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Finite element analysis of bridge transition zone for investigating the effect of moving loads

Analyse par éléments finis de la zone de transition des ponts ferroviaires pour étudier l'effet des charges en mouvement

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ABSTRACT: The problem of track-bridge interaction and its modeling issues have increasingly been emphasized and highlighted due to the presence of high-speed rails. The quality criteria for high-speed rail lines must be stricter than those for conventional rail lines to assure passenger comfort and safety. Experience shows that in transition zones when a train rides from an embankment onto a stiff structure, such as a bridge, tunnel or culvert, an abrupt change occurs in the support stiffness possibly inducing differential settlements. Examples prove that inadequate technical solutions can generate damage that may require long term speed restrictions or lead to short maintenance cycles, significantly increasing the total cost of ownership.
The problems associated with the transition zones require complex analysis. The author reports the results obtained by the investigation of a 3D numerical model of a bridge transition zone by means of an advanced constitutive model and time domain analysis. The mechanical behavior of a railway bridge and its soil environment is presented in the study with special regards to factors like train speed, embankment height and the settlement differences apparently developing in the transition zone.

RÉSUMÉ: Le problème de l'interaction de voie-pont et ses points en question de modélisation sont de plus en plus mis en évidence en raison de la présence des voies ferroviaires à grande vitesse. Les critères de qualité pour les lignes à grande vitesse doivent être plus strictes que celles des lignes ferroviaires conventionnelles afin d'assurer la sécurité et le confort des passagers. Les expériences ferroviaires montrent que dans la zone de transition quand un train chevauche d'un remblai sur une structure rigide, p.ex. un pont, tunnel ou un ponceau, un changement brusque de la raideur du système d'appui s'effectue induisant éventuellement des tassements différentiels. Plusieurs exemples prouvent que des solutions techniques inadéquates peuvent générer des dommages en exigeant les limitations de vitesse à long terme ou conduisant à courte maintenance de cycles et donc augmenter sensiblement le coût total de possession.
Les problèmes associés avec les zones de transition exigent une analyse complexe. L'auteur présente des résultats obtenus par l'analyse des effets dynamiques des charges ferroviaires avec un modèle numérique avancé 3D dans une zone de transition de pont ferroviaire. Le comportement mécanique d'un pont ferroviaire et le comportement géotechnique des terrains mous argileux avoisinants sont présentés dans l'étude spéciale en focalisant des facteurs comme la vitesse de train, de hauteur de remblai et des tassements différentiels apparus dans la zone de transition.

Keywords: transition zone; Plaxis 3D; moving train; differential settlement; dynamic effect

1 INTRODUCTION

High speed trains raise new types of problems due to the increased vehicle velocities and it is easily understood that the quality criteria for high-speed lines must be much more restrictive than those for conventional rail lines to assure passenger comfort and safety. Transition zones are, nowdays, one of the biggest problems of railways that deserve special attention from the rail engineering community (Di Mino, Di Libeto, 2007).

The transition zones can be defined as the regions along the track characterized by the presence of an abrupt variation of its stiffness. This occurs at bridge approaches (most common), level crossings, special track works, tunnels and viaducts (Varandas, 2014).

Figures 1 and 2 show respectively, a schematic profile of a track-bridge approach showing the development of the differential settlement and an unlevelled track-bridge approach evidencing the unevenness in the track geometry (Paixao, 2014). Free track generally experience a higher total settlement within its lifetime caused by soil creep, traffic condition and soil-water response. The total settlement of the train track is negligible in engineering structures such as culverts, bridges, and turnouts, especially in structures with pile foundations. This circumstance results in a differential settlement in the interaction zone between the structure and its approach.

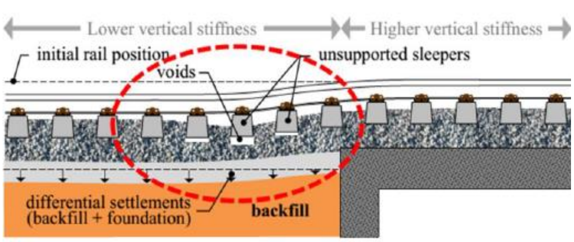


Figure 1. Schematic profile at a track-bridge approach in a differential deformation condition.

Uneven stiffness and damping between bridge and its approach, geotechnical causes, and soil-water response are the key primary causes of track degradation at the bridge transition zone (Kerr & Moroney 1993).

Train load, train speed, traffic condition, height of embankment, type of abutment, and bridge joint are the key secondary causes that can affect differential settlement at the bridge transition zone (Gallage et al. 2013).

Figure 2. Unlevelled track at a track-bridge approach.

