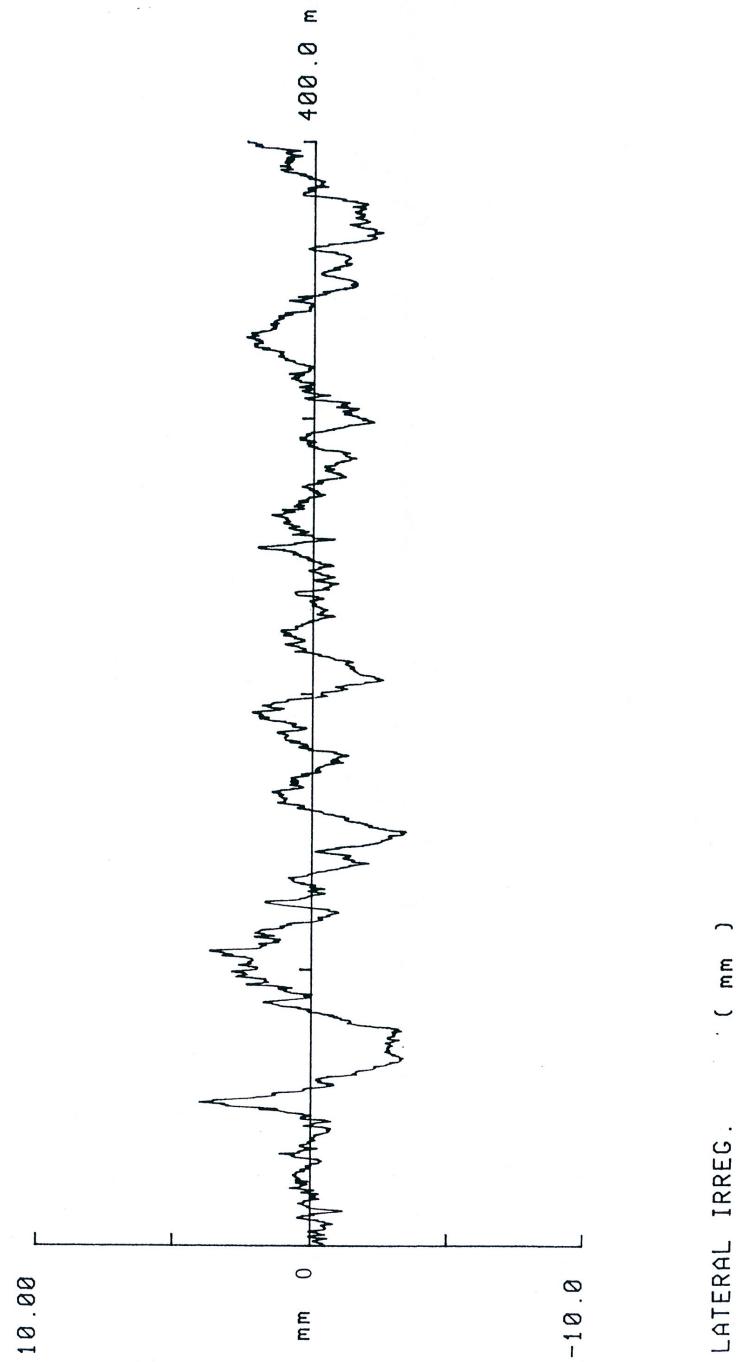


Figure B.13: Horizontal track irregularities for freight trains. Extract from [4, Figure 2.1]

Passenger track: Trackpn. dat



LATERAL IRREG. (mm)

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Figure B.14: Horizontal track irregularities for standard passenger trains. Extract from [4, Figure 2.1]

ERRI D 181/RP 6	VAMPIRE HORIZONTAL TRACK IRREGULARITIES (HIGH-SPEED LINE)	Fig. 2.3
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High speed track: Trackph. dat

