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A Hybrid Geophysical–Geotechnical Approach for Landslide Stability Assessment

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Abstract

Landslides are widespread natural hazards that pose significant threats to human life, infrastructure, agricultural land, and economic activities. In Ethiopia, landslide occurrence is particularly prevalent in tropical and mountainous regions, with the risk increasing substantially during the rainy season from June to September. This study investigates the stability and failure mechanism of a landslide affecting a road section in Bonga Town, southwestern Ethiopia, and proposes appropriate mitigation measures. Numerical modelling was performed using PLAXIS 2D and GEO5 to evaluate slope stability and simulate potential stabilization measures. Field investigations, including seismic refraction and electrical resistivity tomography (ERT), were conducted to characterize the subsurface geological and geotechnical conditions. Laboratory investigations comprised index properties, shear strength, and consolidation tests to determine the engineering properties of the soil. The numerical analysis indicated a factor of safety (FS) of 1.347, which is lower than the minimum acceptable value of 1.5 recommended for stable slopes in relevant Ethiopian Road Administration (ERA) and internationally recognized engineering design standards. The results confirm that the investigated slope is potentially unstable, particularly under adverse environmental and loading conditions. Based on the numerical simulations, field observations, and assessment of the existing failure mechanisms, appropriate slope stabilization and drainage measures were proposed to improve slope stability and minimize the risk of future landslides. The findings provide useful guidance for the rehabilitation of the affected road section and contribute to improved landslide risk management and sustainable road infrastructure development in landslide-prone regions of Ethiopia.

Keywords: Factor of safety; Landslide; Numerical modelling; PLAXIS 2D/GEO5 software.

1 Introduction

Landslides are a worldwide issue that can occur due to natural conditions and are often triggered by human activities. This research identifies the current road situation using a numerical approach, and it helps minimize the transport blockage. It can be represented in various sizes and time scales, and with various possible movements [1]. Irrespective of the design approach employed, a complete understanding of the problem and what it entails requires accurate knowledge of its likely causes and an awareness of the different instability mechanisms that may occur [2]. At the same time, slope stability assessment requires understanding the relevant geological and geotechnical properties of soils and rocks that govern resisting (shearing) forces. Moreover, slope stability analysis is one of the most critical subjects in geotechnical engineering [3]. Slope failures depend on slope geometry, soil type, soil strata, groundwater, and seepage. Calculating the safety factor is the general procedure in the slope stability analysis. Furthermore, the instability phenomenon may present different particularities for distinct ground conditions and is significant in large-scale geotechnical works, such as highways, railways, canals, open-pit mines, tunnels, and even embankment dams. The slope of mines and quarries resulting from the extraction activities constitutes the geotechnical work of mining operations [4]. The miners must be sure that the slopes are stable or that the unsafe factors are manageable for the long-lasting nature of the mines or quarry. The different problems related to the creation of a quarry, the removal of a large quantity of overburden and waste, and the construction of a dump require many strategies to be put in place to minimize the displacement of mining infrastructures [5]. One of the most important strategies is minimizing the slope angle. The road's location and the cut's geometry also affect the slope failure probability. During the production of mines, the steeper the hill, the lower the cost of production, with the risk of abandoning more ore at the mining site [6].

The topography and landscape of Ethiopian infrastructure cover the mountainous terrain. The road's destruction is usually closed during the summer season when rainfall is high. Detailed slope analysis and mitigation measures are needed to prevent damage to the economy and the loss of human lives during the slope's failure [7]. The numerical simulation of the slope stability analysis was used to calculate the safety factor and propose mitigation measures [8]. Scholars proposed slope stability analysis methods that considered both finite element and finite difference methods [9]. However, depending on the complexity of the geological conditions and the groundwater table location, the analysis needs detailed field and laboratory investigation of the soil conditions and the distribution of the groundwater table locations [10].

Ethiopia is a mountainous region and has been subjected to many landslides. Economic, environmental and social losses due to the landslides in the mountainous regions of Ethiopia continue to grow. The significant factors contributing to the development of many landslides are the increasing development of landslide-prone areas, earthquakes, heavy rainfall, and other causative factors such as uncontrolled excavations. Slope stability problems are increased due to incorrect developmental planning and land use in the landslide susceptible zones [11]. Therefore, the movement of earth, debris, and rock mass is one of the most endangering geotechnical and Natural environmental risks in mountainous terrains. The stability analysis of slopes is determined using finite element method (FEM) and finite difference method (FDM) methods like PLAXIS 2D or 3D [12, 13], FLAC 2D or 3D, GeoStudio 2D or 3D, and Zsoil [3]. The FE method divides the slope domain into small, discrete elements (e.g. triangles or quadrilaterals in 2D, tetrahedra in 3D) and solves equations governing stress-strain behaviour and failure criteria.

This area faces different landslide problems [14, 15], which damage property and people's lives during the movement from Addis Ababa to Bonga to Mizan on the main road. Some researchers have studied this area, but the combined effects of laboratory and field tests have not been given more attention due to the budget limitations for conducting detailed geophysical and laboratory tests along Bonga and Mizan towns. The methodologies used in this research are laboratory tests, field tests, finite element methods and limit equilibrium methods. The field and laboratory test result parameters are unit weight of the soil, moisture content, internal friction angle, and soil input parameters' cohesion used in the finite element analysis software (PLAXIS 2D).

The main objective of this study is to determine the optimum mitigation measures for the landslide problem in Bonga Town by evaluating alternative stabilization techniques, including slope geometry modification, drainage systems, retaining structures, and internal slope reinforcement. The study considers the three-dimensional effects of slope instability and incorporates seepage analysis, slope stability analysis, deformation analysis, and soil–structure interaction to comprehensively assess the performance of the proposed mitigation measures. The findings provide a technically appropriate and site-specific basis for selecting effective landslide remediation measures considering the local soil properties, geological conditions, groundwater regime, and slope geometry. Furthermore, the study contributes to reducing the time and cost associated with the investigation and evaluation of alternative remedial measures, thereby supporting the safe and sustainable rehabilitation of landslide-affected areas and road infrastructure in Bonga Town.

This study contributes to a better understanding of landslide problems in Bonga and Mizan towns by integrating field investigations, laboratory testing, and numerical analyses. The paper is organized into four main sections. The first section, Materials and Methods, describes the materials, field investigation techniques, laboratory testing procedures, and numerical modelling approaches employed in the study. The second section presents the Results obtained from the field and laboratory investigations and numerical analyses. The third section provides a Discussion of the findings, including the factors contributing to slope instability and the effectiveness of the proposed mitigation measures. The fourth section presents the Conclusions, summarizing the principal findings and providing recommendations for appropriate and sustainable landslide mitigation measures considering the site-specific soil properties, geological conditions, and topographical characteristics of the study areas.

2 Study area

The study areas are located in southwestern Ethiopia along the Addis Ababa–Jimma–Bonga–Mizan–Tepi road corridor, approximately 555 km southwest of Addis Ababa. The road corridor connects several towns, including Waja Maji, Temenja Yazhi, Yeli, and Mizan Aman, and extends predominantly in a southwest direction from Wacha Town. Geographically, the study area is situated along the southwestern margin of the Main Ethiopian Rift and falls within the Baro River Basin.

The study area is bounded approximately by Easting coordinates ranging from 784,000 to 800,000 m and Northing coordinates ranging from 772,000 to 784,000 m. The geographical location and extent of the study area are presented in Figure 1.

