

Numerical Analysis of Soil Improvement on Retention Pond Embankment Structure

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Abstract— The coastal areas along the northern coast of Java, particularly the Semarang–Demak region, are characterized by thick soft clay deposits with high compressibility and low shear strength. Compounding these challenges, progressive land subsidence increases flood risk, necessitating the construction of retention ponds as hydraulic control infrastructure. This study presents a numerical investigation of the performance of Prefabricated Vertical Drains (PVD) combined with preloading and geotextile reinforcement to stabilize retention pond embankments constructed over coastal soft soil. Finite element analyses were performed using PLAXIS 2D under plane strain conditions. Two scenarios were evaluated: (1) a baseline condition without reinforcement and (2) a fully improved condition incorporating PVD, a sand drainage layer, and geotextile reinforcement. Results indicate that the unreinforced embankment failed to achieve stability under the combined hydrostatic, traffic, and seismic loading. The integrated improvement system effectively dissipated excess pore water pressure, reduced total deformation to 34 cm, and achieved a safety factor of 1.55, satisfying the minimum requirement of 1.5 stipulated by SNI 8460:2017. These findings provide practical guidance for the geotechnical design of retention pond infrastructure in subsidence-affected coastal areas.

Keywords— Finite Element Method, Prefabricated Vertical Drain (PVD), Retention Pond, Soft Soil Improvement.

I. INTRODUCTION

Land subsidence is a phenomenon of gradual ground surface lowering resulting from the consolidation of underlying soil layers, caused by both natural and anthropogenic factors. From a geotechnical perspective, land subsidence occurs when effective stresses within the soil mass increase beyond its structural capacity, triggering the compression of soil particles and the expulsion of pore water from the soil voids [1]–[3]. The primary causes of land subsidence include excessive groundwater extraction, structural loading from buildings and infrastructure, natural consolidation of young alluvial deposits, and tectonic activity. In coastal areas, the impacts of land subsidence are compounded by relative sea level rise, progressively worsening tidal flooding over time. A thorough understanding of land subsidence dynamics is therefore crucial for designing embankment structures capable of adapting to long-term changes in geotechnical conditions, particularly in maintaining the embankment crest elevation to function optimally against relative sea level rise [4],[5].

Various methods for addressing land subsidence in soft soils have been developed and extensively researched over recent decades. These methods can be broadly categorized into: (1) vertical drainage methods such as sand drains and

Prefabricated Vertical Drains (PVD); (2) grouting injection methods to fill subsurface voids and enhance bearing capacity; (3) dynamic compaction methods that utilize impact energy to densify the soil; (4) pile or micropile methods to structurally stabilize the ground; and (5) chemical improvement methods such as cement or lime stabilization [6]–[8]. Each method possesses distinct characteristics, advantages, and limitations that must be considered in accordance with field conditions [8]. The conventional sand drains, although effective in accelerating consolidation, require very large volumes of sand, involve relatively long construction periods, and are susceptible to flow disruption due to soil contamination surrounding the drain [9],[10]. Meanwhile, the grouting method applied was proved effective for coarse-grained soil conditions but is suboptimal in saturated soft clay due to the difficulty of grout penetration into fine soil pores. The dynamic compaction method investigated is effective on granular soils but is not recommended for soft clay as it may actually worsen excess pore pressure conditions. Piles and micropiles can provide good structural stability but require very high initial investment and do not directly address the long-term consolidation problem of thick soft clay layers [11]–[14].

Prefabricated Vertical Drains (PVD) combined with preloading are widely recognized as one of the most cost-effective methods for accelerating consolidation in soft cohesive soils. Compared to other treatment methods, PVD offers several significant advantages that make it the preferred choice for coastal soft soil improvement. First, in terms of time efficiency, PVD shortens drainage paths from vertical to radial, dramatically reducing consolidation time from decades to only a few months cannot be achieved by grouting or dynamic compaction in saturated clay. Second, in terms of cost, PVD is far more economical than concrete piles or micropiles as it does not require expensive structural building materials and its installation using a mandrel is relatively straightforward. Third, PVD is environmentally friendly, being based on polypropylene material that does not disturb the chemical equilibrium of the soil, unlike cement or lime stabilization methods that can permanently alter soil mineral composition. Fourth, PVD can be combined with other reinforcement systems such as geotextiles and preloading to create an integrated improvement system that addresses multiple failure modes synergistically. However, the PVD installation process induces a disturbed zone around the mandrel (smear zonesmear zone) and limited flow capacity of the drain core (well resistance), both of which

reduce theoretical consolidation efficiency and must be accounted for in design [15]-19].

Plane strain finite element analysis using PLAXIS 2D provides a robust framework for capturing the coupled nonlinear hydro-mechanical behavior of PVD-improved soft soils, particularly when combined with back-analysis of field instrumentation data. This approach enables accurate prediction of deformation magnitudes and time-dependent consolidation behavior under realistic loading sequences [20]-[24].

