NUMERICAL MODELING OF TRAIN-TRACK-INTERACTION AT BRIDGE TRANSITION ZONES CONSIDERING THE LONG-TERM BEHAVIOUR

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Abstract. *A train passage will induce elastic and plastic deformations in the ballast and its underlying soil. Considering the short term behaviour the elastic deformations will overweight the plastic and therefore these can be neglected. Regarding the long term behaviour the plastic deformations will accumulate due to repeating traffic loads and lead to settlement and deterioration of the track structure. A smooth settlement along the track (small differential settlement) is acceptable, whereas large differential settlement accelerates the process of deterioration and reduces the safety and serviceability of the track.*

Especially a bridge transition zone is prone to large differential settlements, where the track deteriorates in the backfilling area of bridge abutments mainly due to stiffness changes between embankment and bridge, repeating traffic loads and displacements of the superstructure ends on bridges.

A numerical model will be presented, with a particular focus on the long term behaviour of the track at bridge transition zones under repeating traffic loads. The aim of the simulation is not the computation of every train crossing but the analysis of the long term behaviour of the bridge transition zone based on single train crossings.

The analysis is performed in the time domain, whereas the bridge, the track and the backfilling area are modelled by means of the finite element method (FEM), which can also take nonlinearity of single components into account.

1   INTRODUCTION

A specific section of the track structure is a bridge transition zone. In general a bridge transition zone is a discontinuity in the track structure that can lead to increased track degradation. Technical causes of the track degradation problem of bridge transition zones are [1]:

- a rapid change in the supporting stiffness,

- relative displacements between bridge girder and abutment and

- rail heave due to bridge girder deflection.

Sketches of these are depicted in Figure 1. Due to the rapid change in the supporting stiffness settlement will occur over a short distance (differential settlement) in the embankment under traffic loads which will accelerate the process of track degradation [2, 3].
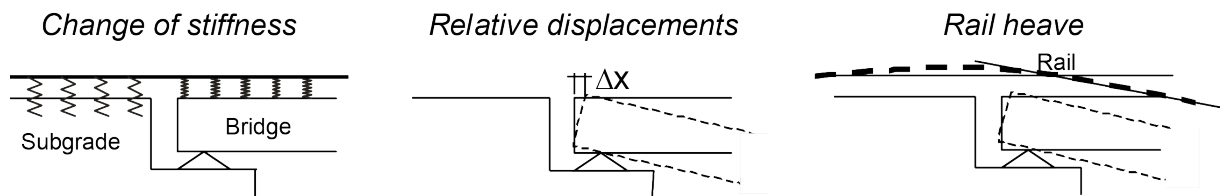

Figure 1: Causes for the track degradation at bridge transition zones.

Relative displacements between bridge girder and abutment can have their causes in temperature fluctuations, acceleration and braking processes or in the deflection of the bridge under traffic load. They lead to a decompaction of the ballast for ballasted tracks and for slab tracks there will be large tractive forces. In addition there will be a rail heave due to the bridge girder deflection under traffic load. According to [1, 4] the increment of settlement increases largely with the associated unloading of the sleepers.

Due to the discontinuities in the bridge transition zone additional dynamic loads will be applied on the system. These will lead to an acceleration of track degradation because of the interaction between train, track and bridge. A change of the track level will lead to a different train-track interaction. Therefore, the embankment is also of interest for an assessment of a bridge transition zone.

In this article a numerical model is presented to describe the dynamical behaviour of the track structure. A particular focus lies on the change in the supporting stiffness between the embankment and the bridge structure. To mitigate differential settlement from the embankment to the bridge different approaches are available. Therefore, assessment criteria for settlement are presented and the effects of different approaches for bridge transition zones on these criteria are compared.

