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# Failure Analysis and Proposed Repair and Retrofit Schemes for a Mechanically Stabilized Earth Wall System in Metro Manila, Philippines

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ABSTRACT: A mechanically stabilized earth wall is an earth-retaining structure consisting of compacted soil mass, facing elements, and reinforcement, which are combined to form a stable slope stabilization system. This system is commonly used for dikes, retaining walls, and bridge abutments. It can also be integrated with other slope stabilization techniques, such as sheet piles and grouted tiebacks, depending on the application. However, the engineering design of an MSE system, particularly when incorporating multiple protective measures, requires thorough inspection. Improper integration of these components can result in structural failures, including excessive deflection and differential settlements. This paper presents the geotechnical assessment and detailed engineering design for the repair and retrofit of a damaged slope protection system, consisting of a 6-m high MSE wall and sheet pile revetment, along a prominent expressway in Metro Manila, Philippines. The site experienced multiple structural and slope failures due to the absence of proper drainage design and failure to integrate various slope protection measures. As a result, the system did not account for the correct stress experienced by the overall stabilization system. Slope stability analysis using the limit-equilibrium methods was conducted to evaluate safety against slope failure. Additionally, deformation analysis using finite element method was performed to assess the serviceability of the slope sections. Based on the analyses, the slope was found to be generally unstable and showed high displacements within the reinforced fill, indicating lateral movement of the MSE wall and pullout of geosynthetic strip reinforcement. Immediate measures were recommended to arrest further damage manifestations, such the installation of a temporary drainage barrier and buttresses to counteract wall overturning moments. Long-term repair schemes were also recommended to permanently address site issues, like the reconstruction of the sheet piles and retrofitting the MSE wall with prestressed grouted tiebacks and reinforced shotcrete facing.

KEYWORDS: Mechanically Stabilized Earth Wall, Slope Stabilization System, Limit Equilibrium Method, Finite Element Method.

## 1 INTRODUCTION

Mechanically stabilized earth (MSE) walls are earth-retaining structures consisting of compacted soil mass, facing elements, and reinforcement, which are combined to form a stable slope stabilization system. This system is commonly used for dikes, retaining walls, and bridge abutments. It can also be integrated with other slope stabilization techniques, such as sheet piles and grouted tiebacks, depending on the application.

Successful application of MSE walls—particularly when incorporating multiple protective measures—heavily relies on backfill material quality, precise compaction per lift, appropriate installation of reinforcing elements, and proper internal and external drainage. Quality control checks from on-site inspection and post-construction monitoring ensure that all components are implemented as designed. Otherwise, improper integration of these components can result in performance issues such as excessive deflection and differential settlements arise, thereby leading to geotechnical/structural failures.

To avoid service downtime or even worsening conditions which can lead to complete collapse, forensic investigations are critical to help identify the hierarchy of geotechnical problems causing distress to the MSE wall. As such, forensic investigation of geotechnical failures shall begin with the thorough understanding of Client needs or objectives, as well as the project site conditions. The consulting forensic geotechnical engineer shall gather as much relevant design and construction information as possible, along with the timeline of damage progression to deduce the prominent failure mechanism in the site of concern.

This paper presents the forensic geotechnical assessment (i.e., field reconnaissance and back-analysis) and subsequent detailed engineering design of repair and retrofit schemes for a damaged 6-m high MSE wall along a prominent expressway in Metro Manila, Philippines. Notably, the MSE wall system is adjacent to a creek whose revetment comprises of contiguous sheet piles. The sheet pile revetment was already completed prior to the construction of embankment for the expressway and the MSE wall along its slip road.

## 2 RECONNAISSANCE SURVEY

The severity of the reported damage was evaluated through a coordinated field reconnaissance. The most prominent manifestation was observed on the asphalt concrete pavement, where longitudinal cracks extended along the shoulder and outermost lane (Figure 1). Initial repairs using concrete infill and sealant temporarily mitigated the issue; however, the cracks continued to propagate over time.

Some sheet piles, originally installed for creek or riverbank protection at least three years before the expressway project, have also exhibited significant tilting. The excessive deflection is likely attributable to a combination of factors, including erosion, scouring along the outer bend, and additional lateral loading from the road embankment, which may have effectively reduced the embedment depth of the sheet piles.

