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Train–Bridge Interaction: Literature Review and Parameter Screening

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Cover photo: Stefan Trillkott, 2010. The Lögde älv bridge during test runs with the Green Train on the Bothnia Line.

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Abstract

New railway lines are continuously being constructed and existing lines are upgraded. Hence, there is a need for research directed towards efficient design of the supporting structures. Increasingly advanced calculation methods can be motivated, especially in projects where huge savings can be obtained from verifying that existing structures can safely support increased axle loads and higher speeds.

This thesis treats the dynamic response of bridges under freight and passenger train loads. The main focus is the idealisation of the train load and its implications for the evaluation of the vertical bridge deck acceleration. To ensure the running safety of train traffic at high speeds the European design codes set a limit on the vertical bridge deck acceleration. By considering the train–bridge interaction, that is, to model the train as rigid bodies on suspension units instead of constant moving forces, a reduction in bridge response can be obtained. The amount of reduction in bridge deck acceleration is typically between 5 and 20% for bridges with a span up to 30 m. The reduction can be higher for certain train–bridge systems and can be important also for bridge spans over 30 m. This thesis aims at clarifying for which system parameter combinations the effect of train–bridge interaction is important.

To this end, a thorough literature survey has been performed on studies in train–track–bridge dynamics. The governing parameters in 2D train–bridge systems have been further studied through a parameter screening procedure. The two-level factorial methodology was applied to study the effect of parameter variations as well as the joint effect from simultaneous changes in several parameters. The effect of the choice of load model was thus set in relation to the effect of other parameter variations.

The results show that resonance can arise from freight train traffic within realistic speed ranges (< 150 km/h). At these resonance peaks, the reduction in bridge response from a train–bridge interaction model can be considerable.

From the screening of key parameters it can furthermore be concluded that the amount of reduction obtained with a train–bridge interaction model depends on several system parameters, both for freight and passenger train loads. In line with the European design code's guidelines for dynamic assessment of bridges under passenger trains an additional amount of damping can be introduced as a simplified way of taking into account the reduction from train–bridge interaction. The amount of additional damping is today given as function of solely the bridge span length, which is a rough simplification. The work presented in this thesis supports the need for a refined definition of the additional damping.

Keywords: dynamics, vibration, railway bridge, bridge deck acceleration, moving load, train–bridge interaction, vehicle model.

Sammanfattning

Nya järnvägslinjer byggs kontinuerligt och befintliga linjer uppgraderas. Det finns därför ett behov av forskning inriktad på effektiv design av de bärande konstruktionerna. Alltmer avancerade beräkningsmetoder kan vara motiverade, särskilt i projekt där stora besparingar kan erhållas från att verifiera att befintliga konstruktioner kan bära ökade axellaster och högre hastigheter.

Föreliggande avhandling behandlar broars dynamiska respons under belastning av gods- och passagerartåg. Huvudfokus är att studera modelleringsalternativ för tåglasten och vilka konsekvenser de har för utvärderingen av brobanans vertikala acceleration. För att garantera trafiksäkerhet vid höga tåghastigheter definierar de europeiska normerna en maximalt tillåten vertikal acceleration i brobanan. Genom att beakta tåg-bro-interaktion, där tågkomponenterna modelleras som avfjädrade stela kroppar istället för konstanta punktlaster, kan en minskning av brons respons erhållas. Reduktionen av brobanans acceleration är typiskt mellan 5 och 20% för broar med en spännvidd på upp till 30 m. Minskningen kan vara högre för vissa tåg-brosystem och kan vara viktigt också för spännvidder över 30 m. Denna avhandling syftar till att klargöra för vilka kombinationer av tåg-broparametrar effekten av tåg-bro-interaktion är viktig.

I detta syfte har en omfattande litteraturstudie genomförts inom området tåg-spår-brodynamik. De styrande parametrarna i 2D tåg-brosystem har studerats vidare i en parameterstudie. Två-nivå faktorförsök har tillämpats för att studera effekten av parametervariationer samt den ytterligare effekten av samtidiga förändringar i flera parametrar. Effekten av valet av lastmodell sattes därmed i relation till effekten av andra parametervariationer.

Resultaten visar att resonans kan uppstå från godstrafik inom ett realistiskt hastighetsintervall (< 150 km/h). Vid dessa resonanstoppar kan en betydande minskning av broresponsen erhållas med en tåg-bro-interaktionsmodell.

Från studien av nyckelparametrar kan man vidare dra slutsatsen att reduktionen som erhålls med en tåg-bro-interaktionsmodell beror på flera systemparametrar, både för gods- och passargerartåg. Enligt de europeiska normernas rekommendationer för dynamisk kontroll av broar för passagerartrafik kan en ökad brodämpning introduceras som ett förenklat sätt att ta hänsyn till minskningen från tåg-bro-interaktion. Mängden tilläggsdämpning anges idag som en funktion av enbart brons spännvidd, vilket är en grov förenkling. Det arbete som presenteras i denna avhandling visar på behovet av en förbättrad definition av tilläggsdämpningen.

Nyckelord: dynamik, vibration, järnvägsbro, brobaneacceleration, rörliga laster, tåg-bro-interaktion, tågmodell.

Preface

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Stockholm, May 2014
Therese Arvidsson

Publications

This thesis is based on two journal papers, labelled paper I and II, and one conference proceeding, paper III.

Appended papers:

I Arvidsson, T., and Karoumi, R., "Train–bridge interaction — a review and discussion of key model parameters", accepted by *International Journal of Rail Transportation*, Feb 2014.

II Arvidsson, T., Karoumi, R. and Pacoste, C., 2014. "Statistical screening of modelling alternatives in train–bridge interaction systems". *Engineering Structures*, 59, 693–701.

III Arvidsson, T., and Karoumi, R., 2014. "Modelling alternatives in the dynamic interaction of freight trains and bridges". In *Proceedings of the Second International Conference on Railway Technology: Research, Development and Maintenance*, Ajaccio, Corsica, France, 8–11 April 2014.

Other Relevant Publications:

1) Andersson, A. (Ed.), Zangeneh Kamali, A., Elgazzar, H., Svedholm, C., Arvidsson, T., Ülker-Kaustell, M., Pacoste, C., Karoumi, R., and Battini, J-M., 2014. "Dynamic Assessment of Railway Bridges on the Bothnia Line Step 2: Simplified Soil–Structure Interaction, Train–Bridge Interaction and Load Distribution". TRITA-BKN, Rapport 150, KTH Royal Institute of Technology, Stockholm.

2) Johansson, C., Arvidsson, T., Martino, D., Solat Yavari, M., Andersson, A. (Ed.), Pacoste, C., and Karoumi, R., 2011. "Höghastighetsprojekt — Bro: Inventering av järnvägsbroar för ökad hastighet på befintliga banor", TRITA-BKN, Rapport 141, KTH Royal Institute of Technology, Stockholm.

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Part A

Introduction and general aspects

List of Abbreviations

ADM	Additional damping method
DOF	Degree of freedom
ERRI	European Rail Research Institute
HSLM	High-speed load model
MF	Moving force
MM	Moving mass
RB	Rigid beam
SIM	Simplified interaction model
TBI	Train–bridge interaction

Chapter 1

Introduction

Increasingly high demands are being placed on railway systems. The strategic plan for transportation within Europe (European Commission, 2011) outlines goals for increased freight traffic and aims at a well-developed high-speed rail network by 2050. The intention is that a majority of all medium-distance passenger traffic should be conducted by rail. At the same time, 50% of the road-based freight traffic should be shifted to alternative methods of transportation, such as rail and waterborne transportation. In Sweden, the Bothnia Line (Botniabanan) was completed in 2010 serving both passenger and freight traffic between a number of cities along the northern coastline. At the time of writing the project Ostlänken (the East link) is to be initialised – a high-speed passenger railway line between Järna and Linköping in southern Sweden. The project is intended to reduce the travel times and, moreover, to clear the existing lines from passenger traffic to enable increased freight traffic (Näringsdepartementet, 2012). Ostlänken is intended to form a part of Götalandsbanan – a future connection between Stockholm and Göteborg. In a later stage the Götalandsbanan may serve as a part of the European Corridor (Europakorridoren). The European corridor, which is still at a conceptual stage (Trafikverket, 2012; Näringsdepartementet, 2014), is intended to provide a high-speed connection to the European rail network; see Figure 1.1.

To meet the increased demands on existing railway lines as well as the urge to construct new lines, research on the railway infrastructure system is vital. With the limited space for new infrastructure, and in order to meet the high comfort requirements at high-speed lines, bridges or viaducts may very well form an increasing part of the railway infrastructure.

Figure 1.1: Conceptual sketch of the European corridor (Europakorridoren) including the Götalandsbanan between Stockholm and Göteborg. Reproduced from Europakorridoren (2014).

1.1 Background

High-speed rail traffic was first introduced in Europe through the Paris–Lyon line in 1981. Excessive vibrations were observed especially in short span bridges, which lead to rapid deterioration of track quality (Zacher and Baeßler, 2009). Bridge deck accelerations proved indeed to be an important parameter besides the dynamic amplification of deformations and stresses. Based on shake table tests undertaken in connection to the work of the European Rail Research Institute (ERRI) committee D214 (ERRI D214, 1999a) a bridge deck acceleration limit was set in the European design codes (Eurocode), CEN (2003). The limits are 3.5 m/s^2 and 5 m/s^2 for ballasted and non-ballasted track, respectively.

For speeds above 200 km/h the bridge deck acceleration and the dynamic amplification of deformations and stresses should generally be determined from a dynamic analysis. The acceleration limit is typically decisive in dynamic analyses (Zacher and Baeßler, 2009; Johansson et al., 2011), which is why research focused on vertical bridge deck acceleration is of interest. In the design of lines that do not carry high-speed traffic dynamic analyses are generally not required.

Freight-train-induced bridge vibration

Resonance is not typically expected from freight train traffic due to the low speeds. However, Majka and Hartnett (2009) point out that resonance may indeed arise within realistic speed ranges. The short axle distances, as compared to passenger trains, leads to lower resonance speeds (the resonance speeds v can be predicted by $v = d_{\mathrm{c}} \times f_0/k$, where d_{c} is the characteristic axle distance, f_0 is the fundamental bridge frequency and $k = 1, 2, 3, ...$). Especially in combination with low bridge fundamental frequencies, the first or second order resonance speeds may very well be within realistic speed ranges. Karoumi and Wiberg (2006) present results from dynamic simulations on a large number of bridges. The analyses show that the bridge deck acceleration is generally higher under the Swedish Steel Arrow train (see paper III) than under the Eurocode high speed load model (HSLM) in the speed range 50–120 km/h. In a number of cases the Steel Arrow train acceleration (50–120 km/h) is even higher than the HSLM acceleration at speeds up to 300 km/h.

Train–bridge interaction

A train–bridge interaction (TBI) model considers not only the dynamic properties of the bridge but also those of the train. Typically, the train subsystem is introduced by considering the train components as rigid masses connected by springs and dampers. These models make the evaluation of passenger comfort criteria possible. Furthermore, TBI models enable analyses of the train–bridge response at the presence of track irregularities, as well as the running safety of trains under cross winds or earthquake loads. In this thesis, our primary concern is the reduction in vertical bridge deck response that, for some train–bridge systems, is obtained from a TBI analysis as compared to a moving force (MF) analysis.

In keeping with the Eurocode (CEN, 2003), a certain amount of additional damping can be introduced in bridges of span 0–30 m to take into account the reduction in bridge response from TBI. In detailed analyses, for example in the upgrading of existing lines, it can be motivated to make the effort of conducting TBI analyses instead of adopting the simplified procedure available in the Eurocode. This thesis aims at exemplifying cases for which a TBI analysis can give different results (more or less reduction in bridge response) as compared to the additional damping in the Eurocode.