This study addresses a gap in the literature by applying equivalent plane strain finite element modeling to retention pond embankments—a loading geometry and operational context that has received far less attention than road embankments in existing literature on PVD-improved coastal soils. The primary objectives of this study are: (i) to quantify total deformation under the integrated PVD–preloading–geotextile improvement system; (ii) to compare the structural performance between reinforced and unreinforced embankment configurations; and (iii) to evaluate the resulting safety factor against the applicable Indonesian National Standard (SNI).

II. LITERATURE REVIEW

A. Land Subsidence in Coastal Areas

Land subsidence in the coastal areas of Semarang has been extensively documented. Abidin et al. [1] demonstrated that subsidence is progressive and spatially concentrated in young alluvial deposits, driven by a combination of natural consolidation and anthropogenic factors including groundwater extraction and structural loading. These findings underscore the urgency of addressing subsidence-related risks in infrastructure planning along the Pantura corridor. This phenomenon directly affects the long-term stability of embankment infrastructure, particularly in maintaining the embankment crest elevation to function optimally under continuously rising relative sea level conditions.

B. Prefabricated Vertical Drain (PVD) and Preloading

The effectiveness of PVD combined with preloading for accelerating consolidation in soft clay has been validated through both laboratory and field evidence. Saowapakpiboon et al. [6] confirmed that PVD application substantially accelerates pore pressure dissipation and reduces consolidation time compared to natural vertical drainage alone. The radial consolidation efficiency produced by PVD is evaluated using the Barron equation [4]: $U_h = 1 - \exp(-8Th / F(n))$, where U_h is the average degree of radial consolidation, $Th = ch \cdot t / de^2$ is the radial time factor, ch is the horizontal consolidation coefficient, t is time, de is the equivalent diameter of the drain influence zone, and $F(n)$ is a geometric factor dependent on the drain spacing ratio. The effects of the smear zone and well resistance resulting from mandrel installation are accounted for using the approach of Indraratna and Redana [5], which modifies the Barron equation to incorporate the disturbed soil zone around the drain, thereby realistically correcting the theoretical consolidation efficiency [25][26].

C. Finite Element Modeling of Coastal Soft Soil

Indraratna and Redana [7] demonstrated that finite element simulation can accurately reproduce the deformation and stability behavior of coastal soft soils. Their work showed that the combination of ground improvement and reinforcement measures yields measurable improvements in structural performance, thereby providing a methodological precedent for the numerical approach adopted in this study. The Mohr–Coulomb (M-C) constitutive model with Undrained A drainage conditions was selected for all clay layers due to its capability to capture shear failure behavior under rapid loading conditions, where excess pore pressures have not yet dissipated. The selection of the M-C model was based on the availability of soil parameters from standard field and laboratory tests, as well as its consistency with geotechnical design practice in Indonesia as stipulated by SNI 8460:2017 [10].

D. Loading on Retention Pond Embankment Structure

Retention pond embankments are subjected to a combination of loads that are characteristically different from conventional road embankments, due to the presence of cyclically fluctuating water pressure. Based on a review of the literature and applicable design standards, three main types of loading are relevant to this structure: hydrostatic pressure, traffic load, and seismic load.

1) Hydrostatic Pressure

Hydrostatic pressure is the dominant load on the walls and slopes of a retention pond embankment that are in direct contact with the stored water. The water pressure force acting on a horizontal surface is calculated based on the principle of static fluid pressure:

$$F = \gamma_w \cdot h \cdot A \quad (1)$$

where,

F = hydrostatic pressure (kN/m),
 γ_w = unit weight of water (kN/m³),
 h = depth from water surface (m),
 A = surface area (m²),

where F is the resultant water pressure force (kN), γ_w is the unit weight of water equal to 9.81 kN/m³, h is the depth of the centroid of the surface from the water surface (m), and A is the area of the pressurized surface (m²). For an inclined surface at angle θ to the horizontal, the water pressure force acting perpendicular to the inclined surface is formulated as:

$$F = \gamma_w \cdot h \cdot A / (2 \sin \theta) \quad (2)$$

The point of application of the resultant hydrostatic force is located at the center of pressure of the surface, which for an inclined surface lies deeper than the centroid of the surface. Equation (2) is important in embankment design because inclined slopes experience a non-uniform pressure distribution, such that the resultant force and its point of application determine the overturning moment that must be resisted by the embankment structure.

2) Traffic Load

Traffic load on the embankment crest is modeled as a uniformly distributed live load (UDL). The magnitude of this load is determined based on road classification in accordance with SNI 1725:2016 on Loading for Bridges [27], which is

also adopted as a reference for arterial road infrastructure. For the primary arterial road classification, the uniformly distributed load intensity applied is:

$$q = 15 \text{ kN/m}^2 \quad (3)$$

This load is applied uniformly over the full width of the embankment crest and reflects the cumulative effect of light to heavy vehicles traversing the embankment during its operational period. In the context of geotechnical design, traffic load imposes additional vertical stress on the embankment body and the underlying soil layers, thereby contributing to accelerated settlement and a reduction in slope safety factor if not adequately accounted for.

