



Numerical Validation of Differential Settlement in the Transition Zone of Embankment-Slab on Pile Over Soft Soils

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ABSTRACT

Differential settlement in the transition zone between embankment segments and rigid structures such as Slab-on-Pile is one of the most critical issues in toll road construction on soft soil, as it may reduce ride comfort, safety, and service life. In the Serang–Panimbang Toll Road Section III Project, the applied ground treatment method consists of Free Draining Granular Material (FDGM) supporting the approach embankment and bored piles as the abutment foundation for the Slab-on-Pile system. This study aims to evaluate the performance of the existing treatment method by comparing settlement predictions derived from one-dimensional analytical methods (Terzaghi's theory for embankments and the Boussinesq and Poulos–Davis methods for abutments) with two-dimensional numerical modeling using PLAXIS. The resulting settlement profile in the transition zone is then analyzed to assess the potential for differential settlement with respect to the applicable serviceability limits. The validation results show a strong agreement between analytical and numerical methods, with settlement deviations of 6.09% for the embankment and 4.93% for the abutment. Nevertheless, the embankment over FDGM experiences a total settlement of 51.90 cm, while the Slab-on-Pile abutment settles only 6.10 cm, producing a differential settlement of 45.80 cm, far exceeding the allowable limit. These findings indicate that the use of FDGM alone is insufficient to accommodate the high compressibility of the thick soft soil deposit at the study location.

Keywords:

*Differential Settlement;
Soft Soil; FDGM; Slab
on Pile; PLAXIS 2D;
Numerical Validation.*

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INTRODUCTION

The development of connectivity infrastructure is one of the Indonesian government's top priorities in supporting economic growth and promoting equitable development. Road infrastructure plays a crucial role in sustaining economic activity due to its function in facilitating the movement of people and logistics. Therefore, road development and management must be grounded in infrastructure asset management principles to ensure optimal and sustainable performance (Sormin et al., 2024; Yuanita et al., 2022). One of the national strategic infrastructure projects (Proyek Strategis Nasional/PSN) is the Serang–Panimbang Toll Road, which extends 83.677 km to connect Serang City with the Tanjung Lesung Tourism Special Economic Zone (KEK) in Banten Province. This project is expected to improve logistics efficiency and open new access to industrial and tourism areas in South Banten. The project site is generally located within agricultural land underlain by deep soft soil deposits. Soft soil is defined as cohesive soil with an NSPT value ≤ 4 , characterized by low bearing capacity and high compressibility, which can lead to large and long-term settlement (Hardiyatmo, 2002).



Figure 1. Location of the Serang-Panimbang Toll Road Construction Project Section III (a) Banten Province (b) Detailed Location

The design and construction of embankments on soft soil present critical geotechnical challenges, particularly regarding stability and settlement issues. Excessive settlement beneath embankment structures can lead to cracking, differential deformation, and even functional failure of the pavement system and associated infrastructure (Wahyudi et al., 2021). The primary factors contributing to such settlement include the application of structural loads on soil with low bearing capacity and the long-term consolidation process required for soft soil to reach stabilization (Aziz et al., 2025). Without prior ground improvement measures, structures built on soft soil are highly susceptible to damage before reaching their intended service life (Sinulingga & Suhendra, 2021).

In the Serang–Panimbang Toll Road Section III Project, two primary soft-soil treatment methods are implemented: soil replacement using Free Draining Granular Material (FDGM) to support the approach embankment and the use of a Slab on Pile structural system. The interface between these two construction types forms a critical transition zone where the flexible embankment structure directly meets the rigid abutment supporting the Slab-on-Pile system. This transition area is highly prone to extreme differential settlement due to the contrasting stiffness of the foundation systems (Peduto et al., 2020). Bridge abutments supported by bored piles generally behave rigidly and remain stable, while the embankment behind them undergoes consolidation settlement in accordance with the characteristics of the underlying soft soil. This differential movement leads to elevation discontinuities or “bumps,” which not only reduce riding comfort but also pose safety hazards due to the potential loss of tire–pavement contact (Zhang, 2010).

