3-Dimensional dynamic hydro-mechanical coupled analysis of the structure – Soil interaction for the TRANSRAPID system

Analyse dynamique 3D avec couplage hydro-mechanique de l'interaction sols-structures, du TRANSRAPID

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ABSTRACT: A 292 km long, very high speed magnetic levitation transportation system (MAGLEV) was planned between Hamburg and Berlin, as part of the German TRANSRAPID transportation system. The MAGLEV system will reach a maximum speed of 250 km/h in conurbation's and up to 450 km/h elsewhere.

The foundation construction is one of the most important parts of the MAGLEV system, since only very small settlements and deflections can be tolerated. This paper presents a design and optimisation workflow using high level numerical design tools. Two different numerical codes (FLAC^{3D} and ANSYS) were used independently for quality assurance, to verify the results and to increase their acceptability to the design and construction engineers. Aim of the study is to set up a parameterised, straight forward numerical design process including the potential of an optimisation due to safety, liability, serviceability and costs. The simulations consider complex hydro-mechanical coupling, soil-structure interaction and modelling of basic material models (Mohr-Coulomb) and sophisticated material models (Cap-Model). Dynamic loading on the structure were verified by modelling the complete passage of a TRANSRAPID. The design loads imposed static and dynamic loading on the structure including centrifugal forces due to curvature of the track and acceleration forces due to braking and accelerating.

The calculations reveal that significant amounts of plastic deformation should be anticipated under these extreme loading conditions. Also the pore water pressures under the foundation slab may change dramatically. In parts of the foundation, the transient pore water pressure may decrease almost to zero, whereas in other parts of the foundation it may increase to approximately 250% of the steady state value.

RESUME: Le réseau ferroviaire allemand TRANSRAPID a un projet de train à très grande vitesse à lévitation magnétique (MAGLEV), reliant Hambourg et Berlin (292 km). Le système MAGLEV autorisera une vitesse de 250 km/h dans les zones urbaines, et 450 km/h ailleurs. La construction de la fondation est une des parties les plus importantes du système MAGLEV puisque seuls de très petits tassements et déflections sont tolérés. Cet article présente un dimensionnement et une optimisation du phasage des travaux réalisés avec des outils numériques performants. Deux logiciels différents (FLAC^{3D} et ANSYS) ont été utilisés indépendamment, pour des raisons d'assurance qualité, afin de vérifier les résultats, valider le dimensionnement et le faire accepter par les ingénieurs. L'objectif de l'étude est de réaliser un modèle numérique entièrement paramétré incluant les étapes de conception et pouvant le cas échéant optimiser le dimensionnement pour des raisons de sécurité, responsabilité, commodité, et de coût. Les simulations tiennent compte d'un couplage hydro-mécanique complexe, d'interactions sol-structures, modélisent des matériaux classiques de type Mohr-Coulomb et des matériaux plus sophistiqués à deux mécanismes de rupture. Les sollicitations dynamiques imposées aux structures sont vérifiées en modélisant le passage complet d'un TRANSRAPID. Le chargement de référence sur les structures inclut des forces statiques et différents chargements dynamiques : de force centrifuges due à la courbure des rails, effet du freinage et de l' accélération.

Les calculs montrent que d'importantes déformations plastiques devraient se produire sous de tels chargement extrêmes. De même, les pressions interstitielles sous la semelle de fondation peuvent changer radicalement. Sous une partie de la fondation, les pressions interstitielles transitoires peuvent se réduire à zéro, alors qu'à d'autres endroits elles peuvent augmenter à approximativement 250% des valeurs de l'état permanent.