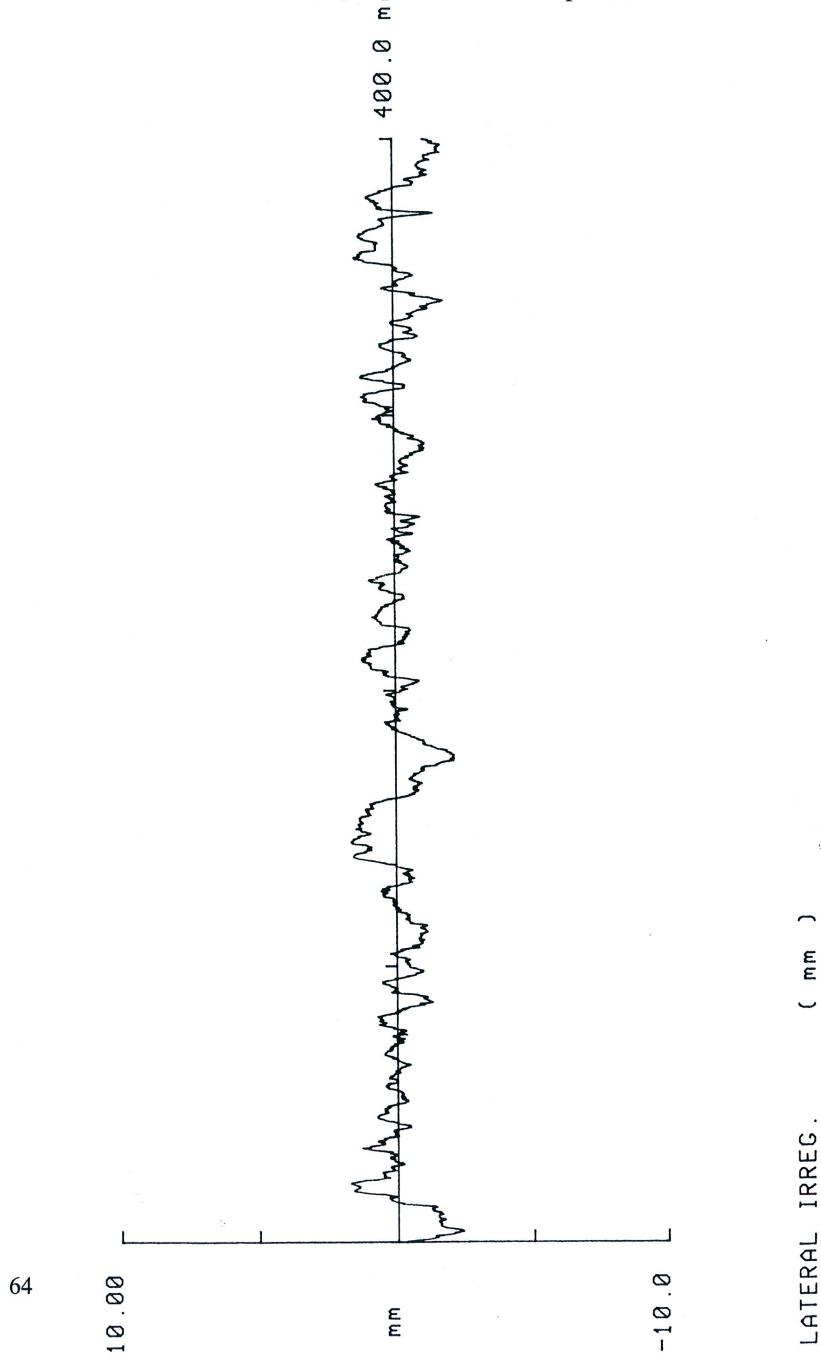


Figure B.15: Horizontal track irregularities for high speed passenger train. Extract from [4, Figure 2.1]

