Loading Requirements for Track Systems

Phase 1 summary report

RESEARCH AND DEVELOPMENT

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Executive Summary

Mott MacDonald was engaged by RSSB to undertake Phase 1 of the research project 'T1073-01 Loading Requirements for Track Systems'. The aim of the research project is to provide an enhanced understanding of the mechanisms for track loading and the response of the track system, specifically to:

- Help improve the way that track systems are designed and ultimately delivered and maintained, through the rationalisation of track loading parameters which are currently inconsistently employed in the design of the railway track; and,
- Deliver a track loading model process enabling the Great Britain (GB) rail industry to carry out track system design more effectively, featuring due attention to any interactions between the track and the rest of the rail transport system.

Phase 1 particularly focusses on establishing the current understanding of the track loading parameters required for design.

The subject of loading of the railway track has been reviewed on the basis of a systematic international literature research, documenting the excitation mechanisms from loads acting on the track, identification of the vertical, lateral and longitudinal loads applied to the track and appraisal of the interactions with the rolling stock and operational aspects of a railway system.

A key aspect emphasised is the fundamental relationship between track loads, railway vehicle properties, and dynamics, which are either inadequately addressed or are circularly referenced across the Standards, Regulations and other technical reference documents that have been considered in Phase 1 of the research project. The resulting lack of clarity concerning the most effective design approach, has the potential to result in inefficiency for designed and constructed track systems, due to either under or over specification.

Review of current practice across the GB and international rail industry has identified that there is considerable variety in the methodology used for design and analysis of track systems, particularly for ballastless track systems. It appears that designers use a methodology deemed to be most appropriate for the system being designed, but generally rely on structural codes, for example EN1991-2 for railway traffic loading and EN1992 for design of concrete slab track. Project specifications for various ballastless track structures for Crossrail, TfL Track Slab renewal and an embedded rail system, have been reviewed to establish the design parameters being used by designers and manufacturers to develop these systems.

The outcome of cross-discipline research, focussing on investigation of the design loading principles adopted in the pavement, earthworks and bridge engineering fields, demonstrates that any lessons learned from other sectors seem to have been already integrated, being part of the current state-of-the-art for railway infrastructure.

This study has identified a basic set of requirements and track loading specifications which form the recommended basis for vertical, lateral and longitudinal loadings used in the design of track systems. Rationalisation of these is required to provide a basis for the harmonisation and standardisation of track design for the GB rail industry.

The outcome of the Phase 1 research shows that:

- There is uncertainty in the GB rail industry about the track loading requirements for the design of track systems, due to the difficulties associated with obtaining data about the impact of loading on track system life, whole life cost and system safety and reliability.
- There are multiple interconnected dependencies for load transfer within the track system and simply specifying a standard dynamic load factor, as is the case in prEN 16432-1, has the potential to result in the construction of track systems that are either under-specified or over-specified.
- The research has identified a need for development of a methodology for design of track systems, including the provision of appropriate load models and guidance on their scope of application.
- Review of project experience within the UK and internationally has found that administrations generally rely on bridge loading codes to establish the required design loading for ballastless (concrete slab) track systems.
- There are potential benefits, although currently these are challenging to quantify, for the industry through the development of a common approach to the design track systems, which is likely to result in improved efficiencies in design, construction and maintenance as well as develop industry confidence in the use of ballastless track systems.
- To facilitate the development of a design loading specification, it is necessary to accurately capture and record the requirements for track loading, which are appropriate for determination of the track parameters,

the railway traffic and the operational requirements, of a particular network.

The Phase 1 research has identified that further work is required to deliver an agreed track loading specification for the GB network. It is proposed that the development of a track design specification for the GB rail industry will require the evaluation of the following requirements:

- Definition and agreement of the total/design vertical load that is to be considered for the design of track systems.
- Definition and agreement of the total/design lateral load that is to be considered for the design of track systems.
- Consideration of the systemic character of railways (railways as a system - subsystem - component), in accordance with the current state-of-the-art, the track being a part of the railway infrastructure.
- Consideration of the rolling stock and its interaction with the track, through the use of fundamental principles for loading, consideration of the dynamic characteristics of both the track and the rolling stock and the impact of variability in these characteristics on the requirements from the design outputs.
- Consideration of all actions and influences likely to occur during the track's intended life.
- Outline the likely deterioration mechanism of the proposed track system.
- Provision of a thoroughly developed design and analysis toolset which is able to meet the current and future demands of the rail industry.

The development of a defined methodology for the design of track systems should combat uncertainty within the GB rail industry about the track loading requirements for design. This will potentially lead to improved efficiency through greater certainty in the performance of a track system, resulting in greater efficiency in design, construction and maintenance, leading to a reduction in whole of life cycle costs.

Abbreviations

ACN	Aircraft Classification Number
AREA	American Engineering Association
AREMA	American Railway Engineering and Maintenance-of-Way Association
ARTC	Australian Rail Track Corporation
BR	British Railways
BS	British Standards
CEN	Comité Européen de Normalisation (European Committee for Standardisation)
CTRL	Channel Tunnel Rail Link
DAF	Dynamic Amplification Factor
DB	Deutsche Bahn (or German Railway)
DLR	Docklands Light Railway
DMG	Design and Maintenance Guide
EN	Euro Norm
EU	European Union
ERRI	European Rail Research Institute
FEM	Finite element model
GB	Great Britain
HS1	High Speed 1
HS2	High Speed 2
INF TSI	Infrastructure Technical Specification for Interoperability
KPIs	Key performance indicators
LOC & PAS TSI	The Locomotive and Passenger Rolling Stock Technical Specification for Interoperability

LM71	Loading Model 71
LU	London Underground
MGT	Million gross tonne
MOD	Ministry of Defence
NR	Network Rail
ORE	Office of Research and Experiments of the International Union of Railways
PCN	Pavement Classification Number
RLE	Rail Link Engineering
RSSB	Rail Safety and Standards Board
RTS	Rail Technical Strategy
RGS	Rail Group Standards
S&C	Switch and crossing
SLS	Serviceability limit state
SNCF	Société Nationale des Chemins de Fer (or French National Railway Company)
TGV	Train à Grande Vitesse (or High Speed Train)
UIC	Union Internationale des Chemins de Fer (or International Union of Railways)
ULS	Ultimate limit state

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1 Introduction

1.1 Objectives and application

Mott MacDonald was engaged by the Rail Safety and Standards Board (RSSB) in March 2015 to undertake Phase 1 (also known as WP01) of the research project 'T1073-01 Loading Requirements for Track Systems'.

The overall objectives of the research project are:

- To obtain an enhanced understanding of the mechanisms for track loading and the response of the track system, such that the track loading parameters which are currently employed in the design of the railway track are critically reviewed, verified and recommendations for improvement and optimisation identified and recorded for future development.
- To help improve, through the rationalisation of track loading parameters, which are currently rather inconsistently employed in the design of the railway track, the way that track systems are designed and ultimately delivered and maintained by optimisation of the required resources and materials.
- To deliver a track loading model process enabling the Great Britain (GB) rail industry to carry out track system design more effectively, taking account of any interactions between the track and the rest of the rail transport system, in particular the rolling stock; the characteristics of the track structure and any underlying track layers and sub-structures will be considered as part of a holistic approach.

The research undertaken in Phase 1 attempts to answer the following questions:

- What are the current track design practices both in GB and elsewhere? What problems have been experienced and why?
- What do customers specify in terms of track loading parameters and why?
- How does a track system designer apply these requirements to provide a compliant design?
- What parameters are taken into consideration and included in the design and why?

- How do tailor made track loading models compare with standard track load models?
- What are the loads applied by the rolling stock (vertical, lateral and longitudinal) for the ranges of British track geometry design parameters in terms of maximum, normal and exceptional dynamic forces?
- How does the influence of specific track characteristics affect the magnitude of the input force applied to the track at a specific location?
- What forces are appropriate for design of the different components of the track system?

The research extends to all trackform types (both ballasted and ballastless) and intends to cover all network (dedicated or not) and operation (passenger, freight, mixed, conventional and high speed) cases encountered in GB. The research primarily covers plain line sections, although an effort is made to include consideration of junctions and other locations where localised increase of dynamic loading may be experienced, such as expansion switches and insulated rail joints.

The research initiative also aligns with the objectives of the Technical Strategy Leadership Group (TSLG) 2012 Rail Technical Strategy (RTS), of which the terms of reference are agreed by the Board of RSSB and aims for 'a simple, reliable and cost-effective infrastructure'.

It is the intention that the outputs from the Phase 1 research will:

- Provide justification for and support to development and validation of track loading parameters suitable for use in track design, within Phase 2 of the Project, based on the information and knowledge gained from Phase 1.
- Allow informed comments to be made and to support input to development of the draft European Norm (EN) standard prEN 16432-1 'Railway Applications – Ballastless Track Systems – Part 1: General Requirements'. This document is currently due for publication by the Comité Européen de Normalisation (CEN). However, there is an opportunity to influence future revision of the document and to provide information to support the development of National Standards or Guidance.
- Assist the GB rail industry in developing a loading regime that can be used with confidence for infrastructure projects particularly where there is a potential for use of ballastless track (eg High Speed 2 - HS2).
- Contribute to the goal of reduced industry costs for new construction and renewals, through the identification of reliable design methodologies which

are potentially more efficient and consistent in the approach to the design of track systems. In addition to potential savings for the construction of the track system, there are significant unquantified benefits, including improved safety through the potential reduction of component failure and train accidents and business benefits because of potentially lower whole life system costs from improved design, more effective and predictable maintenance, leading to enhanced resilience of the track system and improved railway capacity.

Mott MacDonald acknowledges the involvement of RSSB and the stakeholders from the project steering group. Their contributions, when endorsed by Mott MacDonald, have been included in this report.

1.2 Problem statement

Current practice for specifying loading for the design of track systems is perceived to be inconsistent both within Great Britain (GB) and across Europe and considered to be due to a lack of coherent and widely accepted standards providing a set of effective design requirements.

The Locomotives & Passenger Rolling Stock Technical Specification for Interoperability (LOC & PAS TSI) and the Infrastructure Technical Specification for Interoperability (INF TSI), as an example, seem circular regarding reference to track loading, due to the lack of technically consistent documented design requirements.

Furthermore, track loading limits are derived from force limits set out in vehicle acceptance standards, particularly in the case of lateral and dynamic loads. An example of this is Railway Group Standards (RGS); GM/TT0088 'Permissible Track Forces for Railway Vehicles', which sets limits for forces which can be applied to the track by railway vehicles. GC/RT5021 'Track System Requirements' 'mandates requirements for track geometry, track system, track components and S&C to provide for the safe guidance and support of rail vehicles' but in practice the specified forces are not used for design of track. The origin and application of this loading is not fully defined with respect to its application for track design. The use of load limits has traditionally been linked to vehicle acceptance, through the specification of maximum forces applied to the track by vehicles in operation.

To a large extent, current standards for track design loading have been effectively based upon the known satisfactory performance of track systems and empirically developed through experience at particular sites. This point is clearly reflected in a Technical Note prepared by RSSB titled 'Design Loading for Track Systems' (refer Appendix A). This Technical Note provides a brief commentary on the loading requirements for bridges, the permissible loads applied to track by railway vehicles, the loads considered to apply to the design of track systems and the potential relevance of bridge design loading for use in the design of track systems.

Uncertainty around consistent and clear track loading requirements creates potential for over-provision or under-provision of track assets. It is a grey area which is liable to lead to uncertainty and create undefined risks in the provision of design, checking and approval, potentially impacting track component durability and whole life costs, as well as system safety and reliability, each with respective potential commercial repercussions.

The development and use of cross-discipline standards for track design and identification of governing loading regimes for the design of track systems is not apparent. This contrasts with the loading requirements in bridge design standards, which were established many years ago. However, the assumptions upon which they are based are complex, particularly the lateral and dynamic loading aspects. Furthermore, bridge loading is not necessarily directly applicable or appropriate for the design of track systems, although the determination of appropriate loading requirements for track will be no less complex.

Pertinent to this study, the intention of CEN Working Group (TC 256/SC 1/WG 46) is to include general requirements for loading within the draft EN standard prEN 16432-1 'Railway Applications – Ballastless Track Systems – Part 1: General Requirements', that are based on Load Model 71 (LM71) which is used for bridge design. The standard will also allow specifiers to stipulate their own load model, indicating that there is some way to go before consensus is achieved for the vertical loading parameters, particularly in respect of the dynamic effects. The basis for derivation and application of the lateral loading to be applied to the rails is also unclear at present. The critical aspect of load combination and the use of partial safety factors is not addressed at all.

There is a potential risk that specifiers will not use their own models and instead adopt LM71 to save time at a risk of increasing design, capital and operational costs. Conversely, if the loading described is not clear and sufficient to account for all dynamic effects, then failures are likely to manifest within the design lifetime of the track systems built.

The potential consequence for the GB rail industry is that there will remain an ongoing risk of inadequate or over provision for loading requirements. The risks and consequences point to an urgent need for an in-depth study of the track loading requirements, in order to establish a set of parameters that are acceptable to infrastructure owners, designers and manufacturers for a variety of track and loading conditions, and, which adequately mitigate these concerns.

1.3 Methodology

Phase 1 of the research project has involved a staged approach summarised in the following:

- Stage 1: Refining and validating the objectives and scope of the research to be undertaken. This stage initially involved the elicitation and review of the project requirements internally among Mott MacDonald project team members, then review of the proposal with the project stakeholders. This stage relied primarily on subject matter expertise and inputs from the discussions held with the project stakeholders.
- Stage 2: Investigating and analysing the design loading requirements in other transport sectors, for example highways, for the design of pavement structures and also design practice in the earthworks and bridge engineering fields to identify any similarities and to gain valuable insight, including, where possible, lessons that could be extended and applied to rail engineering and the loading of the railway track in particular. The outputs from this stage relied primarily on subject matter expertise, technical workshops and/or daily coordination work between Mott MacDonald project team members.
- Stage 3: Investigating past and current track design practices both in GB and elsewhere, where possible and the historic development of the engineering approach to track design loading. The outputs from this stage relied primarily on a systematic international literature research, partially from subject matter expertise inputs plus those provided by industry stakeholders and from historic project records, to the extent possible.
- Stage 4: Investigating the excitation mechanisms and system aspects behind the loads acting on the railway track and setting out basic functional requirements that an informed approach to track design should consider.
- Stage 5: Drafting of the Draft Technical Summary Report for Stakeholder review and comments and submitting for review.

- Stage 6: Review of the Draft Technical Summary Report and its findings jointly with the project stakeholders.
- Stage 7: Drafting of the Final Technical Summary Report for submission, incorporating stakeholder review and comments.

1.4 Contribution to engineering and future research prospects

This research report provides a review of the current state-of-the-art on the subject of track loading. Within the framework of Phase 1 of the research project 'T1073-01 Loading Requirements for Track Systems', the excitation mechanisms for the loads acting on the track have been considered and discussed, the loads acting on the track have been identified and the interactions between the track and the rolling stock and the operational aspects of a railway system have been considered. This has facilitated:

- An enhanced understanding of the mechanisms governing the loading of the railway track in service.
- An improved insight into the principal laws governing track loads, railway vehicle properties and dynamics.
- The identification of areas of uncertainty with regard to track loading and the potential for harmonisation, as specifically applied to the GB rail industry and within the limits of the research undertaken.
- The drafting of a set of recommendations enabling RSSB and the GB rail industry to proceed with more confidence towards the definition of a principal design loading regime.
- The submission of informed comments on the draft EN standard prEN 16432-1 'Railway Applications – Ballastless Track Systems – Part 1: General Requirements' currently being developed by the CEN (Appendix B).

It is recommended that the results of this Phase 1 research be used to inform potential further research in Phase 2, in accordance with the proposed objectives, following prior consultation with (and agreement between) the key project stakeholders.

1.5 Report structure

The research report structure is:

- Section 1: Sets out the objectives of the research undertaken, its expected application and the approach followed by the research team.
- Section 2: Provides a brief background on the core subject of the research undertaken (the railway track and its loading mechanism) by providing reminders of the basic principles underpinning the origin of track loading sources and interaction with the other components of a railway system (rolling stock and operations). The key mathematical expressions of track loads in service have been included, as captured from the literature review.
- Section 3: Provides a historic review of the loading requirements for track systems and how / why these requirements have evolved over time, to the extent possible. The current state-of-the-art is also addressed; reference is made to current Standards and Regulations and project Performance Specifications, as well as a summary of the experience of track design from projects where this has been made available to Mott MacDonald. In accordance with the requirements in the Work Package Business Case and Specification documents prepared by RSSB, the results of a cross discipline research covering compatible experience from pavement, earthworks and bridge engineering, with regard to vehicle loading requirements are presented, similarities examined and lessons learned, documented and discussed.
- Section 4: Provides recommendations for development of track and component design loading requirements, based on the outcomes from Section 2 and Section 3.
- Section 5: Discussion on the potential economic benefits of improving efficiency in the design of track systems when taking into consideration the whole-of-life-costs.
- Section 6: Summary of the research undertaken, the conclusions drawn and recommendations for future development within Phase 2.

2 The railway track – origin of loads in service

2.1 Definition and interfaces

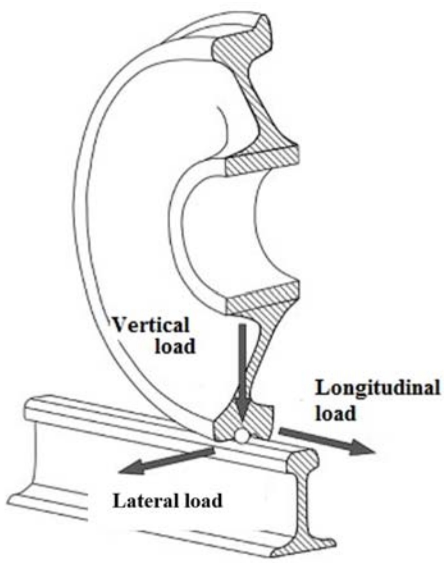
Railway track, in either ballasted or ballastless form, consists of structural materials and components of different elasticities. Its function is to provide the running surface and guidance for railway vehicles to travel and distributes load, exerted by rolling stock, from the top of rail down to the trackbed and substructure below.

2.2 Loads in service

The response of the railway track to the loads acting on it, as for any other structural system, is inherently linked to the nature of the loading (Clough et al. 1993).

The railway track is subject to forces that can be represented as vertical, lateral and longitudinal loading components (see Figure 1) although a combination of these components will always apply. Although other forces do apply and are relevant to track design and management, only the forces acting on the track are generated by the rolling stock (traffic loads) are in the scope of this report. See prEN16432-1 for a full list of the other effects that should be considered.

Figure 1 - Forces acting on the rail in all three directions.
(Source: Zerbst et al., 2009 with modifications)



Vertical loads are exerted on the rolling plane of the rails and transferred to the trackbed through the various elastic track components making up the system; the elasticity of the track components and the track construction layers and the multi-body character of the entire system, enable the generated stresses to decrease with distance from the top of the rail, according to the vertical load distributing characteristics of the track structure.

Lateral (Transverse) loads are transferred to the rails through wheel - rail interaction. From the rails, the loads are transferred down to the trackbed layers through the track system, according to the horizontal load distributing characteristics of the track components, whereby some may also experience torsion or vertical uplift in addition to shear and bending.

Longitudinal loads (originating from the rolling stock) are also exerted on the rolling plane of the rails and, similarly to the lateral loads, the forces are transferred to the trackbed through the components of the track system. The length of track through which the longitudinal loads are distributed may be considerably longer than for both the lateral and vertical loads because of the continuous nature of the rail. For simplicity and to be conservative, the length of load application is often limited in design.

The classification of static loads, quasi-static loads, semi–static loads and dynamic loads is complex and it is recommended that it is formally defined in WP02. The classifications below originate from a number of sources/ references (Lichtberger - 2005, Esveld - 2001, Alias - 1984, Hunt - 1998) and have been applied consistently for the purposes of this report:

- *Static loads:* Non-time-varying loads due to self-weight of the rolling stock vehicle (various conditions may apply, such as loaded or unloaded).
- *Quasi-static or semi-static loads:* These are loads originating from the rolling stock and are applied for the period of time that the cause exists. Vertical and horizontal loads developed as a result of cross-winds or residual centrifugal forces are examples of quasi-static loads.
- *Dynamic loads:* Dynamic loads are time-varying loads. These are the result of:
    - Track irregularities, both vertically and horizontally (due to track geometry faults) and variations of track stiffness in a particular layer or across combined layers of the track system, which give rise to random loads;
    - Discontinuities of the rolling plane, such as at welds, joints, crossings and switches;

- Wheel defects, such as wheel flats and wheel-out-of-roundness; and,
- Geometric, inertia and stiffness imperfections in relation to the rolling stock such as, for example, uneven inflating of the secondary suspension and asymmetries.

2.2.1 Vertical loads

Vertical loads are the primary input in the design, construction, operation and maintenance of the railway track. The static vertical wheel load, in particular, is part of the mathematical expressions of all lateral and longitudinal forces acting on track arising from the rolling stock, either directly or indirectly.

Furthermore, vertical strain developing as a result of the vertical load acting on the track determines the selection of the rail section, the selection and spacing of the sleepers / blocks / bearers, the selection of the rail fastenings and the dimensioning of the resilient layers, including the trackbed. Additionally, vertical loads play an important role in the asset life of both track and rolling stock.

Table 1 provides an overview of the vertical loads acting on the railway track together with their analytical expressions. Further information on how these expressions are derived is provided in the Appendices. The total vertical wheel load Qt equals the sum of all static, quasi-static and dynamic vertical wheel loads at any time:

$$Q_t = Q_0 \pm Q_{wj} \pm Q_{nc} \pm Q_{dyn} \tag{1}$$

The quasi-static vertical wheel load Q_{nc}, which is due to the residual centrifugal force, is only applied during negotiation of curved sections of the horizontal alignment by the rail vehicle (with cant deficiency) and ranges on average between 10 % to 25 % of the magnitude of the static wheel load. The load of the wheel rolling on the outer rail is increased by Q_{nc}, while the load on the inner wheel is decreased by an equal amount.

For Q_w, the transient vertical wheel load due to cross winds causes an overloading of the outer wheel on the lee side of the vehicle and unloading of the other wheel by an equal amount. Increase in the vertical load acting on the track due to cross winds is applicable both to straight and curved track sections.

Contrary to the rather limited magnitude and limited variation of the quasi-static vertical loads, dynamic vertical loads vary considerably in practice. This is because the vertical dynamic loads acting on the railway track as a result of the track - vehicle interaction caused primarily by the condition of rolling stock and track are random (Alias 1984, Prud'homme 1975). As a result, the accurate estimation and calculation of dynamic forces remains extremely complex.

Table 1 - Track Vertical Loads (Source: Mott MacDonald)

Designation / Symbol	Type	Cause / Generation Mechanism	Analytical Expression (j = 1, wheel 1) (j = 2, wheel 2)
Axle Load (Q)	Static	Vehicle mass	$Q = \left(\frac{\bar{M}}{4} + \frac{M'}{2} + m\right) \cdot g$ Vehicle with 2 two-axle bogies
Vertical Wheel Load (Qo)	Static	Vehicle mass	$Q_o = \frac{1}{2} \cdot \left(\frac{\bar{M}}{4} + \frac{M'}{2} + m\right) \cdot g$ Vehicle with 2 two-axle bogies
Vertical Wheel Load due to Cross-Winds (Qw)	Quasi-static	Cross-winds	$\pm Q_{wj} = H_w \cdot \frac{q_o}{2 \cdot e_o}$
Vertical Wheel Load due to Residual Centrifugal Force (Qnc)	Quasi-static	Cant deficiency (curves only)	$\pm Q_{ncj} = F_{nc} \cdot \frac{h_{KB}}{2 \cdot e_o} = Q \cdot \frac{I \cdot h_{KB}}{4 \cdot {e_o}^2}$

Table 1 - Track Vertical Loads (Source: Mott MacDonald)

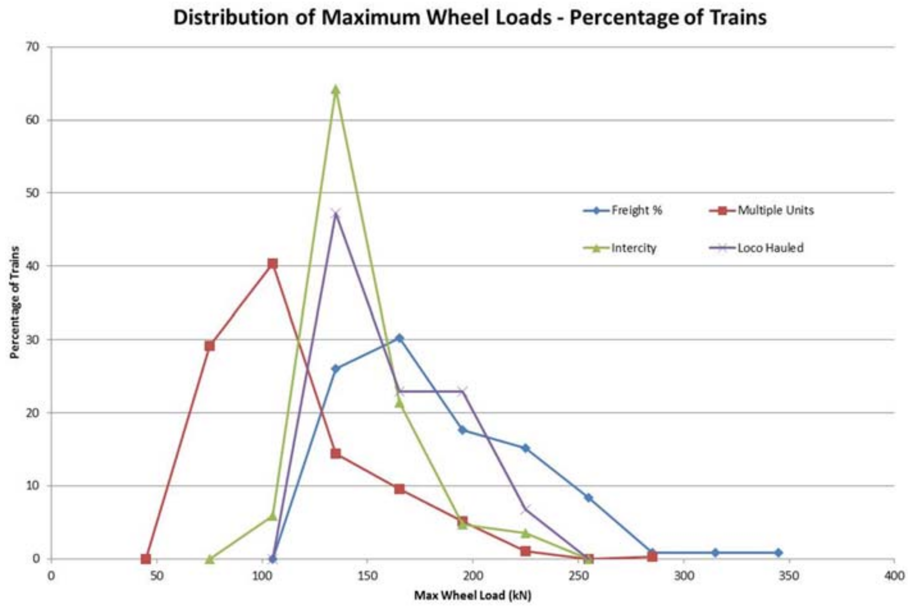
Designation / Symbol	Type	Cause / Generation Mechanism	Analytical Expression (j = 1, wheel 1) (j = 2, wheel 2)
Vertical Wheel Dynamic Load (Qdyn)	Dynamic	Track defects Rolling plane irregularities Discontinuities on the rolling plane Asymmetries or imperfections in the rolling stock	$Q_{dynj} = Q_{dyn1j} + Q_{dyn2j} + Q_{dyn3j}$
Total Vehicle Wheel Load (Qt)	Sum	All	$Q_t = Q_0 \pm Q_{wj} \pm Q_{nc} \pm Q_{dyn}$

Permanent features, such as switches and crossings (S&C) and transient features, can have a significant effect on the force experienced by the track through the amplification of the wheel load as a result of the change in wheel trajectory. An investigation into mitigation of impact forces on S&C components completed by Delft University (Markine, 2009) highlights that the force applied by the wheel on the rail as it passes and impacts the crossing nose was found to be between 2 to 3 times the quasi-static wheel loads and other short wavelength irregularities can be as high as 5.5 to 6 times the quasi-static wheel load.

