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***Three-Dimensional Modelling and Analysis of
Slope Instability in Complex Geological Environments***

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Table of contents

General Overview	1
Chapter 1. Introduction.....	3
1.2. Landslide Impacts on Transport Infrastructures	12
1.3. Methodological approach.....	20
1.4 Outline of the thesis	23
Chapter 2. FLAC3D 7.0 for Three-Dimensional Geomechanical Analysis	28
2.1 Finite difference method solution implemented in FLAC3D	31
Chapter 3 - Geological and Geomorphological Framework of the Montaguto	
Earth-Flow Study Area	41
3.1 Geological and Geomorphological features.....	41
3.2. Structural setting of the landslide area.....	48
3.3. Hydrogeological features	52
Chapter 4 - The Montaguto landslide	55
4.1. Morphological evolution of the Montaguto earthflow.....	59
Chapter 5. Geomechanical and Rheological Parameterization of the	
Montaguto Earthflow Materials	68
5.1 Characteristics of the FAE1 Member	68
5.2 Laboratory Testing.....	70
5.3 Experimental activity	72
5.4. Lithostratigraphic Setting and Physical Properties of the FAE1 Member.....	85
Chapter 6. The “La Montagna” landslide	97
6.1. General aspects of the “La Montagna” landslide.....	98
Chapter 7 – Geological and structural data acquisition for Numerical	
modelling.....	108
7.1 Methodological approach.....	109
7.2 Field data collection phase.....	109
7.3. Stereographic analysis of discontinuity poles.....	112
7.4. Geometric configuration and spatial characterization of discontinuities.....	114
7.5 Sector analysis and geometric parameter.....	119
7.6. From Field Mapping to Computational Input: Establishing the Geomechanical	
Basis for FLAC3D	121

Chapter 8. Slope stability analysis	123
8.1. Wedge Stability Analysis.....	124
8.2 Kinematic interpretation	128
Chapter 9. Three-Dimensional geometric reconstruction of the “La Montagna” landslide	133
9.1 Geomechanical iscretization and feature labeling	134
9.2. NURBS-Based Geometric Synthesis and Topo-Structural Modeling in Rhinoceros	136
9.3. Numerical Implementation in FLAC3D: Mesh Integration and Interface Characterization	139
Chapter 10. Results	142
10.1 Finite Difference Method (FDM) Simulation of 'La Montagna' Landslide Kinematics: Model Architecture and 3D Computational Strategies.....	142
10.2 Morpho-Structural Modeling and Volumetric Characterization of the Landslide Geometry.....	143
10.3 Geomechanical Parameterization and Elastic Stress Initialization in FLAC3D Slope Stability Analysis.....	148
10.4. Numerical Implementation of Elasto-Plastic Constitutive Laws and Shear Failure Mechanisms	152
10.5. Kinematic behavior and shear displacement along structural discontinuities	157
10.6 Limit State Analysis and Factor of Safety (FoS) Determination via the Strength Reduction Technique.....	160
11. Discussion of results	163
11.1 Analysis of Numerical transition and Model validation	163
11.2 Mechanical Significance of the Factor of Safety (FoS 1.02).....	163
11.3 Fracture Hydrodynamics and Meteorological Triggers	164
11.4 Kinematic Interpretation of the Displacement.....	164
11.5 Vulnerability Analysis of Strategic Infrastructure.....	165
11.6 Proposed Mitigation Strategies	166

Conclusions.....	167
References	171
Acknowledgements	183
Appendix.....	185

General Overview

Slope instability is a major geotechnical challenge in land management, posing a serious threat to infrastructure, settlements, and socioeconomic systems.

Landslides, defined as the movement of rock, debris, or soil down a slope (Cruden, 1991), are among the most destructive and complex natural phenomena to predict and often result in severe human and economic losses (Petley et al., 2012).

Landslides occur when predisposing factors (such as lithological makeup, structural features, morphology, and hydrogeological conditions) interact with triggering factors, including heavy or prolonged rainfall, rapid changes in pore pressure, and earthquakes. These processes alter the slope's mechanical and hydraulic stability, weakening resisting forces and leading to failure. Additionally, human activities frequently disrupt the natural balance of slopes.

Specifically, building and maintaining linear infrastructure such as roads, railways, and pipelines causes significant disturbances to hydrological and stress patterns, which can worsen slope instability. Measures such as excavations, embankments, drainage systems, or retaining structures may initiate or reactivate landslides long after they are built. Managing the risks associated with these infrastructures is inherently challenging because of the varying geological and geomorphological conditions across different areas.

Similarly, point infrastructures located in unstable areas, such as strategic buildings or service facilities, are susceptible to slow and cumulative deformations. These can cause progressive structural damage that often remains undetected in the short term but can ultimately compromise the safety and functionality of the structures.

Therefore, assessing these landslide–structure interactions require an integrated approach that combines geological, geotechnical, and structural engineering knowledge.

Over the past decades, climate change has further exacerbated this situation by altering rainfall patterns and increasing the frequency of extreme weather events, thereby increasing the likelihood of landslides. At the same time, the ageing of existing infrastructure (often designed to outdated safety standards) has heightened overall territorial vulnerability. In response to these challenges, scientific research has increasingly emphasized multidisciplinary approaches to evaluate landslide hazard, vulnerability, and risk associated with human activities. In this framework, 3D numerical modelling is crucial because it enables a more comprehensive understanding of instability mechanisms and their effects on infrastructure. Consequently, it is an essential tool for devising effective and sustainable mitigation strategies.

This doctoral research aligns with this framework, specifically focusing on the "La Montagna" sector, recognized as a reactivation of the larger Montaguto landslide complex. The study aims to develop advanced methods for analyzing and mitigating slope instability, with particular attention to landslide–infrastructure interactions and the characterization of large-scale spatial and temporal phenomena.

Chapter 1. Introduction

Landslides are among the most recurrent and destructive natural phenomena worldwide. They mainly affect areas with hilly and mountainous slopes, where they can be triggered by tectonic, climatic, and anthropogenic factors. These gravitational processes often produce widespread and pervasive effects on the landscape.

Landslides can occur via two main mechanisms: (a) reactivating movements along a pre-existing shear surface, as described by Weeks (1969) and Skempton and Weeks (1976), and (b) due to changes in the balance between stabilizing and destabilizing forces, which happen when no initial failure surface is present, as explained by Skempton (1970) and Vaughan and Walbancke (1973).

Pre-existing shear surfaces frequently originate from antecedent geomorphological processes, such as fluvial or coastal erosion, or from glacial and post-glacial events. Conversely, variations in pore-water pressure within the soil mass (typically induced by prolonged or intense rainfall or by anthropogenic activities that modify the weight or stress distribution of the unstable mass) can precipitate the formation of new failure surfaces. Among the various triggering mechanisms, rainfall-induced failures are the most prevalent (Fourie, 1996), and their frequency is expected to increase with climate change (Hulme et al., 2002).

In the Apennine context, the lithologic and structural features of the geological formations promote the initiation and evolution of slope instability phenomena, which pose serious risks to the population and cause significant damage to linear infrastructure and to urban and agricultural settlements.

Italy is widely known as one of the countries most vulnerable to natural hazards and their associated economic damage. Landslides are widespread across the peninsula because of its complex geological and geomorphological features, with about 75% of the land classified as mountainous or hilly. The widespread presence of clay-rich rocks, which lose strength when saturated, plays a crucial role in making slopes unstable. Moreover, the structural weakness of solid rock masses, often cut by faults, joints, and fracture networks, further increases the landscape's vulnerability to landslides.

The latest national assessment of landslides and floods (ISPRA, 2024) shows that between 2021 and 2024, the area considered vulnerable to landslides in the Hydrogeological Asset Management Plans (PAI) increased by about 15%, from roughly 55,400 km² to 69,500 km². This represents approximately 23% of Italy's total land area. At the same time, regions mapped as having high (P3) or very high (P4) landslide hazard expanded from 8.7% to 9.5% of the country's territory. The most significant relative growth was seen in the Autonomous Province of Bolzano (+61.2%), Tuscany (+52.8%), Sardinia (+29.4%), and Sicily (+20.2%). Overall, these trends indicate a growing intensification of hydro-geomorphic risk across Italy.

Climate change significantly influences landslide occurrence by altering the frequency, severity, and location of hydro-meteorological extremes. Rising global temperatures and increased climate variability change rainfall patterns, affecting slope stability, soil properties, and surface–subsurface water movement. The IPCC Special Report on Climate Change and Land (SRCCL) notes that in many arid and semi-arid areas, reduced rainfall combined with increased evaporation is a major driver of land degradation, especially when exacerbated by unsustainable land management practices. These climate and human pressures reveal complex interactions between climate processes and landforms that ultimately control slope behaviour.

In addition to natural controls, human activities significantly increase the risk of landslides.

Population growth, along with the spread of settlements, infrastructure, and economic activities into unstable or potentially unstable areas, increases exposure to slope failures and other hydro-meteorological hazards. As a result, the vulnerability of communities, vital infrastructure, and socioeconomic systems to the effects of landslides continues to grow, underscoring the importance of integrated risk-reduction and land-use planning strategies.

Therefore, areas prone to slope instability are expected to expand, and more populations will face the risks of natural hazards (IPCC, 2022).

Many studies at both national and international levels have shown that indirect economic losses, particularly those stemming from disruptions to major transportation networks, can significantly exceed the direct damage caused by landslide events (e.g., Schuster and Fleming, 1986; Zezere et al., 2005; Sterlacchini et al., 2007). Therefore, effective landslide risk mitigation requires developing and implementing technologies that support reliable alert and early warning systems. These systems must be grounded in the systematic collection, processing, and analysis of precursor measurements to enable timely detection of evolving slope instabilities and prompt execution of appropriate emergency responses.

Multiple prior studies have shown that in many parts of the southern Apennines, the landslide index (the proportion of landslide-affected areas relative to the total area) can reach 90%. This underscores the region's significant geohazard, worsened by rainfall-triggered events and intense seismic activity. A key study published in 2006 produced the Landslide Map of Benevento Province (Guadagno et al., 2006), offering a detailed overview of landslide distribution in the province.

It identified 3,100 landslides, affecting roughly 18% of the area (excluding zones of detachment and triggering related to collapses and rockfalls).

Earth flows were the most common type recorded. Although they rarely cause fatalities, they lead to extensive and lasting damage to infrastructure and urban areas.

Fusco et al. (2023) highlight Campania as one of Italy's regions most prone to landslides, with about 60% of its land area vulnerable. This geomorphic susceptibility aligns with exposure patterns: the 2024 ISPRA report states that 4.7% of the population lives in zones with high or very high landslide hazard, and 19.6% live in areas with some landslide risk.

Earthflows are among the most common types of landslides in the Campania Apennines, especially where clay-rich and geologically complex formations are exposed (Del Prete and Guadagno, 1988; Martino et al., 2004; Bertolini and Pizziolo, 2008; Revellino et al., 2010).

An analysis of historical phenomena affecting Campania highlights considerable vulnerability due to the region's complex geology and geomorphology (such as the Apennines and volcanic pyroclastics), intense rainfall, and seismic activity. Records show peaks between the 19th and 20th centuries, with over 10,000 documented events (Parise et al., 2022; Di Crescenzo et al., 2008; Ferranti et al., 2024).

The Campania region has a high historical landslide frequency, with over 23,000 events recorded in the CNR-GNDICI AVI Project, which documents 20th-century occurrences across about 18,500 affected localities nationwide, many of which are in Campania.

Although there are no century-specific statistics for Campania, combined data from the AVI, GIANO, and POLARIS databases reveal a distinct evolutionary pattern: a rapid increase in the number of recorded landslides from the 19th to the 21st century, driven by improved monitoring and increased human activity.

Landslides occur in diverse geological settings with widely varying kinematic characteristics. Some, called "fast" or "rapid," are difficult to predict and monitor, posing high risks; for example, frequent rockfalls and rapid debris-mud flows in Campania over the past century.

Other landslides move slowly, though they involve large volumes of up to millions of cubic meters, and can be monitored to develop early warning systems to protect populations (Guadagno et al., 2005).

Additionally, hyperconcentrated debris-mud flows during intense hourly rain events can mobilize alluvial fans at the foothills of Campania's main mountain ranges. These processes in Campania are not well studied, mainly because their recurrence intervals often exceed 100 years and because of limited technical, scientific, and historical data (Di Crescenzo et al., 2005).

Campania's landslides mainly occur on the Sorrentine-Amalfi Peninsula, often disrupting important road and rail connections, and along the steep coastal cliffs and beaches of the Phlegraean area and Cilento, both major tourist spots.



Figure 1.1 - Amalfi landslide, 1899

In this context, the Amalfi landslide of December 22, 1899 (Fig. 1.1) resulted in the loss of 11 lives and the destruction of the Hotel Santa Caterina and a portion of the harbour.

The Palma Campania landslide in Napoli on February 22, 1986, was a significant event in which a rapid mudflow detached from Monte Sant'Angelo in the Crocelle area. The flow destroyed a two-story building and resulted in eight fatalities, including two children.

The landslide occurred on a slope covered by Vesuvian pyroclastic deposits, which are prone to rapid movement during heavy rain, as seen in other Campania landslides, such as the 1998 Sarno landslide. Post-event studies (e.g., Guadagno et al., 1988) pointed out inadequate mitigation and human vulnerability, classifying it as a flow-like landslide. Later research shows recurrence on the same Crocelle slope, with smaller volumes due to depletion of the source material (Fusco et al., 2023).

On May 5-6, 1998, the municipalities of Sarno, Quindici, Siano, and Bracigliano experienced severe landslides (Fig. 1.2). These events serve as a significant case study from both scientific and technical-managerial perspectives. There were 161 casualties and extensive economic damage.



Figure 1.2 - Sarno landslides.

Source: <https://www.recyclind.it/ita/3110/5maggio1998lalluvionedisarno/>

Within the geological and geomorphological setting of the Campania region, the Serra delle Forche landslide in Paduli (Benevento) is another significant event. Located in the hilly zone of the Tammaro River basin, west of the main Apennine watershed and in the northeastern Sannio area, the landslide occurs entirely within the Corleto Perticara Formation, which dates to the Cretaceous-Miocene. This formation consists mainly of layered clayey and clayey-marly deposits on the middle and lower slopes, with calcillutites and calcareous marls at the top. The landslide is classified as a complex type, characterized by translational and rotational-translational movements in the source zone, which later develop into earth flows.

On November 23, 1980, an earthquake measuring 6.9 on the Richter scale affected about 17,000 km² across Campania, Basilicata, and Puglia. It caused approximately 2,914 deaths, over 8,000 injuries, and displaced 208,000 people. The seismic event triggered numerous landslides, including the Serra dell'Acquara–Vadoncello landslide.

On January 3, 2002, a complex landslide, classified as a rock avalanche, was triggered on the southwestern slope of Mount Castiello (Positano, SA) by the toppling of a large rock prism. This event caused the collapse, fragmentation, and downslope movement of debris through an inertial flow mechanism. In such cases, the high speeds involved grant the movement significant destructive potential. However, the area lacks infrastructure or settlements; therefore, despite the considerable hazard posed by the unstable residual rock mass, the landslide poses no substantial risk to people or property.

Within the province of Salerno, specifically in the Rizzico locality of the municipality of Pisciotta, an extensive landslide extends along the left-bank slope of the Torrente Fiumicello, near its mouth into the Tyrrhenian Sea. The area exhibits a highly complex structural setting due to intense deformation from the superposition of threefold systems, facilitated by the ductile behavior of the rock mass. Additionally, two sets of normal faults with NW-SE and NE-SW trends are present (De Vita et al., 2013).

The fault defining the upper boundary of the landslide's left flank (where the upper segment contacts the lower segment) is crucial to the landslide's shape and movement. It is classified as a translational rockslide. The slow, ongoing motion has caused ongoing deformations in the provincial road SS47 that passes through the landslide area (see Fig. 1.3).



Figure 1.3 - Details of the damage on the provincial road ex SS447

Within this high-geomorphic-risk framework, the Montaguto landslide (Avellino Province) represents a paramount case study. As one of Europe's most extensive earth flows, it has historically caused massive disruptions to the strategic transport corridor between the Tyrrhenian and Adriatic coasts. In 2006 and 2010, major reactivations interrupted the SS90 "delle Puglie" State Road and severely encroached upon the Benevento–Foggia railway line.

The landslide occurs within a highly complex lithostructural environment, where stratified hydrogeological complexes promote spring formation at high elevations (600-800 m a.s.l.), acting as a primary driver of instability. This doctoral research specifically examines the "La Montagna" sector, which is clearly identified as a significant reactivation of the main

Montaguto landslide body. Understanding this particular sector is essential, as its recent kinematic and morphological changes pose a direct and renewed threat to the valley floor and the stabilization measures previously implemented.

The primary goal of this thesis is to deliver an integrated geological and geotechnical analysis of the "La Montagna" reactivation and its impact on critical infrastructure. The study seeks to reconstruct the developmental stages of the "La Montagna" area through detailed mapping of geological features and multi-temporal spatial analysis, thereby providing an accurate morphological and kinematic profile. This analysis provides a clear understanding of how the landslide's advance affects the transport network by linking the kinematic evolution of the mass (obtained from monitoring and 3D numerical modelling) to the layout of the SS90 highway and the railway line. The results are then compared with current and future mitigation strategies to evaluate their effectiveness, emphasizing key implications for infrastructure planning and regional safety in the highly unstable Apennine region. Ultimately, by bridging advanced 3D modeling with practical risk management, the research aims to establish a solid methodological framework to safeguard key socioeconomic pathways from large-scale slope failures.

1.2. Landslide Impacts on Transport Infrastructures

Landslide hazards pose a significant threat to Italy's transport infrastructure (Liu et al., 2018; Zhao et al., 2020) and roads (Jaiswal et al., 2010; Bornaetxea et al., 2022; Tanyas S. et al., 2022; Taylor et al., 2020). The national transport network is fundamentally built on critical infrastructure systems, known as "lifelines," that form its core. This network includes central transportation systems (road and rail) and urban mobility options (such as metro and tram lines). This infrastructure provides essential mobility and is directly accessible to the public.

In recent decades, the increasing frequency of extreme natural events has led to numerous collapses and damage to road infrastructure, raising significant concern among the public and experts. Driven by climate change, this trend has led to significant economic losses in Italy, mainly due to direct damage from natural disasters such as landslides and floods, which primarily affect transportation networks.

The effects of landslides on infrastructure vary with geological and geomorphological factors (Laimer, 2017; Lan et al., 2008; Martinović et al., 2016) and with the maintenance status of engineered slopes (Voumard et al., 2018).

Linear infrastructure (including roads, railways, pipelines, and energy lines) is a crucial and vulnerable component. Modern society relies on this complex network of infrastructure that supports systemic functions. Lifelines are essential territorial networks connecting settlements and strategic nodes across various systems. They ensure the delivery of vital services, including transportation, energy, telecommunications, water, and healthcare, all of which are critical to community survival and functioning.

According to recent estimates, governments in low- and middle-income countries collectively invest about € 862.29 billion in infrastructure each year, representing roughly 3.4-5% of their GDP (Fay et al., 2019).

However, the quality and performance of infrastructure services vary significantly among countries. In many rapidly urbanizing regions, millions of people face daily challenges due to insufficient infrastructure, often incurring high economic and social costs. Disruptions to infrastructure services are especially severe during extreme natural events. For instance, earthquakes can heavily damage port facilities and interrupt local economic activities, as seen during the 1995 Kobe earthquake, when many passenger cars and heavy vehicles were stranded along cracked roadways (Fig. 1.4).



Figure 1.4 - The landslide affected the bridge.

Source: <https://education.nationalgeographic.org/resource/kobe-earthquake/>

Lifeline safety covers more than just natural disasters like earthquakes; it also includes crises caused by human and natural hazards. For example, the A83 highway in Scotland (see Fig. 1.5) experienced repeated landslides from 2021 to 2025, resulting in weekly disruptions and a total of 270 closure days over four years.



Figure 1.5 - Source: Helensburgh (2016). Landslide on route A83.

URL: <https://www.helensburghadvertiser.co.uk/news/19239592.a83-landslides-helensburgh-public-urged-share-views-five-new-route-options/>

The A83 serves as the primary route connecting Scotland's Central Belt to remote communities and Campbeltown in the southwestern Highlands. Frequent disruptions on this road have led to serious socioeconomic problems for local communities that rely on the corridor and have adversely affected the regional tourism sector.

At around midnight on October 13, 2020, heavy rain triggered a fast-moving rotational landslide in Phong Xuan Commune, Phong Dien District, Thua Thien Hue Province, Vietnam. The landslide buried Ranger Station No. 7 and claimed the lives of 13 rescue workers (Fig. 1.6).



Figure 1.6 - Landslide damage caused to a road in Vietnam.

Source: VNEexpress. <https://e.vnexpress.net/news/news/highways-ravaged-by-landslides-in-central-vietnam-4182129.html>

A significant landslide event is the Pietrafitta landslide in the province of Benevento (Fig. 1.7), which can be classified as an earthflow due to its characteristic slow, intermittent nature and the material involved.



Figure 1.7 - The Pietrafitta landslide. Image from Guerriero et al., 2021

The site is located along the northwestern slope of the Torre Palazzo hill, at an elevation of 200-250 m a.s.l..

The volume mobilized during various reactivation events is estimated at about 300,000 m³ (Mazza et al., 2023).

The SS87 road has been repeatedly impacted by landslide reactivations. In 2016, these movements led to regular road closures. Between November 2016 and July 2017, efforts were undertaken to reduce the geomorphological hazard.

In Italy, around 1,800 highway and railway sites are currently impacted by or vulnerable to landslides. According to Firmi et al. (2021), approximately 7,700 kilometers, which is roughly 45% of the entire railway network, face risks from different kinds of landslides.

The Italian landscape shows exceptional geomorphological diversity (Soldati & Marchetti, 2017), influenced by a highly active and dynamic geodynamic regime (Bartolini et al., 2003; Carminati et al., 2010).

The country is mainly seismically active, lying along the convergent boundary of the Eurasian and African plates (Chiarabba et al., 2005).

This tectonic setting has created diverse landforms and underscores Italy's susceptibility to geohazards, such as landslides. The complex geological and geomorphological conditions greatly raise the risk of slope instability. Recent incidents emphasize the significance of this hazard: in 2015, the Sicilian landslide in Scillato reactivated, causing extensive damage to the Himera viaduct on the Palermo–Catania highway; in 2019, a landslide impacted a section of the A6 motorway from Turin to Savona, leading to irreversible damage to the “Madonna del Monte” viaduct; and in 2020, the Magra River bridge collapsed due to long-term differential displacements of an abutment caused by a landslide in the Northern Apennines (Bozzano et al., 2015; Scarpelli et al., 2024).

Their significant exposure primarily stems from their shared location on steep slopes and their reliance on slope stability.

The magnitude of this phenomenon in Italy is considerable: 94.5% of municipalities are classified as exposed to landslide risk, affecting over 742,000 buildings and 75,000 production units, including critical infrastructure essential to national mobility and economic activity (ISPRA, 2025). These features render hillslopes particularly susceptible to large earthquake-induced landslides.

Historical seismic events, including the 1980 Irpinia earthquake and the 2016 Amatrice sequence, have triggered hundreds of coseismic landslides, with peak ground accelerations (PGAs) exceeding 0.2g and localized soil liquefaction (Esposito et al., 2010). Notably, the 2016 Central Italy earthquake resulted in over 800 landslides that obstructed access routes to inhabited areas in the Sibillini Mountains.

These landslides involved volumes exceeding 1 million cubic meters and runout lengths over 500 meters. Such tectonic settings pose significant hazards, particularly when slope instability disrupts transportation and hampers emergency access to populated areas.

Recent case studies underscore the critical importance of maintaining lifeline operations during the post-catastrophe emergency phases (Chang and Nojima 2001; Casari and Wilkie 2005).

Certain lifelines must operate effectively and efficiently immediately after a disaster and, in some cases, throughout the event. This ensures quick access, rescue efforts, orderly evacuations, and comprehensive support for emergency services (Miller, 2000).

Landslide impacts on transportation networks can be both qualitative and quantitative.