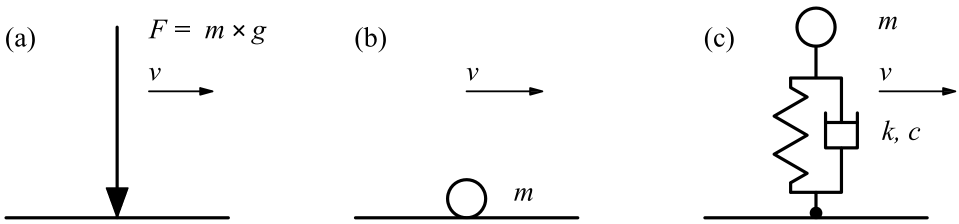


Figure 1.2: The moving force model (a), the moving mass model (b) and the sprung mass model (c).

1.2 Previous work

The very first work dealing with dynamics of railway bridges is said to be the account of experiments performed with rolling loads on girders by Willis (1849). The research was conducted in an enquiry prompted by the collapse of the Chester railway bridge in the United Kingdom. The experiments supported the hypothesis that the dynamic deflection can grow larger than the static one. In the late 19th and early 20th centuries the problem of a moving vehicle over a beam was analytically formulated using: (1) a moving mass over a massless beam by, e.g., Stokes (1849), (2) a moving force over a massive beam by, e.g., Timoshenko (1922), and (3) a moving mass over a massive beam by, e.g., Jeffcott (1929) and Inglis (1934). The latter two idealisations are depicted in Figure 1.2(a) and (b).

Hillerborg (1951), and later for example Biggs et al. (1959), introduced a model including the train suspension system, which thus constituted a simple TBI model; see Figure 1.2(c). From there on, models with up to 35 degrees of freedom (DOFs) for each 4-axle train carriage have been presented to simulate the vertical and lateral train–bridge dynamics as well as the running safety of trains; see for example Zhai et al. (2013a). The track system, modeled with a beam-mass-spring system or with solid elements, is typically included in such studies. Sinusoidal or stochastic uncertainties can be introduced to the rail or the bridge surface to take into account the additional excitation from the unsprung wheel masses traversing an imperfect track.

Simple TBI models, such as the 2D rigid beam (RB) model and the simplified interaction model (SIM) on simply supported beams, Figure 2.1(a) and (b), have been used to study the governing parameters in the TBI system. A thorough literature review of works on TBI is presented in paper I.

1.3 Aims and scope

The overall aim of this licentiate project has been to study the effect of TBI on the bridge response, with particular focus on the vertical bridge deck acceleration. More precisely, the aim was to increase the knowledge of the effect of TBI under variations in the key system parameters. A specific aim has been to identify relevant idealisations and possible simplifications for 2D vehicle models for both passenger and freight train loads. Another specific aim was to identify which bridges and span lengths that are particularly sensitive to heavy freight trains.

In line with the overall aim of the research, one of the main objectives has been to present a literature survey on bridge, train and track dynamics.

The following limitations apply to the research work, though not necessarily to the scope of the literature review:

- The additional excitation originating from track irregularities has not been considered in the models. The reader is referred to paper I, sections 6 and 9, for a discussion on track irregularities.
- The data for the bridges in papers II and III is based on statistics of concrete bridges within the Swedish bridge stock. Many of these bridges are single-track bridges or two-track bridges composed of adjacent bridge decks separated by a joint. Therefore, the parameters are representative for single-track bridges.
- 2D models have been used, which enables the study of vertical train and bridge responses while omitting lateral and torsional responses and 3D effects. This does not pose a problem in the analysis of single-track bridges.
- No comparison with measured data has been performed within the scope of this work.

1.4 Research contribution

The literature survey and the theoretical studies presented in this thesis have resulted in the following scientific contributions:

- A summary of conclusions from the vast number of studies on TBI available in the literature has been provided through the comprehensive literature review.

A rough guide to modelling choices concerning the train load and the track system is given as part of the conclusions in the literature review (paper I, Section 10).

- A two-level factorial method was adopted (papers II and III) to study the effect of modeling choices and uncertainties in key train–bridge system parameters. Hence, it was exemplified for which cases the effect of TBI is important relative to variations in, for example, bridge mass, stiffness and damping.

- Cases have been identified for which reduced bridge response can be obtained by performing a TBI analysis instead of adopting the Eurocode additional damping (paper II). Moreover, for portal frame bridges, as an example, it was identified that the additional damping is not always conservative and should thus be used with caution (Chapter 5).

- The aforementioned theoretical studies, together with the literature survey, have brought attention to the fact that the additional damping in the Eurocode is a rough simplification that should be enhanced and also developed for other bridge types besides simply supported bridges.

1.5 Outline of the thesis

This thesis consists of two parts of which Part A provides an extended summary of the work presented in the papers appended in Part B. Part A Chapter 1 gives an introduction and some general aspects to demonstrate the relation between the appended papers. Chapter 2 presents some details on the TBI modelling used in the present work and the procedure used for parametric studies. Chapter 3 treats the two-level factorial experiment methodology adopted in the appended papers II and III. Chapter 4 provides an extension of the discussion on the governing parameters of TBI systems that is presented in paper I. Chapter 5 presents TBI analyses on bridges along the Bothnia Line (Botniabanan) performed within a project for the Swedish Transport Administration (Trafikverket). Conclusions and a discussion of further research are given in Chapter 6.

As illustrated in Figure 1.3, the literature review (paper I) treats various subjects within bridge dynamic analyses and TBI. The parameters governing the amount of reduction obtained from TBI is thoroughly discussed. This discussion in the staring point for papers II and III which focus on the key parameters in the TBI system for passenger trains and freight trains, respectively. A description of each appended paper is as follows:

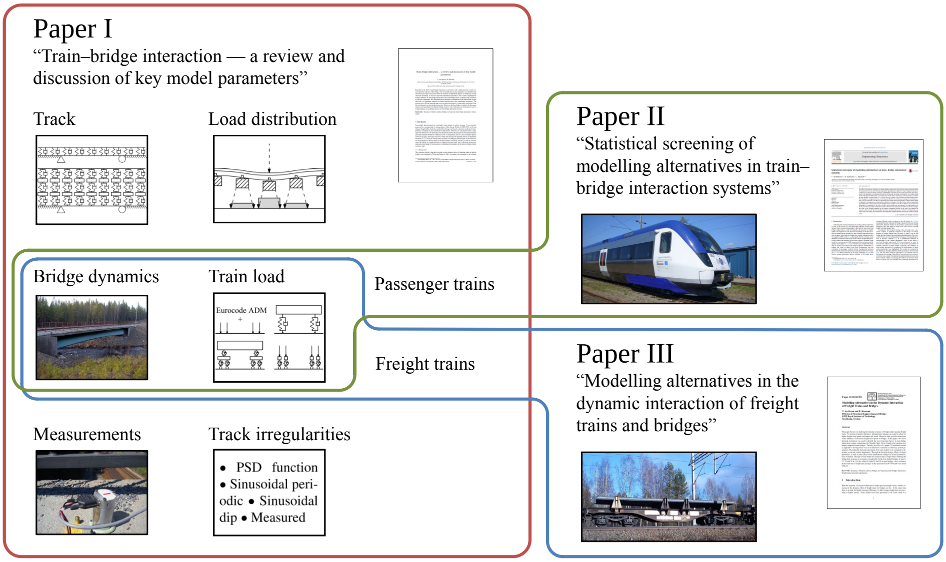


Figure 1.3: Schematic illustration of the relation between papers I–III.

Paper I presents a literature review with a particular focus on TBI models for the evaluation of vertical bridge deck acceleration. The review is complemented by numerical examples comparing different TBI models. Furthermore, general aspects of dynamic analysis of bridges are treated, as well as track models and the effect of track irregularities. TBI models that have been verified with field measurements are also briefly discussed.

Paper II provides a screening of key parameters in the dynamic analysis of beam bridges subjected to passenger train loads (the Swedish Green Train). A two-level factorial experiment is applied to highlight the relative influence of TBI as compared to variations in other key parameters: bridge stiffness, bridge mass, bridge damping ratio, bridge rotational support stiffness and train axle load. Bridge deck vertical acceleration and displacement are studied.

Paper III applies the same methodology as is used in paper II to study freight-train-induced bridge vibration (the Swedish Steel Arrow train). The amount of reduction obtained using a TBI model as compared to an MF model is set in relation to the effect of variations in bridge stiffness, bridge mass and bridge damping ratio.

Chapter 2

Modelling of the TBI system

A TBI analysis requires the solution of two coupled systems of equations: the train subsystem and the bridge subsystem. The two systems can be regarded as coupled via the dynamic interaction forces, which depend on time as the vehicles moves over the bridge, as well as on the bridge and vehicle displacements (not known at the outset).

Through a formulation of the interaction force, the system can be transformed into two uncoupled equations and solved iteratively by enforcing self-consistency between the bridge and vehicle at each time step. Alternatively, the iterations are performed on the system level; the two subsystems are solved in turns for the whole time sequence until convergence is reached for the interaction forces. A finite element (FE) representation of the system yields the following discrete equations of motion for the train–bridge system (Yang and Fonder, 1996; Guo et al., 2012):

$$\mathbf{M}_{\mathrm{b}}\ddot{u}_{\mathrm{b}} + \mathbf{C}_{\mathrm{b}}\dot{u}_{\mathrm{b}} + \mathbf{K}_{\mathrm{b}}u_{\mathrm{b}} = f_{\mathrm{b}} \tag{2.1a}$$

$$\mathbf{M}_{\mathrm{v}}\ddot{u}_{\mathrm{v}} + \mathbf{C}_{\mathrm{v}}\dot{u}_{\mathrm{v}} + \mathbf{K}_{\mathrm{v}}u_{\mathrm{v}} = f_{\mathrm{v}} \tag{2.1b}$$

where subindex b refers to the bridge system and v refers to the vehicle system, $\mathbf{M}$, $\mathbf{C}$ and $\mathbf{K}$ are the mass, damping and stiffness matrices, respectively, and u is the displacement vector. The bridge force vector, f_{b}, is composed of the static (gravity) load from the vehicle as well as the dynamic interaction forces that depend on both the bridge and vehicle motions, as well as time, t. The vehicle force vector, f_{v}, contains the dynamic interaction forces.