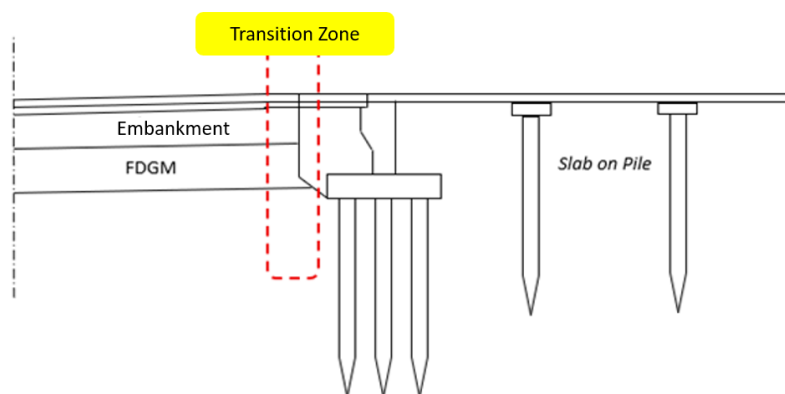


Figure 2. Illustration of the Transition Zone

To ensure serviceability and the safety of toll road users, the magnitude of differential settlement occurring in the transition zone must be controlled so that it does not exceed allowable tolerances. Various national design standards and previous studies have established differential settlement criteria based on multiple considerations, including vehicle dynamic safety, riding comfort and structural serviceability limits to prevent pavement damage. The determination of these allowable limits serves as a critical parameter for evaluating the effectiveness of the ground improvement methods applied in this study. A summary of differential settlement requirements and allowable thresholds from relevant technical standards and recent literature is presented in Table 1.

Table 1. Summary of Differential Settlement Limit Criteria

Reference	Parameter	Permission Limit Value
Zhang (2010)	Absolute Differential Settlement (<i>Bump Height</i>)	≤ 20 mm (2.0 cm)
Pavement Design Manual (MDP) 2024	<i>Change of Grade</i>	$\leq 0.3\%$ (0.003:1)
Abdullah, et al. (2024)	Angular Distortion (δ/L)	1/200 to 1/500
SNI 8460:2017	Angular Distortion (δ/L)	1/300
Wahl (1990)	Angular Distortion (δ/L)	1/200
Geotechnical Guide 4	Rate of Settlement	2.0 cm/year 10.0 cm/10 years

In geotechnical engineering practice, embankment settlement is commonly predicted using one-dimensional analytical methods such as Terzaghi's theory. However, these methods have limitations in capturing complex soil structure interactions and two-dimensional stress distributions (Wehnert & Vermeer, 2004). Meanwhile, for settlement analysis of pile or bored-pile foundations, prediction accuracy is highly dependent on selecting methods that correspond to the dominant load transfer mechanism, whether primarily end bearing or shaft friction (Das, 2012). On the other hand, numerical methods based on the Finite Element Method (FEM), such as PLAXIS 2D, provide more comprehensive analyses, although their accuracy is strongly influenced by the proper calibration of soil input parameters (Abdullah et al., 2024). Calibration of input parameters using field data and validation against analytical calculations or actual observations is essential to ensure reliable modeling results (Saputro et al., 2018).

This study aims to analyze the potential for differential settlement in the transition zone between the embankment and the bridge abutment structure, as well as to validate the reliability of the numerical model through a comparative study. The validation is carried out by comparing two-dimensional finite element simulation results with comprehensive analytical calculations, including Terzaghi's 1D method for embankment settlement prediction and the Poulos&Davis method combined with Boussinesq stress distribution analysis for evaluating abutment pile group settlement. Through this approach, the study is expected to provide an accurate technical assessment of the effectiveness of the existing ground improvement method (FDGM replacement) in meeting serviceability criteria and ensuring toll road user safety.

The abutment structure is supported by a group of bored piles with a diameter of 80 cm and a depth of 40 meters. Given that the hard soil layer is located at a considerable depth, the piles are designed to function as friction piles or floating piles. Prior to conducting the settlement analysis, the axial bearing capacity of the piles was verified using the Meyerhof and Luciano Decourt methods to ensure an adequate factor of safety against the design load. The predicted settlement of the pile group was evaluated using the Boussinesq stress-distribution method and the Poulos & Davis approach. The vertical stress increment ($\Delta\sigma$) in the soil layers beneath the fictitious raft was calculated based on the 2V:1H load spreading assumption using the following equation:

$$\Delta\sigma'_i = \frac{Q_g}{(B_g + z_i)(L_g + z_i)} \quad \dots (3)$$

Where Q_g is the total load of the pile group, B_g and L_g are the width and length of the pile group area and z_i is the depth of the point of interest measured from the base. The value of $\Delta\sigma'_i$ is then used to calculate the consolidation settlement of the soil layers beneath the pile tips.