Data gathered from operational trains suggest that it is not uncommon for vertical dynamic loads to reach magnitudes equal to two to three times the static wheel load. Such data is available in the UK from WheelChex or GOTCHA systems installed on the Network Rail network, which record maximum vertical wheel/rail forces at a number of sites. This force data is monitored and recorded by Network Rail for the purpose of identifying vehicles which have anomalous wheel loads. Generally these occur as a result of wheel damage which can generate large wheel/rail impact forces, but they can also be a result of uneven vehicle loading or suspension failure. The system usually records only vertical wheel/rail forces, so would not be a suitable source of information for lateral or longitudinal forces.

A sample of typical GOTCHA data has been analysed from two sites. The results are shown in Figure 2. It can be seen that the maximum peak wheel load distribution is strongly dependent on the static wheel load, as the lighter multiple unit rolling stock generally has lower peak wheel loads. The higher speed intercity type trains have a relatively narrow spread of peak wheel loads, which is likely to be an indication that the rolling stock parameters which can influence dynamic wheel loads (such as wheel damage) are better controlled. The freight rolling stock has the broadest spread of peak wheel loads and is also responsible for the highest peak wheel loads. This is to be expected due to the variability of loading of freight rolling stock, stiffer suspension, and the higher propensity for freight wheels to suffer from damage.

Figure 2 - Typical Maximum Wheel Load Distribution from GOTCHA data – Percentage of Trains with Peak Wheel Load (Source: Mott MacDonald)



It is considered that data from GOTCHA and WheelChex sites would be a good source of information for assessing how vehicle type and speed influences track forces. However, as the forces are measured at a fixed location the influence of track quality, specific track features (curves, cant, dipped joints, switch and crossings) or the trackform design cannot be assessed.

Investigations completed for RSSB by AEA Technology Rail under T088 found wheel irregularities, such as out-of-roundness, ovality and flats, also influence the vertical force applied to the rail. The amplification effect on the vertical force through investigation of the RMS accelerations at the axle box was found to range from 4.4 to 15.4, depending on the type of rolling stock.

Whilst these figures are provided as a representative data set, they highlight that in the process of establishing appropriate amplification factors for a particular network, it is critical to understand what the impact of track and wheel maintenance regimes will have on the amplification of loads at the wheel rail interface. Large amplification factors may be discounted if a stringent track and wheel maintenance regime is implemented. Influence of track maintenance condition is highlighted in further detail in Section 3.

We do not have any definitive guidance in the UK, highlighting the need for WP02 of this project.

2.2.2 Lateral loads

Lateral loads acting on the railway track are inherently linked to traffic safety and passenger dynamic comfort, but their role in the stress applied to the components of the track is also very important. Lateral loads applied to the track are a combination of quasi-static forces due to centrifugal effects and vehicle dynamic forces arising from the complex interaction of vehicle suspension systems and the condition of wheel and rail, including complex parameters such as wheel and rail transverse profiles and friction/adhesion in the contact between them. To this can be added the effect of external excitation due to cross-winds.

Several authors (Esveld, Iwnicki, Schmid) present useful, detailed descriptions of the forces arising due to vehicle curving behaviour and some details are presented in Appendix C.2.

The quality of the track geometry contributes significantly to the development of dynamic lateral forces. Mitigation measures include restoring the desired profile of the wheels sets at regular controlled interventions and the maintenance of the wheel - rail interface within prescribed tolerances to design or to a target profile that contributes to reducing forces and wear at that interface.

Lateral dynamic loads are complemented by quasi-static loads due to the residual centrifugal force and cross-wind forces; residual centrifugal forces only apply to curved sections of track, whilst cross-wind forces can apply along straight and curved track sections.

Further information on the derivation of the analytical expressions for the lateral loads is provided in Appendix C.2.

The total lateral force ΣY exerted on the track is equal to the algebraic sum of all lateral forces at any moment:

$$\Sigma Y = \text{Sum of all lateral forces acting on the track} \quad (2)$$

Depending on the rolling conditions, the lateral force in a straight section of track and in the absence of cross winds can be described as:

$$\Sigma Y = \pm(T_1 + T_2) \pm S_p \pm P_{dyn} \quad (3)$$

Research completed by British Railways Board into the design of driving axles established that the lateral forces were typically within the ranges provided in Table 2.

Table 2 - Range of Typical Lateral Forces (Source: T.72 – Design of Driving Axles with Axle Hung Traction Motors, British Railways Board, 1967)

	Lateral Force Range		
Load Origin	**Lower**	**Upper**	**Exceptional**
Loads due to the Effect of Cant	0.05Q	0.1Q	-
Flanging Loads	0.25Q	0.35Q	0.75Q
Transverse Friction Loads	0.15Q	0.35Q	0.95Q
Cross Winds (up to 75mph)	0.025Q	0.035Q	-

The force values detailed in Table 2 demonstrate that the highest lateral wheel/rail forces arise in curved track sections or due to lateral track alignment defects. The forces are transmitted both by contact between the wheel flange and the rail and by the friction between the wheel tread and the rail head. In general it has been found that the highest lateral forces occur at the leading axle and that the magnitude of the forces increases with decreasing curve radius. These forces are also strongly influenced by the bogie configuration of the rolling stock and the available friction at the wheel/rail contact. Effects such as forces due to cant or wind loading are much less significant.

Investigations completed for RSSB by AEA Technology Rail under T088 'Vibration environment for rail vehicle mounted equipment' found wheel irregularities, such as out-of-roundness and yaw dampers, also influence the lateral force applied to the rail. The dynamic amplification effect on the lateral force through investigation of the RMS accelerations at the axle box was found to range from 3.8 to 11, depending on the type of rolling stock.

2.2.3 Longitudinal Loads

An overview of the longitudinal loads exerted on the track is provided in Table 3.

Table 3 - Track longitudinal loads (Source: Pyrgidis, 2009

Designation / Symbol	Type	Cause / Generation Mechanism	Analytical Expression (j = 1, wheel 1) (j = 2, wheel 2)
Traction (force)	Static	Train traction Wheel rail adhesion	$F_t = P_t \cdot V$
Thermal forces (N)	Static	Eddy current braking	$N = \pm E \cdot A_R \cdot a_t \cdot \Delta_t$
Braking forces (Nbr)	Static	Vehicle braking Wheel rail adhesion	$N_{br} = 0.25 \cdot \Sigma Q$
Longitudinal creep forces (X)	Dynamic	Wheel profile, rigid wheel linkage, Wheel rail friction, Longitudinal creep phenomena	Straight Track $X_1 = -c_{11} \cdot \left(\frac{x'}{V} - \frac{e_o}{V} \cdot a' - \frac{\gamma_e}{r_o} \cdot y\right)$ $X_2 = -c_{11} \cdot \left(\frac{x'}{V} - \frac{e_o}{V} \cdot a' + \frac{\gamma_e}{r_o} \cdot y\right)$ Curved Track $X_1 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y + \frac{e_o}{R_o}\right)$ $X_2 = -c_{11} \cdot \left(\frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{R_o}\right)$

For many railway networks, braking and acceleration forces are considered to range between 25 % - 30 % of the nominal axle loads. For instance, for Q = 16kN, then N_{br} / N_{acc} = 4kN – 4.8kN. Others use maximum train deceleration rates. Where these forces should be applied is an important question, though most specifications apply them uniformly along the length of the rail supporting the train in question.

Note: Creep forces are also effected as a result of acceleration and braking with high performing rolling stock and on steep grades through a complex non-linear relationship between adhesion and tractive effort. Rhodes et al. (Rhodes 2008) found that the performance requirements specified by relevant standards more than adequately compensate for the influence of grade on the acceleration and braking forces typically found on passenger, freight and mixed-use railways, due to the requirement to provide restraint against creep within CWR arising from thermal effects.

The creep forces generated during curving (differential radius effect) is not covered in this report and requires further investigation in WP02.

Longitudinal forces as a result of gradient have been omitted due to the small effect that these have relative to traction or braking forces. For example, a 3 % grade will contribute 0.05 % to the longitudinal force.

3 Review of design practice for track, other transport systems, Earthworks and bridges

3.1 General

When designing the railway track system, its substructure, and components, it is essential that you take the loading into consideration. The track structure needs to be designed and installed to ensure that it is able to withstand all loads likely to occur, and meet the specified performance requirements.

Over the years, several suggestions have been made by various researchers and administrations respectively as regards the loads that need to be considered in the design of the track structure and its components. Some of them are now being considered obsolete as knowledge in the area has increased over time.

The current section is mainly based on a literature research of available information on track design loading, complemented by review of a range of past project technical specifications, project design experience and the conclusions drawn from a cross discipline research covering the relevance of design practice for other transport systems and engineering applications (highways, earthworks, aviation, maritime and bridge engineering) to rail vehicle loading requirements. The section concludes with a summary of the current state-of-the-art on the subject of track loading, focusing in particular on the identification of gaps in current knowledge and the potential for further investigation.

3.2 Track loads – historic review

3.2.1 Vertical loads

The total vertical load exerted on the track comprises several potential load components (as set out in Section 2), including the dynamic component. The difficulty of specifying a design load value for the vertical track loading is associated mainly with the definition of its dynamic component.

There are two commonly established approaches to the design of track systems. These are further discussed in the following sections.

3.2.1.1 Dynamic Amplification Factor Approach

A large number of methods available in the literature employ the Dynamic Amplification Factor (DAF) (or Impact Factor) approach for determination of the design wheel load. This requires the design vertical wheel load Q_d to be expressed as a function of the static wheel load Q_o or the effective wheel load Q_{eff} (Esveld, 2001), where applicable:

$$Q_d = Q_o \text{ or } Q_{eff} \cdot DAF \tag{4}$$

Where DAF = Dynamic Amplification Factor (dimensionless, always >1).

The expression used for calculation of the DAF is established through on-site testing and has always been expressed in terms of the train's speed. When developing expressions for the DAF, the number of factors being considered depends upon the amount and quantity of track instrumentation used and the assumptions adopted in relating the parameters (Doyle 1980).

In the following sections various expressions that have been developed for determining the DAF are reviewed and discussed.

AREMA formula

The AREMA formula is cited in AREMA (2005) and Hay (1982). The DAF is defined as a function of the vehicle speed and the wheel diameter, namely:

$$DAF = 1 + \frac{D_{33}V}{D_{wheel}100} \tag{5}$$

Where; D_{33} = diameter of a 33-inch reference wheel, V = vehicle speed (in m/h) and D_{wheel} = wheel diameter (in inches).

Translating to metric units, this formula becomes:

$$\mathrm{DAF} = 1 + 15.21\frac{\mathrm{V}}{\mathrm{D}} \tag{6}$$

With V = velocity (km/h) and D = diameter of vehicle wheels (mm).

The tonnage carried by a train and the speed factor (as in the case for sleeper loading) are specified separately (AREMA 2005).

ORE formula

The Office of Research and Experiments (ORE) of the International Union of Railways (UIC) developed a method (ORE 1965), where the impact factor is defined in terms of three, dimensionless speed coefficients α', β' and γ' as:

$$\text{DAF} = 1 + \alpha' + \beta' + \gamma' \tag{7}$$

where α' and β' relate to the mean value of the impact factor and γ' to the standard deviation of the impact factor.

The coefficient α' is dependent upon:

- The suspension of the vehicle
- The vehicle speed
- The track geometry and quality

The correlation between α' and the influence of unintended variation in the track geometry quality is difficult to estimate due to errors in measurement. A theoretically perfect track system with no defects in the intended vertical alignment, α' is zero. For straight track with no vertical alignment defects and very fast traffic, α' was found to approach thirty five per cent, whereas in curved track, values of α' did not exceed eighteen per cent. For the most unfavourable case, α' increases with the cube of the speed and for the locomotives examined, it was empirically expressed as

$$\alpha' = 0.04\left(\frac{V}{100}\right)^3 \tag{8}$$

Where V = vehicle speed (in km/h).

The numerical constant above (0.04) is dependent mainly on the resilience of the vehicle suspension.

The coefficient β' is dependent upon:

- The vehicle speed
- Track cant deficiency

- The centre of gravity of the vehicle

The coefficient β' is the contribution resulting from the wheel load shift in curves (due to cant deficiency) and may be defined by either:

The French (SNCF) formula

$$\beta' = \frac{2d.h}{g^2} \qquad (9)$$

The German (DB) formula

$$\beta' = \left(\frac{V^2(2h + c)}{127\ Rg}\right) - \left(\frac{2h.c}{g^2}\right) \qquad (10)$$

Where; g = gauge width (in m), h = height of the centre of gravity of the vehicle (in m), d = cant deficiency (in m), c = cant (in m), R = radius of curve (in m) and V = vehicle speed (in km/h).

The two formulae are approximately equivalent. The DB formula is accurate, whereas the SNCF formula is approximate to within about one per cent. Values found in literature are mentioned to range between thirteen to seventeen per cent (Birmann 1965).

It was observed, under otherwise equal conditions, that the measured coefficient α' for straight track is in almost all cases larger than the value (α'+ β') for curved track (α' measured and β' calculated). It would therefore seem more appropriate to only consider values of α' as the mean value of the impact factor for use in design.

The coefficient γ' is dependent upon:

- The vehicle speed
- The age of the track
- The likely presence of hanging sleepers
- The vehicle design
- The maintenance conditions of the locomotive power units

The measured coefficient γ' increases with speed, and, as a first approximation, the following formula can be used, if experimental data are not available:

$$\gamma' = \gamma_o = 0.10 + 0.017 \left(\frac{V}{100}\right)^3 \tag{11}$$

Where V = vehicle speed (in km/h).

If the effects of other variables are to be incorporated, the above formula can be generalised as:

$$\gamma' = \gamma_o \,.\, a_o \,.\, b_o \tag{12}$$

Where γ_o = value determined by Equation 11, ao = locomotive factor relating to the maintenance condition (including the effects of locomotive age) and bo = track maintenance factor relating to the standard of the track.

The ORE (ORE 1965) have obtained typical values of the coefficients γ_o, a_o and b_o for various track and locomotive conditions. Implied in these values are the standards of track and locomotive maintenance that allow particular operating speeds to be safely maintained. The ORE emphasise the values of these coefficients were determined entirely from the average observed relative increase in the standard deviation of the mean rail force level. The following values of γ_o, a_o and b_o were recommended:

- For normal track with a maximum permissible speed up to 140 km/h:
  - γ_o =0.11, (from observation this compares with the value of 0.15 obtained from equation 11).
  - Locomotive maintenance factor, a_o = 2.00.
  - Track maintenance factors, b_o = 1.30.
- For special track with an authorised speed of 200 km/h, assuming new vehicles:
  - γ_o =0.24, (from equation 11).
  - Locomotive maintenance factor, a_o = 1.50.
  - Track maintenance factor, b_o = 1.20.

A maximum value of the track maintenance factor, b_o was also determined for a track with relatively poor ballast compaction beneath the sleepers. Under load this track was observed to have 3mm voids between the sleeper and the ballast and this condition coupled with high speed traffic gave a maximum value of b_o equal to 1.70.

In summary, the ORE have observed that the maximum value of the impact factor occurs in straight track. Consequently, the formula for the impact factor (Equation 7), reduces to:

$$DAF = 1 + \alpha' + \gamma'$$

(13)

Using Equations 8 and 12 and the ORE recommended values of γ_o, a_o and b_o from the test data for normal track with permissible speeds of up to 140 km/h, the maximum value of the impact factor can be estimated from (Doyle 1980):

$$\text{DAF} = 1.29 + 0.04 \left(\frac{\text{V}}{100}\right)^3$$

(14)

Where V = train speed (in km/h)

British Railways (BR) formula

A simple model for a discrete irregularity, such as a dipped rail joint has been developed, which illustrates the combined effect of vehicle speed, unsprung mass and track irregularities (Jenkins et al. 1974, Koffmann 1972).

The model was developed for BR main line track consisting of 56kg/m continuously welded rails and with concrete sleepers spaced at 760 mm.

The resultant dynamic wheel load due to a wheel striking a dipped rail joint was determined from:

$$\text{P} = \text{P}_\text{s} + 8.784\ (\alpha_1 + \alpha_2)\ \text{V} \left[\frac{\text{D}_\text{j}\text{P}_\text{u}}{\text{g}}\right]^{0.5}$$

(15)

Where P_s = static wheel load (in kN), P_u = unsprung weight at one wheel (in kN), D_j = track stiffness at the joints (in kN/mm), g = gravitational constant (in m/ s^2), $(\alpha_1 + \alpha_2)$ = total rail joint dip angle (in rad) and V = vehicle speed (in km/h).

Therefore the dynamic factor can be defined as:

$$DAF = 1 + \frac{8.784\ (\alpha_1 + \alpha_2)\ V}{P_s} \left[\frac{D_j P_u}{g}\right]^{0.5}$$

(16)

For BR conditions and with a track consisting of continuously welded 56kg/m rails on concrete sleepers spaced at 760 mm, a value of D_j equal to 88 kN/mm has been considered as adequately representative for use in the above equation (Koffman 1972), together with a rail joint dip of 10mm and a corresponding value of $(\alpha_1 + \alpha_2)$ = 0.015 rad.

Eisenmann's formula

This method adopts a statistical approach to determine the magnitude of the impact factor. Eisenmann's method (Eisenmann et al. 2000, Fastenrath 1977, Eisenmann 1975) has been accepted for use by many European railway administrations and is based on the following observations and assumptions (Esveld 2001):

Measurements, on which this method is based, have shown that the stresses in the rail foot, from a statistical point of view, have a normal distribution.

The mean value is independent of running speed (initially studied up to 200km/h) and can be determined with sufficient accuracy using Zimmermann's longitudinal beam calculation.

The standard deviation is dependent on the running speed and state of the track.

The DAF equals:

$$DAF = 1 + \tau.\varphi.\eta$$

(17)

Where ϕ = a factor dependent upon the track condition (coefficient of variation), η = a speed factor and t = multiplication factor of standard deviation, which relates to the confidence interval being considered.

The value of ϕ is determined by the quality of the track and the following values have been suggested for use:

ϕ = 0.1, for track in very good condition

ϕ = 0.2, for track in good condition

ϕ = 0.3, for track in poor condition

The value of η is determined by the speed of the vehicle, V (km/h) and the following values have been suggested for use:

η = 1, for vehicle speeds up to 60 km/h

$\eta = 1+(V-60)/140$ for vehicle speeds in the range 60 to 200 km/h

$\eta = 1+(V-60)/380$ for vehicle speeds above 200km/h

The constant t characterising the probability of occurrence of the load value, ranges between:

t = 1, corresponding to a probability of occurrence of 68.3 %; usually used to define the design load value to be used in the dimensioning of subgrade layers.

t = 2, corresponding to a probability of occurrence of 95.4 %; usually used to define the design load value to be used in the dimensioning of sleepers (Alias 1984) or ballast (Esveld 2001).

t = 3, corresponding to a probability of occurrence of 99.7 %; usually used to define the design load value to be used in the dimensioning of rails (Esveld 2001).

t = 5, corresponding to a probability of occurrence of 99.9 %; usually used to define the design load value to be used in the dimensioning of rails (Orringer et al. 1998).

Other similar formulae

Several other references have been found as part of the literature review. Doyle (1980) for instance, has reviewed as part of his research several expressions that have been developed to determine the magnitude of the Impact Factor (or DAF). The formula by Agarwal (1974) attempts to relate the track condition to the impact factor by making use of the measured values of the track modulus. The formula by Schramm (1961) on the other hand is a typical expression developed solely on kinematic considerations. The formula by Lombard (1974) is of the same form as the AREA Formula (Prause et al. 1974), but calculated for narrow gauge track structure. The formula by Clarke

(1957) is simply the algebraic combination of the AREA Formula and the Indian Formula (Agarwal 1974) and, as such, is not modelled on any experimental results. The range of available formulae established on different bases emphasises the problem of using highly empirical formulae without adequate knowledge of the track conditions and assumptions used in their derivation.

Summary

All formulae reviewed take into consideration velocity as part of the calculation of the DAF, which highlights that the operating speed is a key contributor to the amplification of the static or quasi-static wheel load. However, there appears to be considerable variance in the other factors that are considered:

- The AREMA methodology considers that the wheel diameter applying the load is the only necessary additional variable.
- The ORE methodology factor for track geometrical considerations, vehicle condition and track maintenance condition, is limited by operating speed. This methodology ignores the contribution of the rolling stock sprung and unsprung masses.
- The BR methodology considers operating speed and focusses on the excitation of the rolling stock sprung and unsprung masses due to the influence of specific track features.

The Eisenmann method is the only methodology that applies a statistical approach to calculating the amplification factor as a result of speed and track condition, but ignores rolling stock condition.

Table 4 summarises the key input parameters to each DAF methodology.

Table 4 - Summary of input parameters to DAF formula

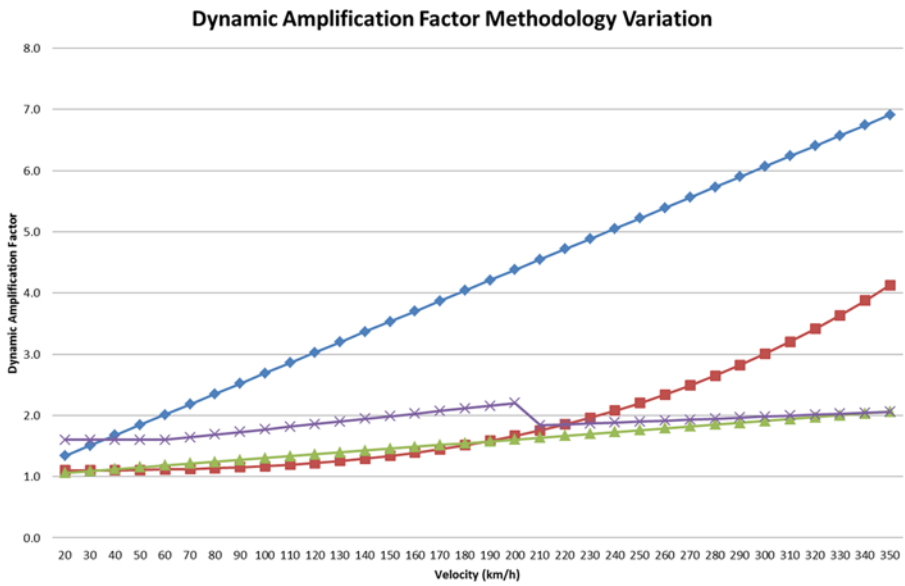
	Vehicle Parameters						Track Parameters						
Formula	Train Speed	Wheel Diameter	Static Wheel Load	Unsprung Mass	Centre of	Gravity of Vehicle	Locomotive Vehicle Condition	Track Stiffness at Joints	Track	Joint Dip Angle	Cant Deficiency in Curves	Curve Radius	Track Maintenance Condition
AREMA	X	X											
Eisenmann	X										X		
ORE	X				X	X			X	X	X		
BRR	X		X	X			X	X					

Generally, DAF formulae have been developed empirically through the investigation of various track maintenance condition and wheel maintenance condition parameters, each of which will have improved since these were first developed. To understand the current applicability of these formulae would require modelling these parameters to understand the influence on the DAF and to take into consideration technological advancements in the wheel-rail interface and rolling stock bogie configuration.

Indicative values for a DAF are shown in Figure 3. They have been calculated for a range of speeds between 20km/h and 350km/h using these parameters:

- Wheel Diameter – 900mm
- Vehicle Maintenance Parameter – 1.5
- Track Maintenance Parameter – 1.2
- Dipped Joint Angle – 0.125 rad
- Joint Stiffness – 10kN/mm
- Sprung mass – 20t
- Unsprung mass – 3t
- Track Quality Index – 0.1
- Probably of occurrence factor – 3

Figure 3 - Variation in DAF with Speed for Different Methodologies (Source: Mott MacDonald)



Spectral Density Functions / Standard Deviations Approach

An approach based on statistical concepts has been proposed and utilised in practice by the French Railways (SNCF) since the 1970s, following the first field trials undertaken after the launch of the first generation of high speed lines. The concept consists of approaching every load group of the total vertical load exerted on the track independently (see Section 2). The dynamic loads are calculated using a statistical approach. Such an approach is particularly appropriate in view of the random character of vertical dynamic loads. Compared to the German method (Eisenmann et al. 2000, Fastenrath 1977, Eisenmann 1975) this is a far more rigorous approach, involving analytical relationships, which take into account the actual and target condition for track stiffness as well as certain rolling stock related parameters.

The design vertical wheel load Q_d is numerically expressed as:

$$Q_{dj} = Q_{oj} + Q_{Hj} + n_p \cdot \left[\sigma\left(Q_{dyn,3j}\right)^2 + \sigma\left(Q_{dyn,1j} + Q_{dyn,2j}\right)^2\right]^{\frac{1}{2}} \tag{18}$$

Where:

Q_o = vertical static wheel load

Q_H = vertical quasi-static wheel load

$\sigma_{(Qdyn3)}$ = standard deviation of the vertical dynamic forces of the un-sprung masses of the vehicle

$\sigma_{(Qdyn1, Qdyn2)}$ = standard deviation of the vertical dynamic forces of the sprung and semi-sprung masses of the vehicle

n_p =multiplication factor, which relates to the probability of occurrence of the load being considered and

j=1,2 = index related to the 2 wheels of the same axle.