3) Seismic Load

Seismic loading on the embankment is analyzed using the pseudo-static method in accordance with SNI 1726:2019 on Seismic Resistance Design Procedures for Building and Non-Building Structures [28]. The pseudo-static approach converts the dynamic effects of an earthquake into equivalent static horizontal and vertical forces acting on the embankment soil mass. The site-corrected surface acceleration is calculated as:

$$\text{PGAm} = \text{FPGA} \times \text{PGA} \quad (4)$$

where,

PGAm = MCEG peak ground acceleration at the ground surface adjusted for site class conditions,

PGA = peak ground acceleration at bedrock,

FPGA = site amplification factor for ground acceleration due to local soil conditions,

where PGAm is the Modified Peak Ground Acceleration, FPGA is the acceleration amplification factor for zero period dependent on site class, and PGA is the design peak ground acceleration at bedrock. The horizontal (kh) and vertical (kv) seismic coefficients used in the pseudo-static analysis are then derived as:

$$kh = 0,5 \times (\text{PGAm} / g) \quad (5)$$

$$kv = 0,5 \times kh \quad (6)$$

where,

kh = horizontal seismic coefficient (g),

kv = vertical seismic coefficient (g),

ad = corrected seismic acceleration (PGAm),

g = gravitational acceleration (m/s^2),

where g is the gravitational acceleration (9.81 m/s^2). The factor of 0.5 in the seismic coefficient determination reflects the allowable reduction for pseudo-static analysis of earth structures, acknowledging that the instantaneous peak acceleration does not fully generate inertial forces of equal magnitude in a soil mass behaving as a multi-degree-of-freedom system. The horizontal seismic inertial force acting on a soil slice in slope stability analysis is then expressed as $F_h = kh \times W$, where W is the weight of the soil slice under consideration.

E. Slope Stability and Safety Factor

Slope stability evaluation of the embankment in the finite element analysis was performed using the Strength Reduction Factor (SRF) method, also known as phi-c reduction. In this method, the soil shear strength parameters (cohesion c and internal friction angle ϕ) are progressively reduced by a

multiplier factor Rf until the system reaches a failure condition indicated by non-convergence of the numerical solution. The safety factor (SF) is defined as:

$$\text{SF} = c / c' = \tan \phi / \tan \phi' \quad (7)$$

where c' and ϕ' are the reduced shear strength parameters at the failure limit condition. SNI 8460:2017 [10] requires a minimum safety factor of $\text{SF} = 1.50$ for conditions of high analytical uncertainty, as applicable to soft soil cases with significant geotechnical parameter variability and complex loading combinations. This requirement serves as the primary benchmark for evaluating the feasibility of the proposed soil improvement system.

Despite these advances, comparative studies evaluating different improvement systems through numerical modeling in the specific context of retention pond loading remain limited. Most existing work has focused on road embankments [2], leaving the distinct loading and operational characteristics of hydraulic control structures—particularly the simultaneous combination of cyclic hydrostatic pressure, traffic load, and seismic load—relatively underexplored. This study addresses that gap by applying a unified loading framework that considers all three load types simultaneously in the finite element model, thereby providing a more comprehensive and realistic assessment of geotechnical performance for retention pond embankments over subsidence-affected coastal soft soils.

III. MATERIALS AND METHODS

A. Study Location and Soil Characterization

The study area location along the Semarang–Demak coastline is shown in Figure 1. The study site is located at coordinates $6^\circ 56' 26.96''\text{S}$ $110^\circ 27' 54.61''\text{E}$ in an alluvial lowland identified as highly susceptible to land subsidence.



Fig. 1. Study Location

The subsurface conditions were characterized through three borehole investigations (BH-1, BH-2, BH-3) to a depth of 80 m. The stratigraphic profile (Figure 2) reveals four main soil units: (1) Fat Clay, Very Soft (0–30 m); (2) Fat Clay, Soft to Medium (16–33.5 m); (3) Fat Clay, Medium to Stiff (33.5–57.5 m); and (4) Fat Clay, Stiff to Very Stiff (57.5–80 m), with a Silty Sand lens (Very Loose) found locally at BH-3 between depths of 4.7 and 9 m.

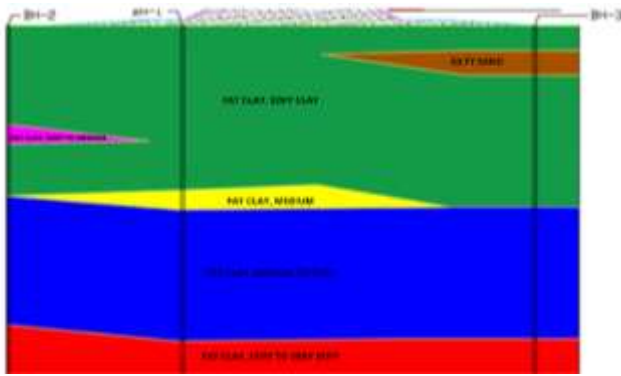


Fig. 2. Soil Stratification

Geotechnical parameters were determined from laboratory testing and correlated with SPT-N values. Table I summarizes the PLAXIS modeling parameters used for all soil units.