Further inspection of the slope protection system revealed that portions of the embankment at the start and end sections of the MSE wall had been eroded, with displaced material sloughing onto the wall fence and creek revetment (Figure 2). Consequently, the geosynthetic strip reinforcements ("geo-straps") of the MSE wall have become exposed in these areas (Figure 3).

Ponding was also observed at the toe of the embankment and slope protection, with the ponded area apparently functioning as the drainage outfall to the creek or river. From a geotechnical standpoint, such ponding increases both the hydrostatic loads and reduces the shear strength of the embankment and the retained soil behind the sheet pile.

Measurements taken during a follow-up site inspection, conducted about a month later, showed that the tension cracks widths increased to 15 to 30 mm and differential settlements of approximately 20 mm. Closer examination of the crack openings on the pavement raised concerns about possible subbase erosion or migration. To investigate, an exploratory excavation pit measuring 0.6 m in width, 3 m in length, and about 1 m in depth was opened to check for signs of material loss. However, no definitive evidence of aggregate subbase migration due to internal water flow was observed.

Figure 1. Pavement cracks with differential settlement.

Figure 2. Tilting of sheet pile walls.

Figure 3. Exposed geosynthetic strips of the MSE wall due to erosion.

These site observations reflect a complex interplay of geotechnical and hydraulic factors. While temporary repair measures have been attempted, the persistence and progression of these issues suggest that subsurface conditions may be a major driver in the observed distress. Understanding these conditions is critical, as they directly influence wall stability, performance of reinforcing elements, and the durability of associated slope protection works.

## 3 SUBSURFACE CONDITIONS

Site characterization for the project draws on a combination of targeted subsurface investigations and reviewed regional datasets. Borehole logs and in-situ tests in a subsurface exploration program define the underlying stratigraphy and material properties. These site-specific findings are supplemented with secondary data from available regional-scale maps that characterize the broader geomorphic, geologic, and hydrologic setting of the area. Such baseline information provides a contextual framework against which the detailed investigation results are interpreted.

Geologic quadrangle maps indicate that the project site is underlain by Quaternary Alluvium (QAl) deposits. This unit, of Holocene age, comprises unconsolidated to loosely compacted sediments such as clay, silt, sand, and gravel deposited by fluvial processes, as well as rock talus. These deposits form part of the Central Luzon Sedimentary Basin.

The geotechnical evaluation utilized drilling data from a borehole located within the vicinity of the failure site, with a termination depth of 30.0 m. Drilling, sampling, and laboratory testing procedures followed applicable American Society of Testing and Materials (ASTM) specifications.

The borehole profile indicates an upper 4.50 m of very loose to medium-dense sand, underlain by stiff silt with appreciable gravel content at depths of 6.0 to 7.5 m. Below these layers, tuffaceous sandstone extends to the borehole termination depth. These findings are consistent with the regional geology, in which overburden soils are typically underlain by tuffaceous sandstones of the Guadalupe Tuff Formation (GTF), the principal bedrock unit in Metro Manila.

The stratigraphic profile and material characteristics established through the subsurface investigation form the basis for evaluating the mechanisms that led to MSE and sheet pile wall distress. With these parameters defined, the back-analysis of the wall failures can proceed using the identified subsurface conditions—combined with design loads and hydraulic factors—contributed to the observed distress and deformation patterns.

## 4 BACK-ANALYSIS OF WALL FAILURE

Slope stability analysis (SSA) is a geotechnical procedure used to evaluate the factor of safety against failure in natural and engineered slopes, including embankments, dams, cut slopes, and retaining systems. In soil–structure systems, the analysis primarily considers sliding-type failures, although other modes of instability—such as flow, fall, topple, or spread—may also be relevant. The main objective is to identify potential failure mechanisms, quantify stability conditions, and implement measures to prevent mass movement that could adversely affect nearby infrastructure.

The assessment of wall performance was undertaken as part of a forensic investigation to establish the most probable causes of the observed distress and deformation. This involved reconstructing the sequence of loading events, drainage conditions, and structural support configurations from the time of construction through the period of damage progression. Site-specific geotechnical and hydrological characteristics were integrated with field reconnaissance observations, subsurface exploration data, and available instrumentation records. Stability and deformation modeling was then conducted under multiple plausible scenarios to replicate observed damage patterns. The comparative evaluation of these scenarios provided insight into the relative contributions of soil properties, reinforcement performance, wall geometry, and external environmental loading to the onset and progression of instability.

