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In-situ dynamic behavior of a railway bridge girder under fatigue causing traffic loading

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ABSTRACT: Stresses in bridges due to traffic loading need to be determined as accurately as possible for reliable fatigue safety verification. In particular, the dynamic traffic effect due to running vehicles has to be considered in a realistic way. In-situ measurements of the dynamic behavior of a one-track railway bridge have been performed to analyze the complex elastic dynamic system consisting of running trains—railway track—bridge structure. The main causes for dynamic effects for fatigue relevant bridge elements have been identified to be the vertical track position, i.e. track irregularities. The dynamic behavior has been modeled with sufficient accuracy using simple models when fatigue relevant dynamic effects are studied. The results of this study allow for the consideration of realistic dynamic amplification factors for fatigue verification.

1 INTRODUCTION

For the examination of existing bridges, dynamic amplification factors for updated traffic loads need to be derived for the deterministic verification of each the structural safety, serviceability and fatigue safety. This paper concentrates on realistic dynamic amplification factors to be deduced considering elastic structural behavior of bridge elements under fatigue loading.

Stresses in the fatigue vulnerable steel reinforcement of concrete railway bridges due to traffic loading have to be determined as accurately as possible for reliable and realistic fatigue safety verification. Therefore, the dynamic traffic effect due to running trains has to be considered in detail.

The aim of this paper is to show that the complex elastic dynamic system consisting of running train—railway track—bridge may be modeled with sufficient accuracy using simple models when fatigue relevant dynamic effects are studied. These simple models may then allow the study of the effect of other velocities and increased trainloads on fatigue relevant bridge elements.

In view of this objective, dynamic measurements are conducted on a one-track railway bridge. First, the measured behavior is compared with the dynamic behavior of a model taking into account only train velocity. Then the dynamic behavior due to railway-track irregularities is investigated with simple models.

2 MEASUREMENT OF THE DYNAMIC BEHAVIOR

2.1 *Description of bridge, railway-track vertical position, trains and measuring system*

The straight single-track bridge consists of a series of seven simply supported prestressed concrete two girder beams with two intermediate and end diaphragms. The span of the beams is 25 m. The dimensions of the cross section given in Figure 1 were confirmed by on-site measurements. For reasons of easy accessibility, the span next to one abutment was chosen for the measurements.

The concrete is un-cracked which is explained by the full post tensioning of the bridge girder.

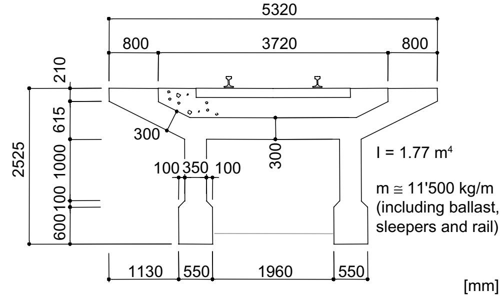

Figure 1. Measured cross sectional dimensions of the prestressed beam.

The fundamental frequency was found by interpretation of measurements and amounts to 6.0 Hz. With the measured deflection and the moment of inertia, the modulus of elasticity *E* was calculated to be around 36 GPa.

All measured passenger trains are of the type ICN (Intercity train of the Swiss Federal Railways). They consist of one or two units of seven cars each. All cars have identical axle configurations. The freight trains consist of one six-axle locomotive and tank cars transporting liquid fuels.

The bridge response allows assuming that all cars have approximately identical axle configurations and weights. Figure 2 shows also the geometric axles—bridge relation with the train positions A and B for respectively maximal and minimal static deflection.

Figure 3 shows the shape and the position of observed vertical railway-track irregularities. They consist of two depressions due to unsupported sleepers. The depths of these depressions were estimated to be 10 millimeters.

Figure 4 shows an elevation of the investigated bridge field with the locations of the measurement points. The measured items are also indicated. The deflection z(t) of the main girder at mid span (LVDT W5) and the rotation at the northern abutment (the relative horizontal displacement between main beam and the lateral wall of the abutment, (LVDT W3)) have been measured.

2.2 *Interpretation of the measured bridge response*

A typical measured dynamic deflection history is shown in Figure 5a for a passenger train and in Figure 5b for a freight train. Analysis of the measurement results reveals some oscillations of higher frequency (f~6 Hz) in both curves.