TABLE I. SOIL MODELING PARAMETERS

Parameter	FAT CLAY Very Soft	FAT CLAY Soft-Medium	FAT CLAY Medium	FAT CLAY Medium-Stiff	FAT CLAY Stiff-Very Stiff	Silty Sand
Model	M-C	M-C	M-C	M-C	M-C	M-C
γ_{sat} (kN/m ³)	15.49	15.69	17.24	17.60	17.64	20.00
γ_{unsat} (kN/m ³)	12.49	13.69	15.24	15.60	15.64	17.80
e	2.02	1.57	1.14	1.09	1.03	0.30
E (kN/m ²)	3,923	4,120	5,150	6,982	19,565	13,890
ν	0.40	0.35	0.30	0.30	0.20	0.40
c (kN/m ²)	8	25	35	50	100	1.96
ϕ (°)	19	21	24	26	31	27
$k_x = k_y$ (m/d)	8.64×10^{-4}	8.64×10^{-4}	8.64×10^{-4}	8.64×10^{-4}	8.64×10^{-4}	0.864

B. Embankment Design and Soil Improvement

The structural profile of the retention pond embankment and the modeled soil improvement system configuration are shown in Figure 3. The embankment consists of 3 m of compacted fill with the following material properties: $\gamma_{sat} = 19.63$ kN/m³, $\gamma_{unsat} = 15.68$ kN/m³, $E = 11,000$ kN/m², $\nu = 0.30$, $c = 13$ kN/m², $\phi = 29.5^\circ$, and $k_x = k_y = 0.000451$ m/day.

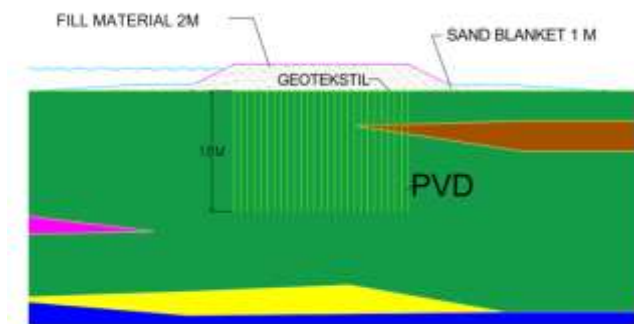


Fig. 3. Recommended Embankment Design

The soil improvement system consists of three integrated components:

(1) PVD System

Prefabricated Vertical Drain (PVD) is a factory-manufactured drainage element made from geosynthetic

material, typically polypropylene, consisting of a corrugated core encased in a geotextile filter.



Fig. 4. Prefabricated Vertical Drain [29]

The primary function of PVD is to shorten the pore water drainage path from the vertical direction to the radial horizontal direction toward the drain core, thereby accelerating the consolidation process of soft soils from what would otherwise take decades to only a few months.

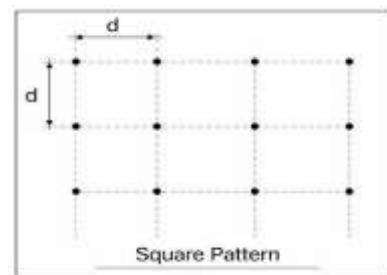


Fig. 5. PVD Installation Pattern

The drain spacing in the modeling was set at 1 m. The PVD installation pattern was assumed to follow a square pattern, so that the vertical drainage effect was modeled equivalently and conservatively in the two-dimensional analysis. In this way, PVD effectively increases the shear strength of the soil, reduces the potential for long-term settlement, and accelerates subground stabilization before or during construction.

Polypropylene PVD (cross-section 100 mm $\times$ 3.8 mm) with tensile strength ≥ 22 kN and flow capacity $\geq 150 \times 10^{-3}$ m³/s at a confining pressure of 300 kPa were installed to a depth of 18 m at 1 m spacing in a square pattern. In PLAXIS 2D, the PVD is represented as a drain element providing vertical drainage conduits under the equivalent

plane strain transformation.

(2) Sand Layer

The sand blanket layer is a high-permeability granular material layer placed on the surface of the soft soil prior to embankment construction. Its primary function is as a horizontal drainage channel that collects and discharges pore water expelled from the soft soil through the PVD heads.

In addition to serving as a drainage medium, the sand blanket also functions as a working platform to facilitate PVD installation and to provide a more uniform load distribution on the underlying soft soil.