Qualitative economic impacts of landslides on transport networks include (Winter & Bromhead, 2012):

- Loss of utility of parts of the road network;
- Disruptions for road users;
- Isolation of rural regions with limited access to essential services, such as healthcare and education.

Quantitative economic impacts associated with road closures are categorized into three groups:

- **Direct economic impacts:** related expenses for the repair or replacement of damaged infrastructure;
- **Direct consequential economic impacts:** associated with infrastructure disruption and service losses (e.g., costs for road closures or implementation of single-lane traffic control with traffic lights) for a limited period;
- **Indirect consequential economic impacts:** after a landslide, access to remote areas becomes limited. As a result, economic activities are disrupted, leading to productivity losses.

However, this classification can be unclear, particularly concerning the direct and indirect economic effects. To address this, Benson (2012) offered the following more precise definitions:

- **Direct losses** refer to the loss of human lives and physical damage to productive and social assets. A study by Donnini et al. (2017) analyzed the direct economic impacts of rainfall-induced landslides on road networks in the Italian regions of Marche and Sicily. The maps produced in the study, combined with cost data from local authorities, showed differences between the study areas.

In Marche, the cost per meter was estimated at approximately 1.916 €/m and 47.58 €/m², while for the main road in Sicily, the estimated costs were 15.897 €/m and 106.93 €/m².

- **Indirect losses:** are losses related to disruptions in service flow caused by direct losses (such as road closures for a specific period);
- **Secondary losses:** concern the impacts on the economic performance of the affected region, such as loss of profitability in a specific economic or agricultural sector.

Quantifying direct, indirect, and secondary losses within systemic risk frameworks requires precise definitions of key concepts, including that of “risk.”

1.3. Methodological approach

This research aims to thoroughly reconstruct the geological–technical model of the landslide known as “La Montagna” to better understand the processes that triggered and developed it.

The study integrates geological, geomorphological, and geotechnical data to analyse the slope's internal structure and the characteristics of the materials involved in the movement.

The collected data will be analysed to determine the conditions that led to failure and the factors that influenced the event's dynamics. This information will be used to develop numerical and interpretative models that simulate deformation patterns and kinematic behaviour, with the goal of reconstructing the landslide process and improving understanding of the mechanisms behind instability in complex materials.

The process is divided into multiple phases, focusing on data collection and processing to create an accurate model by combining historical data, direct surveys, and numerical analyses.

The procedures are outlined in detail below.

1. **Data Acquisition and Integration.** The first phase of the methodology involved gathering and critically reviewing existing data on the landslide. We systematically compiled technical reports, geological and geotechnical surveys (including borehole data and laboratory test results), historical maps, and archival aerial photographs from previous investigations. This documentation provided initial insights into the landslide's geometry, the site's lithologic features and stratigraphy, and the history of the mass movement. The thorough review of existing data established a vital baseline for defining the conceptual model and guided the planning and execution of the subsequent in situ field surveys.

2. **Geological survey.** The initial phase of the methodology involved acquiring and critically reviewing existing data on the landslide. Technical reports, geological and geotechnical surveys (including borehole data and laboratory test results), historical maps, and archival aerial photographs from previous investigations were systematically compiled. This documentation provided preliminary insights into the geometry of the landslide body, the site's lithologic features and stratigraphy, and the mass's movement history. The thorough review of existing data established an essential baseline for refining the conceptual model and guided the planning and execution of subsequent in situ field surveys.
3. **Geotechnical analysis:** To enhance understanding of the physical-mechanical and hydraulic properties of the materials forming the landslide mass, representative soil samples (preferably undisturbed) were collected during on-site investigation campaigns. Standard and specialized laboratory tests were then conducted on these samples. The results of this experimental characterization provided essential geotechnical input parameters (such as shear strength, deformability, and classification properties) necessary for calibrating and successfully executing subsequent numerical models of landslide stability.
4. **Data integration:** Data from detailed topographic surveys and surface geomorphological studies were integrated into a single reference framework and 3D modelling software. This facilitated an accurate determination of surface boundary conditions, such as the current elevation profile and the lateral limits of the landslide zone. Moreover, surface geological data related to structural and stratigraphic evidence along reference geological cross-sections.

5. **Subsurface morphology reconstruction:** This phase focused on reconstructing internal slope geometries and identifying the primary sliding surfaces. The analysis combined data from geognostic boreholes and stratigraphic interpretations to map the spatial arrangement of lithologic horizons and the contact zones between landslide material and underlying bedrock. The resulting models identified transition zones with varying mechanical strengths and helped define the potentially mobile volumes.

6. **Modelling:** The Digital Geometric Model (DGM), created after the reconstruction process, was discretized to define the computational domain for subsequent numerical simulations. Meshing included localized refinement in areas of significant geotechnical interest to ensure sufficient resolution where high deformation gradients occur. The numerical analysis was conducted using the Finite Difference Method (FDM) in FLAC3D, enabling a detailed examination of the three-dimensional mechanical behaviour of the landslide while accounting for complex geometry, material heterogeneity, and realistic boundary conditions. Thanks to the modelling, a detailed geological and geotechnical model of the slope was developed, which is essential for representing failure conditions and interpreting the landslide's kinematic behaviour.

1.4. Outline of the thesis

This paragraph outlines the overall structure of the present thesis.

Chapter 2. Chapter 2 provides a comprehensive overview of the key features and methodological foundations of three-dimensional numerical modelling applied to the analysis of the Montaguto landslide, with specific reference to FLAC 3D, V.7.

This chapter introduces the theoretical and computational framework underlying the finite-difference method implemented in FLAC 3D, highlighting its suitability for simulating complex slope-stability problems involving large deformations, heterogeneous material properties, and nonlinear stress-strain behaviour.

The software's main features include a zone-based discretization method for depicting geological heterogeneity and advanced constitutive models, such as the Mohr-Coulomb model for landslide materials.

It highlights FLAC 3D's ability to handle large-strain dynamics and interface elements for shear rupture along discontinuity planes, key features important for replicating the kinematic and morphological changes seen at Montaguto.

Finally, the chapter describes the workflow for setting up, calibrating, and validating the model, establishing the foundation for the site-specific simulations detailed in the upcoming sections of the thesis.

Chapter 3. Chapter 3 focuses on the main characteristics of the Montaguto study area, where the landslide examined in this research is located. It provides a detailed analysis of the geological and geomorphological features that define the region, including its formation, lithostratigraphy, and key morphostructural elements that influence its development. Special emphasis is placed on the structural framework, as its configuration is vital for understanding the deformation and instability processes affecting the slope.

Additionally, the chapter examines the area's hydrogeological aspects to clarify interactions among groundwater, surface water, and the landslide's overall dynamics.

Chapter 4. Chapter 4 examines the main features of the Montaguto landslide, emphasizing its morphological and kinematic changes over time. This section details the landslide's key characteristics, including its size, type of movement, geological materials involved, and mechanisms underlying its activation and reactivation. Special focus is placed on the morphological development of the landslide body, with the aim of identifying its main growth phases and changes in the slope's geomorphological setting.

Chapter 5. Chapter 5 focuses on the experimental investigation, particularly the detailed characterization of the geotechnical properties of the materials involved in the Montaguto landslide. It systematically describes the collected samples and the laboratory testing procedures applied. The main properties of the materials that make up the landslide body are presented, including both fine-grained soils and coarse-grained fractions.

The focus is on identifying and measuring physical and volumetric properties, including particle size distribution (granulometry), Atterberg limits, void ratio, dry density, and water content. These parameters allow for a comprehensive assessment of the soils' mechanical and volumetric behavior under different stress conditions.

Chapter 6. Chapter 6 describes the characteristics of the “La Montagna” landslide, the main subject of this research work. It illustrates the geological and geomorphological peculiarities of the phenomenon, with particular attention to the preliminary results obtained through DEM (Digital Elevation Model) analyses. These investigations enabled the reconstruction of the three-dimensional geometry of the landslide body, highlighting displacement patterns and accumulation zones. These preliminary results form the basis for subsequent numerical modelling.

Chapter 7. Chapter 7 demonstrates the essential shift from qualitative geological observation to quantitative numerical modelling in FLAC3D. After establishing the methodological approach and conducting on-site surveys, the discontinuities are statistically examined using stereographic projections to identify their fundamental geometric parameters. The study area is then divided into homogeneous structural regions to convert field data into accurate mechanical inputs. Lastly, the geomechanical foundation is established to ensure that the model discretization accurately represents the structural reality of the area.

Chapter 8. Chapter 8 shifts focus to the core stability assessment, transforming the structural data from the previous section into predictive computational models. It starts with a kinematic analysis, which is crucial for detecting potential failure modes (such as planar, wedge, or toppling failures) by examining how discontinuity orientations relate to slope face geometry. Next, it introduces the Factor of Safety (FoS), a key value that measures the balance between resisting and driving forces. This chapter then lays out the analytical framework needed to evaluate whether the slope is naturally stable or if the arrangement of discontinuities makes it vulnerable to failure under certain loading conditions.

Chapter 9. Chapter 9 explains the process of transitioning from raw field data to building the three-dimensional numerical model of the “La Montagna” landslide. The analysis begins with geomechanical discretization, in which each structural feature is labelled and prepared for digital transfer. The main part of the chapter focuses on using Rhinoceros for NURBS-based geometric synthesis, enabling accurate reconstruction of complex topographic surfaces and structural discontinuities. In the end, the geometric model is imported into FLAC3D to create a mesh and perform mechanical analyses of interfaces.

Chapter 10. Chapter 10 details the main research findings, where the geological complexity of the "La Montagna" landslide is converted into a dynamic numerical model using the Finite Difference Method (FDM). The analytical process involves several key stages. First, the 3D computational setup and geomechanical parameters are outlined, including the initialization of elastic stresses in FLAC3D. Next, the use of elasto-plastic constitutive laws is explained to simulate shear failure and the kinematic responses along structural discontinuities. The chapter concludes with a limit-state analysis and the calculation of the Factor of Safety (FoS) using the Strength Reduction Technique (SRT).

Chapter 11. Chapter 11 provides a crucial summary of the entire study, analysing the numerical data to identify risk scenarios and the related engineering solutions.

This work also presents, in the *appendix*, the results of a scientific study on how a landslide affected specific infrastructure in the municipality of Barano d'Ischia, along the Maronti cliff in the Province of Naples, Southern Italy.

It refers to the paper "***Multi-temporal evolution analysis of Maronti Cliff (Ischia Island, Italy) derived from multi-perspective photographic datasets***", published in the *Italian Journal of Engineering Geology and Environment* (DOI: 10.4408/IJEGE.2024-01.S-28).

This work utilized photogrammetry and conducted survey campaigns to capture high-resolution images illustrating the cliff's deformation. These images formed the basis for all subsequent analyses, including qualitative and quantitative evaluations of retreat rates and localized geological and morphological studies. Employing 3D modelling software enables a detailed assessment of key evolutionary processes and the exploration of risk-management scenarios. Moreover, photogrammetric techniques (often using open-source software) enable rapid generation of detailed 3D models and help detect subtle geological features that are challenging to observe with traditional methods.

Therefore, this adaptable approach enhances geological investigations by improving data quality while reducing costs and data-collection time.

The appendix supports the main thesis by providing a practical example of multitemporal methods for analysing the evolution of slope instabilities. It acts as a useful addition to the FLAC 3D numerical approaches utilized for the Montaguto case study.

Chapter 2. FLAC3D 7.0 for Three-Dimensional Geomechanical Analysis

Among the wide range of numerical tools available for modelling unstable slopes, this thesis adopts FLAC3D, V.7.0, because it allows the representation of highly complex geometries, further enhanced by the GRIDDLE plug-in for Rhinoceros.

Flac3D, which stands for Fast Lagrangian Analysis of Continua in Three Dimensions, is a commercial finite-difference software package for geotechnical analysis. It uses an explicit Lagrangian approach and incorporates a mixed-discretization zoning technique (Itasca, 2006).

The software's calculation process involves numerous steps. In each step, an unbalanced force (arising from changes in stress boundaries or displacements across the mesh) is redistributed (Itasca, 2006).

This unbalanced force is the algebraic sum of all net nodal force vectors. In numerical simulations, a model is in equilibrium when the maximum unbalanced force relative to the total applied forces is minimal. If this force approaches a non-zero constant, it suggests sagging or plastic flow within the model. Typically, the model is considered in equilibrium if the ratio of the maximum unbalanced force to the average applied mechanical force is less than 1×10^{-5} (Itasca, 2006).

The finite difference method (FDM) converts a differential problem into an approximate algebraic form. Unlike Finite Element Methods (FEM), which discretize the continuum into smaller segments assuming a trend in a quantity, FDM does not explicitly consider a continuum. Instead, it uses a 3D grid, solving algebraic equations at the nodes after transforming the differential problem into an algebraic one.

Flac3D relies on explicit equations that are dynamic and transformed from differential to algebraic form even in static scenarios. The software solves equations of motion for static problems, effectively treating them as quasi-static dynamic problems.

This method offers several advantages, making it easier to address physical instability issues, such as large deformations and slope instability. To transform a dynamic problem into a quasi-static one (by dissipating kinetic energy), FLAC-3D uses fictitious damping proportional to grid-point velocities.

In the finite difference method, the significance of the mesh size increases. It is worth noting that 'mesh' is not entirely precise; it's not a discretization of the continuum but an overlay of a grid with points where equations of motion can be solved algebraically. Therefore, 'grid point' might be a more accurate term.

The mesh element is called a “zone” (Fig.2.1).

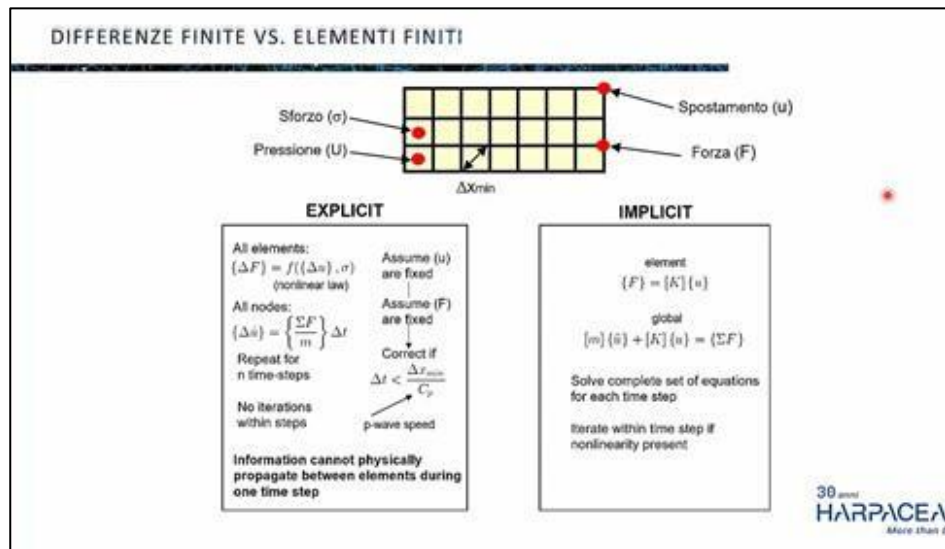


Figure 2.1: Comparison of Explicit vs. Implicit time integration schemes in numerical modeling (FDM vs. FEM)(Harpaceas)

FLAC3D allows the creation of highly complex meshes and overcomes the limitations of structured grids typical in many finite-difference (FDM) codes. In this method, the governing equations are discretized using finite differences at the nodes of a computational grid. A detailed description of the finite-difference scheme used in FLAC3D is provided below (Fig.2.2).

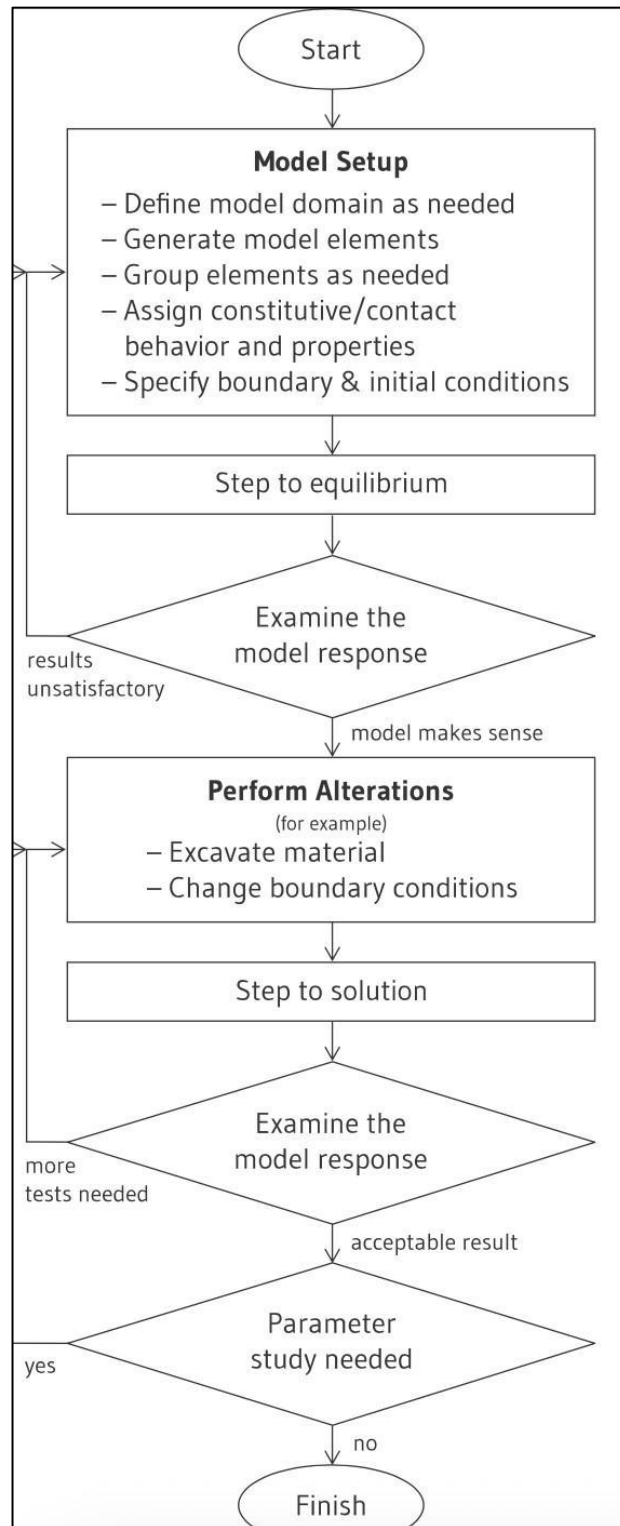


Figure 2.2. Iterative workflow for geomechanical numerical analysis. (Itasca, 2016)

2.1. Finite difference method solution implemented in FLAC3D

2.1.1. Finite Volume Approximation of Spatial Derivatives

The finite-volume formulation of the strain-rate tensor components for a tetrahedral element is presented below as an initial step toward deriving the nodal equations of motion. The tetrahedron's nodes are locally indexed from 1 to 4, with each face numbered according to its opposite vertex. Using Gauss's divergence theorem on the tetrahedron results in the following expression:

$$\int_V v_{i,j} dV = \int_S v_i n_j dS \quad (1)$$

Integrals are taken over the tetrahedron's volume and surface area, respectively, and n denotes the outward unit normal to the surface. For a tetrahedron with a constant strain rate, the velocity field is linear, and n remains constant across each face. Upon integration, Equation (1) simplifies to:

$$V v_{i,j} = \sum_{f=1}^4 \bar{v}_i^{(f)} n_j^{(f)} S^{(f)} \quad (2)$$

where (f) terms relate to the value of the variable associated with face f , and $\bar{v}_i$ represents the average of the i -th velocity component. For a linearly varying velocity field, this can be expressed as:

$$\bar{v}_i^{(f)} = \frac{1}{3} \sum_{l=1, l \neq f}^4 v_i^{(l)} \quad (3)$$

where l relates to the value at node l .

Substituting Equation (3) into Equation (2) and regrouping terms by nodal contribution gives:

$$V v_{i,j} = \frac{1}{3} \sum_{l=1}^4 v_i^l \sum_{f=1, f \neq l}^4 n_j^{(f)} S^{(f)}. \quad (4)$$

Substituting 'v_i' into Equation (1) and applying the divergence theorem yields:

$$\sum_{f=1}^4 n_j^{(f)} S^{(f)} = 0. \quad (5)$$

By employing this relation and dividing Equation (4) by V, we derive the following result:

$$v_{i,j} = -\frac{1}{3V} \sum_{l=1}^4 v_i^l n_j^{(l)} S^{(l)} \quad (6)$$

and the components of the strain rate tensor can be expressed as:

$$\xi_{i,j} = -\frac{1}{6V} \sum_{l=1}^4 \left(v_i^l n_j^{(l)} + v_j^l n_i^{(l)} \right) S^{(l)} \quad (7)$$

2.1.2. Nodal formulation of the Equation of Motion

The nodal formulation of the equation of motion is summarized below, using the principle of virtual work for an equivalent static problem at a specific time. Inertial terms at the nodes are approximated as artificial forces to ensure the solution meets the equilibrium equations, as shown in Equation (8).

$$\sigma_{ij,j} + \rho b_i = 0 \quad (8)$$

Fixing time t , the static equivalent problem is analyzed for the system's equilibrium equation at each instant.

$$\sigma_{ij,j} + \rho B_i = 0 \quad (9)$$

with body forces defined as:

$$B_i = \rho \left(b_i - \frac{dv_i}{dt} \right) \quad (10)$$

Within the finite-volume approximation framework employed, the medium is represented as a continuous assembly of constant-strain tetrahedra subjected to body forces [B].

The nodal forces $[f]^n$ with $n = (1,4)$, acting on a single tetrahedron in static equilibrium under equivalent stresses and body forces, are derived by applying the principle of virtual work. After imposing a virtual nodal velocity $\delta[v]^n$ (which induces a linear velocity field $\delta[v]$ and a constant strain rate $\delta[\xi]$ within the tetrahedron), the external work rate performed by the nodal forces $[f]^n$ and body forces [B] is equated to the internal work rate performed by the stresses σ_{ij} under this velocity field.

Using the notation convention (a superscript denotes the nodal value of a variable) and the Einstein summation over indices i and j , the external work rate can be expressed as:

$$E = \sum_{n=1}^4 \delta v_i^n f_i^n + \int_V \delta v_i B_i dV \quad (11)$$

while the internal work variation is given by:

$$I = \int_V \delta \xi_{ij} \sigma_{ij} dV \quad (12)$$

Using Equation (7), for a tetrahedron with constant strain rate, it is possible to write:

$$I = \frac{1}{6} \sum_{l=i}^4 \left(\delta v_i^l \sigma_{ij} n_j^{(l)} + \delta v_j^l \sigma_{ij} n_i^{(l)} \right) S^{(l)} \quad (13)$$

The stress tensor is symmetric, and a vector T_l with components:

$$T_i^l = \sigma_{ij} n_j^{(l)} S^{(l)} \quad (14)$$

yields:

$$I = -\frac{1}{3} \sum_{l=i}^4 \delta v_i^l T_i^{(l)}. \quad (15)$$

Following the substitution of Equation (10) into Equation (11), the external work rate can be expressed as follows:

$$E = \sum_{n=1}^4 \delta v_i^n f_i^n + E^b + E^l \quad (16)$$

Where E^b and E^l are the external work-rate contributions of the body forces ρb_i and inertial forces, respectively. For a constant body force ρb_i within the tetrahedron, we can write:

$$E^b = \rho b_i \int_V \delta v_i dV \quad (17)$$

While E^l can be written as:

$$E^l = - \int_V \delta v_i \frac{dv_i}{dt} dV. \quad (18)$$

In accordance with the finite volume approximation outlined previously, the velocity field exhibits a linear variation within the tetrahedron.

To clarify, a local coordinate system x'_1, x'_2, x'_3 is used, with its origin at the tetrahedron's centroid, thereby enabling the expression:

$$\delta v_i = \sum_{n=1}^4 \delta v_i^n N^n \quad (19)$$

Where N^n are linear functions and $c_{0n}, c_{1n}, c_{2n}, c_{3n}$ are constants.