In Figure 5a a scaled up rotation history (W3) at the northern abutment is mapped in addition to the deflection history. The shapes of these two plots are rather similar. This indicates that the main girder must vibrate predominantly in its first mode.

For all measured trains the same shape is observed for the effect of each axle group meaning that the effect of an axle group seems not to

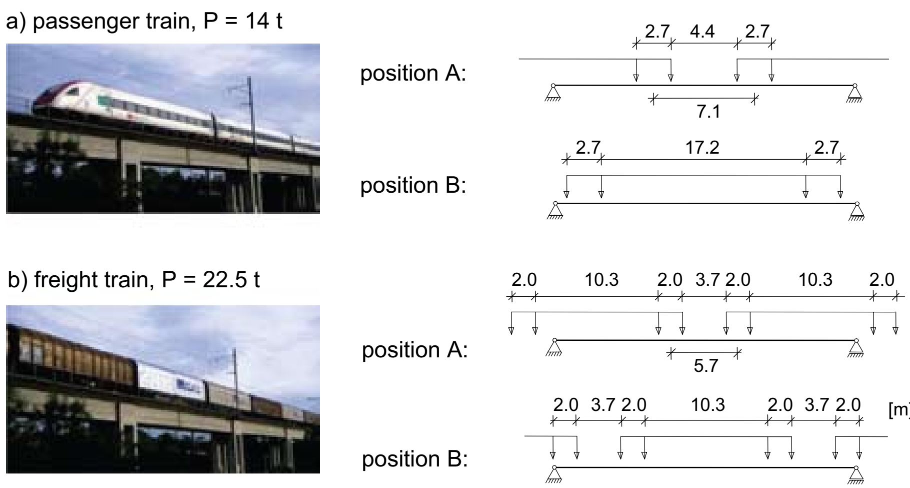

Figure 2. Passenger and freight trains with axle distances and train position A for maximal and train position B for minimal static deflection.

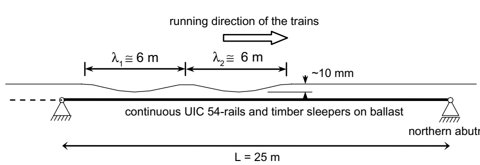

Figure 3. Observed vertical railway-track irregularities on the investigated bridge beam.

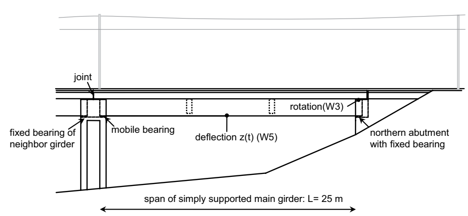

Figure 4. Elevation of the investigated bridge field with the measuring points.