Fig. 6. Sand Blanket Leveling Above Installed PVD [30]

A 1 m thick sand drainage layer ($\gamma_{sat} = 19.5 \text{ kN/m}^3$, $E = 60,000 \text{ kN/m}^2$, $k_x = k_y = 432 \text{ m/day}$) was placed at the base of the embankment to collect and laterally discharge pore water expelled through the PVD heads.

(3) Geotextile Reinforcement

Geotextile is a sheet-form geosynthetic material made from polymer fibers, such as polyester or polypropylene, placed within or between soil layers to fulfill various geotechnical functions.

In the context of embankment reinforcement over soft soils, the primary function of the geotextile is as a reinforcement element that provides tensile resistance at the base of the fill. The geotextile acts by resisting lateral forces caused by embankment loading and preventing the lateral spreading of fill material, thereby enhancing slope stability and preventing base shear failure at the interface between the fill and the underlying soft soil.

A high-tensacity woven polyester geotextile with a tensile strength of 200 kN/m (machine direction) was placed at the interface between the sand layer and the fill. In PLAXIS 2D, this element is modeled as a geogrid with axial stiffness $EA = 200 \text{ kN/m}$, providing tensile resistance against lateral spreading and base shear failure.

C. Loading Conditions

Three types of loads were applied in the finite element model, consistent with the design service life of the embankment:

(1) Hydrostatic Pressure

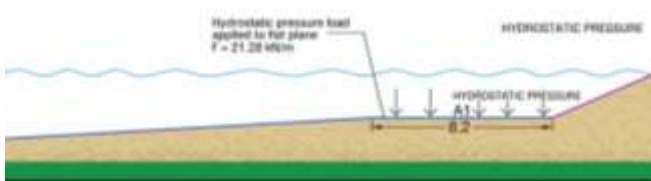


Fig. 7. Hydrostatic Pressure Load on Horizontal Surface

The retention pond is designed for a maximum water depth of 3.17 m. Hydrostatic pressure was calculated per unit length for the horizontal base, upper inclined slope, and lower inclined slope of the embankment geometry using:

$$F = \gamma_w h A \quad (1)$$

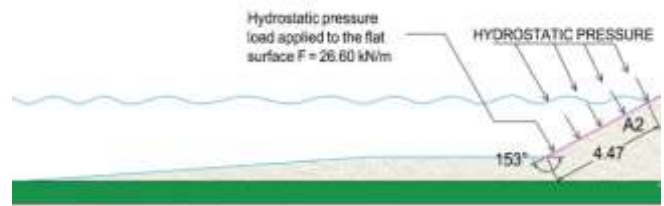


Fig. 8. Hydrostatic Pressure Load on Upper Inclined Surface

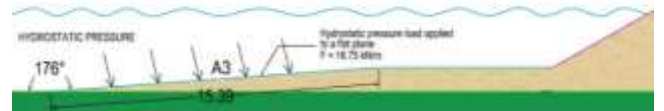


Fig. 9. Hydrostatic Pressure Load on Lower Inclined Surface

Load values applied based on the hydrostatic pressure calculation results with a maximum water depth of 3.17 m and $\gamma_w = 9.81 \text{ kN/m}^3$, the load values input to the finite element model are: (a) horizontal base surface: 21.28 kN/m, (b) upper inclined surface: 26.60 kN/m, and (c) lower inclined surface: 16.75 kN/m. These three values were applied as distributed loads perpendicular to the respective embankment surface areas in direct contact with the retained water.

(2) Traffic Load

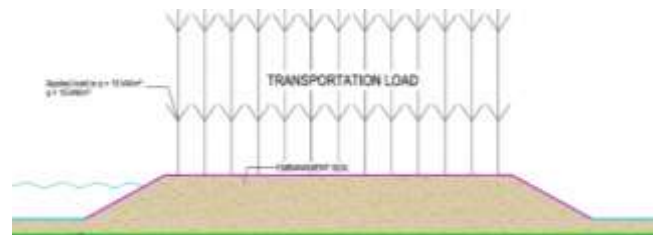


Fig. 70. Traffic Load

Load values applied Based on the provisions of SNI 1725:2016 for the primary arterial road classification [27], the traffic load value used in the modeling is $q = 15 \text{ kN/m}^2$. This load is applied uniformly over the full width of the embankment crest as a uniformly distributed live load (UDL) representing the cumulative effect of light to heavy vehicles during the embankment's operational period.

(3) Seismic Load

Pseudo-static seismic analysis was adopted in accordance with SNI 1726:2019. Using a design peak ground acceleration of $PGA = 0.315g$ for the Semarang coastal area and a site coefficient of $FPGA = 1.6$ (site class SE, soft soil), the adjusted surface acceleration is:

$$PGA_m = F_p^{GA} \times PGA = 1.6 \times 0.315 = 0.504g \quad (3)$$

The horizontal and vertical seismic coefficients are derived as $k_h = 0.5 \times (ad/g) = 0.02569g$ and $k_v = 0.5k_h = 0.01284g$, respectively.