1 INTRODUCTION

The 292 km-long Magnetic Levitation Transportation System (TRANSRAPID), planned to link the major German cities of Hamburg (population 2 million) and Berlin (population 5 million) was intended to reduce the travel time between the two cities to one hour. Approximately 131 km of the track will be elevated and 161 km will be at ground-level. The maximum speed of the train will be 250 km/h in conurbation's and up to 450 km/h elsewhere. The technical requirements of the system are extremely high. The maximum diametrical clearance of the driveway that can be balanced is 5 mm, which imposes strict tolerances with respect to settlements and deflections along the system. The ground surface along the planned driveway consists mainly of settlement-sensitive formations, including loose and medium dense sand, clays, peat and weak lignite. The groundwater table is approximately two meters below the surface. Several slab foundation and combined pile-slab-foundation options have been examined for the driveway frame, based on classical engineering design procedures. (Büchel 1988, Schwindt 1994 and Schwindt 1988)

Figure 1 shows the Transrapid on an elevated frame construction.

Given the high technical requirements, difficult ground conditions and high construction costs (approximately 25 million DM/km, ~$13 million/km), the design of a safe, reliable and economic long-term foundation is a top priority for the project. The initial settlement calculations carried out during the pre-planning and authorisation phase, were based on standard, clas-

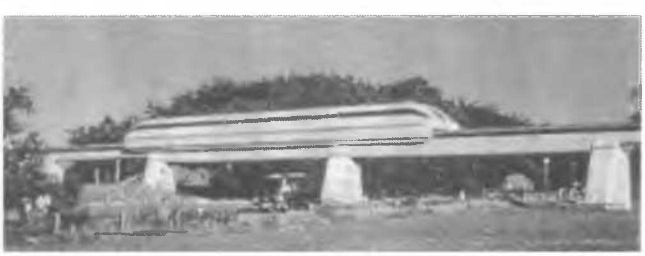

Figure 1. Transrapid on an elevated frame construction.

Table 1. Soil profile.

Depth below surface [m]	soil
0.00 – 8.00	Fine Sand, dense layered
8.00 – 10.50	Fine Sand, dense layered
10.50 – 14.25	Clay, stiff
14.25 – 18.00	Lignite, spongy
18.00 - 25.00	Fine Sand

sical engineering design procedures. While these calculations were sufficient to establish the technical feasibility of this part of the system, important aspects of the soil-structure interaction, such as the complete load-deformation response of the ground, the effect of groundwater (especially the dynamically-induced pore water pressures) and the magnitude and extent of the plastic (irreversible) deformations were not investigated. But one technical requirement was to ensure a negligible amount of plastic deformations in the foundation. The numerical simulations described in this paper were intended to address these question.

2 PARAMETERISED MODEL SET-UP, VERIFICATION OF DISCRETISATION AND MATERIAL MODELLING

The soil profile shown in Table 1, considered to be typical for the Hamburg - Berlin route, was used for the simulations.
Figure 2 shows the grid for the complete model, which includes the soil and the lower part of the frame construction shown in Figure 3.

A parameterised geometric model was build up in ANSYS and the geometry model was prepared for automatic brick meshing. That ensures an automatic remeshing with an optimised number of DOF's due to the optimisation of geometric foundation parameters. Linear analysis was carried out to verify that the density of the discretization (FE-mesh and FD-grid) was acceptable and adequate. This verification was done very easily by using higher and lower order finite elements in ANSYS. This allowed an optimised grid to be designed for the final computations.

The level of discretization has to satisfy the demands of maximum acceptable node/gridpoint spacing to respond accurately to the important frequency content of the dynamic loads imposed on the soil/structure and the stress and strain fields in the soil/structure with sufficient accuracy. Due to the parallel using of finite element and finite difference technology the level of discretization has to be adequate. The optimised grid finally used 88,104 lower order elements and 93,616 nodes.