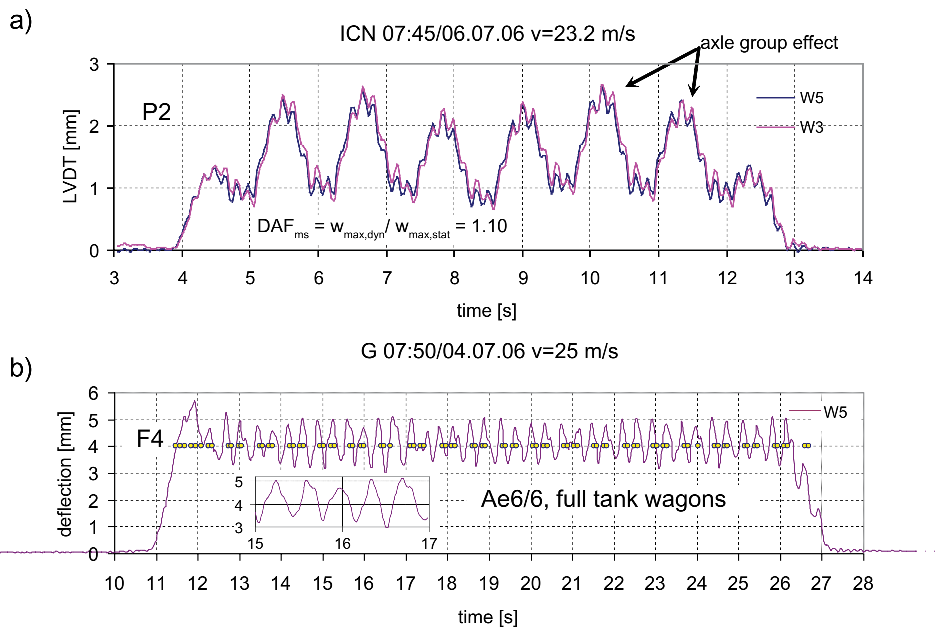

Figure 5. Typical measured mid-span deflection (W5) history under a) an ICN passenger train and b) a freight train. Supplementary, the rotation at the abutment (W3) is plotted in a).

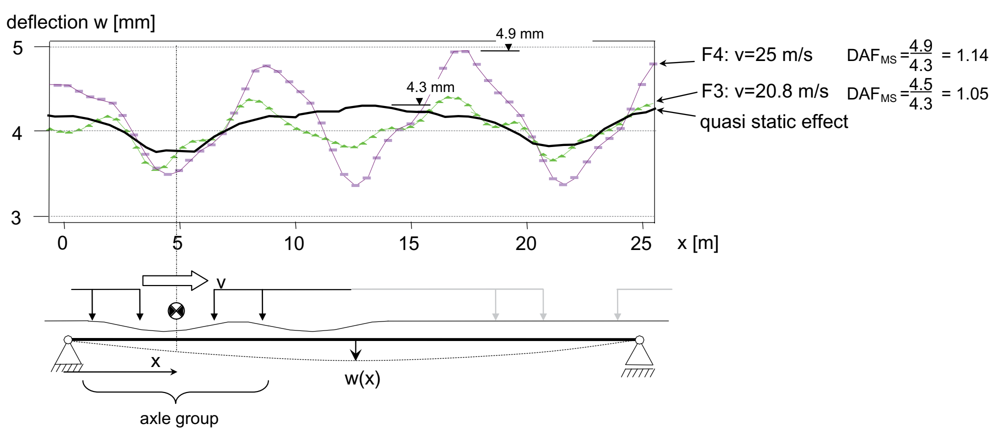

Figure 6. Axle group position versus measured deflection for two freight trains and the calculated quasi-static effect.

have any considerable influence on the effect of the succeeding axle group.

Since the dynamic effect of freight trains is more pronounced and fatigue relevant, the response of freight trains is subsequently discussed in detail. In Figure 6, the deflection is mapped as a function of the position of the axle group for two trains moving at different velocities. Additionally, the calculated quasi-static effect is drawn.

The following was observed:

Intensity of dynamic response for passenger trains: The intensity of the dynamic response (the vibration amplitude of fundamental mode oscillations) varies with train velocity. The largest effect is observed for the velocity v = 23.2 m/s (Figure 5a), which is situated in the middle of the range of the measured velocities (v = 22.4 ... 25.5 m/s).

Characteristics of dynamic response for freight trains: Two deflection peaks (maxima) can be assigned to each axle group passage of the same location on the bridge. A first peak occurs when the 1st axle reaches mid span and a second peak when the 4th axle leaves mid span. A minimum value is observed for all speeds when the axle group moves over mid span which is though in contradiction to the calculated quasi-static effect (Figure 6, in the middle). The track irregularities (Figure 3) have to be accounted for that leading to local contact force variations, which in turn lead to the observed minimum value in the bridge response. The strongest deflection variation (bridge response) is observed for the highest speed within the measured range which is due to the larger bogie force increase due to the two track irregularities and, to a minor extend, the higher resulting excitation frequency (which is closer to the bridge fundamental frequency).

3 SIMULATION OF THE DYNAMIC BEHAVIOR

3.1 *Introduction*

The dynamic behavior of the measured bridge was modeled analytically [Herwig 2008]. Aspects such as the effect of moving loads, the influence of unsprung wheel-set vibrations on the bridge vibrations, the influence of dynamic bridge behavior on the car or the dynamic effect of other velocities and masses per axle were investigated.

The most relevant railway track irregularities will be discussed in this paper. For the investigation of railway-track irregularities, the dynamic behavior of car and bridge were modeled independently based on a study of the influence of the observed bridge behavior on the car. Based on this finding, the car and the bridge are modeled as independent one-mass oscillators.