SNCF formula

The method cited in the French literature (Alias 1984, SNCF 1981, Prud'homme 1975) defines the design load in accordance with equation 18. The method allows for the actual or target design, maintenance etc. and the track quality (expressed in terms of the power spectral density and standard deviation of the rail and track level irregularities respectively) to be considered. Semi-analytical relationships allow for the calculation of the vehicle's dynamic reactions in the low and mid-high frequencies, that is:

$$\sigma(Q_{dyn1.2}) = (V\text{-}40) \text{ X } NL \text{ X } Q_o \text{ X } 10^{-3} \tag{19}$$

Where Q_o = vertical static wheel load (in tonnes, t), V =vehicle speed (in km/h), NL =mean standard deviation for the track level (in mm).

$$\sigma\left(Q_{dyn3}\right) = \left(\frac{0.3k_1}{200}\right) \times V \times \left[\frac{(k_t \times m_w)}{1.780475}\right]^{\frac{1}{2}} \tag{20}$$

Where k_1 = constant that characterizes the roughness of the running surface of rails, V =vehicle speed (in km/h), k_t = track dynamic stiffness (in t/mm) and m_w = wheel unsprung mass (in t).

The constant k_1 derives from measurements made on ground and un-ground rails and expressed in terms of their power spectral density.

The constant n_p in the design vertical loads equation above (18) characterises the probability of occurrence of the load value and ranges within the same values as for the Eisenmann method. The value $n_p = 2$ is recommended for the dimensioning of sleepers and $n_p = 3$ is recommended for the rails (Alias 1984). Other researchers (Giannakos 2014, Demiridis et al. 2010, Demiridis et al. 2007) recommend higher values (see Section 3.2.1.1) based on benchmarks established from measurements.

Other similar formulas

Giannakos (2014) and Demiridis et al. (2010) have proposed variations of the French Railways formula. Giannakos' research has been driven following the failure of a significant number of duo-block concrete sleepers on the Greek mainline railway network (Giannakos 2014, Giannakos et al. 1994). The sleepers, in use since 1972, type Vagneux U2, U3 with RN fastenings for tracks designed for a maximum speed of 200 km/h, demonstrated extended cracking (more than 60 % of all sleepers). The methods cited in the international literature at that time (French, German and American) could not provide any satisfactory justification for the appearance of the cracks as they provided values of actions on the sleepers much lower than the cracking threshold and therefore no cracking should have occurred on that basis. After extensive research in partnership with SNCF and the sleeper manufacturers, the design loads were redefined (Giannakos et al. 1994) to reflect the actual conditions prevailing on the Greek mainline network. The revised design load calculation procedure was capable of predicting the extensive cracking that occurred for the U2/U3 sleepers. The calculations indicated actions in excess of the cracking threshold and, in some instances, even the failure threshold of the sleepers in use.

Demiridis et al. (Demiridis et al. 2010) proposed an extension to the French Railways formula based on analysis and calculation of the dynamic loads induced by the vehicle's unsprung mass (in place of equation 20) and their contribution to the vertical wheel load acting on the railway track. Simulation of the railway track assumed an infinite length Timoshenko beam with suspended masses instead of the infinite single Euler beam used in the original French Railways method; a better simulation of the railway track behaviour in the medium to high frequency range is thus achieved. The proposed method additionally takes into account the Hertzian spring stiffness together with the actual dynamic stiffness values for the elastic elements of the track (the rail pads in particular). The dynamic stiffness values were based on bench tests and field measurements, compared to the static track stiffness values used in the original approach.

3.2.2 Lateral loads

The definition of a lateral design load has proven to be far more complicated in practice than for the vertical load direction and this is reflected in the amount of literature available. The resistance of the railway track to vertical loads (ballasted track in particular) generally stays within the elastic limits, which greatly simplifies their analysis. However, resistance to lateral loads quickly exceeds the limits of linearity, hence resulting in residual displacements. A similar situation arises in the case of ballastless tracks, although it might reasonably be argued that non-linearity in the lateral direction is mainly limited by the response of the rail fastenings.

For the reasons above, lateral design load limits have been traditionally imposed on rolling stock, based on lateral track resistance limits (commonly referred to as the 'track shift limit'). The latter have been defined and validated from field trials, rather than establishing a required track resistance limit adapted to each type of rolling stock.

SNCF has been a significant contributor to the subject of lateral track shift starting with the work of Prud'homme (1967), who experimentally evaluated the lateral strength of a wooden sleeper track under a moving lateral load. These tests were conducted at Vitry-sur-Seine on U33 (46 kg/m) CWR track with wooden sleepers spaced at 0.58m. The test track had a curvature of 800m radius and was new, with no traffic consolidation. Track heating was utilized to increase the longitudinal thermal stresses in the rail. Two test loading vehicle designs were utilized. The first design was a single 'derailleur' vehicle with an 8.2m wheel base and a central axle capable of applying a maximum vertical load of 120kN and a maximum lateral load of 110kN. The second design, designated as the wagon 'tombereau' consisted of two coupled vehicles on parallel tracks. One vehicle propelled the other vehicle and applied a lateral load to one of its axles. The maximum loads for this design were 130kN vertical and 170kN lateral. Two test methodologies were used:

- Method 1: Vertical load kept constant and lateral load incremented after every pass or after every ten passes.
- Method 2: Both vertical and lateral loads kept constant at all passes.

Using Method 1, the cumulative residual deflection was obtained as a function of the lateral load, while Method 2 provided a relationship between residual deflections and a large number of passes. Results from both methods were used to define critical loads and conditions.

Based on these tests, Prud'homme developed the first empirical equation for the lateral strength of a wooden sleeper track under vertical axle loads. The Prud'homme limit is given by:

$$\Sigma\gamma - 10 + Q/3 \qquad (21)$$

Where $\Sigma\gamma$ = total lateral force (in kN) and Q = vertical axle load (in kN).

Over the years, this equation has served as a guideline for acceptable lateral vehicle loads on ballasted track systems. It originally defined the limiting strength of a track section, comprising wooden sleepers, rails and tamped ballast between the sleepers, required to prevent lateral shift under repeated loads. This formula did not account for the curvature and the rail thermal load effects.

Prud'homme later recommended a multiplying factor of 0.85 to cater for these effects. This is known as eighty five percent of the Prud'homme limit (SNCF 1981).

Samavedam et al. (1996) report more recent tests conducted by SNCF on the TGV Paris-South-East line near Tonnere to measure the lateral track resistance for new track designs and to compare this resistance with previous SNCF experiments conducted on older track sections. Tests were conducted using the 'derailleur' wagon, which applied vertical and lateral load to the track through a single central axle. Low speed passes were made with vertical load held constant throughout the testing phase and the lateral load incrementally increased after each set of three passes. The residual lateral displacement was recorded after each pass and the incremental change in displacement due to each vehicle pass obtained. The output data was analysed by Samavedam et al. and the results demonstrated development of instability in the track for a lateral load of ~93.7kN beyond which residual displacements would progressively accumulate without significant lateral resistance.

Cervi, 1994 captures in his paper the SNCF practice for high speed lines, where:

- The maximum repeated lateral axle load which can be applied to the track by the vehicle is defined by eighty five percent of the Prud'homme limit; 0.85(10+Q/3), with the 0.85 factor used to empirically account for curvature and thermal loads.
- The concrete sleeper track must be designed such that the static lateral track resistance equals 24 + 0.41Q for tamped track and 38 + 0.63Q for stabilized track, with the latter required for summer operations. For the TGV

axle load of 170kN, this limit equals 93.7kN for tamped track and 145kN for consolidated track. Note that the limiting resistance represents the maximum stationary lateral load that can be sustained by the track under the vertical load from the vehicle, without any permanent displacement of the track.

3.2.3 Longitudinal loads

It has not been possible to retrieve consistent records from the available literature regarding derivation of the longitudinal design loading values for track systems. Transit Cooperative Research Program (TCRP) proposes within TCRP Report 71 that the following criteria should be addressed as part of the design of direct fixation assemblies to reduce rail breakages and their consequences:

- Restrict broken rail gaps
- Restrict lateral offset of broken rail
- Control excessive traction force
- Ensure longitudinal rail effects do not govern the design of structures

A summary of the bases for calculation of longitudinal loads is provided in Appendix C.3. A review of the track Standards and Regulations covering such values is discussed in the following section.

3.3 Track standards and regulations

3.3.1 RSSB

Railway Group Standard GC/RT5021 provides the requirements for track geometry, track system, track components and switches and crossings. The requirements for vertical, lateral and longitudinal loads are summarised below.

3.3.1.1 Vertical load

A maximum static axle load of 250kN, a vertical dynamic force generated by the static wheel load and the low frequency dynamic forces P2 of 350kN per wheel and an occasional isolated vertical load of 500kN per wheel.

3.3.1.2 Lateral load

A 100kN lateral force generated by a train over a length of 2m is specified.

3.3.1.3 Longitudinal load

A longitudinal force of 1200kN per rail is specified to allow for train acceleration and braking and the thermal forces within the rail.

3.3.2 Network Rail

Network Rail Standard NR/L2/TRK/2102 provides the requirements for track geometry, track system, track components and switches and crossings, based on the requirements in GC/RT5021. The requirements for vertical, lateral and longitudinal loads, are summarised below.

3.3.2.1 Vertical load

A maximum static axle load of 250kN, a vertical dynamic force of 350kN per wheel and a peak occasional isolated force of 500kN per wheel are specified.

3.3.2.2 Lateral load

A 100kN lateral force over a length of 2m is specified.

3.3.2.3 Longitudinal load

A longitudinal thermal tensile force of 700kN per rail and a longitudinal thermal compressive force of 620kN per rail are specified.

Section 6.6.5 also notes:

> *'The track system designer shall confirm that, on structures that require expansion joints, the track system is designed to both adequately anchor the rail and resist longitudinal movement from traction forces and yet able to accommodate the thermal movement of the rail relative to the structure. Such structures shall have site specific design specification and acceptance in accordance with NR/L2/TRK/2500 and NR/SP/CIV/003'.*

3.3.3 London Underground

London Underground Standard LU S1157 identifies the performance, design and configuration requirements for new track components, configurations, layouts and track bed.

3.3.3.1 Vertical load

A static axle load of 12t for passenger vehicles and 17t for engineering vehicles is specified. No dynamic component is specified and specification of the design loading criteria includes the design tonnage and speeds as:

'Track system and component design shall consider the normal track loading generated by speed and tonnage of both service vehicles and engineer's vehicles using any particular route. As a minimum, designers should allow for a 20-year life and take into account:

a) A minimum annual tonnage of 24 MGT;

b) Minimum annual load cycles of 2.4 million 12-tonne axles at 120km/h (nominally configured as 2m between bogie axles and 10m between bogie centres) and 1.4 million 17-tonne axles at 50km/h;

c) The varying track conditions according to track category set out in standard S1159;

d) Maximum operating speeds of 120km/h for passenger trains and 50km/h for engineer's trains or on-track plant'.

It is further specified that:

'The design of track components shall allow for the range of unsprung mass of passenger and engineer's trains on LU and the dynamic loading from this mass'.

3.3.3.2 Lateral load

No value of lateral force is specified; it is implied however in several instances in the Standard that the track system and its components will be deemed to resist the forces exerted by the rolling stock.

3.3.3.3 Longitudinal load

No value of longitudinal thermal tensile force is specified; nevertheless, the temperature range values expected to be encountered in various parts of the LU network are provided so that these loads are calculated indirectly by the designer (Section 2.3.3.1).

There is no mention of traction and/or braking forces or other longitudinal loads.

3.3.4 Docklands Light Railway

DLR Engineering Standard ES-401 defines requirements and gives guidance to designers, contractors and concessionaires on the specific issues to be considered in the design and construction of the permanent way on the DLR.

It is to be read together with ES-709 defining the interfaces with the rolling stock.

3.3.4.1 Vertical load

A maximum static axle load value of 100kN for any passenger service vehicle and a maximum static axle load value of 160kN (subject to a maximum speed of 20km/h when over 100kN) are specified in ES-709. No dynamic component is specified however, a DAF is specified in DLR Engineering Standard ES-710 with regard to the rolling stock - infrastructure (civil works) interface of 1.2 for ballasted track and 1.4 for ballastless tracks, applicable to vertical live loads only.

For the rails, ES-709 specifies a vertical load of 150kN and a lateral load of 30kN applied to the rail head; both loads are static and considered to be applied at mid-span. Design and installation of the track should ensure no permanent deformation of the track system or progressive failure of the fastening system.

3.3.4.2 Lateral load

Lateral loads are not specified.

3.3.4.3 Longitudinal load

Longitudinal loads are not specified. However, values for emergency braking are specified in DLR Engineering Standard ES-710 relative to the rolling stock - infrastructure (civil works) interface as a distributed load of 80kN/vehicle acting over a 22m base.

3.4 Project’s experience

In addition to the summary of track standards and regulations presented in Section 3.3, which are applicable in the UK, a review of available technical documents from various railway administrations has been undertaken by the project team focusing on answering these questions:

- What do customers actually specify in terms of track loading parameters and why?
- What direction, if any was provided with respect to design method?
- How does a track system designer apply these requirements to provide a compliant design?

- What parameters are taken into consideration and included in the design? Why?
- How do tailor made track loading models compare with standard track load models?

3.4.1 Representative project experience

A review of three example design projects completed in the UK was undertaken to identify the different levels of information typically supplied to designers during the design phase for projects. These were: the East London Line Project, the development of a renewal trackform for an existing line section in tunnel, and one for a ballastless track structure for Crossrail, Review of ballasted track requirements for design as these were found to be covered by established construction specifications issued by NR and LUL.

Table 5 summarises the parameters provided in design specifications for the example projects.

Table 5 - Specified parameters for example projects (Source: Mott MacDonald)

Parameter	East London Line	LUL Slab Track FE Analysis	Crossrail	Singapore LTA	Taiwan High Speed
Static axle load	Specified as Load Model RA2	Not specified.	Design axle loads and vehicle speeds specified in the form of design load models.	Design value is specified	Derived from Load Model. UIC Load Model 71 is specified.
Static wheel load	Not specified. Derived from axle load.	Not specified.	Not specified. Derived from axle load.	Design value is specified	Derived from Load Model. UIC Load Model 71 is specified.
Design vertical load	Not specified. Designer guided to GC/RT5112.	Specified in design brief based on in-service loads. Taken as 85kN or -20kN following agreement between designer and LUL. Network specific value based on in-track testing.	Not specified. Designer guided to derive design vertical load by using an impact factor implicitly by the use of a dynamic analysis to either derive amplification factors or to directly apply dynamic loading resulting reliance on engineering judgement.	Specified as Type RL loading in accordance with Highways Agency Standard BD 37/01	Designer provided with a Modified UIC load model

Table 5 - Specified parameters for example projects (Source: Mott MacDonald)

Parameter	East London Line	LUL Slab Track FE Analysis	Crossrail	Singapore LTA	Taiwan High Speed
Design lateral load	Not specified. Designer guided to GC/RT5112. Lateral resistance specified as a minimum value to be achieved, to result in a maximum displacement under load.	Specified in design brief based on in-service loads. Taken as 80kN following agreement between designer and LUL. Network specific value.	**Centrifugal Train Loading** Formula to establish multiplier provided to enable centrifugal loading to be calculated. $\alpha = \frac{s^2}{127R}$ **Nosing Loads** Specified as 100kN coinciding with train bogie position, spread evenly across number of axles within a bogie.	User referenced to UK Highways Agency Design Manual for Roads and Bridges BD 37/01 for lateral design loads	Formula to establish multiplier provided to enable centrifugal loading to be calculated is specified. $\frac{V^2 \times f}{127 \times R}$ Centrifugal force is a multiple of the design wheel load. Load to be considered applied at 1.8m above the running surface and specified to act in combination with the vertical load.
Design longitudinal load	Not specified. Designer guided to consider braking and traction forces. Longitudinal resistance specified as a minimum value to be achieved, to result in a maximum displacement under load.	Specified in design brief based on BS5400. Taken as 20kN following agreement between designer and LUL. Network specific value based on on-track testing.	Acceleration and Braking Loads Designer guided to use 30 % of the vertical load. Thermal Loads Expected operational temperature range specified.	User referenced to UK Highways Agency Design Manual for Roads and Bridges BD 37/01 for longitudinal design loads	Methodology for calculating acceleration and braking forces is specified.

Table 5 - Specified parameters for example projects (Source: Mott MacDonald)

Parameter	East London Line	LUL Slab Track FE Analysis	Crossrail	Singapore LTA	Taiwan High Speed
Tonnage using the network	20MGT per year	Not specified	Not specified. Parameters provided to enable Designer to estimate annual tonnage resulting in reliance on engineering judgement.	Not specified in the standard	Not specified in documentation.
Vertical single axle/wheel load or load model	Not specified	Single wheel load at top of rail	Load model for various vehicles. Source not identified.	Not specified in the standard	Designer provided with a Modified UIC load model

Table 5 - Specified parameters for example projects (Source: Mott MacDonald)

Parameter	East London Line	LUL Slab Track FE Analysis	Crossrail	Singapore LTA	Taiwan High Speed
Loading of track system components	Not specified	Not specified.	Not specified resulting in reliance on engineering judgement.	Not specified in the standard	Methodology for calculating Transverse Load Distribution specified for mono-block and duo-blocker sleepers for ballasted track on bridge deck. Ranges of track stiffnesses are specified. Minimum performance requirements specified for sleepers and fastenings in accordance with EN13230-1 and EN13481-5 respectively. Deformation modulus specified for support layer.

Table 6 summarises the design approached used by the Designers for each of the example projects.

Table 6 - Design approach for example projects (Source: Mott MacDonald)

Design Element	East London Line	LUL Slab Track FE Analysis	Crossrail	Singapore LTA	Taiwan HighSpeed
Method for Design	Documents referenced; Anforderungskatalog zum Bau der Festen Fahrbahn – 4. Überarbeitete Auflage – Stand 01.08.2002" [Requirements Catalogue for the building of Firm Roadways (Slab Track) – Revision 4 - 01.08.2002], Modern Railway Track 2nd Ed. (Esveld, 2001)	Serviceability Limit States / Ultimate Limit States. Ultimate Limit State taken as 1.5x Serviceability Limit State	Specified to be Serviceability Limit States / Ultimate Limit States and Fatigue Loading check.	No specific method is documented.	Performance requirements specified in accordance with relevant standards, typically EN series for various components.
Dynamic Effects	Not specified. Designer referred to GC/RT5112.	Not specified. Agreed to be included in design wheel load between designer and LUL	Not specified. Designer guided to use an appropriate amplification factor resulting in reliance on engineering judgement.	User referenced to Highways Agency Standard BD 37/01 for impact factors	Method for calculating impact factor for structural dynamic effects specified. No method specified for wheel loads on track components.
Load Case / Combinations	Not specified. Designer informed that design axle load is equivalent to RA2.	Load combinations for investigation specified in design brief. Designer required to establish worst loading combination prior to commencing analysis.	Specified as being in accordance with BS5400: Part 2, including allowance for the impact of wheel loading and unloading due to lurching. Load combinations for analysis specified.	User referenced to Highways Agency Standard BD 37/01 for load models	Specific load cases and load combinations are documented for the designer to establish worst case loading condition

The review of these projects highlights that designers typically receive track design specifications that reference documents from other engineering disciplines, ranging from standards to design guidelines to academic texts, which contributes to an inconsistent approach in the design of track structures.

Discussions with LUL representatives highlighted that for unique or innovative track systems, specific vertical and horizontal loading values have been established by in-track monitoring of wheel loads.

International project experience has been summarised from the experience of project team members working on projects such as Athens Metro and Dubai Metro. From a customer's perspective, the following have been established:

Information provided to the designer or specifier of the track:

- Static axle load: This information is normally provided to the designer or specifier of the track system.
- Static wheel load: This information is normally provided to the designer or specifier of the track system.
- Design vertical load: This information is not normally provided as a single value to the designer or specifier of the track system; generally either a DAF or the load combination requirements are specified, leaving the responsibility for the derivation of the design load configuration to the designer or specifier of the track system.
- Design lateral load: Most administrations specify a maximum design value (single lateral load) applied at rail level. This value may, in some cases, derive from civil engineering standards (such as the 100kN value for nosing force set out in BS EN 1991-2 and historical standards such as BD37/01 and BS5400-2), in other cases the design value may be based on the Prud'homme equation values, or in rare circumstances, the result of a simulation or field measurements (or both) of the maximum guiding force applicable to the network, taking account of the operating conditions, maintenance levels and rolling stock in use. The latter approach has been applied in some cases for urban rail networks (for example, Athens Metro).
- Design longitudinal load: This information is usually not provided as a single value to the designer or specifier of the track system; in terms of the thermal loads, most administrations specify the temperature range that the railway track needs to comply with. Traction and braking loads would be specified in most cases either as fractions of the static axle load or absolute values based on distributed loads from standards.

- Tonnage using the network: For main railway lines, this would usually be specified in the network statement, project brief or other document, rather than in a Standard or Regulation. For urban networks, this information is usually either directly provided in a Standard or Regulation or can be deduced from the operational design figures for the line section being studied, if a new project.
- Vertical single axle/wheel load or load model: There is no clear tendency here at all and there is a very large spread in the specifications being provided. This issue is further covered in the next sections of the report.
- Loading of track system components: There is no clear tendency here at all either and there is a very large spread in the specifications being provided. Some administrations do provide values for certain aspects of the track system. For example, Athens Metro provided the loading at the crossing vee. This issue is also covered in the next sections of the report.

Method of approaching the design of the railway track:

- Method for Design: No specific method overall (analytical or numerical) is generally prescribed, the responsibility lying with the designer or specifier of the track system.
- Dynamic Effects: No specific method is generally prescribed for deducing track dynamic loading, the responsibility lying with the designer or specifier of the track system.
- Load Case and Combinations: No specific method is generally prescribed for defining the track loading cases and combinations, the responsibility lying with the designer or specifier of the track system.
- In general, responsibility for establishing the appropriate design methodology typically resides with the designer through the use of engineering judgement and experience to determine the approach to arrive at a satisfactory outcome. Where no clear guidance is provided, the design approach typically involves a blend of standards across a variety of disciplines in which the designer has confidence that the commercial risks associated with design are satisfactorily mitigated.

3.4.2 Typical issues associated with track structures

Poorly specified or designed track structures may result in the premature failure of components within the individual layers of the track structure. Review of the literature has found that fatigue failure is the typical failure mode which arises, as a result of the actual peak wheel load experienced in service exceeding the wheel load assumed during the design phase.

In the ballasted track case, typical failures include:

- Rail breaks, where excessive deflection of the rail under load results in bending capacity being exceeded;
- Concrete sleeper cracking, where rail seat loads generate bending in excess of the flexural strength capacity of the sleeper, resulting in loss of control of the lateral and vertical rail position;
- Ballast crushing, where insufficient surface area is provided at the load transfer point from sleeper to ballast structure, resulting in ballast fouling and detrimental effects to the track system modulus;
- Formation failure, where the bearing capacity of the formation/subgrade is exceeded as a result of the load transfer through the ballast structure being inadequate, resulting in ballast fouling and detrimental effects to the track system modulus;
- Track buckling, where longitudinal compressive forces within the lattice structure of the track system exceed the lateral resistance provided by the ballast structure.

For example, root cause analysis of sleeper failures on the Athens Metro established that the static wheel loads were not adequately increased/amplified during design to encompass the loading experienced in service.

In the ballastless track case, typical failures experienced are:

- Fatigue failure of the anchor studs when the lateral loading transferred to the stud is underestimated, particularly through curved track sections;
- Flexural failure of slab track, where the load transferred to the slab results in bending in excess of the capacity of the slab;
- Failures in the track structure not only affects the ability of the track to perform its of carrying railway traffic safely, but given the interdependencies between each of the layers in the track system, will also result in changes in the performance of each element. For example, flexural failure of the slab track will result in additional deflection of the rail section under loading, which will consequentially reduce the fatigue life and increase the likelihood of a rail break.

3.4.3 Discussion

Based on the above, as well as the broader track design experience of the project team members, we conclude that:

- There is less variation in the methodology used by track designers to approach ballasted track design when compared to ballastless track design.
- There is a considerable variation in the methodology used by track designers to approach ballastless track design; this covers all steps of the process: from the specification of the design loading and the definition of the design load combinations, up to the method of analysis of the track itself.

Two clear tendencies emerge with regard to ballastless track design:

- There are effectively two approaches employed by designers. The first one consists of approaching the track as a concrete slab or deck. This approach involves the adoption of civil engineering standards (particularly the EN 199x Eurocodes series) for the specification of load factors (where not otherwise specified) and load combinations; structural analysis of the track employs limit state principles for verification in accordance with the relevant codes and is usually FEM based. The alternative approach is to model the track as a pavement; structural analysis in this case employs simpler concepts such as for instance that adopted for a beam / plate on an elastic foundation and uses allowable stresses.
- The approach employed in the design and modelling of the railway track is inherently linked to the design loading regime considered in the analysis and vice versa. As identified above, the specification of load factors (where not otherwise specified) and load combinations for the track structure when approached as a slab or deck, would be effectively based on the existing civil engineering standards (particularly the EN 199x Eurocodes series). In the case of the track being approached as a pavement, where not otherwise specified, the design approach is likely to rely on the methods and principles adopted in other transport sectors (see Section 3.5).
- The approach employed in the design and modelling of the railway track structure is inherently linked to the strength limits considered in the analysis and vice versa (for example, concrete axial tensile strength vs. concrete flexural strength).
- Failure to adequately address wheel-loads in service may result in reduced performance of a particular element of the track system, which will then have knock-on effects for other elements in the system.

3.5 Cross-discipline experience and potential application to track engineering

3.5.1 Highways pavement engineering

The methodology for pavement design in the UK, described in Highways England (formally Highways Agency) Design Manual for Roads and Bridges Volume 7 Pavement Design HD 26/06 and in general worldwide, is based upon layered elastic analysis of stress and strain within an idealised pavement structure. The mathematical analysis is simplified by considering a single static loading, based upon a typical vehicle axle or single wheel load. This equivalent wheel load is four tonnes in the UK, derived from a single typical vehicle axle load of eight tonnes or 80kN. This load was developed in the 1960s to represent a common wheel load for an average commercial vehicle. As vehicle weights increased, research showed that potential additional damage to road pavements was proportional to the rising wheel load and in fact rose by a power of four. A damage factor relationship was developed as a means of relating all vehicles back to an equivalent number of standard axles. Similar methodologies exist in most major design methods across the world, relating damage factoring to local traffic configurations.