The same system can be described with a coupled equation of motion. The mass, damping and stiffness matrices are then time-dependent as the vehicles move over the bridge and has to be updated at each time step. With the assumption that

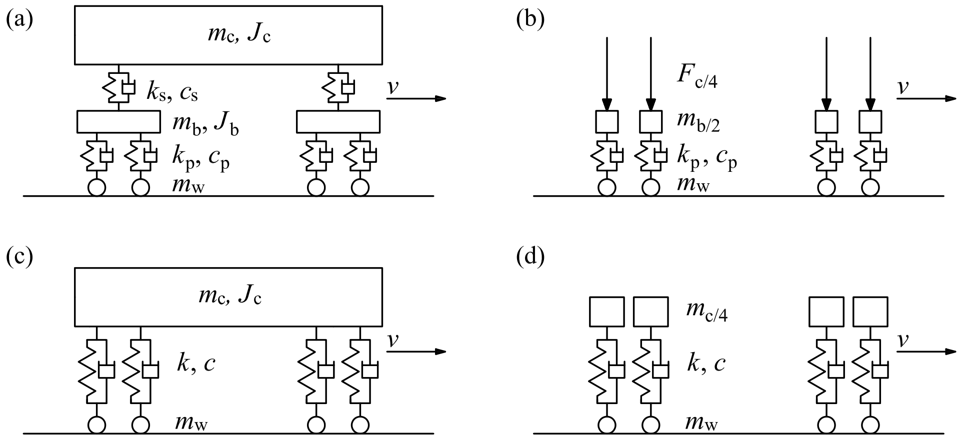


Figure 2.1: The two-layer suspension RB model (a), the SIM (b), the single-layer suspension RB model (c) and the single-layer suspension quarter car model (d), where $F_{c/4}$ is the constant load of the car body; m_c, m_b and m_w are the mass of the car, bogie and wheel, respectively; J_c and J_b are the car and bogie inertia; k_p and c_p, k_s and c_s are the spring stiffness and damping representing the primary and secondary suspension system; finally, k and c are the combined stiffness and damping of the primary and secondary suspension systems.

no external force acts on the vehicle, track irregularities are neglected and that the wheels always remain in contact with the beam, the coupled equation of motion can be written on the form:

$$\begin{bmatrix} \mathbf{M}_{\mathrm{b}} + \mathbf{M}_{\mathrm{v,w}} & 0 \\ 0 & \mathbf{M}_{\mathrm{v,u}} \end{bmatrix} \begin{bmatrix} \ddot{u}_{\mathrm{b}} \\ \ddot{u}_{\mathrm{v}} \end{bmatrix} + \begin{bmatrix} \mathbf{C}_{\mathrm{b}} + \mathbf{C}_{\mathrm{v,w}} & \mathbf{C}_{\mathrm{bv}} \\ \mathbf{C}_{\mathrm{vb}} & \mathbf{C}_{\mathrm{v,u}} \end{bmatrix} \begin{bmatrix} \dot{u}_{\mathrm{b}} \\ \dot{u}_{\mathrm{v}} \end{bmatrix} + \begin{bmatrix} \mathbf{K}_{\mathrm{b}} + \mathbf{K}_{\mathrm{v,w}} & \mathbf{K}_{\mathrm{bv}} \\ \mathbf{K}_{\mathrm{vb}} & \mathbf{K}_{\mathrm{v,u}} \end{bmatrix} \begin{bmatrix} u_{\mathrm{b}} \\ u_{\mathrm{v}} \end{bmatrix} = \begin{bmatrix} f_{\mathrm{grav}} \\ 0 \end{bmatrix} \tag{2.2}$$

where subindex b refers to the bridge system and v refers to the vehicle system consisting of the wheel DOF:s, w, in contact with the bridge and the upper DOF:s of the vehicle, u, not in contact with the bridge. The force on the bridge system, f_{grav}, is the static (gravity) load from the vehicles on the bridge. The submatrices for different beam and vehicle idealisations can be found in, for example, Olsson (1985); Lin and Trethewey (1990); Xia et al. (2000); Au et al. (2001); Museros et al. (2002); and Song et al. (2003).

The assumption of constant contact between the train and the bridge (the "no loss of contact" assumption) is commonly adopted. Particularly for running stability analyses involving lateral dynamics, more advanced contact theories have been used. Common approaches implement Hertz contact for the normal contact and Kalker creep theory for the lateral contact (Zhai et al., 2013b; Goicolea and Antolín, 2012; Romero et al., 2013). In such studies 3D vehicle models are widely adopted despite their computational expense. Important applications include TBI systems under wind and earthquake load.

Considering the vertical bridge response, 2D models are sufficient as long as no other motives for implementing a 3D model are present, such as eccentrically placed track or other important 3D bridge behaviour. The models outlined in Figure 2.1 have been implemented in the present research. Out of these models only model (a), the two-layer suspension rigid beam (RB) model, represent both the car body–bridge and the bogie–bridge interaction. Model (b), the simplified interaction model (SIM), neglects the car body–bridge interaction. The SIM is a relevant idealisation for passenger trains with a two-level suspension system that partly decouples the car body from the dynamic interaction (ERRI D214, 1999c; Museros and Alarcón, 2002; Liu et al., 2009; Goicolea and Antolín, 2012). Model (c) ignores the bogie–bridge interaction as it represents the car body as a rigid beam on a single-layer suspension system. The single-layer RB model is in paper III proved to be a rather adequate model for a freight train model with a stiff secondary suspension system. Model (d), the quarter car model, further neglects the coupling between the axles.

2.1 Modelling details in Abaqus

In this thesis the dynamic interaction between the train and the bridge has been implemented in the commercial FE software Abaqus (Dassault Systèmes, 2011a). Train models travelling over Euler-Bernoulli beams were considered in 2D. Some details on the modelling are as follows:

The train models in Figure 2.1 have been modeled by point masses and rigid beam elements connected by linear spring and dashpot elements. The gravity load from the vehicles was applied in a static step, preceding the dynamic analysis. It is worth noting that, in Abaqus, the dashpots introduce damping in static steps. The amount of damping is determined by the "velocity" at which the load is applied, that is, the displacement divided by the total time during which the load is applied (Dassault Systèmes, 2011a). For the gravity step, a large static time step minimises the effect of damping.

The movement of the train across the beam was enforced by a displacement boundary condition for one of the vehicle nodes (the "vehicle reference node") in each carriage (or in each axle in the decoupled models). An amplitude function for the boundary condition specifies the movement of each carriage or axle. The remaining longitudinal DOFs were constrained to that of the "vehicle reference node" using equation constraints.

The contact between the beam and the vehicle wheel nodes was realised through a surface-to-surface contact definition. The beam acted as a master surface and the vehicle wheel nodes as slave nodes. The interaction was defined using large (finite) sliding, frictionless tangential behaviour, and a "hard" pressure-overclosure contact property, enforced by the penalty method (Dassault Systèmes, 2011a; Saleeb and Kumar, 2011; Martino, 2011). The procedure for solving the TBI system is hence similar to the formulation in Equation 2.1 with iterations for compatibility in each time step.

A no loss of contact condition was enforced throughout the analysis. This infers that the contact forces have to be checked so that they are positive at all times, otherwise separation should occur. The model would obviously not be able to describe the loss and re-establishment of contact that can occur under severe track irregularities (ERRI D214, 1999b; Li et al., 2010; Saleeb and Kumar, 2011; Ang and Dai, 2013; Zhai et al., 2013b).

The default direct integration scheme in ABAQUS was used, which is a Hilber-Hughes-Taylor method with numerical damping. As a direct integration method was used the modal damping ratios were approximated with a Rayleigh damping. The Rayleigh coefficients were chosen to give a damping close to the desired one over an interval containing the most important bridge modes. The bounds of the interval were chosen as the first mode and the second or the third mode.

2.2 Numerical example I: moving sprung mass

The implementation in Abaqus was verified against a benchmark that can be found in Yang and Wu (2001), among others. The problem with a simply supported beam subjected to a moving sprung mass is outlined in Figure 2.2 together with the associated vehicle and beam data. No damping is considered. In the present analysis, a time step of 0.005 s was used, and the beam was divided into 50 Euler-Bernoulli beam elements. For comparison, also the MF model was adopted.

As shown in Figure 2.3, the agreement with Yang and Wu (2001) is good. Although not shown in the figures, the sprung mass accelerations are also in good agreement. The omission of higher modes in the analytical solution from Biggs (1964), also included in Figure 2.3, is most apparent in the acceleration results.

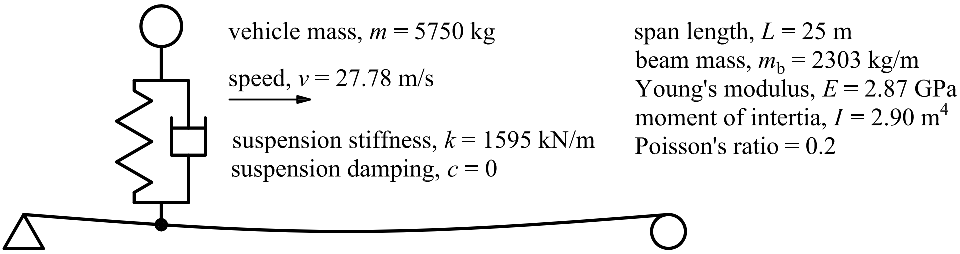


Figure 2.2: The example I sprung mass model.

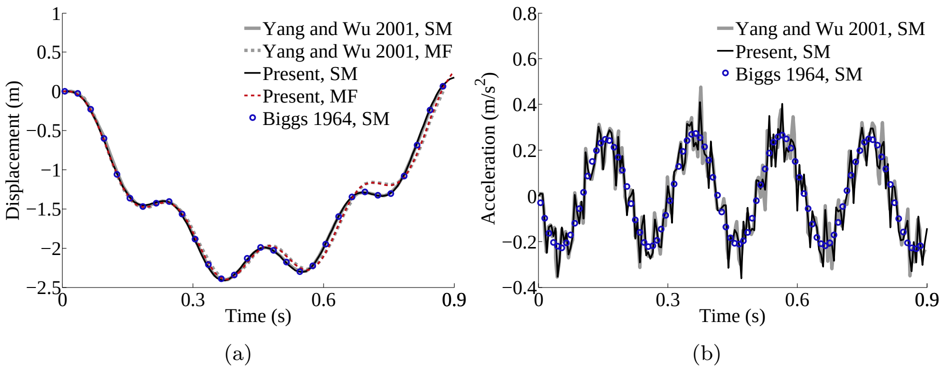


Figure 2.3: Example I: beam mid-span displacement (a) and acceleration (b).

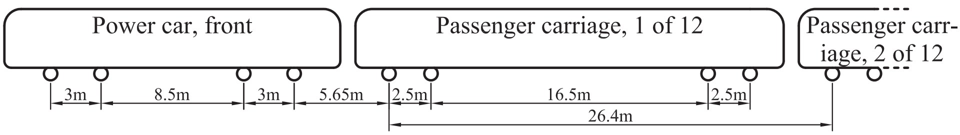


Figure 2.4: The ICE 2 train with one leading and one trailing power car and 12 passenger carriages. Axle distances are as indicated, with a characteristic carriage distance of 26.4 m for the passenger carriages.

2.3 Numerical example II: ERRI D214 results

As a second benchmark, the implementation in Abaqus is compared to results from ERRI D214 (1999c). The ERRI committee D214 results are part of the calculations that lie behind the additional damping, $\Delta\zeta$, in the Eurocode (CEN, 2003). As mentioned, the additional damping is used as a simplified way of taking into account

Table 2.1: Properties of the ICE 2 train model, from ERRI D214 (1999c).

Item	Power car	Carriage
Axle load, F (kN)	196	112
Car mass, m_c (kg)	60,768	33,930
Car inertia, J_c (kg m^2)	1.344×10^6	2.115×10^6
Bogie mass, m_b (kg)	5600	2373
Bogie inertia, J_b (kg m^2)	21,840	1832
Wheel set mass, m_w (kg)	2003	1728
Vertical stiffness, primary suspension, per axle box, k_p (kN/m)	4800	1600
Vertical damping, primary suspension, per axle box, c_p (kNs/m)	108	20
Vertical stiffness, secondary suspension, per bogie side, k_s (kN/m)	1760	300
Vertical damping, secondary suspension, per bogie side, c_s (kNs/m)	152	6

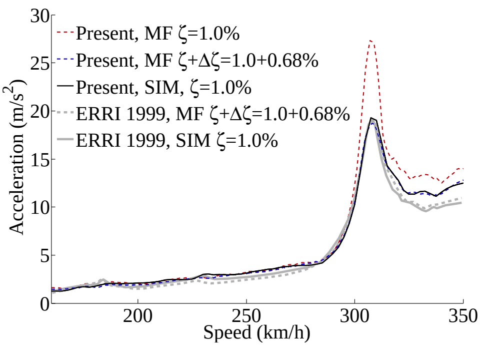


Figure 2.5: Example II: beam mid-span acceleration under the ICE 2 train.

the reduction in bridge response obtained from a TBI model. A fictitious additional damping was introduced in an MF model, which was then calibrated against the SIM (see Figure 2.1(b)).

