DYNAMIC RESPONSE OF FILLER BEAM BRIDGES DURING TRAIN TRANSIT

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INTRODUCTION

A comprehensive investigation of the dynamic behaviour of railway bridges has been recently performed in TDV within the framework of the joint research project “RL-Dynamik”. TDV focussed hereby on investigating single span WIB girders (rolled steel beams cast in concrete) with up to 30 m span length. The results gave the basis of the design proposal in the “Guidelines for the Dynamic Investigation of Railway Bridges” to be published by the Austrian railway authorities.
The investigation comprised un-symmetric single-track structure with 4.85 m width and an edge beam on one side. WIB girders with 3 different concrete grades, 3 different steel profile series (HEA, HEB and HEM) and 2 variants of distribution (7 steel beams with 70 cm distance and 10 steel beams with 48.5 cm distance) were investigated.
In this context, the new program module „**Fast Rolling Stock**“ has been developed in order to allow for performing the analyses with reasonable calculation speed.
Valuable insights could be gained while performing the work and assessing the results as partly presented in this contribution. This concerns the coherence between the governing parameters for the resonance behaviour of the girders. Some surprising phenomena were discovered, e.g. the fact, that for all girder variants the vertical accelerations reach a maximum value at about 17 m span, whereas higher spans again allow higher travel speeds.
An important insight is that the acting momentum essentially influences the behaviour, and that the effective momentum depends to a great extent on the ratio between load distance and span length. The calculation works have been terminated in 2007, and the workout of the “Guidelines” is work in progress. The filler beam girder related findings are in presented in this contribution.

1 SCOPE OF WORK

Description of Investigated Filler-Beam Girders

The investigations were performed for single-span single-track filler-beam girders with span lengths from 5 m to 30 m and plate depths H from 0.45 to 1.10 m. A typical cross-section is shown in Figure 1.

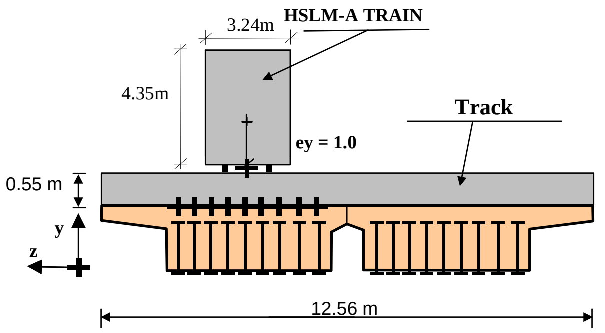

Fig. 1. Typical cross section

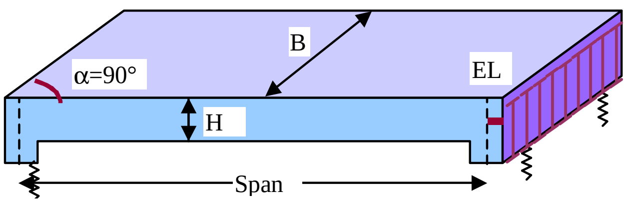

Fig. 2. Structural system

The investigated plate girders are rectangular with support lines orthogonal to the longitudinal direction. The dimensions in lateral directions are governed by operational requirements and are therefore the same for all investigated structures: Plate width (without cantilever as indicated in Figure 2) 4.85 m. Rolled steel sections in longitudinal direction are cast in the concrete plate.

Different variants were investigated with respect to

a) Concrete grade – variants C0 (C25/30), C1 (C30/37), C2 (C35/45)
b) Number of rolled steel sections – variants “Weit” (7 sections) and “Eng” (10 sections)
c) Type of steel sections – variants HEA, HEB and HEM

Structural model

The girder is modelled as girder grid with 7 or 10 longitudinal beams respectively, each subdivided into 16 beam elements. The individual longitudinal beams are composite beams consisting of the embedded steel section and the related rectangular concrete part. The stiffness of the cantilever plates is neglected. The bearing behaviour in lateral direction is described by cross girders connecting the longitudinal girders in every point.