The criteria for design involves consideration of limiting factors for stress and/or strain derived from material characteristic maximum strain values that would lead to loss of serviceability, loss of structural integrity and ultimately failure. In the UK two primary criteria were developed based on observation of how pavements actually fail. The criteria are based on a limit on tensile strain in the upper, usually bound, layers that would lead to cracking and a limit on the compressive strain in the foundation that would lead to deformation (collapse) of the pavement. The UK criteria are presented in TRL Report LR 1132 (Nunn et al. 1982).

The criteria have been established by laboratory testing of materials, measurements of stress and strain values in actual road pavements and by observation and survey of road performance. The modelling considers material performance at the prevailing ambient temperature conditions. In the UK, for example, the temperature for asphalt is standardised at 20°C. For other methodologies a variable analysis of traffic loading and environmental conditions is possible. In the Australian mechanistic design methodology, as presented in the Austroads Guide to Pavement Technology Part 2 Pavement Structural Design (Austroads Ltd 2012), it is possible to model the effect of different temperatures at different depths in the pavement. In addition, the effect of vehicle speed and potential for stationary traffic can also be modelled.

In the UK, the limiting criteria are considered to be empirical as they are based upon likely stress/strain values given a number of loading repetitions. The criteria are compared to calculated stress and strain values from mathematical models, leading to a probability of achieving suitability for design based upon the intended design traffic levels. The relationships are represented in simple nomographs contained in Highways England Design Manual for Roads and Bridges Volume 7 Pavement Design HD 26/06.

Traffic levels for design are established by estimating the number of vehicles that will use the highway over the design period of the pavement, usually 20 or 40 years. Therefore the damage considered is cumulative, based on total traffic that will use the road during its serviceable life. An analysis of past and future traffic is undertaken and then the numbers of each classification of vehicle type, based on weight and wheel configuration, are used to establish a number of passes of the equivalent single wheel load. For vehicles having wheel/axle weights less than or greater than the standard wheel, wear factors are used within the equivalency calculation. In the UK, as explained above the wear factors consider that the rate of damage from an increase in wheel loading rises by a 4th power factor. The UK methodology is presented in Highways England (formally Highways Agency) Design Manual for Roads and Bridges Volume 7 Traffic HD 25/04.

Roads are divided into characteristic sections for design, where significantly different loading conditions exist, for example a main line motorway and junction slip roads would have different traffic levels and therefore different pavement designs.

In summary, the main features of highway load characterisation and design are:

- Loads based on standard wheel and axle (static value)
- Cumulative damage from equivalent standard loads is considered (the rate of damage from an increase in wheel loading rises by a 4th power factor)
- Pavement performance is modelled based on layered elastic theory
- Design criteria seek to limit fatigue and deformation
- Design is based on empirical relationships between loads and failure
- Most methods worldwide follow similar practice.

3.5.2 Earthworks engineering

The geotechnical design of the railway track has traditionally been empirical in nature. Where analytical approaches have been adopted, these have mainly been elastic analyses or related to resilient behaviour rather than assessing long term permanent deformations (Selig and Waters 1994). Additionally, testing to inform the elastic analyses is often based on the cyclic tri-axial testing, which cannot replicate the more complex stress paths known to develop during real dynamic loading (Gräbe and Clayton 2009).

The following sections review selected geotechnical design methods and guidance in use over the last 50 years around the world. The discussion is not intended to be exhaustive, but nevertheless is considered to provide sufficient guidance for drawing useful cross-discipline experience vis-à-vis track loadings and design.

3.5.2.1 International Union of Railways (UIC) Code of Practice

UIC Code 719R, Earthworks and Trackbed for Railway Lines, provides guidance on the required depths of trackbed layers. It covers both new railway lines and the maintenance of existing lines. The approach taken is largely empirical and is based on the experiences of SNCF from the operation of the TGV high speed railway lines and work by the European Rail Research Institute (ERRI). It should be noted that the UIC Code does not specifically cover high speed lines.

In this code, basic thicknesses for the trackbed layers are determined, based on a soil bearing capacity class (P1, P2 or P3, derived from soil quality class and CBR value); subsequent correction factors to account for the sleeper type, the class of traffic and the line speed are applied. The recommended minimum trackbed thicknesses are based on tracks with rail gauge values between 1435mm and 1668mm, a sleeper spacing of 0.6m and a maximum nominal axle load of hauled vehicles up to 250kN (25 tonnes). The required thickness of sub-ballast can be reduced by using a prepared subgrade and classes of imported material are given to facilitate the determination of the permissible reductions.

Dimensioning of the trackbed layers is linked to the static axle load and speed or UIC line classification group in accordance with UIC Code 714R (axle load, speed and tonnage borne related), where applicable.

3.5.2.2 American Railway Engineering Association

The manual of the American Railway Engineering Association (AREA 1996) recommended amongst other methods the classical empirical Talbot equation for the determination of the trackbed layer thickness. The Talbot equation was developed from the results of field tests conducted during the 1910s and 1920s and is based on an allowable subgrade pressure (a value of 138kPa was recommended by AREA) and the vertical stress applied on the ballast surface: the load applied through the track.

Raymond, 1985, modified this approach to reflect that in practice the allowable subgrade pressure varies widely. The Casagrande soil classification was used to relate safe bearing pressures for various subgrade soils. The stress calculations in the track and subgrade remained based on the assumption of a homogeneous elastic half-space representing the ballast, sub-ballast and subgrade; the properties of the individual layers were not considered. The influence of the number of load applications on subgrade soil performance was not included as a design parameter.

3.5.2.3 West Japan Railway Company Standards, Japan

The West Japan Railway Company issues construction and maintenance standards for their high speed (200km/h or 124mph) railway lines (Burrow et al. 2007). The principle of this design method is that the subgrade has a presumed bearing capacity of 288kPa and then the depth of the overlying trackbed layers are prescribed based on the anticipated annual tonnage of traffic on the line. Should the presumed bearing capacity not be achieved, then ground improvement is required in order to provide the required capacity.

Other Japanese railway companies, for example, the Central and East Japan Railway Companies are considered likely to adopt similar methods.

3.5.2.4 British Rail method, UK

In the British Rail method of trackbed design (Heath et al. 1972), a ballast thickness is determined based on limiting the predicted stresses in the subgrade to less than a 'threshold stress' value. The threshold stress is defined as the deviator stress above which plastic strains begin to rapidly increase. Below the threshold stress, the material behaviour is assumed to be elastic and resilient. Determination of the threshold stress is based on cyclic tri-axial tests carried out on undisturbed samples of intact material.

The method essentially comprises the use of a series of charts to define the thickness of the trackbed layers; these charts are based on the threshold stress and the assumption of a single layer linear elastic model developed to predict the stress distribution in the subgrade for a number of different sleeper spacings and applied loads, verified by site measurements. When this method was developed in the early 1970s there was little information on the corresponding track loading spectra with which to inform the model; the applied load is related to the thickness of the trackbed layers through the change in deviatoric or threshold stress induced in the soil. The assumption of a single material, homogeneous and isotropic foundation limits the application of this trackbed design method; it is, now, largely superseded by the approach of Li & Selig (1998a, 1998b).

3.5.2.5 Li & Selig approach

Li & Selig, 1998a, 1998b made improvements to analytical trackbed design through the use of GEOTRACK (Chang et al., 1980), a multi-layered elastic finite element model validated by extensive laboratory testing. The model allows different stiffnesses for the ballast, sub-ballast and formation to be considered. Input parameters are determined from cyclic tri-axial testing, which considers the sensitivity of the number of loading cycles, the soil type and the ratio of the deviator stress (the load induced in the soil from the track) to the soil strength (Li & Selig 1996).

The results from GEOTRACK are used to determine a required ballast thickness as a function of the subgrade's bearing capacity, in order to limit the plastic deformation in the subgrade to an acceptable value for a given number of loading cycles. This has been demonstrated through the results of cyclic triaxial tests, which provided a relationship between the cumulative plastic strain and the soil deviator stress to be developed: the change in deviatoric stress induced in the soil due to track loading (Li & Selig, 1996). Applying the method in practice takes place through the use of design charts.

3.5.2.6 Channel Tunnel Rail Link method

The highest speed trains in the UK run on the Channel Tunnel Rail Link (CTRL also known as High Speed 1) route. On High Speed 1 (HS1) the trains with axle loads of 170kN travel at speeds of up to 300km/hr (186mph) and impart around 0.5 million load cycles to the track every year. This equates to around 8.5 MGT per year. A further 0.5 million load cycles are applied by slower commuter and freight services (with up to 225kN loads) giving an overall applied loading of 18 MGT per year. Although the design of the CTRL was based on the empirical UIC Code 719R, Rail Link Engineering (RLE) recognised the limitations of the approach, namely the lack of an analytical basis and the uncertainty over its extrapolation to high speed lines and therefore reviewed the resulting design in the light of more rigorous methods.

Consequently, RLE reviewed the design method of Li & Selig, taking account of the displacements measured in the three European study sites reported by Woldringh & New (1999). The results obtained have shown that the Li & Selig method slightly overestimates the measured deformations, where measurements took place (O'Riordan 2003). This may be due to the dynamic load correction applied in GEOTRACK, overestimating the actual dynamic response (O'Riordan & Phear 2001). As a result, when RLE used Li & Selig to validate the results of trackbed design to UIC Code 719R, they used a reduced axle load correction to account for dynamic effects. This resulted in reasonable agreement between the two design methods (O'Riordan & Phear 2001). However, it is worth noting that UIC Code 719R generally predicts a thicker trackbed compared to Li & Selig; the exception is where there are high axle loads (Burrow et al, 2007). In this case, the adjustment of the static axle loads to account for the dynamic effects associated with high train speeds results in the application of particularly high axle loads.

3.5.2.7 Network Rail standards

Network Rail standard NR/L2/TRK/2102 refers to the design and construction of track. This standard sets out the principle that a sufficient depth and width of ballast shall be specified to:

- Distribute dead and live loads to the formation, subgrade and structures without overstressing them;
- Enable the track to be maintained to line and level;
- Provide longitudinal and lateral stability to the track system; and
- Facilitate the rapid dispersal of water.
- The forces specified in NR/L2/TRK/2102 are set out in Section 3.3.2.

Network Rail standard NR/SP/TRK/9039 Formation Treatments, relates the required thicknesses of the trackbed layers to undrained subgrade moduli depending on sleeper type and stiffness. It is presumed that the undrained subgrade moduli are derived from short term 'dynamic' tests with no account taken of local defects (fatigue failures) and associated impacts/dynamic forces and deterioration in the longer term. The origin of the required thicknesses is not clear, but the design approach suggested is very much one of learning from experience of the maintenance history of any particular line.

It is understood that Network Rail, at the time of writing, is in the process of developing a standard or design guideline where target formation stiffnesses will be specified to aid the design process. Standardising the target formation stiffnesses across a particular network or line category would assist in reducing doubt from track designers and constructors about the required formation stiffness.

3.5.2.8 European Codes of Practice

BS EN 1997-1:2004 + A1:2013, Geotechnical Design, Part 1: General Rules, does not explicitly consider the design of railway trackbed foundations, but it does contain a number of mandatory principles and other recommendations that are relevant to embankment design for infrastructure. In particular, a list of the limit states to be checked and complied with as part of the design process. It is recommended that, amongst others, the following limit states are checked:

- Deformation of the embankment leading to loss of serviceability;
- Excessive deformations in transition zones, such as the approaches to bridge abutments;
- Degradation of base course materials due to high traffic loads.

When considering the ground properties it is also a principle that account must be taken of any processes which may change those properties from values measured during testing. This includes the effects of time, climate and dynamic loading, all of which are relevant to railway embankment and trackbed design.

The definition of actions, the requirements for partial safety factors and the rules for combination of actions, are contained in EN 1990:2002. The values of actions are contained in EN 1991, including traffic loads and actions to be considered for checking the dynamic response of a structure (where relevant). Design requirements for earthworks subject to vertical loading from railway traffic is also covered in EN1991-2 Clause 6.3.6.4, but the Eurocodes are silent on acceptance criteria for the ground in terms of permissible settlements of track structures and where account needs to be taken of the relationship between speed and the response of the ground in terms of providing continued support to the track structure.

3.5.2.9 Summary

In summary, the geotechnical design of the railway trackbed is:

- Largely empirical in nature;
- Where analytical approaches have been adopted, these have mainly been elastic analyses or related to resilient behaviour;
- The applied loads are usually considered in terms of the change in the deviatoric stress in the soil subgrade, converted from a static axle load;
- More recently consideration has been given to the impact of long term permanent deformations on trackbed performance;
- The present state-of-the-art design methods for trackbed foundations are based on resilient modulus and permanent deformation; these parameters are estimated from conventional cyclic triaxial tests that do not reproduce the complex stress paths that the trackbed is subjected to; and
- North American and UK methods follow broadly similar practice.

3.5.3 Development of bridge load codes

3.5.3.1 Vertical loading

In the early days of the railways, the load models considered were uniformly distributed loads. Later, Railway Engineers developed load models that defined the heaviest rolling stock (axle loads and spacing) as precisely as possible.

Within the UK, vertical loading used for bridge design has evolved as summarised within a simplified timeline:

- Pre-1923: train loading used an average tons per foot (ignoring dynamic effects or axle configurations).
- 1923 to 1973: static railway loading represented by Type RA British Standard unit loading (for various gauges) or Type RB (for gauges of 4ft 8½" and over) and defined within BS 153: Part 3A. Appropriate additions were made to the type RA1 loading to account for impact (dynamic) effects.
- 1973 to 2010: European loading UIC 71, later adopted as the RU load model within BS 5400 Part 2. This static loading was multiplied by appropriate dynamic factors.
- 2010 onwards: Implementation of the Eurocodes, where UIC 71 is represented by Load Model 71, multiplied by appropriate dynamic factors.

The UK National Annex also requires the addition of a load classification factor.

Further discussion on the development of the above loading is provided below.

Type RA1 loading

The type RA1 load model (representing 1 British Standard unit) is defined within the historic document BS 153: Part 3A and for the mainline railways within the UK, the loading recommended by 'Requirements for passenger lines and recommendations for goods lines of the Ministry of Transport in regard to railway construction and operation (1950)' was 20 units of type RA1 loading.

For short spans, 20 units of RA1 loading is represented by two axles of 250kN at a spacing of 1.829m representing 25t axles at 6' spacing. For longer spans, a load model consisting of four axles between 150 and 200kN, along with a trailing load of 65kN/m is considered. The RA1 load models were developed to represent typical back-to-back goods trains.

For simply supported spans, type RA1 loading can also be represented by Equivalent Uniformly Distributed Load (EUDL).

Appropriate additions were made to the type RA1 loading to account for impact effects caused by the hammer blow of locomotives, rail joints and track and wheel irregularities. A basis for determining these dynamic effects was provided by the Report of the Bridge Stress Committee of 1928, based on dynamic testing of real bridges under a range of locomotives and speeds. The impact, or dynamic factor used is dependent upon the bridge elements under consideration (for example, longitudinal or transverse members), the permissible line speed and whether fatigue is being considered.

A series of factors are available by which the railway live loading can be reduced to account for load dispersion through the rail/sleepers/ballast and bridge structure. These factors reflect the element being considered and the track form.

The Type RA1 loading is still considered within UK underbridge assessments undertaken in accordance with Network Rail Standard NR/GN/CIV/025 and for defining the static load effects of rolling stock.

Type RU/RL loading

In accordance with BS 5400 Part 2, RU loading (including SW/0 loading for continuous beams) represents combinations of normal vehicles running (or projected to run at the time of publishing the British Standard) on railways within Europe, including the UK. This loading was adopted for bridge design carrying main lines of standard UK gauge (1.4m) or above.

RL loading is a reduced loading for passenger rapid transit railway systems that is considered for lines where locomotives or main line rolling stock do not operate. The loading was developed by LUL to represent the typical traffic that was expected to use its network.

The basis for RU loading is load model UIC 71 defined within UIC Leaflet 702. This load model was the first to envelope European rail traffic and does not represent a real train as it is a generic model which envelopes a range of axle loads and spacings. This load model was developed principally for longitudinal simply supported spans (noting that SW/0 loading is considered for continuous beams).

The development of load model UIC 71 was based on the results of a 1971 questionnaire issued by the International Union of Railways. The detailed questionnaire was completed by the European Railways and provided detailed information on their rolling stock (and train composition) in service. As a result of the questionnaire, six train types were determined to represent critical real trains. Motive power is now electric and diesel rather than steam and this presented axle patterns for locomotives that were similar to bogied freight wagons, noting that freight wagons are often heavier than the locomotives.

Within the six trains defined within UIC 776-1 Appendix 101, it was possible to define bending moments and shear forces corresponding to static live loads as an envelope curve (for train types in UIC 71). The following critical real vehicles (from existing European rolling stock) are considered within UIC 71:

- 2CC locomotives operating up to 120km/h
- Wagons (1) operating up to 120km/h (Guterwagon)
- Wagons (2) operating up to 120km/h (Guterwagon)
- Passenger trains operating up to 250km/h (Reisezug)
- Turbotrain operating up to 300km/h
- Special vehicles operating up to 80km/h (Sonderfahrzeug)

RU loading (as derived from UIC 71) is represented by four concentrated loads of 250kN (applied as 125kN per rail) at 1.6m spacing and a uniformly distributed load of 80kN/m before and after the concentrated loads. The concentrated loads represent two coupled locomotives and allow for determining the effect of individual axle loads on shorter loaded lengths. The uniformly distributed load represents the maximum load per metre of trains in service and is likely to be more relevant for longer loaded lengths.

RL loading is represented by a single 200kN concentrated load (applied as 100kN per rail) coupled with a uniformly distributed load of 50kN/m for loaded lengths of up to 100m. For loaded lengths in excess of this, the distributed load of 50kN/m is considered for the first 100m and reduced to 25kN/m for lengths in excess of 100m. To allow for lurching, this axle force is split 56/44 between the two rails.

The RU loading defined is equivalent to static loading and is multiplied by appropriate dynamic factors to account for impact, oscillation/resonance and other effects that may be caused by track or wheel irregularities. This approach was introduced within UIC 776-1R and requires checks to ensure that the deflection of the bridge is within the limits set out in UIC 776-3R, thus taking account of the natural frequency of the bridge. The research to inform the development of the dynamic factors included testing of real bridges by the International Union of Railways and the Office for Research & Experiment (ORE).

Within BS 5400 Part 2, a dynamic factor is determined for the design of bending moment effects and shear effects and is a function of the influence line for deflection of the element under consideration. The allowances for dynamic effects have been calculated so that they cover the effects of slow moving heavy and fast moving light vehicles (for example, heavy wagons of speeds up to 120km/h, passenger trains up to 200km/h and exceptional vehicles up to a speed of 200km/h).

For speeds greater than 200km/h, a special dynamic study would be required to determine the dynamic effects in accordance with the requirements of the relevant Railway Authority.

Concentrated loads, as defined above, applied to the rail can be distributed both longitudinally by a continuous rail to more than one sleeper and transversely over an area of deck due to dispersion by the sleeper and ballast. The distribution of concentrated loads applied to a non-ballasted track can be calculated based on the relative stiffnesses of the rail, the supporting elements and the bridge deck itself.

Deck plates and similar local elements are required to be designed to support a nominal load of 250kN for RU loading and 168kN for RL loading at any point of support of a rail. These loads are deemed to include all allowances for dynamic effects and lurching.

Load Model 71

With the implementation of the Eurocodes, the requirements of BS EN 1991-2 are not significantly different to that of the superseded BS 5400 Part 2. This is the result of the European collaboration achieved through the International Union of Railways, where consistent railway bridge design requirements have been adopted over many years.

Within the Eurocode requirements, the vertical static load effects are represented by Load Model 71 (LM71). This load model represents standard rail traffic operating over the European railway network as discussed previously and is defined within UIC 71. As such, the load model characteristic values considered are as defined within BS 5400 Part 2 for RU loading.

For continuous beams, the SW/0 load model is used as previously defined within BS 5400 Part 2. However, an additional load model SW/2 is included within the Eurocodes to represent the static load effects of heavy rail traffic that operate on some parts of the European network (for example, Scandinavia), although it is acknowledged that the other load models discussed, such as LM71 and SW/0, adequately represent the traffic operating on the UK network.

Provision is made within the Eurocodes to vary the characteristic values to account for different types, weights and volumes of rail traffic by the introduction of a factor ?. For international railway lines, a factor of ? not less than 1.10 is recommended and this is the value included within the UK National Annex. Within Network Rail Standard NR/L3/CIV/020 “Design of Bridges”, an ? factor of 1.21 is required – this value has been adopted so that Network Rail can achieve the in-service performance requirements outlined within the High Speed Technical Specification for Interoperability.

The static load effects determined (and enhanced using the ? factor) are multiplied by dynamic factors to account for dynamic effects. These factors take account of the following effects induced on a bridge due to moving rail traffic and consider effects for track with standard maintenance and for track that is carefully maintained:

- The inertial response (impact) of the bridge due to the rapid rate of loading caused by the speed of the rail traffic crossing the structure.
- Potential bridge resonance that may be caused by the passage of successive loading with approximately uniform spacing that excites the structure, such as where the frequency of excitation matches the natural frequency of the bridge.

- Increase in wheel loads caused by track or vehicle imperfections.

As for the dynamic factor formulas defined within BS 5400 Part 2, those adopted within the Eurocodes are based on the work undertaken by the International Union of Railways (and others) and defined within UIC 776-1R.

The Eurocode provides guidance regarding the longitudinal and transverse distribution of load to account for the rails, sleepers and ballast. This guidance also considers the effect of eccentric loads (transversely) due to track cant.

Where the line speed exceeds 200km/h, the Eurocode provides guidance as to when a dynamic analysis is required as part of the bridge design. If a dynamic analysis is required, this requires consideration of the High Speed Load Models, HSLM-A and HSLM-B (for short spans). These load models represent the dynamic load effects of articulated, conventional and regular high speed trains in accordance with the requirements of the European Technical Specification for Interoperability for infrastructure.

The Network Rail Standard NR/L3/CIV/020 specifies additional loading requirements for bridges carrying directly fastened or embedded rails (this does not include rails attached to longitudinal timbers). A single static vertical design load of 600kN is applied directly to the parts of the structure that support the rail and is considered for ultimate limit states only. The load provided includes for partial load factors, dynamic and lurching effects. The basis for this loading is not fully understood, however the research undertaken would suggest that this applied load either represents a load effect caused by wheel flats, or represents a train derailment and subsequent impact load onto the bridge deck locally supporting the track.

American Railway Engineering and Maintenance-of-way Association (AREMA) models the vertical load as a series of static axle loads of 360kN with a spacing of 1.5m. For short lengths, these are increased by 60 % for impact loading.

Further detail on the original of the vertical wheel load s is provided in Appendix A.

3.5.3.2 Lateral loading

An allowance has been made historically within bridge design standards (BS 153, BS 5400, Eurocodes are examples) for lateral loads applied by trains to the track. This nosing force is taken as a concentrated load of 100kN (and 10 tons within BS 153) acting horizontally perpendicular to the track and at rail level. The 100kN is considered a nominal (BS 5400) or characteristic (BS EN

1991-2) load and applied in the least favourable position for the element under consideration.

In accordance with Network Rail Standard NR/L3/CIV/020, this nominal/characteristic loading may be distributed over three adjacent sleepers in the proportions of ¼:½:¼ for UK bridge design.

Historically, no justification or origin of the nosing load used has been provided. Research work undertaken in the early to mid-1990's by the European Rail Research Institute (ERRI) under project D181 considered lateral forces on railway bridges, although it did not critically challenge the nosing force requirements within design standards. As part of this work, BR Research calculated a range of lateral forces for typical UK vehicle configurations, speeds and bridge structures. The work undertaken by ERRI and BR Research observed:

- There is a strong influence of vehicle type and speed on the lateral loads determined.
- The work suggested that lateral force requirements for ballasted tracks may also be relevant to structures, employing the established Prud'homme criterion.
- There may be grounds to reduce the design nosing force on structures under certain circumstances, namely where axle loading is less than 22.5t, or speed of operation is less than the normal design speed for the vehicles.

Where a track on a bridge is curved, allowance for centrifugal action of moving loads is considered within the design of bridge elements. In the derivation of the centrifugal loads, similar combinations of vehicle weights and speed to those discussed previously have been considered (for example, heavy wagons of speeds up to 120km/h, passenger trains up to 200km/h and exceptional vehicles up to a speed of 200km/h). As such, the centrifugal force is determined by multiplying the static vertical loading (determined from the relevant load model considered) by a factor. The factor determined is a function of the greatest speed envisaged on the curve, a reduction factor accounting for speed and influence length and the radius of the track curvature. The effect of track cant is also considered.

The load determined for centrifugal effects is taken to act outwards in a horizontal direction at a height of 1.8m above the running service/rail level.

AREMA methodology applies a centrifugal load, dependent on the train speed, track radius and cant.

Further detail on the origins of the lateral wheel loads is provided in Appendix A.

3.5.3.3 Longitudinal loading

Traction and braking forces are considered acting at rail level and in a direction parallel to the tracks. These loads are considered as uniformly distributed over the corresponding influence length for the element of the structure being considered and the direction of the traction and braking forces applied take account of the permitted direction of travel on each track.

Within BS 5400 Part 2, the longitudinal loads are taken as 30 % of load on the driving wheels for traction and 25 % of the load on braked wheels for braking. For bridges supporting ballasted track, up to one-third of the longitudinal loads can be assumed to be resisted by track outside of the bridge structure (provided that no expansion switches or similar rail discontinuities are located on or within 18m of the end of the bridge).

The Eurocode requirements within BS EN 1991-2 for traction and braking are similar to the loading defined within UIC 776-1R. However, within the UK National Annex, the opportunity has been taken to include the provisions for traction and braking as defined within BS 5400 Part 2 and summarised above.

The Eurocode also provides guidance for the consideration of track-structure interaction due to variable actions, such as traction/braking forces and thermal effects and the influence of interaction between the structure and the track on the stresses in continuous rails. This is generally derived from UIC recommendations and provides guidance on the modelling and loading of the track and structure.

AREMA applies a longitudinal braking or traction force dependent on the length of the bridge or rail between discontinuities.

Further detail on the origins of the longitudinal wheel loads is provided in Appendix A.