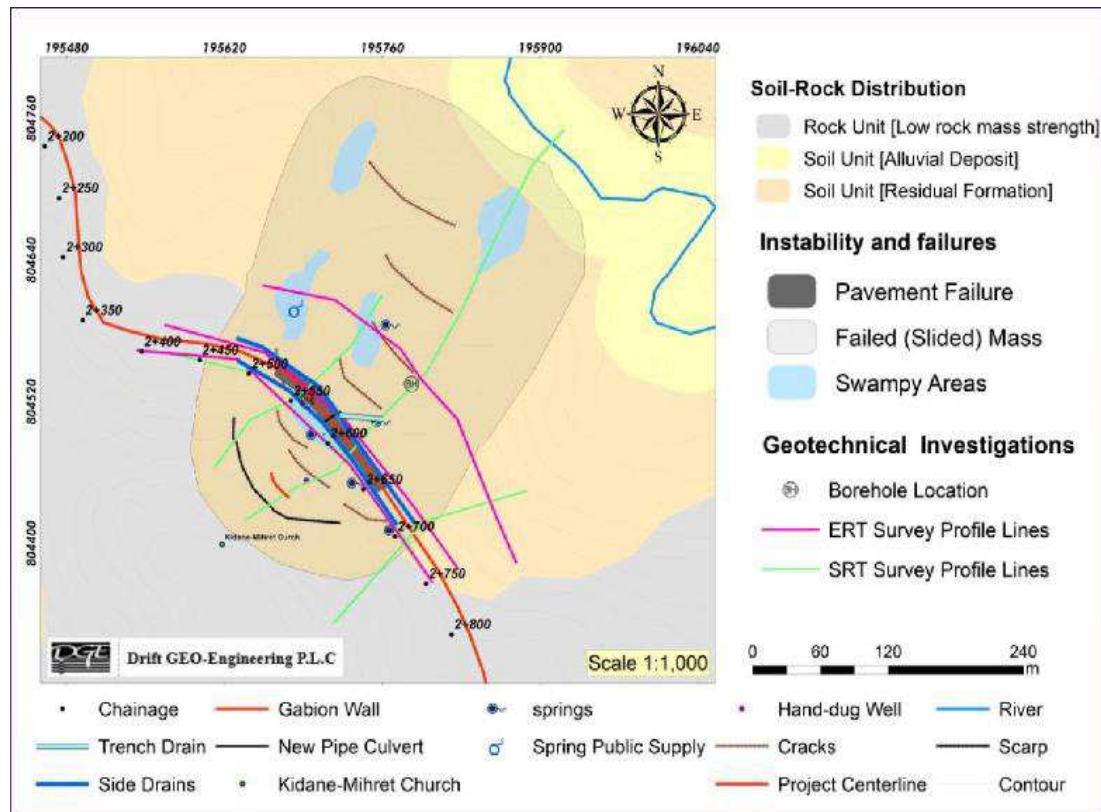


Fig.1 Engineering geological mapping along km 2+520 - 2+660, Bonga-1

In this stretch, the failed mass is the entire hillside that moved onto the road. However, the movement is more pronounced on the main asphalt road and thus needs due attention for remediation compared to above and below the road [16]. Cracks and springs have also been observed in different locations. Some ponds and shallow groundwater are also noticed above the road section due to the poor design of subsurface drainage, which is, in fact, the most prominent triggering factor in this area[17]. An effort was made to maintain the failed road section using rock fill, which was successful. However, there is still an active movement. It is noted that the rock fill is found at a shallow depth of the moving mass[18]. It is believed that either anti-slide piles on the left-hand side (LHS) or well-designed soil reinforcement and stabilization on the whole road would be effective. The slope instability is shown in Fig.2 below.



Fig.2 Photos showing instability section along Km2+520-2+660, Bonag-1

3 Materials and methods

The investigation of slope stability was carried out in different stages: preliminary, detailed investigation, and analysis phases. The initial stages involved a desk study to collect information about the site and the associated problems, like landslide failures, by detailed physical investigation through the roads using pickup trucks[9]. Site topography, groundwater, geological, and geotechnical properties were determined in the inquiry. By combining geotechnical properties, hydrological data and geophysical methods, this study uses finite element methods to reconstruct failure mechanisms and their causative factors and provide mitigation measures for the selected road sections[19].

3.1 Geophysical investigation

This study employed Seismic Refraction Tomography (SRT) and Electrical Resistivity Tomography (ERT) surveys to investigate the geophysical conditions of the Bonga to Mizan road landslide area. This method was chosen due to its cost-effectiveness, rapid data collection, and ability to reveal shallow bedrock and overlying materials. Based on the P wave velocity, different layers of geological conditions in soil or rock were investigated. The geophysical investigation was displayed in a two-dimensional way [20].

Both methods were placed around seven selected sites. B1-SRS-Cr-1, B1-SRS-Cr-1, B1-SRS-Cr-1, and B1-SRS-Cr-1 in the case of Seismic Refraction Tomography (SRT), and B1-ERT-A1-R2, B1-ERT-A1-R1, and B1-ERT-A1-L2 were in the case of Electric Resistivity Tomography (ERT).

3.2 Geotechnical investigation

The geotechnical investigation was conducted by ASTM to comprehensively characterize the subsurface conditions and investigate the relationship between the lithology of the landslide-prone areas and the electrical resistivity profiles. Four boreholes were drilled at selected locations considered representative of the overall study area. Two boreholes were located along the Bonga 1 section between stations 2+520 and 2+660 km, while the other two were located along the Bonga 2 section between stations 4+300 and 4+420 km. Undisturbed soil samples were collected from the boreholes for subsequent laboratory testing.

For Bonga 1, borehole B1BH01 was located at UTM coordinates of 195599.5 m E, 804260.3 m N, with an elevation of 1701.6 m, and was drilled to a depth of 25 m. Borehole B1BH02 was located at 195694.0 m E, 804276.1 m N, with an elevation of 1693.1 m, and was drilled to a depth of 15 m. For Bonga 2, borehole B2BH03 was located at 194786.0 m E, 803588.0 m N, with an elevation of 1639.0 m, and was drilled to a depth of 20 m. Borehole B2BH04 was located at 194818.0 m E, 803571.0 m N, with an elevation of 1639.0 m, and was drilled to a depth of 15 m.

The investigation comprised two main stages: laboratory testing and field investigations. The laboratory tests were performed on the collected soil samples to characterize their physical and engineering properties, while the field investigations were undertaken to assess the in-situ subsurface conditions and electrical resistivity characteristics of the landslide-prone areas.

3.2.1 In situ tests

In accordance with ASTM standards, Standard Penetration Tests (SPT) were conducted at depths of 15, 20, and 25 m to assess the in-situ soil resistance and subsurface conditions. Groundwater levels were measured 24 h after completion of each borehole to allow sufficient time for groundwater stabilization and minimize the influence of drilling-induced disturbance. In addition, field observations of seepage zones, nearby springs, and surface-water accumulation patterns were systematically recorded to identify potential groundwater flow paths and assess seasonal variations in groundwater conditions.

Field observations of groundwater seepage were complemented by historical rainfall records obtained from nearby meteorological stations to estimate groundwater conditions during the monsoon season. Borehole logs were also examined for evidence of perched water tables, saturated zones, and variations in moisture conditions, providing additional information on seasonal groundwater fluctuations. These observations and rainfall data were incorporated into the PLAXIS 2D numerical analysis to simulate groundwater conditions representative of peak monsoon periods and to evaluate their potential influence on slope stability and landslide occurrence.

Both disturbed and undisturbed soil samples were collected from the boreholes at representative depths and transported to the laboratory for further geotechnical characterization. The laboratory testing program was conducted in accordance with the relevant ASTM standards to determine the physical and engineering properties of the soils.

3.2.2 Laboratory tests

To characterize the soil properties and determine the composition of the materials contributing to landslide occurrence, a comprehensive laboratory testing program was conducted. The tests included specific gravity and unit weight (ASTM D2937), direct shear strength (ASTM D3080), natural moisture content (AASHTO T265), particle-size distribution (AASHTO T88), Atterberg limits (AASHTO T89/T90), and water chemical analysis (ASTM D516). Soil classification was performed in accordance with the AASHTO M145 and ASTM D2487 classification systems, while unconfined compressive strength (UCS) tests were conducted in accordance with the relevant ASTM standard. The number and type of laboratory tests varied depending on the lithological formations and soil types encountered at different depths within the boreholes, ensuring that the testing program adequately represented the subsurface materials and their engineering characteristics [21, 22].

3.3 Slope stability analysis

The natural terrain profile after the failure was considered for the analysis by defining the subsoil layer based on the findings from geophysical and geotechnical investigations[23]. The water table condition during the monsoon period was estimated based on the present water table recorded from field observations, springs and seepage levels, and the saturation layer observed from geophysical tests[24].

The slope stability analysis was done using PLAXIS 2D and GEO 5 software. The most critical section was selected after field work, such as finalizing the level of the existing landslide crown, road level, and the toe area of the landslide, and desk work, such as topographical surveys, geophysical testing, and geotechnical investigations [25]. This approach ensured the accuracy of the derived geometrical setup for modelling to represent the real-world condition of the site [26]. Four strategically placed boreholes adequately covered the slope lithology, as confirmed by the consistency of subsurface conditions stemming from the geophysical study [27].

PLAXIS 2D was employed for a comprehensive slope analysis, and GEO 5 software was used to apply the analysis to the mitigation measure [28]. The software's behaviour is also easy to understand. The input parameters for both methods for the simplified model of the critical section were deduced from the soil properties obtained from geotechnical tests and confirmed by the field tests. Based on the detailed description and field and laboratory tests, the flowchart of the work is given in Fig.3.

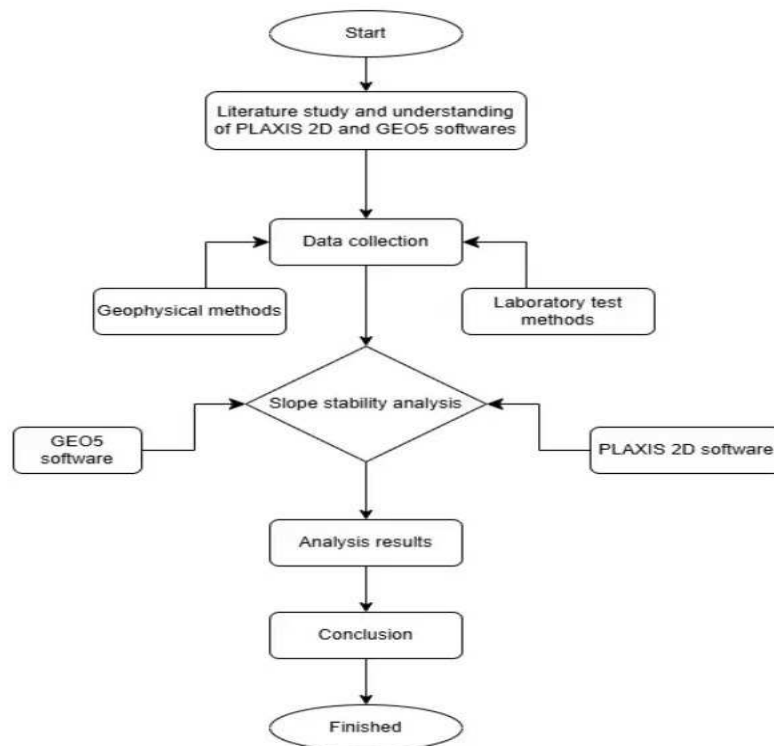


Fig.3 Flow chart diagram

4 Results

4.1 Geophysical investigation

The SRT and ERT surveys revealed significant spatial variations in subsurface materials, ranging from colluvial soils and weathered formations to saturated deposits and fractured bedrock. The interpreted resistivity profiles identified distinct low-resistivity zones, which were generally associated with elevated moisture content, saturated or clay-rich materials, and zones of increased weathering and fracturing. These zones may represent potential planes of weakness and preferential groundwater pathways, thereby reducing the shear strength of the subsurface materials and contributing to slope instability. The integration of the SRT and ERT results with borehole and geotechnical data provided a more comprehensive interpretation of the subsurface stratigraphy and the hydrogeological conditions controlling landslide susceptibility..

