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Numerical modeling of the deformation in railway foundation – A case study

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ABSTRACT: For modeling the development of railway track deformation, a quasi-static model describing the stress - permanent strain - cycle number - relationship of granular soils has been developed in terms of the cyclic triaxial test results from literature. The in-situ measurement carried out in the section Wittenberge-Dergenthin of the extending railway line Hamburg-Berlin is analyzed by using the developed model and the procedure. The research indicates that the essential parameters included may be assessed by using the results of the vibration table tests on soils. The comparison of the predicted settlement with the measured values shows satisfactory agreement.

1 INTRODUCTION

The increasing application of the slab tracks, which are more difficult to adjust geometrically, requires that the total settlement as well as the differential settlement of the railway foundation must be limited within a much smaller value compared to those for ballasted tracks.

In this paper, a case study is presented for the section Wittenberge-Dergenthin of the extending railway line Hamburg-Berlin, where the slab superstructure was applied. For monitoring the stability and deformation of the track, extensive in-situ measurement was carried out during and after the construction. In the analysis, a quasi-static model developed by the authors was used, see (Kempfert & Hu 1999). The essential parameters included were determined from the back-analysis of the vibration table tests.

2 QUASI-STATIC MODEL

The real dynamic loading from railway traffic and the physical mechanism of soil deformation under such condition are very complicated. The consideration of all aspects with one mechanical model seems to be not realistic, at least in the present stage. Simplification must be made for practical purpose.

A simplified procedure called quasi-static model has been proposed and numerically implemented by the authors for describing the permanent deformation of granular soils under cyclic loading, see Fig. 1. The basic assumptions are as follows:

-The maximum of the cyclic dynamic loading is seen as input parameter "quasi-static stress σ".

-In-situ measurement indicates that during train passing the resulted excess pore water pressure in granular soils is usually very low and decreases within very short time down to zero after train passing. Upon this, the factor of pore water pressure may be excluded.

-The cyclic strain part ε^{qe} is generally much smaller than the permanent strain ε^{cp}. The quasi-elastic modul E can be assumed to be independent of the cycle number.

-The viscous dashpot η_1 is introduced to simulate the permanent strain ε^{cp} for the cyclic stable case and is dependent on the cyclic number N. This dependency can be described using the empiric relationships from cyclic triaxial tests, see Table 1.

-For the failure case, the conventional visco-plastic formulation can be used.

3 THE RAILWAY LINE SECTION WITTENBERGE-DERGENTHIN

3.1 *Project*

In 1993/94 the railway line Hamburg-Berlin was extended by using the slab superstructure (type Züblin) on the section Wittenberge-Dergenthin (6 km long). The old track was the ballasted superstructure and showed a bad state. The design train speed is 160 km/h, optionally up to 200 km/h.

As illustrated in Fig. 2, the slab superstructure of the Züblin type consists of an upper bearing concrete plate with a thickness of 28 cm and a 30-cm thick

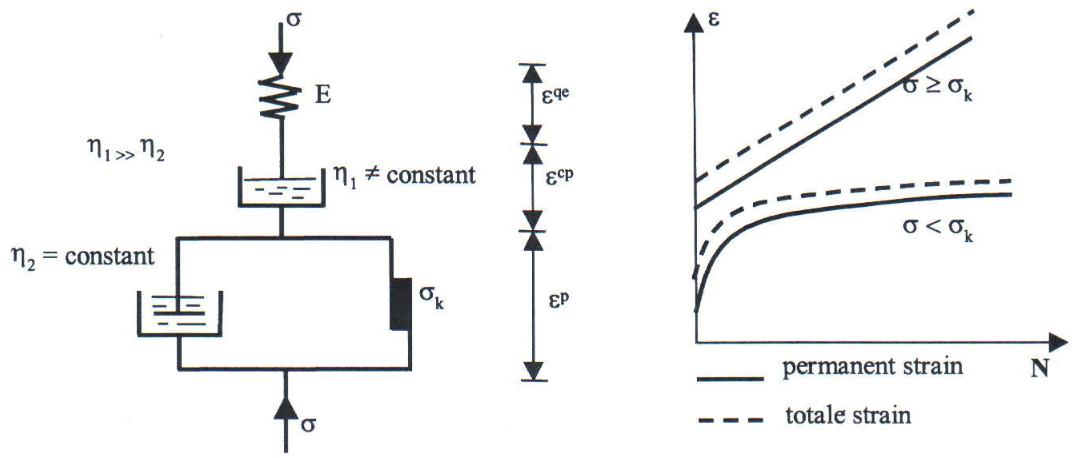


Figure 1. a) one-dimensional quasi-static model; b) schematic strain - cycle number - relationship.