Support conditions and cross girders at the ends

The structure has been quasi-rigidly and statically determinate supported in 4 points. The 4 support points are arranged along the support line, each with 50 cm distance from the edge. The eccentric position of the support point has been considered for taking into account vibration effects due to horizontal movement of the structure. The actual restraint point was assumed 10 cm below the end cross girder (identical for all spans).
End cross girder: First investigations with omitting the higher depth and the excess length EL of the end cross girder showed, that considering this higher lateral stiffness is essential for small spans. Therefore, a detailed modelling of the end cross girders was required (thickness = plate thickness + 10 cm). Additionally, an equivalent excess length of 55 cm was considered.

Loads and Masses

In addition to the self-weight of the structure the 55 cm roadbed (specific weight 20.0 kN/m^3) was considered as distributed mass. Additional masses were applied on the left-most and right-most girders for considering the weight of the cantilever plates with superimposed dead load. The moving traffic loads were not considered as effective masses.
The traffic load was applied on the track in accordance with EN 1991-2. The moving load was distributed in lateral direction over the affected longitudinal girders and in longitudinal direction over 3 crossties (distance 0.65 m) with the ratio 0.25, 0.5, 0.25. This longitudinal distribution is in accordance with EN 1991-2 and was implemented after first calculations gave very unfavourable results for girders with small span length, where the small axle distances (2 m to 3.5 m) govern the excitation.
EN 1991-2 also specifies High Speed Load Model trains to be investigated. HSLM-A trains and HSML-B trains must be investigated up to a span length of 7.0 m. For span lengths higher than 7.0 m only HSLM-A trains need be considered, because HSML-B trains are not anymore relevant in this span range.

2 PHENOMENOLOGICAL INVESTIGATIONS

Preliminary investigations have been performed for studying the principal behaviour of the systems with varying different structural parameters. These studies were performed for different structures of the series with concrete C0, and with 7 HEB steel sections arranged in the cross-section (series C0-HEB-weit).

Influence of the Loading

Figure 3 shows the maximum vertical acceleration for different trains. We see that the behaviour is governed by resonance effects, i.e. the peaks occur where the excitation frequency governed by the typical bogie distance matches the natural frequency of the structural system.
The actual size of the peaks is however essential for assessing the allowable travel speeds and the question whether smaller peaks – as occur for 2 trains on the left side in Figure 3 – are just below or just above the limit value changes the allowable travel speed dramatically. Therefore it is essential to apply accurate calculation methods and model parameters to get maintainable results.