The equilibrium condition for the equivalent system is hereby established by requiring that, at each node, the total of the equivalent static forces $-[f]$ originating from all contributing tetrahedra, in conjunction with the nodal contributions $[P]$ from applied loads and concentrated forces, sums to zero. To articulate this condition, a notation is adopted whereby a variable with a superscript $\langle l \rangle$ signifies its value at the node labelled $[[.]]$ within the global index. The symbol $[[.]]^{\langle l \rangle}$ indicates the summation of contributions at the global node l from all tetrahedra converging at that node. Utilizing these conventions, Newton's law at the nodes can be expressed as:

$$F_i^{\langle l \rangle} = M^{\langle l \rangle} \left(\frac{dv_i}{dt} \right)^{\langle l \rangle} \quad (20)$$

Where $l=1, n_n$ is the total number of nodes involved in the medium representation, and the nodal mass $M \langle l \rangle$ is defined as:

$$M^{\langle l \rangle} = [[m]]^{\langle l \rangle} \quad (21)$$

Moreover, the unbalanced force is given by:

$$F_i^{\langle l \rangle} = \left[\left[\frac{T_i}{3} + \frac{\rho b_i V}{4} \right] \right]^{\langle l \rangle} + P_i^{\langle l \rangle}. \quad (22)$$

This force equals zero when the medium has reached equilibrium.

Once equilibrium is achieved with zero unbalanced force, the computational strategy shifts to time integration, where key differences emerge between explicit (DF) and implicit (EF) methods.

The key difference between the explicit method (DF) and the implicit method (EF) is that DF can track non-linear stress-strain relationships without extra iterations in the computation cycle. In contrast, EF requires iterative interactions to accurately model non-linear problems.

For a given problem size and type, the DF's computational time grows roughly with $n^{3/2}$, where n is the number of grid points.

Meanwhile, the EF's computation time tends to increase with n^3 for the same problem size. It is also important to note that physical instability observed in the explicit method does not necessarily indicate numerical instability.

Conversely, the implicit method poses a distinct challenge, as it is currently unfeasible to simulate physical instability. This renders it difficult to identify the precise trigger.

The Discrete Element Method (DEM) provides a promising approach for modelling extensive problems with minimal memory requirements. This is attributable to the absence of a stiffness matrix for storage, which significantly reduces the computational burden. Nonetheless, it is important to consider that increased RAM may be required to accommodate the method's inherent complexity.

Conversely, the Finite Element Method (FEM) may demand substantial memory resources to store the stiffness matrix. This aspect should be carefully evaluated when determining the feasibility of implementing this method. It is noteworthy that the initial method requires no additional computational time for simulations involving large displacements, unlike the EF method (Itasca, 2018).

FLAC3D includes an integrated programming language, FISH, that allows users to create new variables and functions. This provides a flexible way to customize analyses to meet specific requirements. For example, FISH enables (Itasca, 2016):

- User-prescribed property variations within the grid, such as a nonlinear increase in modulus with depth.
- Plotting and printing of user-defined variables (i.e., custom-designed plots).
- Implementation of special grid generators.
- Servo-control of numerical tests.
- Specification of unusual boundary conditions.
- Variations in time and space.

- Automation of parameter studies.

For this thesis, the Mohr-Coulomb constitutive model was employed within the software.

The Mohr-Coulomb model serves as an elastoplastic failure criterion. This indicates that the material exhibits elastic behaviour until the stress reaches a specific threshold, beyond which it undergoes plastic deformation (i.e., irreversible deformation).

The key parameters defining material behaviour according to this model are:

- **Young's Modulus (E) and Poisson's Ratio (ν):** These define the elastic behavior of the material (deformation before failure).
- **Cohesion (c):** The shear strength of the material in the absence of normal stress. It represents the material's internal "bonding strength."
- **Angle of Internal Friction (ϕ):** The shear strength due to friction between the material's particles.
- **Dilation Angle (ψ):** This parameter quantifies the volume change that occurs during plastic shear deformation. A positive dilation angle indicates an increase in volume (dilatancy), whereas a zero or negative angle indicates non-dilating or compacting behavior.
- **Tensile Strength (σ_t):** The maximum tensile stress the material can withstand before fracture.

In FLAC 3D, the failure envelope for this model follows a Mohr-Coulomb criterion (shear yield function) with a tension cutoff (tension yield function). The position of a stress point on this envelope is governed by a non-associated flow rule for shear failure and an associated rule for tension failure (Itasca, 2016). The Mohr-Coulomb criterion in FLAC 3D is expressed in terms of the principal stresses, σ_1 , σ_2 , and σ_3 , which are the three components of the generalized stress vector for this model ($N=3$). The components of the corresponding generalized strain vector are the principal strains, ϵ_1 , ϵ_2 , and ϵ_3 .

2.1.3. Finite Difference Method Implemented in FLAC3D

FLAC3D employs a distinctive solution strategy based on three core methodologies (Itasca Consulting Group, 2023):

- **Finite volume method** approximates first-order spatial and temporal derivatives within discrete volumes, based on the assumption of linear variation in the variable over finite spatial extents and time steps.
- **Discrete element modelling** substitutes the continuous medium with a discrete network, concentrating all external and internal forces at the nodes of a 3D computational lattice that represents the domain.
- **Pseudo-dynamic relaxation** leverages the inertial component of the momentum equation as an artificial mechanism to drive the system toward static equilibrium.

Using these three methods, the equations of motion for continuum media are reformulated into a discrete version of Newton's second law at the grid points. This results in a system of ordinary differential equations, which is then solved numerically using an explicit finite-difference scheme in time. The spatial derivatives used to derive the equivalent medium are obtained from the strain-rate definition expressed in terms of velocities.

To accurately capture velocity gradients and spatial variations, the domain is divided into constant-strain-rate elements (usually tetrahedral or hexahedral) with vertices aligned to the grid nodes. A diagram of the tetrahedron is provided in Fig.2.3:

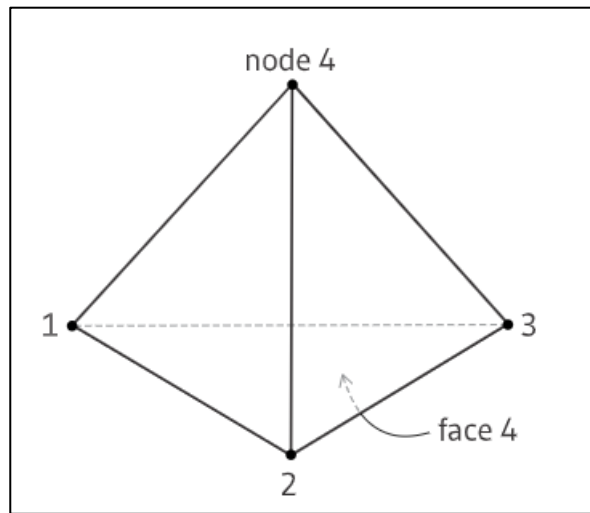


Figure 2.3 - Tetrahedral element discretization and nodal topology.

Chapter 3. Geological and Geomorphological Framework of the Montaguto Earth-Flow Study Area

This section outlines the main geological, geomorphological, and hydrogeological features and conditions of the landslide area and surrounding areas.

3.1. Geological and Geomorphological features

The Montaguto earthflow is situated in the southern Daunia Mountains on the south-facing slope of La Montagna Mt.

This part of the Apennine Range exhibits tectonic complexity, with large-scale structures and deformed bedding that significantly affect landslide shape and behaviour (e.g., Fiorillo, 2004; Revellino et al., 2010; Grelle et al., 2011). In this region, the prominent geological outcrops are the Faeto Formation Flysch and the Villamaina Unit (Pescatore et al., 1996) (Fig. 3.1 and 3.2).

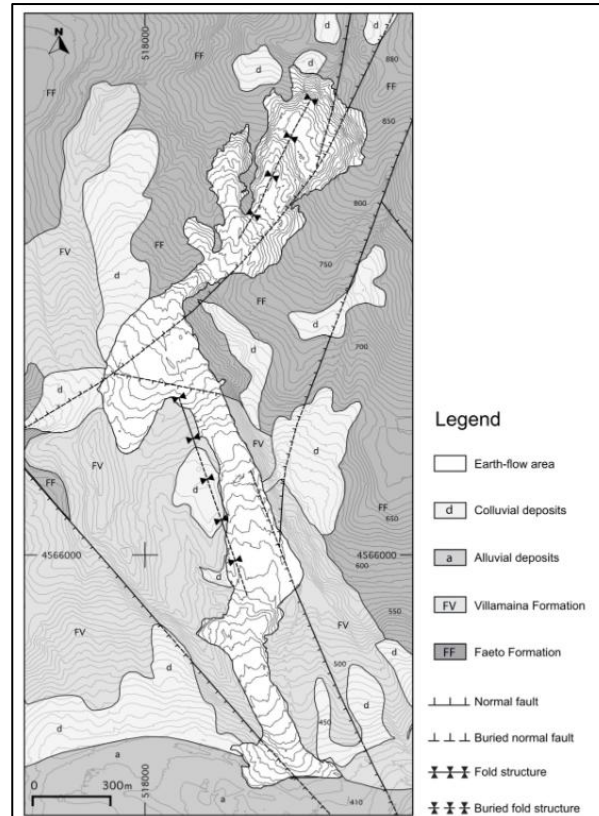


Figure 3.1 - Geological map of the Montaguto earthflow

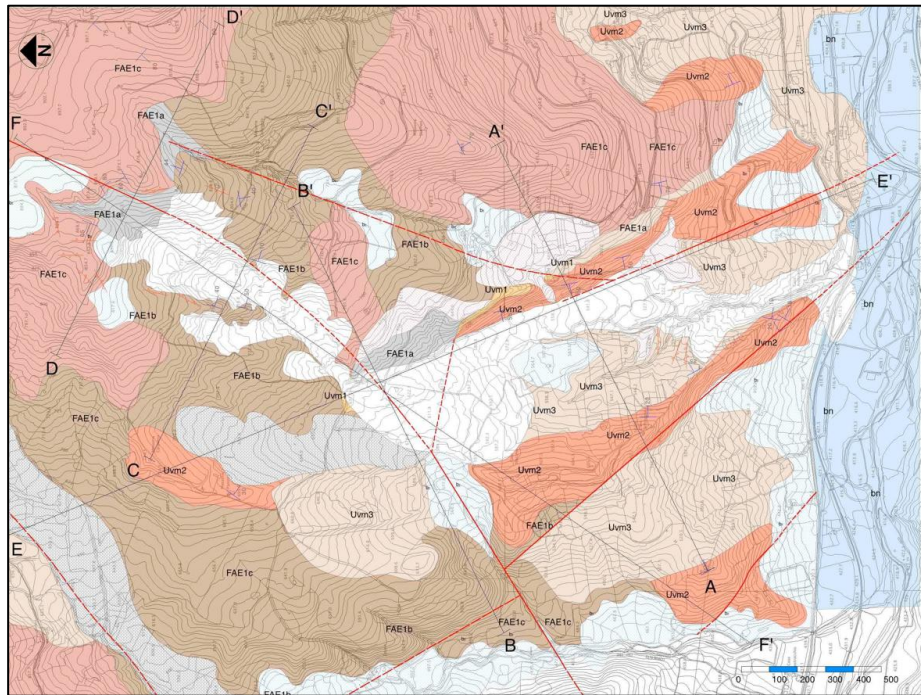


Figure 3.2 - Geological map of the Montaguto earth-flow

In the study area, exposed successions predominantly comprise multilayered calcareous, clastic, and pelitic rocks dating from the late Aquitanian to the middle–upper Tortonian.

Additionally, siliciclastic successions from the upper-lower Messinian and upper-lower Pliocene are observed. The former deposits originate from the external Lagonegrese Basin, deposited in a bathyal environment, and constitute the Daunia tectonic unit.

The latter are wedge-top depozone successions, including a portion of the Villamaina unit, developed in an inner circalittoral setting and unconformably overlying the terrains of the Daunia Unit. These sequences also encompass siliciclastic deposits from infralittoral to outer circalittoral–epibathyal environments, unconformably above all previously described deposits (Ariano Irpino supersystem).

The unconformities and associated deposits from the Miocene to Pliocene are critical for understanding the region's tectono-sedimentary evolution and regional geodynamic development.

Within the Daunia Unit, primarily calcareous-clastic successions of formation rank are identified and can be distinguished from bottom to top as follows:

- Monte Sidone calcarenites, marls, and clays (Senatore, 1988) (Aquitanian–Langhian);
- Faeto Flysch (Crostella & Vezzani, 1964) (Langhian–Tortonian);
- Toppo Capuana Argillaceous Marls (Crostella & Vezzani, 1964; Senatore, 1988; Pinto, 1993) (middle–upper Tortonian).

The Monte Sidone Calcarenites, Marls, and Clays (CMS) are the oldest recognizable deposits within the Daunia Unit. The formation's stratigraphy consists of red and green clays or marls interlayered with calcarenites and calcirudites, often exhibiting graded textures in beige shades.

The calcirudites are dominated by bioclastic material, including benthic organisms such as alveolines, echinoid radioles, bryozoans, and bivalve shell fragments, with a generally sparse matrix.

Clay chips occur at bed bases, and rare ripple marks indicate a predominant NE flow. Metasomatic chert layers are also commonly observed.

The thickness of calcarenite beds ranges from 30 cm to 1 m, whereas calcarenites are less common, with thicknesses between 20 and 70 cm. They are medium- to coarse-grained and display parallel or oblique lamination, often in Tb–c sequences.

Upsection, the pelitic fraction becomes dominant over the calcareous component, forming beds or layers of green and red argillaceous marls and levels of bentonitic clay (smectites). Higher in the stratigraphy, thin orthoquartz sandstone beds mark the Numidian Flysch lithosome (Ogniben, 1969).

Because of the characteristics of the calcareous resediments, the overall succession is interpreted to have originated from a range of gravity-driven flows, from debris flows to highly diluted turbidity currents. The total thickness of the formation is approximately 500 meters.

The Faeto Flysch (FAE) lies stratigraphically above the Monte Sidone Calcarenites, Marls, and Clays Formation, with a calcareous marl layer and a chaotic horizon interpreted as a slump horizon in between.

The FAE deposits consist of alternating layers of calcirudites, bioclastic calcarenites, calcilutites, marly limestones, and white marls, interspersed with beds or layers of light green and brown clay. Overall, these are regarded as resediments deposited by diluted gravity flows, such as turbidity currents. to result from resedimentation processes induced by diluted gravity flows, such as turbidity currents. The Faeto Flysch is divided into three members, listed from bottom to top:

- (a) argillaceous-calcareous-marly member.
- (b) calcareous-marly member.
- (c) argillaceous-marly-calcareous member.

The argillaceous-calcareous-marly member is continuous with the Monte Sidone Calcarenes, Marls, and Clays Formation. It features alternating layers of limestone, marl, or clay. The calcareous part mainly consists of bioclastic calcirudites, such as alveolines, nummulites, and bryozoans, often showing signs of reworking. These are typically well-cemented, 30–40 cm thick, and yellow-brown.

Calcarenes are vertically graded, range in thickness from 5 to 40 cm, and are medium- to fine-grained. They display planar-parallel, large-scale, wavy laminations in the upper parts, with convolute structures common throughout the bed. Complete Bouma (1979) sequences are frequently present.

Imprints at the base of the beds indicate flows originating from the northeast. The pelitic component comprises green and brown clays and marly clays, with maximum thicknesses of up to 1 meter. In the upper part of this member, the calcareous fraction increases in abundance compared to the pelitic fraction. The total thickness of this member is estimated at approximately 280 meters.

The calcareous-marly member is the most exposed regional unit and constitutes the thickest and most prominent stratum within the formation. At its base, it comprises calcareous-clastic debris-flow deposits that alternate with marls and pelites, gradually transitioning upward into a rhythmic calcareous-marly sequence.

The calcarenite beds are medium- to fine-grained, white, and typically 3 to 30 centimeters thick. These beds are notably rich in bioclasts, including bryozoans, echinoid radioles, alveolines, and nummulites. Sedimentary structures within the calcarenites indicate deposition by turbidity currents. Numerous flute casts at the base of these beds, formed by such currents, suggest sediment transport from the northeast. Higher in the stratigraphic sequence, the calcarenites transition into calcilutites and marls, which are characterized by extensive bioturbation and contain burrow casts as well as Zoophycos. The pelitic layer consists entirely of white marls, which are abundant in planktonic foraminifera. The overall thickness of this member is approximately 280 meters.

In the calcareous-argillaceous-marly member, the pelitic fraction is dominant and consists of white marls and green clays, with individual beds up to 1 meter thick. Calcarenites are medium- to coarse-grained, graded, and exhibit planar-parallel and wavy laminations.

Upsection, the calcareous beds gradually transition into marly beds, often preserving a complete Bouma sequence.

The pelitic fraction includes marls that become increasingly dominated by green clays toward the top of the unit. Flute casts are observed again, indicating sediment input from the northeast. Lithologically, this member is like the basal one but differs in its stratigraphic position, the absence of metasomatic chert nodules and layers, and the presence of marls in the lower part. Its thickness does not exceed about 100 meters.

The Villamaina Unit (D'Argenio et al., 1975) in the southern Apennines consists of a succession of deposits that overlies tectonic units derived from the Lagonegrese-Molisano Basin domain (the Frigento-Monte Arioso Unit and the Fortore–Groppa d'Anzi Unit).

In the Irpinian sector, these deposits have been reconstructed using the hierarchical criteria of Young (1975) within a regional framework of systematic units (Irpino supersystem), with the Villamaina synthem serving as the rank.

Stratigraphically and depositional-wise, the unit represents an evolutionary stage of a foredeep turbiditic depositional system during deformation (wedge-top depozone), accumulating on top of the Frigento-Monte Arioso Unit and the Fortore–Groppa d’Anzi Unit. In the Daunian Subapennines, these deposits rest on the middle to upper levels of the Daunia Unit (Pescatore et al., 1996).

At San Martino, the unit consists of poorly cemented conglomerates and sandstones with minor clay layers; upward, it transitions into gray-blue clays and silty clays containing macrofossils such as *Ostrea* and *Dentalium*. The grey-blue clays gradually grade upward into silty clays rich in macrofossils like bivalves and cirripedes, and then into sandier deposits that mark the end of the succession. The best exposures are found at San Martino (km 42 of S.S. 90). The total thickness is approximately 150 meters. Depositional environments indicate shallow marine settings that do not exceed the inner circalittoral zone. The age corresponds to the upper part of the lower Messinian.

The presence of multiple lithofacies associations permits subdivision of the unit into three partially laterally and vertically arranged members: sandy–conglomeratic, pelitic–sandy, and pelitic.

The sandy–conglomeratic member is composed of polygenic microconglomerates arranged in beds and lenses, which are occasionally fused, lenticular, or poorly stratified. It is predominantly heterometric with a sandy matrix. The rocks are yellowish, quartz-feldspathic, and quartz-lithic sandstones containing mica, exhibiting a grey-beige colouration. The formation is massive, primarily composed of laminated sections, and consists of medium- to coarse-grained material. It is also locally cemented with abundant calcareous cement.

Additionally, grey bioclastic calcirudites and calcarenites occur in lenticular beds. Sandstones are interbedded with grey-blue to brownish pelites, marls, marly clays (medium-thick and fractured in a conchoidal manner), whitish silty clays, and fine brown sandstones. In some areas, the sandstones are finely laminated.

The pelitic-sandy member comprises brownish and grey-blue pelites, marly clays (medium- to thick, with conchoidal fracture), whitish silty clays, and finely laminated fine brown sandstones. It also features interbedded quartz-feldspathic, quartz-lithic, and mica-bearing sandstones, mostly medium- to fine-grained, supported by a calcareous matrix. The member has a thickness of approximately 50 meters.

The pelitic member consists of grey-blue, silty clays with conchoidal fractures in beds. It also contains rare, thin, degraded sandstone layers and often includes beds with benthic remains such as *Ostrea*, *Dentalium*, and bivalves.

The Pliocene deposits in the intra-Apennine basins of the Campano-Lucanian Apennines are traditionally referred to as the “Ariano Unit” (now part of the Ariano Irpino Supersynthem). They are widely exposed between the valleys of the Ufita and Ofanto rivers, extending into the Baronia region and the Cervaro Stream valley, which marks the limit of their range. These deposits consist of low-marine depositional systems (conglomerates, sands, and clays) that sometimes grade into continental environments.

They have recently been revised within the hierarchical scheme of systematic units, especially the Andretta synthem and, specifically, the Baronia Formation (Di Nocera et al., 2006). In the Ariano Irpino geological sheet no. Four hundred thirty-three of the new Geological Map of Italy (1:50,000 scale) shows these deposits as BNA1 and BNA2 in the Avella Stream valley (Panni Basin) and dates them to the lower Pliocene.

3.2. Structural setting of the landslide area

The La Montagna-San Martino-Mezzana area, where the Montaguto landslide occurred, lies on a composite substrate. According to the Geological Map (Fig. 3.2), the upper part of the terrain (La Montagna) consists mainly of deposits from the lower member of the Faeto Flysch (FAE), specifically the argillaceous-calcareous-marly layer (FAE1). In contrast, the middle and lower sections are covered mainly by deposits of the Villamaina Unit (VMN). Mesoscale geological surveys identified three lithofacies associations in the lower part of the Faeto Flysch, showing partial lateral variation (heteropy), each with a distinct calcarenite/pelite ratio. Within the deposits of the Villamaina Unit, three lithostratigraphic units, corresponding to members in a latero-vertical relationship, were identified and mapped.

The Villamaina Unit rests unconformably on the Faeto Flysch within a compressional structural depression (synform), further highlighted by normal faults that form conjugate tensile systems-oriented NW–SE and SW–NE. The NW–SE systems are offset by the SW–NE systems. As a result, the southern part of the La Montagna relief features an area where various structural features develop, related to both folding and tensile faulting.

Stratigraphic and geometric reconstruction of the subsurface, extending to the longitude of the Montaguto settlement, also reveals a large-scale compressional structure aligned with the Apennine trend (NW–SE). This structure corresponds to a fold–fault system, with its overturned (eastern) limb directly beneath the settlement and a zone of reverse faulting immediately to its east.

The landslide-affected area lies within the standard (western) limb of this large fold. The normal limb exhibits hectometric-scale fold geometries consistent with Z-type parasitic folds, aligning with internal-limb deformation (fold-normal) of an asymmetric fold tilting toward the Adriatic.

The geological cross-sections A-A', C-C', E-E', and F-F' (Figs. 3.3, 3.4, 3.5, 3.6) highlight this geometric characteristic in their central parts. By examining the geological map in Fig. 1 and comparing sections A-A' through F-F', it becomes evident that the structural framework (both in its remnants and surface expression) has influenced the landslide's kinematic evolution. Specifically, the upslope part of the erosive activity has limited the flow within morpho-structural boundaries, directing movement in a NE-SW direction and sense of travel on the unstable masses.