### 4.1 *Limit Equilibrium Analysis*

The stability of selected slope sections within the project area was evaluated using the Limit Equilibrium Method (LEM) as implemented in *Slide2*. LEM determines slope stability by computing the factor of safety (FS), defined as the ratio of available shear resistance to the mobilized shear stress along a predefined or computed failure surface. The method considers the balance between driving forces, which promote downslope movement, and resisting forces, which oppose it. Common factors leading to reduced FS and slope instability include seismic loading, adverse geologic conditions, surcharge loads, elevated porewater pressures, and surface erosion.

In LEM, the soil mass is discretized into vertical slices bounded by an assumed or computed slip surface. For each slice, the weight, normal and tangential reactions, and shear forces are resolved. Prior to analysis, the slope geometry and stratigraphy are idealized, and relevant material properties, boundary conditions, loading scenarios, and analysis methods are defined. In this study, the General Limit Equilibrium (GLE) formulation with the Morgenstern–Price method was adopted for both circular and non-circular slip surface searches. This approach assumes a functional relationship between interslice shear and normal forces, with the normal force acting at the centroid of the slice base and varying linearly along its length.

Structures found atop the slopes will induce surcharge loads that may contribute to instability (Duncan et al., 2014). Surcharge loading was incorporated based on permanent and transient loads acting at or near the slope crest. Permanent loads included pavement and structural dead loads derived from material test reports, with the pavement contributing approximately 18 kPa. For traffic loading, AASHTO LRFD (2020) prescribes an equivalent uniform surcharge corresponding to 0.6 m (2 ft) of earth fill for HL-93 highway loading, equivalent to approximately 10 kPa (250 psf).

Shear strength parameters were estimated using the linear elastic–perfectly plastic Mohr–Coulomb (MC) model. Due to the absence of advanced laboratory testing (e.g., consolidated triaxial, direct shear), shear strength properties were derived from SPT N-values using established correlations (Bowles, 1996; Look, 2007). Table 1 presents the adopted parameters for dry and saturated unit weights ($\gamma_{dry}$ and $\gamma_{sat}$), drained cohesion intercept ($c'$), and drained angle of internal friction ($\phi'$).

Limit equilibrium analysis for the MSE wall section (Figure 4) indicates that some critical slip surfaces have FS values below the allowable threshold. The slip surface with the global minimum FS (0.987) is shown in green. Analysis results suggest that ground improvement using soil–cement columns (SCC) and geotextile reinforcement constrained slip surfaces to the zone between the geotextile and the MSE wall levelling pad. Critical slip surfaces extend through the reinforced fill and daylight at the finished grade line beneath the pavement layers.

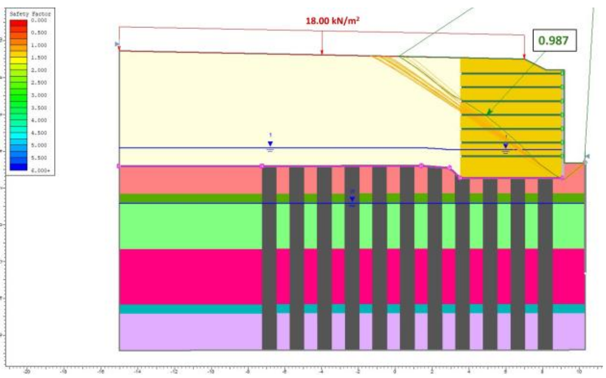


Figure 4. LEA critical sliding surfaces at MSE wall section.

Table 1. Mohr-Coulomb Model Parameters.

| Parameter | $\gamma_{dry}$ [kN/m$^3$] | $\gamma_{sat}$ [kN/m$^3$] | $c'$ [kPa] | $\phi'$ [°] |
|---|---|---|---|---|
| Reinf. Earth Fill | 19 | 20 | 0 | 36 |
| Embankment | 19 | 20 | 0 | 34 |
| M. Dense Sand | 17 | 18 | 0 | 31 |
| V. Loose Sand | 14 | 16 | 0 | 28 |
| Stiff Silt | 16 | 18 | 28 | 21 |
| Dense Gravel | 18 | 19 | 0 | 39 |
| Sandstone | 22.5 | 22.5 | 22 | 26 |

Elevated porewater pressures, associated with a perched groundwater table above the geotextile (Piezometric Line 1), further reduced stability. Even under static loading without the full highway live load, the calculated FS at the MSE wall section indicates that the 5.5 m reinforcement length is likely insufficient.