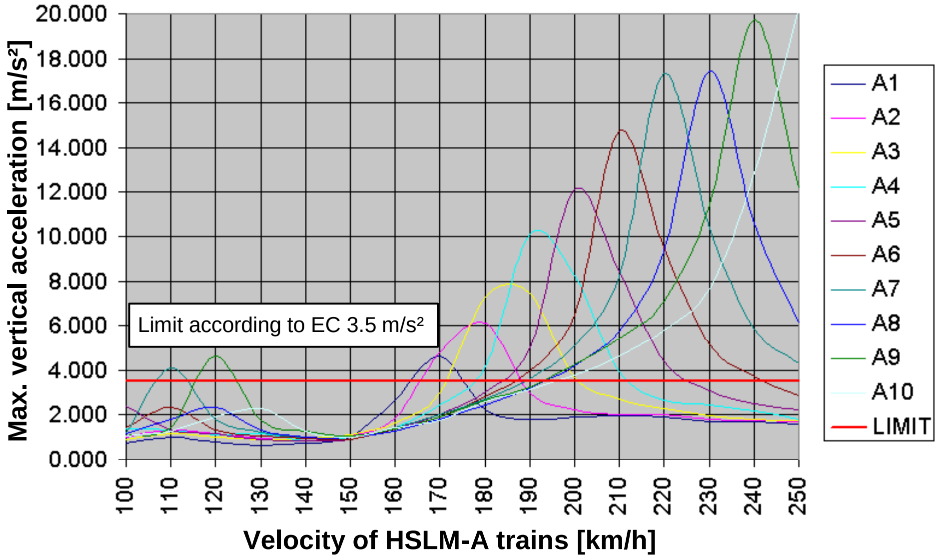

Fig. 3. Maximum vertical acceleration for different trains

Calculation method

Modal analysis versus Direct Time Integration

A new extremely fast calculation tool had to be developed due to the huge amount of structures to be investigated. Intensive research was done to find the appropriate method. The modal analysis method and the direct time integration method are known as suitable analysis methods. Both have advantages. The modal method reduces the equation system to be solved considerably [2], however, complicated and time consuming transformations of the loading terms are required.
Comparative considerations showed that an optimized direct time integration scheme seems to be more effective at the end. This strategy has therefore been prosecuted and the respective software tool “Fast rolling stock” developed, based on the Newmark method for time integration [3]. The solution reduced the computing time for the time stepping analysis of one train passage by a factor of several hundreds with respect to the analysis with the standard bridge analysis tool RM.

Accuracy – Time step length

The Newmark time stepping scheme is unconditionally stable also for larger time step lengths. However, results may become very inaccurate, if the used time step length exceeds a certain value. This value is determined by the contributions of the highest frequency to be considered. As a rule of thumb we can say, that 10 time steps must describe a vibration period, i.e. if contributions with 20 Hz shall be taken into account, we need a time step length of 0.005 sec.
Therefore, the required time step length is dependent on the natural frequencies and in consequence on the span length. The huge amount of calculations required saving computing time wherever possible. Therefore a formula was developed, where the used time step length was defined as a function of the span length: $\Delta t = (0.001 + 0.004*(STW-8.0)/22.0$ sec

Damping values

The structural damping is very important for the magnitude of vibration amplitudes. EN 1991-2 prescribes very conservative design values dependent on the span length:

$\varsigma = 1.5 + 0.07 * (20 - L)$ for $L < 20$ m and $\varsigma = 1.5$ for $L > 20$ m respectively.

An additional span-length dependent damping for span lengths below 30 m may be assumed:

$\Delta\varsigma = (0.0187L - 0.00064L^2)/(1 - 0.044L - 0.0044L^2 + 0.000255L^3)$

Rayleigh damping matrices are used in time history calculations, therefore the respective equivalent Rayleigh coefficients were evaluated with using the actual damping ratio and the relevant natural modes. The natural frequencies of the first and 2nd bending modes were assumed relevant.

Dynamic coefficients

The design code requires also applying “dynamic coefficients” for taking into account any track irregularities. Therefore, the calculated accelerations had to be multiplied by the factor (1+Phi’’/2), where Phi’’ is a function of the basic natural frequency and span length. Eigenfrequency analyses has therefore been performed although not essentially required for the time history analysis.

Calculation of Natural Frequencies

These Eigenfrequency analyses have been performed in advance for all structures. The results are the base for calculating damping parameters and dynamic coefficients, for studying the influence of various parameters and for defining required time step lengths. Fig. 4 and 5 show the lowest natural frequencies for 2 different plate depths (0.6 m and 1.1 m) as a function of the span length.

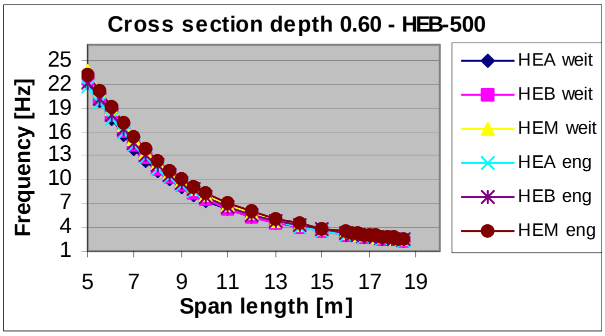

Fig. 4. 1st natural frequency for depth 0.6 m

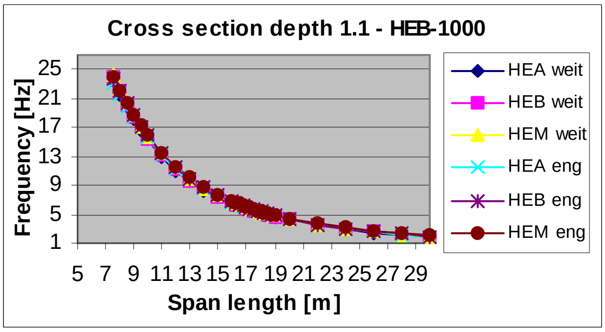

Fig. 5. 1st natural frequency for depth 1.1 m

We see in these diagrams, that the first natural bending frequencies vary between less than 1 Hz for large span length and some 25 Hz for the smallest span lengths. However, we also see that the differences between different steel section arrangements are very small.