2   ASSESSMENT CRITERIA FOR SETTLEMENT

A railway track will settle as a result of permanent deformation in the ballast and the underlying soil due to the repeated traffic load. To quantify the total settlement of a ballasted track, the accumulated plastic deformations must be estimated for each load cycle or else extrapolated from one cycle. There exist different criteria from which the settlement can be estimated. The criteria can be generally classified in three different groups. These are:

Criteria based on single variables of the superstructure

Single variables are used to estimate the track settlement, e.g. the dynamic force on the sleeper F_{sl} [5] or the deflection of the sleeper u_{sl} [6]. Using the dynamic force F_{sl}, the total settlement s_n after N cycles for a ballasted track can be estimated by:

$$s_n = s_c F_{sl}^a (1 + c \ln(N)), \tag{1}$$

with s_c, a and c as parameters. The total settlement depends mainly on the number of load cycles and the dynamic force on the sleeper. According to [6] the settlement law can be written in an incremental form, defined as:

$$\frac{\mathrm{d}s}{\mathrm{d}N} = \alpha u_{sl}^{\beta}, \tag{2}$$

with α and β as parameters. The total settlement for a ballasted track is then computed by integrating Eq.(2) over a number of load cycles.

It should be mentioned that the parameters in Eq.(1) and Eq.(2) need to be adjusted for the analyzed track section. An overview of empirical settlement laws for railway tracks is given in [7].

Criteria exceeding a given threshold value

A common criterion for the permanent deformation in the underlying soil is the effective strain γ_{eff} [8]. If the effective strain in the soil is exceeding a given threshold value (the so-called Vucetic criterion), permanent settlement in the soil can be expected. According to [9], first the resulting effective vibration velocity $v_{\mathrm{eff,z}}$ in a depth z is computed:

$$v_{\mathrm{eff,z}} = v_{\mathrm{eff,sl}} \cdot \exp(-z/a), \tag{3}$$

with $v_{\mathrm{eff,sl}}$ as the resulting effective vibration velocity of the sleeper and a as track parameter. From there then the effective strain in a depth z can be computed as:

$$\gamma_{\mathrm{eff}} = \frac{v_{\mathrm{eff,z}}}{c_s}, \tag{4}$$

with c_s as shear wave velocity of the soil. An alternative approach is to compare the Vucetic criterion with the second invariant of the strain tensor [10], which is defined as:

$$\gamma_{\mathrm{eff}} = \sqrt{\frac{2}{3}\left((\epsilon_{xx} - \epsilon_{yy})^2 + (\epsilon_{yy} - \epsilon_{zz})^2 + (\epsilon_{xx} - \epsilon_{zz})^2\right) + \gamma_{xy}^2 + \gamma_{yz}^2 + \gamma_{xz}^2}. \tag{5}$$

If the effective strain γ_{eff} is exceeding the Vucetic criterion, then the permanent settlement can be estimated by an approximate solution, e.g. [11].

Another criterion is given in [1], where a threshold value for the resulting effective vibration velocity of the sleeper is given, above which permanent settlement for a ballasted track can be expected ($v_{\mathrm{eff,sl}} \geq 18$ mm/s).

An appropriate constitutive law as criterion

Another strategy is to use an appropriate constitutive law to determine the permanent deformations within the ballast and embankment, whereby every load cycle needs to be computed. However, the computation of every cycle is not recommended for problems involving a large number of cycles because of the accumulation of numerical errors. In this respect, it shall be noted that there also exist the so-called explicit accumulation approaches, which determine the long-term behaviour based on the computation and extrapolation of selected single cycles [12].