## Appendix C

# Speeds which do not require dynamic compatibility checks

Line category Locomotive class	Freight wagon	Locomotive	Passenger carriage	Multiple unit	Special vehicle
a10 <sup>a</sup>	-	-	-	-	-
a12 <sup>a</sup>	-	-	-	-	-
a14 <sup>a</sup>	-	-	-	-	-
A	120	120 <sup>b</sup> / 160	160 <sup>c</sup>	160 <sup>c</sup>	120
B1	120	120 <sup>b</sup> / 160	160 <sup>c</sup>	160 <sup>c</sup>	120
B2	120	120 <sup>b</sup> / 160	-	-	120
C2	120	120 <sup>b</sup> / 160	140 <sup>c</sup>	140 <sup>c</sup>	120
C3	120	120	-	-	120
C4	120	120	-	-	120
D2	120	120 <sup>b</sup> / 160	120 <sup>c</sup>	120 <sup>c</sup>	120
D3	120	120	-	-	120
D4	120	120	-	-	120
D4xL	120 <sup>d</sup>	120	-	-	120 <sup>d</sup>
D5	100	-	-	-	100
E4	100	-	-	-	100
E5	100	-	-	-	100
E6	80	-	-	-	80
L4	-	120 <sup>b</sup> / 160	-	-	-
L6	-	120	-	-	-

<sup>a</sup> Light railways – normal operating speeds are generally significantly less than speed at which additional dynamic checks would need to be considered.  
<sup>b</sup> Three or more adjacent couples locomotives.  
<sup>c</sup> Additional values for max "p" (see Table F.2).  
<sup>d</sup> Option.

Figure C.1: Speed limit (in km/h) in relationship Line Category/Locomotive Class and vehicle type.  
Extract from [10, Appendix F]

## Appendix D

# MU-Groups and MU-Classes

### D.1 Definition

Multiple units can be grouped according to type of traffic service (high speed - long distance, intercity - regional and commuter/suburban) or to the kind of running gear (conventional bogies, articulated bogies and single axles).

In some cases due to potential excessive dynamic load effects in bridge line category checks are not sufficient to demonstrate compatibility. To minimise the need for undertaking a dynamic check of individual trains, several typical and wide spread MU-designs have been grouped in MU-classes. For these groups of vehicles, load models covering the specified design parameter ranges have been developed to allow the efficient dynamic analysis of bridges. For practical reasons, the number of MU classes was limited and for trains outside the range of parameters covered, the process of checking an individual train existing at the time of publication of this standard as state of the art shall be used.

Each MU-class is defined by:

- ranges of train parameters covered and;
- a corresponding load model for carrying out dynamic checks on bridges.

Each MU-Group comprises of several MU-Classes. Table

MU-Group	MU-Class
conventional bogie(CB)	$CB_1$ $CB_2$
articulated bogie(AB)	$AB_1$ $AB_2$ $AB_3$ $AB_4$
single axle(SA)	$SA_1$ $SA_2$