The Poulos and Davis (1980) method evaluates pile–soil–pile interaction effects that cannot be captured by single pile analysis. This method is based on elasticity theory, in which the deformation of one pile influences the surrounding piles through soil deformation. The first step is to calculate the settlement of a single pile (S_1) under the working load (Q) using the following equation:

$$S_1 = \frac{Q I}{E_s d} \quad \dots (4)$$

Where E_s is the soil elastic modulus, d is the pile diameter, and I is the influence factor for total displacement.

Next, to obtain the pile group settlement (S_g), the superposition principle of interaction factors (α) is applied. The settlement of any pile i within the group due to the load applied on pile j is defined as follows:

$$S_1 = S_1 \times \sum_{j=1}^n (P_j \alpha_{ij}) \quad \dots (5)$$

Where S_1 is the single pile settlement under a unit load, n is the number of piles in the group, and α_{ij} is the interaction factor between pile i and pile j , the value of which depends on the pile spacing to diameter ratio (s/d), the pile length to diameter ratio (L/d), and the relative pile soil stiffness (K).

Numerical Modeling

The numerical simulation was performed using the finite element software PLAXIS 2D with a plane strain model. In the global stability analysis, the factor of safety was determined using the shear strength reduction (Phi–c reduction) procedure. This method works by progressively reducing the soil cohesion (c) and friction angle (ϕ) until failure occurs, providing a more comprehensive approach compared to conventional limit equilibrium methods (Manurung et al., 2021). For the consolidation settlement analysis, PLAXIS 2D implements constitutive models such as Soft Soil and Modified Cam Clay to predict the magnitude and rate of consolidation settlement, which are essential for evaluating the long-term behavior of embankments on soft soil (Avhycanti & Setiawati, 2021).

The model geometry includes the embankment structure, FDGM layer, abutment, foundation piles, and slab on pile system, following the actual field dimensions. Constitutive model selection was adjusted to match the mechanical behavior of each material. The embankment fill and concrete structural elements were modeled using the Mohr–Coulomb (MC) and Linear Elastic models, while the clay layers were modeled using the Soft Soil (SS) model. The stiffness parameters λ^* and κ^* were derived from laboratory test results by converting the compression index (C_c) and swelling index (C_s).

The model is considered valid when the deviation between the numerical settlement results and the analytical predictions falls within an acceptable tolerance ($< 10\%$). The calibrated model is then used to map the differential settlement profile along the transition zone (Abdullah et al., 2024).

Table 2. Summary of Numerical Model Material Input

Material Type	Set Type	Model	Drainage Type
Very Soft Clay	Soil & Interfaces	Soft Soil	Undrained A
Soft Clay	Soil & Interfaces	Soft Soil	Undrained A
Stiff Clay	Soil & Interfaces	Soft Soil	Undrained A
Stiff Silt	Soil & Interfaces	Mohr-Coulumb	Undrained A
Very Stiff Clay	Soil & Interfaces	Mohr-Coulumb	Undrained A
Embankment	Soil & Interfaces	Mohr-Coulumb	Drained
FDGM	Soil & Interfaces	Mohr-Coulumb	Drained
Abutment	Soil & Interfaces	Linear Elastic	Non-Porous
Slab	Plate	Plastic	-
Rigid Pavement	Plate	Plastic	-
Concrete Spun Pile	Embedded Beams	Elastic	-
Bored Pile	Embedded Beams	Elastic	-

The selection of observation points (nodes) for the settlement curve was carried out strategically at locations critical to road functionality, as illustrated in Figure 5. The *Abutment Point* was placed at the top of the abutment structure, while the *Embankment Point* was located on the embankment surface at a horizontal distance of 5.5 meters from the abutment axis. This location represents the end of the 5-meter approach slab, where elevation discontinuity or a potential “bump” is anticipated to occur.