3 NUMERICAL MODEL

A 3D numerical model has been developed in the time domain to analyze the change in the supporting stiffness between the embankment and the bridge structure and to assess different strategies for transition zones to mitigate the differential settlement. The track, consisting of the superstructure (rail, rail pads, sleepers and ballast), a bridge and the underlying soil, has been modelled by means of the FEM in MATLAB. It is assumed that the material behaviour is linear elastic and the track is symmetrical, so only a half of each track component needs to be modelled. The FEM mesh of the track and of the bridge structure are depicted in Figure 2.
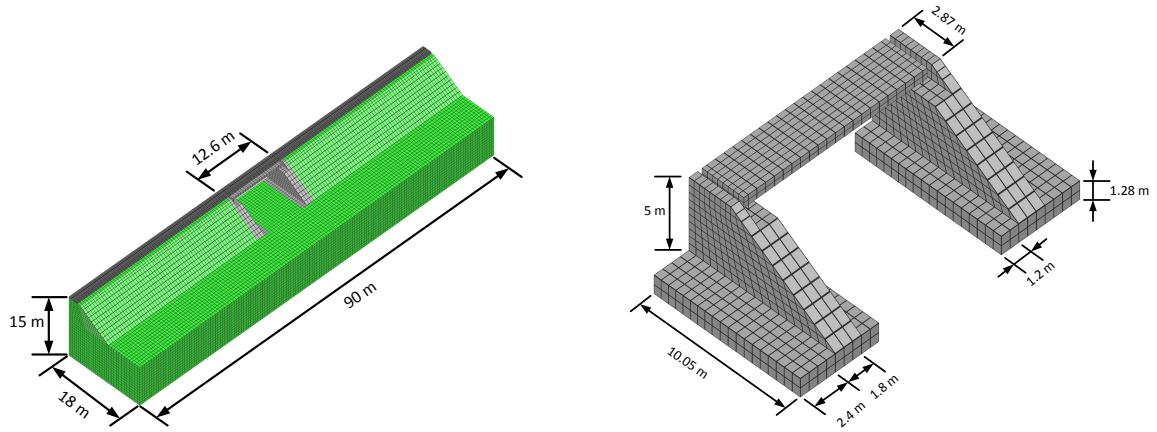

Figure 2: Mesh of the track (left) and bridge section (right).

The total length of the track is 90 m, consisting of 151 sleepers equally spaced, resulting in a distance of $d = 0.6$ m between two consecutive sleepers. The rail has been modelled as Euler-Bernoulli beam elements of 0.15 m length with a bending stiffness of $EI = 6.32 \times 10^6$ Nm^2 and a mass per unit length of $m = 60.3$ kg/m. The rail pads have been modelled as spring-damper elements with a stiffness value of $k_{rp} = 150 \times 10^6$ N/m and a damping value of $c_{rp} = 13.5 \times 10^3$ Ns/m. The sleepers have also been modelled as Euler-Bernoulli beam elements with a total mass of $m = 338$ kg. The ballast, embankment, soil, bridge and abutment have been modelled as 8-node solid elements and their parameters are given in Table 1.

In order to avoid reflections from the truncated FEM domain, local transmitting boundaries have been implemented [15]. The bridge deck is supported by the abutments, with one end fixed and the other end movable. From the bearings, the bridge has an overlap of 0.3 m on each side and from the end of the bridge a gap of 0.3 m to the abutments. For the bridge a damping

Layer	Young's Modulus (N/m^2)	Poisson's ratio	Density (kg/m^3)	Thickness (m)
Ballast	200×10^6	0.2	1500	0.35
Embankment	100×10^6	0.3	1700	5
Soil	100×10^6	0.3	1700	9.65
Bridge	34×10^9	0.2	2500	0.7
Abutment	20×10^9	0.2	2500	

Table 1: Track parameters [13], [14].

ratio of 2 % has been chosen, which is described by the rayleigh damping. Interface elements have been implemented between the abutments and the embankment to permit a vertical relative displacement between them [16]. The time step for all computations has been chosen to $\Delta t = 2 \times 10^{-4}$ s. To this point the vehicle has been considered as moving vertical loads with an axle load of $P_0 = 196.15$ kN, where the axles of a bogie have a distance of $d_a = 3$ m and the bogies a distance of $d_b = 11.46$ m. These parameters are consistent with an ICE 2 traction car. For all simulations only one passage of the traction car has been considered.

4   THE BRIDGE TRANSITION ZONE

In the following, the problem of the change in the supporting stiffness between the embankment and the bridge structure is analyzed where the focus is lying on the assessment criteria for settlement, the sleeper displacement (Eq.(2)), the vibration velocity of the sleeper (Eq.(4)), the force on the sleeper (Eq.(1)) and the effective strain (Eq.(5)). Further on, the effects of different approaches for bridge transition zones on these criteria are analysed and compared.