4.1.1 Seismic Refraction Tomography (SRT)

The maximum P-wave velocity recorded along this survey section was approximately 5.0 km/s. Based on the measured P-wave velocities and their spatial distribution, three distinct geophysical layers were identified, as described below.

The first layer is characterized by P-wave velocities of less than 1.2 km/s and is interpreted as residual soil comprising predominantly silty clay and sandy clay. The thickness of this low-velocity layer varies considerably along the road section. Its maximum thickness, approximately 30 m, was identified at around 250 m along B1-SRS-Cr-2, between 200 and 240 m along B1-SRS-Cr-1, and near the central portion of B1-SRS-Cr-3. In contrast, the residual-soil layer is considerably thinner, with a thickness of approximately 3–4 m, in several locations. The first 120 m of B1-SRS-Cr-1, both ends of B1-SRS-Cr-2, and the section between 160 and 200 m of B1-SRS-AI-R4 exhibit relatively thin surficial soil deposits. The considerable variation in thickness indicates pronounced spatial heterogeneity in the residual soil and weathering profile.

The second layer is characterized by P-wave velocities ranging from 1.2 to 2.4 km/s and is interpreted as highly weathered, fragmented, and fractured rock. This interpretation is supported by the lithological information presented in Table 1 and the borehole data. In particular, borehole B1-BH-1, located approximately 120 m along B1-SRS-AI-R4, encountered this layer at a depth of about 20 m, showing good agreement with the seismic refraction results. In general, the upper boundary of this layer becomes progressively shallower toward the right-hand side of the survey section and deeper beneath the central portion of the road. The second cross-line also identified a relatively high-velocity layer at a shallow depth of approximately 3–5 m toward the far left-hand side, close to the riverbank, suggesting localized variations in the degree of weathering and bedrock depth.

The third and basal layer is characterized by P-wave velocities greater than 2.4 km/s and is interpreted as moderately weathered and fractured bedrock. This high-velocity layer occurs at variable depths along the survey lines. A prominent high-velocity anomaly was identified at a depth of approximately 10 m between 160 and 230 m along B1-SRS-AI-R4, as well as toward the end of the second part of the profile near the river. Given its relatively high P-wave velocity (>2.4 km/s), this layer is interpreted as competent to moderately weathered bedrock. Overall, the bedrock surface occurs at an average depth of approximately 27–30 m along the road section, although localized zones exhibit substantially shallower bedrock, reflecting the heterogeneous nature of the subsurface geological conditions.

The seismic refraction results therefore indicate considerable lateral and vertical variations in the thickness of the residual soil, degree of rock weathering, and depth to bedrock. These variations are important for assessing groundwater pathways, potential weak zones, and the geotechnical conditions influencing slope stability along the investigated road section.

Table 1 Summary of Seismic Refraction Tomographic Section of Bonga-1 along and across the road

SRT Profiles along and across Km2+520-2+660 [Bonga-1]		Lyr	P-wave Velocity (m/s)	Average Depth (mbgl)	Inferred formation from the SRT
		1	<1200	4 ~ 30	Soft Clay silt (Residual soil unit)

B1-SRS -Cr-1		2	1200 – 2400	7 ~ 20	Silty clay with gravel and highly weathered rock
		3	>2400	> 30	Moderately weathered and fractured Rock unit
B1-SRS -Cr-2		1	<1200	3 ~ 25	Soft Clay silt (Residual soil unit)
		2	1200 – 2400	7 ~ 28	Silty clay with gravel and highly weathered rock
		3	>2400	> 8	Moderately weathered and fractured Rock unit
B1-SRS-Cr-3		1	<1200	13 - 25	Soft Clay silt (Residual soil unit)
		2	1200 – 2400	4 - 12	Stiff Clay silt (Residual soil unit)
		3	>2400	—	Weathered Rock unit
B1-SRS -AI -R-4		1	<1200	4 - 23	Soft Clay silt (Residual soil unit)
		2	1200 – 2400	10 - 26	Stiff Clay silt (Residual soil unit)
		3	>2400	—	Weathered Rock unit

4.1.2 Electric Resistivity Tomography (ERT)

The electrical resistivity tomography (ERT) surveys identified a wide range of subsurface resistivity values, approximately 5–180 $\Omega \cdot m$, indicating substantial variations in lithology, moisture content, and degree of weathering. All three ERT profiles exhibited a broadly similar resistivity distribution pattern, consisting of three principal zones. The upper zone is characterized by relatively high resistivity values and generally extends to a depth of approximately 5–10 m, suggesting relatively dry and less weathered soil or surficial materials. This is underlain by a low-resistivity zone with an average thickness of approximately 20–25 m. The low-resistivity zone is interpreted as predominantly saturated residual soil and/or highly weathered and fractured rock, where increased moisture content and possible groundwater accumulation result in reduced electrical resistivity. Beneath this zone, a second relatively high-resistivity layer occurs at an average depth of approximately 30–40 m and is interpreted as relatively competent, less weathered bedrock.

However, localized variations in the thickness and depth of the low-resistivity zone were observed along the profiles. In particular, the low-resistivity zone extends to greater depths along portions of the first 160 m of B1-ERT-A1-R1 and between approximately 120 and 160 m of B1-ERT-A1-L2. These deeper low-resistivity anomalies may indicate localized groundwater accumulation, preferential seepage pathways, or zones of intensely weathered and fractured materials. Such features are particularly important for landslide assessment because increased saturation can elevate pore-water pressure and reduce the effective shear strength of the soil and weathered rock mass.

The ERT results presented in Figures 4–6 therefore provide important evidence of the spatial variability of subsurface materials and groundwater conditions. When integrated with the seismic refraction tomography (SRT), borehole logs, and laboratory test results, the ERT data provide a more reliable interpretation of the subsurface geological and hydrogeological conditions controlling slope instability along the investigated road section.

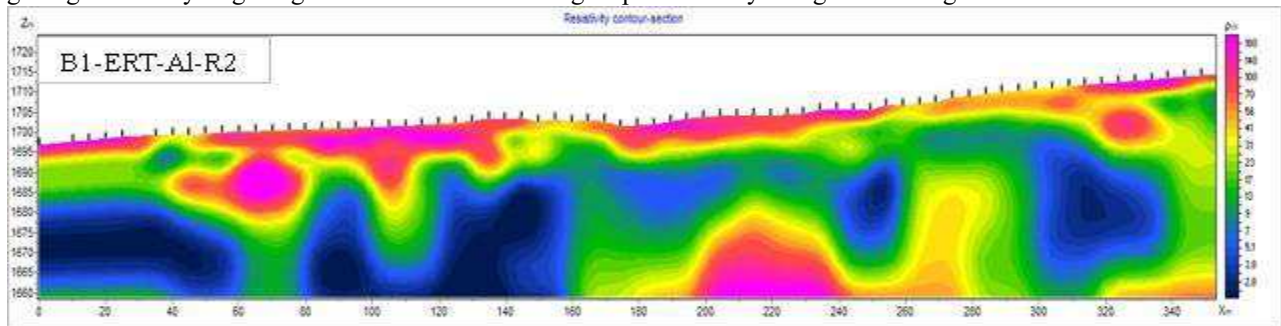


Fig.4 ERT Profile Line along RHS Road Section of Station 2+520- 2+660 [Bonga-1]

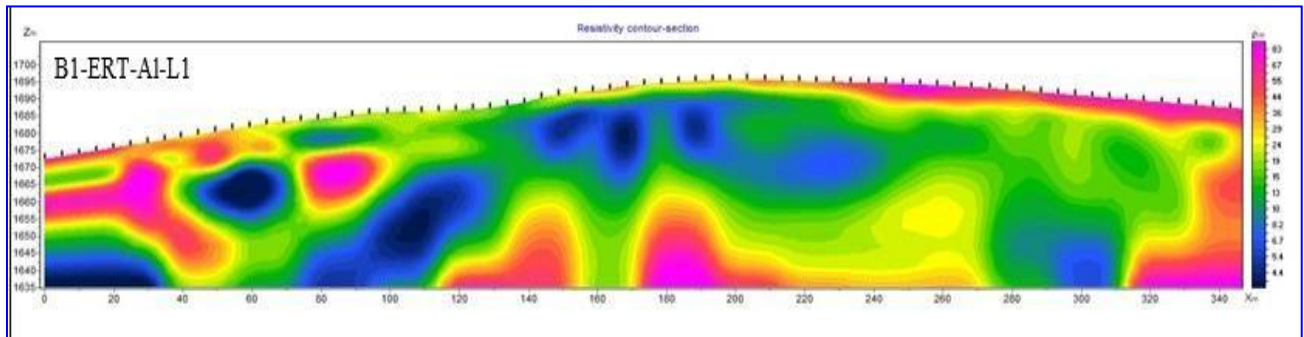


Fig.5 ERT Profile Line along LHS Road Section of Station 2+520- 2+660 [Bonga-1]