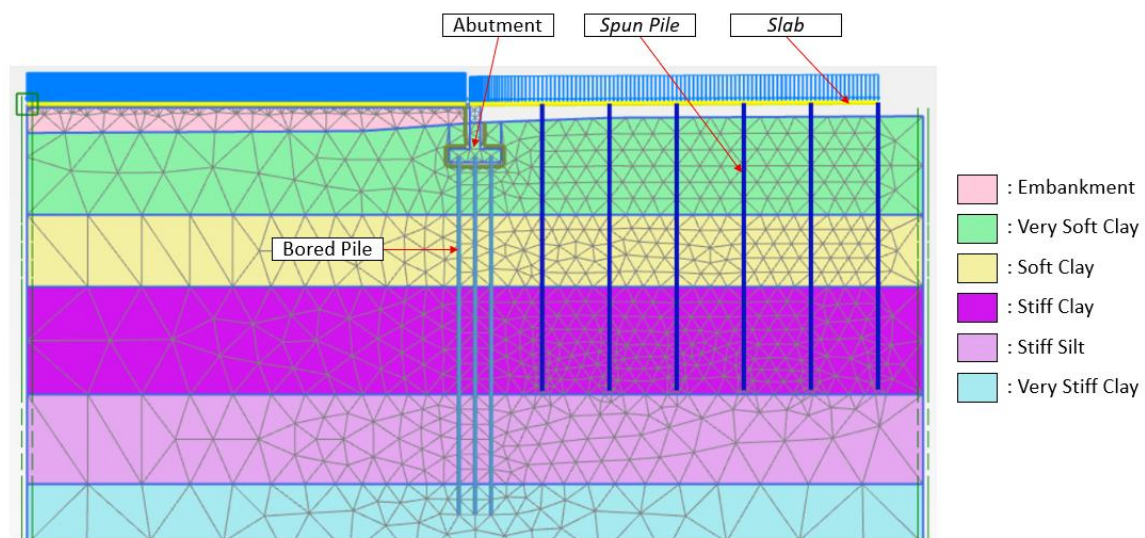


Figure 4. Geometry and Mesh Numerical Models

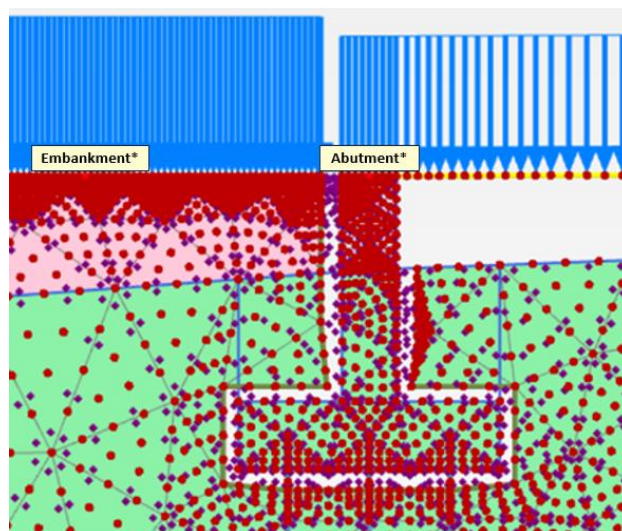


Figure 5. Selection of Embankment and Abutment Observation Points (Nodes)

RESULTS AND DISCUSSION

Characteristics of Existing Soil

This study adopts a case study from the Serang–Panimbang Toll Road Section III Project, focusing on the transition zone between STA 69+575 and STA 69+675. To ensure a high level of reliability in the input data, borehole logs and laboratory test results were subjected to correction and parameter correlation to verify the consistency of the soil layers. Inaccuracy in consolidation settlement prediction often arises from uncertainties in field conditions and laboratory data interpretation. Therefore, determining compressibility parameters such as the compression index (C_c) plays a vital role, and selecting an appropriate empirical correlation is crucial to achieving agreement between predicted and actual field conditions (Wahyudi et al., 2021).

The soil data processing involved three main stages. First, corrections were applied to the field NSPT values to obtain standardized corrected NSPT values. Second, the soil stratigraphy was regrouped based on consistency ranges into categories of very soft, soft, stiff, and very stiff clay layers. Third, a cross-check was conducted for each physical and mechanical soil parameter to identify and eliminate anomalous data, ensuring that all parameters used were relevant and consistent with their respective consistency groups. Table 3 presents a summary of the soil data used in both analytical and numerical calculations.