4.1   ASSESSMENT OF THE TRANSITION PROBLEM

The passage of the moving loads has been performed for three different velocities ($v_0 = 20, 60, 93.75$ m/s). Figure 3 shows the maxima of the four different assessment criteria along the track for the different load velocities.

The criterion **sleeper displacement** (upper left) shows that the displacement increases before and behind the bridge for higher train velocities. The displacement on the embankment is in a range between 1.1 mm for $v_0 = 20$ m/s and 1.3 mm for $v_0 = 93.75$ m/s and nearly symmetrical around the bridge. On the bridge the sleeper displacement (relative displacement between sleeper and bridge) is nearly equal for all the velocities. The difference between the displacement on the embankment and the bridge is approximately 1 mm. This indicates the abrupt stiffness change between the embankment and the bridge structure. The criterion **vibration velocity of the sleeper** (upper right) shows great differences between the different train velocities (from 10 m/s for $v_0 = 20$ m/s to 55 m/s for $v_0 = 93.75$ m/s on the embankment). The vibration velocity reduces on the bridge for all the train velocities. For the velocity of $v_0 = 20$ m/s the vibration velocity is nearly symmetrical around the bridge. For the higher velocities a peak of the vibration velocity is observed on the embankment behind the bridge. For the velocity of $v_0 = 93.75$ m/s the vibration velocity shows an increase of approximately 30%.

The criterion **force on the sleeper** (lower left) shows a difference of approximately 2 kN for the different velocities on the embankment. On the bridge the force amplitude increases by approximately 15 % and is nearly the same for the different velocities. Again, this increase indicates the stiffness change between the embankment and the bridge structure. Also the force is nearly symmetrical around the bridge for all the velocities.

The criterion **effective strain** (lower right) increases for higher train velocities. In a depth of 1.35 m the effective strains show an increase from 3.1×10^{-4} to 3.7×10^{-4} for the velocities of $v_0 = 20$ m/s and $v_0 = 93.75$ m/s, respectively. In a depth of 4.35 m the effective strains for these velocities reduce to 1×10^{-4} and 1.3×10^{-4}. Towards the bridge structure the effective strain increases, whereas the effective strain is for the lowest train velocity nearly symmetrical around the bridge. For the higher velocities a peak is also observed behind the bridge structure. For the velocity of $v_0 = 93.75$ m/s an increase of nearly 30 % is observed at a depth of $z = 1.35$ m.