In bridge design Finite Element Method is the most commonly used tool for the calculation of stresses and strains in structures. The analysis of the superstructure and the substructure as well as certain foundation problems are usually appear as separate tasks in the design work flow. The most commonly used FEM codes frequently use the linear elastic constitutive model for modelling soil behavior, which is a rough approximation.

Using up-to-date three dimensional geotechnical FEM programs the behavior of soil environment condition can be modelled more accurately applying advanced constitutive models. The effects of soil and initial stress conditions, unloading-reloading phases and the soil-structure interaction can be considered as well. The dynamic modul of the latest softwares allow of investigating the wave propagation in the subsoil and their effects on the structure.

The problems associated with the transition zones require a complex analysis. For efficient modelling of the mechanisms resulting in gradual line deteriorations in the transition zones the understanding of the 3D and dynamic effects associated with the problem seems to be essential. To enhance our understanding regarding the problem a 3D numerical model has been developed and presented for time domain analysis.

2 MODELING APPROACH

2.1 Railway bridge and its surroundings

The effect of moving of a passing single bogie over a bridge has been analyzed using the PLAXIS 3D Dynamic package (Brinkgreve et al. 2010). Figure 3 shows a section of the bridge and the soil profile. The top 10 m of the subsoil in the current model was soft clay, resting on 10 m of stiff sand. On top of the soft layer, a medium dense sand embankment and a dense backfill was built to support the railway line.

The length of the backfill zone was 15.4 m. The height of the embankment was H=3.6 m and 7.2 m, with a slope of 1:1.5. The ballast layer was designed with 0.35 m thickness. The ground-water table was assumed to be at the top of the soft clay layer. The abutments rest on piled foundation and wingwalls are attached to them. The length of the 10 piles (per abutment) with the diameter of 0.8 m, which were placed in two rows with a distance of 2.5 m, was 15.0 m. The span of the bridge was 17.6 m, the open size was 2.6 and 6.2 m.

The piles were modeled as embedded beam elements, the concrete pile caps, abutments, wingwalls and the superstructure were solid elements. The length of the model was 96 m, while the breadth was 60 meters. The total depth of the box model was 20 m. Standard fixities and absorbent boundaries were applied in order to model free-field conditions and reduce wave reflections from the boundaries. The 60E1 rails and the standard B70 prestressed RC sleepers were also modeled as beam elements. Input properties for rail and sleepers are shown in Table 1. A total of 121 sleepers were placed in the model in a 60 cm grid. Figure 3 shows the PLAXIS 3D model.

Table 1. Input properties for rail and sleeper

Parameter	Sleeper B70	Rail 60E1
A (m^2)	0.0513	0.0077
γ (kN/m^3)	25	78
E (MPa)	36000	200000
I_3 (m^4)	0.0253	0.00003
I_2 (m^4)	0.00024	0.00000513

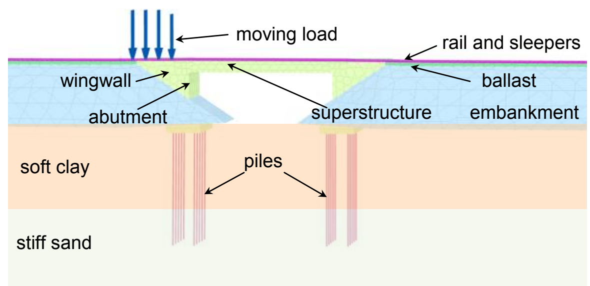


Figure 3. PLAXIS 3D model.

The moving train was modelled with the LM71 Eurocode load model consisting of eight dynamic point loads of 125 kN vertical force. PLAXIS 3D defines dynamic loads using a time-force signal. For the simulation of dynamic loads, point loads were predefined in a set of points. In the model, every single dynamic point load has its own multiplier. Group of point loads were simultaneously activated and deactivated by changing their load multipliers to simulate the rolling vehicle. For the simulation of different travel velocities (120 km/h and 250 km/h), the dynamic time step was varied, while the distance between dynamic point loads were held constant. For example, a train with 120 km/h speed passes 1.6 m in 0.048 sec; hence, the time interval must be set for 0.048 s for the fixed dynamic point loads. The total elapsed time between the first load and the last load was 2.88 sec. An additional time of 0.62 sec was considered to allow complete dissipation of the waves induced by the passing train (Shahraki et al, 2014).