Based on experiences and results from previous studies [Ludescher 2003] in the domain of dynamic modeling of bridge—vehicle interaction, a simple dynamic model was chosen to investigate the influence of relevant parameters on the dynamic bridge response under fatigue loading.

The dynamic model consists of a one-mass oscillator [Clough 1993, Fryba 1996] established according to Figure 7. The one mass oscillator represents modal mass and stiffness of the car for vibrations in vertical direction. The modal mass corresponds approximately to ½ of the total car mass subsequently called "car body" and the stiffness corresponds to the total bogie suspension stiffness. The bridge is loaded by two such "car bodies". The natural frequency for the "car body" is assumed 1.0 Hz in the case of a passenger car and 2.25 Hz in the case of a loaded freight car.

3.2 *Car response*

First the dynamic response of the car in terms of car body displacements and contact force amplifications was investigated (simulation for the car), for introducing the forces of the axle group as excitation for the bridge (simulation for the bridge).

These simulations (for the car) showed that the dynamic bridge behavior is insignificantly altered by unsprung wheel set vibrations and that the bridge behavior has a negligible influence on the dynamic behavior of the car. Therefore, the model of Figure 7 left is accurate enough accurate for the simulations of the car. The car model is excited dynamically by imposed displacements representing the passage over railway-track irregularities.

The wavelength of the observed railway-track irregularities (6 m) is large enough such that the entire bogie of the real train is able to follow the shape and so the foot of the one-mass oscillator just follows the vertical railway-track course. The train velocity for the simulation determines, together with the railway-track, the excitation function. Table 1 shows the input data for the simulations.

Given that the bridge girder is loaded by two car halves (Figure 2, position A), the simulation is made with two one-mass oscillators that are distant from each other by the same interval as the bogies of the real train.

The displacement and force histories are represented in Figure 8 for the passenger train and in Figure 9 for the freight train.

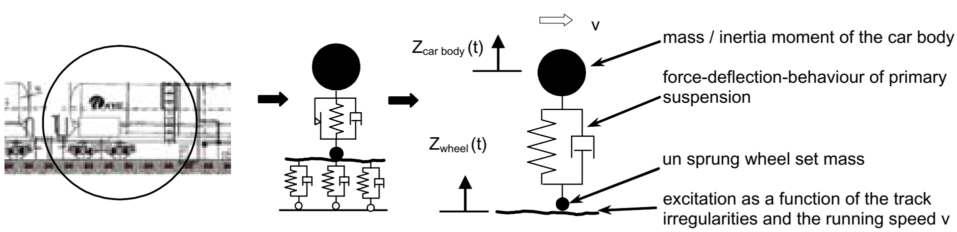

Figure 7. Simplified car model for obtaining of dynamic bogie forces (adapted from [Ludescher 2003]).

Table 1. Put data for the simulations of train passage over track depressions.

Train	v		$m_{car} = 2 \cdot P$	k_{bogie}	λ_{track}	a_{track}	$f_{exc} = v/\lambda_{track}$	f_{car}	f_{exc}/f_{car}
	[m/s]	[km/h]	[t]	[kN/m]	[m]	[mm]	[Hz]	[Hz]	[-]
ICN	22.4...25.5	81...92	28	1200	6	5	3.7...4.3	1.0	3.7...4.3
Freight	20.8...25.5	75...92	45	9000	6	5	3.5...4.3	2.25	1.6...1.9

with:
v: train velocity
m_{car}: mass per bogie
k_{bogie}: stiffness per bogie
a_{track}: track depression amplitude
λ_{track}: track depression wave length
f_{exc}: excitation frequency for a single bogie
f_{car}: uncoupled fundamental car frequency
P: mass per axle