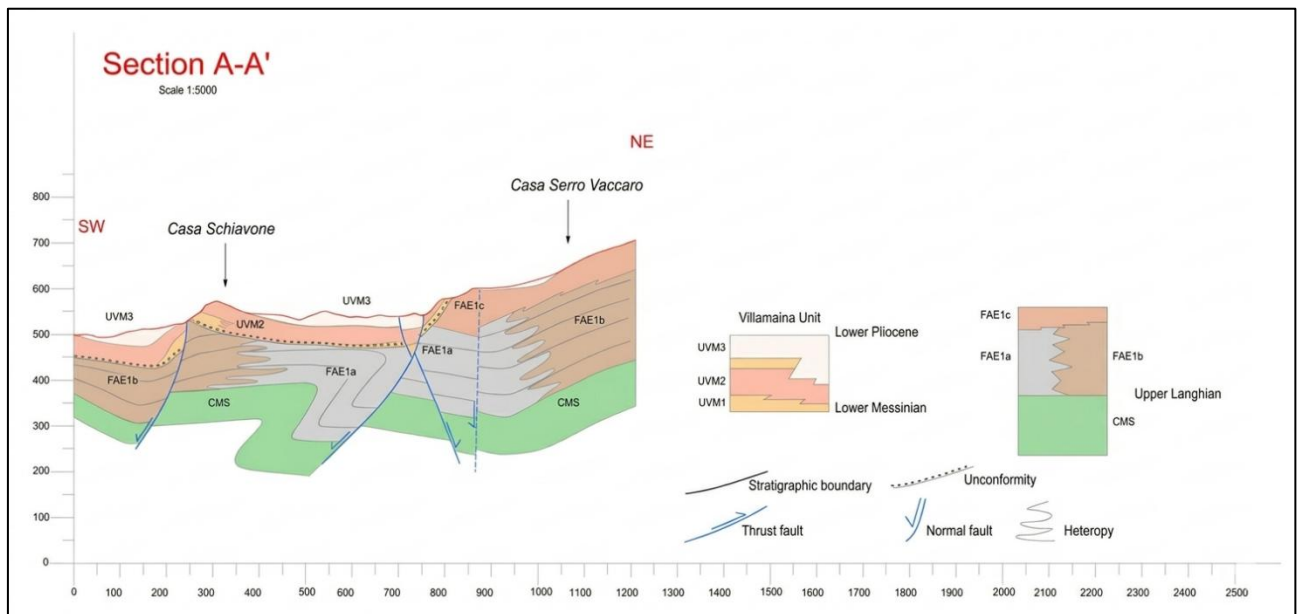


Figure 3.3 - Cross-section A-A'

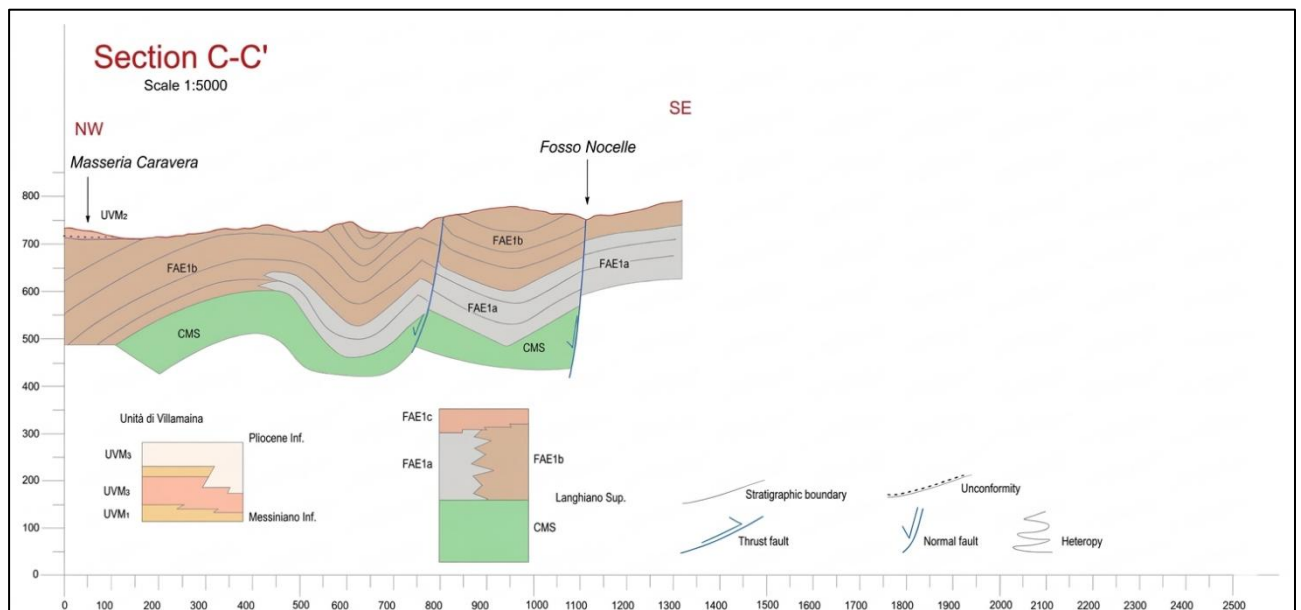


Figure 3.4 - Cross-section C-C'

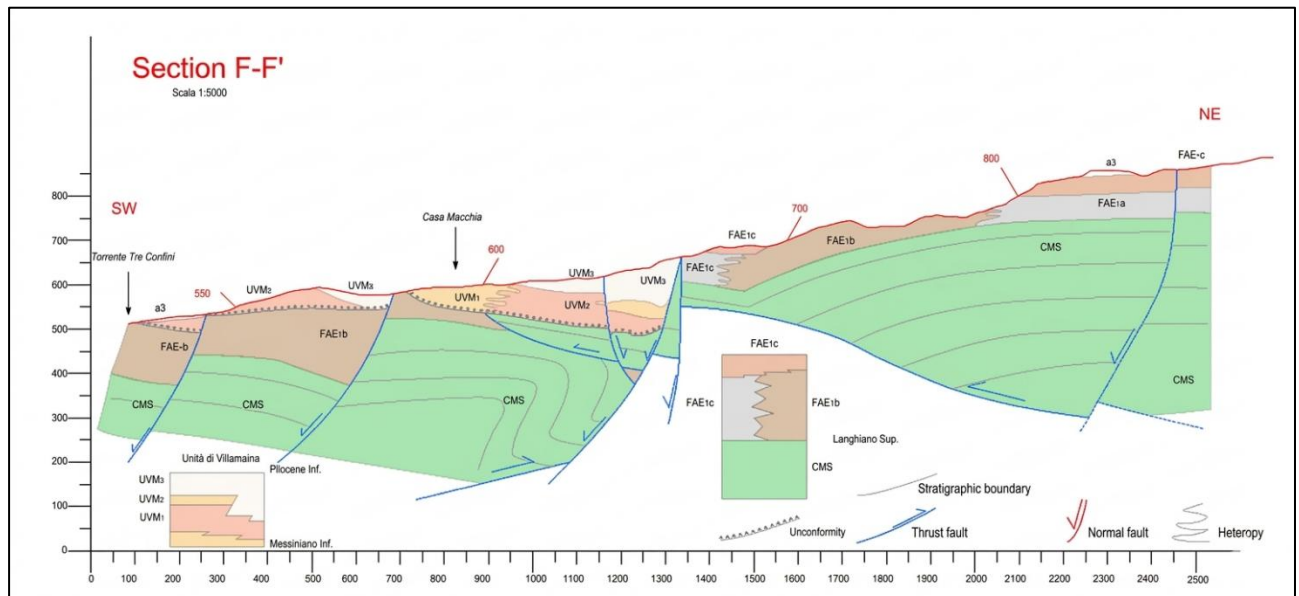


Figure 3.5 - Cross-section F-F'

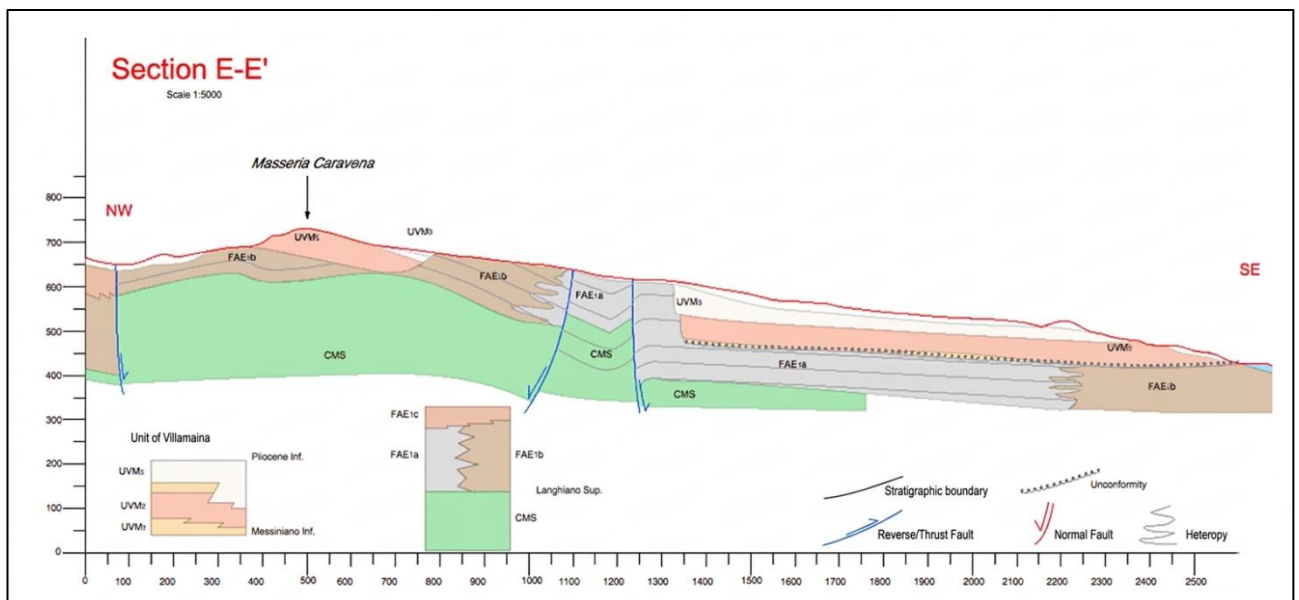


Figure 3.5 - Cross-section E-E'

At higher elevations on the southern slope of La Montagna, three lithofacies (FAE1a, FAE1b, and FAE1c) are exposed.

Lithofacies FAE1a consists of grey and light-brown clays and marly clays arranged in beds and layers, with very rare intercalations of light-grey marls and marly marls.

Laterally, this lithofacies grades into brown clays with intercalations of calcarenites and calcirudites in medium- to thick beds, graded and displaying transfacial structures, collectively characterizing lithofacies FAE1b. This heterotopic contact is visible in the upper-left erosion sector of the landslide-affected area. In this sector, bedding alternates between normally and reversely inclined layers, and tight mesoscale folds with small interlimb angles develop. Regular bedding predominates, confirming the presence of Z-symmetry of folding even at the mesoscale. Upslope, on the right side of the landslide's upper crown, the succession transitions to lithofacies FAE1c.

The calcarenite and calcirudite beds of FAE1c are arranged in a gentle “traverse-slope” orientation, with an inclination of $L0\ 334/50^\circ$, which is less than the slope's maximum gradient.

Between approximately 600 m and 400 m a.s.l., the lower part of the southern extension of the La Montagna relief is developed.

In San Martino and Mezzana, deposits of the Villamaina Unit are preserved. Their distribution and the shape of the stratigraphic boundaries reveal a structural setup typical of a synformal syncline. The presence of normal faults slightly disrupts the geometric continuity of this structure while emphasizing its low morphostructural position.

In the mid-slope areas, the clay-rich deposits of the pelitic member UVM3 are evident. Outcrops of this member consist of poorly layered, weakly marly, silt-clay deposits that, at certain stratigraphic levels, contain benthic macrofossils preserved in situ.

3.3. Hydrogeological features

Near the landslide area, the exposed lithotypes can be classified into seven hydrogeological complexes, as outlined below:

- **Detrital complex:** limited in size and confined to specific zones east of the landslide area. The complex exhibits permeability through porosity, ranging from medium to low, depending on the lithologic features of the deposits. Because of its small extent, this complex has minimal hydrogeologic significance and little relevance to landslide phenomena.
- **Alluvial complex:** located along the alluvial belt of the Cervaro River at the base of the landslide accumulation zone. It has medium-to-high permeability due to its porosity. This complex features a moderately significant aquifer with a shallow groundwater table (averaging 2–4 meters deep) that is directly hydraulically connected to the Cervaro River. Depending on the hydrological season, the river can act as a drainage channel or as a recharge area. However, groundwater flow within the alluvial aquifer does not influence the landslide zone.
- **Clayey-silty complex:** corresponding to the UVM3 clayey member of the Villamaina Formation, this complex is widely exposed on both sides of the landslide. It has mixed permeability, generally low to very low, particularly in the shallow alteration zone. Hydrogeologically, it contains a discontinuous, low-yielding, near-surface aquifer of limited importance.
- **Arenaceous-conglomeratic complex:** corresponding to members UVM1 and UVM2 of the Villamaina Formation, this complex is exposed in limited outcrops on both sides of the landslide. It is characterized by mixed, mostly low-permeability rocks. The low relative permeability makes it act as a relatively impermeable unit.
- **Calcareous-clayey-marly complex:** the lithotypes in this complex mainly outcrop east of the landslide body. Permeability is mainly due to fracturing, with values ranging from low to medium. Hydrogeologically, it contains a poorly productive aquifer, usually limited to the thin weathered surface layer.

- **Calcareous-clayey complex:** exposed on both sides of the landslide area, it is characterized by fracture-related permeability ranging from medium to high. As in the previous unit, groundwater flow is generally limited to the shallow weathered zone. However, as fracture density and aperture increase, circulation tends to deepen, forming discontinuous water tables with highly variable hydraulic heads. Under these conditions, the complex can supply perennial springs of moderate hydrogeologic significance, such as the Caraventa and Del Ponte springs, with average annual discharges usually exceeding 7.0 L/s.
- **Clayey complex:** cropping out in a small sector east of the landslide body, it exhibits very low porosity-related permeability. It can thus be considered a relatively semi-permeable unit.

The geological setting of the area strongly influences groundwater flow and spring distribution. Numerous springs are located within and around the earth-flow body, at elevations between approximately 600 and 850 meters above sea level. In the source zone, the aggregate discharge in May 2010, measured using manual timing methods (bucket and stopwatch), was approximately 2.0 L/S. Downslope, along the eastern margin of the earth flow, a group of springs that contribute to a substantial pond (Rane Lake, as detailed in the Results section) exhibited a comparable total discharge of approximately 2.0 Liters per second during the same period. The spatial arrangement of these springs, combined with the variable permeability of the exposed rocks and structural complexity, suggests that the hydrogeological system on the southern slope of La Montagna Mountain comprises multiple interconnected aquifers (Diodato et al., 2014).

Precipitation characteristics are evaluated using meteorological data from weather stations in Orsara di Puglia, Monteleone di Puglia, Savignano Irpino, Faeto, and Bovino, all within a 10 km radius of the earth flow.

The stations' elevations range from 650 meters above sea level (Orsara di Puglia and Bovino) to 900 meters above sea level (Faeto).

Historical annual rainfall data from 1921 to 2009 indicate an average of approximately 800 mm per year, with annual totals ranging from 200 mm to 1400 mm. Although meteorological data do not differentiate between snowfall and rainfall, observations in this region of the Apennines suggest that over 90% of precipitation falls as rain (Diodato, 1997; Revellino et al., 2013).

Chapter 4. The Montaguto landslide

The Montaguto landslide (Fig. 4.1) is located in the southern Daunia Mountains, along the left slope of the Cervaro River valley (Fig. 4.1, location map) and is classified as a translational earthflow, with localized superimposed viscous flow.

It shows intermittent activity, marked by backward expansion in the source area and by structurally driven displacements within the same zone.

The landslide body extends approximately 3 km, covers 67 hectares, and involves nearly 6 million cubic meters of material.

The earth flow width ranges from roughly 75 m at the narrow neck to almost 450 m at the upper source zone. The total relief, measured from the toe near the Cervaro River to the crest of the 90-m-high headscarp, is approximately 440 m. Excluding the headscarp, the average slope gradient is about 7.2° .

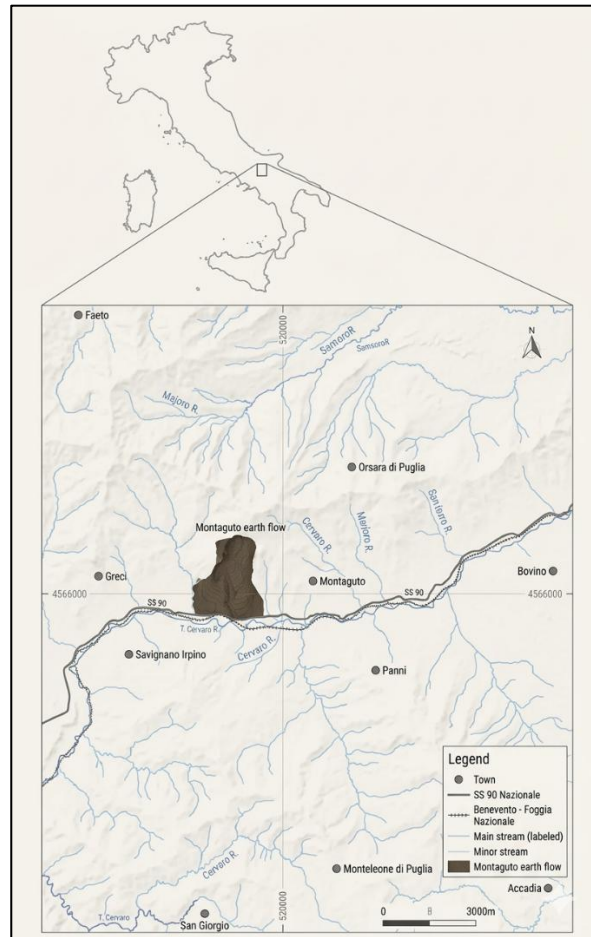


Figure 4.1 - Schematic map of the study area showing the Montaguto earth flow footprint. The spatial relationship between the landslide body and the strategic transport corridor (SS90 and national railway) is shown. The base map includes the main and minor stream drainage patterns and the surrounding municipal territories for regional context

The landslide movement shows a typical multi-source pattern with two main lobes.

The source area boundary appears structurally complex, influenced by the arrangement of flysch formations and the presence of a soil and weathered layer that is periodically affected by block-translational movements associated with retrogression. The source area extends about 900 meters in a NE–SW direction. It is characterized by a prominent scarp with locally steep slopes exceeding 40° , and laterally irregular flanks with variable inclinations, generally greater than 45° . These morphological features reflect the structural setting of the outcropping rocks.

Along the upper part of the scarp, several shallow “rafts” covered by herbaceous and shrub vegetation can be observed moving. At the same time, immediately downslope, clayey flow material accumulations are present. At the base of the scarp, a morphological terrace associated with subsided bodies can be identified, with two morpho-depressions developing. These depressions once contained small landslide ponds, which are now drained through siphon systems.

The flanks are primarily affected by flow-like and translational processes. However, rockfalls occur along lithoid outcrops, depending on slope gradient and the lithologic characteristics of the exposed rocks.

On the left flank, two source lobes are identifiable and do not appear to be significantly active currently. Immediately downslope, a newly formed lobe impacts farmland. In this area, the displaced material moves intermittently, controlled by rainfall and spring runoff. Periodically, internal shear interfaces develop within the landslide deposit, dividing sections that move at different speeds.

At an elevation of about 650 m above sea level, a structural scarp marks the tectonic boundary between the Faeto Flysch and the clays of the Villamaina Unit, marking the start of the central channel zone.

The channel extends roughly 1,600 m northwest–southeast, maintaining a nearly constant width.

Its upper part lies above an ancient landslide deposit, while its lower section is carved into the rocks of the Villamaina Unit. The overall shape is nearly straight, bordered on the left by outcrops of the “Arenarie di Villamaina.” In this area, where the slope decreases from approximately 10° in the source region to about 7°, the landslide material beneath a thin surface crust (10-40 cm) exhibits viscous–plastic behaviour.

Along the channel, localized counter-slope areas form due to differential movement within the landslide mass, and small landslide ponds develop in these sections. The lower part of the channel has an even gentler gradient, with a slope of around 20°.

From this point onward, the terminal accumulation zone develops, where the landslide deposit occupies a narrow valley that extends downslope to the base of the slope, with a total length of approximately 500 meters. In this sector, about 300 meters upslope from the distal limit of the accumulation zone, the deposit thickness was measured and found to exceed 20 meters. This suggests a significant buildup of fine-grained, reworked material, likely resulting from multiple depositional phases associated with successive movements of the main landslide body.

Figure 4.2 shows the topographic profile of the landslide to highlight its unique features.

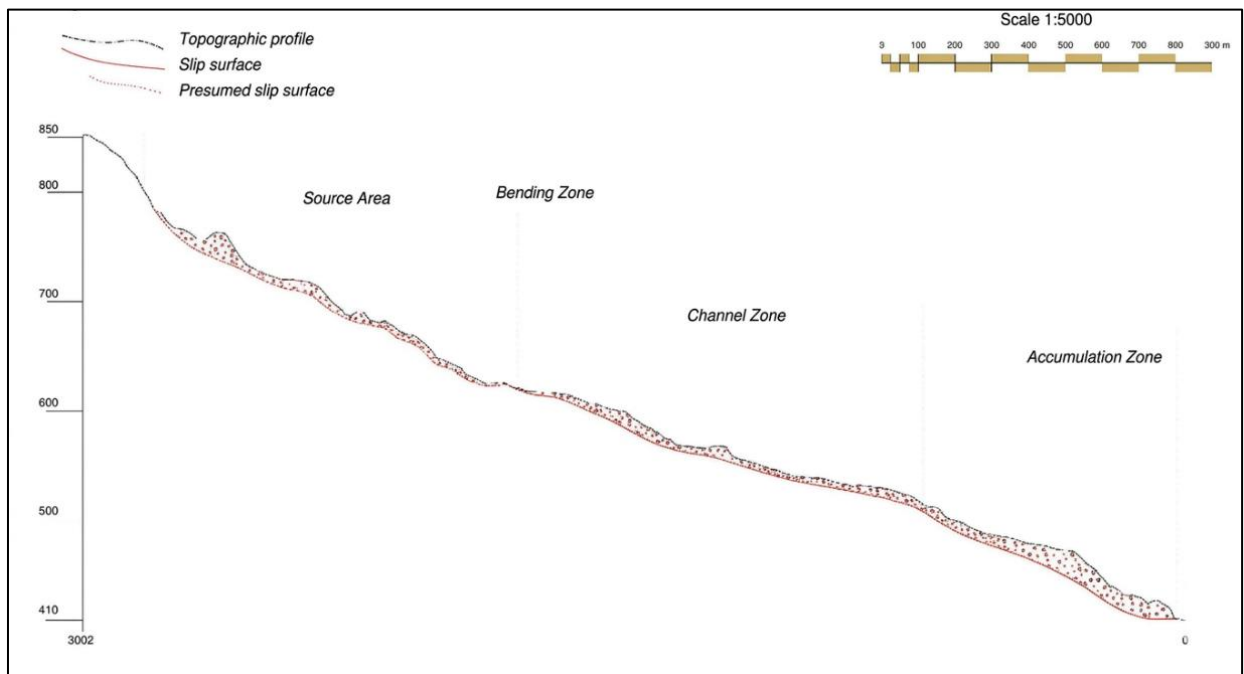


Figure 4.2 - Longitudinal section of the Montaguto landslide (2010 data). The profile highlights the different morphological zones: source area, bending zone, channel zone, and accumulation zone. Solid and dotted red lines represent the observed and presumed slip surface, respectively.

4.1. Morphological evolution of the Montaguto earthflow

Since 1954, the landslide has exhibited prolonged periods of slow movement, punctuated by episodes of rapid acceleration (Guerriero et al., 2013), including those observed in 2006 and 2010. The latter event represents one of the most intense activity phases, measured by both displacement rates and affected volume. Over time, the shape of the landslide body has changed considerably, reflecting ongoing material redistribution and the spatial–temporal variability of movement rates.

The main stages of landslide activity are outlined below.

1954 - The landslide, dating to 1954, is already quite large, extending about 1,000 meters. Its width varies from approximately 200 meters at the source area to around 400 meters in the accumulation zone. The edge of the source area is roughly 700 meters above sea level (a.s.l.).

Upslope, there are two smaller features outside the main landslide body, with source areas at 775 meters and 750 meters a.s.l., respectively. These features likely involve only the uppermost meter of bedrock and affect the sides of an upslope drainage channel. The source area is complex, featuring three lobes along the right flank and a series of scarps.

At about 650 meters a.s.l., the landslide narrows to roughly 40 meters, aligned with the tectonic boundary between the Faeto Flysch rocks and the clays of the Villamaina Unit.