Table 1. Basic relationships in terms of the reanalysis of the results in literature, $(\sigma_1 - \sigma_3)_c / (\sigma_1 - \sigma_3)_{s,f} < K$

double-logarithmic function: $\varepsilon_a^{cp} = a \cdot (1 + \alpha \cdot \log N)$;

α = constant for a given relative density;

$$a = \beta \cdot (\sigma_3 / P_a)^\chi \cdot q^2 .$$

semi-logarithmic function: $\log \varepsilon_a^{cp} = \log a + \alpha \cdot \log N$;

α = constant for a given relative density;

$$a = \beta \cdot (P_a / \sigma_3)^{1/3} \cdot q^\chi \quad (\sigma_3 < 35 \text{ kN/m}^2);$$

$$a = \beta \cdot (\sigma_3 / P_a)^\chi \cdot q^2 \quad (\sigma_3 > 35 \text{ kN/m}^2).$$

$P_a = 100$ kN/m²; $q = (\sigma_1 - \sigma_3)_c / (\sigma_1 - \sigma_3)_{s,f}$; β and χ : two curve fitting coefficients.

down bearing layer of cement-bound mineral material. Under the slab superstructure a frost protection layer was constructed with a minimum thickness of 40 cm and increases to 60 cm depending on the slope ratio 1:20. The dewatering of the track is designed using a drainage pipe embedded in bituminous impervious layer between the rails. The collected water is leaded to the outside through the intake shafts and the cross drainage pipes built every 50 m. In the slab superstructure, the rail location can be adjusted only by using the compensation in the rail fastening system and therefore very limited. Here, the maximum correction limits are 22 mm up, 4 mm down, and 5 mm laterally, respectively.

Under such boundary conditions, a stiffer foundation must be built preventing unallowable settlement of the track. For this purpose, some requirements related to the foundation have been stipulated by the Germany Railway (DB AG). For the investigated project, the essential points are as follows:

frost protection layer: $E_{v2} \geq 100$ MN/m²;
$D_{pr} > 1{,}0$; $k \geq 1 \times 10^{-5}$ m/s
substructure/foundation: $E_{v2} \geq 45$ MN/m²;
$D_{pr} > 0{,}97$ (0-0,5 m);
$D_{pr} > 0{,}95$ (0,5-2,5 m);

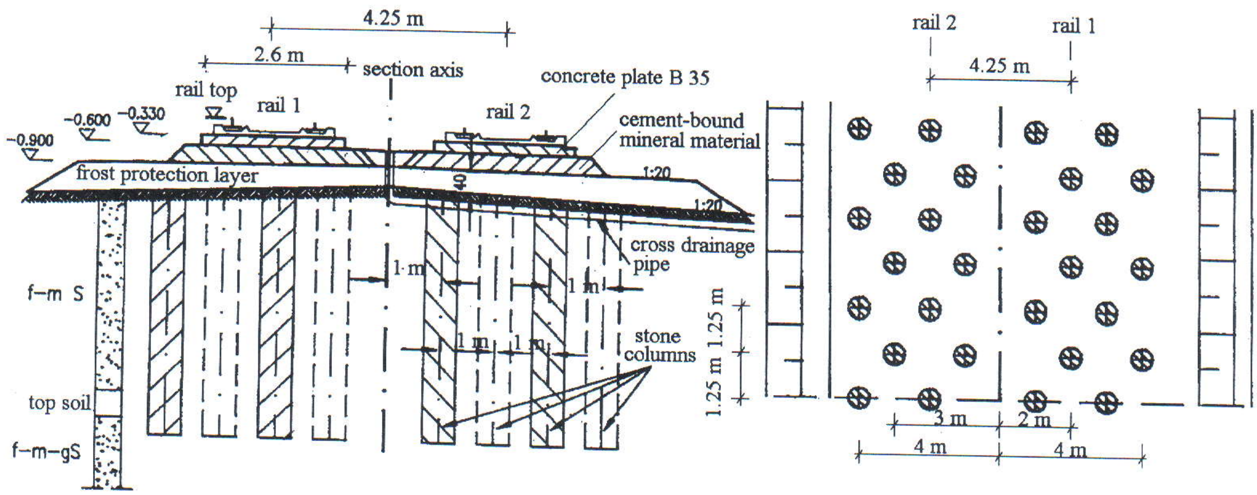
E_{v2}: deformation modul from the second loading cycle of in situ plate compression test.