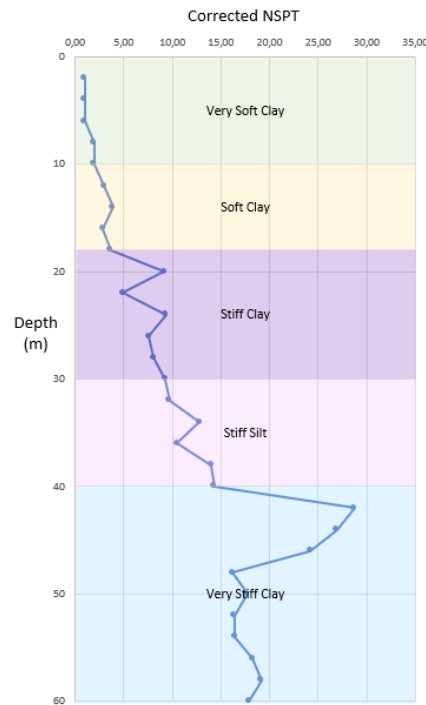


Figure 6. Existing Soil Layer Stratigraphy

Table 3. Recapitulation of Existing Soil Characteristics

	Very Soft Clay	Soft Clay	Stiff Clay	Stiff Silt	Very Stiff Clay
NSPT Correction	1.40	3.37	9.19	12.27	20.20
γ_{sat} (kN/m³)	14.05	16.38	18.15	18.15	18.80
σ' (kN/m²)	20.25	89.32	195.60	285.25	440.00
wc (%)	102.70	53.90	42.50	22.89	34.40
and	2.75	1.45	1.08	1.08	0.93
Sr (%)	100.00	100.00	100.00	100.00	100.00
Gs	2.68	2.68	2.68	2.62	2.68
LL (%)	97.95	89.85	76.90	76.90	75.00
PL (%)	38.60	39.45	41.40	41.40	41.40
PI (%)	59.35	50.40	35.50	35.50	33.60
Cc	0.792	0.719	0.602	0.602	0.585
Cv (cm²/sec)	0.00064	0.00119	0.00274	0.00274	0.00317
Cs	65.80	158.22	379.90	576.54	949.43
Pc (kN/m²)	0.297	0.172	0.127	0.127	0.117
Cu (kN/m²)	12.6	20.2	69.4	99.9	121.2
c' (kN/m²)	8.4	13.5	46.3	121.2	80.8

Numerical Model Validation with Analytical Calculations

Before performing the numerical simulation, analytical calculations were conducted to establish benchmark values for settlement prediction. Based on the one-dimensional consolidation analysis (Terzaghi) under the critical assumption of an overconsolidated (OC) condition, the total settlement of the embankment segment was predicted to reach 55.90 cm. This magnitude is primarily governed by the high compressibility of the 10 meter thick very soft clay layer near the surface, which exhibits a high compression index (C_c). Meanwhile, for

the abutment structure, the pile-group analysis produced an estimated settlement of 6.41 cm (Poulos & Davis method) and 5.44 cm (Equivalent method). The value of 6.41 cm was adopted as the validation reference target, representing the behavior of the pile foundation system in effectively transferring loads to deeper soil layers.

The reliability of the PLAXIS 2D finite element model (FEM) was verified by comparing the predicted final settlements against the conventional analytical calculations. Validation was conducted at two critical observation points: (1) the embankment surface point, representing the consolidation behavior of soft soil under embankment loading, and (2) the abutment head point, representing the response of the pile foundation system. A summary of the settlement comparison results is presented in Table 4.

Table 4. Comparison of Total Settlement Results of Analytical and Numerical Methods

Vantage Point	Analytical Methods (cm)	Numerical Method (cm)	Deviation (%)
Embankment	55.30	51.90	6.09
Abutmen	6.41	6.10	4.93

As shown in the Table 4, the deviation in the predicted embankment settlement is only 6.09%. This indicates that the use of the Soft Soil material model, with compressibility parameters (C_c , C_s , λ^* , and κ^*) calibrated under the OC condition is capable of accurately representing the compressibility behavior of the soft soil at the study location, comparable to the prediction obtained from the Terzaghi 1D method. Meanwhile, the numerical model predicts an abutment settlement of 6.10 cm, which falls within the range of the pile-group analysis results obtained from the Poulos & Davis method. This demonstrates that the soil pile interaction has been properly represented in the numerical model and that the piles are able to effectively mobilize skin friction without experiencing bearing capacity failure.