From a top-down view, the accumulation area is extensive and can be divided into three deposit levels over time. The most recent deposit is fan-shaped, covering roughly 40,000 square meters, and lies above an older, less distinct deposit that spans about 100,000 square meters and is thinner. This older layer rests on a third, even older deposit, of which only its lower part is visible. On the slope, other flow-like features also cover surfaces ranging from 3,000 to 10,000 square meters.

1976 - In 1976, the landslide became significantly more complex, extending about 2,000 meters. The source area is entirely different from the 1954 one, with its rim now at roughly 775 meters above sea level, matching the 1954 source rim. It follows a typical multi-source pattern with two lobes, both formed by retrogression from the 1954 lobes. This pattern is likely linked to a period of intense activity (likely due to persistent heavy rainfall, which still needs verification), causing not only the expansion of the source area but also the development of minor landslides both within and outside the main body. Most minor phenomena within the main landslide occur along its edges, with source areas situated along the boundary of the main body and genetically connected to the retrogression process.

Their kinematic characteristics are strongly influenced by the type of exposed materials along the flanks: on the right side, minor events mainly show translational movement, while on the left side, they display flow-like behavior. Outside the main landslide, flow phenomena are observed both upslope of the source area and downslope of the accumulation area. Importantly, the downslope flow phenomena are covered by vegetation, indicating temporary stabilization.

In this evolutionary stage, a rectilinear channel area is observed immediately downslope of the source area, starting at 650 m a.s.l. and extending in a NW–SE direction. This channel is superimposed on an extensive ancient accumulation with a maximum width of 450 m.

On the left side, the channel borders sandstone-conglomerate rock outcrops that control its path, ensuring that material movement follows consistent trajectories. From approximately 550 m a.s.l., onward, the accumulation area develops, characterized by a lobate morphology with a length of about 600 m and a maximum width of approximately 200 m. The thickness of the accumulation cannot be reliably estimated.

1985 - The 1985 landslide exhibits a similar level of complexity and size to the 1976 landslide. The source area is partly covered by vegetation, and some smaller features within the main 1976 landslide body are no longer visible. The channel area remains essentially unchanged from 1976, as does the accumulation area, which shows only a slight increase in width and length.

Similarly, smaller features outside the main landslide are either obscured by vegetation or no longer visible, except for an area downstream of the accumulation zone. In this region, a previously vegetated feature on the right side of the outflow channel now appears uncovered and larger.

Concerning the minor features on the left side of the channel, only the rims of the source zones and their related scarps are visible.

1991 - In 1991, the landslide covered an area like that in 1985. Recent viscous phenomena are visible along the left flank of the source area, extending to an area of up to 10,000 m². The landslide drainage network appears more developed than in 1945, 1976, and 1985, featuring a series of landslide ponds along the main channel. The channel area remains similar to 1985, although a new viscous phenomenon exceeding 15,000 m² has formed. The accumulation zone is also comparable to 1985 but shows a notable increase in vegetation cover, including trees.

External features beyond the central landslide bodies are no longer recognizable, except for that upslope of the source area, especially on the left side of the outflow channel, which appear more developed and display translational features compared to 1985. Similarly, downstream of the accumulation zone, the phenomenon on the right side of the channel seems more extensive.

2003 - The landslide exhibits morphological complexity similar to that observed in 1991 (Fig. 4.3), but not in extent, as an apparent retrogression is evident in the source area, with its rim reaching an elevation of 825 m a.s.l. Numerous minor features, both recent and newly formed, are present within the source area; the recent ones are, in some cases, covered by shrub vegetation, upon which the newly formed features tend to develop. In the upper part of this area, characterized by an amphitheater-like structure, landslide accumulations begin to form in the lower sector, where the slope gradient decreases. The source lobes, located around 700 m a.s.l., are covered with arboreal and shrub vegetation, indicating partial and temporary stability.

At the same elevation, where the landslide channel begins, longitudinal shear interfaces are observed, separating materials moving at different speeds. The channel area shows no substantial differences from 1991, although intense cultivation of the landslide materials prevents full recognition of the minor features from that year.

Nonetheless, a drainage network with a series of ponds exists; these ponds are currently empty, likely due to seasonal conditions. The accumulation area remains similar to 1991, with even more extensive coverage of arboreal and shrub vegetation.

Finally, regarding minor phenomena outside the main landslide body, recent features are common on the right flank of the mid- to upper-channel area, and the phenomenon downstream of the accumulation area remains visible.

2006 - The material channels within the small valley located downslope of the accumulation area, observed in 1991, extend to the foot of the slope and spread out in a fan shape.

During its descent, it also incorporates material mobilized by the viscous phenomenon seen in 1991 below the accumulation area.

The source area, affected by prolonged periods of intense rainfall, is influenced by numerous minor phenomena related to the ongoing retrogression process; many translational movements of soil rafts occur near the rim of the source area, resulting in a highly irregular morphology. Along the scarp, many viscous phenomena arise, leading to the formation of new deposits that overlay existing ones.

In this zone, at the base of the previously mentioned amphitheater-shaped morphology, some landslide ponds form, fed by springs.

Further downslope, still within the source area, multiple longitudinal shear interfaces develop, along which relative movement occurs between materials moving at different downhill velocities.

At approximately 700 m a. s. l. A series of recent flow lobes is observed, attributed to precipitation-induced material mobilization. In the lower channel area, longitudinal interfaces also separate materials with different water contents, with the more water-rich material moving downslope faster. Mid-channel lobes are still in the same positions as in 1991.

Several counter-slope areas are present, corresponding to locations where landslide ponds have formed.

The accumulation area develops within the incised valley immediately downslope of the 1991 deposit, where the material reaches an approximate thickness of 10 meters (geometrically measured) and spreads to the foot of the slope in a fan-shaped pattern.

On April 26th, 2006, for the first time, the earth flow cut through the east-west trending Italian national road (Fig. 3 .4), which connects the provinces of Foggia and Avellino on the east and west coasts, damaging some buildings, and stopping approximately 30 meters from the Benevento- Foggia national railroad.

2007 - The landslide exhibits a similar level of complexity and extent to that in 2006 (Fig. 4.4).

In the source area, the viscous phenomena observed in 2006 tend to overlap with those at the center of the previously described amphitheater-shaped morphology, causing deposits associated with translational phenomena to flow immediately downslope and fragment into smaller blocks.

Further downslope, on the left flank of the source area, newly formed viscous flow lobes are observed. Upstream of this zone, outside the main landslide body, numerous wide fractures aligned north-south are visible. Near 700 m a.s.l., the viscous flow lobes identified in 2006 are still present. New longitudinal shear interfaces are visible, along which material moves downslope at different speeds.

The channel area appears similar to that in 2006, except for recent viscous flow lobes in slightly different positions. In this region as well, material moves downslope with varying velocities, creating longitudinal movement interfaces similar to those in 2006. Finally, the accumulation area shows a similar extent and features a morphology stabilized by civil protection efforts: the lower deposit section is re-profiling with a stepped structure.

2008 - The landslide shows no significant changes compared to 2007.

New viscous flow lobes are observed along the scarp, and the accumulations at 700 m a.s.l. are visible, with slightly altered shapes.

The flow lobes in the channel area and the landslide ponds remain distinguishable. The most notable differences include a new viscous-flow accumulation at 500 m a.s.l., which is the source lobe, and the absence of vegetation in the toe zone due to human removal.

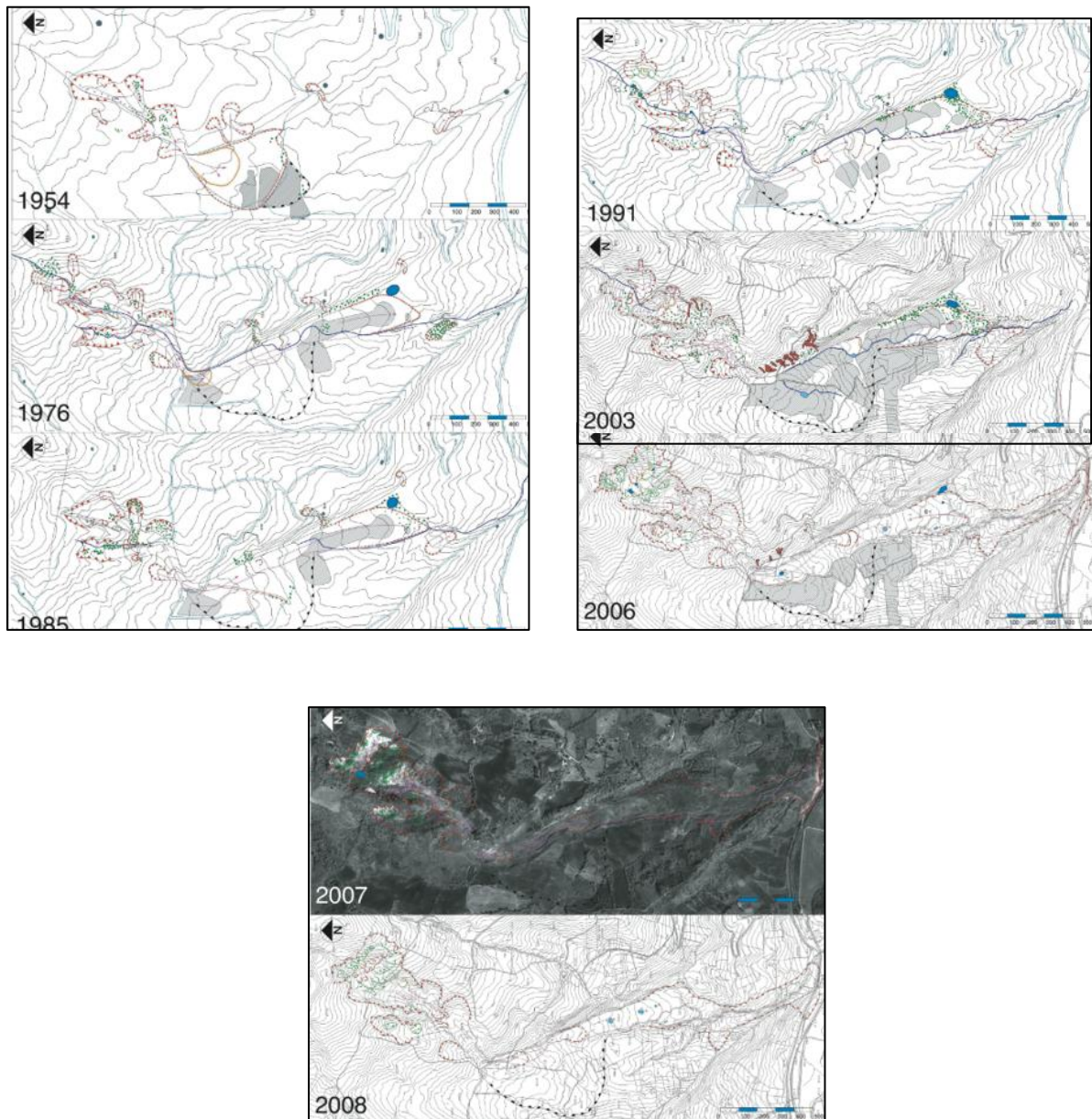


Figure 4.3 – Multi-temporal evolution of the Montaguto earthflow (Guerriero et al., 2013)

2009 - In July 2009, the morphological structures and hydrological features along the earth-flow body closely resembled those of 2007, except for some notable changes.

These changes included an expanded northeastern headscarp, a shallow internal landslide in the neck zone, the reformation of Lago delle Rane at its 2006 location, the destruction of the artificial drainage channel upslope of the toe, and a new failure surface in the southwestern toe area.

2010 - In June 2010 (Fig. 4.5), the morphological and hydrological features along the earth flow body closely resembled those in 2009. However, notable differences included a new pond in the western source area, the reformation of six ponds at their 2006 locations, and the development of an internal landslide headscarp, pond, and back-tilted surface on the eastern flank just south of UTM 4565800 N. Furthermore, toe expansion impacted both SS90 and the Benevento-Foggia railway. The 2010 reactivation was like the 2006 event, involving part of the earthflow toe, with over 300,000 m³ of material mobilized and displacement rates reaching 0.5 m/h (Guerriero et al., 2013).



Figure 4.4 – The Montaguto earth-flow on 27th April 2006.



Figure 4.5 - The Montaguto earth-flow on April 3, 2010

Chapter 5. Geomechanical and Rheological Parameterization of the Montaguto Earthflow Materials

The present geotechnical characterization is based on data derived from laboratory tests conducted at the Departmental Laboratory of Applied Geology and Monitoring of the University of Sannio. Both undisturbed and disturbed samples, specifically collected for this study, were analyzed. The results were further compared with the existing literature on formations outcropping in the Sannio-Daunia area (Guadagno et al., 2006).

Based on these investigations, lithological classifications, and a preliminary interpretation of the experimental results, the geotechnical and geomechanical properties of the rock masses in the landslide area are defined.

5.1. Characteristics of the FAE1 Member

Chapter 3 describes the geological setting and lithological features of the formations cropping out in the Montaguto area affected by the project. The lithotypes belong to the Faeto Flysch Formation, which is subdivided into three members characterized by distinct lithological features.

These successions consist of alternating layers and beds composed of clays, marly clays, marls, calcarenites, and calcisiltites, typical of a turbiditic depositional environment.

The resulting rock masses, derived from these lithologically heterogeneous successions, are highly tectonized and include structural bodies interconnected by various tectonic features, such as folds, normal and reverse faults, and thrust systems, reflecting the tectonic evolution of the Dauno Subapennine.

Before analysing the geological, technical, geotechnical, and geomechanical properties of these lithotypes, it is essential to emphasize their marked lithological complexity. This complexity significantly affects the determination of their overall mechanical behaviour, mainly due to the alternation of competent rock layers and weaker, more deformable materials.

Specifically, the successions may locally be dominated by either pelitic or more competent lithologies and are often characterized by dense alternations of thin layers and beds. Lithofacies commonly exhibit heteropic contacts.

These features are the result of brittle tectonic processes that have generated multiple families of discontinuities, including stylolitic surfaces, fracture cleavages of different scales, axial plane cleavages, joints, and faults.

Such discontinuities exert a strong control on the mechanical behaviour of the rock masses, particularly in proximity to major tectonic structures.

Based on these considerations, the formations described above must be classified as “lithologically and structurally complex” (*sensu* Esu, 1977; AA.VV., 1985; Picarelli, 1986). This condition is widely documented in the Campania Apennines (Guadagno et al., 2006; Revellino et al., 2010) and is especially relevant in the Montaguto area (Guerriero et al., 2014; Pinto et al., 2016).

For the present geotechnical characterization, data from the 2020 design investigations were integrated with results from supplementary investigations conducted in 2023 by Tecno-in on newly collected samples. Additional laboratory tests were performed at the Departmental Laboratory of Applied Geology and Monitoring on both undisturbed and disturbed samples collected specifically for verification and integration purposes.

All datasets were compared with literature data referring to the same geological formations in the Sannio-Daunia sector (Guadagno et al., 2006).

The investigations were conducted under a cooperation agreement between the Department of Science and Technology (DST) of the University of Sannio and ERG WIND S.r.l.

5.2. Laboratory Testing

In June–July 2023, in situ investigations included continuous core drilling to reconstruct the subsurface stratigraphy and evaluate the geotechnical properties of the materials within the study area.

The collected samples were subjected to detailed visual inspection and laboratory testing to determine their primary physical and mechanical properties. These data provided the basis for developing the geological and geotechnical reference models.

Figure 5.1 shows the location of the boreholes, which reached depths of up to 35 m below ground level. These investigations allowed the identification of the main lithological variations and stratigraphic sequences relevant to the geotechnical analysis.

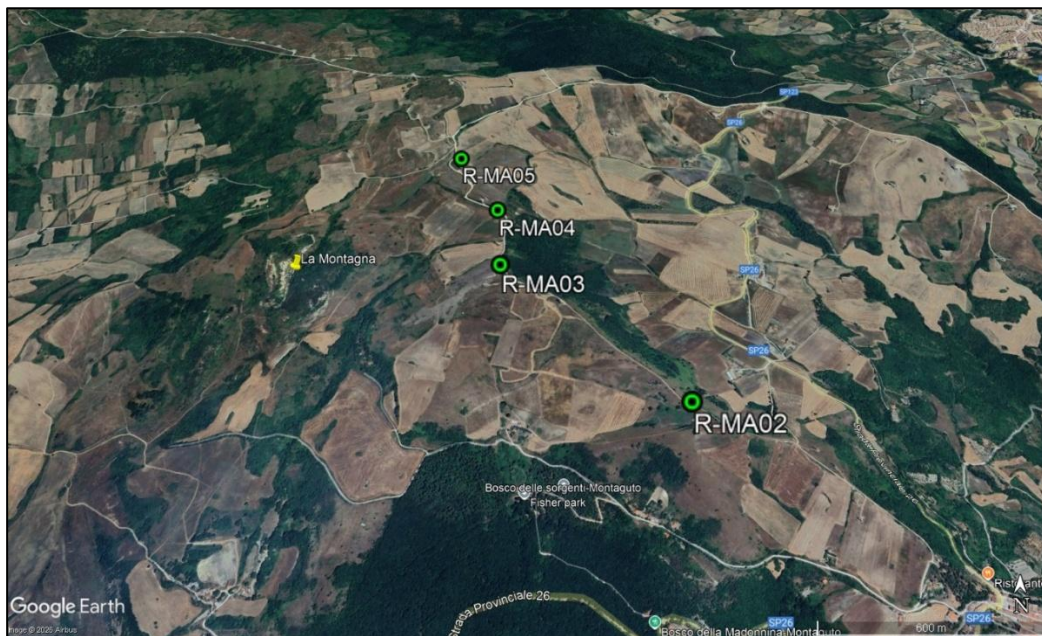


Figure 5.1 - Map of borehole locations within the study area

The following laboratory tests were performed on the collected samples:

INDEX PROPERTIES:

- Natural water content (w);
- Specific gravity of solids (Gs);
- Bulk and dry unit weight (γ , γ_d);
- Particle size distribution (sieve and hydrometer analysis);
- Atterberg limits (Liquid Limit, LL; Plastic Limit, PL) and consistency indices.

MECHANICAL PROPERTIES:

- Compressibility characteristics (oedometer tests);
- Residual shear strength parameters.

5.3. Experimental activity

Following the general overview of the in-situ samples, this section presents the experimental work performed firsthand.

Detailed analysis is provided for the three samples, RMA02 (sampled at depths of 1.50-2.00 m and 3.50-4.00 m, respectively) and RMA03 (7.00-7.50 m) - University of Sannio) analysed directly by the author.

This method combines previous data with precise geotechnical parameters obtained through targeted laboratory tests.

The analysis of the representative samples (Figures 5.1 – 5.2 – 5.4) collected during fieldwork was conducted at the Applied Geology Laboratory of the University of Sannio after thorough oven-drying.

The samples were subjected to the following laboratory tests:

Index properties:

- Natural water content (w): UNI EN ISO 17892-1;
- Specific gravity of soil solid (G_s): UNI EN ISO 17892-2;
- Bulk and dry unit weights (γ , γ_d): UNI EN ISO 17892-3;
- Particle size distribution (via sieving and hydrometer analysis): UNI EN ISO 17892-4
- Atterberg limits (Liquid Limit, LL, and Plastic Limit, PL) and related consistency indices: UNI EN ISO 17892-12.



Figure 5.2 - Sample RMA02, collected at a depth of 1.5-2.00 meters.



Figure 5.3 - Sample RMA02, collected at a depth of 3.5-4.00 meters.



Figure 5.4 - Sample RMA02, collected at a depth of 7.00 - 7.50 meters.

5.3.1. Water content

Determining the natural water content (w) of a soil sample is a fundamental laboratory procedure. This process involves weighing the specimen in its undisturbed state (as-sampled) and then oven-drying it and weighing it again.

The water content is defined as the ratio of the weight of water to the weight of the dry solids, expressed as a percentage.

In geotechnical engineering, water content is a critical parameter for assessing soil compressibility and shear strength.

In this study, measurements were taken immediately after sampling, and the specimens were carefully sealed during transport to the laboratory to prevent moisture loss through evaporation, thereby ensuring the accuracy and reliability of the data.

5.3.2. Specific Gravity of Solids (G_s)

The specific gravity of soil solids (G_s) is defined as the ratio of the unit weight of the solid particles to the unit weight of distilled water at 4°C. It is the ratio of the mass of a given volume of solids to the mass of an equal volume of gas-free distilled water. This dimensionless parameter is essential for calculating weight-volume relationships, such as void ratio and degree of saturation.

$$G_s = \frac{W_s}{V_s \cdot \gamma_w}$$

where:

- G_s is the specific gravity of soil solids (dimensionless);
- W_s is the weight (or mass) of the oven-dried soil solids;
- V_s is the volume occupied by the soil solids;
- γ_w the unit weight of water at 4°C (typically taken as 9.81 kN/m³ or 1.0 g/cm³).

Because laboratory testing temperatures are typically above 4°C, current standards allow using water density at 20°C as a reference. This approach results in an error of less than 0.3%, which is considered negligible for geotechnical purposes.

The specific gravity (G_s) was determined using the pycnometer method. The soil was first oven-dried at 110°C for 24 hours and then ground to a fine powder in a ceramic mortar. A specimen of approximately 20 g (W_1) was weighed and prepared for testing. A 100 mL glass pycnometer was filled with distilled water to the graduation mark (aligned with the base of the meniscus) and weighed (W_2).

After emptying the pycnometer, the pre-weighed soil specimen was introduced, and the flask was partially refilled with distilled water.

To remove any air trapped in the intergranular spaces, the pycnometer was connected to a vacuum pump.

Once de-airing was complete, the pycnometer was filled to the mark and weighed again (W3). The specific gravity, defined as the ratio of the unit weight of the soil solids to that of water, was then calculated using the following formula:

$$G_s = \frac{W_1}{W_2 - (W_3 - W_1)}$$

where:

- G_s = specific gravity of soil solids;
- W_1 = mass of the oven-dried soil specimen;
- W_2 = mass of the pycnometer filled with water;
- W_3 = mass of the pycnometer containing both soil and water.

The final value was obtained by applying a correction factor based on the water density at the measured temperature, expressed in g/cm³.

5.3.3 Determination of Bulk and Dry Unit Weights

The natural bulk unit weight and the dry unit weight are determined by measuring the weight of a representative soil specimen of known volume.

These parameters are defined by the following relationships:

$$\gamma_n = \frac{W}{V}$$

where:

- γ = bulk unit weight of the soil;
- W = total weight of the soil specimen (solids + water);
- V = total volume of the specimen.

$$\gamma_d = \frac{W_s}{V}$$

where:

- γ_d = dry unit weight of the soil;
- W_s = weight of the oven-dried soil solids;
- V = total volume of the specimen.

The bulk unit weight was determined using the cylindrical cutter method. The sampler's height and diameter were measured with a digital caliper to ensure accuracy. The weight of the empty cutter was also recorded. The cylinder's internal volume was then calculated using the following geometric formula:

$$V_f = \pi r^2 * h$$

where:

- V_f = volume of the cylindrical cutter (cm^3); r = radius of the cutter (cm);
- h = height of the cutter (cm).

The cylindrical cutter was then manually driven into the undisturbed soil sample, taking care to avoid creating voids. The edges were leveled with a spatula, leaving the cutter filled with soil. The filled cutter was then weighed, and the natural bulk unit weight (γ) was calculated by subtracting the weight of the empty cutter (W_1) from the weight of the filled cutter (W_2), and dividing the result by its volume (V):

$$\gamma = \frac{W_2 - W_1}{V}$$

where:

- γ_n = natural bulk unit weight (g/cm^3)
- W_1 = mass of the empty cylindrical cutter (g);
- W_2 = mass of the cutter containing the moist soil specimen (g);
- V = internal volume of the cutter (cm^3).

After measuring the bulk unit weight, the water content (w) of each specimen was determined using the methodology described in the previous section. Finally, the dry unit weight (γ_d) was derived from the relationship between the bulk unit weight and the moisture content.

5.3.4. Particle Size Distribution Analysis

The objective of the particle-size analysis is to determine the distribution of grain sizes in the soil samples. Given the predominantly fine-grained nature of the material, the investigation was conducted using a combined approach:

- Wet sieving: utilized for the coarse-grained fraction (particles with a diameter $d > 0.075$ mm, retained on the No. 200 sieve);
- Sedimentation analysis (Hydrometer test): employed for the fine-grained fraction (particles with a diameter $d < 0.075$ mm).