3.5.4 Airfield pavement engineering

Basic pavement design methodology in the aviation sector is based upon the similar principles of materials' elastic behaviour as for highways. In the UK most airfield pavement design follows Ministry of Defence (MOD) practice, which is presented in Design and Maintenance Guide (DMG) 27 A Guide to Airfield Pavement Design and Maintenance (Defence Estates, 2006), which is an update of the previous PSA Green Book, which was for many years considered as the reference document for airfield pavement design worldwide. The methodology is again largely empirical, based upon modelling of the behaviour of actual pavements and experience of what has and hasn't worked in the past.

Loading requirements are established by reviewing the actual aircraft using the airfield facility. The most commonly used type of aircraft for each airfield facility becomes the design aircraft and then all other arriving and departing aircrafts are equated to the design aircraft for an evaluation of traffic using the airfield facility on a daily basis. This process is called a Mixed Traffic Analysis and the equivalency is based upon actual number of passes of each aircraft and its potential damaging factor compared to the design aircraft.

The pavement thickness design is based upon the damaging effects of passes of the design aircraft and equivalents. Aircraft wheel loading varies depending upon the aircraft mass and its undercarriage configuration (number of wheels, sets of axles, single, dual, multiple etc.). Obviously damage increases as mass increases, however, larger aircraft tend to have more wheels in each undercarriage and several sets of undercarriage. For example, some large transport aircraft, particularly military types, have multiple wheels and are designed to land upon unmade or poorly maintained runways. The damaging effect of an individual large aircraft is relatively low due to the load spreading of many wheels. The damaging factor is represented by an Aircraft Classification Number (ACN), which relates to its overall mass and wheel configuration. The ACN, in effect, equates the actual main landing gear of the aircraft to a single equivalent load although the main landing gear maybe single, dual, tandem, tridem etc. The conversion is based on the reaction of a reference theoretical pavement to a given limiting stress or a number of applications of the load. Whilst this approach is broadly empirical, it does, theoretically, take account of both static and dynamic loads, because the modelling and limiting factors are based upon observation of the performance of actual pavements.

Airfield runways and other airfield pavements are assigned a Pavement Classification Number (PCN), which, for design purposes, must be equivalent to or exceed the ACN of the design aircraft. ACN/PCN is based upon experience and empirical factors giving a probability of survival over the design period of the

pavement. The design also takes into account the number of passes of the design aircraft and so lower trafficked areas, such as taxiways and aprons, will result in thinner pavement structures.

In summary the main features of the aviation sector load characterisation and design are:

- Loads based on a single equivalent load classification number (ACN):
- Cumulative damage of mixed traffic related to a design aircraft;
- Design is based on empirical relationships between loads and failure;
- Most methods worldwide follow similar practice.

3.5.5 Maritime pavement engineering

Maritime sector pavements are also based upon similar elastic principles and analysis of stress and strain, which lie behind the design requirements for highway and airfield pavements. The design methodology used for most maritime situations worldwide is contained in the Structural Design of Heavy Duty Pavements for Ports and Other Industries (Interpave, 2008). The elastic properties of the pavement materials are used to describe the behaviour of each component. The analysis is based on consideration of fatigue performance, by establishing criteria to limit stresses to which the pavement can be exposed for one load pass and then, by reducing those stresses to account for the fatigue effect and cumulative damage of repeated load repetitions during the design life of the facility. The methodology is empirical in that the criteria were established by back analysing stress and strain in typical, 'real' pavements modelled using finite element modelling.

Vehicles using maritime and heavy duty industrial pavements tend to be limited to a small number of defined vehicles employed for specific purposes; for example, cranes and fork lift trucks. These vehicles also tend to operate on very constrained and repetitive paths, which are often planned and designed to achieve operational efficiency.

The wheel loading characteristics of these vehicles are well defined as they are related to the vehicles' load carrying capacity. The load carrying capacity and corresponding wheel loads are usually therefore available from the vehicle's manufacturer and often give data on laden and unladen weights and for various loading configurations.

Loading for pavement design is based upon establishment of an equivalent wheel loading that will use the facility. The wheel loading for design is the most onerous, based on the mix of traffic to be operational in each characteristic section. The equivalent wheel load calculation takes into account both static and dynamic loads.

The static load is the heaviest wheel load stated by the vehicle manufacturer. Consideration is then given to proximity wheel loads, if a vehicle has multiple wheels close together and factors are then added to account for the dynamic conditions related to that vehicles' path and mode of operation. The dynamic conditions are braking, slewing, turning etc. The vehicles in a port operation tend to operate to a very defined pattern, which is planned for optimum efficiency; therefore speed is usually fairly constant and not taken into account as a dynamic factor. For each condition that is present, a calculation increases the effective wheel load and this load is then used to calculate pavement thickness and specification.

All vehicles using each section of the facility are then equated to a number of passes of the equivalent effective wheel load and the pavement is designed accordingly.

In summary the main features of maritime sector load characterisation and design are as follows:

- Design based on material characteristics and elastic behaviour;
- Limiting factors established to reduce stress to account for repeated load conditions;
- Loads based on characteristics of actual vehicles including factoring for dynamic loads;
- An effective maximum load is calculated based on the most onerous vehicle;
- Empirical methodology based on analysis of 'real' pavements; and
- Most methods worldwide follow similar practice.

3.5.6 Pavement engineering – comparison with the rail sector

Similarities to the practice for design of the railway track can be found in all three pavement engineering sectors (highways, airfields and maritime). The major similarities identified are:

- Operations are based on a mixture of traffic, with the exception of mainline interoperable corridors for passenger traffic only.
- Design characteristics based upon different loading conditions and traffic.
- Basic principles of pavement design followed in track engineering for ballastless track design.

- A combination of static and dynamic loads is required for design.
- Design is dependent upon a set of specific loading conditions that are representative of the operational conditions.

There are however a number of important differences that need to be considered when comparing the design of pavements for other forms of transport to the design of track systems for railway traffic. These are:

- Pavement structures do not comprise mechanical elements within a system of interconnected individual components, but are rather a continuous layered structure comprising bound material layers with good load distribution properties which facilitates approximation using linear, half-space elastic theory or similar.
- The pavement structure is not required to steer the vehicles; whilst creep forces (forces arising due to deviation between the rolling direction of the wheels and the direction of axle displacement) develop in both cases at the wheel – rolling plane interface, the physics of motion are rather simpler in the case of a paved road compared to that of a railway.
- The wheels of paved road vehicles have the ability to turn in the direction being steered, whereas steering of railway vehicles is provided by the natural guiding mechanism of the railway axles, the wheel - rail contact characteristics (equivalent conicity) and the rolling plane dynamics (rail and track undulations) which provide the excitation mechanism.
- Paved road vehicle wheels can all rotate independently; whilst this is true for certain categories of railway vehicles and bogie technologies (employed in tramway systems in particular), this represents a small fraction of railway vehicles. However, the presence of independently rotatable wheels for a railway vehicle, does not cancel the fundamental differences in the steering mechanism of the wheelset due to the 'hard' rolling steel on steel nor the complexity of the vehicle's response to excitation.
- There are no actions from road vehicles, which tend to cause tension in the pavement structure in the direction normal to travel whereas the rectangular rigidly mounted wheels of a railway vehicle cannot rotate in plan and tend to prise the rails apart; this emphasises the systemic (component based) nature of the track structure which is required to resist such forces.
- Only vertical forces are considered in the design of the layered road pavement systems; lateral forces are also not considered for the design of aircraft pavements and the pavement acts as a large shear diaphragm with the dominant forces in the direction of travel and not normal to the direction of travel. However, for the design of the track system components, such as rails, fastenings and sleepers, it is necessary to consider

the lateral component of the loading as well as forces in the vertical and longitudinal directions.

- There are no discrete components on the surface of a pavement structure, such as the rails of a track system, which could be subjected to lateral forces.

The design parameters for pavements have been developed and refined over many years of applied research and experience of performance in practice based on data from monitoring, empirical characterisation and analytical or mechanistic modelling; this practice is consistent with that adopted for the development of railway track.

The physical modelling, loading mechanism and structural response analysis used for the design of ballastless track systems, was based on the design of continuous reinforced concrete road pavements, adapted to the rail sector through years of site trials and research (Eisenmann et al. 2000).

The mathematical modelling of track wear in relationship to axle weight (ORE 1979, ORE 1982) is similar to the 'power of four' rule used for pavement wear factors in pavement design.

3.5.7 Earthworks and bridge engineering – comparison with the rail sector

For the design of earthworks and bridges, in particular the latter, there is a much stronger coherence with the loadings considered for the design of the railway track than there is for the design of pavements. For vertical loads, the nominal / static values are considered together with their quasi-static (residual centrifugal force) and dynamic components (factored nominal loads) for the design of track systems, earthworks and bridges.

For lateral loads, there is a similarity of design approach for consideration of the lateral motion dynamics, through common application of the nosing force. The nosing force is representative of the guiding forces described in Section 2 which are generated by lateral displacement of a vehicle axle resulting in contact between the wheel flange and the rail. Similarly, in the longitudinal direction, the values of the design forces are specified to cater for the inertial forces generated during acceleration and braking of rolling stock and also for thermal loads in the structure and the track. In the case of ballastless tracks, specific consideration has to be given to continuous welded rail and high speeds.

What potentially differentiates the design of pavements, from the design of track systems and the design of structures, is the loading origins and the physical mechanisms for their application. These are different for pavement systems compared to track, whereas the loads and physical mechanisms are the same for bridges and the track, although spatially different.

Differences in the loading mechanism and the nature of the structural system are considered in the following:

- In the case of bridge structures, the traffic loading is generally considered on the basis of static equivalents, with the exception of short span bridges and high speed / high frequency loading cases, where dynamic analysis is sometimes required, as described in Section 3.5.3. The random character of the track loads at a particular point of the structure is smoothed out due to the damping properties of the track and the structure itself. Consequently, train signature reproducibility as measured on the track is rarely achieved.
- Bridge structures are designed using limit state design methods where design at the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) requiring the application of partial load factors to the characteristic value of the load are considered; on the basis of limit state principles, the extra capacity of materials beyond the yielding limit under exceptional loads can be effectively used for design at ULS. However, allowable (working) stress design methods, utilising unfactored loads, are still utilised for the design of the slab for ballastless track systems(Liu et al. 2011).
- The engineering properties of bridge structures vary little during their life time, assuming that the structures are properly maintained. This is not necessarily the case for a track structure, where components of the track system wear at different rates and have considerably different life expectancies. Wear at the contact / rolling plane, due to the natural guiding mechanism of vehicles, has a direct, knock-on effect on the longevity of track components and the track system as a whole.
- Due to the considerably longer design life of bridge structures compared to the railway track system components (including for ballastless tracks), the respective probability of failure needs to be considered. Dependent upon the limit state case considered, the future capacity required for bridge structures may not be directly compatible with that of the track system.

3.6 Discussion

Critically looking at the review of this section in terms of the vertical, lateral and longitudinal track loadings, project experience and cross-discipline experience, the following key points have been identified:

- The empirical approach historically implemented by track engineers has been to build on previous construction experience, assessment of operational integrity and retrospective analysis of failure. Other engineering sectors (pavement, earthworks, bridge engineering) have adopted a similar approach to the development of the understanding of their structural systems.
- The track structure is not currently designed as a whole system to take into account the loading, load paths and components.
- There is no common approach to analytical techniques concerning the track structure and its behaviour under dynamic loading;
- The examples of work undertaken by various railway administrations have proposed a range of dynamic factors based on the methodology assumed by their research.
- The inherent elastic nature of the track system itself acts as a filter to the loads carried by each layer within the system, meaning that simply applying a dynamic amplification or impact factor at the wheel-rail contact point is likely to lead to the over-specification of the support system.
- Track system behaviour due to external factors other than vehicle loading should also be considered, such as wind and thermal loading.
- The literature review highlights that much of the work is biased towards the vertical loads, reflected by the statistical confidence level in the dynamic factor which is used to determine the total vertical load. No evidence has been found that this is embedded in national design standards or project practice. The same level of confidence is not however evident in the case of the lateral loads which potentially affects any load combinations required for design.

The lateral design load definition presents the highest difficulty, arising not only from the highly non-linear and random relationship of the track/vehicle interaction in the lateral direction, but further due to the difficulty in establishing the contributors to lateral forces in service. This is particularly important in understanding the lateral resistance that must be provided by a ballast structure, or in the case of ballastless track, what loads need to be resisted by the baseplate assembly. The lateral load from the rail depending on the track system may with resilience and the beam effect of the rail be shared between rail holding points thereby reducing the lateral loading from the assumed force at the WRI.

The desktop analysis of track related research and relevant cross-discipline expertise has indicated the significance of the parameters highlighted for track loading in Table 5. As a result, we can reasonably conclude that a loading regime must take these parameters into consideration as a minimum.

Table 7 - Summary of load contributors within the track system (Source: Mott MacDonald)

Force Direction	System Location	Load Source	Contribution to Load
Vertical	Internal	The mass and inertial properties of the rolling stock (car body, bogie, axle mass and loading conditions; centre of gravity)	Contributor to all expressions of the vertical load (static, quasi-static, dynamic)
		The stiffness properties of the rolling stock (primary and secondary suspension, stiffness and damping)	Required for the definition of the vertical dynamic load
		The geometric properties of the rolling stock (carriage dimensions, bogie spacing, wheel spacing)	Required for the definition of the vertical dynamic load. Also defines the point of application of any cross wind forces and thus rule the respective quasi-static load
		The wheel and rail profiles and their equivalent conicity	Influences the definition of the contact patch size and the Hertzian spring
		The characteristics of the track's horizontal alignment (curvature, cant, cant deficiency, rail gauge)	Influences the definition of the vertical quasi-static load (due to the residual centrifugal force).
		The quality of the railway track (track geometry faults)	Influences the vertical dynamic load.)

Table 7 - Summary of load contributors within the track system (Source: Mott MacDonald)

Force Direction	System Location	Load Source	Contribution to Load
Vertical, *cont.*	Internal *cont.*	The quality of the rails' rolling plane (crossing nose in S&C, dipped welds, rail undulations, roughness and joints, if any)	Influences the vertical dynamic load.
		Wheel geometry (ovality, out-of-roundness, flats)	Influences the definition of the vertical dynamic load (due to the stochastic application to the track structure).
		The stiffness of the railway track as a whole (pads and ballast)	Influences the vertical dynamic load.
		Vehicle speed	Influences the vertical dynamic load.
	External	The prevailing wind speeds (cross winds force)	Influences the definition of the vertical quasi-static load due to cross winds.
		Other forces, where applicable, transmitted through the track's substrate (for example, earthquake forces).	

Table 7 - Summary of load contributors within the track system (Source: Mott MacDonald)

Force Direction	System Location	Load Source	Contribution to Load
Lateral	Internal	The wheel - rail contact interface (equivalent conicity, contact surface friction coefficient)	Contributor to the development of longitudinal and lateral creep forces
		The centring of the wheelset on the track (axle lateral displacement)	
		The mass and inertial properties of the rolling stock (car body, bogie, axle mass and loading conditions, centre of gravity)	Influences the residual centrifugal force applied to the track structure through curves
		The stiffness properties of the rolling stock (primary and secondary suspension, stiffness and damping)	Influences the resistive force applied by track structure on wheel to guide wheel through track geometry features
		The characteristics of the track's horizontal alignment (curvature, cant, cant deficiency, rail gauge)	Influences the overall lateral force as a result of residual centrifugal force
		The quality of the railway track (track geometry faults)	Influences the overall lateral force due to flange contact with the rail
		Vehicle speed.	
	External	The prevailing wind speeds (cross winds force);	Influences the quasi-static vertical load due to cross winds.
		Other forces, where applicable, transmitted through the track's substrate (for example, earthquake forces).	

Table 7 - Summary of load contributors within the track system (Source: Mott MacDonald)

Force Direction	System Location	Load Source	Contribution to Load
Longitudinal Note: longitudinal creep forces being considered in the lateral loads above)	Internal	The mass and inertial properties of the rolling stock (car body, bogie, axle mass and loading conditions).	Influences the amount of force able to be applied to the track structure through friction contact with the rail
		Power unit power output.	Influences the tractive effort applied to track structure during acceleration
		Power unit braking performance.	Influences to the braking forces applied to track structure during braking
		Vehicle speed.	
	External	Ambient temperatures.	Influences the compressive or tensile forces in the rail

4 Review of design requirements of track

4.1 General

This section presents a review of the loading regime requirements for the design of the track system, considering also application to various track components and layers. This draws on the results of the desktop analysis of track related research, project experience and relevant cross-discipline input presented above. This review forms the basis for recommendations for further research set out in Section 6.

4.2 Track system

Design of the track system requires:

- The evaluation of vertical and lateral track deflections under load and verification that they remain within acceptable limits.
- Checks and assurance that the structural capacity of any layer or component within the track system is not exceeded.

The behaviour of the track system under loading is dependent on a number of interconnected material characteristics and therefore designers should consider these behaviours.

From the findings presented in Section 3, it has been established that the loading of a track structure should consider these inputs:

- The vertical loads should be considered on the basis of an actual train load model, or where a variety of rolling-stock is operated on a particular line, a representative design load model such as LM71, suitable for the track category.
- The vertical loads must then be factored to take into consideration the effects of speed, wheel condition and track condition, plus specific localised track features which can cause dynamic amplification of the load. prEN16432-1 proposes a dynamic amplification factor of 1.5. This is likely to be inadequate given the complex relationship between the wheel

condition, allowable track condition and the forces carried by the track structure which can result in a range in dynamic loads experienced over the life of the track structure (see Section3.2).

- The design lateral load value should be based upon consideration of the track/rolling stock interaction and specification of the design parameters through modelling (for example, using multi-body dynamic simulation). As accepted figures are not available, further research to establish applicable ranges for defined rolling stock design and condition categories would be worthwhile. Consideration of the statistical variation due to condition of train and track would provide confidence in a suitable load range that should be applied.
- For both vertical and lateral loading, consideration should be given to whether the load is to be applied from a single point (wheel) to the rail, or via two wheels within a bogie, or indeed multiple bogies. Multi body simulation would help decide a realistic balance of applied loading for a bogie, but account would need to be taken of the component structure to decide how the worst case loading and suitable fatigue cases should be applied.
- Lateral loads should be applied in conjunction with appropriate vertical loads and the resilience of the track system.
- Suitable load combinations are not defined in the literature or available from the examples found in this review and multi-body simulation and careful consideration of potential worst case combined load cases are worth exploring further.
- The influence of environmental loads (such as temperature gradient) and externally induced forces (such as earthquake) should be taken into consideration in addition to the rolling stock loads specified.
- The influence of relative change of vertical and lateral track stiffness along the track system (for example, between different track-forms or any other transition) can be important and should be included in the design remit.

4.3 Rails

Rail design is rarely conducted as part of the design process in current practice. Rail section selection is primarily based on axle load and tonnage borne and is usually specified by the Customer. Rail section selection can be influenced by procurement related factors, such as the availability of a particular section from manufacturers and its anticipated durability. Requirements for specification and performance are set out in BS EN 13674.

British Railway Track (PWI, 6th Edition) describes the ultimate limit state (ULS) and service limit state (SLS) concepts and a fatigue strength methodology to enable validation of rail stresses where required. A key input to this methodology is the amplification factor, inclusive of the quasi-static and dynamic effects, which is applied to the static vertical wheel loads and the lateral loads that apply to the rail.

4.4 Fastenings

Fastenings must be capable of resisting the longitudinal loading effects of creep, acceleration and braking forces and thermal effects. They must also maintain gauge by resisting lateral forces and uplift due to lateral loading effects as well as transfer lateral and vertical loads through to the structure beneath.

Fastenings systems are also required to provide a level of compliance, eg through provision of elastic rail pads, to allow a degree of load distribution and impact attenuation.

Performance requirements for rail fastenings are defined in EN 13481 and are based on assumed loading criteria for categorised wheel loads. This standard series defines the requirements for longitudinal resistance, clamping force and uplift force.

EN 13481 also specifies a load application angle for testing of components which aims to represent the range of lateral to vertical load (L/V or Y/Q) ratios anticipated in service for the worst combination of geometrical conditions and operating speed. The highest L/V ratio identified within EN13481 is 1.0.

This is a fatigue-related performance requirement, given the nature of the test. However, the standard does not give consideration to the effects of exceedances of the L/V ratio and as such cannot be used for identification of design loads.

The design of fastening assemblies should consider:

- Vertical and lateral loads to be carried by the fastening assembly, distributed according to properties of rail and support stiffness.
- Resistance to gauge spread and rail rollover.
- Resistance to longitudinal forces as a result of thermal effects and longitudinal creep.
- Vertical deflection of resilient components (rail pads and baseplate pads for example).
- Attenuation of impact forces, noise and vibration.
- Resistance to rail uplift forces as a result of negative bending in front of and behind an axle.

4.5 Sleepers

Sleepers (including blocks and bearers) must be capable of resisting the shear forces and bending moments generated by the loading applied to the rail-seat. The load borne by the sleeper is influenced by the distribution of load due to the combined elastic behaviour of the rail and fastenings.

Sleepers, blocks and bearers must also be capable of transferring the rail-seat load to the ballast structure or concrete slab structure in such a way that the bearing capacity of the support layer is not exceeded.

Performance requirements and requirements for the design of sleepers, blocks and bearers are covered in BS EN 13230.

Note: For supports with S&C (ie bearers), the vertical load applied needs to adequately factor the influence of the crossing nose on the dynamic wheel load due to the change in system stiffness at this particular location and the apparent dip due to the flange-way gap, which gives rise to impact forces.

4.6 Ballast

Design of the ballast layer is not normally undertaken, it being a standard specification according to track category, as opposed to loading, under Network Rail standards. However, the depth of the ballast and other layers and specification of other components above the formation should be adequate for the distribution of load to ensure that the strain within the formation remains elastic under foreseeable conditions and the design life of the track. The load distributing capability of ballast layers should not be over-estimated, due to the risk of plastic deformation in the formation that would reduce the ability of the formation or sub-ballast to drain effectively. Additional components such as sleeper pads can improve the load distributing capability of a ballasted track system.

The ballast structure must also provide adequate resistance to lateral shift of the track as a result of guiding forces generated by the movement of rolling stock through a curve and on straight track (nominally based upon the vehicle that generates the highest lateral force) and also be resistant to the buckling effects as a result of thermal expansion. Similarly to the vertical dimensioning of ballast, lateral dimensioning is usually given through standard specifications for width and shape of the ballast shoulder outside of the sleeper ends, established through experience and maintenance practice.

Additionally, the ballast structure must provide resistance to longitudinal forces generated by acceleration and braking of the vehicle.

4.7 Ballastless-track structure

Development of a suitable loading regime for design of a ballastless track structure should include consideration of the nature of the proposed structure as this may dictate how a worst case combination of load needs to be applied. Structural modelling should be appropriate to the trackform being studied (continuous pavement, discrete or joined slabs) and the interface with the underlying structure (earthworks, tunnel or bridge support).

The specification of a design loading regime for ballastless track is addressed in prEN 16432 Part 1, however, it is noted that the standard may not adequately consider the influence of dynamic effects. For example, it is recommended that a dynamic factor of only 1.5 is applied to the nominal vertical load, which is derived from the requirements for the acceptance of rolling stock. As discussed above, a higher dynamic load case might be considered advisable for engineering design - further work is recommended.

4.8 Switches and crossings

Switch and crossing (S&C) layouts present specific features to the wheel-rail interface that must be considered by track designers. Key locations within S&C layouts that result in an increase in loading are the crossing nose (knuckle) and switch rails (particularly the wheel flange contact areas close to the switch toe).

The crossing nose at fixed crossings results in an increase in vertical load as a result of the wheel passing over the flange-way gap (also the knuckle at fixed diamond crossings). This results in a change of trajectory and an impact on the leading edge of the vee on the "running on" side of the crossing nose. Simulations (Markine, 2009) have identified that dynamic forces generated within S&C units are strongly linked to the geometry of wheel and rail profiles and the material characteristics of bearers, crossing nose, pads, ballast etc.

At the switch toes, there will be an almost instantaneous increase in lateral wheel load (known as jerk) as the vehicle adjusts for the change in direction. The interface between the switch blades and the stock rail within a S&C unit presents a change in gauge as experienced by the axle, which results in lateral shift in the axle as centring forces are initiated. A similar condition occurs, but greatly reduced, at switch diamonds.

Where swing nose crossings are used, vertical impacts at the crossing nose will be reduced as the irregular vertical trajectory of the wheel through the crossing is significantly reduced, compared with a fixed crossing. Passing through a swing nose crossing will still generate additional lateral forces due to axle centring as the axle adjusts for the wide gauge (rail profile change) present at the point where the point rail (swing nose) is mated to the splice rail, just as there is for the switch rail adjacent the stock rail.

Due to the fixed position of S&C units, the influence of these features is highly repetitive on each layer of the track structure.

Design of each layer within a S&C unit should be designed for the considerations described in Sections 4.2 to 4.7, with additional consideration to the impact of:

- Wheel trajectory changes as a result of switch toes and crossing noses
- Changes in stiffness characteristics due to:
  - Longer and deeper bearers
  - Crossing nose element

- Additional track mass
- Additional rails pinned to bearers

4.9 Check rails

Check rails aim to reduce the risk of derailment in tight radius curves and S&C units and/or reduce the rate of rail side wear, by reducing the lateral force on the outer wheel through contact with the back of flange of the inner wheel. This contact has the effect of sharing lateral force across both the check rail and the outer rail.

A check rail will only be of benefit where the horizontal radius is sufficiently tight to result in contact between the wheel flange and the check rail.

In curves where check rails are used, the design process should recognise that the lateral forces may be shared between the outer rail and the check rail, although the worst case for component design should be for the concentration of all the load on each side in turn.

5 Assessment of economic benefits to track systems

Broadly speaking, the development of a set of clear and concise loading requirements for track systems has the potential for improving the chances of success for innovative track systems.

It is challenging to place a cost on the current varied approach to the design of track systems, given that it is generally not known whether a track structure has been over, under, or efficiently specified. This is because it requires a failure to establish that under-specification has occurred and then, typically, the costs of rectifying such issues are either not publicised, are hidden within maintenance costs, or may even be subject to commercial action or legal proceedings.