Table 2. Material properties of soil layers

parameter		subsoil soft clay	subsoil stiff sand	embankment medium sand	backfill dense sand	ballast gravel	bridge concrete
model		HS-small	MC	HS-small	HS-small	MC	LE
E	(kPa)		90 000			100 000	$3\ 10^7$
E_{50}^{ref}	(kPa)	10 000		36 000	48 000		
E_{oed}^{ref}	(kPa)	10 000		36 000	48 000		
E_{ur}^{ref}	(kPa)	25 000		108 000	144 000		
G_0^{ref}	kPa)	55700		100 800	114 400		
m	(-)	0.80		0.51	0.45		
$\gamma_{0.7}$	(-)	0.000274		0.00014	0.00012		
c'_{ref}	(kPa)	6.0	1.0	1.0	1.0	1.0	
ϕ'_{ref}	(deg)	26.0	40.0	35.5	38.0	45.0	
ψ	(deg)	0.0	10.0	5.5	8.0	15.0	

2.2 Construction phases and material properties

The following construction phases were defined: 1–initial phase, 2–excavation, 3–piling, 4–construction of the abutment (pile cap+ abutment+wingwalls), 5–placing the backfill and embankment material at both sides of the abutments up to 1.8 m, 30 days construction time, 6–consolidation with u_{res}=10 kPa residual excess pore pressure as consolidation criterion, 7–placing the backfill and embankment material at both sides of the abutments up to 3.6 m, 30 days construction time, 8–consolidation u_{res}=10 kPa, 9–placing the backfill and embankment material at both sides of the abutments up to 5.4 m, 30 days construction time (in case of 7.2 m high embankment), 10–consolidation, (u_{res}=10 kPa), 11–placing the backfill and embankment material at both sides of the abutments up to final height, 30 days construction time (in case of 7.2 m high embankment), 12–consolidation, (u_{res}=5 kPa), 13–construction of the super-structure, 14–placing the ballast, 15–placing the sleepers, 16–placing the rails, 17– train passing (120 km/h and 250 km/h).

Plastic drained calculation method was chosen in phases 1-4 and phases 13-16, consolidation calculation was carried out in phases 5-12. In phase 17, the dynamic calculation option was selected to model the stress and strain waves and vibrations induced in the soil. In this phase, all dynamic point loads on the rails were active, and turned on and off by their time dependent multipliers.

To model the soil behavior, the HS-small constitutive model was applied. The ballast layer was modelled by MC, for the concrete bridge Linear Elastic model was applied. Material properties of soil layers are listed in Table 2. The Poisson's ratio was ν_{ur}=0.2 for all layers as recommended by PLAXIS.

3 RESULTS

3.1 Global behavior of the bridge model

The aim of this study was to determine the settlement in the bridge transition zone while changing the embankment height (as well as the abutment height) and the train speed. Altogether four models were built. In order to evaluate the settlement caused by a moving train, several cross-sections were selected (3 on open track, 4 on backfill and 2 on the bridge) and the total displacements were determined on top of the ballast when the moving train was exactly above the cross-section.

Figure 4 shows the deformed mesh of a model (7.2 m high embankment, 120 km/h speed). One can see the effect of the moving train as it pushes the embankment into the soft subsoil, differencial settlement occurs at the boundaries between regions of different stiffness and the piled bridge is almost unmoved.

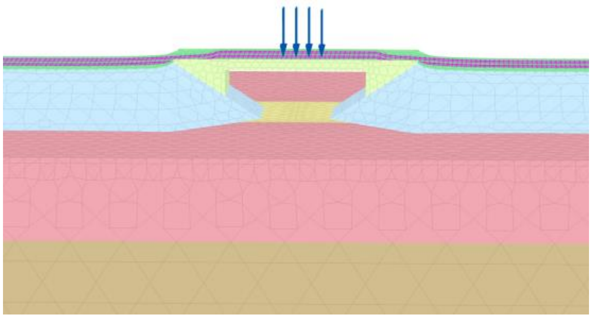


Figure 4. Deformed mesh of a model (7.2 m high embankment and 120 km/h train speed).

Figure 5 shows the total displacement for the same case as it is shown in Figure 4 when the train is on the bridge (construction phase 17). The following can be established:

- the highest settlement occurs on open track, on top of the ballast,
- the settlement decreases as approaching to the bridge abutment; behind the abutment it is one third of the maximum displacement on open track,
- vertical displacement of the superstructure is less than 20 mm,
- upward movement around the embankment toe refers to the possible rotational sliding.