### 4.2 *Finite Element Analysis*

Deformation analysis was conducted to simulate the stress–strain response of the soil–wall system under various loading and construction stages. This type of analysis calculates stresses, displacements, and secondary response variables simultaneously, providing insight into both stability and serviceability performance. In the present study, it was used to identify probable failure mechanisms and assess the adequacy of existing reinforcement, with results forming the basis for proposed mitigation measures.

One of the primary criteria in the design of earth-retaining systems is control of serviceability-related deformations, including vertical settlement or heave, lateral deflection, and internal strain in reinforcement elements. The analysis was performed using the Finite Element Method (FEM) in *PLAXIS 2D*, which models the soil–structure system as a continuum composed of piecewise-homogeneous, deformable porous media. The governing equations for deformation and porewater flow are derived from conservation of mass, momentum, and energy, and are solved numerically as a nonlinear boundary value problem. FEM allows progressive accumulation of stresses and strains to be tracked through staged construction, enabling evaluation of system behavior up to failure or a specified point in time.

The Hardening Soil (HS) model (Schanz et al., 1999) was used for the soil layers (Table 2), capturing stress-dependent stiffness and strain-hardening/softening behavior. The model incorporates a Mohr–Coulomb (MC) failure envelope, with MC shear strength parameters estimated from SPT N-value correlations (Bowles, 1996; Look, 2007) as listed in Table 1.

Table 2. Hardening Soil Model Parameters.

| Parameter | $E_{50}$ [MPa] | $E_{oed}$ [MPa] | $E_{ur}$ [MPa] | $\nu_{ur}$ [-] |
|---|---|---|---|---|
| Engineered Fill | 50 | 50 | 150 | 0.2 |
| M. Dense Sand | 25 | 25 | 50 | 0.2 |
| V. Loose Sand | 25 | 25 | 75 | 0.2 |
| Stiff Silt | 20 | 20 | 60 | 0.2 |
| Dense Gravel | 50 | 50 | 125 | 0.2 |

In the table above, $E_{50}$ is the secant stiffness in standard drained triaxial test, $E_{oed}$ is the tangent stiffness for primary oedometer loading, $E_{ur}$ is the unloading-reloading stiffness, and $\nu_{ur}$ is Poisson's ratio for unloading-reloading. These stiffness parameters are calibrated to a reference pressure of 100 kPa.

Borehole data indicate that the soil profile is underlain by highly weathered sandstone with zero Rock Quality Designation (RQD). This layer was modeled using the Hoek–Brown (HB) criterion (Brown & Hoek, 1980; Hoek et al., 2002) to account for nonlinear strength variation with confining stress. Geological Strength Index (GSI) values were applied to represent reductions in intact rock strength due to jointing and bedding planes (Table 3).

Table 3. Hoek-Brown Model Parameters.

| Rock Type | $E_{rm}$ [MPa] | $\sigma_{ci}$ [MPa] | $m_i$ [-] | $GSI$ [-] | $D$ [-] |
|---|---|---|---|---|---|
| Sandstone | 36 | 0.65 | 17 | 0.2 | 0 |

In the previous table above, $E_{rm}$ is the rock mass Young's modulus, $\sigma_{ci}$ is the absolute value of the uniaxial compressive strength of the intact rock, $m_i$ is the intact rock parameter, and the $D$ is the disturbance factor. These parameters were calibrated from lab test data as well as typical values suggested in the literature (Hoek, 2006).

Geosynthetic strip elements were assigned axial stiffness values from their product specifications, with adjustments to account for the looped installation configuration that doubles tensile capacity and stiffness. The long-term design tensile strength of the geosynthetic strip is 25.3 kN/m, while the static axial stiffness is 722 kN/m. The typical length of the reinforcement is 5 m with a vertical spacing of 0.75 m and horizontal spacing of 1.5 m.