For the purchaser or infrastructure owner or maintainer, there is a clear benefit in establishing a set of clear and concise loading requirements for the design. There is potential for cost efficiencies to be achieved along the supply chain, from component supply through to design, construction and maintenance.

5.1 Specifier

Typically found within the Infrastructure Owner's organisation, the specifier in the current environment will find that they must review a number of different sources and references; from standards, to academic texts, to examples of equivalent projects and cross-discipline research. This tends to be a hidden cost within an infrastructure owner's organisation, unless it is delivered by a consultant or contractor. A set of clear and concise loading requirements and guidance for their application should result in a single reference, or suite of references, which would expedite this process.

5.2 Design

From the review of example projects in Section 3.4, it is evident that there are a variety of practices used by track design engineers especially when considering ballastless or innovative track systems. This variation will inevitably result in the effort being expended in establishing the design requirements or challenging the specifier on the interpretation, as the Designer is seeking to ensure their commercial risks are mitigated.

Clear and concise requirements would result in an overall reduction in design effort as industry improves its knowledge base, resulting in better specifications and improved scope for checking and auditing.

Also for ballasted track, a clear guideline on loading would help establish a clearer expectation for design life and the risks or improvements achievable through improved track (and train) condition. Improvements to maintenance planning and prioritisation and whole life costing should result.

5.3 Component manufacturing

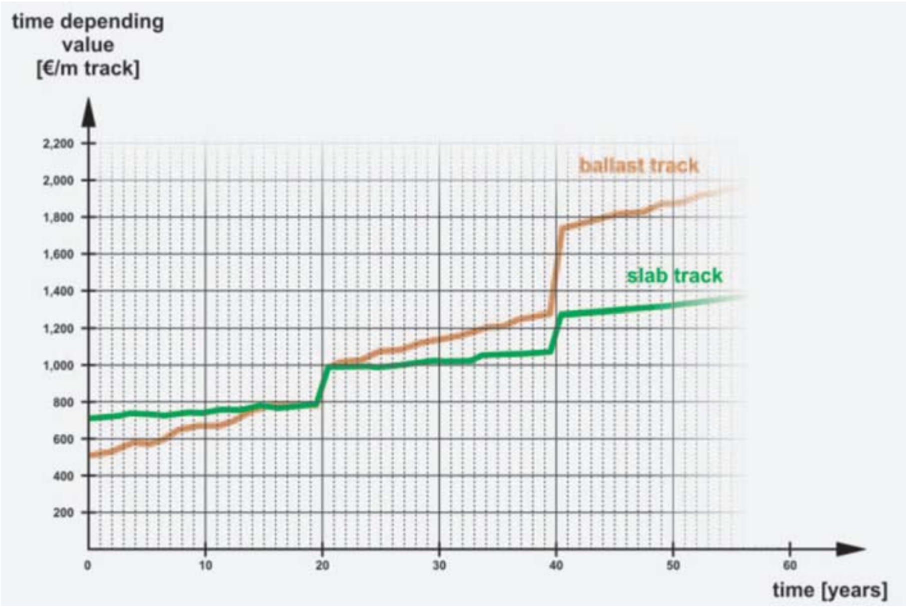
The track component manufacturing sector of the rail industry is already governed by established performance requirements contained within the relevant Standards. However, the industry could still benefit through the development of a clear and concise loading regime in the following ways:

- Potential for reduced reliance on 'design assisted by testing' practices and validation of proprietary software tools;
- Improved use of desktop based analysis to understand ultimate and fatigue characteristics of components in different conditions and as part of the whole system;
- Potential for improved efficiency in the use of construction materials, for example, better understanding of the loads being applied to slab-track may result in the reduction in size of structural elements;
- Greater reliability in service, resulting in purchasers and specifiers having greater confidence that the product will achieve its minimum design life.

5.4 Construction and maintenance

Various studies completed over the past 10-15 years have suggested that ballastless track systems will generally be more cost effective than their ballasted counterparts from the whole-of-life-cycle cost (WLCC) perspective, despite being 20%-40% more expensive to construct. This is perceived to be due to the anticipated lower maintenance cost and longevity of the structural components - 50-100 years for ballastless track systems compared to 30-40 years (at best) for ballasted track systems (Michas, 2012).

Figure 4 - Indicative model for the cumulative cost of slab-track and ballasted track (Source: Schilder, R et al., 2007)



Schilder, R et al. 2007, estimate the break-even point for slab-track systems compared to ballasted track systems is approximately 20 years, based on assumptions for construction and the maintenance regime, as well as applied tonnage of traffic and various financial parameters (such as the internal rates of return). It is noted that these assumptions were based on a particular network known for its stringent approach to wheel and track maintenance and that ballast would last 40 years before renewal and rails 20 years.

For this particular scenario, it is suggested that slab-track systems should be considered by Infrastructure Owners as a viable cost effective solution in the long term. This approximation is supported by long-term studies by the Austrian Federal Railways (ÖBB) which found that the average break-even point between the two systems was approximately 24 years, based on a daily track loading of around 70,000 tonnes (approximately 25 Million gross tonnes per annum) and an estimated life expectancy for the concrete elements of around 80 years (Pichler, D et al, 2013). It is important to note that it is critical to have the earthworks/formation support prepared to a higher specification to ensure that the WLCC assumptions and therefore the economic justification, is correct (for the ballasted track as well as for ballastless).

It is also important to consider the influence of track form on whole project costs, such as resulting from the dimensioning of tunnels and bridges. For such locations, a ballastless structure for the track may offer weight and volume savings through shallower construction depth, or may be the only viable option due to access constraints and availability targets.

A well-designed slab track maintained in accordance with its maintenance schedule should require minimal maintenance over its life cycle, with fewer interventions and less risk of faults leading to speed restrictions.

However, costs associated with rectifying under-specified slab-track structures can be exacerbated as they are typically used in challenging maintenance or access environments and are more complex to successfully solve. Consequently, the cost of getting a design wrong can be disproportionately high.

Additional costs, which can add to the overall cost of failure of a degrading track structure which is struggling to meet the design life target, may include:

- Penalty costs associated with temporary speed restrictions due to poor track geometry.
- Access costs for possessions to undertake preventative/restorative maintenance.

This study has found there is currently a paucity of information available on the costs associated with rectifying under-specified ballastless systems that have subsequently failed.

6 Conclusions and recommendations

6.1 Conclusions

The research undertaken within Phase 1 of the research project 'T1073-01 Loading Requirements for Track Systems' has:

- Examined the way in which loads are applied to track systems by railways.
- Reviewed the loads that have historically been cited for the design of track structures.
- Summarised design practice for track projects in GB and internationally.
- Examined the approaches adopted for loading of earthwork and bridge structures and design practice applied for other transport systems.
- Made recommendations for potential future track loading regimes.
- Considered the economics of different track systems.

Loading of the railway track has been reviewed on the basis of a systematic literature review across GB and international sources enabling the vertical, lateral and longitudinal loads acting on the railway track in service, to be documented. The fundamental calculations covering the source of various load components are included in Appendix C.

The report documents the equations for calculation of the load applied to the track system. These are based on a set of parameters, such as rolling stock characteristics, track geometry and ambient environmental conditions.

The key aspect established and highlighted in this report is the complex relationship between track loads, the properties of the railway vehicle properties, and the dynamic response of the vehicle with the track system. Another key aspect highlighted is the number of approaches available to estimate parameters such as the dynamic amplification factor (DAF) and the variation in outcome across the different methods.

The outcome of cross-discipline research, to investigate the design loading requirements for pavement structures (as used in highways), earthworks and bridge engineering, has identified potential technical similarities with the loading of railway track. It has also provided a valuable insight into the potential for extension of the requirements for application to rail engineering. This demonstrates that any lessons learnt from other sectors seem to have been integrated, already being part of the current state-of-the-art for railway track engineering practice.

Key findings include:

- From Pavement Engineering:
  - The design parameters in the pavement design sectors described previously have been developed and refined over many years of applied research and return of experience, by monitoring, empirical characterisation and analytical / mechanistic modelling; this aspect is, to a great extent, consistent with railway design practice for ballasted track.
  - The design of ballastless track systems, for example the physical modelling, loading mechanism and structural response analysis, was based on the design of reinforced continuous concrete road pavements, adapted to the rail sector through years of site trials and research (Eisenmann et al. 2000).
- From Earthworks and Bridge Engineering:
  - In the case of civil structures, traffic loadings are generally considered on the basis of static equivalents, with the exception of short span bridges and high speed/high frequency loading cases, where dynamic analysis is required, as described in Section 3.5.3. The random character of track loads at a particular point of the structure is smoothed out due to the damping properties of the track and the structure itself. Consequently, train signature reproducibility as measured on the track is rarely achieved.
  - Civil structures are designed using limit state design methods and are dependent on the structural system being considered and the potential formation of plastic hinges at ULS; this allows for the extra capacity of materials beyond the yielding limit to be effectively used. Allowable (working) stress design methods are still applicable for parts of the track system (Liu et al. 2011), such as is the case for the slab in ballastless tracks.
  - Civil structure properties vary little during their life time, assuming that the structures are properly maintained. This is not necessarily the case for a track structure, where the components of the track system wear at

different rates and have considerably different life expectancies. Wear at the contact or rolling plane, due to the natural guiding mechanism of vehicles, has a direct, knock-on effect on the longevity of track components and the track system as a whole.

- Due to the considerably longer design life of civil structures compared to the railway track system components (including ballastless tracks), the respective probability of failure needs to be considered. Dependent upon the limit state case being examined, the future capacity required for civil structures may not be directly compatible with that of the track system.

Evolution in the rail sector during the last forty years, particularly due to the increase in speed of travel, has led to considerable progress, not in connection with the principles of track loading, but in terms of specific track system response, taking account of track stiffness, track - vehicle interaction and modelling of the track response to load.

Design of the rails and ballast within a track structure is rarely a design activity that is undertaken currently, as the design of these layers in the track system is generally based on well-established specifications. Fastenings and sleepers require an understanding of the load transfer between components to ensure an adequate design, but also have well established performance requirements which have to be achieved prior to approval for use in service.

The outcome of the Phase 1 research shows that:

- There is uncertainty in the GB rail industry about the track loading requirements for the design of track systems, due to the difficulties associated with obtaining data about the impact of loading on track system life, whole life cost, system safety and reliability. This has been evident in the case studies, which in practice are often bound by confidentiality restrictions. This uncertainty about 'cause and effect' of loads applied to track systems, may lead to over-provision or under-provision of track assets.
- There are multiple interconnected dependencies for load transfer within the track system and simply specifying a standard dynamic load factor, as is the case in prEN 16432-1, has the potential to result in the construction of track systems that are either under-specified or over-specified.
- The research has identified a need for development of a methodology for the design of track systems, including the provision of appropriate load models and guidance on their scope of application.

- Review of project experience within the UK and internationally has found that administrations generally rely on bridge loading codes to establish the required design loading for ballastless (concrete slab) track systems.
- There are potential benefits, although currently unquantifiable, for the industry through the development of a common approach to the design of track systems. This is likely to result in improved efficiencies in design, construction and maintenance as well as the development of industry confidence in the use of ballastless track systems.
- To facilitate the development of a design loading specification, it is necessary to accurately capture and record the requirements for track loading, which are appropriate for the track parameters, the railway traffic and the operational requirements, of a particular network.

Based on the findings of the research, we have concluded:

- The vertical loads should be considered on the basis of an actual train load model, or where a variety of rolling-stock is operated on a particular line, a representative design load model such as LM71, which is suitable for the track category.
- The vertical loads must then be factored to take into consideration the effects of speed, wheel condition and track condition, plus specific localised track features which can cause dynamic amplification of the load. A methodology which enables the influencers of the dynamic amplification to be incorporated into the calculation is required to ensure that the track system is efficiently designed.
- The lateral load value for design should be based upon consideration of the vehicle/track interaction and specification of the design parameters through modelling (for example, using multi-body dynamic simulation). Further research is recommended to establish applicable ranges for defined rolling stock design and condition categories as accepted figures are not readily available. Consideration of the statistical variation due to condition of train and track would provide confidence in a suitable load range that should be applied.
- For both vertical and lateral loading, consideration should be given to whether the load is to be applied from a single point (wheel) to the rail, or via two wheels within a bogie, or indeed multiple bogies. Multi-body simulation would help decide a realistic balance of applied loading for a bogie, but account would need to be taken of the component structure to

decide how the worst case loading and suitable fatigue cases should be applied.

- Lateral loads should be applied in conjunction with appropriate vertical loads to determine the resilience of the track system.
- Suitable load combinations are not defined in the literature or available from the examples found in this review and multi-body simulation and careful consideration of potential worst case combined load cases should be explored further.
- The influence of environmental loads (for example, due to changes in temperature) and externally induced forces (for example due to earthquake) should be taken into consideration in addition to the rolling stock loads specified.
- The influence of relative change of vertical and lateral track stiffness along the track system (such as between different track-forms or any other transition) can be important and should be included in the design remit.

6.2 Recommendations

The Phase 1 research has confirmed the need for further work to deliver a harmonised track loading specification for the GB network. It is anticipated that the work involved would need:

- Determination of and definition of the total and design vertical load that should be used for the design of track systems. The two commonly applied approaches are the German methodology as developed by Eisenmann, and the French method cited in the French literature. These should be explored further for applicability to the GB rail industry. Both methods are included as recommended methods for determination of the vertical dynamic wheel load, in the current version of the prEN 16432-1:2015. Both methods allow for the operational conditions of the line (speed) and the track condition (track geometry faults) to be taken into account, as they have been demonstrated to be significant and are fundamentally linked to the magnitude of the track vertical dynamic load.

For the purpose of determining an appropriate design track loading specification for the track systems of a particular network, a staged approach comprising the following steps are recommended for Phase 2:

- Categorising rolling stock in service for a particular network in order to capture and record the following:

    - Weight characteristics (sprung, semi-sprung, unsprung masses as specified by the rolling stock supplier)
    - Dimensional characteristics (as specified by the rolling stock supplier including the wheel dimensions and profile, wheel spacing and vehicle length)
    - Suspension stiffness characteristics (primary and secondary suspension stiffness and damping values as specified by the rolling stock supplier)
    - This step is considered necessary to enable the creation of a rolling stock database for a particular network that will serve to benchmark loads applied to the track system and to support desktop simulations
- Collating bands of key track parameters for a particular network, such as rail profile, minimum curve radius, maximum cant and cant deficiency, track stiffness range (good/average/bad) depending on the track configuration and track maintenance levels (safety, target, acceptance, actual).
- Research into the basic environmental parameters for a particular network, such as historic records (from past projects and meteorological data) of prevailing winds and their speeds, where available and applicable, ambient (design) temperature values.
- Determination of representative design loading values per network, route, and section of track.
- Agreement on the preferred definition of the total or design lateral load. A similar process to that proposed for the vertical loading can provide equivalent results for the track lateral loading.
- Complete a multi-body dynamic simulation exercise using the representative loading and environmental values to establish the influence on the static wheel load to define suitable impact factors in both the vertical and lateral loading directions.

The determination of an appropriate track loading specification applicable to specific track configurations and track equipment, such as switch and expansion joints, isolated rail joints, junction work and transitions, requires a similar approach to the one described for plain track, involving as a first step the identification of the track equipment to be considered. Due to the variety of equipment in service on the GB network, it may be necessary to limit the range. The information already identified, together with key parameters of the track equipment which is needed by manufacturers, would provide a basis for modelling the loading mechanism, the response of the track system, and permit development of appropriate track loading specifications for the various track system components.

Field measurement trials would be beneficial to correlate the models with the actual loads observed in service and to provide validation that the expected loads will not lead to exceedance of the accepted performance limits. A certain amount of track response measurement is also recommended in order to capture the track dynamic performance characteristics as seen in service.

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Loading Requirements for Track Systems
Phase 1 summary report

Appendix A: Design loading for track systems

A.1 Reasons for this note

In the course of development of a specification for research project T1073 'Loading requirements for track systems', it has become apparent that there are open questions regarding the applicability of the bridge loading requirements in BS EN 1991-2 'Traffic Loads on Bridges' (BSI 2003) for the design of track systems. The Load Models specified for bridge structures in BS EN 1991-2 include requirements for the application of load in the vertical and horizontal directions. Some of these load models have been suggested for design of ballastless track systems in draft standard pr EN 16432-1 'Railway applications - Ballastless track systems - Part 1: General requirements' (BSI 2014).

This note provides a brief commentary on, the loading requirements for design of bridges in BS EN 1991-2, the permissible loads applied to the track by vehicles in GM/TT0088 'Permissible Track Forces for Railway vehicles'(RSSB 1993), and the loads required for the design of track systems in accordance with GC/RT5021'Track System Requirements' (RSSB 2009).

The potential relevance of bridge design loading for the design of track systems is considered.

A.2 Bridge loading to BS EN 1991-2 Traffic Loads on Bridges

A technical report has been produced for RSSB titled 'Background to Structural Design Eurocodes Relating to Rail Traffic Loading on Bridges' (RSSB 2009). This report summarises the background and the sources of information used in the development of the current standard for rail traffic loading, BS EN 1991-2:2003 'Traffic Loads on Bridges'.

The key load models used to account for vertical and horizontal forces from rail traffic are:

1 Vertical - Load Model 71 (LM71), Load Model SW/0 (LM SW/0), and the High Speed Load Model (HSLM)

2 Horizontal - Braking and acceleration forces acting in a longitudinal sense in the direction of travel along the track, and centrifugal and nosing forces acting in a lateral sense in a direction normal to the direction of travel along the track.

Additionally, vertical loading is specified for accidental loading situations where a train derailment occurs on a railway bridge, and loading is applied to parts of the bridge away from the track.

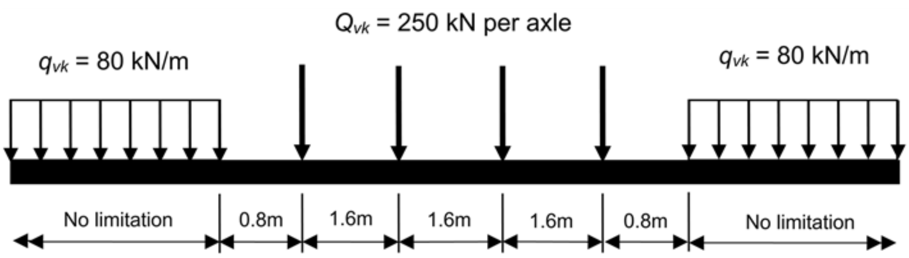
Vertical load models

The vertical load models used for the design of railway bridges will be considered in this section.

1 LM71 and LM SW/0

For bridge loading in accordance with BS EN 1991-2, the primary load model used to represent normal traffic for design of railway bridges is LM71 (see Figure 1).

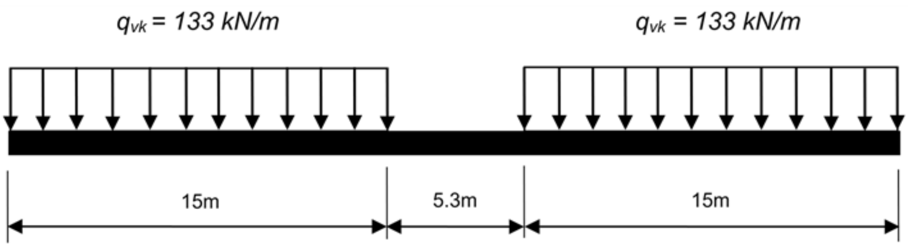
Figure 1 - LM71



LM71 was developed primarily for use on simply supported bridges, but it was found to not adequately cover the load effects from normal traffic on continuous bridges (i.e. multiple span bridges for which the adjacent spans are connected structurally at the intermediate supports).

LM model SW/0 was developed to address this situation (see Figure 2).

Figure 2 - Load model SW/0



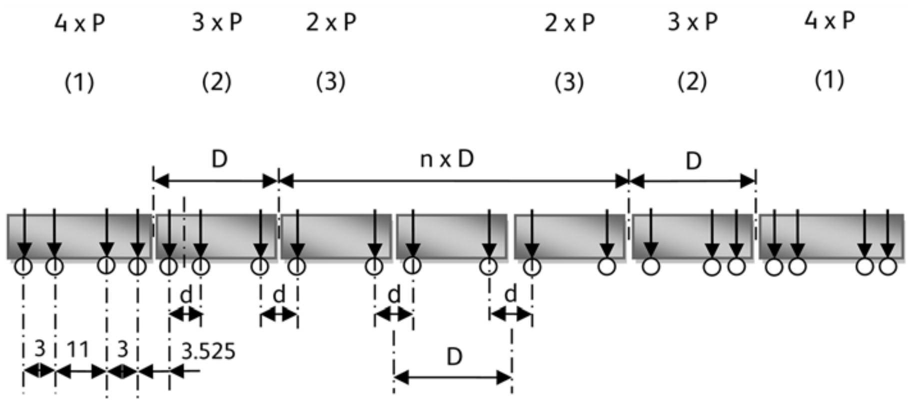
LM71 and LM SW/0 are assumed to replicate the effects of mixed traffic comprising passenger vehicles capable of travelling at speeds up to 200 km/h and freight vehicles capable of travelling at speeds up to 100 km/h.

Additional loads for the design of bridge structures supporting directly fastened and embedded rails are included within 10.2.6 of Network Rail Standard NR/L2/CIV/020 'Design of Bridges & Culverts' (Network Rail 2010).

2 High Speed Load Model

LM71 had generally proved to be 'fit for purpose' up to the early 1990s *'when unacceptable levels of bridge acceleration were experienced at some (generally short span) bridges on continental high speed lines, when train speeds above 200 km/h were introduced'*. Consequently, the European Rail Research Institute (ERRI), undertook research which led to the development of acceptance criteria for ballast destabilisation in the form of a limit on ballast acceleration and the development of a High Speed Load Model (HSLM) to replicate the effects of '*universal trains*' which represented the high speed rolling stock in service at the time. The work is summarised in the final report D214/RP 9 'Rail Bridges for Speeds > 200 km/h' (ERRI 1999).

Figure 3 - High Speed Load Model (HSLM) A



Notes

1. Power car (leading and trailing power cars identical).
2. End coach (leading and trailing coaches identical).
3. Intermediate coach.
4. See Table 1 for vehicle dimensions and axle loads.

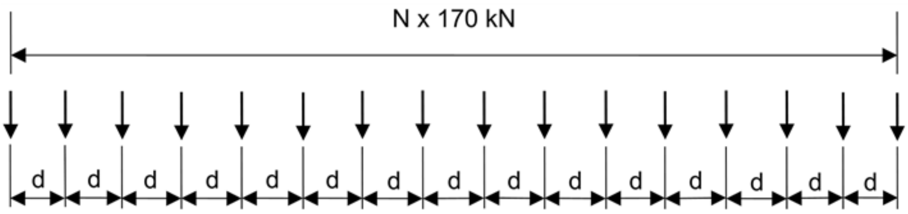
The load model HSLM comprises two separate Universal Trains with variable coach lengths, HSLM-A and HSLM-B (see Figures 3 and 4 and Table 1).

The acceptance criteria, and HSLMs A and B, are included in BS EN 1990 Annex A2 'Basis of design' (BSI 2008) and BS EN 1991-2 respectively. Acceptance criteria for the stability of 'direct fastened tracks with track and structural elements designed for high speed traffic', is also included within BS EN 1990 Annex A2. Extensive dynamic analyses and site measurements were behind the development of HSLM and the provision of the acceptance criteria for bridges.

Table 1 - HSLM A Vehicle Dimensions

Universal Train	Number of intermediate coaches (n)	Coach length (D - m)	Approximate train length (m)	Bogie axle spacing (d - m)	Axle load (P - kN)
A1	18	18	365m	2.0	170
A2	17	19	364m	3.5	200
A3	16	20	361m	2.0	180
A4	15	21	356m	3.0	190
A5	14	22	349m	2.0	170
A6	13	23	340m	2.0	180
A7	13	24	353m	2.0	190
A8	12	25	341m	2.5	190
A9	11	26	327m	2.0	210
A10	11	27	338m	2.0	210

Figure 4 - High Speed Load Model (HSLM) B



Notes

1. The axle spacing 'd' is dependent upon the bridge span length (L) and the number of axles (see Figure 6.14, BS EN 1991-2) – 2.5m < d < 6.0m, 2.5 < N < 20.
2. Application of HSLM B is limited to bridges with span L < 7m.

Horizontal load models

Longitudinal (in the direction of travel)

For bridge design, longitudinal loads are considered to take account of traction and braking forces from railway vehicles, applied to the track. These forces are applied at the top of the rails and are '*considered as uniformly distributed over the corresponding influence length $L_{a,b}$ for traction and braking effects for the structural element considered*'.

Characteristic values are provided for the traction and braking forces to be considered with the coexistent vertical loads attributable to load models 71, SW/0, SW/2, and HSLM in clause 6.5.3(2) of BS EN 1991-2. These forces are summarised in Table 2.

'*The characteristic values of traction and braking forces*' are not required to be multiplied by the dynamic factor (6.4.5.2) or by the reduction factor f (6.5.1(6)). '*Load models SW/0 and SW/2 traction and braking forces need only be applied to those parts of the structure*' which are loaded with the vertical loading indicated in Figure 2 (6.5.3(2) NOTE 1). The '*characteristic values are applicable to all types of track construction e.g. continuous welded rails, with or without expansion devices (6.5.3(4))*'.

Table 2 - Traction and braking loads from BS EN 1991-2

Load type	Force	Load model
Traction	Qlak = 33 (kN/m) x La,b (m) ≤ 1000 kN	LM71 LM SW/0 LM SW/2 HSLM
Braking	Qlbk = 20 (kN/m) x La,b (m) ≤ 6000 kN	LM71 LM SW/0 HSLM
	Qlbk = 35 (kN/m) x La,b (m)	LM SW/2

The UK National Annex (UK NA) to BS EN 1991-2 has taken the option to specify the traction and braking loads formerly adopted in the previous British standard, BS 5400-2:2006 'Specification for loads' (BSI 2006) (see Table 3). This is because the traction and braking loads are generally more severe than BS EN 1991-2, particularly in the span range 10m to 15m, and ensures that existing safety levels for bridge support structures are maintained.