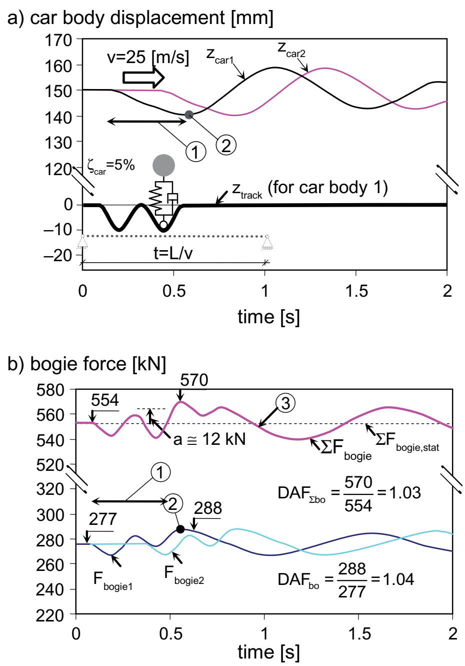

Figure 8. Simulated car body displacement and bogie force history of two adjacent passenger car halves riding over railway-track depressions with a 25 m/s speed, bogie distance = 7.1 m, ζ_{car} = 5%.

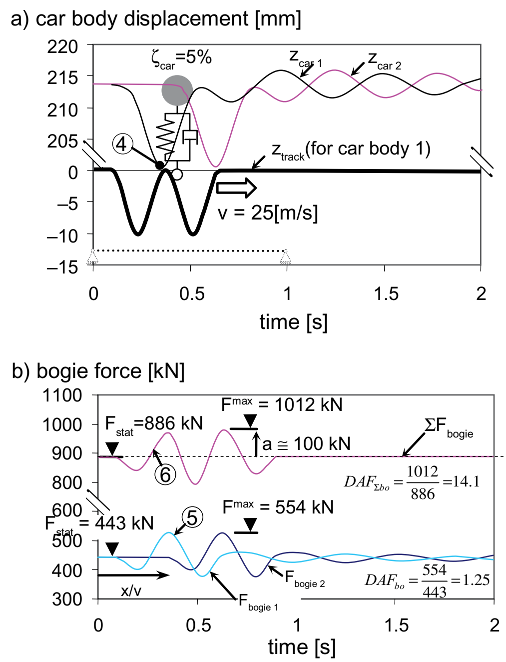

Figure 9. Simulated car body displacement and bogie force history of two adjacent freight car halves riding over railway-track depressions with a 25 m/s speed, bogie distance = 5.7 m.

As shown in Figure 8, the car body of the passenger car moves steadily downwards whilst the bogie force response follows in time with the shape of vertical railway-track position (1). In this case the main origin of bogie force variations is the imposed suspension length variation by the railway track. In order to represent the total action on the bridge, the two bogie force evolutions are added up (3). Cyclic force variation of about 4 Hz is resulting. The maximum force is 570 kN, leading to an amplification factor $DAF_{\Sigma bo}$ = 1.03.

As shown in Figure 9 and in contrast to the passenger car, the freight car body reacts faster in terms of vertical movements. The car body moves downwards together with the railway-track depression in a way such that it reaches its lowest point when the track is again up after the first depression (4). Therefore, the contact force varies considerably (5). The sum of two bogie force evolutions (6) leads to cyclic force variation of about 3 Hz and an amplification factor $DAF_{\Sigma bo}$ = 1.14, which is lower than the dynamic amplification factor for the single bogie force evolution of $DAF_{bo} = 1.25$. This is because the phase-shift between the force evolutions of the two bogies is such that maximum bogie forces do not occur at the same time.

The chosen damping ratio $\zeta = 5\%$ is rather low in the case of the passenger cars. However, since the ratio between the excitation frequency and the fundamental frequency is rather high ($f_{exc}/f_{car} = 4.2$), there is no significant influence of the damping ratio on the response.

3.3 *Bridge response*

The bridge dynamic excitation results from the global bogie force variation ΣF_{bogie} (t) of previous car simulations. It is described by the harmonic force $F_{exc}(t) = F_{stat} + F_0 \cdot \sin(f_{exc} \cdot 2\pi \cdot t)$ with the excitation frequencies $f_{exc} = 4$ Hz and 3 Hz, corresponding approximately to the frequency of the bogie force variation of previous passenger and freight car simulation.

The one mass oscillator model does not account for the distance between excitation force-application point and the centre of the effective bridge mass. [Ludescher 2003] has shown that effective mass and stiffness for the first mode in the real bridge are not considerably altered when the excitation force is acting not exactly at mid-span, but the dynamic response of the model may be slightly stronger. Figure 10 shows the bridge response due to passenger train with 4 Hz and Figure 11 the bridge response due to freight train with 3 Hz excitation frequency.