Out of the available results, a 10-m span beam with EI = 3,803,120 kNm^2, a distributed mass of 10,000 kg/m and damping ratio 1% was chosen for the comparison. The fundamental frequency of the beam was 9.7 Hz. The train load was the ICE 2 train with properties according to Figure 2.4 and Table 2.1. Additional assumptions are described in Section 5.2.

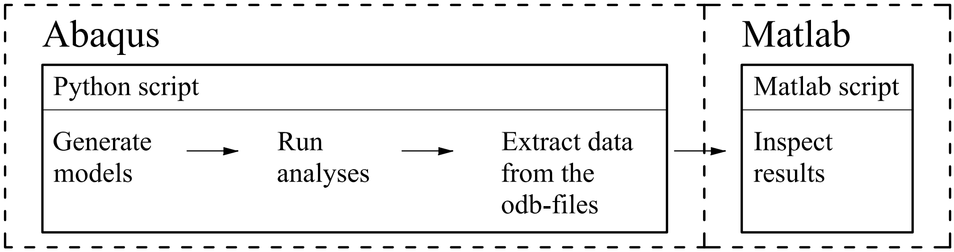


Figure 2.6: The procedure used to build models and perform TBI simulations in Abaqus, and to thereafter extract output data for processing in Matlab.

As seen in Figure 2.5, the results from the SIM in the present implementation in Abaqus tally well with those of the ERRI committee D214. The discrepancies between the results that is seen above 320 km/h can partly be explained by the omission of higher order modes in the ERRI committee D214 analyses.

2.4 Scripting

The implementation in Abaqus as described in this chapter was used to perform the parameter screening presented in papers II and III. In order to perform the 640–5440 TBI analyses for each bridge the analysis procedure was scripted. The Python scripting (van Rossum, 2008) capabilities within Abaqus were used. Guidelines on how to control Abaqus using Python scripts can be found in the Abaqus scripting manual (Dassault Systèmes, 2011b). The models and the corresponding input files were first generated using a Python script, which also generated a batch script to subsequently perform the analyses in Abaqus from the system command prompt. In this way, analyses could be performed with systematic variations in the system parameters for a range of speeds. The extraction of output data from the Abaqus output database (odb-) file was automated via another Python script. The analysis results could then be processed in Matlab (The MathWorks, Inc., 2012). The analysis procedure is outlined in the flow chart presented in Figure 2.6.

Chapter 3

Two-level factorial experimentation

The two-level factorial experiment is a parameter screening procedure that is used to identify the most influential parameters in a system. The procedure differs from ordinary one-factor-at-a-time methods in that the parameters are systematically varied simultaneously, allowing for the identification of interaction effects. The two-level (2^n) factorial design is performed for two levels of n factors and requires 2^n runs. The factors can be quantitative or qualitative (on-off factors).

The main effect of a factor is defined as the effect of a change in the parameter from its low to its high value. It is thus evaluated as the difference in result between all runs with the factor at its low value and all runs with the factor at its high value.

The interaction effect is the joint effect from simultaneous changes in several factors. Consider an experiment with three factors: A, B and C. The two-parameter interaction between factor A and factor B (A*B) is evaluated as the difference between the effect of factor A at the low and high level of factor B. The interaction effects are symmetrical so that the same interaction effect can be evaluated as the difference between the effect of factor B at the low and high level of factor A. The three-parameter interaction A*B*C is evaluated as the difference in the A*B interaction effect at the low and high level of factor C, or vice versa according to symmetry.

The results of all runs in the experiment can thus be described as a linear combination of main and interaction effects, together with an error term and a replication term. A model equation can be formulated according to Johnson et al. (2011):

$$\begin{aligned} Y_{ijkl} &= \mu + \alpha_i + \beta_j + \gamma_k + (\alpha\beta)_{ij} + (\alpha\gamma)_{ik} + (\beta\gamma)_{jk} \\ &\quad + (\alpha\beta\gamma)_{ijk} + \rho_l + \varepsilon_{ijkl}, \end{aligned} \tag{3.1}$$

where Y_{ijkl} is the estimated response for observation y_{ijkl} which is the l:th replication with factor A at level i, factor B at level j and factor C at level k. In the equation, μ is the grand mean, α_i, β_j, γ_k are the estimated effect of factor A, B and C, respectively, at level i, j and k. The estimated parameter interaction effect of the ith level of factor A and the jth level of factor B is given by $(\alpha\beta)_{ij}$, and so forth. The effect of the replicate l is given by ρ, and ε_{ijkl} are the error terms which are assumed to be random variables having a normal distribution with zero mean and variance σ^2. Furthermore, the effects are restricted to the conditions $\alpha_1 = -\alpha_0$, $\beta_1 = -\beta_0$, $\gamma_1 = -\gamma_0$. Equally, the sums of the two- and three-way interaction term effects are assumed zero, as well as the sums of the replication effects. As can be realised from Equation (3.1), the change in the mean response μ from a change from level 0 to level 1 in one term corresponds to two times its estimated effect α_i, β_j, γ_k, $(\alpha\beta)_{ij}$, ..., which is what we above referred to as the effect of that term.

The concept of the two-level factorial experiment is well described in the book by Box et al. (2005). Some practical guidelines on the choice of factor levels are given by Mee (2009).

3.1 Limitations

The major limitation of the two-level factorial experiment is that the model describes only linear variations. Non-linearities in the variation between two levels of a factor will not be detected. This infers a restriction on how to choose the high and low value for the factors: preferably, the chosen response variable should be reasonably linear for changes in the quantitative factors within the range of the experiment. This restriction does not apply for the qualitative factors as the nature of the on-off concept is that the variation in between the levels is not defined. An example of a qualitative factor is the choice of catalyst A or catalyst B in a chemical experiment. The amount of catalyst, on the other hand, would be a quantitative factor.

Another limitation with the present implementation of the two-factorial concept is to determine which effects are significant. Normally, in a replicated experiment the significance of each factor would be estimated based on the error term. The estimation can be based on the standard error as in Box et al. (2005) or on the ANOVA concept as described in Johnson et al. (2011). Box et al. (2005) argue strongly against the use of ANOVA for the two-factorial screening procedure as the decisions are taken too mechanically based on a certain level of probability. In the present application, the experiments are performed without replication as the FE model produces identical results for unchanged input data. Then an estimation of the error can be obtained by pooling a number of insignificant high-order interaction

Table 3.1: Numerical example: influence on the response Y_{ij} (beam fundamental frequency in Hz) from factors S (beam stiffness) and M (beam mass).

Level i of factor S	Level j of factor M	Response Y_{ij}
0	0	26.74
1	0	29.29
0	1	25.49
1	1	27.92

effects. This error estimate could be used in the same way as an error estimate based on replications. The graphical method by Daniel (1959) provides means for determining which high-order interaction effects that are insignificant and can be pooled, as well as a visual way of determining which effects that are significant. According to Daniel's method the two-level factorial effects are plotted on a normal probability plot. Insignificant effects then form a straight line whereas significant effects deviate from the straight line. Examples can be found in papers II and III where half-normal score plots are used to determine which factors that are important.

It should furthermore be noted that the two-level factorial procedure identifies interactions whenever factor effects are not additive. In applications where we expect multiplicative behaviour (i.e. we are interested in effects as a percentage of the response) the parameter interactions may be distorted when large main effects are present. If one main effect (say in a factor A) is large compared to the mean response, μ, then the effect of the other factors will, in absolute numbers, be larger at one level of factor A than at the other level of factor A even if their percentage effects are equal at both levels of factor A. A logarithmic transformation makes effects that are multiplicative on the raw scale additive on the logarithmic scale (Mee, 2009). Box et al. (2005) point out that a logarithmic transformation affects the result if the ratio between the largest and the smallest observation is large. This issue is briefly discussed in paper III in connection to main effects of over 50% of the mean response. In the paper, no spurious interactions were seen in the results but relevant interactions were overshadowed by the scale effect from one large main effect. A procedure to further examine the results and identify these interactions was adopted.

3.2 Numerical example

Table 3.1 presents an example of a two-level factorial experiment with two factors. The influence of factors S (stiffness) and M (mass) on the fundamental frequency

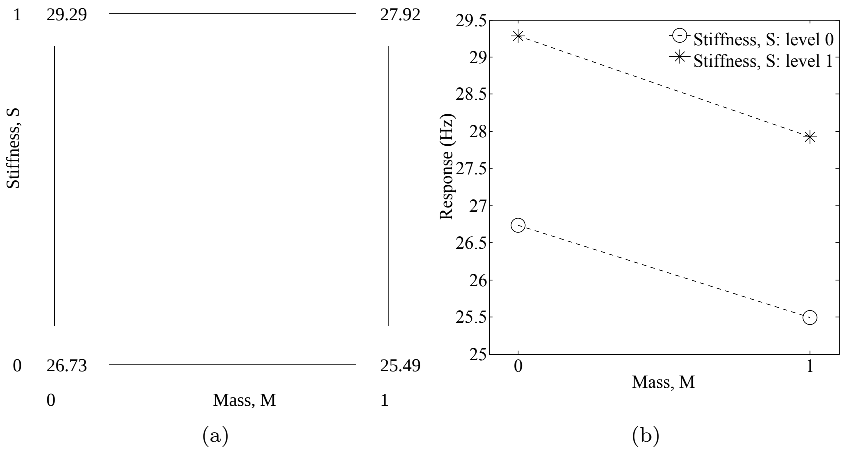


Figure 3.1: Two-way diagram (a) and marginal mean plot (b) for the results in Table 3.1.

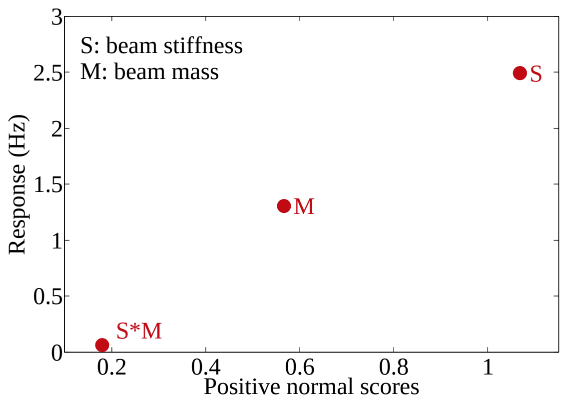


Figure 3.2: Normal score plot of the factor effects calculated from the results in Table 3.1.

of a 6-m span beam is evaluated. The level one stiffness value represents a 20% increase in beam stiffness from the level 0 value. A 10% increase was considered for the beam mass. Four runs are needed in an unreplicated 2^2 factorial to evaluate the main effects S and M, a well as the interaction S*M. From the results of the four runs the effect of factor S is evaluated as the difference between the average response at its high and its low level:

$$\begin{aligned} S &= \frac{Y_{10}+Y_{11}}{2} - \frac{Y_{00}+Y_{01}}{2} \\ &= \frac{29.29+27.92}{2} - \frac{26.74+25.49}{2} = 2.49. \end{aligned} \tag{3.2}$$

An increased beam stiffness increases the fundamental frequency. Similarly, the effect of M is:

$$\begin{aligned} M &= \frac{Y_{01}+Y_{11}}{2} - \frac{Y_{00}+Y_{10}}{2} \\ &= \frac{25.49+27.92}{2} - \frac{26.74+29.29}{2} = -1.31. \end{aligned} \tag{3.3}$$

An increase in mass reduces the fundamental beam frequency. The parameter interaction effect S*M is half the difference in effect of S at the high and the low level of M, or vice versa:

$$\begin{aligned} S*M &= \frac{(Y_{11}-Y_{01})-(Y_{10}-Y_{00})}{2} \\ &= \frac{(27.92-25.49)-(29.29-26.74)}{2} = -0.06. \end{aligned} \tag{3.4}$$

The response Y_{ij} can now be described by inserting the factor effects into Equation 3.1:

$$\begin{aligned} Y_{ij} &= \mu + S_i/2 + M_j/2 + (SM)_{ij}/2 \\ &= 27.36 + (2.49/2)x_1 + (-1.31/2)x_2 + (-0.06/2)x_1x_2, \end{aligned} \tag{3.5}$$

where x_1 and x_2 take the value +1 or −1 for level 1 or level 0 of i and j, respectively. The error and replication terms from Equation 3.1 are absent as the experiment was performed without replication.