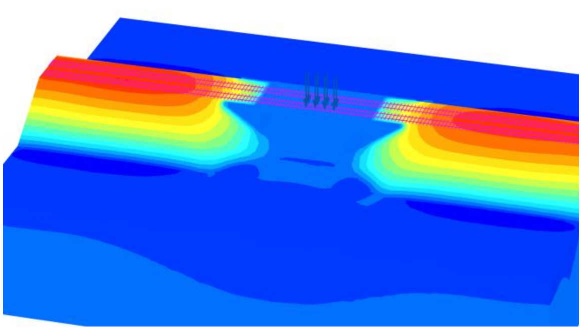


Figure 5. Vertical displacement (7.2 m high embankment and 120 km/h train speed).

Figure 6 shows the total displacement in a longitudinal section of the bridge and its transition zone when the train approaches to the bridge. The vertical displacement decreases with the depth, the depth of influence is about on top of the stiff sand.

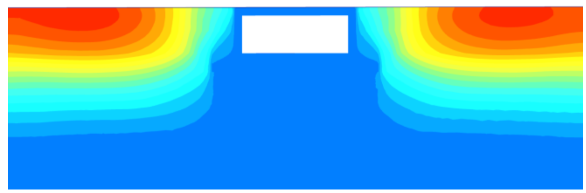
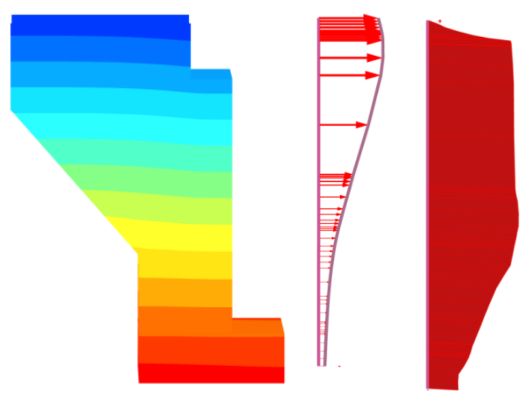


Figure 6. Longitudinal section of the bridge and its surroundings (7.2 m high embankment and 120 km/h train speed).

Figure 7a shows the horizontal movement of the left side abutment before the placement of the superstructure. The horizontal movement of the abutment is small, less than 15 mm. The greatest horizontal movement appears on the bottom part of the abutment and it decreases towards the top of the abutment. Figure 7b shows the horizontal movement of a middle pile. The highest movement occurs below the concrete pile cap and decreases with depth.



a) b) c)

Figure 7. Horizontal displacement of the abutment (a) and one of the piles (b) and the normal force (c) prior to the construction of the superstructure.

Figure 7c represents the normal forces along the pile. It is almost constant on the upper 8 m afterwards in the bottom part of the soft clay layer the resistance and the development of negative skin friction are in balance. In the stiff sand the resistance is dominant.

3.1 Results related to the transition zone

Figure 8 shows the vertical displacements caused by the moving train for six checkpoints (open track: A-top of the ballast +7.55 m, B- top of the subsoil, C-in the subsoil -3,0 m; backfill zone: D-top of the ballast +7,55 m, E-top of the subsoil; F-top of the ballast on the bridge +7.55 m). The following observations can be stated on the basis of Figure 8:

- the greatest displacement on open track occurs on top of the ballast (A),
- in the backfill zone, the displacement reduces significantly (D),
- the lowest displacement occurs on the bridge on top of the ballast (F),
- reduction of the settlement with the depth is obvious (A-B-C and D-E),
- at -3.0 m in the subsoil, the effect of the passing train is definitely decreased (C).

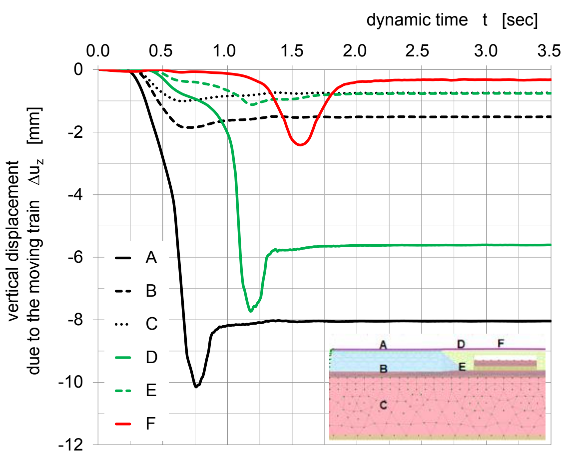


Figure 8. Vertical displacement due to the moving train in different checkpoints (7.2 m high embankment and 120 km/h train speed.

Figure 9 shows the total vertical displacements from beginning of the construction to the moment of the passing train along the longitudinal profile for four different cases, while modifying the embankment heights and the train speeds. The following could be stated regarding the total settlement:

- on open track the higher the embankment, the higher is the settlement,
- much lower on the backfill zone than on the open track,
- it starts to decrease approximately 20 m from the bridge abutment nearly independently of the embankment high and train speed,
- higher speed causes less settlement but the difference is not much,
- it is low on the bridge for each case.