The laboratory data were processed to construct the particle-size distribution (grading) curve for the material under investigation.

This distribution is plotted on a semi-logarithmic chart, with particle diameter on the abscissa (log scale) and cumulative percentage passing on the ordinate (linear scale).

To categorize soil fractions, this study uses the classification system proposed by the AGI (Italian Geotechnical Association, 1990), which defines boundaries for clay, silt, sand, and gravel by specific diameter ranges.

This system classifies soil particle sizes into the following categories based on particle diameter (d) (see Table 1):

Table 1. Soil fractions according to AGI (1990) classification

Soil Fraction	Particle Diameter Range (mm)
Clay	$d < 0.002$
Silt	$0.002 \leq d < 0.06$
Sand	$0.06 \leq d < 2.0$
Gravel	$2.0 \leq d < 60.0$
Cobbles	$d > 60.0$

5.3.5. Test procedure

For the wet-sieving analysis, a representative oven-dried soil sample of approximately 60 g was used.

To ensure complete disaggregation of soil clusters, the sample was gently processed in a ceramic mortar. The material was then placed in a sieve stack with the following aperture sizes: 2 mm, 1 mm, 0.5 mm, 0.25 mm, 0.125 mm, and 0.075 mm (No. 200 sieve).

The stack was mounted on a mechanical shaker specifically configured for wet sieving. During the process, distilled water was introduced through the upper inlet while the apparatus operated at its maximum oscillation frequency.

The outflowing water was collected from the lower outlet, and its clarity was continuously monitored; the procedure was considered complete when the effluent appeared visually clear, typically after about 4 minutes.

After rinsing, each sieve containing the retained fraction was placed in a drying oven at 110 °C for 24 hours. After drying, the soil retained on each sieve was weighed on a precision balance.

These mass values were subsequently converted into a percentage retained relative to the initial total dry mass using the following formula:

$$\% \text{ retained} = \frac{\text{mass retained (g)}}{\text{mass of sample (g)}} \times 100$$

The percentage retained was subsequently converted into the cumulative percentage passing, which serves as the fundamental parameter for plotting the particle-size distribution curve. This value was calculated as the difference between 100% (representing the total initial dry mass) and the cumulative percentage of soil retained on each sieve.

5.3.6. Sedimentation Analysis (Hydrometer Test)

The sedimentation test is used to determine the particle-size distribution of the soil fraction finer than 0.075 mm. Rather than direct measurement, this method estimates particle diameters by monitoring the settling rate of soil particles in a distilled-water suspension in a 1000 mL graduated cylinder.

To determine the equivalent diameter of silt and clay particles, Stokes' Law (1851) is applied. This physical law assumes each particle is a perfect sphere and relates its terminal settling velocity to its diameter, based on the fluid's density and dynamic viscosity.

The relationship allows the measured settlement depth and time to be correlated with the corresponding particle size.

$$D = \sqrt{\frac{1800 \cdot \eta_L \cdot V}{\gamma_s - \gamma_L}}$$

where:

- D = diameter of the sphere (mm)
- μ = dynamic viscosity of the fluid (g·s/cm²)
- ρ_p = specific weight (density) of the particle (g/cm³)
- ρ_f = density of the fluid (g/cm³)
- V = settling velocity of the sphere (cm/s)

5.3.7. Test procedure

A representative 60 g sample of oven-dried soil, consisting entirely of the fraction passing the 0.075 mm sieve, was prepared for the sedimentation test.

The specimen was prepared from oven-dried material, gently ground in a mortar, and pre-screened by dry sieving. The soil was then added to 125 mL of a sodium hexametaphosphate solution. This chemical acts as a dispersant (or deflocculant), neutralizing surface charges to prevent particle aggregation and ensuring that individual grains remain uniformly dispersed.

The specimen was allowed to soak in the solution for 24 hours to ensure complete hydration. After this period, the mixture was agitated in a mechanical laboratory mixer for 15 minutes. The resulting suspension was then transferred to a 1000 mL graduated glass cylinder and filled to the mark with distilled water. To achieve maximum particle dispersion, the cylinder was inverted by hand 30 times within 1 minute immediately prior to the start of the test.

Initially, a soil hydrometer was carefully inserted, and the first density reading was taken after 30 seconds. Follow-up measurements were recorded at 1, 2, 5, 15, 30, 60, 120, 240, 480, and 1440 minutes. Each reading was used to calculate an equivalent particle diameter based on Stokes' Law. Finally, the percentage passing from the sedimentation test was normalized and combined with sieve analysis data to produce a continuous particle-size distribution curve.

5.3.8. Determination of Atterberg Limits

Grain size analysis provides quantitative information on the particle-size distribution of a soil; however, it does not provide insights into the characteristics of the clay minerals present. In cohesive soils, these minerals play a crucial role, significantly affecting the material's physical and mechanical behaviour. Specifically, for clays, this behaviour is strongly influenced by the presence of water, which is adsorbed onto the individual particles, forming bonds of varying strength.

5.3.9. Determination of the Liquid Limit (LL)

The Liquid Limit (LL) is defined as the gravimetric water content at which a soil specimen transitions from a plastic to a liquid state. In accordance with standardized procedures, the soil is placed in a Casagrande brass cup and bisected using a calibrated grooving tool. The limit is specifically identified as the moisture content required for the two halves of the soil pat to flow together and close the groove over 13 mm, following 25 controlled drops from a standardized height of 10 mm.

According to this definition, determining the liquid limit directly can be challenging because it is difficult to achieve groove closure of exactly 13 mm on the 25th blow.

However, plotting water content (on a linear scale) against blow count (on a logarithmic scale) yields an approximately linear relationship for blow counts between 15 and 35.

Therefore, the test involves preparing multiple samples with varying water contents and measuring the number of blows required to close the groove under each condition. From the resulting graph, the liquid limit can be interpolated as the water content corresponding to 25 blows.

5.3.10. Determination of the Plastic Limit (WP)

The plastic limit is the water content at which a soil begins to lose its plastic behavior. In laboratory conditions, it is determined by manually rolling a small thread of soil until it reaches a diameter of 3.2 mm. At this stage, the appearance of cracks indicates a loss of plasticity due to reduced moisture.

5.4. Lithostratigraphic Setting and Physical Properties of the FAE1 Member

The landslide body develops within the calcareous–marly–clayey member (FAE1) of the Faeto Flysch Formation. The deformed sequences involved in the mass movement can be distinguished into three main lithofacies: FAE1a, FAE1b, and FAE1c.

Within these units, lithic components are often well represented and play a primary role in controlling the formation's overall geomechanical behaviour.

The physical, volumetric, and index properties of the pelitic levels derived from site investigations are summarized in Tables 2 and 3.

These parameters were obtained from laboratory analyses carried out on:

- Disturbed samples collected during Standard Penetration Tests (SPT);
- Undisturbed samples retrieved for advanced geotechnical characterization.

Table 2. Physical and volumetric properties of samples collected in the Montaguto area

Sample ID	Depth		Bulk density (KN/m ³)	Dry density (KN/m ³)	Water content (%)	Specific gravity of solids G _s	Porosity (%)	Void ratio (-)	Degree of saturation (%)
	from	to							
R-MA02-1_C10	2,00	2,50	18,65	16,02	16,44	2,57	37,79	0,61	69,68
R-MA02-1_SPT1	6,00	6,30							
R-MA02-1_SPT2	7,50	7,95							
R-MA02-1_SPT3	9,00	9,45							
R-MA02-1_SPT4	12,00	12,45							
R-MA02-1_CL1	14,00	14,30	21,09						
R-MA02-1_C2	15,00	15,50							
R-MA02-1_CR5	23,00	23,30	19,13	16,58	15,34	2,68	38,18	0,62	66,61
R-MA02-1_CR6	25,00	26,00	17,29	15,02	15,10	2,60	42,15	0,73	53,80
R-MA02-1_C3	30,10	30,70							
R-MA02-1_CR7	32,70	33,00	19,86	17,51	13,42	2,70	35,18	0,54	66,79
R-MA02-1_CR8	37,40	37,70		9,91	2,73				
R-MA02-2_SPT1	1,50	1,95							
RMA02 - University of Sannio	1,50	2,00	19,20	15,20	26,00	2,60	37,00	0,51	68,20
RMA02 - University of Sannio	3,50	4,00	19,12	15,10	22,11	2,60	40,76	0,69	83,49
R-MA02-2_C11a	3,00	3,50	18,45	15,61	18,25	2,62	40,54	0,68	70,26
R-MA02-2_SPT2	7,50	7,95							
R-MA02-2_SPT3	9,00	9,45							
R-MA02-2_CR4	14,00	14,50	21,09	20,26	4,12	2,71	25,14	0,37	33,21
R-MA02-2_C2	16,00	16,60							
R-MA02-2_CL1	22,50	23,00	21,45						
R-MA02-2_CR5	26,00	26,50	19,45	17,62	10,41	2,70	34,67	0,53	52,91
R-MA02-2_CR6	28,70	29,30	21,64	19,32	12,03	2,71	28,59	0,40	81,31
R-MA03-1_CR1	1,20	1,60	18,67	15,99	16,79	2,52	36,46	0,57	73,64
R-MA03-1_SPT1	1,60	2,05							
R-MA03-1_SPT1	3,00	3,11							
R-MA03-1_CR2	5,00	5,00	17,80	16,91	5,24	2,69	37,07	0,59	23,91
R-MA03-1_SPT3	5,00	5,45							
R-MA03-1_CR3	6,50	6,70	18,30	16,32	12,10	2,66	38,53	0,63	51,26
R-MA03-1_CL1	8,10	8,30	26,25						
RMA03 - University of Sannio	7,00	7,50	19,40	16,96	14,13	2,60	33,46	0,50	72,97
R-MA03-1_CL2	12,70	12,90							
R-MA03-1_CL3	24,50	24,80	22,56						
R-MA03-1_CL4	25,50	25,70	24,54						
R-MA03-1_CL5	28,00	28,40	25,73						
R-MA03-1_CL6	32,80	33,00	25,59						
R-MA03-2_C1	2,20	2,40	23,75						
R-MA03-2_C2	5,50	6,00							
R-MA03-2_C1a	6,00	6,50							
R-MA03-2_CL3	9,50	10,00	21,39						
R-MA03-2_CR1	12,00	12,50	20,87			2,65			
R-MA03-2_C2	14,00	14,80	26,33						
R-MA03-2_CL4	16,00	16,40	23,65						
R-MA03-2_CL5	18,40	19,00	22,57						
R-MA03-2_CL6	22,00	22,70	23,59						
R-MA03-2_CL7	27,30	27,80	24,78						
R-MA04-CR01	3,40	4,00	17,18	13,72	25,24	2,52	45,48	0,83	76,14
R-MA04-CR01	6,70	7,00	17,22	14,58	18,15	2,51	41,83	0,72	63,21
R-MA04-CL01	9,00	9,30	19,61						
R-MA04-CL02	14,60	14,90	23,62						
R-MA04-CL03	20,00	20,30	24,71						
R-MA04-CL04	26,00	26,30	20,88						
R-MA05-CL01	4,10	4,60	17,33	13,34	29,92	2,51	46,89	0,88	85,09
R-MA05-CL02	6,00	6,85	23,95						
R-MA05-CL03	14,30	14,50	24,05						
R-MA05-CL04	18,00	18,40							
R-MA05-CL05	34,60	35,00	22,08						

Table 3 – Geotechnical index properties of samples collected in the Montaguto area

Sample ID	Depth		Grain size distribution (%)					Atterberg limits (%)				
	from (m)	to (m)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)		WL	WP	IP	IC	IL
R-MA02-1_C11b	2,00	2,50	26,50	19,50	34,50	19,50	0,00	50,00	20,11	59,89	1,12	-0,12
R-MA02-1_SPT1	6,00	6,30	10,50	12,00	23,50	53,00	0,00					
R-MA02-1_SPT2	7,50	7,95	11,50	12,00	23,50	53,00	0,00					
R-MA02-1_SPT3	9,00	9,45	12,50		49,04	38,46	0,00					
R-MA02-1_SPT4	12,00	12,45	12,00		46,99	41,01	0,00					
R-MA02-1_CL1	14,00	14,30										
R-MA02-1_C2	15,10	15,50										
R-MA02-1_CR5	23,00	23,30	23,50	17,00	29,50	30,00	0,00	31,50	20,65	10,85	1,49	
R-MA02-1_CR6	25,70	26,00	39,00	22,50	18,00	20,50	0,00	42,20	25,64	16,56	1,64	
R-MA02-1_C3	30,10	30,70										
R-MA02-1_CR7	32,70	33,00	19,50	18,00	33,00	29,50	0,00	37,50	19,64	17,86	1,35	
R-MA02-1_CR8	37,40	37,70	10,00	10,50	23,00	56,50	0,00	n.d.	n.d.	n.d.		
R-MA02-2_SPT1	1,50	1,95	20,50	24,50	42,00	13,00	0,00					
RMA02 - University of Sannio	1,50	2,00		20,50	35,40	20,00	0,00	33,40	24,98	17,71	0,42	
RMA02 - University of Sannio	3,50	4,00	20,90	37,50	22,30	0,00		39,12	24,18	14,94	1,14	
R-MA02-2_C11a	3,00	3,50	21,00	21,50	37,50	20,00	0,00	44,25	25,56	18,69	1,39	-0,39
R-MA02-2_SPT2	7,50	7,95	9,50		22,72	58,28	9,50					
R-MA02-2_SPT3	9,00	9,45	9,50		15,97	74,53	0,00					
R-MA02-2_CR4	14,00	14,50	15,50	17,50	34,50	32,50	0,00	31,80	19,51	12,29	2,25	
R-MA02-2_C2	16,00	16,60										
R-MA02-2_CL1	22,50	23,00										
R-MA02-2_CR5	26,00	26,50	25,50	21,00	24,00	29,50	0,00	38,40	21,93	16,47	1,70	
R-MA02-2_CR6	28,70	29,30	37,00	29,00	27,50	6,50	0,00	35,00	21,83	13,17	1,74	
R-MA03-1_CR1	1,40	1,60	18,50	21,50	30,50	29,50	0,00	57,10	39,60	17,50		
R-MA03-1_SPT1	1,60	2,05	10,00		11,74	78,26	0,00					
R-MA03-1_SPT2	3,00	3,11	9,50		69,85	20,65	0,00					
R-MA03-1_CR2	4,50	5,00	31,00	35,50	29,00	4,50	0,00	47,75	29,86	17,89	2,38	
R-MA03-1_SPT3	5,00	5,45	13,00	11,50	45,00	30,50	0,00					
R-MA03-1_CR3	6,50	6,70	30,00	36,00	34,00	0,00	0,00	48,60	29,91	18,69		
R-MA03-1_CL1	8,10	8,30										
RMA03 - University of Sannio	7,00	7,50						45,73	28,02	17,71	1,78	
R-MA03-1_CL2	12,70	12,90										
R-MA03-1_CL3	24,50	24,80										
R-MA03-1_CL4	25,50	25,70										
R-MA03-1_CL5	28,00	28,40										
R-MA03-1_CL6	32,80	33,00										
R-MA03-2_CL1	2,00	2,40										
R-MA03-2_CL2	5,50	6,00										
R-MA03-2_C1a	6,00	6,50										
R-MA03-2_CL3	9,50	10,00										
R-MA03-2_CR1	12,00	12,50	19,00	35,00	40,50	5,50	0,00	36,30	23,46	12,84	1,73	
R-MA03-2_C2	14,00	14,80										
R-MA03-2_CL4	16,00	16,40										
R-MA03-2_CL5	18,40	19,00										
R-MA03-2_CL6	22,30	22,70										
R-MA03-2_CL7	27,50	27,80										
R-MA04-CI01	3,40	4,00	29,00	32,50	37,50	1,00	0,00	59,90	30,22	29,68	1,17	-0,17
R-MA04-CR01	6,70	7,00	33,50	36,50	28,50	1,50	0,00	52,00	36,39	15,61	2,17	
R-MA04-CL01	9,00	9,30										
R-MA04-CL02	14,60	14,90										
R-MA04-CL03	20,00	20,30										
R-MA04-CL04	26,00	26,40										
R-MA05-CI01	4,10	4,60	36,50	35,00	28,00	0,50	0,00	51,70	37,84	13,86	1,57	-0,57
R-MA05-CL01	6,00	6,85										
R-MA05-CL02	14,30	14,50										
R-MA05-CL03	18,00	18,40										
R-MA05-CL04	29,00	29,40										
R-MA05-CL05	34,60	35,00										

As shown in Figure 5.5, the pelitic layers exhibit moderate moisture conditions, with a degree of saturation (S_r) ranging between 30% and 80% (Fig. 5.6). The bulk unit weight (γ) varies between 17 and 21 kN/m³, while the water content (w) ranges from 13% to 24% (Fig. 5.7).

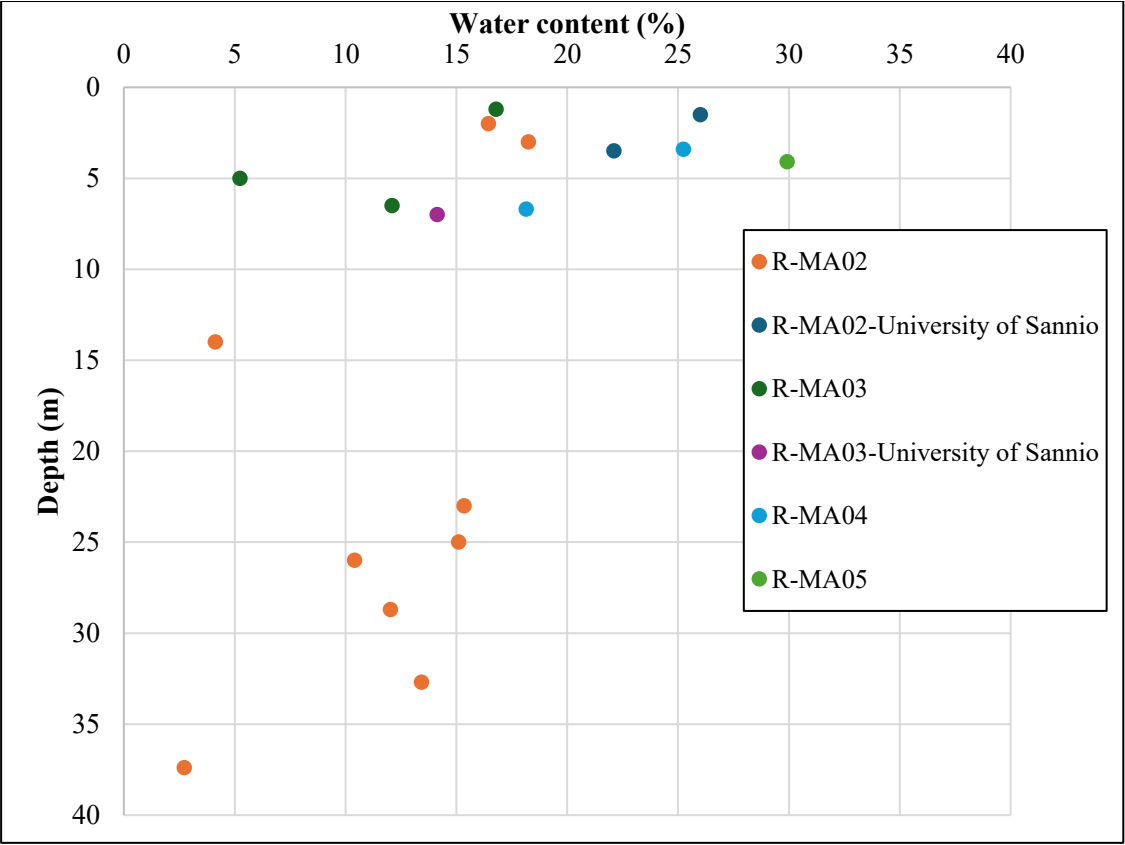


Figure 5.5 – Variation of water content with depth

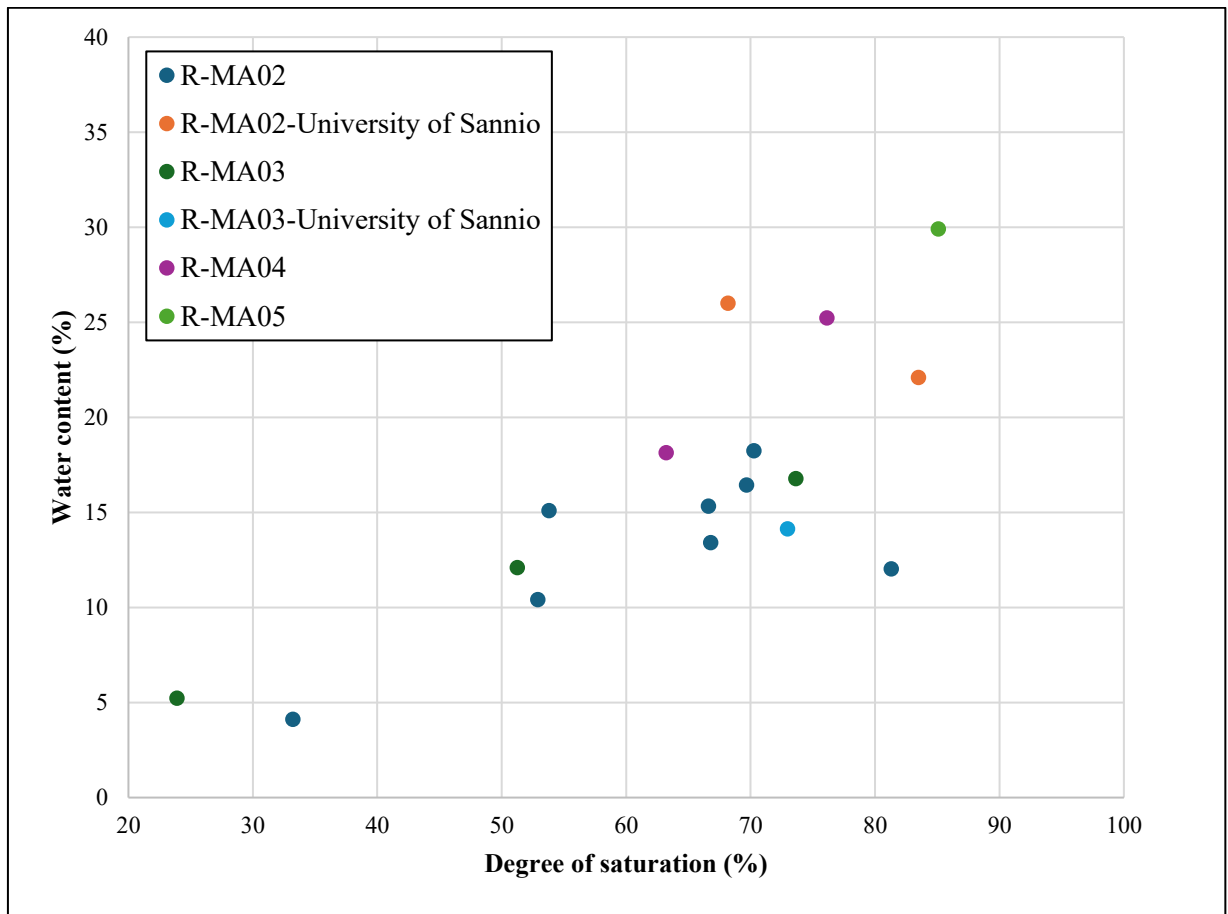


Figure 5.6 - Relationship between natural water content (w) and degree of saturation (S_r)

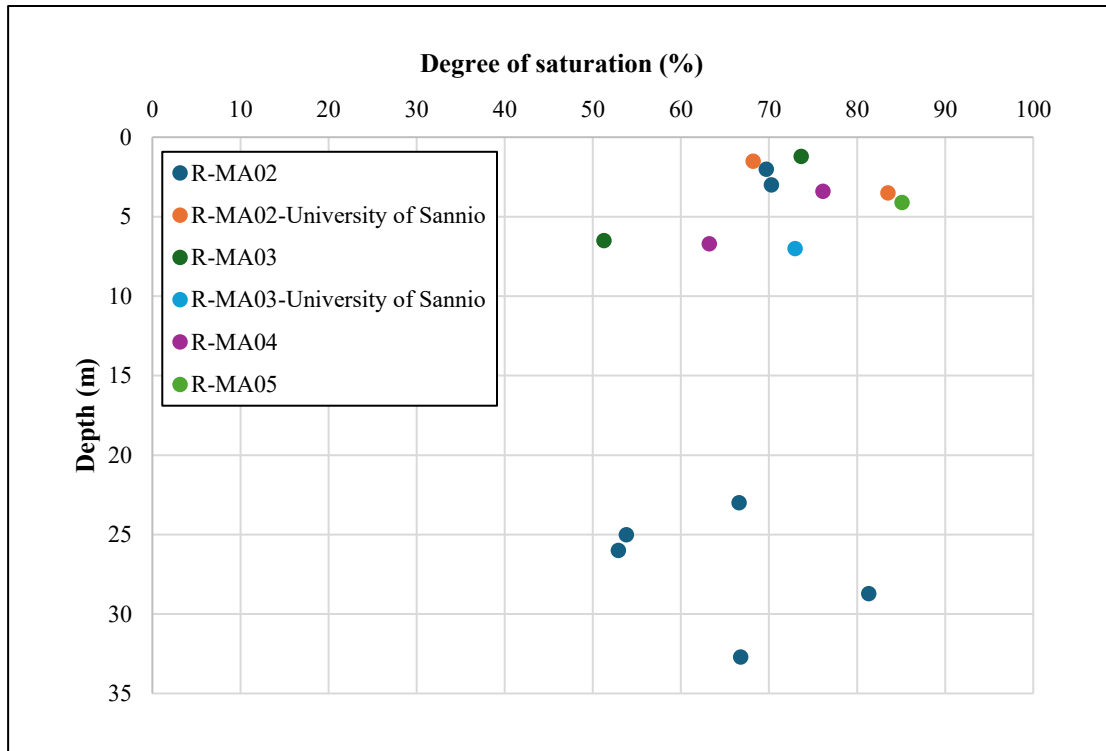


Figure 5.7 - Variation of the degree of saturation with depth

The clay fraction of the analysed samples ranges from 11% to 30%, as determined by standard ASTM procedures.