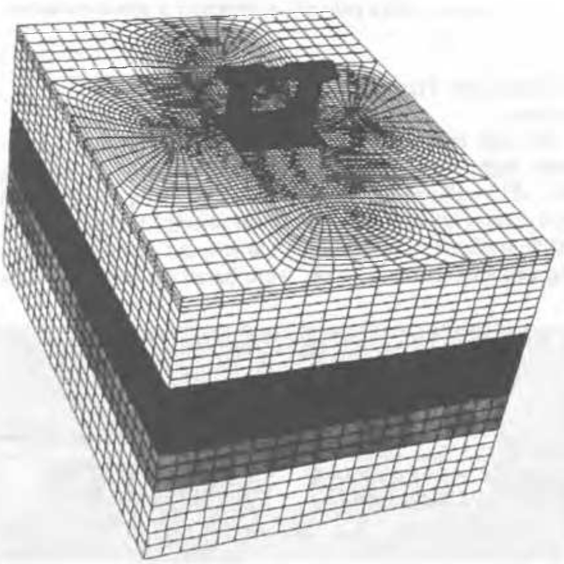

Figure 2. Complete geometric model
(soil + lower concrete part of the frame construction).

The optimised grid from the ANSYS pre-processor was then transferred to FLAC^{3D} using a specially developed converter, called ANSFLAC. Beside the grid, material numbers and node entities can be transferred.

The soil component of the model is 25 m deep and extends 30 m by 40 m laterally. The lower concrete part of the frame construction corresponds to the centre pillar of a 62 m long double girder with a gradient height of 7 m. The zoned lower part of the frame construction is shown in Figure 3. The 1.5 m thick slab plate has a lateral extension of 6 m by 8 m. The lower part of the frame construction was assumed to behave as a purely elastic material, using material data for a B30-type concrete. A primary stress state with an earth pressure coefficient of 0.4 was assumed in all simulations. Standard geomechanical far-field boundary conditions were assumed for the ground (static: fixed normal displacements / dynamic: nearly non-reflecting boundaries). Local damping, set at 3% of critical damping, was used in the dynamic analysis.

To gain a good understanding of the important aspects of the non-linear material behaviour a parametric study was carried out considering several different elasto-plastic material models and coupled hydro-mechanical behaviours with undrained conditions. Starting with basic material models of Mohr-Coulomb type, finally a sophisticated CAP-material model with fully dynamic hydro mechanical coupling was used.

The following simulations were included

- Static:
  - elasto-plastic Mohr-Coulomb behaviour, (ANSYS, FLAC^{3D})
- Dynamic: (FLAC^{3D})
  - elasto-plastic, Mohr-Coulomb, constant pore water pressure (RF-1)
  - elasto-plastic, Mohr-Coulomb, hydro-mechanically (HM) coupled response (RF-2)
  - elasto-plastic, Mohr-Coulomb with additional moving cap (Double-Yield-Model), HM-coupled response (RF-3)

All of the elasto-plastic material models obey a non-associated flow rule, with residual strength values and tension cut-off. Volumetric compaction of the sand and variable stiffness during loading and unloading were taken into account by the Double-Yield-Model. To ensure the accuracy of the Double-Yield-Model Oedometer test results were used for calibration. Figure 4 shows a comparison between measured and numerically calculated values for an Oedometer test.
Extensive elasto-plastic analysis with the code ANSYS (implicit FEM) and FLAC^{3D} (explicit FDM) were carried out to verify the adequacy of the numerical models including boundary conditions, loading and material modelling. One aim of this analysis was to examine the influence of different element integration used in FE and FD on the simulation results. A secondary results, but very important for the quality insurance and reliability of the analysis, is the controlling of boundary conditions, loading and numerical accuracy by using two different codes with different spatial, stress and time integration.

3 STATIC CALCULATIONS

The static calculations were done for the load case where two trains, travelling in opposite directions, passed each other. The estimated added dynamic load advantage factor of approx. 1.3 was included.
The static calculations were intended to compare the numerical results with those obtained earlier using the classical engineering methods.