Table D.1: Relationship MU-groups - MU-classes

#### D.1.1 Train parameters of MU-Class CB\_1

#### D.1.2 Train parameters of MU-Class CB\_2

#### D.1.3 Train parameters of MU-Class AB\_1

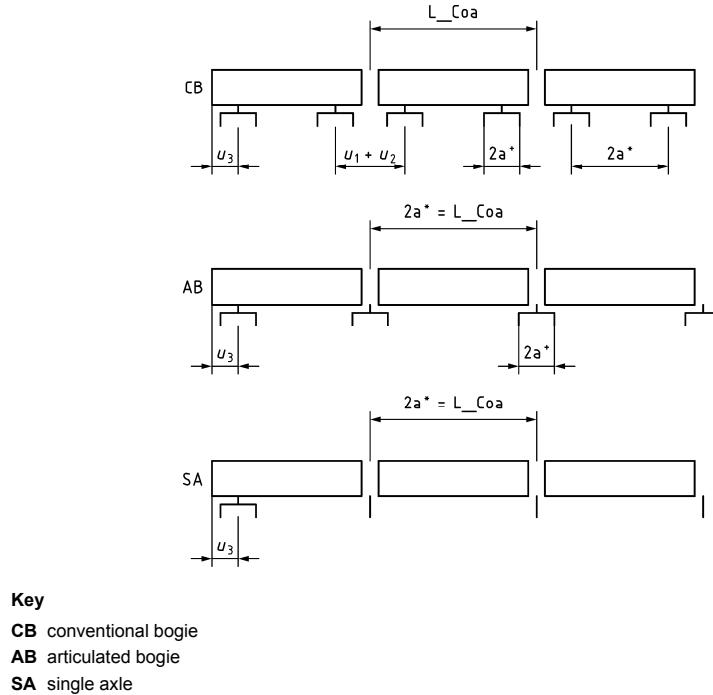


Figure D.1: Train parameters related to MU-Groups. Extracted from [10, Annex C]

Name	Parameter	Unit
$2a^*$	Bogie spacing between pivot centres within a vehicle	m
$2a^+$	Axle spacing in bogie	m
$u_1 + u_2$	Bogie spacing between pivot centres of adjacent vehicles	m
$u_3$	Overhang of end coaches	m
$L_{Coa}$	Coach length	m
No_Coa	Number of coaches within an unit	-
No_Units	Number of units within a train	-

Table D.2: Explanation of train parameters. Extracted from [10, Annex C]

- [D.1.4 Train parameters of MU-Class AB\\_2](#)
- [D.1.5 Train parameters of MU-Class AB\\_3](#)
- [D.1.6 Train parameters of MU-Class AB\\_4](#)
- [D.1.7 Train parameters of MU-Class SA\\_1](#)
- [D.1.8 Train parameters of MU-Class SA\\_2](#)

max No_Units	2
max No_Coa	8
L_Coa	$23.8m \leq L_Coa \leq 25.3m$
$2a^*$	$16.8m \leq 2a^* \leq 18.0m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$(u1 + u2)$	$7.0m \leq (u1 + u2) \leq 7.6m$
$u3$	$4m \leq u3 \leq 6m$

Table D.3: Train parameters for conformity with MU-Class CB\_1

max No_Units	2
max No_Coa	7
L_Coa	$25.3m \leq L_Coa \leq 27.5m$
$2a^*$	$18.0m \leq 2a^* \leq 19.5m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$(u1 + u2)$	$7.2m \leq (u1 + u2) \leq 8.0m$
$u3$	$4m \leq u3 \leq 6m$

Table D.4: Train parameters for conformity with MU-Class CB\_2

max No_Units	4
max No_Coa	5
$2a^*$	$14.9m \leq 2a^* \leq 16.0m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$u3$	$3m \leq u3 \leq 5.5m$

Table D.5: Train parameters for conformity with MU-Class AB\_1

max No_Units	4
max No_Coa	5
$2a^*$	$18.8m \leq 2a^* \leq 19.5m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$u3$	$3m \leq u3 \leq 5.5m$

Table D.6: Train parameters for conformity with MU-Class AB\_2

max No_Units	2
max No_Coa	11
$2a^*$	$17.0m \leq 2a^* \leq 17.5m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$u3$	$4.5m \leq u3 \leq 5.7m$

Table D.7: Train parameters for conformity with MU-Class AB\_3

max No_Units	2
max No_Coa	10
$2a^*$	$18.7m \leq 2a^* \leq 19.2m$
$2a^+$	$2m \leq 2a^+ \leq 3m$
$u3$	$4.3m \leq u3 \leq 5.3m$

Table D.8: Train parameters for conformity with MU-Class AB\_4

max No_Units	3
max No_Coa	10
$2a^*$	$9.2m \leq 2a^* \leq 9.8m$
$u3$	$4.25m \leq u3 \leq 6.25m$