Table 3 - Traction and braking requirements in the UK National Annex to BS EN 1991-2

Vertical Load Model	Load attributable to	Loaded length, L (m)	Longitudinal load (kN)
LM71 LMSW/0 HSLM	Traction (30 % on driving wheels)	Up to 3m From 3m to 5m From 5m to 7m From 7m to 25m Over 25m	150[1] 225[2] 300[3] 24 (L-7) + 300 750[4]
	Braking (25 % on braked wheels)	Up to 3m From 3m to 5m From 5m to 7m Over 7m	125[1] 187[2] 250[3] 20 (L-7) + 250[5]

Notes

1 Two wheels assumed for traction (75 kN per wheel) and two wheels assumed for braking (62.5 kN per wheel).

2 Three wheels assumed for traction and three wheels assumed for braking.

3 Four wheels for traction and four for braking.

4 Limit lower for maximum traction load compared to BS EN 1991-2 (1000 kN).

5 No limit on maximum value of braking load

In all cases for bridges supporting ballasted track, it is assumed that up to one third of the calculated longitudinal load is resisted by the track either side of the bridge, provided there are no rail discontinuities (such as expansion switches) within 18 m of the end of the bridge.

BS EN 1991-2 assumes that the traction and braking forces are uniformly distributed over the length of the bridge span. The assumption of uniformly distributed load is perhaps not realistic for the design of local details on the bridge (such as cross girders), but is likely to be a satisfactory approach for design of the bridge supports (abutments and piers).

Although modern braking systems may be designed to apply traction and braking loads uniformly along the length of the train, there are situations where the traction and braking forces are applied unevenly and forces are concentrated at individual axles. This is recognised in the UK National Annex (NA) to BS EN 1991-2 'Traffic loads on bridges' (BSI 2008), where the proportions of vertical load assumed to act on the driving and braked wheels are stated (see Table 3). The derived forces are proportional to the number of axles of the LM71 vehicle that can be accomodated within the span length of the bridge. The longitudinal loading values derived for LM71, are assumed to be the same where the accompanying vertical load model is LM SW/0 or HSLM (see Tables 2 and 3).

For routes requiring conformity with the Technical Specification for Infrastructure (INF TSI), an upper limit of 2.5 m/s2 is set for longitudinal forces applied to the track. For a vehicle weighing 80 tonnes with two twin axle bogies, this represents an axle force of 200 kN (50 kN per axle if 25 % vertical load on the braked wheels is assumed) which is less than the force for which bridges over 5 m span length are likely to be designed using the UK NA to BS EN 1991-2 (see Table 3).

Lateral (normal to the direction of travel)

There are two types of lateral load applied in the design of bridges:

1 Centrifugal forces

2 Nosing force

Centrifugal force

Centrifugal forces are required to be applied for the design of bridges '*where the track on a bridge is curved over the whole or part of the length of the bridge*' (clause 6.5.1(1), BS EN 1991-2). The centrifugal force is required to be '*combined with vertical traffic load*' but not 'multiplied by the dynamic factor Φ_2 or Φ_3 (6.5.1(3)).

Centrifugal forces are calculated from the following equations in BS EN 1991-2:

$Q_{tk} = V^2/127r\ (f \times Q_{vk})$ kN

or

$q_{tk} = V^2/127r\ (f \times q_{vk})$ kN/m

Where:

Q_{tk}, q_{tk} are characteristic values of the centrifugal force

Q_{vk}, q_{vk} are characteristic values of the vertical loads (LM71, SW/0,SW/2) excluding dynamic effects

V is the maximum line speed in km/h

f is a reduction factor which assumes that freight vehicle speeds are limited to a maximum of 120 km/h

r is the radius of curvature of the track in m

The loaded length for centrifugal forces is assumed to be compatible with that for the vertical loading applied to the bridge e.g. LM71. LM71 comprises concentrated load components (assumed to represent individual axles) as well as uniformly distributed load components (assumed to represent the average vehicle loading). Provision is made for calculation of the centrifugal force component which is compatible with the vertical load component.

'*For load model HSLM, the characteristic value of centrifugal force should be determined using Load Model 71*' (6.5.1(4)).

'*For LM71 and SW/0 centrifugal forces*' should be calculated using the load classification factor α applied to the vertical loads (6.5.1(9)).

Nosing force

The nosing force represents the maximum contact force applied to the rails by the wheel flanges of railway vehicles, due to lateral track alignment irregularities, '*on both straight track and curved track*' (6.5.2(1), BS EN 1991-2). It is intended to ensure that sufficient resistance is provided to avoid damage to the track as a result of damage to the supporting bridge structure.

The characteristic value of the force is '*taken as a concentrated force acting horizontally, at the top of the rails, perpendicular to the centre-line of the track*' (6.5.2(1), BS EN 1991-2). The value of the nosing force Qsk = 100 kN. It is not required to be multiplied by the dynamic factor Φ (6.5.2(2)) but it should be multiplied by the load classification factor (6.5.2(3)). '*The nosing force shall always be combined with a vertical traffic load*' (6.5.2(4)).

Network Rail standard NR/L2/CIV/020, 10.2.1, permits the nosing load to be distributed over three adjacent sleepers in the proportions ¼:½:¼ as for LM71 (6.3.6.1(1)) (see also Figure 6).

A.3 Loading requirements for track

The loading requirements specified in standards for permissible vehicle/track forces in the vertical and horizontal directions, is discussed in the following.

Permissible vehicle/track force, GM/TT0088, and GC/RT5021

GM/TT0088

A British Rail Research (BRR) report 'Design Vehicle Loading on Structures and Track' (BRR 1996), has previously considered the value of the permissible vehicle/track force (commonly referred to as the P2 force) of 322 kN per wheel contained in Railway Group Standard GM/TT0088 'Permissible Track Forces for Railway vehicles'. This was thought to be necessary because it seemed high in relation to the bridge design loading set out in the draft Eurocode for bridge loading (ENV1991-3), which was current at the time.

The P2 force of 322 kN is a dynamic force which represents the maximum permitted force from a wheel of the vehicle in service (i.e. static load + dynamic increment).

GC/RT5021

Design requirements for track systems are specified in GC/RT5021, Issue Five, 'Track System Requirements', 3.1.1.

This standard states that '*Track systems for all new construction shall be designed to have performance characteristics capable of sustaining the following forces:*

a *A maximum static axle load of 250 kN (25.5 t).*

b *A vertical dynamic force, generated by the static wheel load and the low frequency dynamic forces P2, of 350 kN per wheel and an occasional isolated vertical load of 500 kN per wheel.*

c *A longitudinal force of 1200 kN per rail, to allow for train acceleration and braking, and the thermal forces within the rail.*

d *A lateral force generated by a train of 100 kN over a length of 2m.*'

A.4 Loading requirements for the design of bridges and track systems

The loading requirements for bridges and track are compared in this section.

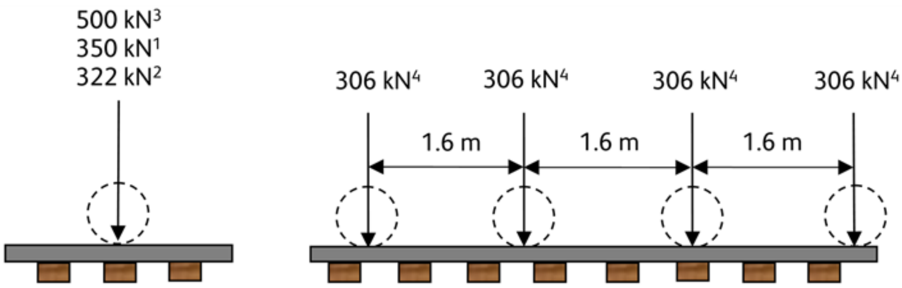
Vertical loading

The maximum value for the P2 force is compared with the maximum wheel loading obtained from LM71 for the design of bridges (see Figure 5).

The maximum vertical axle load specified for LM71, which is used for the design if bridges, is 250 kN (125 kN per wheel). This represents an effective dynamic amplification factor of 2.58 (322/125), when compared to the maximum force which can be applied to track by vehicles in GM/TT0088. The dynamic factor which is applied to the static value of the axle loads within LM71, in accordance with section 6.4.5 of BS EN 1991-2, is dependent on the track quality. The maximum value of the dynamic factor Φ_3 for '*track with standard maintenance*' is 2.0. The value of the wheel force then becomes equal to Φ_3 x LM71 (2 x 125 = 250 kN) which, when compared to the GM/TT0088 force of 322 kN, represents only 78 % of the permissible vehicle/track force (250/322 = 0.78).

However, for the design of bridges, BS EN 1991-2 requires additional factors, as well as the dynamic factor Φ_3, to be applied to the LM71 wheel load in order to obtain the maximum value for design (RSSB T696). These factors include an allowance for variation of wheel loads to allow for uneven distribution of load in freight wagons (clause 6.3.5), and the load classification factor (α) (clause 6.3.2 (3)), which is used to account for traffic which is '*heavier or lighter than normal rail traffic*'. The effect of maximum 'eccentricity of vertical loads' in 6.3.5, is to increase the maximum wheel load from 125 kN to 139 kN. The GB value for   is generally taken as 1.1 for TSI compliant routes. In this case, the maximum wheel load then becomes Φ_3 x α x 139 = 2.0 x 1.1 x 139 = 306 kN, which is very similar to the P2 force of 322 kN within approximately 5 %. Comparison of the maximum wheel loads specified for the design of bridges and track systems is included within Figure 5.

Figure 5 - Comparison of LM71 and the load model implied by GM/TT0088



NOTES

1. GC/RT5021 – low frequency dynamic force, P2
2. GM/TT0088 – P2 force for vehicle acceptance
3. GC/RT5021 – occasional isolated vertical load per wheel
4. BS EN 1991-2 – quasi-static axle load, LM71

The BRR report referred to in 3 (BRR 1996) concluded that, although the P2 force was slightly higher than the maximum wheel load obtained from the application of LM71, it is broadly compatible owing to the attenuation of load that takes place between the rail, the sleepers, the ballast, and the supporting bridge structure. It further concluded that other types of bridge structure, such as unballasted structures, '*may not provide the same degree of attenuation*'.

The lighter axle loads (17t to 21t) but greater speeds (> 200 km/h) associated with the high speed load models (HSLMs) (see Figures 3 and 4 and Table 1), create dynamic behaviours in bridge structures that were not anticipated prior to the introduction of high speed rail travel. Development of the HSLMs for the design of bridges, and the provision of acceptance criteria for the dynamic response of the structure, required an extensive set of dynamic analyses and site measurements.

It is expected that similar studies would be necessary for the development of appropriate load models, and the determination of acceptance criteria, for track systems. For track systems, it is expected that the velocity and frequency of soil displacement associated with ground surface waves due to dynamic impact forces transmitted through the track system, will be important for the prevention of track destabilisation. It will also be necessary to consider the dynamic response of the track system, including its support structure. Additionally, the likelihood that its dynamic characteristics will match the frequency of loading due to the appropriate load model(s) for the design of the track system and its supporting structure, will need to be considered.

Horizontal loading

The bridge design requirements for horizontal loading are intended to ensure that the bridge structure is capable of resisting the maximum expected force (in any horizontal direction) on the rail and any track support structure (eg. ballast or a concrete slab). It would therefore seem logical that the bridge and track support forces are higher than those intended for acceptance of rolling stock.

The magnitude of the longitudinal loading for the design of bridges, is dependent upon the span of the bridge, and whether traction or braking forces are being considered (see Tables 2 and 3). The forces for bridge design are specified per track, whereas the maximum load capacity for the design of the track system (see 3) is specified per rail and includes an allowance for thermal effects.

For the purposes of comparison, the maximum force permitted in two rails according to GC/RT5021 is 2400 kN, the maximum bridge length required to achieve a longitudinal braking force equivalent to this value is 120m according to BS EN 1991-2 (Table 2) and 114m according to the UK NA to BS EN 1991-2 (Table 3). For loaded lengths greater than these two limits, the longitudinal rail forces will exceed the specified maximum design force in GC/RT5021 (1200 kN per rail).

For the design of bridges to resist loading in the lateral direction, the maximum value will be obtained due to application of both the centrifugal and the nosing forces on curved sections, and the nosing force only on straight sections (see 2). The maximum design loading requirement for the track system, is to resist a lateral force of 100 kN (see 3).

For application of the nosing force, a key difference between the design of bridge and track system components, arises due to the assumption for length of rail over which the force is assumed to be distributed as:

- GC/RT5021 - 2m for the rail
- NR/L2/CIV/020, 10.2.1 - Two sleeper spacings (say 1.2m to 1.3m) for design of bridges

On this basis an unfactored design force of 50 kN/m (100kN/2m) would be required for the design of the rail to GC/RT5021, and a design force of around 80 kN/m (100kN/1.2m) would be required for design of the bridge based on NR/L2/CIV/020.

The rolling stock acceptance criteria defined in GM/TT0088 is a concentrated force of 71 kN, which is based on the Prud'homme force for an axle load of 183 kN.

The rolling stock acceptance forces are distinguished in terms of those forces which may be assumed to be averaged over a length of rail of 2m or more and those that apply where short wave lateral irregularities in the lateral track profile occur. The latter case seems to be relevant for concentrated load applications such as those applied to track system components.

A point load might be assumed to be more relevant for the design of local details such as track fixings or welded joints. For supporting structures, it would be expected that the load would be further distributed once the effect of distribution through the ballast is taken into account.

A.5 Design requirements for track systems

For the design of bridges, it is assumed that the track and its fixings have been designed to be sufficiently resilient, and the primary focus is to ensure that the bridge is robust enough to support the track without damaging it, including the rails, the sleepers (where provided), and the track fixings.

Comparison of LM71 and the implied load model for track based on the permissible vehicle/track force in accordance with GM/TT0088, and also the loading requirements for track design in GC/RT5021 (see Figure 3), demonstrates the key physical differences in the model configurations. The application of a single wheel load compared to a sequence of wheel (or axle) loads, may be considered to be indicative of the different design needs for track systems and their components and a supporting structure such as a bridge.

To ensure adequate resistance of the track and its components, a single point load which represents the wheel force, may be fine for ensuring (by measurement of track forces) that permitted values are not exceeded at a point on the track network. However, real trains comprise several consecutive axles, and for design of railway bridges (taken to mean structures with spans greater than 1.8 m), it is often necessary to take into account the effect of more than one axle. However, it is also the case that the shorter the span, then the greater the influence of an individual axle will be.

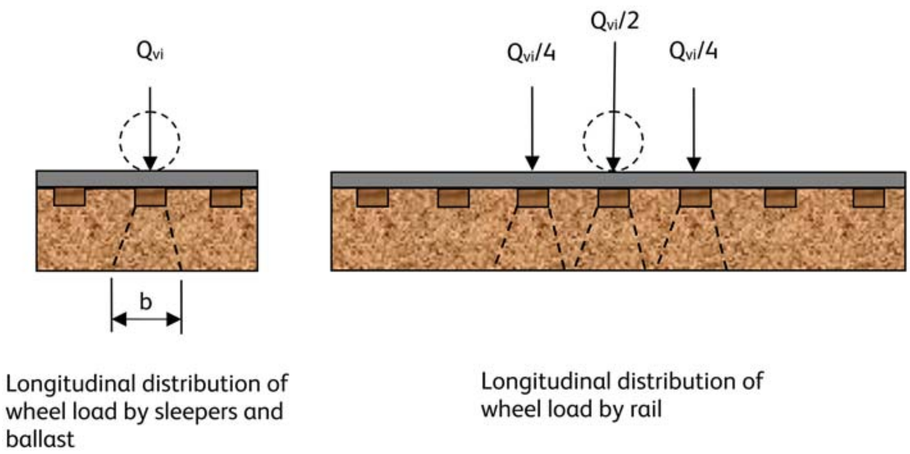
When considering LM71, it should be borne in mind that the wheel spacing is not the same as for typical trains that run on the network. However, LM71 has provided a safe load model, which replicates the effects of trains (passenger and freight) that use the mainline railway network, for the design of railway bridges over many years.

When distinguishing between load model requirements for track systems compared to bridges, consideration of Figure 3 illustrates the difference in the load that the supporting structure sees. The design of the track system may well be dominated by the influence of a concentrated local load that affects the response of the rail, the rail fixings, the sleeper, and the ballast (and/or slab). The response will be influenced by whether support to the track system is provided by the ground or the bridge.

For the design of bridges, load attenuation through sleepers and ballast (and/or slab) for design of local structural elements, is generally assumed (see Figure 6), but not necessarily taken into account for design of the main structural members of the bridge spanning in the direction of the track.

Based on consideration of bridge and track loading requirements, it therefore seems to be necessary to distinguish between the track, comprising the rails, fixings, load attenuation components (such as rail pads), sleepers (including load attenuation components such as under sleeper pads), and the support structure, including ballast, slab, the ground, and the bridge structure, as well as combinations of these.

Figure 6 - Distribution of wheel load



The maximum load experienced by each of the track system components, will depend upon what components are provided and their stiffness. However, the load for which a track system is designed must be the same at the wheel/rail interface at the point of load application, until such time as there is a structural response of the track system, including its supporting structure. The maximum load carried by the track system components will also depend upon their proximity to the applied load.

The maximum axle load and the associated permitted speed ranges, are specified in connection with line classification information for traffic performance parameters in the Infrastructure Technical Specification for Interoperability (INF TSI). The appropriateness of particular load models which satisfy the design criteria set in the INF TSI, is likely to be dependent upon the particular component of the track system and its support structure.

John Lane, 2015

Appendix B: Review of prEN 16432-1:2015 Loading Requirements

B.1 Outline

The review has taken place within the framework of the Phase 1 of the Research Project 'T1073-01 Loading Requirements for Track Systems' and solely as regards the loads specification in Clause 5.1.1 to 5.1.4 of the prEN 16432-1:2015 Standard included, inclusive of any sub-clauses.

B.2 Review comments

Clause 5.1.1: The scope of load definition and design aim should be clarified. This applies to the entire section under study (Clause 5.1.1 to 5.1.4).

Clause 5.1.2: It is considered that the recommendation to use Load Model 71 (EN 1991-2: 2003) in Clause 5.1.2.1 and 5.1.2.2 derives from the German practice for designing slab track (Eisenmann et al., 2000). The standard as drafted provides however flexibility to consider alternative loadings (Clause 5.1.2.1, 5.1.2.3 and 5.1.2.4).

Allowance for cross wind loads (vertical quasi-static loads) should be added in Clause 5.1.2.5. Provision for the factor kq to take into consideration vertical quasi-static loads due to residual centrifugal force mentioned in Clause 5.1.2.5 aligns with usual track engineering practice and values referred herein; flexibility to consider real rolling stock and track alignment data (potential optimisation) is provided.

The dynamic factor value of kd = 1.50 mentioned in Clause 5.1.2.6 is considered to refer to V=300km/h, ϕ = 0.1 (very good track), t = 3, η = 1.63 (for V=300km/h), see also Eisenmann et al., 2000. It is considered that no specific value of kd should be provided as the value should be driven by the operational, track and rolling stock characteristics of the line under study.

There is no mention of the total vertical load to be considered and this should be calculated (analytical expression).

Clause 5.1.3: The factor value of k1 = 1.00 (Clause 5.1.3.2), specified as common for all vehicles, is not in line with the current BS EN 14363:2005 (differentiates between passenger and freight vehicles); the symbol for the wheel load employed is not the one usually employed in the literature (Qo).

The use of the term 'exceptional' …lateral load (5.1.3.3) is considered as not appropriate, especially, given the previous employment of the same term in Clause 5.1.1 and 5.1.2. The quotient limit in Clause 5.1.3.3 is higher than the limit value within the current BS EN 14363:2005 (limit value of 0.80). The criterion is ambiguous and the same applies for its application in practice; the criterion normally refers to the quotient of the guiding force exerted by the wheel of the leading wheelset on the rail, in case of wheel flange - rail contact, divided by the effective wheel load. It is unclear, in this respect, how this criterion can be applied in practice by the designer / specifier on its own.

There is no mention of the total lateral load to be considered; the role of the loads in Clause 5.1.3.3 and whether, they need to be considered in combination with the loads in Clause 5.1.3.2 (we consider not) is not clear.

Clause 5.1.4: It is considered that values should be specified in relation to the static wheel load or other. Eddy current brake application loads have not been reviewed.

Appendix C: Loads on the track

C.1 Vertical loads

C.1.1 Static vertical loads

Axle load

The term weight or load per axle (axle load) describes the static load Q, which is individually transferred, by each axle of a vehicle and in general of a train, to the rails. Considering symmetrical loading of the various vehicle parts, the axle load equals the quotient of the total vehicle weight divided by the total number of axles.

For example, in the case of a vehicle with two two-axle bogies, the following mathematical equation applies:

$$Q = \left(M\frac{\overline{M}}{4} + \frac{M'}{2} + m \right) \cdot g$$

(22)

Where $\overline{M}$ = car body mass (t), M’ = bogie mass (t), m = railway axle mass (axle body + wheels + axle-boxes) (t) and g = acceleration due to gravity (m/s^2).

Wheel load

The term weight or load per wheel (wheel load) refers to the static wheel load Q_o, which is individually transferred by each wheel of the vehicle to the corresponding rail.

Considering a symmetric loading of the vehicle, the following mathematical equation applies:

$$Q_o = \frac{Q}{2}$$

(23)

Where Q = load per axle (kN).

In practice and particularly along curves, the wheel loads within the same wheelset are not equal to each other (either due to, for instance, the loading of the vehicle springs - if no cant deficiency, for instance, or due to the vertical component of the residual centrifugal force - if cant deficiency, respectively).

C.1.2 Quasi-static vertical loads

Vertical wheel load due to cross winds

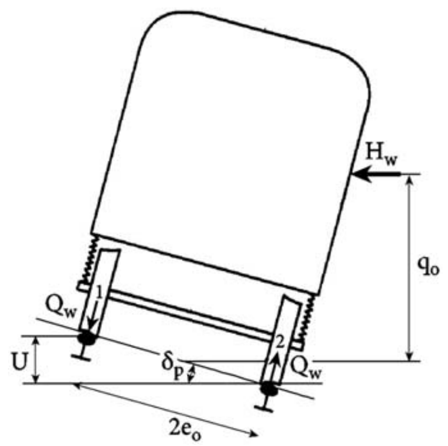
The vertical wheel load Q_w due to cross winds is described by the following mathematical equation (see Figure 5):

$$\pm Q_{wj} = H_w \cdot \frac{q_o}{2 \cdot e_o} \tag{24}$$

Where j = (1, 2) = index related to the two wheels of the same railway axle, H_w = cross wind force applied on the geometrical centre of the lateral surface of the car body, q_o = vertical distance between the geometrical centre of the lateral surface of the car body and the rolling plane and 2_{eo} = distance between the vertical axes of symmetry of the running rails.

When the force H_w is directed from wheel 2 towards wheel 1, then wheel 1 load increases by Q_w, while wheel 2 load decreases by the same amount. As a result of cross winds, where applicable, the vertical load acting on the track may be applied during motion both in tangent alignment and in curves. The wind force may be either estimated from site measurements or calculated using the applicable codes in vigour.

Figure 7 - Vertical load ±Qw due to cross winds - Motion in curve (Source: Esveld, 2001)



Vertical wheel load due to residual centrifugal force

The vertical wheel load Q_{nc}, which is due to the residual centrifugal force F_{nc} (see Figure 6), is expressed as:

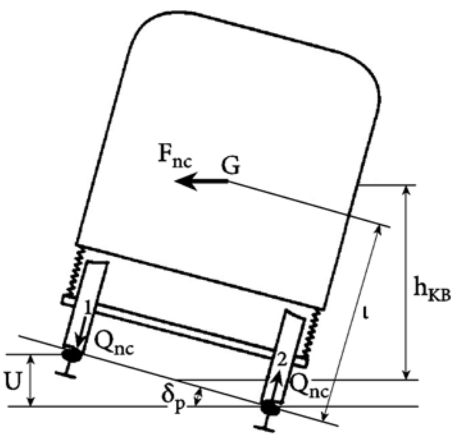
$$\pm Q_{ncj} = F_{nc} \cdot \frac{h_{KB}}{2 \cdot e_o} = Q \cdot \frac{I \cdot h_{KB}}{4 \cdot {e_o}^2}$$

(25)

Where F_{nc} = residual centrifugal force, I = cant deficiency, h_{KB} = distance between the vehicle's centre of gravity G (KB) and the rolling plane, U = track cant, R_c = horizontal alignment curve radius and δ_p = angle of cant.

The vertical wheel load, which is due to the residual centrifugal force, is only applied during negotiation of curved sections of the horizontal alignment by the rail vehicle. The load of the wheel on the outer rail is increased by Q_{nc}, while the load of the inner wheel is decreased by the same amount respectively.

Figure 8 - Vertical load $\pm Q_{nc}$ due to residual centrifugal force
(Source: Esveld, 2001)



C.1.3 Dynamic vertical loads

The vertical dynamic wheel load Q_{dyn} is the sum of individual dynamic loads, namely:

$$Q_{dyn1}, Q_{dyn2}, Q_{dyn3}$$

Where Q_{dyn1} = dynamic wheel load due to the vehicle's sprung masses, Q_{dyn2} = dynamic wheel load due to the vehicle's semi-sprung masses and Q_{dyn3} = dynamic wheel load due to the vehicle's un-sprung masses.

Hence, Q_{dyn} equals:

$$Q_{dyn2} = Q_{dyn1} + Q_{dyn2} + Q_{dyn3} \tag{26}$$

Where j=1,2 = index related to the two wheels of the same railway axle.

Experimental results (Knothe 2001, Alias 1984, Prud'homme 1975) suggest that the study of vertical dynamic loads can be conducted at different frequency ranges, depending on the oscillation cause and mechanism.

Depending on the excitation frequency ($f=V/\lambda$, where, f is the excitation frequency in Hz, V is the vehicle speed in m/s and λ is the excitation wavelength in m, see Table 8), dynamic loads acting on the track in the vertical direction may be characterised as (Knothe 2001):

- Low-frequency dynamic loads, corresponding to f=0-40Hz.
- Medium-frequency dynamic loads, corresponding to f=40-400Hz.
- High-frequency dynamic loads, corresponding to f=400-2000Hz; note that the limit of 2000Hz is conventionally set.