The existing railway foundation was built on an embankment with changing height (H = 0 to 6 m). It's slope angle is between 45° and 50°. The filing material is mainly composed of the silty fine- and medium-sized sands having loose, locally medium density. Under the embankment, the silty fine- and middle-sized sands having the medium density are encountered. The ground water table is about 3 to 7 m below the rail top.

In consideration of the loose soils encountered and the high requirements in the construction of the slab superstructure, the combined technique of the foundation company Keller "vibro displacement/ flotation compaction" has been used for improving the strength and deformation behavior of the filling materials/soils in the embankment/subsoil. The stone columns were constructed using the old ballast materials. The column raster is illustrated in Fig. 2. The quality and improvement of the embankment/subsoil after the construction were checked by using different monitoring methods, see (Kempfert & Berner 1997).

3.2 *Measurement program*

For monitoring the stability and deformation of the neighboring track in operation during the construction, and also observing and investigating the long-term behavior of the railway track, four measuring stations were installed along the section, see Fig. 3 (left). To measure the absolute settlement of the



a) cross section

b) raster of stone columns

Figure 2. Typical cross section of the slab superstructure (type Züblin) and the foundation construction, project Wittenberge-Dergenthin.

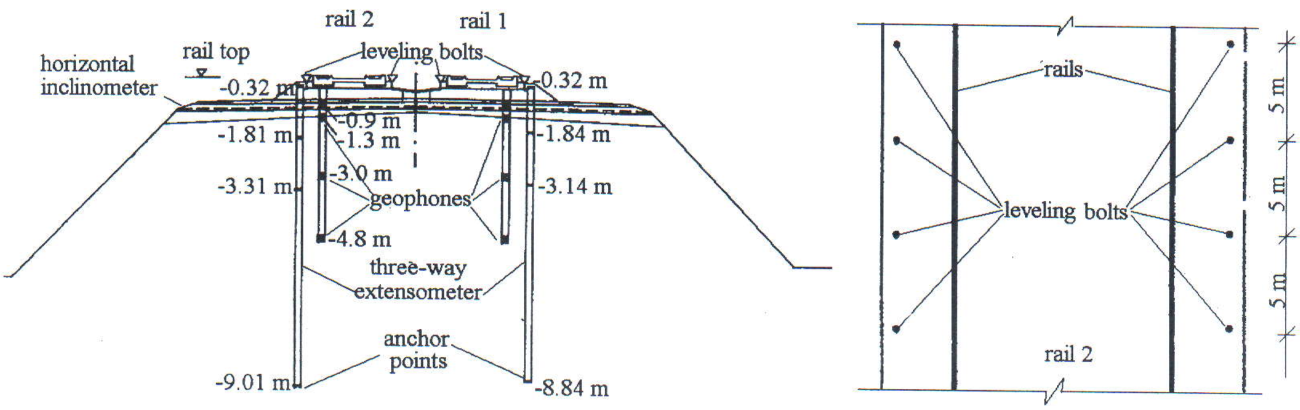


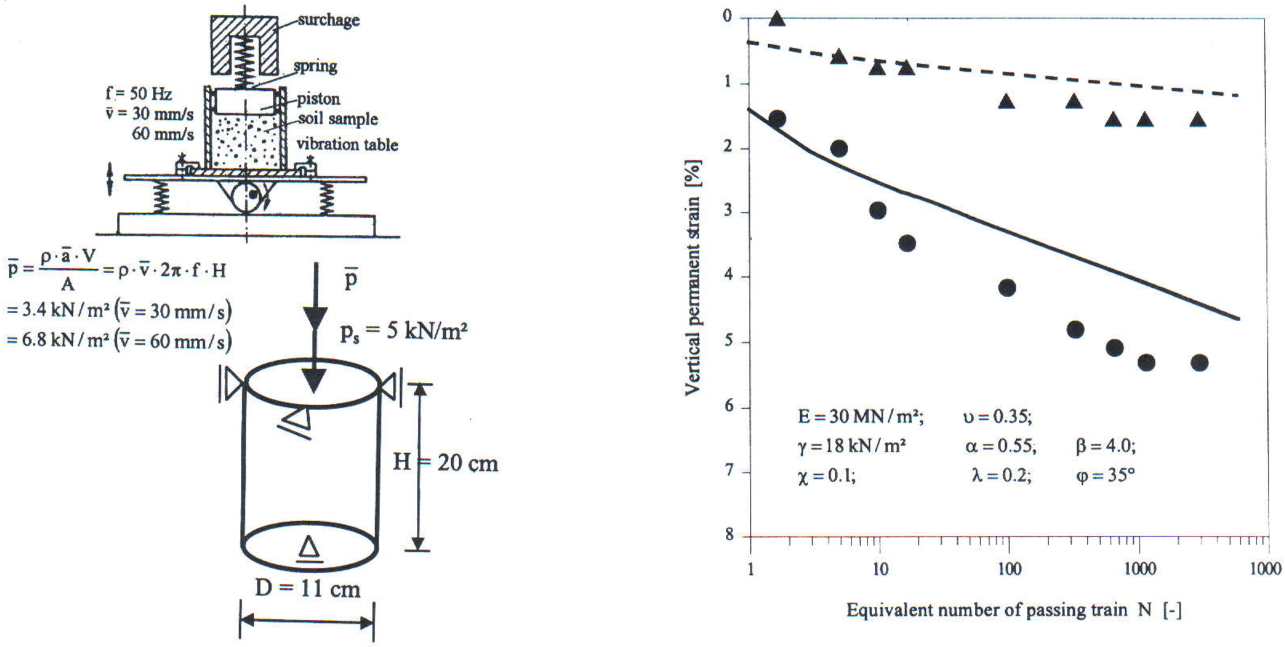