Considerable plastic deformation, up to 1 cm (1/3 of the total deformation), was observed to develop for the assumed loads and soil parameters. Mainly shear failure with a beginning ground breakage plane can be identified. The total deformations

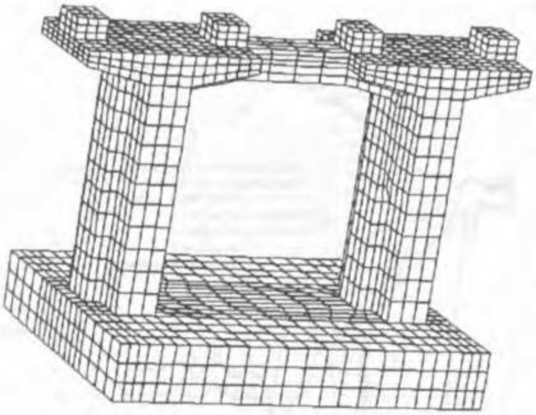

Figure 3. Lower concrete part of the frame construction.

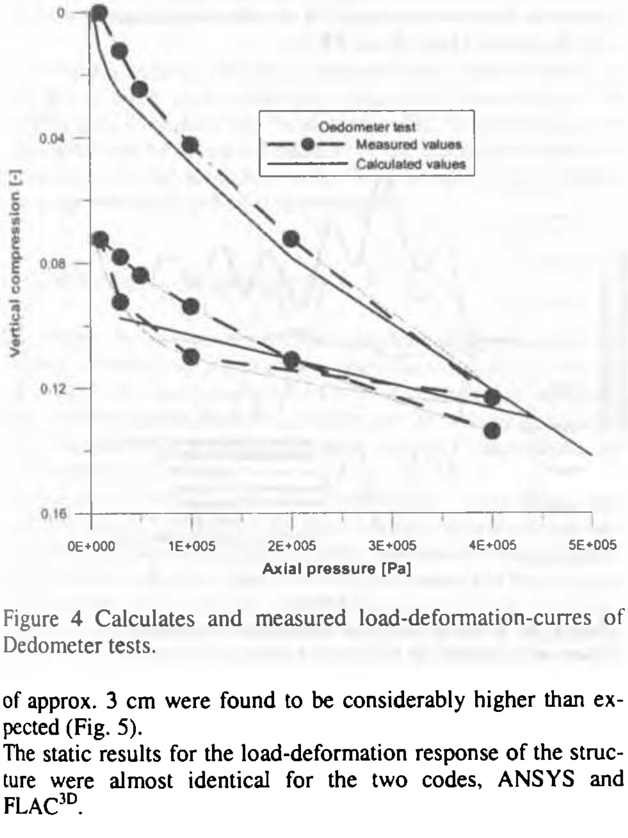

Figure 4 Calculates and measured load-deformation-curres of Dedometer tests.

of approx. 3 cm were found to be considerably higher than expected (Fig. 5).

The static results for the load-deformation response of the structure were almost identical for the two codes, ANSYS and FLAC^{3D}.

4 DETERMINATION OF DYNAMIC LOADS

Transient analysis was used to establish values of dynamic loads which act on the lower part of the concrete frame construction during the passage of a Transrapid train with

2 sections (train length of 45 m). The aim of the calculations was to show the possibility to determine the transient load history and to approximate the frequency content of the loading in the centre pillar. Additionally the results were compared to the amplitudes of the defined static and dynamic load cases.

For simplification the 62 m long TRANSRAPID track was modelled as a quadratic concrete cross section and was assumed to be decoupled from the upper concrete part of the foundation a linear transient analysis of the passage is performed. The loading of the track due to the 15 electromagnets was modelled as a time dependent rolling load sequence. The speed of the train was assumed to be 450 km/h.