In summary it can be noted that differential track settlements will be produced between the embankment and the bridge structure due to the abrupt stiffness change and to the fact that the
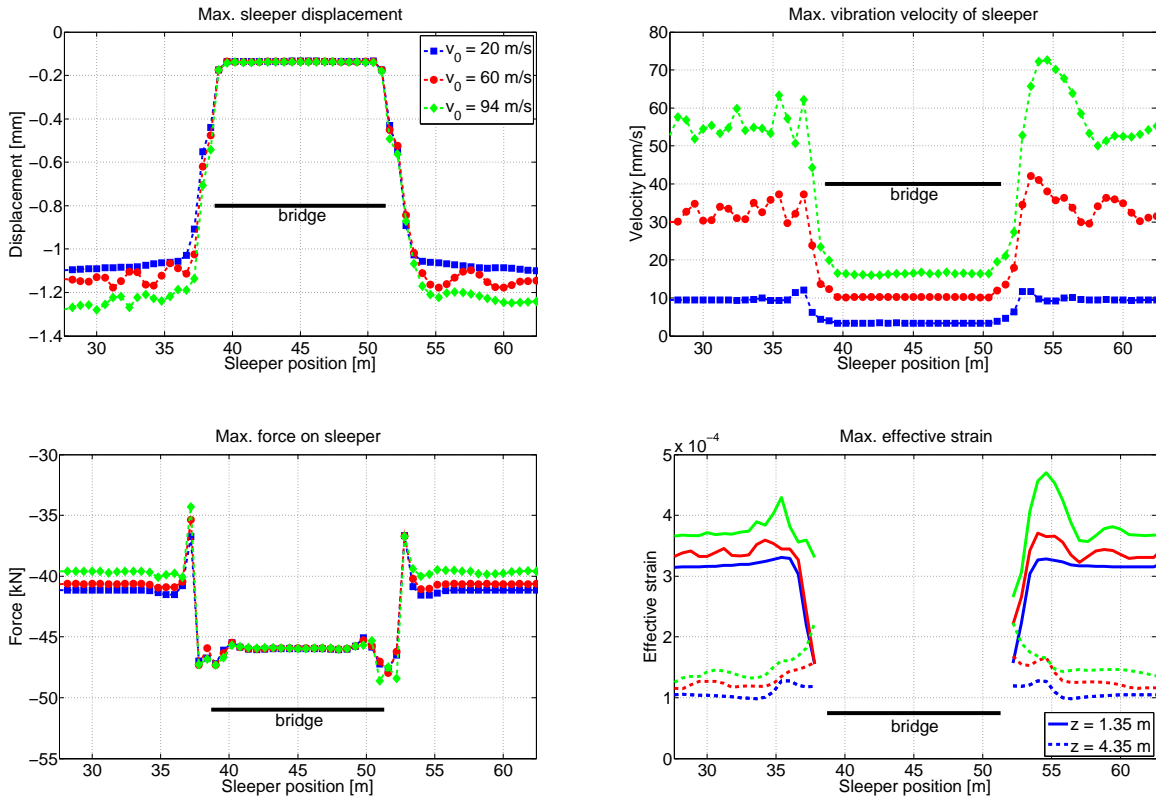
Figure 3: Maximum sleeper displacement (upper left), maximum vibration velocity of the sleepers (upper right), maximum force on sleeper (lower left) and maximum effective strain in two different depths (lower right) for $v_0 = 20, 60, 93.75$ m/s between 27 m and 63 m.

embankment is susceptible to settlement while the bridge structure is not.

The criteria sleeper displacement and force on sleeper are relatively insensitive to the influence of entering or leaving the bridge and of the different train velocities. This may be explained by the fact that they do not take the influence of the underlying soil sufficiently enough into account. Therefore, a complete assessment of the whole track should also include an assessment of the underlying soil in addition to that of the superstructure.

4.2 DIFFERENT APPROACHES FOR BRIDGE TRANSITION ZONES

In order to mitigate the abrupt stiffness change from the embankment to the bridge structure, different approaches are available to smooth the transition. In the following, the effects for a small selection of different approaches is presented. These are:

- A 0.5 m thick rigid transition slab of 6 m length before and behind the bridge (same material parameters as the abutment, see Table 1).

- Under ballast mats over the whole track (stiffness of $k = 0.1$ N/mm^3).

- Backfilling area before and behind the bridge (slope of $1 : 1$, $E = 1.5 \times 10^8$ N/m^2, $\rho = 2000$ kg/m^3, $\nu = 0.3$).

The approaches are depicted in Figure 4. For this comparison only the velocity of $v_0 = 93.75$ m/s has been considered because its effects on the transition zone with no optimisation were the largest.
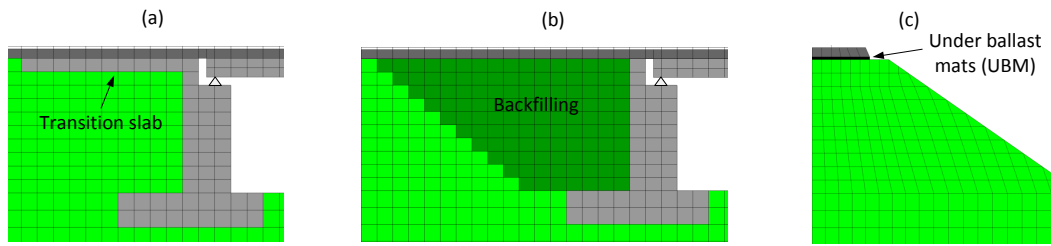
Figure 4: Approaches for transition zones: (a) transition slab, (b) backfilling area, (c) under ballast mats (UBM).