Influence of the concrete grade

Considering, that the concrete grade only influences the natural frequencies and no other parameters like mass or damping, it is justified to omit detailed rolling stock calculations for other grades than C0, if a direct correlation between the natural frequencies of these structures can be found. This relation was found in detailed comparisons of eigenfrequencies; allowable travel speeds can be increased by 2% or 4% for girders made of concrete C1 or C2 respectively.

3 SERIAL ROLLING STOCK ANALYSES

General

The first calculation was done for the variant C0-HEB-weit. All calculations have been performed for travel speeds from 80 km/h to 300 km/h, with a step of 5 km/h, up to a vertical structural acceleration of 6 m/s^2. The vertical structural accelerations of all node points beneath the track from x/L = 1/8 to 7/8 were considered. The individual values of the maximum to 10th highest acceleration amplitudes were stored together with the relevant trains. Fig. 6 shows a typical result diagram for a structural acceleration of 6 m/sec^2.

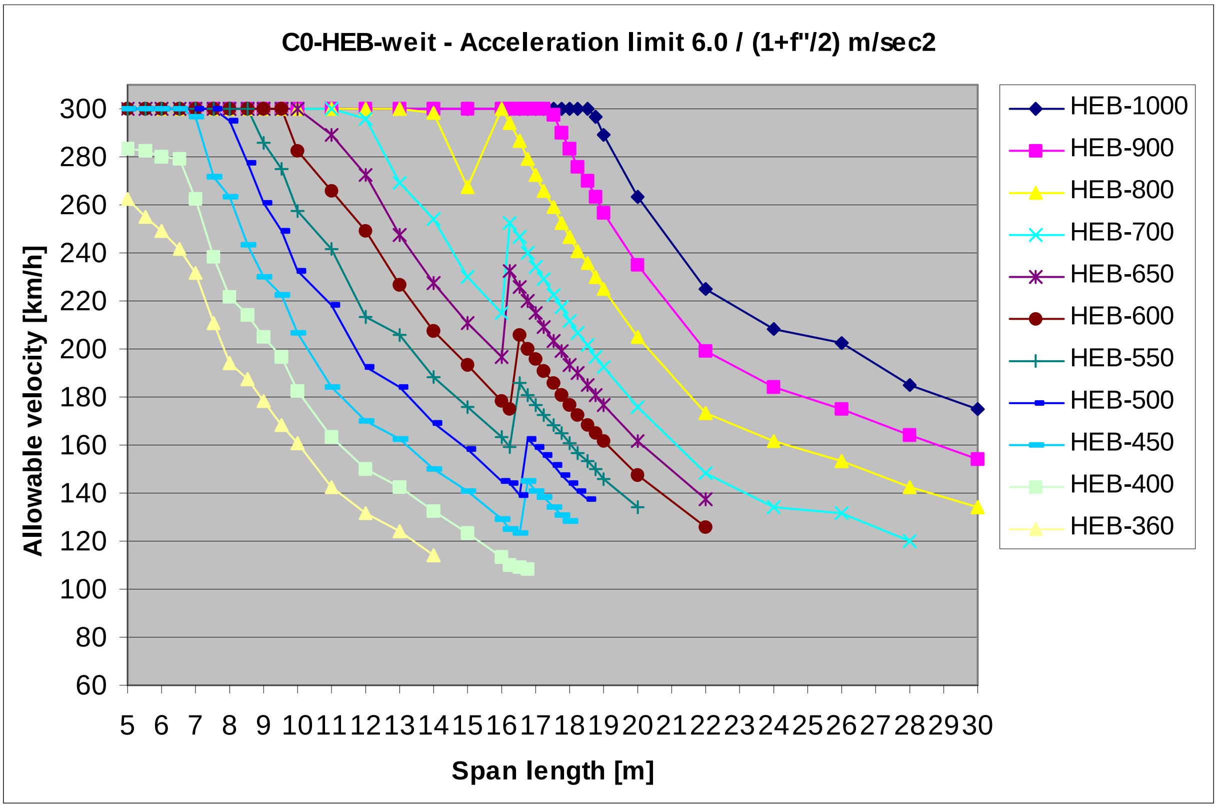

Fig. 6. Allowable travel speeds for existing structures (limit 6.0 m/s^2)

Similar diagrams have been prepared for new structures with the design code acceleration limit of 3.5 m/s^2 (Figure 7).