FEM analyses were carried out for the MSE wall section with the adjacent sheet pile (Figure 5). The geometry was based on available design drawings, with the global water table positioned 2.0 m below original ground level in accordance with the borehole data. The construction sequence was modeled in phases to replicate stress and strain history: (1) initial stress state assignment, (2) installation of sheet pile, (3) ground improvement with SCC and geotextile, (4) construction of reinforced fill lifts with geosynthetic strips, (5) placement of general backfill, (6) application of pavement dead load, and (7) installation of new sheet pile.

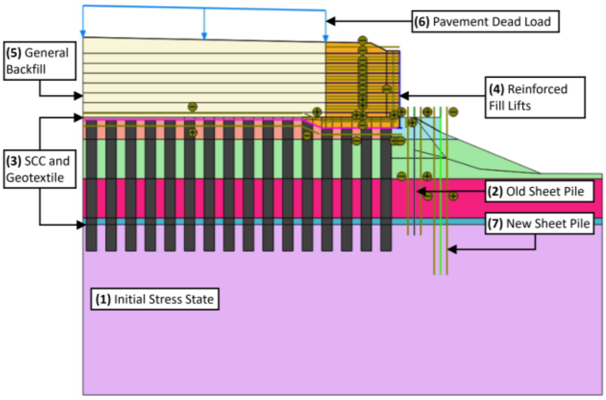


Figure 5. FEA geometry for MSE wall section.

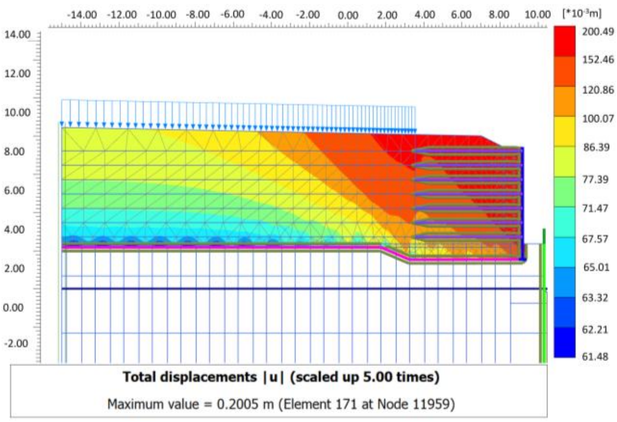


Figure 6. Color plot of total (resultant) displacements in the MSE wall section with the adjacent sheet pile.

A 9.0 m Type-II U-sheet pile was assumed for the existing sheet pile in the absence of reliable as-built information. Interface elements were incorporated to simulate shear and normal stiffness at material contacts. For geosynthetic strip–fill interfaces, manufacturer-provided interaction coefficients were used; for other contacts, typical literature values were adopted.

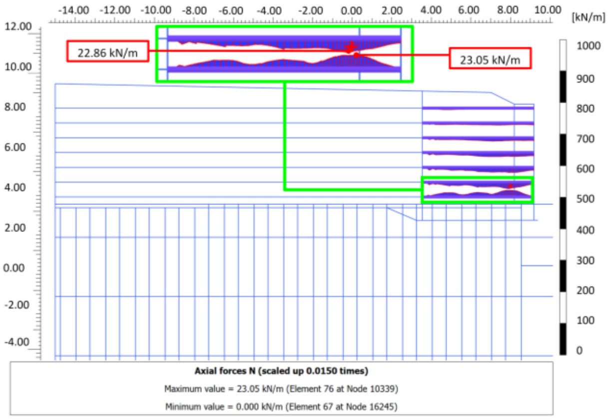


Figure 7. Axial force distribution in geosynthetic strip reinforcement.

FEM results show large lateral displacements (up to 0.2 m) concentrated within the reinforced soil zone (Figure 6), indicating outward movement of the MSE wall. The analysis also captured a potential pullout mechanism at the geosynthetic strip–fill interface. The two lowermost strips developed peak axial forces of 23.05 and 22.86 kN/m (Figure 7), approaching their design tensile strength of 25.3 kN/m.

### 4.3 *Back-Analysis Results and Discussion*

The LEM and FEM were applied to evaluate the likely failure modes contributing to the observed pavement distress, MSE wall deformation, and existing sheet pile deflection. While LEM provides an efficient means of assessing stability limit states through FS calculations, FEM offers a more comprehensive simulation of soil–structure interaction, allowing for simultaneous consideration of stresses, strains, and deformation mechanisms that influence serviceability.