In Figure 5 the assessment criteria for settlement are plotted for the different optimisation strategies as well as for the transition zone with no optimisation.

The transition slab and the backfilling area reduce the **sleeper displacement** (upper left) before and behind the bridge structure, which indicates a modification of the track stiffness on the embankment. However, the transition onto the slab takes place more suddenly than onto the backfilling area. For the under ballast mats the displacement is only shifted to higher displacements.

The **vibration velocity of the sleeper** (upper right) is reduced on the transition slab, whilst immediately behind the slab an increase of the vibration velocity is observed. This indicates that the problem of the transition is only shifted to the end of the slab. For the under ballast mats the vibration velocity is only shifted to higher velocities. For the backfilling area the vibration velocity is reduced, while behind the bridge the peak in the vibration velocity is still observed
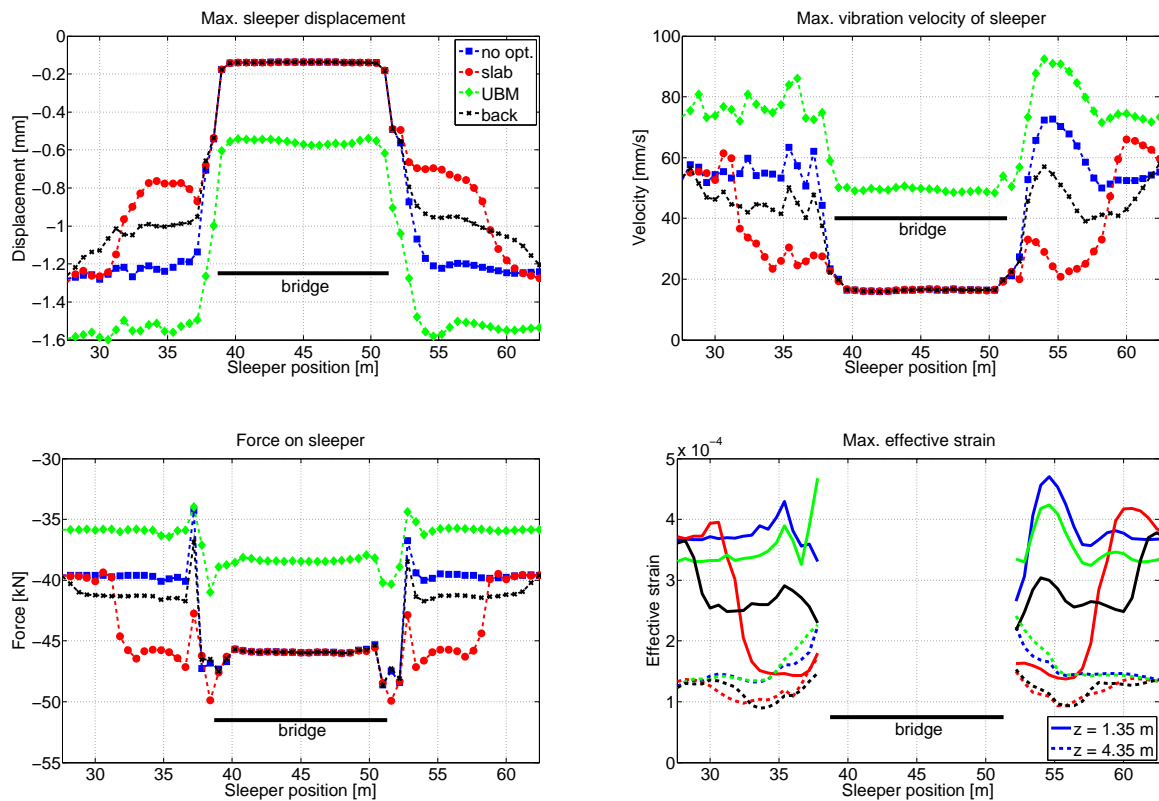

Figure 5: Assessment criteria for $v_0 = 93.75$ m/s between 27 m and 63 m for different approaches (no optimisation = blue square, transition slab = red dots, under ballast mats = green diamonds, backfilling area = black crosses).

but with a smaller magnitude than for the transition zone without optimisation.