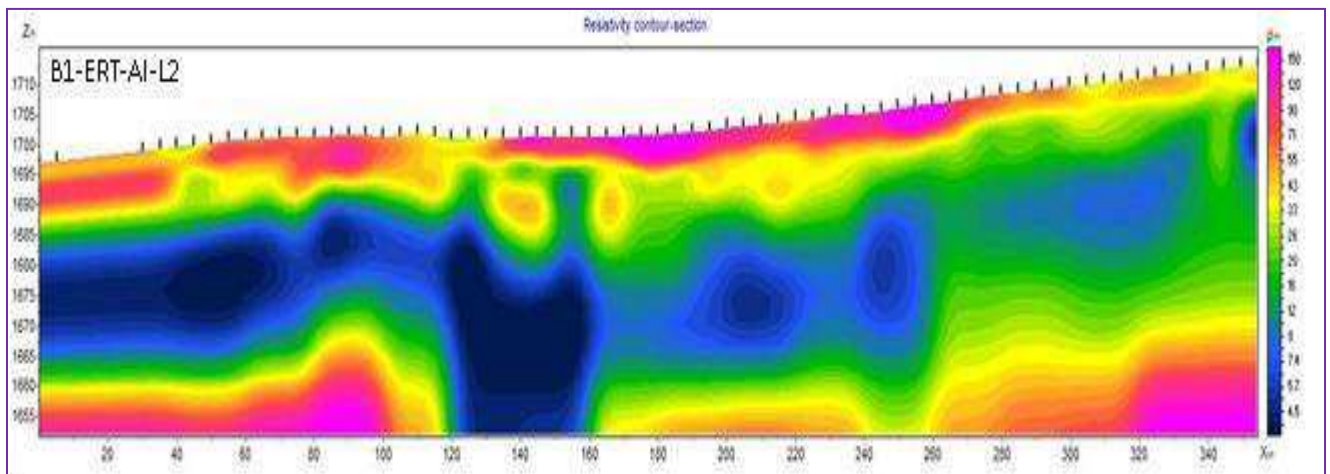


Fig.6 ERT Profile Line along LHS Road Section of Station 2+520- 2+660 [Bonga-1]

Detailed engineering geological maps were prepared at scales ranging from 1:400 to 1:1,000 for the two road sections affected by slope failures. These maps provided a detailed representation of the surface geological and geomorphological conditions associated with the observed failures and facilitated the identification of potential causative and triggering factors. The integrated geological assessment was also used to support the selection of appropriate and site-specific slope stabilization and remediation measures.

The investigation comprised four Seismic Refraction Tomography (SRT) profiles, three Electrical Resistivity Tomography (ERT) profiles, and two geotechnical boreholes within the investigated sections. The subsurface geological conditions and the probable slip-surface geometry were interpreted by integrating the SRT and ERT results with borehole logs and geotechnical observations. The estimated slip-surface depths and corresponding subsurface characteristics are summarized in Table 2.

Table 2 Summaries of geophysical investigations for slip surface identification

Km2+520-2+660 [Bonga-1]	Geophysical Survey results		Drilling results	Interpreted soil mass depth (meters below ground level) sensitivity to failure in both investigations
	SRT	ERT	SPT N-values	
	Probable thickness of soft soil (m)	Thickness of soft and saturated soil (m)	Thickness of soft soil (m)	
B1BH-1	4	3	10	5.0
B1BH-2	5	4	5	

4.2 Geotechnical investigation

4.2.1 In situ tests

The SPT N-values provided additional information on the subsurface conditions and the relative density and consistency of the encountered materials. In both boreholes, the penetration resistance increased markedly with depth, reaching an N-value of approximately 50 at depths of about 4.5 m in BH-1 and 3.0 m in BH-2. This abrupt increase in penetration resistance indicates a transition from relatively weak and compressible near-surface materials to a denser and more competent subsurface layer, suggesting a significant change in lithology or degree of weathering. The high N-values indicate that the underlying materials possess relatively high shear resistance and load-bearing capacity and are therefore comparatively competent from a geotechnical perspective.

Despite the presence of this relatively competent layer, the groundwater table was encountered at a shallow depth of approximately 1.0 m in both boreholes. Such a shallow groundwater table indicates saturated near-surface conditions and may substantially influence slope stability. Elevated pore-water pressures associated with groundwater saturation can reduce the effective stress and shear strength of the soil, while groundwater seepage may promote internal erosion, softening, and weathering along soil-rock interfaces or other potential planes of weakness. Consequently, the shallow groundwater conditions are considered a potential contributing factor to the observed slope instability, particularly during periods of intense or prolonged rainfall.

4.2.2 Laboratory tests

The geotechnical parameters used for the slope stability analysis of this road section were derived from borehole investigations and laboratory test results. Two boreholes, B1BH-1 and B1BH-2, were drilled to depths ranging from 15 to 25 m to characterize the subsurface conditions and obtain representative soil samples. Based on the SPT results, relatively competent material was encountered at approximately 10.0 m below ground level in B1BH-1 and 5.0 m in B1BH-2, as indicated by SPT refusal. This abrupt increase in penetration resistance suggests a transition from relatively weak upper soil materials to a more competent underlying geological formation. Accordingly, the potential slip surface was interpreted to occur near the interface between the upper, relatively weak soil formation and the underlying competent layer, at an average depth of approximately 5.0 m.

Groundwater was encountered at a depth of approximately 3.0 m below the natural ground surface in both boreholes. The presence of shallow groundwater is considered an important factor influencing slope stability because increased saturation and associated pore-water pressures can reduce effective stress and shear strength within the potentially unstable soil mass.

The subsurface profile consists of four principal geological and geotechnical units. The uppermost layer comprises imported fill material classified as elastic silt (MH). This layer is underlain by stiff, reddish-brown elastic silt (MH),

interpreted as residual soil. The third layer consists of very stiff, greyish-brown lean clay (CL), derived from the weathering and decomposition of trachyte bedrock. The fourth and deepest layer comprises highly weathered, light-grey basalt containing some silt and is classified as low-plasticity silt (ML). The laboratory test results and corresponding engineering properties of the identified soil layers are summarized in Table 3 and were subsequently used as input parameters for the slope stability analysis.

Table 3 Summaries of geotechnical investigations for slip surface identification

Cut slope subsurface profile at Km 2+520- 5+660, Bonga-1	Geotechnical Property			
	Unit weight (kN/m ³)	Cohesion (kN/m ²)	Friction angle (degree)	Groundwater table reference
Imported fill material [MH],0.0-4.5m	19.00	5.0	35.0	Partially above
Highly plastic SILT [MH], 4.5-9.5m	17.28	22.0	17.0	Below
Lean CLAY with Trachyte [CL], 9.5-21.0m	17.65	9.0	22.0	Below
Low plastic SILT with basalt [ML], 21- 25m	15.86	4.0	23.0	Below

Slope stability analysis was performed under static and dynamic loading conditions, considering both dry and wet states of the slope. The static loading conditions accounted for the effects of road-cut construction, while the dynamic analysis incorporated site-specific seismic loading for the selected slope section. However, since the subject road project is located in a seismic-free zone according to Ethiopia's seismic hazard zonation, seismic effects were not considered in the final analysis because of their negligible influence on the stability of the cut slopes. The failure analysis also considered the slope excavation at km 2+560, where the natural slope angle of 9° was steepened to 18°. During the investigation, approximately 4.50 m of the road section had been excavated and improved, incorporating both surface and subsurface drainage systems. The slope instability increased considerably from the static dry condition to the static wet condition. The primary factor contributing to slope instability was the rise in the groundwater table and the resulting increase in moisture content within the slope materials. The factor of safety obtained from the analysis was 1.347. The results indicated that the slope materials undergo significant deformation under wet conditions or when the groundwater table rises, demonstrating the critical influence of groundwater conditions on the stability of the investigated slope.

Based on the stability analysis, the possible depth of the slip surface, which is similar to the one observed during field investigation and geotechnical investigations at the sliding section, is predicted to be around 5.50m below the road formation level (see figure above) [6, 29]. This output will be used as an input parameter to recommend remedial measures for sliding [30-32]. The stiffness soil parameters used in the PLAXIS 2D software are shown below in Table 4.

Table 4 Stiffness soil parameters

Cut slope subsurface profile at Km 2+520- 5+660, Bonga-1	Geotechnical Property					
	Unit weight (kN/m ³)	Young's modulus (kN/m ²)	Poisson's ratio	Density (kg/m ³)	Vp (km/s)	Vs (km/s)
Imported fill material [MH],0.0-4.5m	19.00	5000	0.3	1936.79		
Highly plastic SILT [MH], 4.5-9.5m	17.28	1576	0.44	1761.46	1	0.31
Lean CLAY with Trachyte [CL], 9.5-21.0m	17.65	2000	0.47	1799.18	1.8	0.379
Low plastic SILT with basalt [ML], 21- 25m	15.86	1587	0.5	1616.71	2.5	0.982

4.3 Slope stability analysis

The geotechnical investigations at the site identified several critical conditions affecting the stability of the highway's hill and valley sides. The GEO5 analysis indicated that the groundwater level rises to nearly half the thickness of the fill material, as illustrated in the figure below. The calculated factors of safety obtained using different analytical methods ranged from 1.15 to 1.30, which are below the minimum values required by the applicable design standards, indicating an inadequate level of slope stability under the investigated conditions [25].