Total Settlement Profile of Transition Zone

Based on the validated model, the total settlement profile along the transition zone was mapped. Figure 7 presents the vertical displacement contours at the end of the construction stage, showing a pronounced contrast in behavior. At the left end of the model (red zone), the ground experiences the maximum settlement. This phenomenon is governed by two contributing factors. The first factor is the loading effect associated with the embankment geometry. At the free-field area near the model boundary, the design embankment height is greater than in the region near the abutment. According to Terzaghi's consolidation principles, the magnitude of settlement is proportional to the applied stress increment ($\Delta\sigma'$). Consequently, the subsoil in this area receives the highest overburden load and undergoes the greatest settlement.

The second factor is the boundary condition. The left-end area is a free zone unaffected by structural stiffness, allowing the soil to undergo unrestricted one-dimensional compression. In contrast, approaching the abutment area (yellow to blue zones), the settlement progressively decreases due to the restraining effect of the rigid abutment wall and bored piles. Through soil–

structure interface friction, the abutment partially restrains the adjacent embankment mass, preventing it from settling fully.

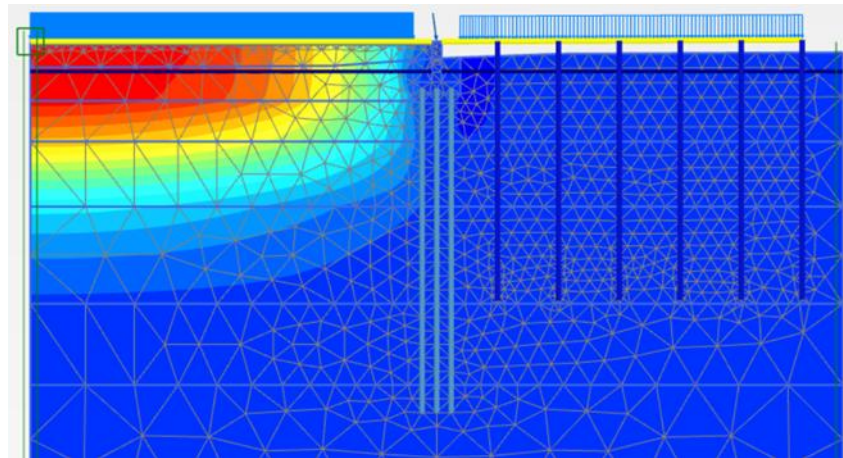


Figure 7. Total Displacement Profile and Vertical Direction Soil Movement (U_y) of Model

Evaluate *Safety Factors* at Each Stage of Work

In addition to the settlement evaluation, a global stability analysis was conducted at each critical construction stage to ensure that no slope failure or bearing capacity failure occurred during construction or throughout the service life. A summary of the Safety Factor (SF) values obtained from the shear strength reduction (Phi-c reduction) analysis is presented in Figure 8. Based on these results, the lowest SF value was recorded during the *Concrete Spun Pile Construction* stage, at 4.087, while during the operational stage under full traffic loading, the SF reached 6.777. All safety factor values across the construction phases consistently exceed 1.5, which is the minimum required permanent stability threshold stipulated in SNI 8460:2017. This indicates that despite the substantial settlement occurring within the soft soil, the embankment geometry and the planned ground reinforcement system possess sufficient strength reserves to mitigate stability related risks.