Signal analysis of the results reveals two dominate frequencies:

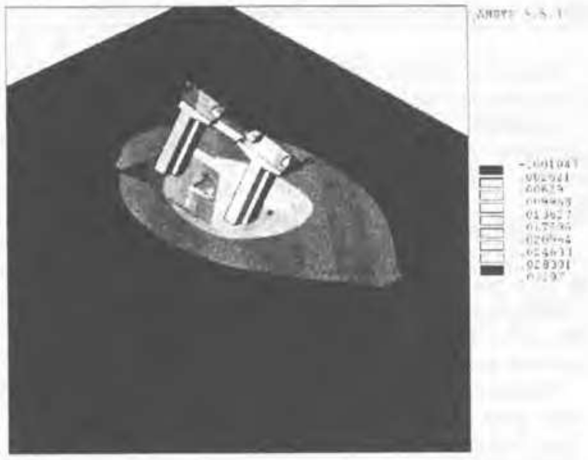

Figure 5. Calculated total vertical ground deformations [m] during passage of two Transrapid-trains (static loading).

- one of approximately 1 Hz, generated by the complete train passing,
- the other of approximately 4 Hz, associated with the frequency of the 15 electromagnets.

Passage of the train takes 0.85 s, so calculations were carried out for 1.5 s. The global damping was varied between 3 and 10 %. Depending on the damping value chosen for global damping, the maximum dynamic loads exceeded the static loads by a factor of 1.2 up to 1.5.

Exemplary, Figure 6 shows the dynamic load for one of the load suspension points.

5 RESULTS OF DYNAMIC HYDRO-MECHANICAL COUPLED CALCULATIONS

The transient non-linear hydro-mechanical coupled analysis was done within FLAC^{3D}. Selected results of the dynamic hydro-mechanical coupled calculations RF-1, RF-2 and RF-3 are shown, in summary form, in Table 2.

Plastic deformations extend locally and temporarily to a depth of up to approximately 15 m below the surface. Shear and volumetric failure are dominant. Nearly all of the dynamic movements in the ground have been attenuated after approximately 1.5 s. In the case RF-3 the ground vibrations show a

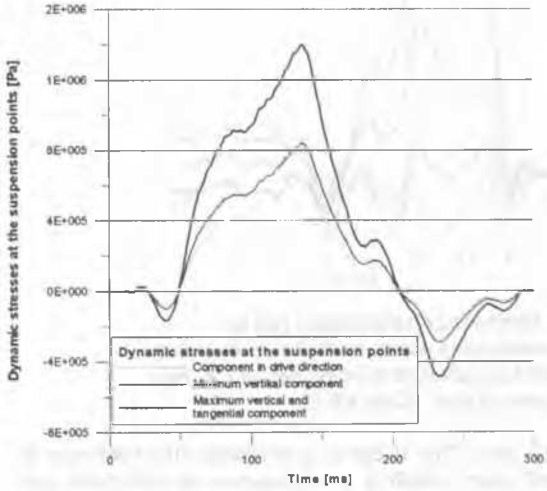

Figure 6. Variation of dynamic excitation stresses (Pa) for one load suspension point.

Table 2. Summary of most important calculation results.

	RF-1	RF-2	RF-3
Maximum displacement at the foundation slab [mm]	4.0	3.5	13.5
Maximum residual displacements at the frame construction after train passage [mm]	2.2	2.0	11.0
Maximum dynamic inclination at the foundation slab during train passage [mm/m]	0.90	0.87	3.5
Maximum residual inclination at the frame construction after train passage [mm/m]	0.33	0.30	1.7
Maximum dynamically generated pore water pressure increase / decrease in the ground during train passage [%]	----	144/ 100	233/ 100

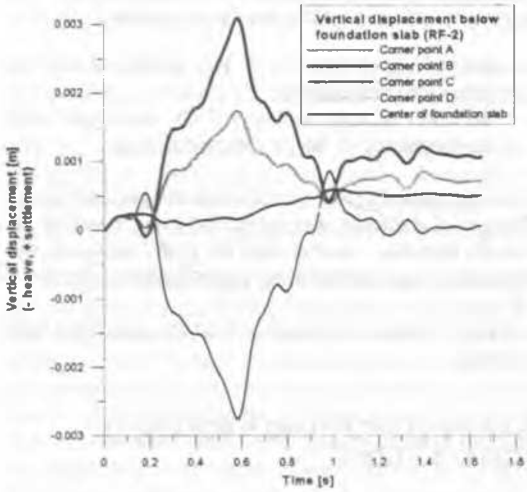

Figure 7. Vertical displacements [m] of selected observation points at the lower boundary of the slab foundation as a function of time (Case RF-2).