Figure 3. Measurement program.

track, the leveling bolts were firmed up on the surface of the concrete plate (margins) with a distance of 5 m along the section, see Fig. 3 (right). A total measuring length of 50 m was covered round each measuring station. The measurement of the absolute settlement was carried out using fine leveling instrument. The three-way extensometers were installed direct on the concrete plate for measuring the settlement distribution in depth. The anchor points were cemented down to 8,6 m under rail top. In addition, the horizontal inclinometers were installed in the frost protection layer for measuring the horizontal settlement distribution in cross section. The other measurements, such as vibration velocity in foundation were also carried out, see (Kempfert & Berner 1997).

4 NUMERICAL PREDICTION AND COMPARISON WITH MEASUREMENT

4.1 *Calculation parameters*

For the numerical simulation, the empirical parameters included in Table 1 were assessed from the vibration table tests. In Fig. 4 a) the test apparatus and the calculation model are illustrated. The soils were prepared to an initial density index of 0.49. The cylindrical soil samples have a initial height of 20 cm and a diameter of 11 cm. They were first statically under a vertical stress of 5 kN/m², and then dynamically under an harmonic vertical vibration loaded. A frequency of 50 Hz was applied and the effective vertical vibration velocity 30 and 60 mm/s,



a) Test apparatus and model

b) calculated and measured results

Figure 4. Back-calculation of the parameters from the vibration table tests.