The Liquid Limit (LL) varies between 38 and 60, while the Plasticity Index (PI) ranges from 10 to 23. Based on these values, the materials can be classified as medium- to high-plasticity clays, generally exhibiting normal to inactive activity (Figs. 5.9 and 5.10).

It should be noted that prolonged weathering and softening processes may lead to an apparent increase in clay content.

The presence of sandy and gravelly fractions reflects the occurrence of arenaceous interlayers and laminae within the sequence.

Consequently, the consistency index (I_c) is consistently higher than 1.4 and often exceeds 1.9, indicating that the clayey materials are predominantly in a semi-solid to solid state.

A representative example is sample RMA-02 (University of Sannio), which exhibits a shrinkage limit of 15.3%, a value close to its natural water content, further supporting the interpretation of its material composition.

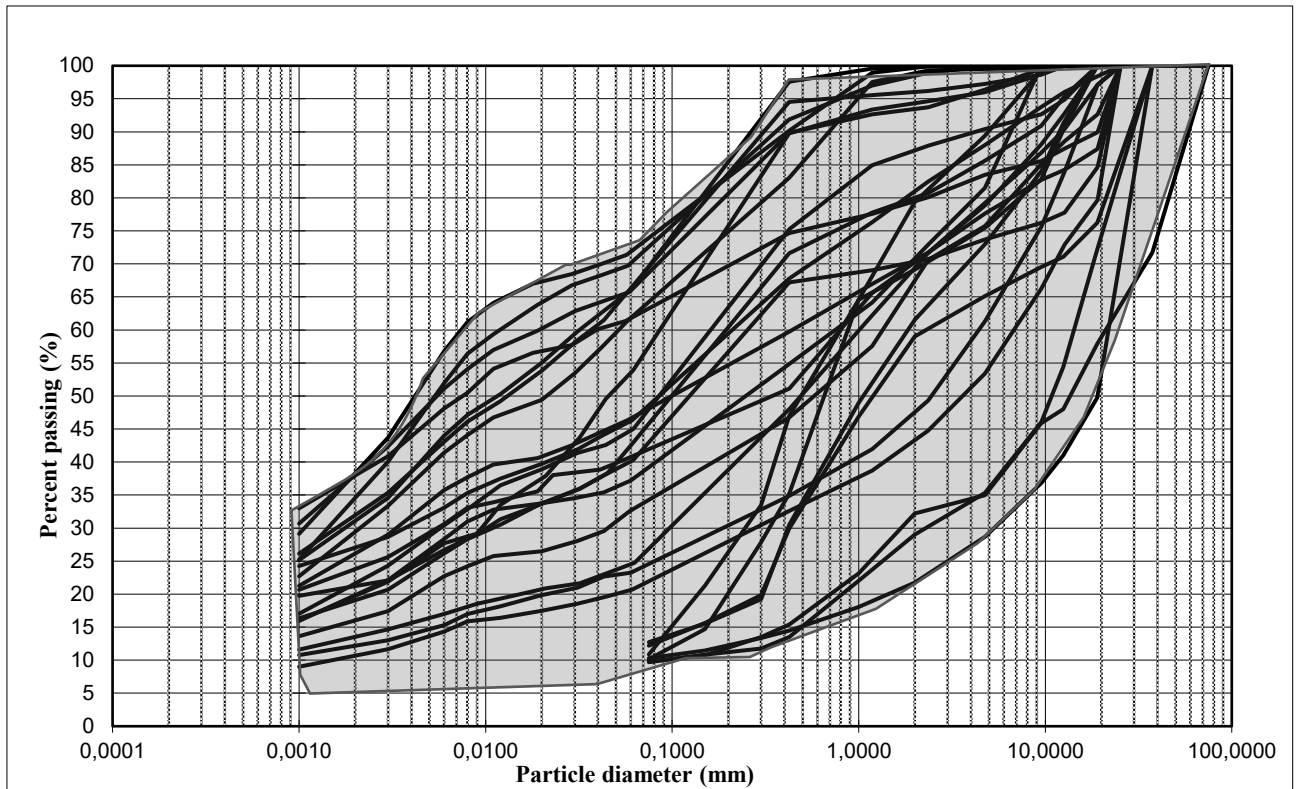


Figure 5.8 – Grain size distribution of analyzed samples

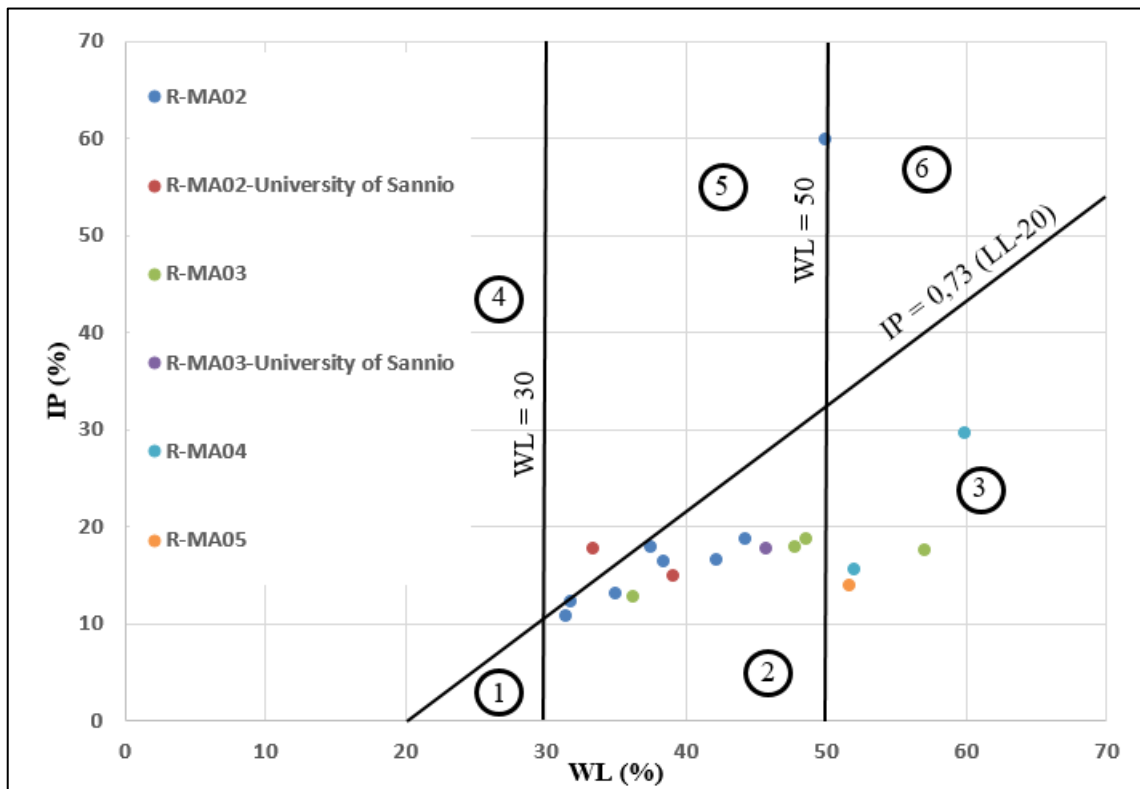


Figure 5.9 – Plasticity chart (Casagrande, 1948)

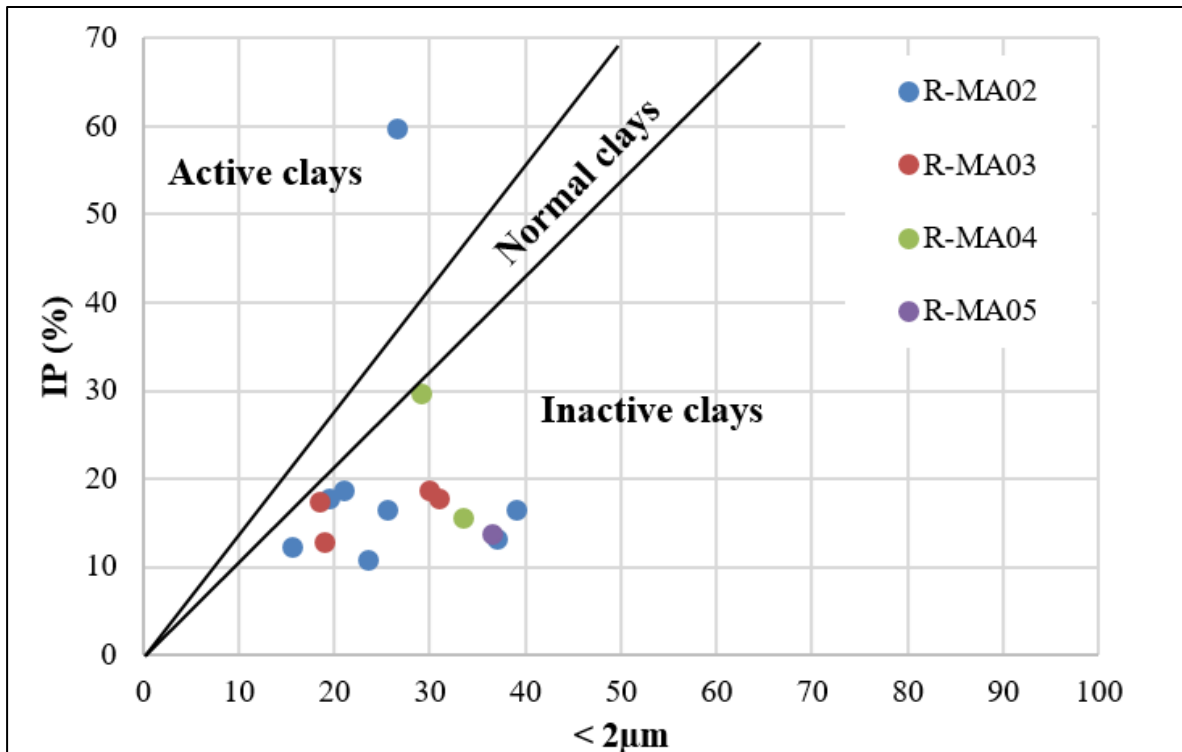


Figure 5.10 – Activity chart (Skempton, 1953)

Beyond their geological history, the significant degree of cementation and the quasi-rock behavior of these materials are primarily related to their high calcium carbonate content.

For instance, sample RMA-02 (University of Sannio) shows a carbonate content of approximately 44%.

Shear strength parameters were derived from direct shear tests performed on undisturbed specimens. Within a normal stress range of 150–400 kPa, the materials exhibit an apparent cohesion (c') of about 40 kPa and an internal friction angle (ϕ') of approximately 32° .

In contrast, triaxial compression tests on the same material yielded significantly lower strength parameters, with a friction angle of about 23° and negligible cohesion (ranging from 0.67 to 0.87 kPa).

This discrepancy highlights the complex mechanical behavior of marly–clayey sequences.

The intense tectonic deformation has generated a dense network of discontinuities and structural defects, which strongly control the overall mechanical response of the soil mass.

Direct shear tests performed on remoulded specimens at the liquid limit yielded friction angles between 23° and 25° , with a residual friction angle (ϕ'_r) of approximately 13° .

These values represent a key reference for evaluating the shear resistance mobilized along pelite-calcarenite interfaces (see Table 5 4).

Regarding the more competent lithologies, Uniaxial Compressive Strength (UCS) tests performed on core samples yielded values ranging from 22 to 69 MPa.

Point Load Tests (PLT) provided index values $Is_{(50)}$ ranging from 0.25 to 6.38 MPa, indicating significant variability in rock strength.

Table 5. Mechanical properties and shear strength parameters of the materials investigated

Sample ID	Depth (m)		Consolidated drained direct shear test (CD, peak values)		Consolidated drained direct shear test (CD, residual values)		Uniaxial compression strength on rock specimens	Tangent elastic modulus Et (at 50% of failure load)	secant elastic modulus Es at 50% of the failure load (GPa).	Poisson's ratio.	Point load strength index
	da	a	Friction ϕ°	Coesion c (kPa)	Friction ϕ°	Coesion c (kPa)	s (MPa)	Et (GPa)	Es (GPa)	n (-)	Is ₅₀
R-MA02-1 CI1b	2,00	2,50									
R-MA02-1 SPT1	6,00	6,30	32,68	25,09	23,94	0,87					
R-MA02-1 SPT2	7,50	7,95									
R-MA02-1 SPT3	9,00	9,45									
R-MA02-1 SPT4	12,00	12,45									
R-MA02-1 CL1	14,00	14,30									
R-MA02-1 C2	15,10	15,50					1,50				
R-MA02-1 CR5	23,00	23,30									6,38
R-MA02-1 CR6	25,70	26,00									
R-MA02-1 C3	30,10	30,70									
R-MA02-1 CR7	32,70	33,00									0,25
R-MA02-1 CR8	37,40	37,70									
R-MA02-2 SPT1	1,50	1,95									
R-MA02-2 CI1a	3,00	3,50									
R-MA02-2 SPT2	7,50	7,95	32,98	22,60	23,75	0,67					
R-MA02-2 SPT3	9,00	9,45									
R-MA02-2 CR4	14,00	14,50									
R-MA02-2 C2	16,00	16,60									
R-MA02-2 CL1	22,50	23,00									
R-MA02-2 CR5	26,00	26,50					0,80				
R-MA02-2 CR6	28,70	29,30									
R-MA03-1 CR1	1,40	1,60									
R-MA03-1 SPT1	1,60	2,05									
R-MA03-1 SPT2	3,00	3,11									
R-MA03-1 CR2	4,50	5,00									
R-MA03-1 SPT3	5,00	5,45									
R-MA03-1 CR3	6,50	6,70									
R-MA03-1 CL1	8,10	8,30									
R-MA03-1 CL2	12,70	12,90					42,30				
R-MA03-1 CL3	24,50	24,80									0,12
R-MA03-1 CL4	25,50	25,70									0,22
R-MA03-1 CL5	28,00	28,40					3,50				
R-MA03-1 CL6	32,80	33,00					39,90				
R-MA03-2 CL1	2,00	2,40					69,21	51,55	43,93	0,31	
R-MA03-2 CL2	5,50	6,00					49,72	35,53	39,08	0,34	
R-MA03-2 CI1a	6,00	6,50									0,07
R-MA03-2 CL3	9,50	10,00									0,19
R-MA03-2 CR1	12,00	12,50									0,19
R-MA03-2 C2	14,00	14,80									
R-MA03-2 CL4	16,00	16,40					44,30				
R-MA03-2 CL5	18,40	19,00					22,21	16,98	18,96	0,43	
R-MA03-2 CL6	22,30	22,70									0,12
R-MA03-2 CL7	27,50	27,80					33,87	20,73	38,12	0,41	
R-MA04-CI01	3,40	4,00					37,50	20,85	35,56	0,39	
R-MA04-CR01	6,70	7,00	31,40	25,57	23,27	0,60					
R-MA04-CL01	9,00	9,30	26,95	31,03	19,48	0,36					
R-MA04-CL02	14,60	14,90					9,20				
R-MA04-CL03	20,00	20,30					13,30				
R-MA04-CL04	26,00	26,30					76,44	35,17	33,20	0,29	
R-MA05-CI01	4,10	4,60					5,30				
R-MA05-CL01	6,00	6,85	25,04	37,76	18,78	0,82					
R-MA05-CL02	14,30	14,50					50,10				
R-MA05-CL03	18,00	18,40					24,50				
R-MA05-CL04	29,00	29,40									1,32
R-MA05-CL05	34,60	35,00									0,45

As previously established, the study area is characterized by lithostructurally complex successions. From a geomechanical standpoint, these sequences are classified as Class B1 rock units according to the AGI (1979) classification.

To identify the most representative geomechanical parameters and shear strength characteristics, a multi-methodological approach was used. Since the rock mass mainly consists of weak marly and calcareous units, an overall Rock Quality Designation (RQD) of 40–50% was estimated. Additionally, applying the Bieniawski (1989) Rock Mass Rating (RMR) system (Fig. 6.8) produced values between 52 and 60 (Table 4), classifying the formation as a "Fair" quality rock mass.

Parameter			Range of values					
1	Strength of intact rock material	Point-load strength index	>10MPa	4 – 10MPa	2 – 4MPa	1 – 2MPa	For this low range – uniaxial compressive test is preferred	
		Uniaxial compression strength	>250MPa	100 – 250MPa	50 – 100MPa	25 – 50MPa	5-25 MPa	1-5 MPa
	Rating	15	12	7	4	2	1	0
2	Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%	
	Rating		20	17	13	8	3	
3	Spacing of discontinuities		>2m	0.6 – 2m	200 – 600mm	60 – 200mm	<60mm	
	Rating		20	15	10	8	5	
4	Condition of discontinuities (See E)		Very rough surfaces	Slightly rough surfaces	Slightly rough surfaces	Slickensided surfaces or gouge <5mm thick or Separation 1-5mm continuous	Soft gouge >5mm thick or Separation >5mm continuous	
			Not continuous	Separation <1mm	Separation <1mm			
	Rating		30	25	20	10	0	
5	Ground water	Inflow per 10m tunnel length (l/m)	None	<10	10 - 25	25 – 125	>125	
		(Joint water press)/ (major principal σ)	0	<0.1	0.1 – 0.2	0.2 – 0.5	>0.5	
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing	
	Rating		15	10	7	4	0	
B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)								
Strike and dip orientations			Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable	
Ratings	Tunnels and mines		0	-2	-5	-10	-12	
	Foundations		0	-2	-7	-15	-25	
	Slopes		0	-5	-25	-50	-60	
C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS								
Rating			100 ← 81	80 ← 61	60 ← 41	40 ← 21	<21	
Class number			I	II	III	IV	V	
Description			Very good rock	Good rock	Fair rock	Poor rock	Very poor rock	
D. MEANING OF ROCK CLASSES								
Class number			I	II	III	IV	V	
Average stand-up time			20 yrs for 15m span	1 year for 10m span	1 week for 5m span	10 hrs for 2.5m span	30 min for 1m span	
Cohesion of rock mass (kPa)			>400	300 – 400	200 – 300	100 – 200	<100	
Friction angle of rock mass (deg)			>45	35 – 45	25 – 35	15 – 25	<15	
E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY CONDITIONS								
Discontinuity length (persistence)			<1m	1.3m	3 – 10m	10 – 20m	>20m	
Rating			6	4	2	1	0	
Separation (aperture)			None	<0.1mm	0.1 – 1.0mm	1 – 5mm	>5mm	
Rating			6	5	4	1	0	
Roughness			Very rough	Rough	Slightly rough	Smooth	Slickensided	
Rating			6	5	3	1	0	
Infiling (gouge)			None	Hard filling <5mm	Hard filling >5mm	Soft filling <5mm	Soft filling >5mm	
Rating			6	4	2	2	0	
Weathering			Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed	
Rating			6	5	3	1	0	

Figure 5.8 - Rock mass classification RMR system (Bieniawski, 1989)

Table 4. Rock Mass Rating (RMR) results (Bieniawski, 1989) for the Montaguto area

Index	Value	
	C ₁	C ₂
A1 – Point load index	7	7
A2 – RQD (%)	8	13
A3 – Discontinuing spacing	5	8
A4 – Discontinuity condition	25	25
A5 – Hydraulic condition of discontinuity	7	7
Base RMR	52	60
Orientation of discontinuities (slope)	-25	-25
RMR class	27 (IV)	33 (IV)
Orientation of discontinuities (foundation)	-7	-7
RMR class	47 (III)	55 (III)

In the slope stability analysis, the rock mass is classified as Class IV (Poor Rock) according to the adopted classification system. This classification corresponds to an estimated cohesion (c) range of 100–200 kPa and a friction angle (ϕ) between 15° and 25°.

The determination of this reference geotechnical and geomechanical parameters was based on an integrated analysis of a comprehensive dataset comprising field surveys, *in situ* investigations, and laboratory testing. By synthesizing the lithostratigraphic framework (derived from outcrop mapping, geological surveys, and borehole logs) with the experimental results, a consistent set of representative values has been established for the rock masses within the study area.

Chapter 6. The “La Montagna” landslide

Overview

Deep-seated landslides occur globally, involving a variety of materials and displaying different velocities and movement patterns (e.g., Petley and Allison, 1997).

Certain deep-seated landslides exhibit slow, short-term movement (weeks to months) before initial failure and, less frequently, long-term movement (years to decades) that may lead to collapse. The progression of movement within brittle regimes is associated with deformation processes around the unstable mass, regardless of whether these processes are brittle or ductile. Typically, this mechanism presents as a sudden slope failure following a phase of limited slow movement. In ductile regimes, materials show extended, slow movements, with failure occurring only under external loads. In such contexts, creep (Fleming & Johnson, 1975; Emery, 1979; Van Asch, 1984) progresses at a consistent velocity and is primarily driven by changes in soil mechanical properties and discontinuities.

On this basis, the objective of this study is to enhance the understanding of the dynamics of deep-seated landslides that develop along clayey stratifications.

Due to the landslide advancing the upper boundary of the pre-existing Montaguto earthflow upslope, which is renowned for its intricate evolutionary history and its impacts on local structures and linear infrastructure (e.g., Guerriero et al., 2013a, 2013b, 2014, 2015), the findings of this study offer novel insights into the mechanisms governing headward retreat in large earthflows affecting structurally complex formations (e.g., Pinto et al., 2016) and into the contributory role of soft clayey layers.

The analysis, which integrates remote sensing observations with laboratory test data, is further enhanced by three-dimensional numerical modelling.

6.1. General aspects of the “La Montagna” landslide

This chapter examines the behaviour of the recent deep-seated “La Montagna” landslide, situated in the Province of Avellino, which failed on November 7, 2018, following a period of accelerated creep (Guerriero et al., 2019; Fig. 6.1).