The seismic load values applied are $PGA_m = 0.504g$, horizontal seismic coefficient $k_h = 0.02569g$ and vertical seismic coefficient $k_v = 0.01284g$.

D. Finite Element Modeling Procedure

Plane strain finite element analysis was performed in PLAXIS 2D using the Mohr–Coulomb constitutive model with Undrained A drainage type for all clay layers. The numerical model encompasses the following construction stages: (1) initial stress generation (K0 procedure); (2) activation of PVD drain elements and sand layer; (3) staged embankment construction with consolidation phases; (4) application of hydrostatic and traffic loads with continued consolidation; and (5) pseudo-static seismic loading with stability analysis (ϕ/c reduction method). Two modeling scenarios were investigated: a baseline condition without any ground improvement or reinforcement, and a fully improved condition incorporating PVD, sand layer, and geotextile reinforcement.

IV. RESULTS AND DISCUSSION

A. Soil Structure Geometry and Initial Model Conditions

Fig. 11 presents the cross-section of the structure modeled in PLAXIS 2D under plane strain conditions (*plane strain*).

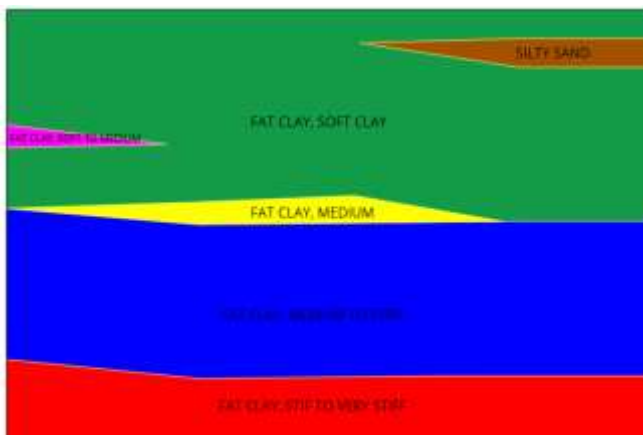


Fig. 81. Initial Soil Structure

B. Baseline Performance – Unreinforced Condition

Finite element analysis of the unreinforced embankment revealed that structural instability was reached before the full loading sequence could be completed. The safety factor dropped below the minimum permissible threshold during the initial embankment loading stage, and the analysis terminated prematurely due to non-convergence associated with the formation of a failure surface. The total displacement contour plot (Figure 12) shows a characteristic bulging deformation pattern concentrated beneath the embankment base, with displacement magnitudes exceeding 475 mm in the peak zone—indicating bearing capacity failure of the underlying very soft clay.

The excess pore pressure analysis (Figure 13) confirms severe pore pressure accumulation beneath the embankment (maximum value: +16.35 kN/m²; minimum: -61.18 kN/m² at Day 55), reflecting the inability of the low-permeability soft clay to dissipate pore pressures under the applied loading rate. These results are consistent with theoretical expectations for normally consolidated soft clay with NSPT = 1 and $c_u = 8$ kN/m² subjected to rapid loading [7].

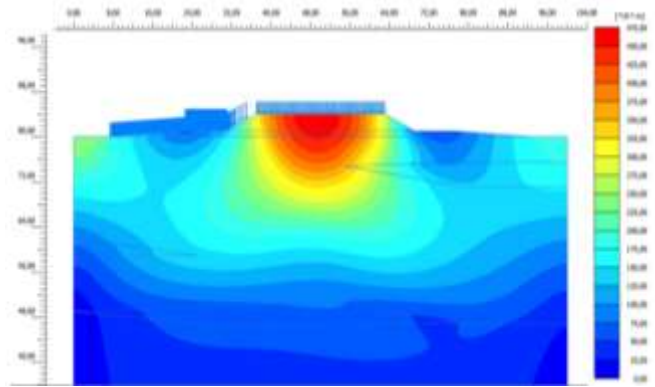


Fig. 92. Deformation Results Without Reinforcement

C. Improved Condition – PVD, Sand Layer, and Geotextile

The integration of the PVD system with the sand drainage layer and basal geotextile reinforcement fundamentally altered the hydro-mechanical response of the embankment. The fully improved model successfully completed all loading stages and achieved a stable equilibrium condition.

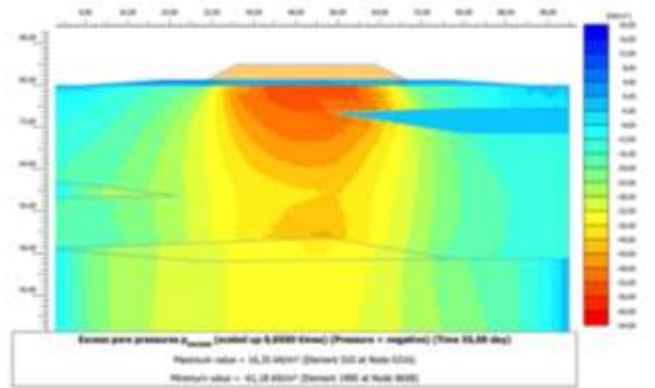


Fig. 103. Excess Pore Pressure Results Without Reinforcement

The total vertical displacement was 0.2048 m (settlement, downward) at the embankment crest, with a maximum horizontal displacement of 0.3772 m (Fig. 14) recorded at the embankment toe, resulting in a controlled deformation profile consistent with a consolidating rather than failing soil mass.