LEM results suggest a critical "compound failure" mode, in which sliding surfaces pass through the reinforced soil zone and daylight at the finished grade line beneath the pavement layers. The presence of SCC columns and basal geotextile restricts sliding beneath the wall toe, redirecting movement upward and inducing an overturning mechanism in the wall facing. Despite the presumed ground improvement, FS values remain below the typical design target of 1.5 for high-consequence structures (Berg et al., 2009), indicating marginal stability.

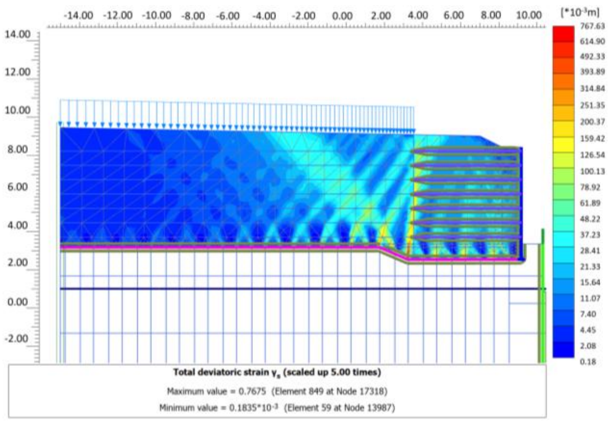


Figure 8. Color plot of total deviatoric (distortional) strain plot in the MSE wall section with the adjacent sheet pile (FS = 1.33).

FEM results provide further detail, capturing a potential pullout mechanism in the two lowermost geosynthetic strips, with peak axial forces of approximately 23 kN/m approaching their design tensile strength. While the recommended geosynthetic strip configuration should result in a double-reinforcing action, field evidence (Figure 3) suggests that only single-reinforcing action may have been achieved, reducing available tensile capacity. Deviatoric strain plots (Figure 8) identify a sliding surface associated with pullout failure, which, when analyzed for FS, yields a value of 1.33—slightly higher than the LEM-derived FS but still below acceptable thresholds.

Differences in predicted sliding surfaces between the two methods are partly attributed to the omission of stiffness properties in LEM for the MSE wall, SCC, and geotextile. The FEM-derived global failure surface extends approximately 11 m behind the wall facing, matching the observed zone of longitudinal tension cracking 9 to 11 m from the wall and encompassing both the road pavement and embankment.

Internal (perched) groundwater significantly reduces FS in both sections. In LEM, this reduction is straightforward to quantify; in FEM, it requires calibration of permeability and unsaturated soil parameters, which was beyond the scope of this study. Nonetheless, similar effects are expected once appropriate hydraulic parameters are applied.

Certain site phenomena could not be fully modeled in either method. Notably, localized scouring beneath the U-ditch drainage outlet at the MSE wall terminations (Figure 3) has caused loss of reinforced fill and exposure of geosynthetic strips, reducing facing support and accelerating distress. Additionally, the concave plan geometry of the MSE wall (Figure 2) may induce arching effects that inhibit outward deformation, suggesting that the 2D analyses may be conservative when applied to back-analysis and retrofit design.

## 5 REPAIR AND RETROFIT SCHEMES

Repair and retrofit strategies were formulated based on the results of the stability and deformation analyses to address the observed failures affecting the pavement, MSE wall, and existing sheet pile wall. The proposed interventions are classified into three categories: (1) Temporary or immediate measures to prevent further deterioration prior to major work, (2) Permanent or long-term solutions to restore and enhance structural stability and serviceability, and (3) Complete reconstruction of the MSE wall.

### 5.1 *Temporary Measures*

Site inspections have confirmed progressive propagation of longitudinal pavement cracks, with widths of 15–30 mm and differential settlements of approximately 20 mm. To prevent further deterioration, immediate measures are recommended to limit water ingress and restrain ongoing deformation until permanent works can be implemented.