On the transition slab and the backfilling area the **force on sleeper** (lower left) is increased, which indicates also the track stiffness modification, whereby the increase of the force onto the transition slab is more sudden than onto the backfilling area. The under ballast mats reduce the force on the sleeper over the whole track.

For the **effective strain** (lower right) a reduction takes place under the transition slab but directly behind the slab an increase is observed, which again indicates that the problem is only shifted to the end of the slab. The under ballast mats reduce the effective strain at a depth of $z = 1.35$ m over the whole track and at a depth of $z = 4.35$ m the effective strain shows hardly any change. Also in the backfilling area the effective strain is reduced which is a result of the stiffer material of the backfilling material.

In can be concluded that the transition slab only shifts the problem to the end of the slabs. The under ballast mats increase the sleeper displacement and the vibration velocity of the sleeper but reduce the force over the whole track as well as the effective strain. The backfilling area smooths the transitions between the embankment and the bridge structure.

4.3   AN OPTIMISED BRIDGE TRANSTION ZONE

In order to smooth the transition and to fit the track stiffness between the embankment and the bridge, a transition zone is chosen with a backfilling area (slope 1:2) and under ballast mats placed only on the bridge. In Figure 6 the results for the train velocity of $v_0 = 93.75$ m/s are shown.
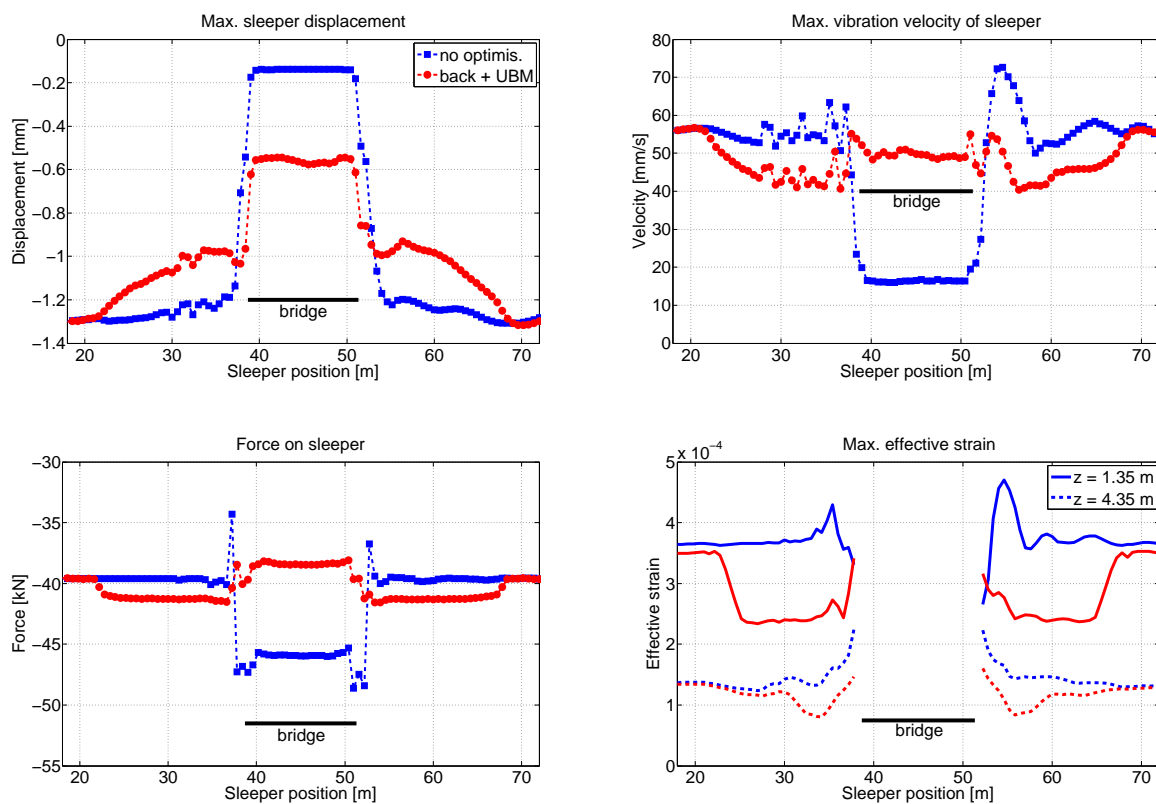