It is clearly visible that the bridge reproduces in terms of displacements and forces nearly at the same time the excitation by harmonic force.

In both cases, the amplification factor for the bridge response (DAF_{tr}) is larger than the amplification factor for the bogie forces (DAF_{bo}). This is because the "bridge" accumulates dynamic energy under the excitation by the harmonic force.

The excitation frequency resulting from the passenger-car is higher and therefore closer to the bridge fundamental frequency but the wheel force increase is relatively small. The wheel force increase of freight cars is more pronounced which leads to higher DAF_{tr} for freight cars.

3.4 *Comparison between the model and the real bridge response*

In Figure 12, the measurement result of an axle group and the simulation result for an equivalent time interval are compared. The contribution of the train velocity with moving loads is not included in the simulation result; but previous studies indicated that it is insignificant. It can be seen that the model reacts similar to the real bridge.

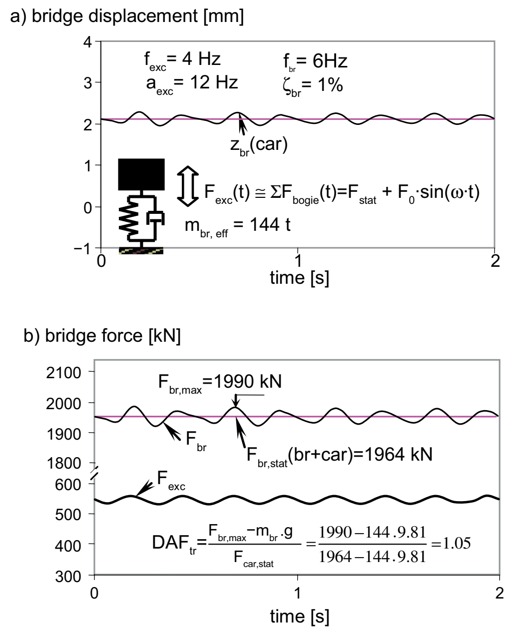

Figure 10. a) displacement and b) force history of by harmonic force excited bridge model due to the passenger train.

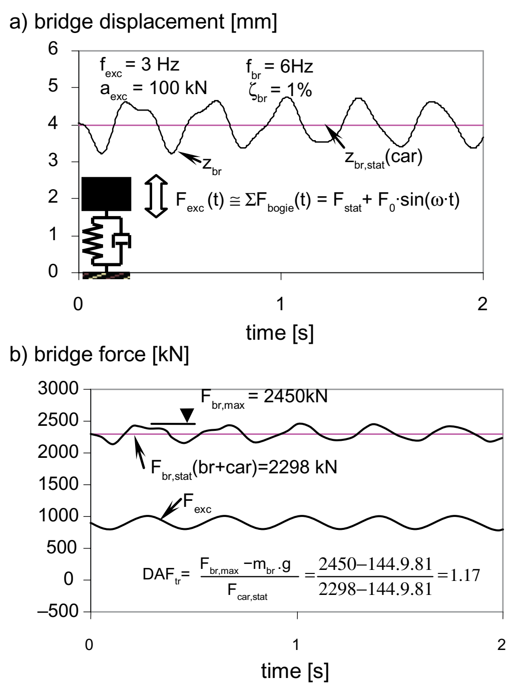

Figure 11. a) displacement and b) force history of by harmonic force excited bridge model due to the freight train.

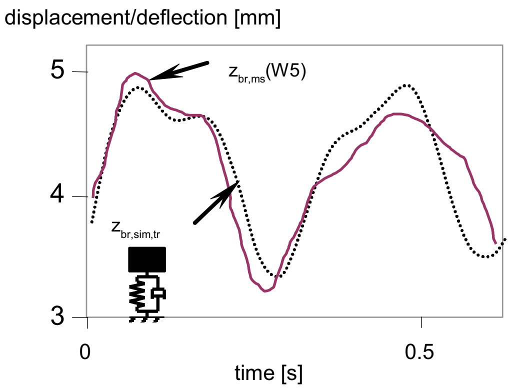

Figure 12. Comparison between the measurement result for the effect of one axle group of freight train F4 with the one-mass oscillator simulation result. (The effect of moving loads is not contained in the simulation result).