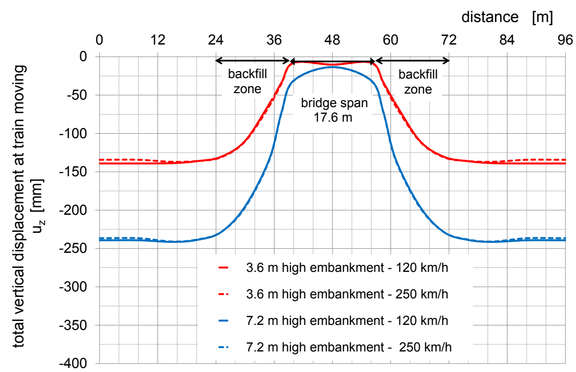


Figure 9. Vertical displacement at train moving of different models.

Figure 10 shows the peak vertical displacements due to the moving train along the longitudinal profile varying the embankment height and train speed. The following could be stated:

- on open track the settlement is higher at lower train speed,
- shapes of settlement curves change in the backfill zone due to the stiffest backfill material,
- on the backfill zone higher velocity causes higher settlement,
- on the bridge vertical movement of the rails is less than 3 mm.

Considering the passengers' comfort and the damage of the rail geometry, investigation of the vertical velocity could be essential.

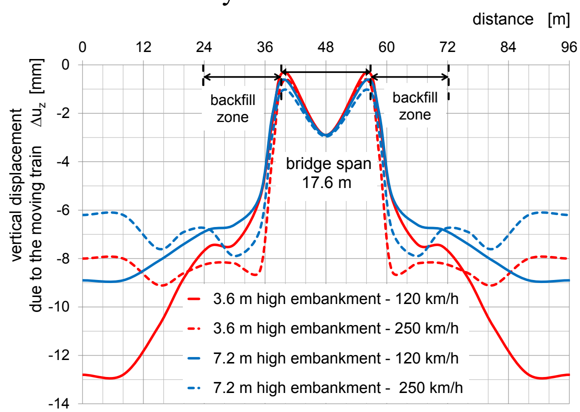


Figure 10. Vertical displacement of different models due to the moving train.

Figure 11 shows the vertical velocity for five checkpoints in different layers on open track at 20 m. Results relate to the case of 7.2 m high embankment and 120 km/h train speed. Velocity amplitudes decrease with depth as expected. The highest velocity belongs to checkpoint A, which is located on top of the ballast. Checkpoints B, C and D show smaller velocities as the wave goes deeper in Z-direction.

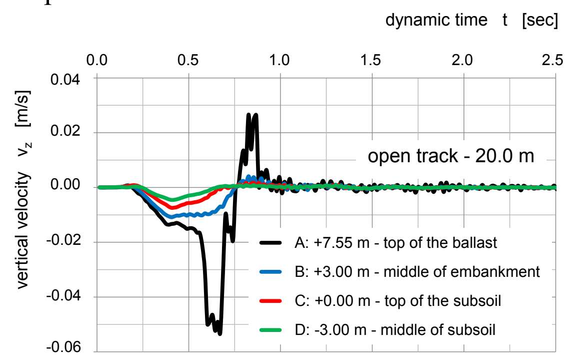


Figure 11 Vertical velocity vs dynamic time at 120 km/h, 7.2 m high embankment.

4 CONCLUSIONS

The presented paper investigates the modelling of a typical railway bridge transition zone by means of the PLAXIS 3D dynamic modul. The results presented here are only a subset of a larger parametric study focusing only on the effect of embankment height and train speed. Based on the results some useful conclusions could be stated.

The presented results show that the real life problem of a train travelling in a transition zone can be modelled by Plaxis 3D using the HSsmall constitutive model and dynamic package.

It is clearly illustrated that the influences of the different parameters (such as the speed of the vehicle and the embankment height) can be modelled efficiently.

The effect of the speed is less than expected. Magnitude of the settlements related to the abutment and its surroundings are more favorable than the results obtained from conventional calculations.

Displacement of the abutment and the piles such as the bottom part of the abutment has higher horizontal movement towards the span corresponds to real-life measurement results.

With this type of modeling it is possible to optimize the construction processes, to determine the allowable negative skin friction, to establish the construction time of the rail superstructure in order to predict the long term behaviour of the track.

The presented experience in dynamic 3D modelling of the problem allowed us to make design recommendations for the required length of the transition zone in different soil conditions and different types of structures.

5 ACKNOWLEDGEMENTS

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