The first measure involves covering the pavement cracks with polyethylene (PE) sheets—commonly referred to as tarpaulin—to act as temporary drainage barriers (Figure 9). These will prevent surface water infiltration into the cracks, which could otherwise accelerate subgrade weakening, reduce shear strength, and increase hydrostatic loading in the embankment due to inadequate internal drainage. The sheets shall be securely fastened using traffic barriers or equivalent means.

Monitoring records indicate that the MSE wall is out-of-plumb and undergoing outward displacement. To provide interim structural restraint, a temporary buttress using gabion baskets or sandbags is proposed at the wall toe (Figure 9). This countermeasure will partially offset the overturning moment acting on the wall and stabilize its facing until the installation of long-term reinforcement systems.

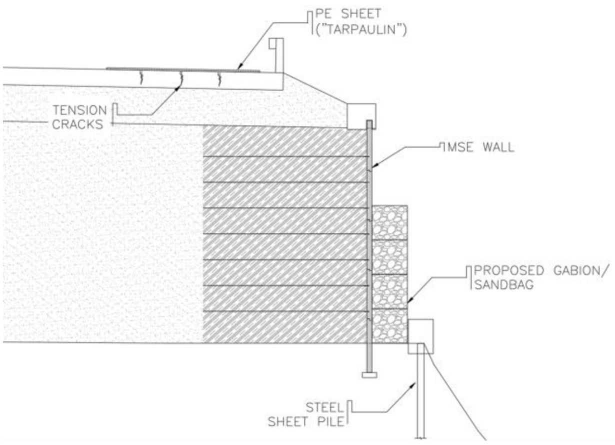


Figure 9. Polyethylene sheet cover and temporary buttress.

### 5.2 *Permanent/Long-Term Solutions*

Measured deflections of 2 to 3 m in the existing sheet pile wall indicate yielding and irreversible deformation of the steel section. A new sheet pile wall is therefore recommended, to be constructed with a minimum offset of 3 m from the MSE wall facing to allow working space for subsequent MSE wall retrofit works. Deadman anchors will be installed near the top of the new sheet pile wall to provide additional lateral restraint.

Reconnaissance surveys identified localized washout at both ends of the MSE wall. To ensure smooth transition from the wall termination to the natural ground and to eliminate scour at U-ditch outlets, the MSE wall should be extended at both ends. Terminations will be daylighted to the existing ground profile and provided with erosion protection. The external drainage (U-ditch) will be connected to catch basins or piped outlets. To accommodate the MSE wall extension, the existing slope will be regraded and stabilized using soil nails with shotcrete facing (Figure 10).

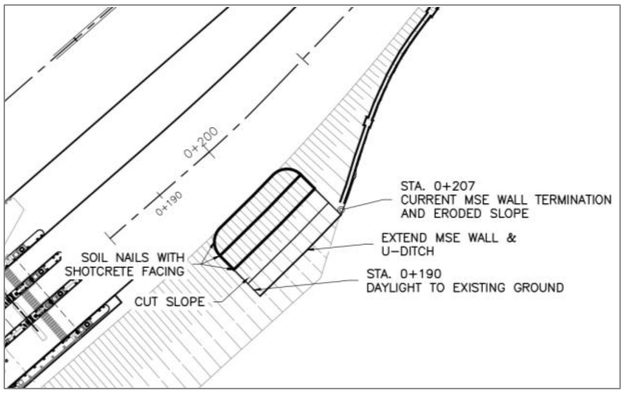


Figure 10. MSE wall repair layout.

The current geosynthetic strip-reinforced MSE wall exhibits insufficient margin of safety, requiring reinforcement to improve stability and serviceability. The retrofit scheme involves drilling for steel soldier piles (preferably double-channel sections) rather than driving them, to avoid inducing further wall displacement. The drilled shafts will be concreted below existing grade level.

Grouted, prestressed tieback anchors will be installed to prevent additional wall movement (Figure 11). Reinforced shotcrete will be applied to the wall facing to increase bearing thickness during prestressing. Anchor and pile positions will be marked using reinforced shotcrete pilasters to facilitate future inspections and maintenance (Galvan, 2010).