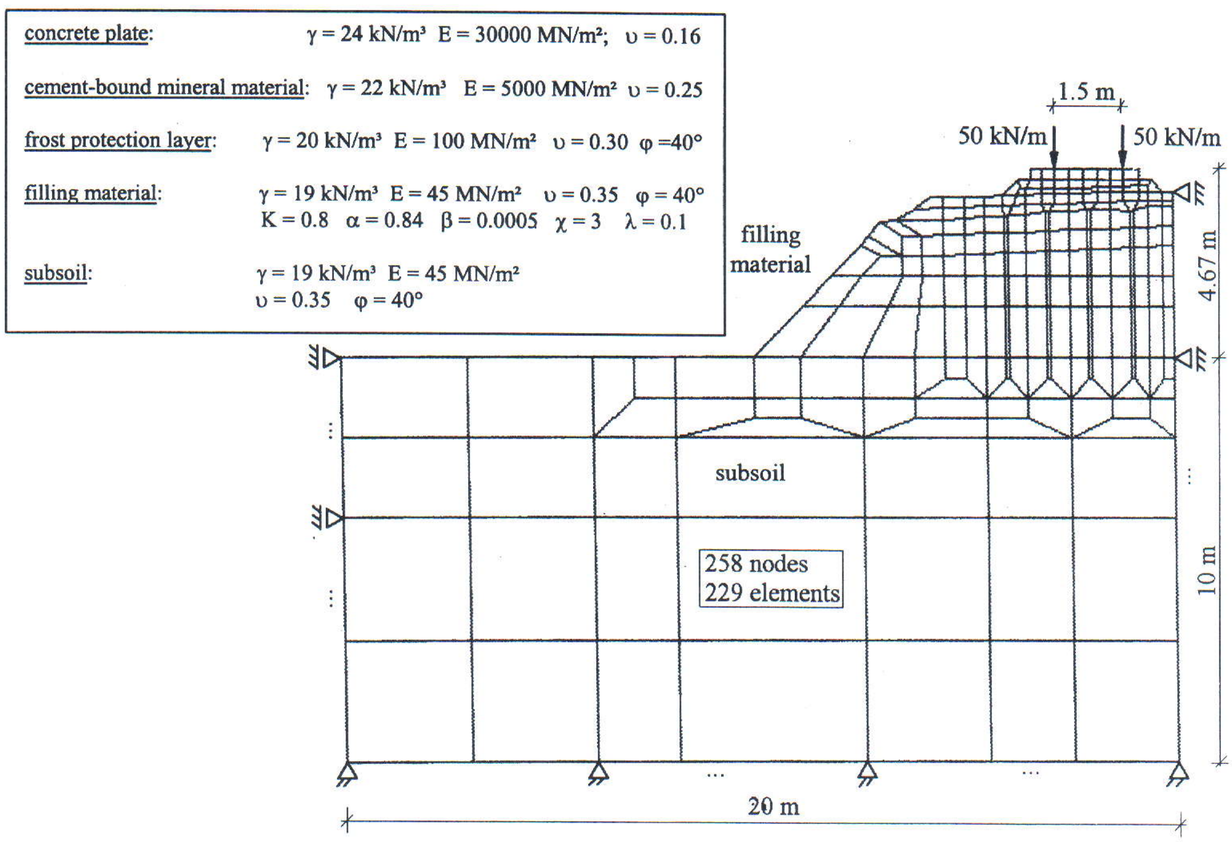


Figure 5. Computation section, FE-mesh and applied parameters.

respectively. The duration of the dynamic loading was 300 minutes. This corresponds to about 3000 train passings, provided that the average train passing duration is 6 seconds. The final density index D came to 0.61 and 0.79, respectively. The dynamic vibration was simulated as a quasi-static vertical stress in terms of the d'Alembert's principle, see Fig. 4. The research showed that the double-logarithmic function in Table 1 is appropriate. The back-calculated parameters and the comparison between the calculated and measured vertical permanent strain are presented in Fig. 4 b). The new parameters for the compacted soils were deduced from the curve after D_{pr} = 97% (density index D = 0.8) in the vibration table tests and are given in Fig. 5. Here, the semi-logarithmic function in Table 1 is appropriate for the description of the stress - strain - cycle number - relationship of the compacted material.

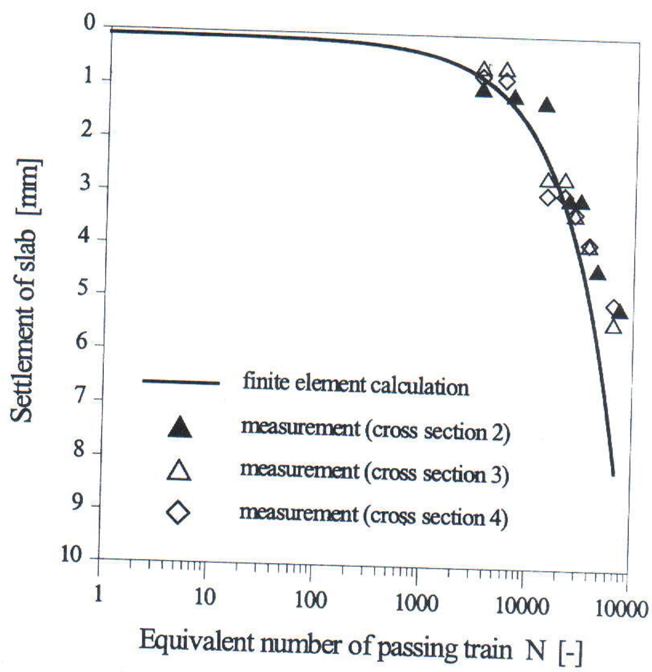
4.2 *Computation cross section, FE-mesh and calculation procedure*