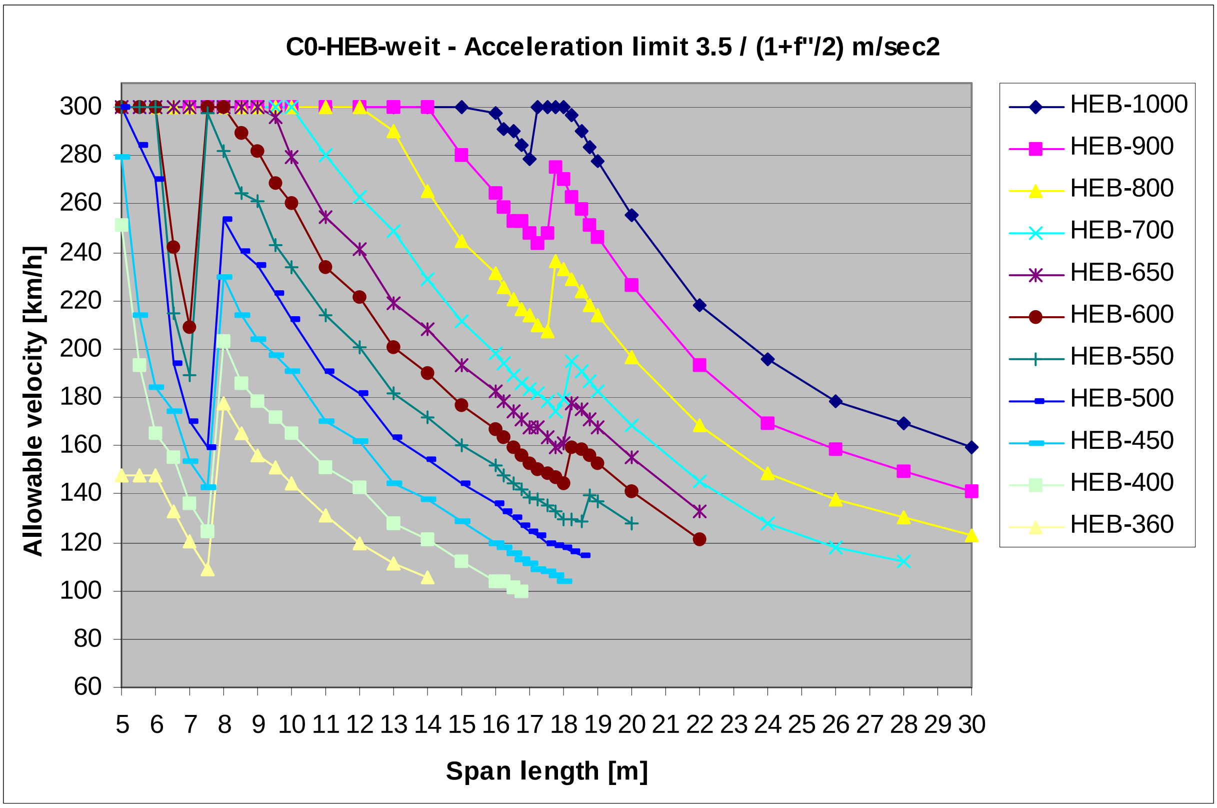

Fig. 7. Allowable travel speeds for new structures (limit 3.5 m/s^2)

In the span range above 10 m these diagrams look very similar, with the level of allowable travel speeds some 15% below the values for the 6.0 m/s^2 limit. However, in the range between 5 m and 10 m span length, resonance effects become decisive, which are not relevant for higher spans. These peaks with values between 3.5 m/s^2 and 6 m/s^2 cause a big reduction of allowable speeds for span lengths approximately 7 m, as can be seen in Fig. 7.