4.3.1 Boundary constraints

Xmin and Xmax (left and right boundaries):

Static analysis: The left and right boundaries were horizontally fixed ($u_x = 0$) to represent the lateral confinement provided by the undisturbed soil mass at the sides of the slope. This assumption is based on the lateral restraint typically observed in natural and cut-slope systems, where soil masses located sufficiently far from the slope remain relatively undisturbed and provide a confining boundary effect. Vertical movement (u_y) was permitted to accommodate settlement and deformation resulting from gravitational loading. This boundary condition is consistent with commonly adopted practices for finite element method (FEM)-based slope stability analyses. *Dynamic analysis:* According to the Ethiopian Geological Survey (EGS), the study area has experienced little to no significant historical seismic activity. Therefore, seismic effects were considered negligible in the dynamic analysis, and no additional lateral boundary constraints were imposed beyond those required by the numerical formulation.

Ymin (bottom boundary):

Static analysis: The bottom boundary was fully fixed ($u_x = 0$) and ($u_y = 0$) to represent the behavior of competent bedrock or deeply consolidated soil layers, which are assumed to undergo negligible displacement under static loading. This assumption is supported by the site-specific geological investigations, which identified competent rock strata beneath the slope. Fixing the bottom boundary in both horizontal and vertical directions provides numerical stability and prevents unrealistic rigid-body movement or excessive downward displacement of the model domain.

Ymax (top boundary):

Static analysis: The top boundary was left completely free (u_x) and (u_y) unrestricted) to permit natural deformation of the ground surface under gravitational loading and other external influences. This condition represents the physical behavior of an exposed ground surface, where no artificial mechanical constraints are present. A free upper boundary therefore allows realistic simulation of surface settlement and horizontal deformation. *Dynamic analysis:* For the dynamic analysis, no mechanical constraint was imposed on the top boundary, allowing the ground surface to deform freely in response to seismic excitation and associated dynamic stresses.

4.3.2 Slope stability analysis using GEO 5 software

The stability of the investigated slopes was assessed using GEO5, a finite element method (FEM)-based geotechnical engineering software package. GEO5 incorporates internationally established geotechnical analysis and design methods and provides a range of integrated modules for geotechnical applications, including slope stability, rock slope stability, reinforced slopes, spread footings, abutment design, plate analysis, gravity retaining walls, sheeting design, and settlement analysis. For the present study, the slope stability module was used to evaluate the stability of the investigated road sections under the specified loading and groundwater conditions. GEO5 provides two principal analysis approaches: the All Methods approach, which evaluates slope stability using different analytical methods, and the Optimization approach, which identifies the most critical potential failure surface and corresponding minimum factor of safety [33].

4.3.3 Factor of safety (FS)

Slope stability is governed by several factors, including the physical and mechanical properties of the soil. The physical properties include water content, soil composition, particle-size distribution, and soil texture, while the mechanical properties are primarily characterized by shear strength, cohesion, and the angle of internal friction. These parameters directly influence the resistance of the soil mass to shearing and, consequently, the overall stability of the slope. In addition, groundwater conditions and slope geometry can significantly alter the stress state and pore-water pressure within the slope, thereby influencing its susceptibility to failure. Therefore, accurate characterization of these parameters is essential for reliable slope stability assessment and the selection of appropriate mitigation measures. The factor-of-safety (FS) criteria adopted for the slope stability assessment are presented in Table 5.

Table 5 Recommended Minimum Safety Factors [34]

Cost and consequence of slope failure	Uncertainty of analysis conditions	
	Small ^a	Large ^b
Cost of repair comparable to the incremental cost to construct a more conservatively designed slope	1.25	1.5
The cost of repair is much greater than the incremental cost to construct a more conservatively designed slope	1.5	2.0 or greater

^aThe uncertainty regarding analysis conditions is smallest when the geologic setting is well understood, the soil conditions are uniform, and thorough investigations provide a consistent, complete, and logical picture of conditions at the site.

^bThe uncertainty regarding analysis conditions is largest when the geologic setting is complex and poorly understood, soil conditions vary sharply from one location to another, and investigations do not provide a consistent and reliable picture of conditions at the site.

The stability of the selected slope was evaluated using several limit equilibrium methods, including Bishop, Fellenius, Spencer, Janbu, and Morgenstern–Price. The corresponding factors of safety (FS) were 1.30, 1.15, 1.31, 1.29, and 1.29, respectively. Figure 7 presents the slope stability analysis and critical slip surface obtained using GEO5. The subsurface profile shown in Figure 5 comprises imported fill material (MH), highly plastic silt (MH), lean clay with trachyte fragments (CL), and low-plasticity silt with basalt fragments (ML). In addition, a finite element analysis was performed using PLAXIS 2D, which yielded a factor of safety of 1.437. This value is below the minimum factor of safety specified in Table 5, indicating that the investigated slope does not satisfy the required stability criteria under the analyzed conditions. The relatively low FS values obtained from both the limit equilibrium and finite element analyses indicate a potential for slope instability and failure, particularly under unfavorable groundwater or loading conditions.

Slope failure along the road corridor could result in significant socioeconomic and environmental consequences, including disruption of transportation, damage to agricultural land and nearby houses, and destruction of vegetation. To improve slope stability, appropriate remedial measures should therefore be implemented. These measures include soil reinforcement, installation of horizontal drainage pipes to control groundwater and reduce pore-water pressure, and consideration of unsaturated soil mechanics in the design and evaluation of the proposed stabilization measures.

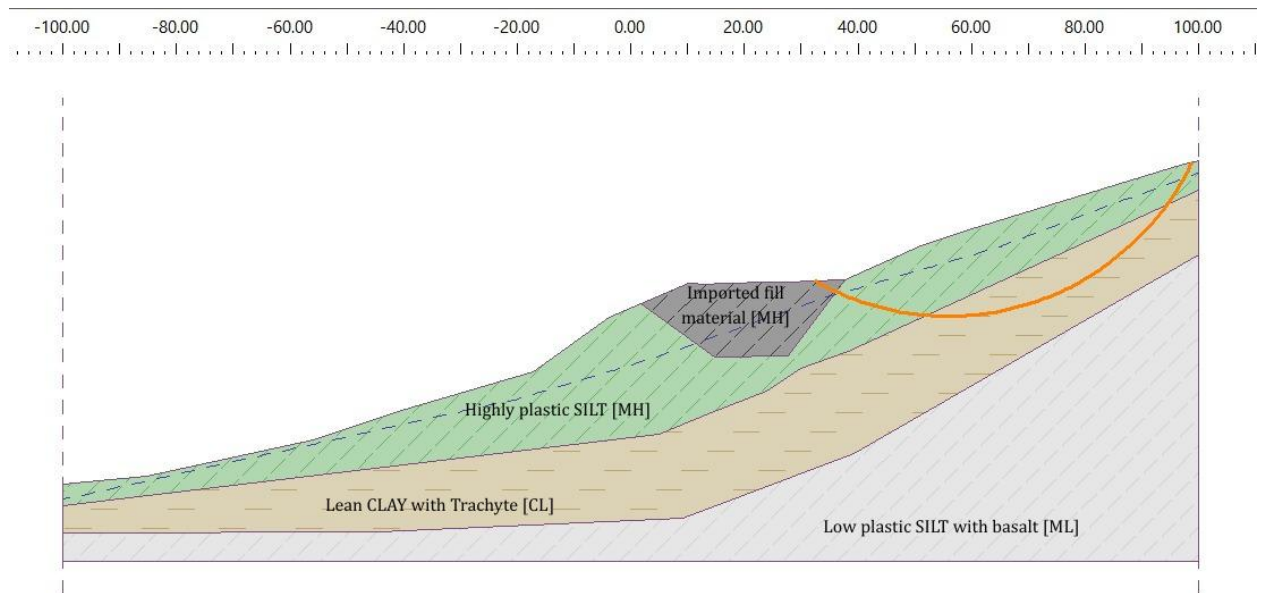


Fig.7 Line slide analysis

5 Discussion

Previous research indicates the mechanism of slope stability analysis using different numerical approaches. The analysis is different from location to location. The soil's uncertainty increases both horizontally and vertically. The findings from one soil condition are not identical to those from the other location. Previous researchers used laboratory tests to characterise the soil types in this case study. They used laboratory test input parameters such as the soil's unit weight, angle of internal friction, and cohesion. Geophysical tests and laboratory tests are implemented in this research. Numerical analysis software, such as PLAXIS 2D and GEO5, was used to determine the safety factor of the selected road section[8, 12]. The GEO5 software calculated FS using methods like Bishop, Fellenius, Spencer, Janbu, and Morgenstern. The minimum and maximum safety factors lie between 1.15 and 1.3, less than the minimum required standards [13].

Based on the GEO 5 software in Fig.7, Table 6 shows the slip surface results for the part in the orange diagram with the most critical safety factor values.