Figure 6: Assessment criteria for $v_0 = 93.75$ m/s between 18 m and 72 m for the transition zone with no optimisation (blue square) and with a backfilling area + under ballast mats on the bridge (red dots).

Due to the backfilling area the **sleeper displacement** (upper left) is reduced slightly on the embankment and on the bridge the sleeper displacement increases due to the under ballast mats. The displacements show a difference of approximately 0.4 mm between the embankment and the bridge structure, whereas for the transition zone without optimisation the difference is approximately 1 mm.

The **vibration velocity of the sleeper** (upper right) is also reduced on the embankment due to the backfilling area and increases on the bridge due to the under ballast mats. As a result of the optimisation, the vibration velocity of the sleeper has been leveled and is in a range of $v_{sl} = 42 - 58$ mm/s, whereas the vibration velocity of the transition zone without optimisation is in a range of $v_{sl} = 18 - 73$ mm/s.

The same can be said for the **force on the sleeper** (lower left), where the range is between $F_{sl} = 38 - 42$ kN for the optimised transition zone and the range for the transition zone without optimisation is between $F_{sl} = 34 - 48$ kN.

The **effective strain** (lower right) is also reduced in the backfilling area because of the stiffer material and no increase at the beginning or the end of the backfilling area is observed.

As result it can be stated that an insertion of a backfilling area smooths the transition towards the bridge structure and can mitigate differential settlement in the transition zone. The under ballast mats on the bridge levels the transition from the backfilling area onto the bridge structure.

5 DISCUSSION & OUTLOOK

In this study, several assessment criteria for settlement have been presented as well as their sensitivity to different optimisation strategies for bridge transition zones. The obtained results so far indicate that the velocity of the moving loads has a minor influence on the criteria sleeper displacement and force on the sleeper. Also the moving direction of the loads plays a minor role for these criteria. In contrast, the criteria vibration velocity of the sleeper and effective strain are sensitive for the velocity and showed an increase up to 30% when the loads are leaving the bridge. This contrast indicates that at least one criterion for the superstructure and one criterion for the underlying soil should be used to assess the whole track structure at bridge transition zones. For a final assessment the influence of the vehicle (wheel, bogie, car) and of additional dynamical loads (wheel out of roundness, etc.) need to be addressed because they will have a significant influence at least on the sleeper displacement and the force on the sleeper. In this respect, the influence of the bridge structure (relative displacement between girder and abutment, rail heave, etc.) also needs to be taken into account.

For an optimisation of the bridge transition zone the transition slab only shifts the problem to the end of the slab. In contrast, the under ballast mats increase the displacement as well as the velocity of the sleeper but reduce the force on the sleeper and the effective strain in the soil. This means that the general problem of a transition still exists. The backfilling area smooths the transition between the embankment and the bridge structure and reduces the magnitudes of the peaks. A combination of a backfilling area and elastic elements seems to be an effective optimisation strategy but more investigations on this topic need to be done.

In the future works, a feed-back loop for the long-term behaviour of the track shall be implemented to evaluate the evolution of the deformed track and its behaviour. This shall lead to improved optimisation strategies for bridge transition zones.

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