Table 4 - Track dynamic vertical loading as a matter of the excitation frequency (Source: Demiridis, 2006)

Loading / Excitation		Static	Dynamic		
Excitation range		- (0Hz)	Low (0-40Hz)	Medium (40-400Hz)	High (400-2000Hz)
Excitation cause Loading mechanism Railway track		-	Substructure / Trackbed Mid and long wavelengths λ (λ>0.5m, V=20m/s) (λ>2.0m, V=80m/s)	Track panel Short wavelengths λ and rail undulations (λ<0.5m, V=20m/s) (λ<2.0m, V=80m/s)	Rails Rail roughness and discontinuities (λ<0.05m, V=20m/s) (λ<0.20m, V=80m/s)
Excitation cause Loading mechanism Rolling stock		Static loads (own weight)	Sprung and semi-sprung masses	Un-sprung masses Wheel tyre short wavelengths λ (λ<0.5m, V=20m/s) (λ<2.0m, V=80m/s)	Un-sprung masses Wheel tyre roughness and discontinuities (λ<0.05m, V=20m/s) (λ<0.20m, V=80m/s)
Effect	End user	-	Vehicle accelerations	Rolling noise	Rolling noise
	Environment	-	Vibrations	Rolling noise	Rolling noise
	Railway track	Wear Fatigue Settlements	Substructure / Trackbed fatigue	Track panel fatigue	Rails fatigue
	Rolling stock	-	Carriage and bogie fatigue Vehicle stability	Wheelset fatigue	Wheelset fatigue

This discrimination takes into consideration:

- The excitation causing mechanism at the level of the railway track.
- The effect of loading - result of the excitation, or the way this is being perceived.
- The non-linear behavior of certain track materials, where applicable (Thompson 1999, Grassie 1989).

C.2 Lateral loads

C.2.1 Quasi-static lateral loads

Cross wind forces

In case of cross winds, a lateral force H_w is transferred through the axles to the rail rolling plane. This force is considered as quasi-static and its direction depends on the wind's direction; it is considered to act on the geometrical centre of the car body's lateral surface. It may be either estimated from site measurements or calculated using the applicable codes in vigour.

The cross wind force H_w induces a lateral displacement to the vehicle's axles and exacerbates the vehicle's overturning mechanism.

Residual centrifugal force

When a vehicle of mass Mt negotiates a horizontal alignment curve where the curve radius is R_c, running at a velocity V, a centrifugal force F_{cf}, acting on the vehicle's centre of gravity is generated, which pushes the vehicle towards the outside of the curve (see Figure 7).

$$F_{cf} = M_t \cdot \frac{V^2}{R_c} \tag{27}$$

Due to the cant U applied on the track, the lateral component of the weight B_{ty} (or FB as denoted in Figure 7) is simultaneously applied. B_{ty} acts to the opposite direction of that of the centrifugal force and its value equals to:

$$B_{ty} = M_t \cdot g \cdot \delta_p = M_t \cdot g \cdot \frac{U}{2 \cdot e_o}$$

(28)

The difference between forces F_{cf} and B_{ty} (or FB as denoted in Figure 7) expresses the residual centrifugal force Fnc. At the level of the railway axles and consequently at the rail rolling surface, mathematical equation 29 applies:

$$F_{nc} = \frac{Q}{g} \cdot \left(\frac{V^2}{R_c} - g \cdot \frac{U}{2 \cdot e_o}\right) = \frac{Q \cdot I}{2 \cdot e_o}$$

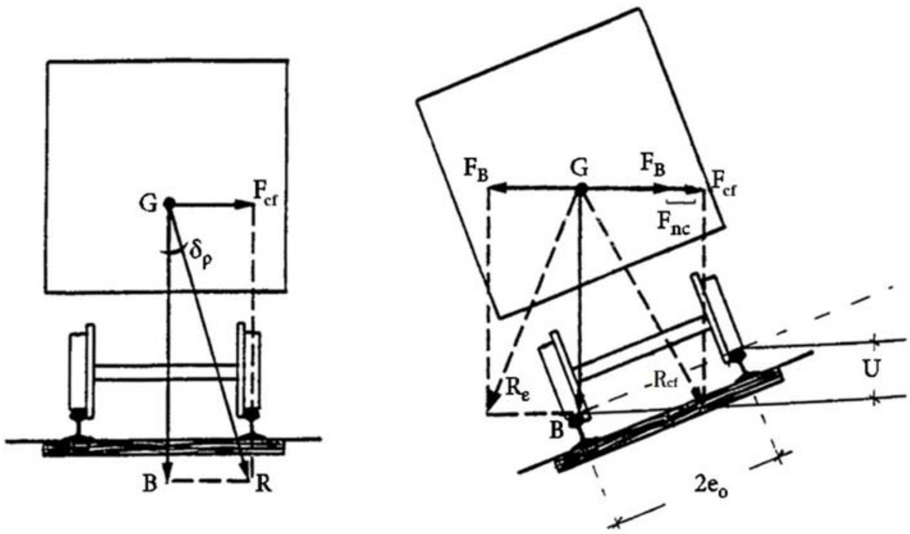
(29)

Where I = cant deficiency (mm).

The value $\left(\frac{V^2}{R_c} - g \cdot \frac{U}{2 \cdot e_o}\right)$ represents the lateral residual acceleration Y_{nc}.

The increase of the vehicle's velocity V as well as the decrease of the curve radius R_c and the decrease of cant U contribute to the increase of the residual centrifugal acceleration.

Figure 9 - Rail vehicle on curve - Residual centrifugal force F_{nc} (SourceL Alias, 1984)



F_{nc} is considered as a quasi-static force; it is undesirable as it not only causes the lateral displacement of the vehicle's wheelsets (risk of flange contact), but may exacerbate passenger comfort problems.

Indicatively, when Q = 18t, V = 150 km/h, Rc = 1500m, g = 9.81 m/sec2, 2eo = 1.50 m and U = 130 mm, then F_{nc} = 5.6 kN and Υ_{nc} = 0.307 m/sec2.

C.2.2 Dynamic lateral loads

C.2.2.1 Gravitational forces

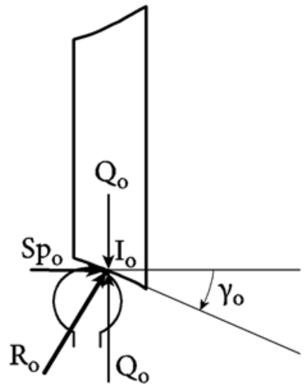
At every wheel/rail contact point, the reaction force Ro may be analysed in two components; namely, Q_o and Sp_o (see Figure 8). The lateral component Sp_o is defined as the gravitational force; this force is exclusively due to the conicity of the wheels and acts, through the axle, on the rolling plane of rails.

The gravitational or restoring force is considered a dynamic force, equal to:

$$Sp_j, j = 1, 2Q_o \tan\gamma_o \qquad (30)$$

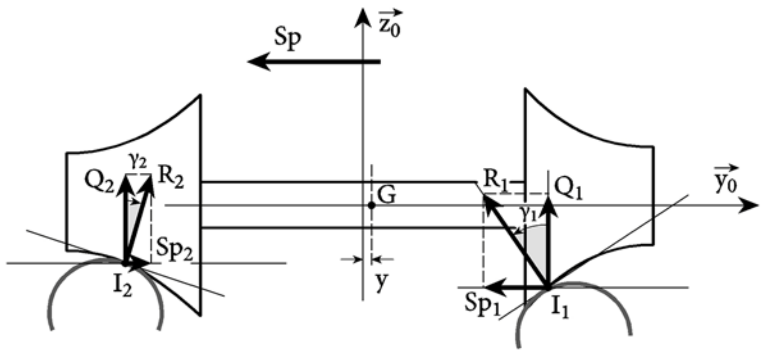
Where Q_o = static wheel load (kN) and γ_o =angle between the tangent plane at point I_o and the horizontal plane.

Figure 10 - Gravitational force per wheel (Source: Pyrgidis, 1990)



In case of an individual wheelset, there are two restoring forces, one per wheel (Sp_j, j = 1, 2) as shown in Figure 9.

Figure 11 - Total gravitational force (per axle) (Source: Pyrgidis, 1990)



The lateral displacement of the wheelset being symbolised by γ, the following mathematical equations apply:

$$Sp_1 = Q_1 \tan\gamma_1 = Q_1\gamma_1$$
$$Sp_2 = Q_2 \tan\gamma_2 = Q_2\gamma_2$$

(31), (32)

Where Q_1, Q_2 = vertical component of the reactions R_1 and R_2 at the contact points I_1 and I_2 respectively and γ_1, γ_2 = angles formed by the horizontal plane and the tangent planes at the contact points I_1 and I_2 respectively (as γ_1, γ_2 are very small quantities, $\tan\gamma_1 = \gamma_1$ and $\tan\gamma_2 = \gamma_2$ applies).

From the mathematical solution of the wheel - rail contact geometry and assuming that the wheels are of conical shape with curved treads of constant radius, while the heads of the rails are spherical, these following linear relationships are concluded:

$$\gamma_1 = \gamma_o + \frac{\gamma_e}{R \cdot \gamma_o} \cdot y$$

$$\gamma_1 = -\gamma_o + \frac{\gamma_e}{R \cdot \gamma_o} \cdot y$$

$$\gamma_e = \frac{R \cdot \gamma_o}{R - R'}$$

(33), (34), (35)

Taking into account the mathematical equations (33) to (35) and assuming equal weight distribution per wheel, the following mathematical expressions are concluded for the total gravitational force (Pyrgidis 1990):

$$S_p = S_{p1} + S_{p2} = 2 \cdot Q_o \cdot \frac{\gamma_e \cdot y}{R \cdot \gamma_o}$$

(36)

Substituting Equation 35 into Equation 36 results in:

$$S_p = 2 \cdot Q_o \cdot \frac{1}{R - R'} \cdot y$$

(37)

Where R = curvature radius of the wheel tread, γ_e = equivalent conicity of the wheel and R' = radius of curvature of the rolling surface of the rail head.

Considering the mathematical equations (34) and (35), we may conclude that:

- The total gravitational force is proportional to the displacement y of the axle's centre of gravity. This means that, if for any reason, the axle is displaced laterally, then the gravitational force tends to reinstate it to its initial equilibrium position (for y = 0, Sp = 0).

- The total gravitational force is inversely proportional to the curvature radius R of the wheel tread and proportional to the equivalent conicity γ_e. Hence, worn wheels (small R values / large γ_e values), automatically generate a greater gravitational force.

It is to be noted that in the case of conical wheels (triangular cross section), the total gravitational force is zero (R = ∞, $\gamma_e = \gamma_o$, $\gamma_1 + \gamma_2 = 0$, Sp=0); the same applies in the case of cylindrical wheels (orthogonal cross section) (R = ∞, γ_o = 0, S_p=0).

Indicatively, for $2Q_o$ = 13.50t, R = 0.344m, R' = 0.30m and y = 5mm, then: S_p = 1.53t or 15.3kN.

C.2.2.2 Creep forces

It is noted that the Kalker's linear theory is used in the following sections; the formulae shown are for presentation reasons only.

Running on straight track

In the case of a conventional railway axle running on a tangent alignment section, the analytical expressions of the creep forces resulting from the application of the Kalker's linear theory are given by the mathematical equations (38) to (43) below (Pyrgidis 2009, Pyrgidis 1990).

$$X_1 = -c_{11} \cdot \left(\frac{x'}{V} - \frac{e_o}{V} \cdot a' - \frac{\gamma_e}{r_o} \cdot y \right)$$
$$X_2 = -c_{11} \cdot \left(\frac{x'}{V} + \frac{e_o}{V} \cdot a' + \frac{\gamma_e}{r_o} \cdot y \right)$$
$$T_1 = -c_{22} \cdot \left(\frac{y'}{V} - a \right) - c_{23} \cdot \left(\frac{a'}{V} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o} \right)$$
$$T_2 = -c_{22} \cdot \left(\frac{y'}{V} - a \right) - c_{23} \cdot \left(\frac{a'}{V} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o} \right)$$
$$M_1 = -c_{23} \cdot \left(\frac{y'}{V} - a \right) - c_{33} \cdot \left(\frac{a'}{V} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o} \right)$$
$$M_2 = -c_{23} \cdot \left(\frac{y'}{V} - a \right) - c_{33} \cdot \left(\frac{a'}{V} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o} \right)$$

(38), (39), (40), (41), (42), (43)

Where,

X_1, X_2: longitudinal creep forces applied on both wheels

T_1, T_2: lateral creep forces applied on both wheels

M_1, M_2: spin moment on both wheels

x: longitudinal displacement of the wheel set

y: lateral displacement of the wheel set

α: yaw angle of the wheel set

ϕ: angle of rotation of the wheels and the wheel set

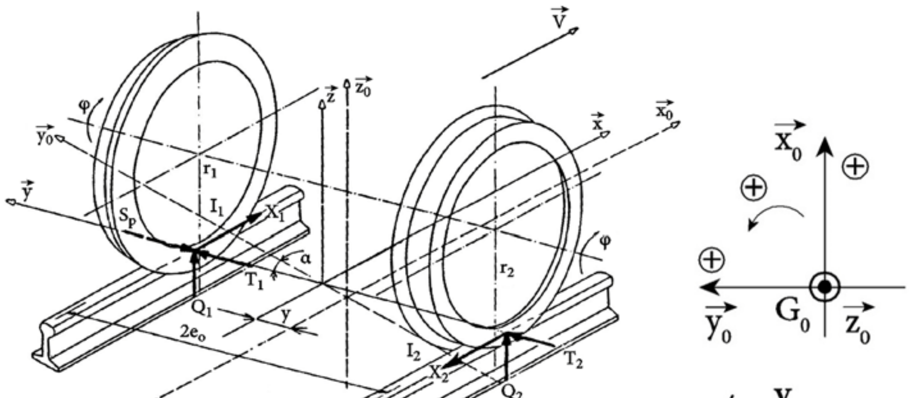
x', y': derivative of displacements x, y, of the yaw angle α and of angle of rotation ϕ

c_{11}: longitudinal coefficient of Kalker

c_{22}: lateral coefficient of Kalker

c_{23}, c_{33}: spin coefficient of Kalker

Figure 12 - Forces applied on the rail - wheelset in random position on the track (Source: Pyrgidis, 1990)



From the mathematical equations (36) to (39), we may conclude:

The longitudinal creep forces X_1 and X_2 applied on both wheels create a pair of forces, which tends to rotate the railway axle around axis $\vec{z}_0$ (below). For x = 0, the forces X_1 and X_2 are equal in terms of magnitude and opposite in terms of direction.

The lateral forces applied on each wheel, when C_{23}, C_{33} = 0 (spin), are equal and act in the same direction.

The increase of the displacement velocity V reduces the damping terms $\frac{x'}{V}, \frac{a'}{V}, \frac{y'}{V})$ that tend to stabilise the axle.
The increase of the equivalent conicity γ_e and the decrease of the wheel rolling radius r_o lead to an increase of the value of the longitudinal creep forces.

Running in curves

In the case of a conventional axle negotiating a curve, the analytical expressions of the creep forces resulting from the application of Kalker's linear theory are given by the mathematical equations (44) to (49) below:

$$X_1 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{V} \cdot a' + \frac{e_o}{R_c}\right)$$
$$X_2 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y + \frac{e_o}{V} \cdot a' - \frac{e_o}{R_c}\right)$$
$$T_1 = -c_{22} \cdot \left(\frac{y'}{V} - a\right) - c_{23} \cdot \left(\frac{a'}{V} - \frac{1}{R_c} - \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right)$$
$$T_2 = -c_{22} \cdot \left(\frac{y'}{V} - a\right) - c_{23} \cdot \left(\frac{a'}{V} - \frac{1}{R_c} + \frac{\gamma_o}{r_o} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right)$$
$$M_1 = -c_{23} \cdot \left(\frac{y'}{V} - a\right) - c_{33} \cdot \left(\frac{a'}{V} - \frac{\gamma_o}{r_o} - \frac{1}{R_c} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right)$$
$$M_2 = -c_{23} \cdot \left(\frac{y'}{V} - a\right) - c_{33} \cdot \left(\frac{a'}{V} + \frac{\gamma_o}{r_o} - \frac{1}{R_c} - \frac{\gamma_e \cdot y}{R \cdot \gamma_o \cdot r_o}\right)$$

(44), (45), (46), (47), (48), (49)

In the case of small radii curves and assuming a displacement velocity that is approximately equal to the equilibrium velocity of the line, the inertia and damping forces may be disregarded when compared with the elastic forces. Moreover, by ignoring the spin impact, the following mathematical equations apply:

$$X_1 = -c_{11} \cdot \left(-\frac{\gamma_e}{r_o} \cdot y + \frac{e_o}{R_c}\right)$$
$$X_2 = -c_{11} \cdot \left(+\frac{\gamma_e}{r_o} \cdot y - \frac{e_o}{R_c}\right)$$
$$T_1 = T_2 = -c_{22} \cdot (-a)$$

(50), (51), (52)

Longitudinal creep forces result in the horizontal rotation of the axle and along with the lateral creep forces activate the sinusoidal movement (hunting) of the bogie axles, thereby causing undesired vehicle oscillations (hunting).

The presence or absence of the longitudinal and lateral component of the creep force depends on how the wheels are connected to the axle, while the presence or absence of spin moment depends on the angle formed by the wheel rolling surface plane and the rotation angle.

To eliminate wheelset hunting, it is essential that the rigid link between the two wheels and the axle body be broken; thus the two wheels are then able to rotate at different angular velocities, while at the same time $\omega_1 r_1 = \omega_2 r_2 = V$, where ω_1, ω_2, the angular velocities of the two wheels and r_1, r_2, their rolling radii respectively.

This ensures rolling of the two wheels without creep and elimination of longitudinal creep forces; the technology of bogies with independent wheels is based on this principle.

C.2.2.3 Forces due to vehicle oscillations

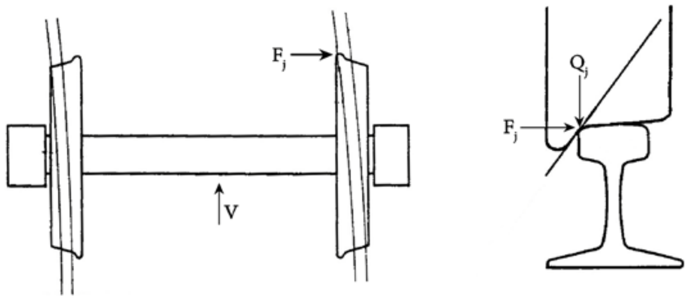
Lateral dynamic loads P_{dyn} may arise due to alignment faults similarly to vertical dynamic loads arising out of level and twist faults.

They may be estimated either by in-situ measurements or track - vehicle interaction models (for example, through the use of Vampire®) on the basis of track alignment spectral density functions.

C.2.2.4 Flanging forces

When the lateral displacement y of a railway axle is equal to the flange clearance between the wheel flange and the rail, the outer side of the wheel flange comes in contact with the inner part of the rails (see Figure 11) resulting in flanging forces. These forces are considered to be a consequence of inadequate rolling radius difference.

Figure 13 - Origin of flange contact forces (Source: Esveld, 2001)



Lateral dynamic loads are exerted on the contact point, called flanging forces F_j (j = 1 or 2); they are of dynamic, random nature. Flanging forces are highly undesired since:

- They compromise dynamic passenger comfort
- They are a source of increased rolling noise
- They cause wear to the wheels and rails
- They exacerbate bogie and wheelset fatigue

Under certain circumstances, they may result in the higher friction forces, which when combined with vertical unloading, increasing the risk of derailment of the vehicles.

The calculation of flanging forces may be carried out either in the field, using appropriate measurement apparatus, or theoretically (mathematical models simulating the vehicle's dynamic behaviour) (Liu et al. 2011, Joly et al. 1996).

C.3 Longitudinal loads

C.3.1 Thermal forces

When the temperature changes, the length of the rails increases or decreases by a value Δl equal to:

$$\Delta l = \alpha t \cdot \Delta t \cdot l_o \tag{53}$$

Where Δl = variation of the length of the rail (expansion or contraction) (in mm), αt = steel thermal expansion coefficient (in grad-1), $\Delta t = t_{re} - t_{in}$= actual (recorded) temperature- initial temperature (in ˚C) and l_o =initial rail length (in mm).

This displacement is countered by the friction forces T_{fr} developed between rails and sleepers and between sleepers and ballast, where applicable; for a directly fastened track this would be the fastenings.

The rail is under compressive stress when the temperature increases and under tensile stress when the temperature decreases.

In the case of continuous welded rails (CWR) a pair of compressive or tensile forces N (thermal forces) develops axially in the rail:

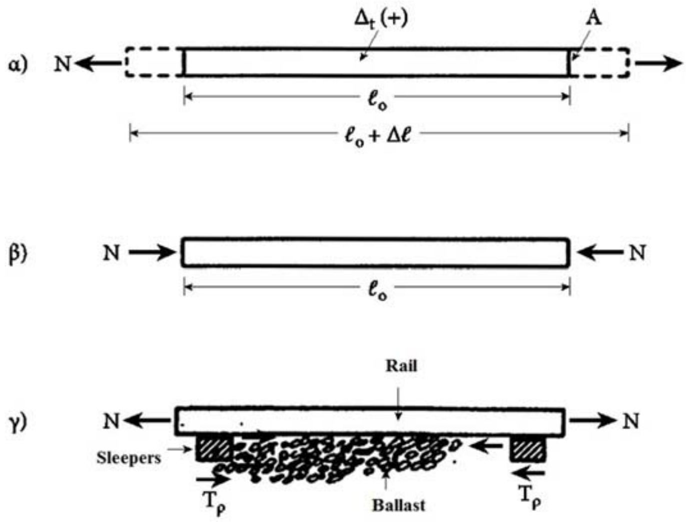
$$N = -E \cdot A_r \cdot \frac{\Delta l}{l_o} = -E \cdot A_r \cdot \alpha_t \cdot \Delta_t \tag{54}$$

Where A_r = rail cross section (in mm2) and E = steel modulus of elasticity (in kN/mm^2).

Thermal forces may be approximated to static forces (Lichtberger 2005).

The value of thermal forces remains constant almost throughout the length of the continuous welded rails section and only reduces at a distance from the ends of the rail (the breathing or expansion zone), where the value of the N force gradually decreases to zero.

Figure 14 - Temperature forces N (Source: Profyllidis, 2014)



C.3.2 Braking and acceleration forces

Acceleration forces N_{ac} and braking forces N_{br} may be approximated to static (usually distributed) loads developing during the acceleration and the braking of trains respectively (Lichtberger 2005).

More specifically, during acceleration, longitudinal forces build up in the track due to the static friction between the wheel(s) and the rail(s). Tension is generated in front of a driven railway axle; while, compression is generated behind it. The magnitude of these longitudinal forces depends on the vertical wheel load as well as on the coefficient of adhesion. Longitudinal forces not exceeding 5 % of the longitudinal thermal forces may be disregarded (Lichtberger 2005).

When the vehicles decelerate, compression tends to build in front of the first braking axle and tension behind it; this is the opposite of what happens during acceleration. Another difference, when comparing the above two cases is that, during braking, all axles are usually taking part in the process. Braking forces may reach to as much as 15 % of the maximum thermal forces. Indicatively, the values of braking forces for various vehicles are as follows (Lichtberger 2005):

- For electric traction units, they reach values equal to 12 %-15 % of the nominal axle load

- For diesel traction units, they reach values up to 18 % of the nominal axle load
- For two-axle freight wagons, they reach values up to 25 % of the nominal axle load

For many railway networks, braking and acceleration forces are considered to range between 25 % - 30 % of the nominal axle loads.

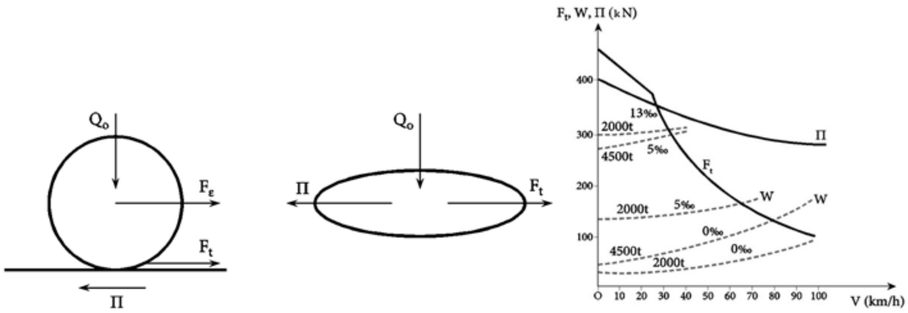
C.3.3 Traction or adhesion forces

If *W* the total train resistance, *Ft* the traction effort on the driving wheel treads and *Π* the adhesion force generated on the driving wheels' wheel/rail contact surface, the fundamental mathematical relationship, which must apply at all times in order to ensure train movement is:

$$\Pi > F_t > W \tag{55}$$

The force acting on the axles is called traction effort F_ε (Figure 13).

Figure 15 - Forces during train movement (Source: Pyrgidis, 1990)



The traction effort on the treads is not constant; it varies with the speed of the locomotive and generally reduces as the speed increases. Hence, we have the equation:

$$F_t = P_t \cdot V \tag{56}$$

Where P_t = power output (in kN).

The maximum traction effort on the treads develops at start up. The increase in speed initially results in the linear reduction of the traction effort on the treads and to its further reduction rapidly thereafter. However, the increase in speed also results in an increase of the train's movement resistance.

The traction effort on the treads is given by the manufacturer of the rolling stock, while the adhesion force is calculated either experimentally or with the help of empirical formulae found in the literature; the train's total motion resistance is calculated by a competent traction engineer based on network data.

The motor's traction effort $F\varepsilon$ cannot produce transportation work if it lacks a point of application (the contact patch); a certain amount of resistance develops at the contact patch (the adhesion force Π), this being defined as equation 57:

$$\Pi = Q_o \cdot \mu \tag{57}$$

Where Q_o = vertical load per wheel (kN) and μ = adhesion coefficient.

The adhesion force depends on many parameters, which may be classified in three categories:

- Manufacturing features of the vehicle and the track
- Construction materials of the wheel and the rail
- Environmental conditions

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