The data is visualised in the two-way diagram in Figure 3.1(a). The interaction term S*M is, as expected, very small, which can be seen also from the marginal mean plot in Figure 3.1(b) as the lines are practically parallel. The normal score plot is given in Figure 3.2, although this kind of plot is more interesting when a larger number of factors are considered (see for example Figure 6 in paper II).

3.3 Comments on the application to train-induced bridge vibration

The factorial experimental procedure can be applied to evaluate the influence of different factors on the dynamic response of bridges due to passing trains; this is

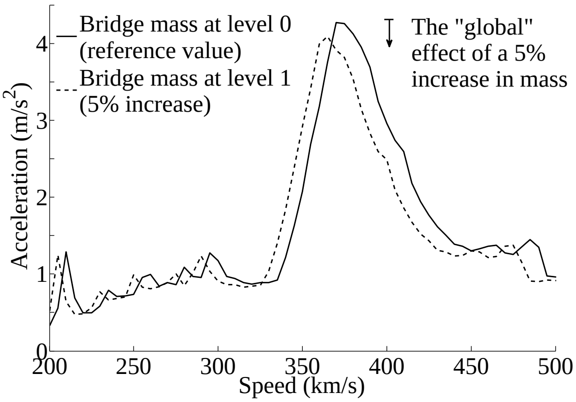


Figure 3.3: The effect of a 5% increase in bridge deck mass on the bridge deck acceleration under a train passage.

done in papers II and III. In this kind of application, the influence of the train speed must be recognised. Some factor variations (such as bridge stiffness and bridge mass) affect the natural frequencies of the bridge and thus the resonance speeds. We typically want to differentiate the effect of the change in resonance speed from the overall or "global" effect of a factor. Figure 3.3 illustrates this issue: an increase in mass alters the resonance speed so that the response at a certain speed increases or decreases depending on which speed we choose to look at. However, the overall, "global", effect of the 5% increase in mass is a 5% decrease in acceleration. To detect this effect a speed range must be chosen that covers the resonance peak for both factor combinations.

Chapter 4

Discussion on the TBI system parameters

TBI models for passenger trains passing simply supported bridges has been studied by for example Olsson (1983); Dahlberg (1984); Museros (2002); Museros and Alarcón (2002); Majka and Hartnett (2008); Liu et al. (2009); and Doménech and Museros (2011). The amount of reduction obtained from a TBI model as compared to an MF model has been shown mainly governed by: the bogie–bridge frequency ratio, the bogie–bridge mass ratio and the bridge–carriage length ratio. The reduction from TBI is largest at high mass ratios and, for realistic ranges of train and bridge parameters, at a bogie–bridge frequency ratio of 1.0–1.5. The frequency ratio is important as the train components interact with the bridge only if their frequencies are reasonably close to each other. For passenger trains the bogie–bridge interaction is most important as the train suspension system is typically designed to isolate the car body from vibrations. The mass ratio is important as the effect of interaction is notable only if the mass of the train components is considerable compared to the bridge mass. Finally, Museros (2002) showed that the relation between the bridge length and the carriage length directly affects the amount of reduction, and that it moreover has an indirect effect as the bridge length decides realistic ranges for the frequency and mass ratios. The reader is referred to Section 4 in paper I (including Figures 9 and 10) for further details. In this section a discussion is developed on a number of other important factors and modeling issues arising from these relations.

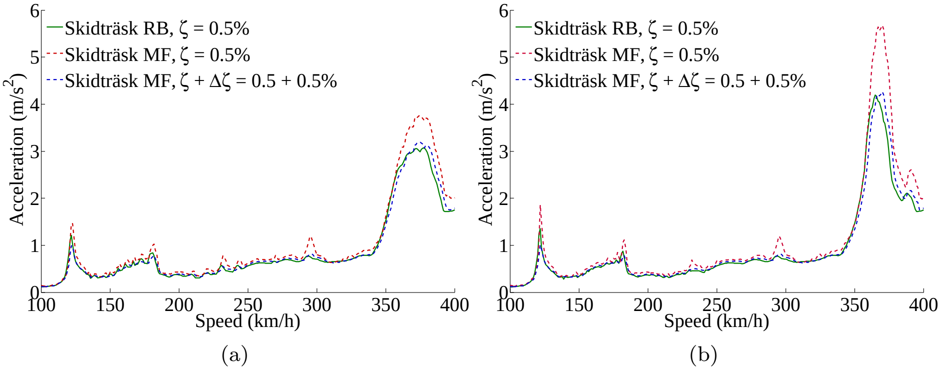


Figure 4.1: Bridge deck mid-span acceleration for the Skidträsk bridge under the ICE 2 train with (a) 12 carriages and 2 power cars, and (b) with 24 carriages and 2 power cars.

4.1 Train type

From the fact that the mass ratio, frequency ratio and length ratio are important follows obviously that different trains will yield reductions from TBI of different magnitudes. The load effect from the ICE 2 train (see Figure 2.4 and Table 2.1) as compared to the Swedish Green Train (used in paper II) on the Skidträsk bridge may serve as an example. The Skidträsk bridge is a 36-m single-track, simply supported, steel-concrete composite bridge in Sweden. The 2D beam model of the bridge has a fundamental frequency of 3.86 Hz; see paper II for details. The amount of reduction obtained from a TBI analysis as compared to an MF analysis was in paper II found to be around 30% at resonance under the Green Train load. Figure 4.1(a) shows that the reduction in bridge deck acceleration is around 20% at the resonance peak at 370 km/h for the ICE 2 train. The fact that the reduction is lower under the ICE 2 load can possibly be explained by: (1) the ICE 2 train has a different set of train characteristics resulting in a different bogie–bridge frequency ratio, and (2) the ICE 2 train bogies are lighter than the powered bogies of the Green Train resulting in a lower bogie–bridge mass ratio.

An MF model with additional damping was manually fitted to the results of the two-layer suspension RB model in Figure 4.1; an additional damping of 0.5% was found reasonable. Using the same procedure, the estimation of additional damping from the Green Train results would instead be 1.0%. In the Eurocode, the amount of additional damping is given as a function of span length and no additional damping is given to bridges with a span above 30 m.

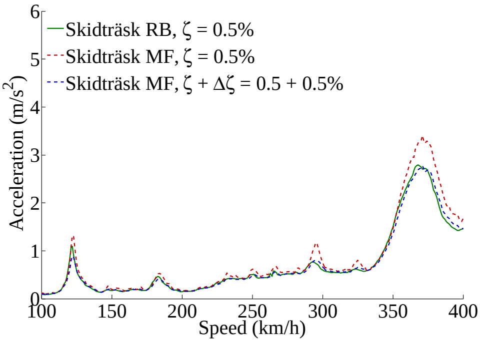


Figure 4.2: Bridge deck mid-span acceleration for the Skidträsk bridge under a theoretical 12-carriage ICE 2 train set without the two power cars.

4.2 Train length

The amount of reduction from TBI increase with the number of regularly spaced axles. This is exemplified in Figure 4.1 which shows the bridge deck acceleration under (a) an ICE 2 train set with 12 carriages and 2 power cars, and (b) a theoretical ICE 2 train set with 24 carriages and 2 power cars. The difference between the MF model and the RB model is in (a) 20%, and in (b) 25%, at the resonance peak at 370 km/h.

An MF model with additional damping was manually fitted to the results of the RB model in Figure 4.1. For both the 12+2-carriage and the 24+2-carriage trains an additional damping of 0.5% was found reasonable. Based on these results the additional damping describes the effect of TBI very well in this particular respect: both the effect of an additional amount of damping and the effect of TBI grows larger with more pronounced bridge resonances.

4.3 Train set configuration

Figure 4.2 shows the bridge deck acceleration for the Skidträsk bridge under a theoretical ICE 2 train set without the two power cars. Based on the comparison between Figure 4.2 and Figure 4.1(a) the amount of reduction from TBI seems to be similar for train sets with and without power cars. This may very well be because the response at resonance in both cases is governed by the repetitive carriage axle loads. An additional damping of 0.5% was found reasonable also based on the results from the train set without power cars.

4.4 Type of resonance

According to the author's knowledge no studies have been performed to reveal whether the reduction from TBI is the same at the primary resonance peak as at the secondary or higher order resonance peaks (i.e. $v = d_{\mathrm{c}} \times f_0 \times 3.6/k$, for $k \geq 2$). ERRI D214 (1999c) studied the reduction at the highest resonance peak within the interval 150–350 km/h. A similar approach was used by the author in papers II and III. For short spans with a high fundamental frequency the resonance peaks within realistic speed intervals are typically higher order resonances. The primary resonance for these bridges corresponds to very high speeds. Museros and Alarcón (2002) and Doménech and Museros (2011) adopted another approach and studied specifically the primary resonance.

Chapter 5

Case study: bridges along the Bothnia Line

In this chapter, some of the author's results from an ongoing investigation for the Swedish Transport Administration (Trafikverket) are included. The project provides an example of the use of TBI in a feasibility study for increased speeds on an existing railway line.

5.1 The Bothnia Line bridges and previous work

The Bothnia Line is a single-track railway line in northern Sweden, built between 1999 and 2010. For most of the 87 railway bridges along the line, no dynamic analysis was performed as the Eurocode (CEN, 2003) design guidelines for high-speed traffic were not yet implemented in Sweden at the time of design. Therefore, a dynamic assessment for future high-speed trains was performed by Johansson et al. (2013). The work presents a first stage out of three in the planned assessment and adopts simple 2D models for dynamic analyses of 76 of the bridges. It was found that the design limit on the vertical bridge deck acceleration was exceeded for about 50% of the bridges. In a second stage, Andersson et al. (2014) implemented more advanced numerical models. It was believed that the following modelling aspects can reduce the predicted bridge deck acceleration: the use of load distribution, the consideration of soil–structure interaction and the consideration of TBI. Some of the author's results concerning TBI will be described here. Field measurements and model calibration are intended as a third stage in the project.