Table D.9: Train parameters for conformity with MU-Class SA\_1

max No_Units	2
max No_Coa	14
$2a^*$	$12.8m \leq 2a^* \leq 13.5m$
$u3$	$4.25m \leq u3 \leq 6.25m$

Table D.10: Train parameters for conformity with MU-Class SA\_1

# Bibliography

- [1] Jean-Armand Calgaro, Marcel Tschumi, and Haig Gulvanessian. *Designers' Guide to Eurocode 1: Actions on Bridges*. 2010.
- [2] UIC Code. “518: Testing and approval of railway vehicles from the point of view of their dynamic behaviour—Safety—Track fatigue—Ride quality”. In: *International Union of Railways, 3rd ed., Paris* (2005).
- [3] ERRI D181 Committee. *ERRI D 181/DT 329 E: LATERAL FORCES ON RAILWAY BRIDGES, PARAMETRIC STUDY PART1,PART 2*. UIC, 1995.
- [4] ERRI D181 Committee. *RP6: Final report, D181 LATERAL FORCES ON RAILWAY BRIDGES*. UIC, 1996.
- [5] Manuel Cuadrado Sanguino et al. “Analysis of Lateral Displacements in Large Railway Viaducts Under Traffic Loads. Impact on Ride Safety and Passenger Comfort.” In: (2008).
- [6] BS EN. “14363: 2005”. In: *Railway applications-Testing for the acceptance of running characteristics of railway vehicles-Testing of running behavior and stationary tests* (2007), pp. 6–36.
- [7] *EN 1990 AMD 2 : 2005(E) Annex A2 to EN 1990*.
- [8] Coenraad Esveld. “Modern railway track”. In: (2001).
- [9] CEN Eurocode. “1: Actions on structures, Part 2: Traffic loads on bridges”. In: *Brussels: European Standard EN 2* (1991), p. 2003.
- [10] CEN Eurocode. “prEN 15528:2013, Railway applications Line categories for managing the interface between load limits of vehicles and infrastructure”. In: *Brussels: European Standard EN* (2013).
- [11] L. Fryba and L. Frýba. *Dynamics of railway bridges*. T. Telford, 1996. ISBN: 9780727720443.
- [12] British Standards Institution. *Eurocode: Basis of structural design*. BSI, 2002.
- [13] Simon Iwnicki. *Handbook of railway vehicle dynamics*. CRC Press, 2006.
- [14] Colin MacDOUGALL, Robert G Driver, and Gilbert Y Grondin. “A state-of-the-art review of high performance steel for bridge design”. In: *Transports Quebec 11e Colloque*. 2004.
- [15] A.V. Metrikine. *Dynamics, Slender Structures and an Introduction to Continuum Mechanics: CT 4145*. Delft University of Technology.
- [16] M Sajad Mohammadi and Rishiraj Mukherjee. “Wind Loads on Bridges: Analysis of a three span bridge based on theoretical methods and Eurocode 1”. In: (2013).
- [17] International Union of Railways(UIC). *Design requirements for rail-bridges based on interaction phenomena between train, track and bridge*.
- [18] Rui Filipe da Silva Dias. “Dynamic Behaviour of High Speed Railway Bridges. Vehicles Lateral Dynamic Behaviour”. In: (2007).

- [19] Yan Quan Sun and Manicka Dhanasekar. “A dynamic model for the vertical interaction of the rail track and wagon system”. In: *International Journal of Solids and Structures* 39.5 (2002), pp. 1337–1359.
- [20] UNE-ENV 12299: *Aplicaciones ferroviarias. Comodidad de viaje para los viajeros. Medición y evaluación.* 2000.
- [21] UNE-ENV 13803-1 EX: *Aplicaciones ferroviarias. Parámetros de proyecto de trazado de la vía. Ancho de vía de 1 435mm y mayores. Parte 1: Plena vía.* 2006.