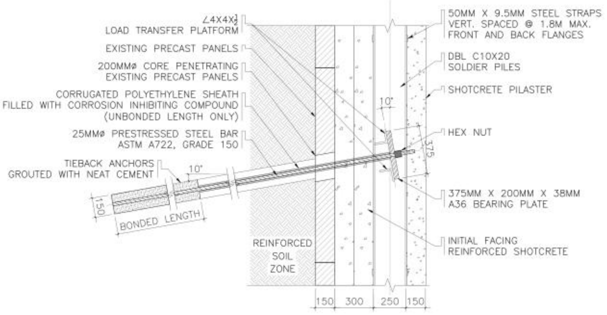


Figure 11. MSE wall retrofit using prestressed anchor, shotcrete facing, and soldier pile construction: section view.

Post-construction drainage installation is inherently challenging. However, drilled weepholes, consisting of PVC pipes wrapped in filter fabric with annulus filled by permeable grout, can provide partial relief of internal porewater pressure. Routine maintenance will be necessary to prevent clogging and monitor for soil migration.

Following completion of wall replacement and retrofitting, road pavement will be reconstructed. Geosynthetics are recommended within the base or subbase layers to improve reinforcement and lateral stability while permitting water flow. Alternatively, geo-composites may be used to combine reinforcement with internal drainage, discharging water through collector pipes connected to existing drainage networks.

It is worth noting that the existing MSE wall design lacks provisions for extreme flood events. Although the proposed shotcrete facing will act as a water intrusion barrier, floodwater may enter via the drilled weepholes. A hydrologic and hydraulic study is therefore recommended to determine the frequency and risk of flooding or river overtopping. The study will also define the optimal elevation of weepholes to minimize floodwater intrusion.

### 5.3 *Reconstruction Option*

As an alternative to retrofitting, complete reconstruction of the MSE wall may be considered to address both stability and serviceability deficiencies. This option involves dismantling the existing wall and replacing the reinforced fill with expanded polystyrene (EPS) geofoam blocks as lightweight backfill material.

Geofoam is a low-density, high-strength material significantly lighter than conventional soil fills. Substituting geofoam for soil backfill greatly reduces the driving forces acting on the MSE wall, thereby increasing the factor of safety against sliding and overturning, and decreasing foundation pressures. This also mitigates settlement and reduces the potential for deformation-related serviceability issues.

In this configuration, the geofoam blocks would be placed in layers behind the wall facing, with a geotextile separation layer to prevent contact between the geofoam and pavement materials. The MSE wall facing system could be designed with minimal or no geosynthetic strip reinforcement, relying instead on the reduced loads from the lightweight fill. Where necessary, discrete facing connections can be provided for alignment stability. The top surface of the geofoam would be covered with a protective soil layer to accommodate pavement and drainage features.

Drainage considerations remain critical, as EPS materials are impermeable; surface runoff and subsurface water must be redirected using drainage blankets or collector pipes to prevent hydrostatic buildup behind the facing. Additional protective measures, such as geomembrane wrapping or concrete panels, may be required to shield the geofoam from hydrocarbon exposure, UV degradation, and mechanical damage during service life.

Although initial costs for geofoam are higher than conventional fill materials, potential long-term benefits include reduced maintenance and faster construction time due to lightweight placement. For the subject site, this approach offers the dual advantage of eliminating the need for meticulous retrofit while achieving a higher stability margin under various loading conditions.

## 6 CONCLUSIONS

This paper presents the geotechnical assessment and detailed engineering design for the repair and retrofit of a damaged slope protection system, consisting of a 6-m high MSE wall and sheet pile revetment, along a prominent expressway in Metro Manila, Philippines. The site involves multiple structural and slope failures due to the absence of proper drainage design and failure to integrate various slope protection measures.

Geotechnical assessment of the MSE wall with sheet pile revetment revealed that inadequate drainage and poor integration of slope protection measures led to instability, excessive wall displacement, and reinforcement pullout. Stability and deformation analyses confirmed insufficient safety margins and significant serviceability concerns. Immediate mitigation measures, such as temporary drainage barriers and toe buttresses, were proposed to arrest further damage, while long-term solutions include sheet pile reconstruction, MSE wall retrofitting with prestressed tiebacks and reinforced shotcrete, internal drainage installation, and pavement rehabilitation with geosynthetic reinforcement. The findings emphasize the necessity of integrated design, proper drainage, and consideration of soil–structure interaction to ensure the long-term stability of MSE wall systems in critical infrastructure.

## 7 ACKNOWLEDGEMENTS


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