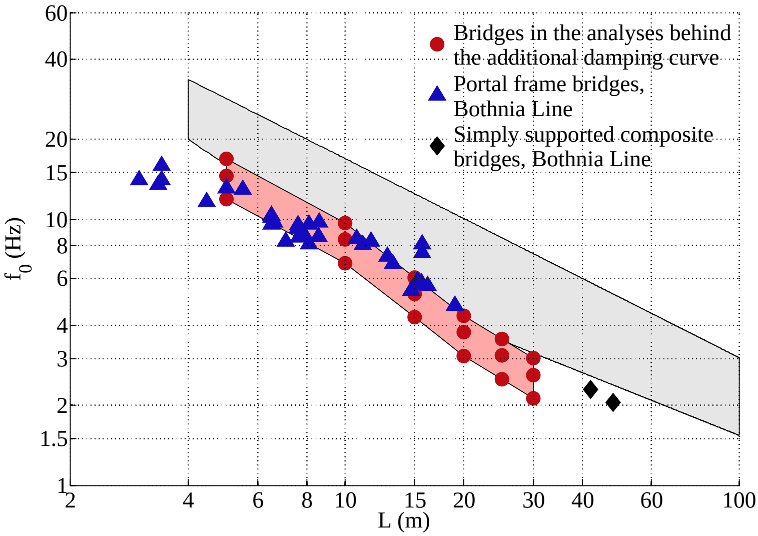
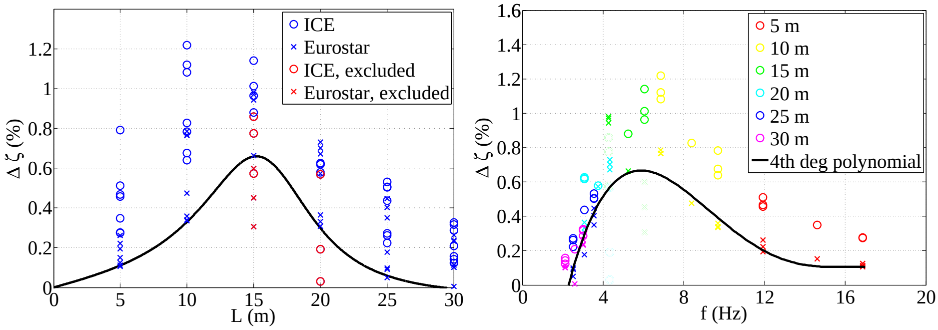


Figure 5.1: The fundamental frequencies of the ERRI D214 (1999c) bridges used in the analyses behind the Eurocode additional damping. The approximate frequency range covered by the ERRI D214 bridges is indicated in light red. The frequency range specified for high-speed railway bridges in the Eurocode is indicated in grey. The frequencies of some of the Bothnia Line bridges are included for comparison.

Additional damping according to the Eurocode, CEN (2003), for bridges of span 0–30 m, was implemented already in stage one of the project. The objective of this part of the work was to investigate whether reduced response from TBI can be obtained for two simply supported bridges with spans over 30 m. For this purpose, similar models as those used by ERRI D214 (1999c) in deriving the additional damping in the Eurocode was adopted. Another objective was to investigate whether the additional damping according to the Eurocode can be deemed reasonable for the portal frame bridges along the line. Comparative TBI analyses on portal frame bridges are motivated as the additional damping in the Eurocode was derived for simply supported bridges only.

5.2 Analyses by the ERRI committee D214

The additional damping in the Eurocode was derived by ERRI D214 (1999c) from TBI analyses of simply supported beam bridges of span 0–30 m. The fundamental frequencies of the bridges are shown in Figure 5.1 (where they are compared to the fundamental frequencies of some of the Bothnia Line bridges). A fictitious addi-



(a) (b)

Figure 5.2: The ERRI D214 (1999c) results behind the Eurocode additional damping defined as a function of span length (a). Analyses in which no resonance occurred were deemed unrepresentative by the ERRI committee and are indicated in red. In (b), the results are plotted against bridge fundamental frequency instead of span length. An example of a curve fitted to the lower bound values is included.

tional damping was introduced in an MF model, which was then calibrated against results from TBI analyses with the SIM. The ICE 2 train, seen in Figure 2.4, with one leading and one trailing power car and 12 passenger carriages was considered together with the Eurostar train. Additionally, the following was assumed for the SIM: (1) the unsprung mass of the wheelset was neglected, (2) the stiffness and damping of the axles were taken as equal to those of the primary suspension. Analyses were performed in the speed range 150–350 km/h with no consideration taken to track irregularities. In the ERRI committee D214 analyses, the deformed shape of the beam was described using modal superposition with a maximum of three modes and a cut-off frequency at 30 Hz. Modal superposition was not used in the analyses performed within the present work.

The results from the ERRI committee D214 are shown in Figure 5.2(a) together with the additional damping as a function of span length. As mentioned, the choice to give additional damping as a function of span length is a simplification. This can be realised from the large spread in the results in Figure 5.2(a). Another possible choice is to give the additional damping as a function of bridge frequency, even though this is also a simplification of a much more complex relation. The ERRI committee D214 results plotted against frequency is shown in Figure 5.2(b). The curve shows that some additional damping is to be expected for bridges with frequency 2–3 Hz. Steel-concrete composite bridges with spans of about 30–40 m, that are given no additional damping according to the Eurocode, can be expected to have a fundamental frequency within this range.

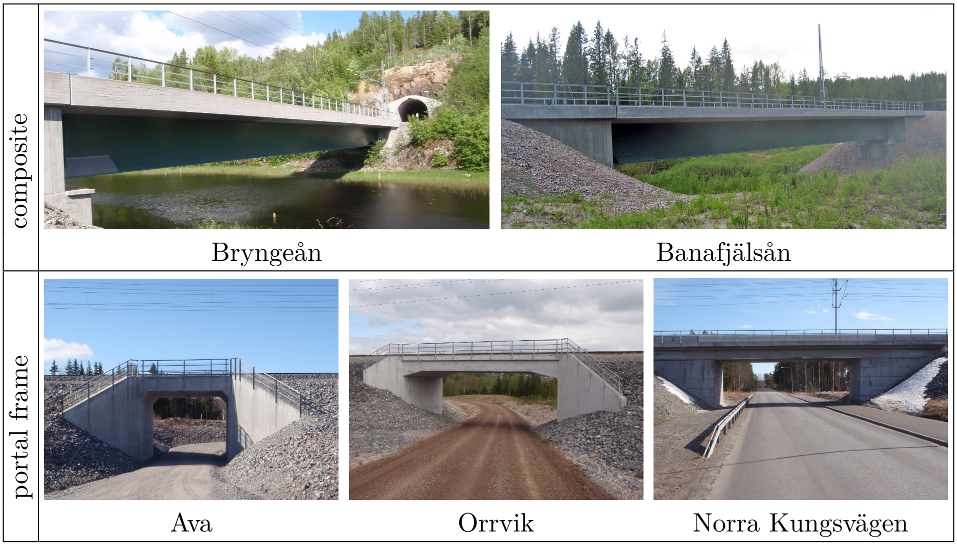


Figure 5.3: The studied Bothnia Line bridges, photos from BaTMan (Trafikverket, 2014).

5.3 TBI analyses of the Bothnia Line bridges

TBI analyses have been performed for two simply supported single span bridges and three portal frame bridges seen in Table 5.3. The train load was the ICE 2 train with parameters according to Figure 2.4 and Table 2.1. The reasons behind the choice of train model were: (a) to make the analyses comparable to those performed within the ERRI committee D214, (b) there are no vehicle parameters available for the HSLM load model that is normally used in design calculations. Based on Figure 5.2(a) we can expect slightly higher reductions from TBI with the ICE 2 train than with the Eurostar train.

The two-layer suspension RB model and the SIM, outlined in Figure 2.1(a) and (b), were used in the analyses. As mentioned, the SIM takes into account the bogie–bridge interaction while the car body–bridge interaction is ignored. The RB model takes into account also the car body–bridge interaction. For bridge spans above around 30 m, or with a fundamental frequency much lower than the bogie frequency (closer to the car body frequency), the SIM deviates slightly from the RB model. The SIM then predicts less reduction in response. This is why also the RB model was adopted here. The vertical bogie frequency of the ICE 2 passenger carriages is 6.11 Hz whereas the vertical car body frequency is 0.64 Hz.

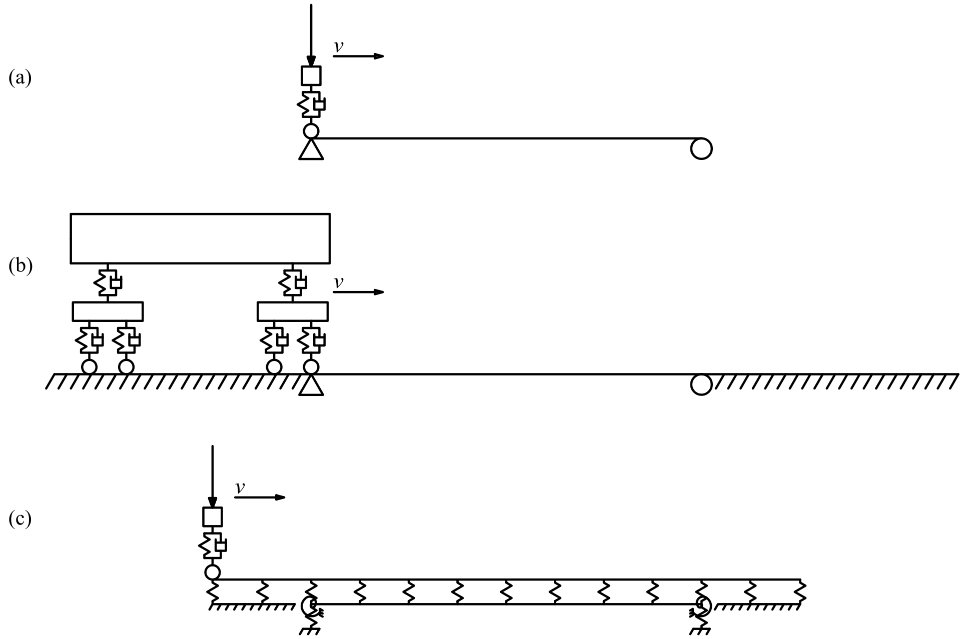


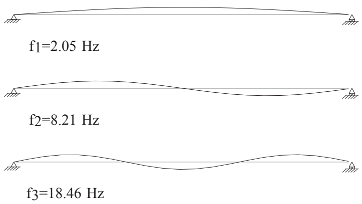
Figure 5.4: Schematic sketch of the model used for TBI analyses in Abaqus for: (a) the SIM on simply supported bridges, (b) the two-layer suspension RB model on simply supported bridges, and (c) the SIM on portal frame bridges.

Figure 5.4(a) and (b) present schematic sketches of the Abaqus model used for TBI analyses of the simply supported bridges. The analyses were performed as described in Chapter 2 with additional assumptions according to the previous section.

The simplified 2D beam model described in Johansson et al. (2014) was adopted to idealise the portal frame bridges. The rigidity of the frame walls as well as the elastic interaction with the soil is taken into account by vertical and rotational springs at the supports. The mass of the frame walls and the soil resting on the foundation is lumped into the beam model over a length of 0.1 m from each support. The damping associated with the soil–structure interaction was not considered. In Abaqus, the stiffness contributions from the springs are not automatically included in the Rayleigh damping matrix. Therefore, equivalent dashpots were introduced in parallel with the vertical and rotational springs at the supports to achieve the desired overall damping ratio.

A track was introduced to the beam models with elastic supports (the portal frame bridge models) to guide the axles over the bridge; see Figure 5.4(c). A relatively

Table 5.1: Bridge data for the Bryngeån bridge, from Johansson et al. (2013).

Modes of vibration	Bridge properties	
	Bridge type	beam bridge, simply supported
	Span length (m)	48
	E (GPa)	200
	I_x (m^4)	0.86
	m (kg/m)	19,000
	ζ, excluding additional damping (%)	0.5
	$\Delta\zeta$, additional damping (%)	0

stiff bed modulus was assumed for the track (300 MN/m and m along the track) to reduce the load distributive effect of the track. It is assumed that the presence of the track does not affect the amount of reduction in bridge deck acceleration from the SIM as compared to the MF model. This has been verified only for a 10-m simply supported bridge.