Table 6 Slope stability verification (all methods)

Methods of slope stability	Factor of safety (FS)	Remarks
Bishop	$FS = 1.30 < 1.50$	Not acceptable
Fellenius/Petterson	$FS = 1.15 < 1.50$	Not acceptable
Spencer	$FS = 1.31 < 1.50$	Not acceptable
Janbu	$FS = 1.29 < 1.50$	Not acceptable
Morgenstern-Price	$FS = 1.29 < 1.50$	Not acceptable

In light of the results above, two methods were employed: the Morgenstern Price and Bishop methods. It was determined that the FS values were not secure, as the FS value was less than the permitted FS value of 1.5. This is because the slope in question exhibits a considerable degree of steepness, coupled with the vulnerability of the soil on young rocks at the edge of the hill, which displays a relatively low soil bearing capacity. Consequently, it is imperative to reinforce the hill[35]. This process can be effectively achieved by implementing a counterweight model, which represents the optimal repair solution and is cost-effective[9, 36].

Table 7 General summary of the properties of soils from the landslide-affected area

Borehole	Depth (m)	NMC (%)	Location of groundwater level (m)	Unit weight (kN/m ³)	Specific gravity	Direct Shear	
						ϕ (°)	C (kN/m ²)
B1BH-1	0.0 – 4.5	30.5-35.9	4.5m	19	2.624-2.689	35	5
	4.5 – 9.50			17.28		17	22
	9.50 – 21.0	16.4-20.5		17.65	2.659- 2.689	22	9
	21.0-25.0			15.86	2.738	23	4
B1BH-2	0.0 – 2.0		3.0				
	2.0 – 3.50	24.7			2.72		
	3.5 – 15.0	17.8-31.3			2.69-2.73		

According to Table 7 above, the natural moisture content is high. The sample was taken during summer (kiremt in Ethiopian) in July. In the region, rainfall is high from June to September [37, 38]. Most of the time, the road fails during the rainy season. The suction value was high during the dry season, so the stability of the area was good, and the traffic was safe. The problem comes from when the rain falls, it decreases the shear strength of the soil. Silty and clay soil have high natural moisture content. The water infiltrates its particle elements between the bonds of one soil layer and the other soil layer easily[39].

As shown in Table 4, the subsurface profile consists of different soil types that vary with depth. Gravelly and sandy soils generally exhibit better drainage and shear-strength characteristics than clayey and silty soils because of their higher permeability. The relatively high permeability of granular soils facilitates rapid dissipation of excess pore-water pressure, thereby contributing to improved slope stability. In addition, the unit weight and frictional resistance of gravel and sand are generally higher than those of fine-grained soils, resulting in greater shear strength and improved resistance to slope deformation and failure. It is used to measure the total and effective stresses, which is fed to the shear strength [19, 27].

Shear strength of a soil is a combination of cohesion of the soil, angle of internal friction, suction value and the moisture content. The shear strength of the soil also affected by the natural moisture content. It disintegrate when its amount is increasing and the soil didn't support the load comes from the truck [34]. So this disintegration or decrease of the shear strength would result landslide problems along weak planes. One of the limitation of this study was the suction value was not included. It has positive contribution for the shear strength of the soil. The PLAXIS 2D software results are shown below from Fig.8 to Fig.11. This includes the geometry of the slope, the mesh obtained, and the Factor of safety before and after construction.

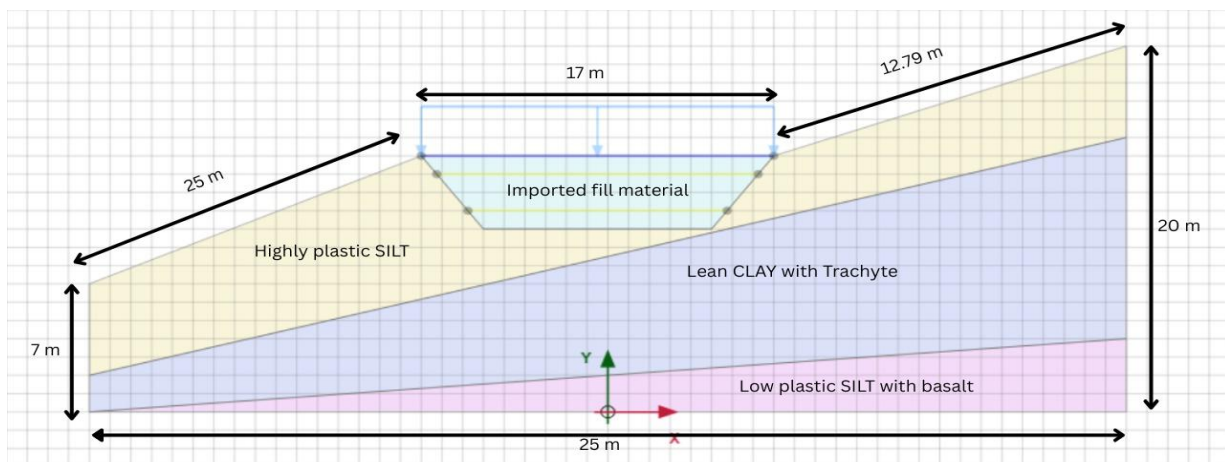


Fig.8 Geometry of the slope and soil type using PLAXIS 2D

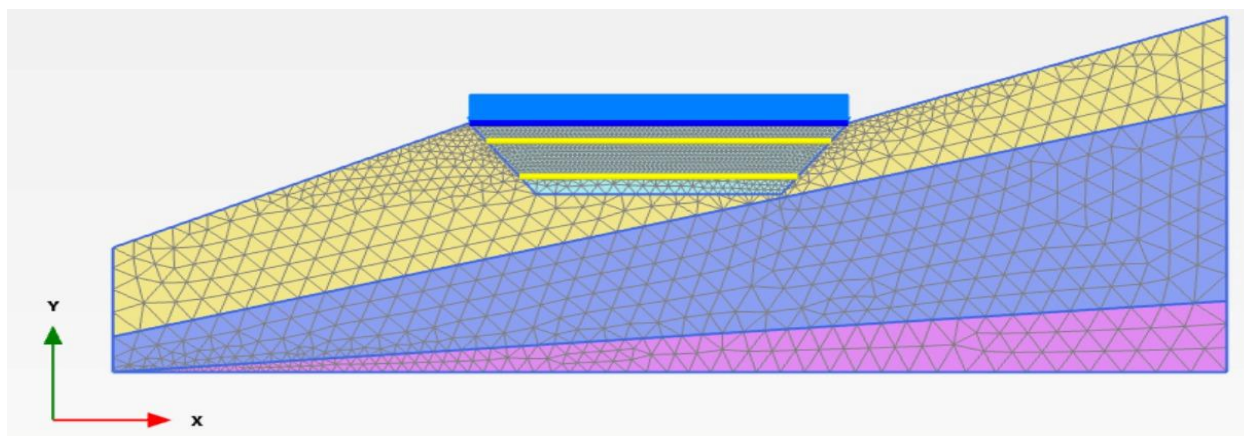


Fig.9 Mesh obtained from PLAXIS 2D

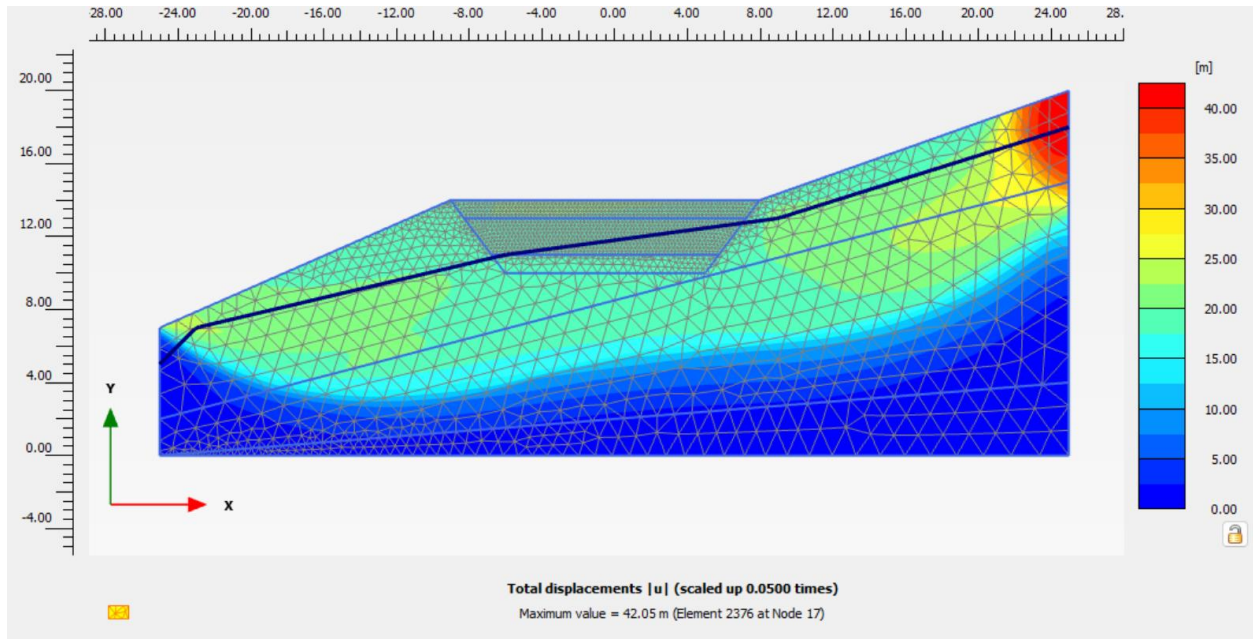


Fig.10 FS before construction (Phase_1)