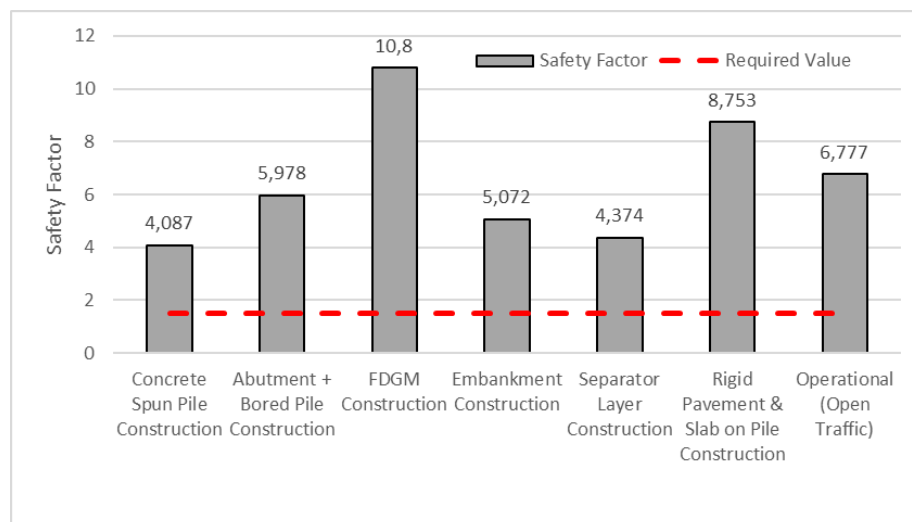


Figure 8. Recapitulation of Each Staged Construction's Safety Factor

Consolidation Time and Evaluation of Rate of Settlement

Aside from the magnitude of total settlement, the post construction rate of settlement is a key indicator for determining the long term serviceability of toll road infrastructure. Referring to geotechnical practice standards in Indonesia, such as those outlined in *Panduan Geoteknik 4*, settlement performance evaluation typically focuses on the condition at which 90% degree of consolidation (U_{90}) is achieved. This criterion is based on the consideration that once U_{90} is reached, the remaining settlement will be relatively small and slow, thereby minimizing the risk of sudden deformation (DPPW, 2002).

The difference in long term settlement behavior between the two structural components is presented in Figure 9. The graph illustrates the development of cumulative settlement over time to identify the point at which U_{90} is reached. At the embankment point, U_{90} is achieved in year 116, indicating that the underlying soft clay requires more than a century to complete its primary consolidation process. Meanwhile, at the abutment point, U_{90} is reached in year 80. Further evaluation at the Abutment Node shows excellent stability. At U_{90} , the total cumulative settlement of the abutment is only 5.5 cm. When this value is considered relative to the duration of settlement 80.15 years the average settlement rate of the abutment remains far below the allowable limit of 2 cm/year and the long term deformation criterion of 10 cm/10 years. This low settlement rate demonstrates that the bored-pile foundation system performs effectively as a rigid support capable of limiting vertical movement within safe bounds throughout the service life.

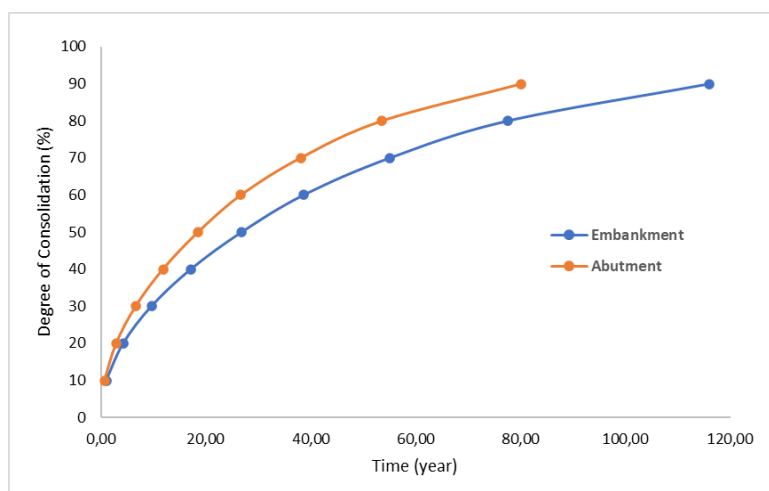


Figure 9. Achievement Time of Degree of Consolidation at Embankment and Abutment

Table 5 shows that the embankment segment does not meet the performance requirements during the early operational phase. Referring to the criteria in *Panduan Geoteknik 4*, the allowable residual settlement rate for toll roads is a maximum of 2 cm/year. The data indicate that in Year 1, the settlement rate reaches 5.1 cm/year, and in Year 2, it reaches 2.1 cm/year. Both values exceed the allowable threshold, indicating that primary consolidation is still highly active during the first two years of service. The long-term performance evaluation for the first ten years after construction also shows noncompliant results. The total accumulated settlement from the start of operation to Year 10 is 16.14 cm. This value significantly exceeds the long-term deformation limit required for toll road infrastructure, which is a maximum of

10 cm over 10 years. This indicates that the embankment structure still exhibits substantial settlement potential even after a decade of operation, directly affecting riding safety and comfort and increasing the frequency of maintenance throughout its service life.