5.4 The Bryngeån bridge and the Banafjäl bridge

The Bryngeån bridge is one out of two simply supported steel-concrete composite bridges that did not fulfil the design limit for vertical bridge deck acceleration according to the analyses in stage one of the project. The Bryngeån bridge has a span of 48 m and is therefore given no additional damping due to TBI. However, with a bridge frequency of 2.05 Hz we can expect a small amount of additional damping according to Figure 5.2(b). Modes of vibration and bridge properties are given in Table 5.1.

The mid-span acceleration under the ICE 2 train load in the speed range 100–300 km/h is shown in Figure 5.5. As can be observed, there is little difference between the MF model and the SIM. As mentioned, the RB model represents a slightly more complex model of the train taking into account also the car body–bridge interaction. As seen in Figure 5.5, the RB model predicts indeed slightly reduced responses compared to the SIM. An additional damping has been manually fitted to the results of the RB model. As could be expected, the amount of additional damping that can be motivated for this bridge is rather low: about 0.1%. The additional damping of 0.1% leads to a 5% decrease in the predicted

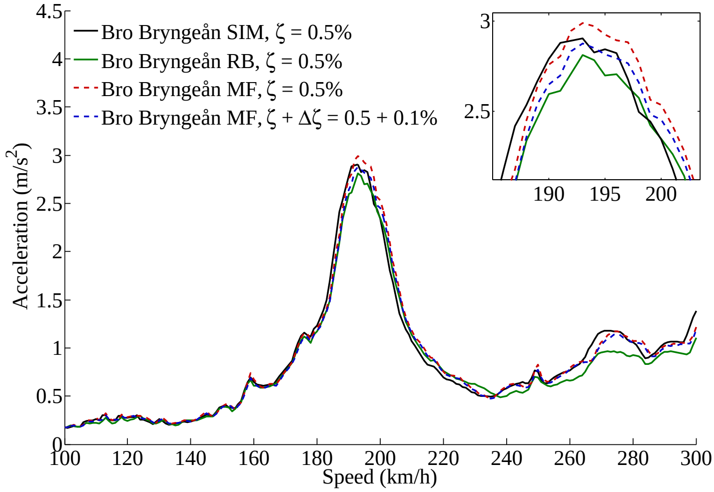


Figure 5.5: Acceleration at mid-span for the Bryngeån bridge. Results from the SIM, the RB model and the MF model. An MF model with additional damping has been manually fitted to the RB results. The first resonance of the fundamental frequency and the characteristic axle distance is seen at: $v = f_1 \times d_c \times 3.6 = 2.05 \times 26.4 \times 3.6 = 195$ km/h.

maximum bridge deck acceleration under the most critical HSLM load model (A7), as an example.

The results for the 42-m span Banafjälsån bridge were similar and an additional damping of 0.1% could be motivated also for that bridge.

5.5 Portal frame bridges

The results from TBI analyses of three portal frame bridges are presented in Figures 5.6–5.8. The Ava bridge, Table 5.2, has a span length of 6.5 m while the Orrvik bridge and the Norra Kungsvägen bridge both have span lengths of about 15–16 m; see Tables 5.3 and 5.4.

As can be observed from Figure 5.6 the bridge deck acceleration obtained with the MF model and the SIM are similar for the Ava bridge, that is, the reduction obtained from including TBI is small. The Eurocode additional damping assumption seems reasonable for this bridge. For the Orrvik bridge, Figure 5.7, the bridge acceleration obtained from the SIM is considerably lower than from the MF model. There is

Table 5.2: Bridge data for the Ava bridge, from Johansson et al. (2013).

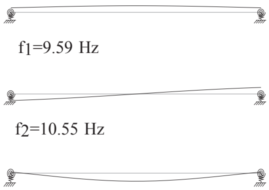
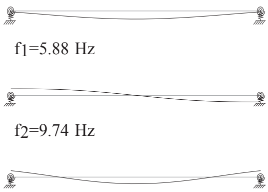
Modes of vibration	Bridge properties	
	Bridge type	portal frame, open
	Span length (m)	6.5
f1=9.59 Hz	E (GPa)	20.4
	I_x (m^4)	0.14
f2=10.55 Hz	m_plate (kg/m)	17,126
	$m_\mathrm{foundation}$ (kg)	368,267
	k_v (MN/m)	1604
f3=22.25 Hz	k_φ (MNm/rad)	959
	ζ, excluding additional damping (%)	2.45
	$\Delta\zeta$, additional damping (%)	0.16

Table 5.3: Bridge data for the Orrvik bridge, from Johansson et al. (2013).

Modes of vibration	Bridge properties	
	Bridge type	portal frame, open
	Span length (m)	15.25
f1=5.88 Hz	E (GPa)	20.4
	I_x (m^4)	0.48
f2=9.74 Hz	$m_\mathrm{plate}(kg/m)$	22,625
	$m_\mathrm{foundation}(kg)$	459,410
	k_v (MN/m)	1984
f3=11 Hz	k_φ (MNm/rad)	2208
	ζ, excluding additional damping (%)	1.83
	$\Delta\zeta$, additional damping (%)	0.65

a good match between the results of the SIM and those of the MF model with additional damping according to the Eurocode. Also for this bridge, the Eurocode recommendations seem reasonable.

From Figure 5.1 we can see that the fundamental frequency of the Norra Kungsvägen bridge (8.09 Hz) is considerably higher than those of the 15-m bridges considered by the ERRI committee D214. We therefore expect that the results from a TBI analysis on this bridge will not match the results predicted by the Eurocode additional damping. Figure 5.8 confirms that the additional damping according to the Eurocode is indeed too high for this bridge.

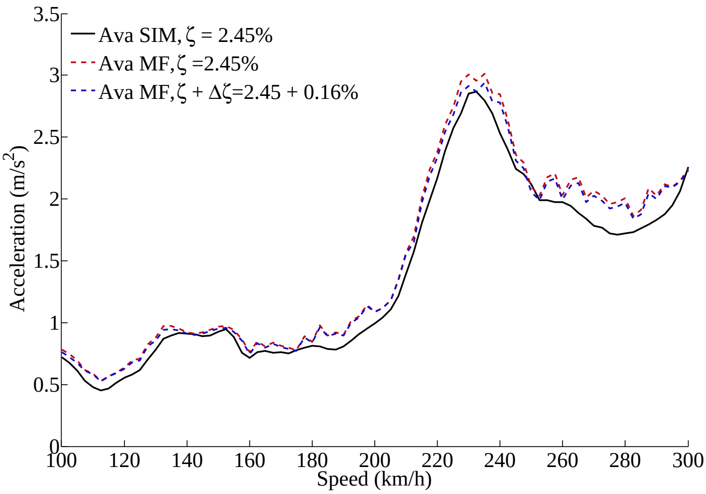


Figure 5.6: Acceleration at mid-span for the Ava bridge. Results from the SIM and the MF model. An MF model with additional damping according to the Eurocode is also included. The peak is a resonance induced by the power car and the third bridge mode at 22.25 Hz (a first bending mode), see Table 5.2.

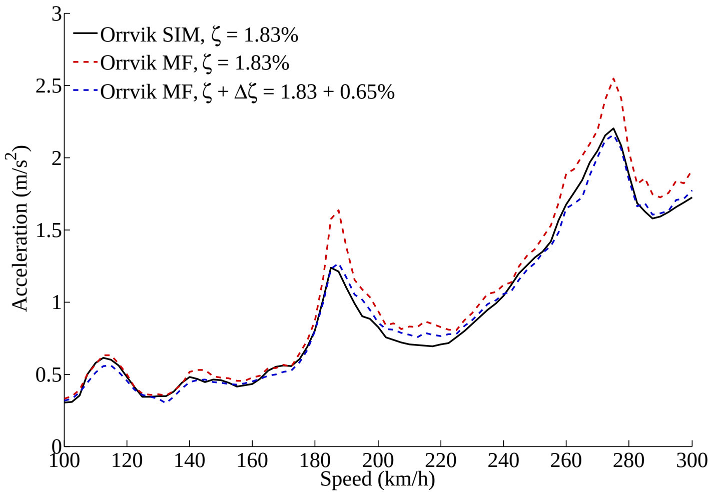
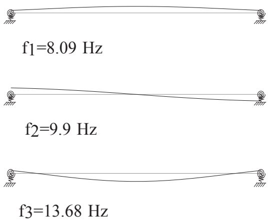


Figure 5.7: Acceleration at mid-span for the Orrvik bridge. Results from the SIM and the MF model. An MF model with additional damping according to the Eurocode is also included. The second resonance of the fundamental frequency and the characteristic axle distance is seen at: $v = f_1 \times d_c \times 3.6/2 = 5.88 \times 26.4 \times 3.6/2 =$ 279 km/h. The third resonance is seen at around 190 km/h.

Table 5.4: Bridge data for the Norra Kungsvägen bridge, from Johansson et al. (2013).

Modes of vibration	Bridge properties	
	Bridge type	portal frame, open
	Span length (m)	15.7
	E (GPa)	20.4
	I_x (m^4)	1.59
	$m_{\text{plate}}(kg/m)$	21,712
	$m_{\text{foundation}}(kg)$	547,456
	k_v (MN/m)	2294
	k_φ (MNm/rad)	6581
	ζ, excluding additional damping (%)	1.80
	$\Delta\zeta$, additional damping (%)	0.65

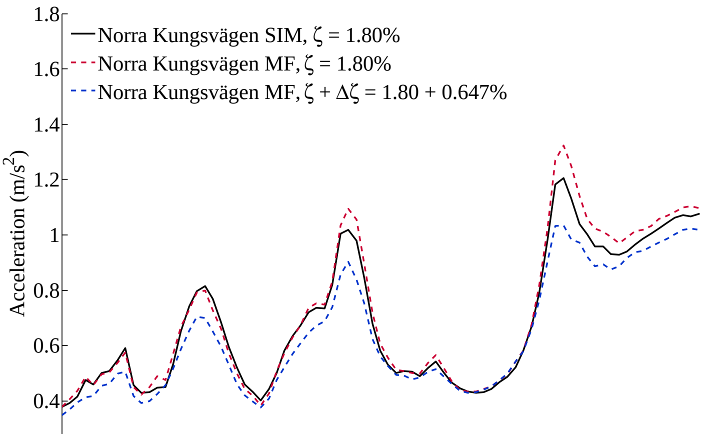


Figure 5.8: Acceleration at mid-span for the Norra Kungsvägen bridge. Results from the SIM and the MF model. An MF model with additional damping according to the Eurocode is also included. The third resonance of the fundamental frequency and the characteristic axle distance is seen at: $v = f_1 \times d_c \times 3.6/2 = 8.09 \times 26.4 \times 3.6/3 = 256$ km/h. The forth resonance is seen at around 190 km/h.

5.6 Case study conclusions

The initial motive to the investigation was that the additional damping due to TBI as prescribed in the Eurocode is a rather rough simplification of a complex relation; refined TBI analyses might reduce the calculated acceleration levels for some of the bridges.

The results show that, if the RB model is adopted, a moderate reduction in response due to TBI can be expected under the ICE 2 train load for the two studied simply supported bridges: the Banafjälsån bridge and the Bryngeån bridge. Both bridges have rather low fundamental bridge frequencies, around 2 Hz, that approaches those of the car body modes. Presumably, this is why the RB model, that also includes the car body–bridge interaction, shows larger reductions as compared to the SIM. A small amount of additional damping, about 0.1%, can be motivated for both bridges based on the RB analyses. Under the HSLM load this additional damping corresponds to a reduction in bridge deck acceleration of about 5% for both bridges.