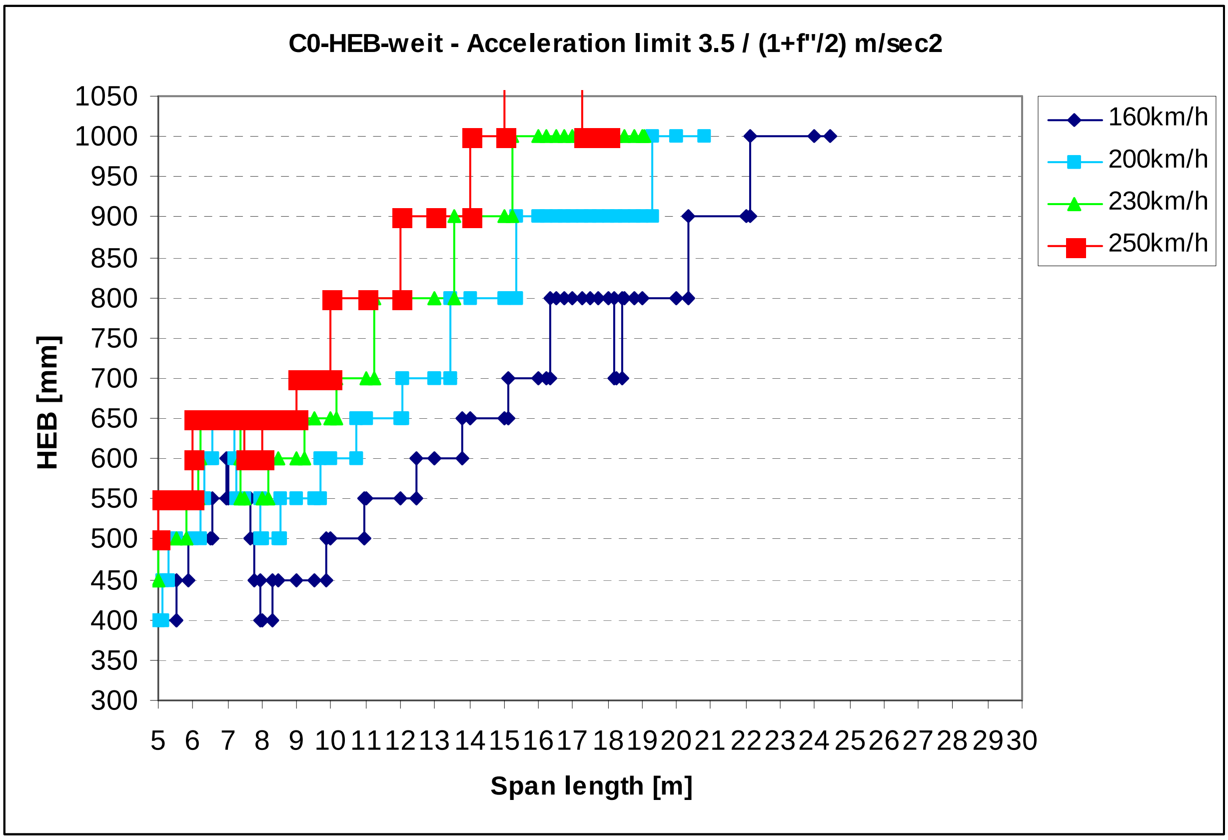

Fig. 8. Required plate depth and steel sections as a function of the span length (limit 3.5 m/s^2)

As an extension, additional diagrams have been created for the design speeds 160 km/h, 200 km/h, 230 km/h and 250 km/h (effective speeds 196, 240, 276 und 300 km/h). These diagrams show for the design speeds the minimum steel section as a function of the span length (Figure 8).

3 SUMMARY

Serial investigations of the dynamic behaviour of WIB girders from 5 to 30 m span length were used for preparing diagrams for the preliminary design process of high-speed railway bridges. These diagrams show directly the allowable travel speed for the investigated range of bridges. The huge calculation amount (some 2000 structural systems with 40 time-history calculations each) required the development of a special calculation tool. The results will be published in the national Austrian "Guidelines for Dynamic Investigations of Railway Bridges" together with the results of the partner project "ComTest" related to dynamic measurements for existing bridges [4].

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