Table 5. Rate of Settlement of Embankment

Time (Year)	Degree of Consolidation (%)	Cumulative Settlement (cm)	Rate of Settlement (cm/year)	Service Status
1	9.65	5.105	5.105	Not Acceptable
2	13.64	7.219	2.114	Not Acceptable
3	16.71	8.842	1.622	Not Acceptable
4	19.30	10.209	1.368	Not Acceptable
5	21.57	11.414	1.205	Not Acceptable
6	23.63	12.504	1.089	Not Acceptable
7	25.53	13.506	1.002	Not Acceptable
8	27.29	14.438	0.933	Not Acceptable
9	28.95	15.314	0.876	Not Acceptable
10	30.51	16.142	0.828	Not Acceptable

Differential Decline of Transition Zones

The magnitude of differential settlement at $U_{90}\%$ is calculated as the difference in settlement between the embankment observation point (end of the approach slab) and the abutment structure, as follows:

$$\Delta Sc = Sc(90\%) \text{ Embankment} - Sc(90\%) \text{ Abutment}$$

$$\Delta Sc = 47.70 \text{ cm} - 5.50 \text{ cm} = 42.20 \text{ cm}$$

Based on the safety criteria, the differential settlement value (ΔSc) of 42.20 cm far exceeds the absolute tolerance limit of 2.0 cm recommended by Zhang (2010) to prevent vehicle dynamic hazards (loss of traction) on high-speed highways. From a serviceability perspective, assuming an effective transition length equal to the approach slab length (5 meters), the resulting angular distortion is approximately $1/8.4$. This value is far above the structural allowable limit of $1/300$ specified in the 2024 Pavement Design Manual (MDP) and SNI 8460:2017 to prevent structural damage within the transition zone.

These evaluation results confirm that the existing soft clay improvement method using a 5-meter-deep FDGM replacement is ineffective for controlling settlement under very deep soft-soil conditions (> 15 meters). The FDGM layer functions only as a drainage layer and a near-surface ground improvement, but it does not increase the stiffness of the deeper soft clay layers beneath it.

CONCLUSION

Based on the conducted study on differential settlement prediction in the transition zone, the following conclusions can be drawn: (1) The validation between analytical methods and numerical modeling shows a high level of agreement, with settlement deviations of 6.09% for the embankment and 4.93% for the abutment. This demonstrates that the analytical approaches (Terzaghi 1D, Boussinesq, and Poulos–Davis) can still represent the global settlement behavior at the study location. (2) The embankment supported by FDGM experiences a total settlement

of 51.90 cm, driven by the high compressibility of the deep soft clay layers. In contrast, the Slab-on-Pile abutment undergoes only 6.10 cm of settlement due to the stiff soil–pile interaction. The resulting differential settlement of 45.80 cm far exceeds the allowable limits. (3) Global stability analysis throughout all construction stages indicates that the structure remains safe, with Safety Factor (SF) values ranging from 4.087 to 10.80. All SF values are significantly above the minimum requirement of 1.5 as specified in SNI 8460:2017, indicating a very low risk of failure or bearing capacity collapse despite the deep soft soil. (4) The abutment structure supported by bored-pile groups demonstrates excellent long-term performance, with a total settlement of only 6.10 cm. The associated settlement rate remains far below the allowable limit of 2 cm/year, confirming that the foundation design effectively transfers loads to the deeper soil layers. (5) The embankment segment improved only by FDGM replacement experiences substantial consolidation settlement, reaching 51.90 cm. The consolidation process is extremely slow, with 90% degree of consolidation (U_{90}) achieved only after approximately 116 years. The early operational settlement rates are also unacceptable, exceeding the criteria of 2 cm/year and 10 cm over 10 years. (6) Overall findings indicate that FDGM alone is insufficient to control long-term settlement in deep soft-soil conditions. Although global stability is ensured, the settlement and serviceability criteria are not satisfied, posing a high risk of operational issues such as bumps, differential settlement, and pavement cracking within the transition zone.

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