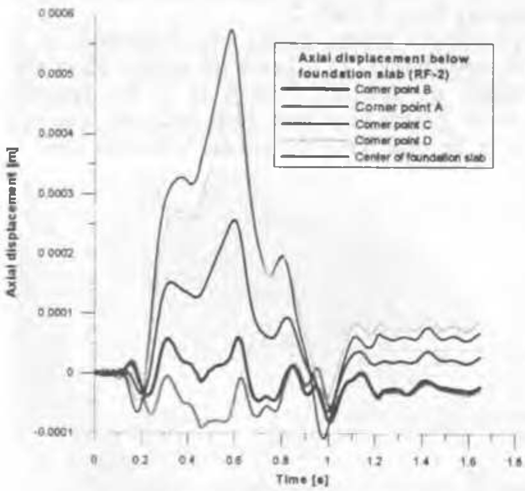

Figure 8. Horizontal displacements [m] at selected observation points on the lower boundary of the slab foundation in the direction of the train as a function of time (Case RF-2).

small decay only. This is due to a resonance-like behaviour in the first soil layer, which is a consequence of reflections produced by the strong stiffness contrast at the first discontinuity layer.

Figures 7 to 12 show examples of time varying displacement

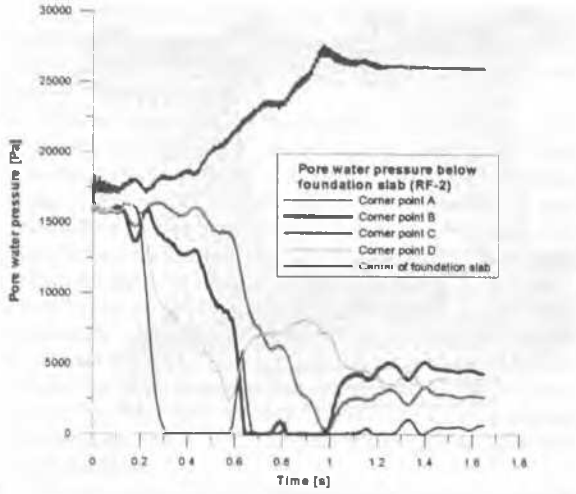

Figure 9. Pore water pressure [Pa] of selected observation points on the lower boundary of the slab foundation as a function of time (Case RF-2).

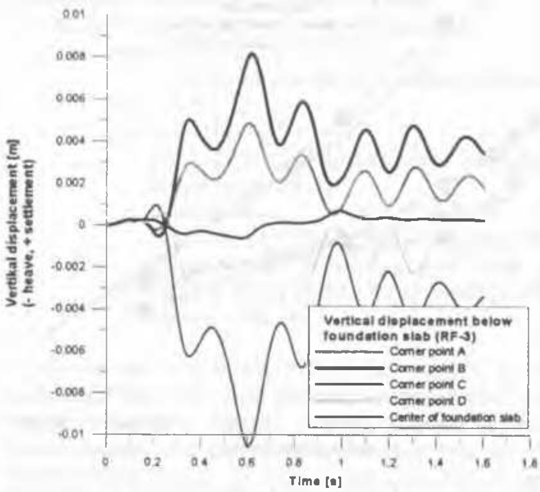

Figure 10. Vertical displacements [m] of selected observation points on the lower boundary of the slab foundation as a function of time (Case RF-3).