The embankment is assumed to be symmetrical and the loads from the train vehicle appear on the two rails at the same time. The wheel load of a train is idealized as a line load with a value of 50 kN/m. This value has included the dynamic amplification factor suggested by Kempfert & Hu (1999). Because of the lack of the tests on the stone columns, the embankment is simplified as a homogenous layer. The influence of the underlying deposit on the deformation is subordinate and therefore assumed to be conventional elastic-plastic material. The chosen cross section, the FE-mesh as well as the parameters are shown in Fig. 5.

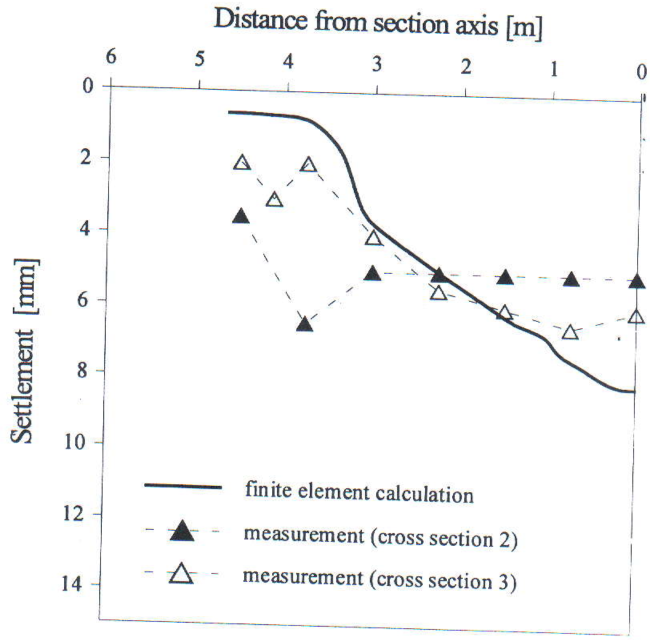
4.3 *Comparison of the calculation results with in-situ measurement*

In Fig. 6 a) the calculated and measured absolute settlement of the concrete plate depending on the number of passing trains is put together. The measured points are illustrated in form of the average values in section 2, 3 and 4, respectively. The absolute settlement until N = 70000 lies in the scale of ca. 5 mm. The calculated results have a similar tendency as the measured. But the calculated settlement seems to be a little larger than the measurement. This may be put down to the simplified assumption of the homogenous material in the embankment, because the stone columns may have a higher stiffness than that of the filling material.

The settlement distribution in the frost protection layer is presented in Fig. 6 b) for N = 60000. In principle, the calculated distribution has a good agreement with the measurement. In the neighboring area of the two parallel rails the predicted settlement is larger than those measured. This difference may be from the simplification of the calculation model, as the assumed parallel train passing of the two rails appears rare in practice. In comparison with this, the measured settlement on the other side, namely near the slope of the embankment, is larger than that predicted.



a) settlement development



b) settlement distributions, N = 60000

Figure 6. Comparison between the calculated and the measured settlement.

5 CONCLUSIONS

In this paper, the case study is presented for the section Wittenberge-Dergenthin of the extending railway line Hamburg-Berlin. The analysis was carried out by using the proposed quasi-static model. The parameters used in the calculation were determined from the vibration table tests instead of the cyclic triaxial tests. The comparison of the calculated settlement with the measured values shows satisfactory agreement. This research indicates that the developed model and procedure are applicable for the prediction of the long-term settlement and deformation of the railway foundation. The vibration table tests may be a practical way in determining the necessary parameters.

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