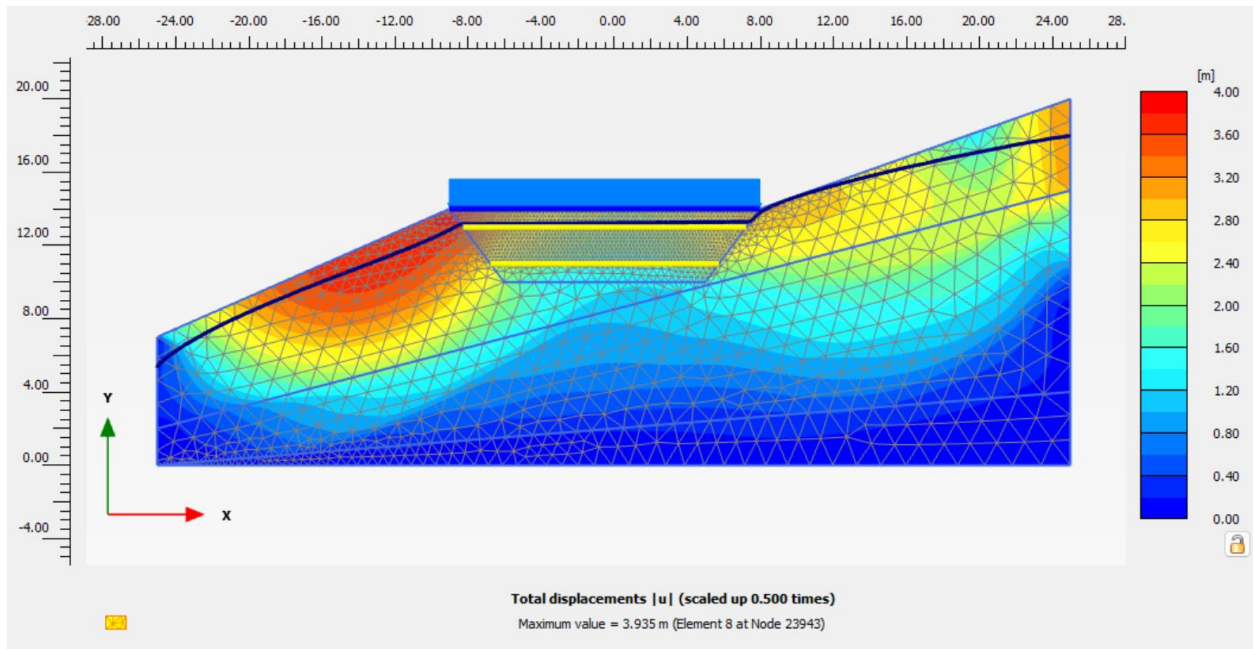


Fig.11 FS after construction (Phase_6)

5.1 Risk mitigation measures

Different slope instability mitigation measures are required depending on the subsurface stratification, groundwater conditions, soil properties, and geometry of the slope. For the unreinforced Bonga road-section slope, the analysis indicates that the factor of safety (FS) decreases to approximately 0.88 at the critical failure condition, confirming that the slope is unstable. The failure mechanism generally initiates near the slope crest, where tensile stresses may cause detachment of the near-surface soil, followed by progressive shearing and development of a slip

surface through the zone of plastic deformation. An FS below 1.5 indicates that the unreinforced slope does not satisfy the adopted stability criterion and is therefore susceptible to failure under the considered conditions.

The incorporation of retaining walls significantly improves the stability of the slope by increasing the factor of safety and providing additional resistance against downslope soil movement. When the potential slip surface intersects the retaining wall, the wall mobilizes resistance against the lateral movement of the sliding soil mass. This interaction can generate passive earth pressure, as the soil mass moves toward and bears against the retaining structure, thereby increasing the overall resistance to sliding.

For the unreinforced condition, the calculated FS was 0.88. In the retaining-wall analysis, the interface friction angle between the wall and the backfill soil was assumed to be 0° , representing a conservative design condition. This assumption neglects any beneficial shear resistance developed at the soil–wall interface and consequently results in a higher estimated lateral earth pressure acting on the retaining wall. Therefore, the adopted assumption provides an additional margin of safety in the design and evaluation of the proposed retaining-wall system.

5.1.1 Cellular confinement system

In the reinforced test sections, four different types of geocell materials were installed over a geotextile layer with varying aperture sizes and heights to investigate their geometric effects and associated soil-confinement mechanisms. The geocell configurations were selected to evaluate the influence of cell geometry on reinforcement performance. The aperture dimensions ranged from 358×258 mm to 485×353 mm, while the geocell heights varied from 100 to 150 mm in the expanded (open) state [24, 27]. While the nominal thickness of the geocells used was 1.4 mm, a constant parameter, the specimens were classified due to their closed dimensions of 440 mm and 660 mm. So, 440 x 100 abbreviation denotes the aperture and the height dimensions of geocells in the closed state, respectively [19].

5.1.2 Ground reinforcement fabric

A single layer of geotextile was used as a separation layer between the subgrade, representing the natural soil with low bearing capacity, and the base course, representing the stabilized layer. The geotextile was incorporated in both the unreinforced reference section and the geocell-reinforced test sections to provide separation between the subgrade and base materials and to minimize intermixing during loading. [20]. The properties of the geotextile material used for this purpose are given in the Table according to the manufacturer's values[40].

6 Conclusions

The landslide problems along the Bonga–Mizan road section have persisted for many years despite repeated maintenance and rehabilitation efforts by the Ethiopian Roads Administration (ERA), Jimma District. The recurrent slope failures are associated with the complex interaction of groundwater conditions, variable subsurface materials, and high and prolonged rainfall. This study investigated the major slope-instability zones along the road using an integrated geophysical, geotechnical, and numerical approach.

Seismic Refraction Tomography (SRT) and Electrical Resistivity Tomography (ERT), together with borehole investigations and laboratory testing, provided valuable information on subsurface stratification, weathering conditions, groundwater distribution, and the engineering properties of the soil and rock materials. The laboratory investigations included grain-size and hydrometer analyses, natural moisture content, dry and bulk density, consolidation, unconfined compressive strength, unconsolidated-undrained triaxial, direct shear, and specific gravity tests. The field and laboratory results were subsequently incorporated into PLAXIS 2D and GEO5 analyses to evaluate the existing slope conditions and determine the corresponding factors of safety.

The numerical results demonstrated that groundwater and increased moisture content are among the principal factors contributing to slope instability along the investigated road sections. The combined finite element and limit equilibrium analyses provided a reliable basis for evaluating the stability conditions and identifying the critical failure mechanisms. Based on these findings, appropriate site-specific mitigation measures, including geogrids, geotextiles, and groundwater-control measures, were recommended to improve slope stability and reduce the recurrence of landslides. The proposed measures can contribute to reducing road closures, maintenance requirements, traffic disruption, and the associated economic and environmental impacts.

The main limitation of this study was the restriction of time and financial resources, which prevented investigation of the entire approximately 100-km Bonga–Mizan road corridor. The study therefore focused on the major and most critical slope-instability sections identified along the road. Because subsurface geological, soil, and groundwater conditions vary spatially, the findings should not be generalized to the entire road corridor without site-specific

investigation. Future studies should extend the investigation to the unassessed sections and develop a comprehensive landslide susceptibility and risk assessment for the entire corridor.

For future field investigations, the use of the piezocone penetration test (CPTu), in combination with SPT, geophysical surveys, and laboratory testing, is recommended. CPTu can provide continuous information on soil stratification, cone resistance, sleeve friction, and pore-water pressure, thereby improving the characterization of groundwater and hydrogeological conditions. Such integrated investigations would provide a more comprehensive understanding of the mechanisms controlling landslide initiation and progression and would support the development of more effective and sustainable stabilization measures along the Bonga-Mizan road corridor.

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References

- [1] C. Siregar, H. Nuggraha, G. Azhar, and D. Warlina, "Slope Stability Analysis Using Plaxis & Slope/W (Case Study: Bagbagan-Jampangkulon Road Section STA 8+ 400, STA 8+ 200, and STA 13+ 000)," in *IOP Conference Series: Earth and Environmental Science*, 2024, vol. 1321, no. 1: IOP Publishing, p. 012031.
- [2] R. Bulko, J. Mužík, and F. Cigán, "Slope Stability Using Self-Drilling Anchor Rods," *Civil and Environmental Engineering*, vol. 20, no. 2, pp. 1267-1275, 2024, doi: 10.2478/cee-2024-0092.
- [3] W. Mohd Robi, M. F. Md Dan @ Azlan, M. A. Khalid, M. G. Keat Heng, and N. H. Basri, "The Slope Failure of Weathered Granite Boulders by Using the Finite Element Method," *Journal of Sustainable Underground Exploration*, vol. 3, no. 2, 2023, doi: 10.30880/jsue.2023.03.02.004.
- [4] M. L. B. J.P. Castagna, and R.L Eastwood, " Relationships between compressional-wave and shear-wave velocities," *GEOPHYSICS*, vol. 50, pp. 571-581, April 1985.
- [5] J. R. a. H.-J. v. Heijst, "Constraints on the correlation of P- and S-wave velocity," *GEOPHYSICS*, pp. 482-489, 2002.
- [6] F. Khosro Anjom, F. Adler, and L. V. Socco, "Comparison of surface-wave techniques to estimate S- and P-wave velocity models from active seismic data," *Solid Earth*, vol. 15, no. 3, pp. 367-386, 2024, doi: 10.5194/se-15-367-2024.
- [7] Z. Wang, C. Sun, and D. Wu, "Simultaneous Estimation of P and S Wave Velocities by Integrated Inversion of Guided P and Surface Wave Dispersion Curves," *Surveys in Geophysics*, vol. 45, no. 2, pp. 429-458, 2024, doi: 10.1007/s10712-023-09814-8.