The simulation result shows how the displacement increases and decreases due to the effect of the railway-track irregularities as it was observed for the bridge response. The effect of other velocities was investigated in [Herwig 2008].

4 DISCUSSION

Modeling of dynamic behavior: The presented study shows that the dynamic behavior as measured from a prestressed concrete bridge girder is well represented by simple models which allow identifying fatigue relevant dynamic effects. It may be noticed that the dynamic bridge behavior has no significant influence on the dynamic behavior of the car because of the high difference in stiffness between the car suspensions and the bridge structure. Also, it is sufficient to represent the first vibration mode of the bridge girder. The bridge structure may be attributed to a rigid base for the car, and car and bridge may be modeled separately.

Fatigue relevant dynamic actions: Track irregularities are found to be the main cause leading to amplifications of the bridge dynamic response. Typical cases are local settlements in the transition zone embankment-bridge or a difference in elevation of the railway track due to a rail joint (misalignment) or bad welding. The wheel force variation due to railway track depressions leads to an almost immediate bridge response.

As observed in the measurements and simulations, the dynamic amplification factor varies with varying train velocity. A maximum occurs at the resonance speed for maximum car excitation due to track irregularities. High contact force amplitude leads to high action effect amplitudes. Maximum dynamic wheel forces do however not necessarily occur always at the location leading to maximum action effect such as the bending moment.

It should be noted that maximum dynamic amplifications due to both train velocity and track irregularities should actually not just be added to obtain the total dynamic amplification factor, since it is rather unlikely that the maximum dynamic effect of both effects occurs at the same time for the occasional case of a carriage with a high fatigue relevant load.

Fatigue relevant action effect: The action effect of interest (i.e. the maximum moment) is caused by more than one bogie. It must not be expected that all the bogies increase due to dynamic effects their contact force at the same time. Consequently for fatigue loading, the sum of the contact forces leads to lower amplification factors than those for single bogies.

For the investigation of the fatigue safety, the dynamic effect of high traffic loads rather than medium or lightweight cars running at high speeds are of interest.

Dynamic amplification factors for high traffic loads are distinctly lower than for trains with lighter carriages as has been shown by many investigations. In particular, wheel force amplification and corresponding action effects (forces) in the bridge element due to track irregularities decrease with increasing weight of carriage [Herwig 2008, Ludescher & Brühwiler 2009] and it may be deduced:

– In the case of dynamic amplification due to *excitation from train movement* (train velocity), maximum dynamic effects occur only with regular axle spacing in narrow velocity domains. Other velocities lead to moderate dynamic effects. Here the effect of carriage weight is less pronounced.
– In the case of dynamic effects due to *track irregularities*, one needs to consider that the track quality varies over time, and since the overloaded carriage (as leading action) is an *occasional* event, it is reasonable to consider track irregularities as a quasi-permanent state.

Application: As a consequence and since the static load considered in the FLS verifications is extreme (high), the dynamic amplification factor according to EN 1991-2 may be reduced accordingly. At fatigue limit state FLS, frequent values of dynamic action effects are considered to represent service load conditions. Based on the foregoing considerations, the following dynamic amplification factor φ_{FLS} is suggested [SB4.3.3, 2007]:

$$\varphi_{FLS} = 1 + 0.5(\varphi' + 0.3\varphi'')$$

with φ' and φ'' according to EN 1991-2.

5 CONCLUSIONS

The following conclusions are valid for the investigated bridge as well for fatigue relevant shorter bridges:

– The dynamic behavior of a bridge can be represented by simple models when fatigue relevant action effects are studied. It is sufficient to represent the first vibration mode of the bridge girder.
– Track irregularities are the most important cause for fatigue relevant dynamic amplifications of action effects. This effect occurs at each train passage for each car.
– Dynamic amplification factors for elastic structural behavior at fatigue limit state shall consider the effects of two main parameters involved, i.e. (1) train velocity and (2) track irregularity.

The investigated prestressed concrete bridge girder itself is actually only little fatigue vulnerable due to its relatively long span and the fully prestressed concrete cross section. Since the investigated bridge was rather stiff, the assumptions on the modeling are however applicable for bridges of shorter spans.

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