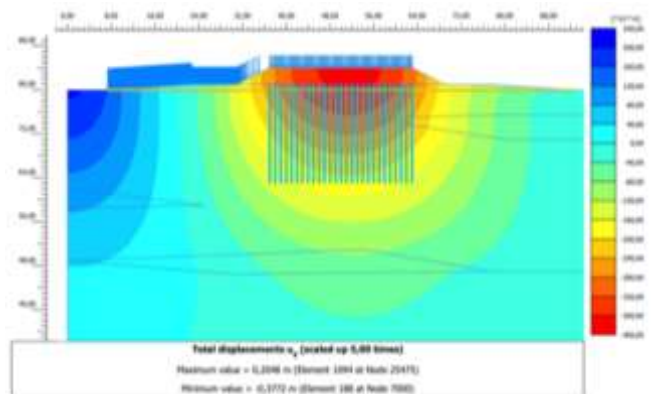


Fig. 114. Deformation Results With Reinforcement

The excess pore pressure distribution in the improved condition (Fig. 15) shows a dramatic reduction compared to the unreinforced case. The maximum excess pore pressure decreased to 0.1143 kN/m² at Day 40, with a minimum of -34.48 kN/m². The spatial concentration of the remaining excess pore pressure shifted to the deeper zone below the 18 m depth not addressed by PVD, confirming that the drain

network effectively intercepted and channeled upward pore water flow within the treated zone.

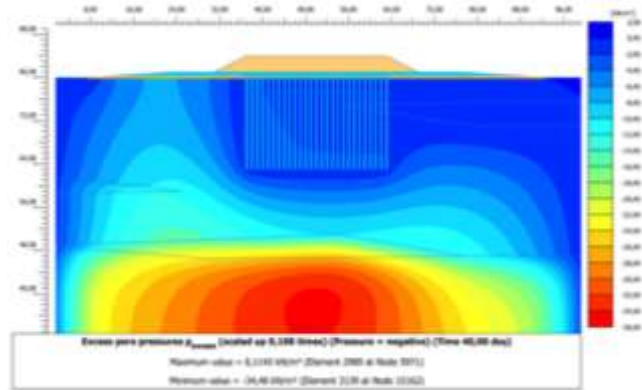


Fig. 12. Deformation Results with Reinforcement

The improvement in pore pressure dissipation directly translated into enhanced effective stress development and accelerated shear strength gain in the soft clay. This is evidenced by the ability of the improved model to sustain the applied hydrostatic, traffic, and seismic loads without structural failure—a condition that could not be achieved in the unreinforced scenario. These findings are consistent with reported field and numerical observations of PVD-accelerated consolidation in coastal soft clays.

D. Safety Factor Analysis

Slope stability was evaluated using the Shear Strength Reduction (SSR) method in PLAXIS 2D. For the unreinforced condition, the analysis terminated without convergence, indicating that no finite safety factor could be obtained—confirming absolute instability. For the improved condition, the evolution of the safety factor (Figure 16) exhibits three distinct phases:

1. a plateau phase at $SF \approx 1.20$ during consolidation under the embankment's self-weight, reflecting gradual strength gain;
2. an increasing phase during seismic loading as the PVD-improved consolidated clay mobilized its enhanced shear resistance; and (iii) a stable plateau at $SF = 1.55$ as the geotextile progressively carried the lateral tensile forces that could no longer be sustained by the soil matrix alone.

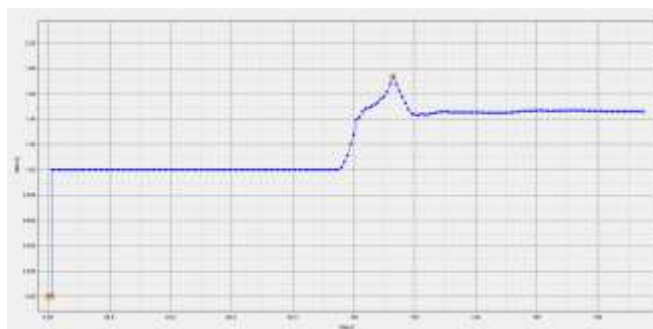


Fig. 13 Safety Factor

The computed final safety factor of $SF = 1.55$ satisfies the minimum requirement of $SF_{min} = 1.50$ for conditions of high analytical uncertainty as specified in SNI 8460:2017 [10]. The role of the geotextile is critical: without its tensile contribution, the reduction in soil shear resistance under seismic inertial forces would drive the safety factor below the

acceptable threshold. The composite load-sharing mechanism between the consolidated clay and the geotextile reinforcement represents the key design principle underlying the recommended system.