- [8] M. N. Luc Leroy *et al.*, "Comparative Analysis of the Slope Stability Using Slide and Plaxis 2D Software: A Case Study of Tombel Pozzolan Quarry (South-West Cameroon)," *Applied and Environmental Soil Science*, vol. 2024, pp. 1-20, 2024, doi: 10.1155/2024/8260177.
- [9] H.-W. Chiu, Y.-H. Tsai, C.-W. Tang, C.-Y. Chu, and S.-L. Chen, "A Case Study of Using Numerical Analysis to Assess the Slope Stability of National Freeways in Northern Taiwan," *Applied Sciences*, vol. 15, no. 2, 2025, doi: 10.3390/app15020635.
- [10] S. M. Kassa and B. Z. Wubneh, "Use of machine learning to predict california bearing ratio of soils," *Advances in Civil Engineering*, vol. 2023, no. 1, p. 8198648, 2023.
- [11] S. Raja and C. S. N. Murthy, "Influence of Structural Discontinuity on Slope Stability using Numerical Modelling and Sirovision," *Journal of Mines, Metals and Fuels*, pp. 53-69, 2024, doi: 10.18311/jmmf/2024/34911.
- [12] H. R. N. C A Siregar^{1*}, G A Azhar², and D Warlina², "Slope Stability Analysis Using Plaxis SlopeW (Case Study: Bagbagan-Jampangkulon Road Section STA 8+400, STA 8+200, and STA 13+000)," *IOP Conference Series: Earth and Environmental Science*, 2024, doi: doi:10.1088/1755-1315/1321/1/012031.
- [13] S. T. a. R. K. Sharma, "Integrated Slope Stability Assessment and Solutions using PLAXIS 2D and GEO5," *Journal of Mining and Environment (JME)*, 2024, doi: 10.22044/jme.2025.15439.2961.
- [14] S. E. A. Sudhakar D Pardeshi, and Suchitra S Pardeshi, "landslide hazard assessment: recent trends and techniques," *PSpringerPlus*, 2013.
- [15] W. J. Kockelman, "Some Techniques for Landslide Hazards," *Bulletin of The Association of Engineering Geologists*, vol. XXIII, pp. 29-52, 1986.
- [16] I. Alcántara-Ayala, "Landslides in a changing world," *Landslides*, 2025, doi: 10.1007/s10346-024-02451-1.
- [17] C. Bonnard and J. Corominas, "Landslide hazard management practices in the world," *Landslides*, vol. 2, no. 4, pp. 245-246, 2005, doi: 10.1007/s10346-005-0014-z.
- [18] V. Chauhan, L. Gupta, and J. Dixit, "Machine learning and GIS-based multi-hazard risk modeling for Uttarakhand: Integrating seismic, landslide, and flood susceptibility with socioeconomic vulnerability," *Environmental and Sustainability Indicators*, vol. 26, 2025, doi: 10.1016/j.indic.2025.100664.
- [19] A. P. Dyson, A. Tolooiyan, and D. V. Griffiths, "Numerical Modelling Techniques for Stability Analysis of Slopes Reinforced with Shallow Roots," *Geotechnics*, vol. 3, no. 2, pp. 278-300, 2023, doi: 10.3390/geotechnics3020016.
- [20] T. E. Elahi, M. A. Islam, and M. S. Islam, "Parametric Assessment of Soil Nailing on the Stability of Slopes Using Numerical Approach," *Geotechnics*, vol. 2, no. 3, pp. 615-634, 2022, doi: 10.3390/geotechnics2030030.
- [21] R. I. Annually, "Astm standards," 1995.
- [22] T. M. Allen, "AASHTO geotechnical design specification development in the USA," in *Modern Geotechnical design Codes of Practice*: IOS Press, 2013, pp. 243-260.
- [23] I. Satyanarayana, G. Budi, and S. Murmu, "Stability analysis of a deep highwall slope using numerical modelling and statistical approach—a case study," *Arabian Journal of Geosciences*, vol. 14, no. 3, 2021, doi: 10.1007/s12517-021-06476-x.
- [24] B. Evirgen, H. O. Kara, M. S. Uzun, A. A. Gültekin, M. Tos, and V. Öztürk, "The effect of the geometrical properties of geocell reinforcements between a two-layered road structure under overload conditions," *Case Studies in Construction Materials*, vol. 20, 2024, doi: 10.1016/j.cscm.2023.e02793.
- [25] A. Fawaz, "Slope Stability Analysis Using Numerical Modelling," *American Journal of Civil Engineering*, vol. 2, no. 3, 2014, doi: 10.11648/j.ajce.20140203.11.
- [26] B. G. Habtemariam, K. B. Shirago, D. D. Dirate, and P. S. Valvo, "Effects of Soil Properties and Slope Angle on Deformation and Stability of Cut Slopes," *Advances in Civil Engineering*, vol. 2022, no. 1, 2022, doi: 10.1155/2022/4882095.
- [27] A. Hakam *et al.*, "Design of MSE Wall with geotextile reinforcement for temporary mitigation of landslide in Padang Panjang – Sicincin road section (STA 64+100)," *E3S Web of Conferences*, vol. 604, 2025, doi: 10.1051/e3sconf/202560416004.
- [28] W. Hassan, B. Alshameri, M. N. Nawaz, and S. U. Qamar, "Experimental study on shear strength behavior and numerical study on geosynthetic-reinforced cohesive soil slope," *Innovative Infrastructure Solutions*, vol. 7, no. 6, 2022, doi: 10.1007/s41062-022-00945-2.

- [29] A. S. M. M. Kamal, F. Hossain, B. Ahmed, M. Z. Rahman, and P. Sammonds, "Assessing the effectiveness of landslide slope stability by analysing structural mitigation measures and community risk perception," *Natural Hazards*, vol. 117, no. 3, pp. 2393-2418, 2023, doi: 10.1007/s11069-023-05947-6.
- [30] G. MARIES, "Seismic investigations and physical property studies of natural resources in Finland and Sweden. Efficient exploration of groundwater and mineral resources.," PhD, Digital Comprehensive Summaries of Uppsala Dissertations
- from the Faculty of Science and Technology 1936, Uppsala University, 2020.
- [31] M. L. B. a. R. L. E. J.P Castagna, " Relationships between compressional-wave and shear-wave velocities in clastic silicate rocks," *Geophysics*, vol. 50, pp. 571-581, 1985.
- [32] J. R. a. H.-J. v. Heijst, "Constraints on the correlation of P- and S-wave velocity heterogeneity in the mantle from P, PP, PPP and PKPab traveltimes," *Geophys. J. Int.*, pp. 482–489, 2002.
- [33] P. J. K. U. Arun¹, Rose Simon³, Oshin Ann Mathews⁴, E. M. Anju⁵, "A Comparison Study on Stability of Kuranchery Slopes Using GEO5 and PLAXIS 2D Software," *International Journal of Research in Engineering, Science and Management*, vol. 3, no. 3, 2020.
- [34] J. M. Duncan, S. G. Wright, and T. L. Brandon, *Soil strength and slope stability*. John Wiley & Sons, 2014.
- [35] Ali Fawaz¹, Elias Farah², Fadi Hagechade¹, "Slope stability analysis using numerical model.pdf," *American Journal of Civil Engineering*, pp. 60-70, 2014, doi: 10.11648/j.ajce.20140203.11.
- [36] D. Kang *et al.*, "Study on landslide hazard risk in Wenzhou based on slope units and machine learning approaches," *Sci Rep*, vol. 15, no. 1, p. 7511, Mar 3 2025, doi: 10.1038/s41598-025-91669-7.
- [37] C. Villaça, P. P. Santos, and J. L. Zêzere, "Modelling the rainfall threshold for shallow landslides considering the landslide predisposing factors in Portugal," *Landslides*, vol. 21, no. 9, pp. 2119-2133, 2024, doi: 10.1007/s10346-024-02284-y.
- [38] M. Takci, I. DeveliOglu, H. F. Pulat, and H. E. Demireli, "Laboratory modeling and analysis of slopes of different geometry under the effect of precipitation," *Turkish Journal of Engineering*, vol. 7, no. 4, pp. 349-357, 2023, doi: 10.31127/tuje.1191246.
- [39] J. Li *et al.*, "Evaluating failure mechanisms of excavation-induced large-scale landslides in Xinjing open-pit coal mine through integrated UAV imagery and 3D simulation," *Landslides*, 2025, doi: 10.1007/s10346-025-02460-8.
- [40] P. S. Wulandari and D. Tjandra, "Analysis of Geotextile Reinforced Road Embankment Using PLAXIS 2D," *Procedia Engineering*, vol. 125, pp. 358-362, 2015, doi: 10.1016/j.proeng.2015.11.075.