Contrary to the results from the Skidträsk bridge, presented in Section 4.1, the reductions in acceleration from TBI are small. For the 36-m Skidträsk bridge the reduction in acceleration was up to 20% (see Figure 4.1(a)). The bridge has a slightly higher fundamental frequency (closer to the vertical train bogie frequency), and is slightly lighter than the Banafjälsån bridge and the Bryngeån bridge.

The additional mass in portal frame bridge structures (from the frame walls and the soil on the foundations), as well as the differences in frequencies and mode shapes between portal frames and simply supported bridges, lead us to call into question whether the Eurocode additional damping covers portal frame bridges. In this work, the elastic soil–structure interaction was included in the models. This lead to reduced bridge frequencies (and to the introduction of rigid body modes) as compared to models with fixed supports. Under these assumptions, the fundamental frequencies of the portal frame bridges match well the frequencies of the bridges in the analyses behind the additional damping; see Figure 5.1. This is believed to be the reason for the god fit between some of the TBI results and the results with additional damping as provided in the Eurocode. For the Norra Kungsvägen bridge, with a fundamental frequency considerably higher than the ERRI committee D214 bridges, the additional damping according to the Eurocode proved to be too high and thus non-conservative. In conclusion, the analyses show that the additional damping is not directly applicable to portal frame structures.

Chapter 6

Discussion and conclusions

6.1 Parametric studies

Two-level factorial experiments were adopted to study parameter variations in TBI systems. Contrary to one-factor-at-a-time procedures, factorial experiments are able to show parameter interactions. Moreover, they provide a neat way of presenting the results: the relative importance of each parameter is easily observed from a normal score plot. The parametric studies in papers II and III showed that:

- TBI is only important at resonance – an observation which is consistent with the conclusions in ERRI D214 (1999c); Liu et al. (2009); and Rocha et al. (2012).
- The reduction from TBI is slightly larger for acceleration than for displacement; this was also observed by Doménech and Museros (2011).
- For some of the train–bridge systems, the effect of TBI was smaller or comparable to the effect of other parameter variations (e.g. in axle load, beam stiffness, beam mass, beam damping and rotational support stiffness) while it was the most important effect for other train–bridge systems. This applies both to passenger train and freight-train-induced bridge vibration.
- A number of parameter interactions were found: load model*beam damping, load model*beam stiffness, load model*axle load, and load model*rotational support stiffness. These results confirm that the effect of TBI depends on several other system parameters.

- The parameter interaction effects were rather low (mostly below 10%) for factor variations corresponding to reasonable model uncertainties. If larger variations were introduced the interaction effects would be more important.

- Considerable reductions in response from TBI were obtained for clear resonance cases for the freight train applications in paper III. Possible explanations are the considerable length of the freight trains and the high train–bridge mass ratios (especially as the bridge data were chosen based on mainly single-track bridges).

- The SIM (Figure 2.1(b)) gives similar results as the slightly more complex two-layer suspension RB model (Figure 2.1(a)) for passenger train applications; thus, it presents a relevant simplification of the train load. The RB model can give a slight additional reduction in the results for bridges with frequencies well below that of the train bogie frequency (Chapter 5).

- For freight trains with a stiff secondary suspension system the car body–bridge interaction is important which is why an RB model should be adopted (with a single- or two-layer suspension, Figure 2.1(a) and (c)), or alternatively the simplified quarter car model (Figure 2.1(d)). The SIM is not a suitable model for this type of freight trains.

6.2 Freight-train-induced bridge vibration

Although freight trains travel at lower speeds than passenger trains the, typically, shorter axle distances imply that the resonance speeds are lower. In combination with low bridge fundamental frequencies, the first or second order resonance speeds may very well be within realistic speed ranges. If resonance indeed occurs it can grow large due to the considerable length of the trains. The analyses presented in Karoumi and Wiberg (2006), as well as the examples in paper III, indicate that single-track steel-composite bridges and prestressed concrete bridges can be susceptible to freight-train-induced vibrations. This is especially true for single-span bridges (20–45 m) with a fundamental frequency below approximately 5 Hz. As an example, the models in paper III predict a resonance peak at 95 km/h for the steel-concrete composite Skidträsk bridge under the Swedish Steel Arrow train. The acceleration peak was much reduced with a TBI model, as compared to the MF model.

6.3 Additional damping

The Eurocode (CEN, 2003) defines additional damping for bridges of span 0–30 m. Throughout the work of this thesis examples have been identified for which the results from a TBI analysis deviate from what is predicted by the Eurocode additional damping:

- The reduction from TBI was shown to be considerable (20–30%) under the ICE 2 train and the Green Train for the Skidträsk bridge with span over 30 m (see Figure 4.1 and paper II).
- The reduction in acceleration from TBI for the 6-m bridge in paper II was slightly lower than what is predicted by the additional damping (see paper II, Table 4).
- It was observed in Chapter 5 that the additional damping seem applicable to some of the portal frame bridges along the Bothnia Line in northern Sweden. For the Norra Kungsvägen bridge, however, the reduction in acceleration predicted by the additional damping was non-conservative compared to the results from a TBI analysis (see Figure 5.8).

Additionally, the following show that the additional damping is a rough simplification: (1) the large spread seen in the results of the analyses behind the additional damping criteria in Figure 5.2(a), and (2) the fact that the span length has not been found to be the most important parameter, but rather: the train–bridge frequency ratio, the train–bridge mass ratio and the bridge–carriage length ratio.

The above reasoning leads to the choice between:

(i) to use the present additional damping in the Eurocode and define bridge types for which it may be worth conducting a TBI simulation, or

(ii) to refine the additional damping criteria in the Eurocode to better describe the effect of TBI for a wider range of bridge characteristics.

As a rough recommendation on choice (i), a TBI analysis is relevant whenever the frequency of the bridge lies well outside the frequency range covered by the bridges in the ERRI committee D214 analyses; see the light red area in Figure 5.1 and Figure 8 in paper II. For passenger train applications we may expect large reductions in bridge deck accelerations at resonance in low mass bridges with a frequency close to the bogie frequency of the train. Due to the influence of the mass ratio, higher reduction can be expected for light single-track bridges than for heavy double track bridges. If resonance occurs under freight train passages we might expect considerable reductions by considering TBI as we then typically have large train–bridge mass ratios and a pronounced resonance due to the long train sets.

The author finds choice (ii), a further development of the additional damping concept, more relevant as: (1) it can easily be included in the dynamic analysis, and (2) it describes the effect of TBI rather well as TBI and damping are both important only at resonance.

We must recognise, though, that the additional damping is a small amount of damping that is added to an initial bridge damping estimate that is generally associated with a large uncertainty. The Eurocode recommendations on damping ratios are essentially based on lower bound values from numerous damping ratio estimates, determined from measured free vibrations after train passages. The author believes that a "consistent degree of approximation" should be sought in order for the additional damping concept to be fully relevant: if the additional damping contribution from the TBI is investigated in detail then an equivalent amount of effort should be put into the bridge damping estimate. The studies by Ülker-Kaustell and Karoumi (2011, 2013) present examples of detailed investigations into the bridge damping. For a simply supported steel-concrete composite bridge, they observed damping ratios that, at low amplitudes of vibrations, are in line with the Eurocode recommendations. However, they show that the damping ratios increase considerably at high amplitudes of vibration.

6.4 Further research

During the course of the present research the author has identified many interesting future research directions: a list of suggestions is found as part of the conclusions in the literature review, paper I. Some of the issues that are most closely related to the content of this extended summary will be further developed here.

Proposal for refinement of the additional damping for simply supported bridges

The need for a refinement of the additional damping in the Eurocode was motivated in the previous section. The author sees several alternatives for enhancing the additional damping definition for simply supported bridges; further studies should determine which is the most feasible. Possible definitions include: (1) additional damping based on the train–bridge frequency ratio, train–bridge mass ratio and bridge–carriage length ratio, (2) additional damping based on bridge frequency and further differentiated based on bridge mass and bridge length, or (3) additional damping based on bridge frequency and differentiated based on bridge type (e.g. bridge deck material, single-track or multi-track). Alternative (1) would be the most solid alternative. It would however require the knowledge of train characteristics (masses and suspensions). Therefore, alternatives (2) or (3) could be better choices. Then, for each combination of bridge characteristics, a range of train characteristics needs to be analysed to be able to give a lower bound recommendation as a function of bridge characteristics only.

Hence, to develop a refined additional damping definition analyses are needed for a range of bogie–bridge frequency ratios, bogie–bridge mass ratios and bridge–carriage length ratios. A set of dimensionless parameters can be used to cover such parameter ranges, as was implemented by Museros et al. (2002) and Doménech and Museros (2011).

Based on the rather limited analyses presented in Chapter 4, the amount of additional damping that should be introduced to fit the results from a TBI model seem independent of the train set length: both the effect of TBI and of an additional amount of damping increase with increasing resonance. Similarly, based on the analysis in Chapter 4 the train set configuration (presence or absence of power cars) do not affect the estimation of additional damping when the resonance is induced by the repetitive carriage axle loads. According to the knowledge of the author no studies have revealed whether the effect of TBI is the same at the first order resonance as at higher multiples of resonance. It should therefore be further investigated whether an additional damping fitted to the fundamental resonance is valid also at higher order resonance peaks that occur at lower speeds.

Furthermore, an extension of the additional damping definition to include also speeds above 350 km/h is interesting. New railway lines in Sweden will be designed for 320 km/h (Trafikverket, 2012), and as calculations is to be performed up to 1.2 times the maximum permitted speed the limitation on the present additional damping criteria at 350 km/h is too low.

Additional damping for other bridge types

A definition of additional damping for bridge types other than simply supported beam bridges can be motivated based on the discussion on portal frame bridges in Chapter 5. The need for a unique definition of additional damping should be examined for: (1) portal frame bridges (2) multi-span continuous bridges, and (3) series of simply supported bridges.

Guidelines for the dynamic assessment of bridges under freight train traffic?

No dynamic analysis is generally required for bridges along lines with speeds below 200 km/h. As bridge resonance may arise due to freight train traffic, special design rules for dynamic analyses of bridges under freight train traffic may be relevant. The reduction from considering TBI can be large, as was shown in paper III. TBI analyses or the development of an additional damping curve for freight trains could thus be of interest. The use of more detailed load models is particularly relevant in the upgrading of existing lines for higher train speeds and higher axle loads.

Field measurements

There is a need for further field measurements to verify TBI models. Some published comparisons between measured data and 3D TBI models are discussed in paper I. It would further be interesting to perform measurements on the simplest possible full scale railway bridge (that can be sufficiently well described with a 2D beam model) to verify the amount of reduction obtained in our 2D TBI models as compared to MF models. A well-maintained track is preferable to reduce the effect of track irregularities. However such comparisons are difficult due to the inherent discrepancies between theoretical models and the actual bridge structure in, for example, damping, bridge member stiffnesses and support conditions.

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Part B

Appended papers

Appendix I

Train–bridge interaction — a review and discussion of key model parameters

Accepted for publication in the International Journal of Rail Transportation, February 2014.

Appendix II

Statistical screening of modelling alternatives in train–bridge interaction systems

Engineering Structures, volume 59, pages 693–701, 2014.

Appendix III

Modelling alternatives in the dynamic interaction of freight trains and bridges

Proceedings of the Second International Conference on Railway Technology: Research, Development and Maintenance, Ajaccio, Corsica, France, 8–11 April 2014.

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