TABLE II. SUMMARY OF ANALYSIS RESULTS BY MODELING SCENARIO

Performance Indicator	Without Reinforcement	PVD + Sand Layer + Geotextile
Max. Total Displacement	> 475 mm (failure)	204.8 mm (controlled)
Max. Excess Pore Pressure (kN/m^2)	61.18 (Day 55)	34.48 (Day 40)
Analysis Completion	Premature (non-convergence)	Full sequence completed
Safety Factor (SF)	< 1.0 (unstable)	1.55
SNI 8460:2017 Compliance	Non-compliant	Compliant ($SF_{min} = 1.50$)

E. Validation of the Numerical Model

To ensure the reliability of the PLAXIS 2D simulations, the model's predictive capability was validated against published case studies of soil improvement. Previous field monitoring studies demonstrated that the combination of PVD and preloading on similar thick alluvial clay deposits yielded a comparable consolidation behavior and settlement magnitude. Furthermore, the settlement values obtained from the finite element analysis align reasonably well with classical 1D consolidation analytical solutions for highly compressible clay subjected to equivalent embankment loads. This consistency provides confidence in the accuracy of the numerical model in representing the actual field behavior.

F. Design Implications for Retention Ponds in Subsidence-Affected Coastal Areas

The results demonstrate that independent ground improvement (PVD and drainage) or independent structural reinforcement (geotextile alone) is insufficient to meet safety requirements under the combined loading scenarios considered. The synergistic action of accelerated consolidation will reducing excess pore pressure and enhancing effective stress and tensile reinforcement, resisting lateral deformation and seismic inertia which is essential for stable retention pond embankments on very soft coastal clay.

An important design consideration specific to retention ponds is the fluctuating hydrostatic load. Unlike road embankments that experience predominantly static surcharge loads, retention ponds impose cyclic changes in effective stress as water levels vary with storm events and operational cycles. This dynamic loading mode necessitates a conservative approach in consolidation design and reinforcement dimensioning, and supports the use of PVD at a relatively close spacing (1 m) to ensure rapid reconsolidation between loading cycles.

The land subsidence context adds a further layer of complexity: progressive regional settlement will gradually reduce the effective freeboard of the embankment over its design life. This underscores the importance of accurately predicting post-construction settlement is including the secondary compression component in organic-rich coastal clays and incorporating appropriate elevation allowances into the design embankment crest level.

While the numerical simulation successfully demonstrates the effectiveness of the combined PVD, sand blanket, and geotextile system, several practical limitations

and their corresponding mitigations must be considered for field implementation. Firstly, the installation of PVDs using a steel mandrel inherently causes soil disturbance, known as the 'smear effect', which reduces the horizontal permeability of the clay. To mitigate this, the mandrel size should be minimized, and the actual PVD spacing may need slight adjustments based on trial embankment results. Secondly, strict quality control is required to prevent mechanical damage to the geotextile from heavy construction equipment. This risk can be minimized by placing an initial protective sand cushion layer before allowing heavy machinery to operate. From an economic perspective, procuring large volumes of high-permeability clean sand for the sand blanket can significantly increase construction costs. Future implementations could explore the use of Prefabricated Horizontal Drains (PHD) or locally available granular materials as cost-effective alternatives. Ultimately, long-term maintenance and monitoring of residual settlement remain essential to ensure the sustained resilience of the retention pond embankment.

V. CONCLUSION

This study has demonstrated, through plane strain finite element analysis using PLAXIS 2D, that the construction of retention pond embankments over coastal soft clay area requires an integrated soil improvement and reinforcement strategy. The key findings are as follows:

- (1) The unreinforced embankment failed to achieve stability under the design loading combination, with excess pore pressure accumulation and bearing capacity failure preventing the completion of the loading sequence.
- (2) The integrated system comprising PVD to a depth of 18 m at 1 m square spacing, a 1 m thick sand layer, and 200 kN/m basal geotextile reinforcement successfully controlled deformation (total settlement: 34 cm) and reduced excess pore pressure to near-ambient levels within the treated zone.
- (3) A safety factor of 1.55 was achieved under the combined hydrostatic, traffic, and pseudo-static seismic loading, satisfying the minimum SNI 8460:2017 requirement of 1.50 for high uncertainty conditions.

These results confirm that PVD-accelerated consolidation and geotextile reinforcement act synergistically, with each component addressing a distinct failure mode that cannot be adequately mitigated by the other component alone. Future research should incorporate field instrumentation data for model calibration, account for long-term secondary consolidation in organic clay layers, and evaluate the sensitivity of embankment performance to variability in PVD spacing and geotextile stiffness under progressive subsidence loading scenarios.

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