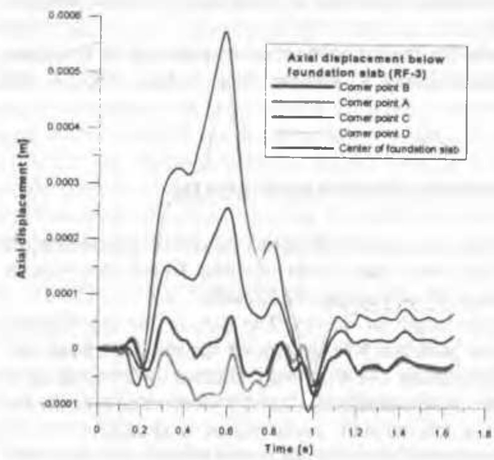

Figure 11. Horizontal displacements of selected observation points on the lower boundary of the s lab foundation [m] in the train direction as a function of time (Case RF-3).

components and pore water pressures at the four corners A, B, C and D of the slab foundation, and below the centre of the slab foundation. In respect to the short loading time undrained condi-

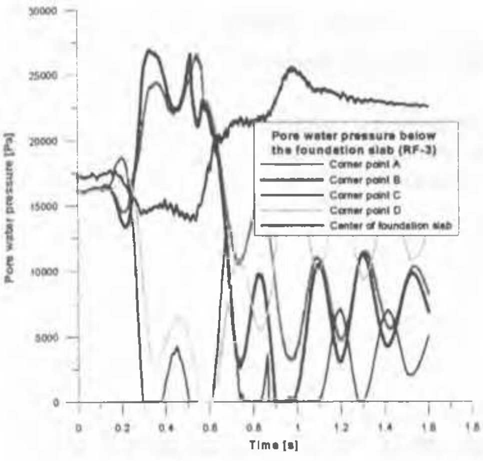

Figure 12. Pore water pressure development [Pa] at selected observation points on the lower boundary of the slab foundation as a function of time (Case RF-3).

tions were assumed. The high amount of the plastic strains in case RF-3 result from extensive compaction behaviour of the CAP-Model. Like the static load case, sufficient stability of the foundation can be found but because of the amount of plastic deformations the serviceability under the used definition of material properties and loading is questionable.

6 DISCUSSION OF RESULTS

The paper discuss on a complex geomechanical problem the process of setting up a design and optimisation workflow by using high level numerical tools. The results show the necessity and demonstrate the feasibility of complex non-linear geotechnical 3-dimensional hydro-mechanical coupled calculations to study practical problems.

In the case discussed, dynamic loading and elasto-plastic soil structure interaction of the foundation for a high-speed train system is the particular problem but other problems of comparable complexity can be analysed in the same manner. The example indicate that the complete three-dimensional model, the non-linear load-deformation behaviour and soil-structure interaction can be investigated.

Additionally we want to point out, that the hardware and manpower requirements for that kind of complex geomechanical studies is not exorbitant high. Only the non-linear transient coupled hydro mechanical analysis (using FLAC^{3D} on PC) were CPU time sensitive.

The extent and magnitudes of dynamically-generated pore pressures, plastic regions and possible areas of failure have been identified and evaluated. A realistic representation of the behaviour of ground was achieved by including the following aspects of the problem:

- 3D simulation,
- full hydro-mechanical coupling,
- dynamic generation of pore pressures,
- non-linear static calculations,
- non-linear transient dynamic simulation (real-time),
- elasto-plastic material behaviour, including a moving cap,
- realistic dynamic excitation in all 3 spatial directions.

Consideration of these factors allow more realistic simulations with respect to the following design issues

- serviceability limits,
- ultimate limit state design,
- evaluation of alternative foundation concepts,
- optimisation of foundations (including cost optimisation),
- reduction of operational risk.

The material parameters used in this study correspond to very poor ground conditions with no ground improvement. Combined with the extreme loading conditions the simulations have resulted in large plastic deformations. These deformations are not expectable for serviceability and should be reduced considerably with application of ground improvements and/or alternative foundation concepts. Further numerical simulations should also include cyclic material loading with a high number of train passages in order to assess the long-term behaviour of the plastic strains.

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