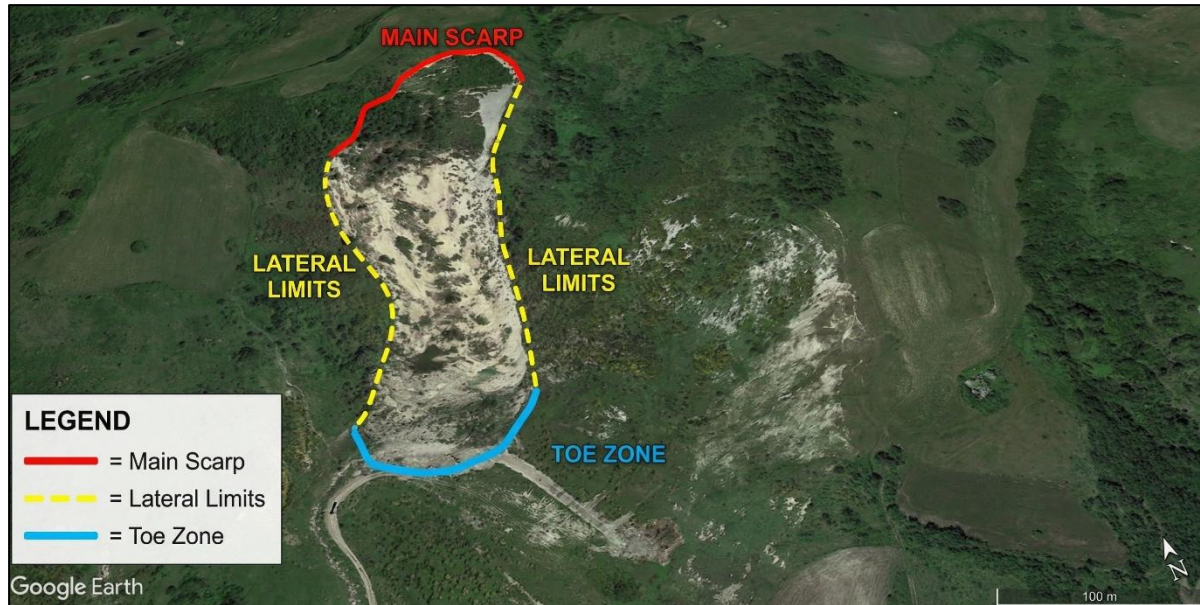


Figure 6.1. Morphological framework of the “La Montagna” landslide. The image (processed from Google Earth) illustrates the main features of the mass movement: the main scarp (red line), the lateral limits or flanks (yellow dashed lines), and the toe zone (blue line).

The La Montagna landslide lies within the upper western portion of the Montaguto earthflow source area in southern Italy (Fig. 8).

It is characterized by an upper, deep-seated translational slide that transitions downslope into a debris flow. The landslide extends about 390 m and mobilizes an estimated 300,000 m³ of fine-grained flyschoid material from the Flysch of the Faeto Formation (Pinto et al., 2016).

The upper translational block measures roughly 125 m in length and 110 m in width, with an average reconstructed thickness of about 16 m and a maximum of around 20 m, based on its topographic expression (i.e., soil bulging).

This portion of the slope is predominantly affected by extensional deformation, with sets of normal faults locally associated with strike-slip and oblique-slip fault segments. Individual fault scarps range in height from 0.3 m to 2 m. The upper translational block terminates at a slope break, where the sliding mass transitions into debris due to enhanced extensional deformation and toppling.

The landslide exhibits a flow-like morphology, and the deposit consists mainly of fine flyschoid material with scattered debris and boulders. The flow deposit thickness ranges from 5 to 14 m, and its distal zone displays the typical bulging morphology associated with earthflow toes. Field evidence indicates that, according to the evolutionary model proposed by Pinto et al. (2016), the geometry of the La Montagna landslide is primarily controlled by the slope's geological structure.

The upper translational block advances primarily along a narrow (~10 cm) clayey layer, which is extensively exposed along its eastern flank (Fig. 6.2). Geological data reported by Pinto et al. (2016) suggest that this sector of the slope is influenced by folding, in which the clay layer also serves as the landslide's basal shear zone. The shear surface exhibits a concave-upward geometry, laterally constrained by steep slip surfaces.

Bedding and discontinuity measurements from outcrops along the landslide margins have enabled the reconstruction of its three-dimensional geometry.

The upper translational block has a trapezoidal cross-section and a longitudinal shape resembling a parallelogram.

Downslope of the knick zone, the basal slip surface aligns with the pre-failure topography, and the landslide morphology assumes a more flow-like form under this structural control.



Figure 6.2 - Detail of the basal shear zone and slip plane. The field observation highlights a thin clayey layer (approximately 10 cm thick) acting as the main slip surface. The basal shear zone marks the mechanical discontinuity between the stable substrate and the overlying landslide material.

In the upper sector, the landslide movement is confined by well-defined lateral boundaries, especially evident on the eastern side, where they form a distinct oblique fault. Further downslope, along the flow pathway, the strike-slip faults that mark the landslide edges are less prominent than in the source zone and consist of multiple segments spread across the moving core.

Although the La Montagna landslide was likely creeping for several years (with initial signs observed in 2010), the primary reactivation occurred between late October and early November 2018.

In this event, the upper block shifted by approximately 15 meters, compressing the underlying debris and creating a deposit with a flow-like morphology.

To define the kinematic character of the landslide, a series of activities was carried out, including geological and morphological surveys, acquisition of georeferenced drone images suitable for topographic restitution, and the establishment of GPS benchmarks.

Figure 6.3 shows the image of the specifically surveyed area obtained through high-definition aerial photogrammetric surveys. It encompasses areas of particular interest, including the toe of the landslide mass, the water accumulation zones, the deformed drainage trench, and the corresponding portion of the deformed mass. This image was therefore used to make appropriate comparisons with pre-2018 instability conditions.

Figure 6.4 illustrates the elevation differences between the 2019 survey and pre-movement surveys.



Figure 6.3 - Detailed photogrammetric orthomosaic of the investigated area. The image shows the current state of the landslide toe and the deformed drainage network. This survey area was specifically selected to evaluate morphological evolution by comparing post-event deformations with pre-2018 historical data.

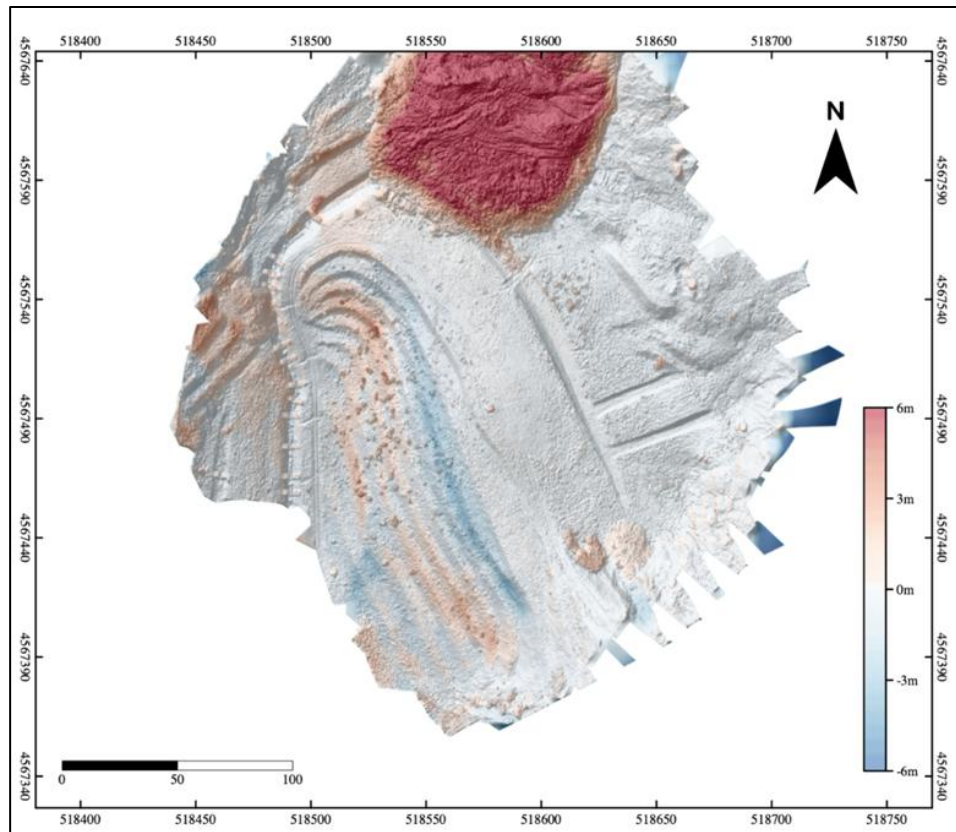


Figure 6.4 - DEM of Difference (DoD) of the Montaguto landslide toe. The map shows the topographic changes between the two survey periods. Red areas indicate positive elevation differences, with values reaching up to +6 m, while blue areas represent negative differences (subsidence or erosion) down to -6 m. The shaded-relief (hillshade) background highlights surface texture and structural deformations

The analysis provides a comprehensive, quantitative delineation of the processes that occurred in the region, resulting in the formation of an accumulation exceeding 6 meters in thickness and deformation of the area containing drainage trenches and associated channels.

As previously reported, the placement of GPS points enabled the monitoring of mass movements over an extended period. The comprehensive distribution of these monitored points is illustrated in Figure 6.5.

Additionally, the figure depicts the displacement vectors and the temporal progression of deformation at monitoring points no. 8 and 9.

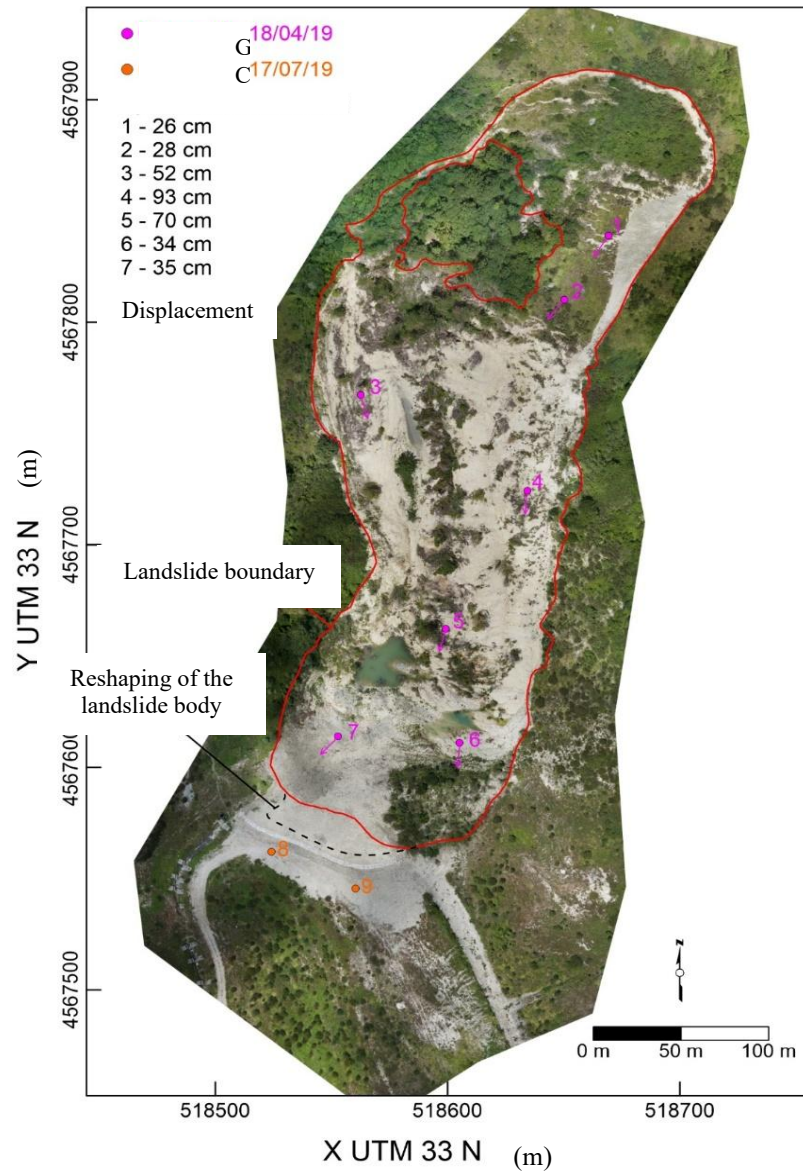


Figure 6.5 - Surface displacement monitoring at the Montaguto landslide. The orthophoto shows the landslide boundary (red line) and the location of the Ground Control Points (GCP) surveyed on 18/04/19 (magenta) and 17/07/19 (orange). Displacement vectors (1–7) quantify the mass's movement over the three-month period, with values ranging from 26 cm to 93 cm. The dashed line indicates the reshaping of the landslide body in the toe sector.

As previously mentioned, although the La Montagna landslide had been moving slowly for several years, the most significant movement occurred between the final week of October and the first week of November 2018.

During this period, the landslide displaced about 17 meters (Guerriero et al., 2022), exhibiting a hyperbolic pattern of movement and a parabolic change in velocity, with an acceleration phase followed by gradual deceleration.

Such trends are typical geodetic indicators of progressive slope failure in landslides (Saito, 1965).

In this episode, the upper block collapsed into debris, forming a deposit with flow-like characteristics. From September 29 to November 17, 2019 (Guerriero, 2022), a low-cost extensometer monitored ground displacement. The data showed creep-like behaviour, with a phase of steady acceleration and movement rates reaching up to 110 mm/h, consistent with a typical hyperbolic displacement pattern. The maximum velocity was recorded on the afternoon of November 9, 2019.

Before November 5, the landslide velocity increased gradually, with slight sinusoidal fluctuations. On November 6, the landslide reached a failure condition, and the movement rate rose sharply, peaking at 110 mm/h on November 9. Thereafter, the velocity gradually decreased until the movement completely ceased on November 15.

According to Saito's linear relationship (Saito, 1969), the observed displacement trend allowed the slope failure time to be estimated, which, based on the analysis, occurred in the morning of November 6, 2018.

Overall, a single-cycle sinusoidal pattern of the acting force was reconstructed, consistent with the behaviour of catastrophic landslides characterized by sudden failure (Ekström et al., 2013).

During the initial phase, the unbalanced downslope force increased steadily, accelerating the landslide. In the subsequent phase, this force decreased and became negative, slowing the landslide.

The decrease in force and the resulting reduction in movement velocity can be attributed to a redistribution of mass along the slope.

The reconstructed force time series also shows slight sinusoidal oscillations preceding the collapse of the final slope. These features provide evidence of the nonlinear nature of the progressive failure process, which is typical of landslides developing in structurally complex geological settings.

While the phenomenon has previously been described as a translational slide turning into a debris flow, a detailed analysis of field evidence and material make-up suggests a more nuanced classification.

- **Matrix-supported composition:** the landslide deposit mainly consists of fine-grained flyschoid material, with scattered debris and boulders serving as secondary components rather than the primary structure. This dominance of fines indicates earthflows rather than debris flows; the latter generally require a primarily coarse-grained fraction and are driven by rapid, fluid-saturated surges.
- **Distal morphology:** the distal part of the flow shows noticeable bulging, a feature typical of earthflow toes. This suggests movement driven mainly by material viscosity and excess pore-water pressure within the fine-grained matrix, rather than by frictional grain-to-grain collisions.
- **Kinematic mechanism:** although the event reached peak velocities of 110 mm/h during the November 2018 crisis, the displacement followed a clear hyperbolic acceleration pattern (tertiary creep). This behavior aligns with the plastic deformation of saturated clayey materials rather than the rapid, non-viscous collapse of non-cohesive debris.

In summary, the decrease in driving force and terminal velocity results from the downslope redistribution of mass, which promotes a gradual return to kinematic stability. Additionally, the sinusoidal oscillations observed in the force-time series prior to the final collapse strongly suggest the non-linear nature of the progressive failure process. This mechanical behavior is typical of landslides forming in structurally complex geological settings, where internal shears and lithological discontinuities influence the failure path.

The 2019 event clearly showed that the shift between creep phases is the "engine" driving the disaster. Before May 2019, the "La Montagna" slope was in a steady-state creep stage. Displacements, tracked using Ground-Based Interferometric Synthetic Aperture Radar (GB-InSAR) and GPS, measured only a few millimeters per month.

During this period, the internal structure of the flysch remained stable due to its residual cohesion.

The heavy spring 2019 rainfall led to a rapid acceleration of the system. Velocity rose from millimetres per month to several centimetres per hour, peaking at meters per day within the active lobe. Unlike rockfalls, which occur suddenly, the slope exhibits a viscous kinematic behavior governed by the Faeto Flysch. The marly clays serve as a "viscous brake."

The 2019 reactivation was caused by the shift from secondary to tertiary (accelerating) creep. The main factor was exceptional cumulative rainfall over the previous months (exceeding 350–400 mm per quarter), which greatly increased pore-water pressures within the flysch. This water effectively "hydraulically lifted" deep rock blocks, lowering effective friction and leading to the surge in velocity seen during the primary detachment.

The theoretical curve in Figure 6.6 illustrates this behaviour, which is characterised by three major kinematic stages (e.g., Varnes, 1978; Saito, 1969; Crosta and Agliardi, 2003; Jiang et al., 2013; Wang et al., 2018).

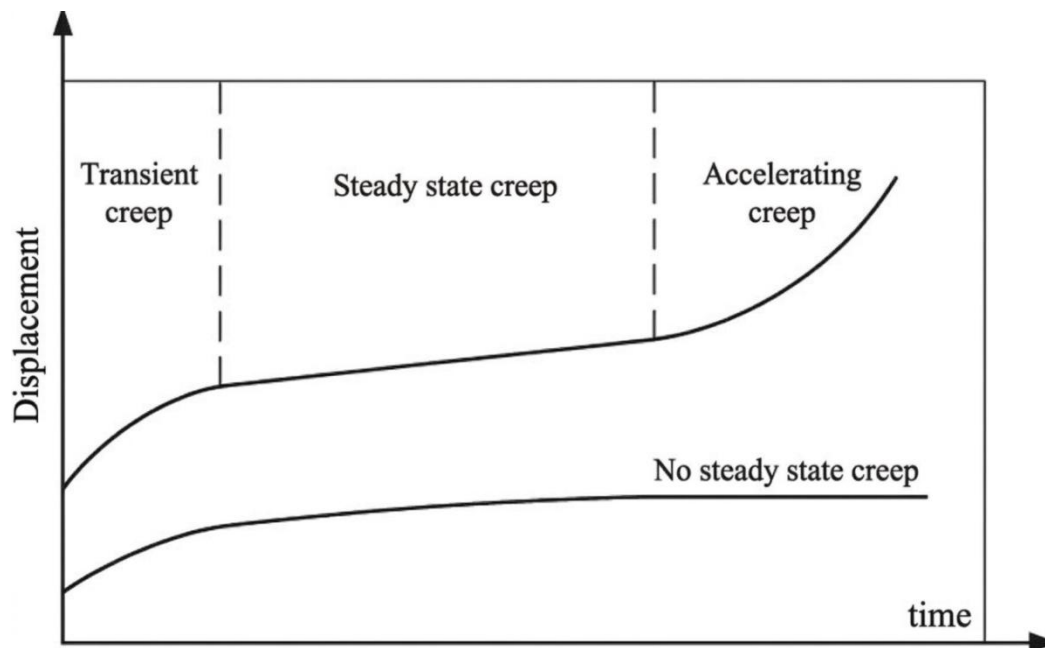


Fig. 6.6 - Conceptual models of landslide creep behavior. The upper curve illustrates the classic three-stage creep process: primary (transient), where the rate decreases; secondary (steady-state), characterized by constant velocity; and tertiary (accelerating), leading to ultimate failure. The lower curve represents a stable or dormant condition where no steady-state creep is achieved after the initial transient phase.

Chapter 7. Geological and structural data acquisition for Numerical modelling

Overview

The kinematic complexity and highly heterogeneous deposits of the "La Montagna" landslide required a shift to three-dimensional modelling. Although 2D simulations provide a basic estimate, they are inadequate for capturing scenarios in which irregular topography, asymmetrical failure surfaces, and boundary effects, such as lateral resistance, significantly influence movement.

This section outlines the development of the advanced numerical model, focusing on the following key phases:

- Converting real-world geometry into a solid computational domain (mesh).
- Defining and calibrating the physical and mechanical properties of the soils and rock masses involved.
- Simulating internal stresses and deformations to validate the kinematic hypotheses formulated during the survey and monitoring phases.

Therefore, implementing a 3D approach is not just about achieving higher computational accuracy; it also serves as a crucial interpretive tool for verifying data collected from in-situ monitoring and geological surveys.

7.1. Methodological approach

Research on the "La Montagna" landslide focuses on deep-seated landslides occurring within clay layers. It uses Digital Elevation Models (DEM), laboratory test results, and 3D numerical modelling with FLAC3D.

7.2. Field data collection phase

Field data collection played a crucial role in validating and integrating the geological-geotechnical model of the study area. The fieldwork, conducted directly on the landslide, involved collecting detailed structural measurements and morphological observations essential to understanding the ongoing deformation processes.

The attitudes of the main discontinuities and lithological contacts were measured with a geological compass for Dip and Dip Direction. These data were compared with existing bibliographic and cartographic sources to verify accuracy and detect local variations in the structural setting. This process refined the geological interpretation and improved the three-dimensional model of the landslide body. To support this, several field photographs (Fig. 7.1 and 7.2) were included to document key geomorphological features of the study areas.



Figure 7.1 - The La Montagna Landslide



Figure 7.2 – Structural setting of the landslide area

The initial phase, essential for creating a reliable numerical model (particularly in complex cases like the “La Montagna” landslide), involves detailed geotechnical characterization. This was achieved through several field survey campaigns that systematically identified and measured discontinuities compromising the mechanical integrity of the rock and soil mass.

For each measurement station, the spatial orientation parameters were established using the Dip and Dip-Direction notation. The main goal of the field campaign was to identify the dominant "families" or Joint Sets of discontinuities that influence slope kinematics.

By gathering a statistically significant sample of orientations, it became possible to move from individual point measurements to the definition of systematic sets.

This survey identified five main structures, subsequently named Fault 1 through Fault 5, each playing a specific mechanical role in the geomechanical system. These discontinuities were represented as dedicated interfaces in the FLAC3D model.

- **Bedding and basal sliding systems:** Characterized by low-angle dip orientations, they are frequently associated with weak clayey interlayers. In the numerical model, they serve as potential basal sliding planes (e.g., "IDrear", "basal surface") that facilitate the translation of the landslide body.
- **Shear and release systems:** Composed of sub-vertical or high-angle discontinuities, they act as detachment surfaces (tension cracks) or lateral release planes. They are crucial for defining the volumetric boundary and lateral confinement of the unstable mass.

7.3. Stereographic analysis of discontinuity poles

Field survey data were analyzed using stereographic projections (Schmidt-Lambert equal-area net, lower hemisphere) to identify the primary discontinuities affecting the slope's geometry. Since the dataset consisted of discrete primary structural features rather than diffuse joint populations, the focus was on accurately determining their orientation parameters. Statistical filtering and clustering were not applied because they would not yield meaningful insights given the limited sample size. The data processing aimed to achieve the following objectives:

- **Kinematic and Morphological Verification:** stereographic analysis helped confirm the spatial relationship between the detected discontinuities and the slope's shape. This step is crucial to verify that the identified structures are kinematically consistent with the movements within the landslide, such as by examining how planes daylight relative to the topography.
- **Derivation of Modelling Input Data:** the projection of poles enabled accurate extraction of mean Dip and Dip-Direction values needed for the next numerical implementation stage.

The distribution of poles illustrated in Figure 7.3 shows the arrangement of the five primary structures (F1–F5) identified within the study area. The orientation values reported in Table 6 serve as the primary geometric parameters for simulating the mechanical behavior of the flysch within the FLAC3D numerical code.

Table 6. Discontinuity orientations and their normal poles

Fault	Dip Direction (°)	Dip (°)	Pole Trend (°)
F1	276	29	96
F2	142	22	322
F3	261	87	81
F4	176	74	356
F5	215	20	35

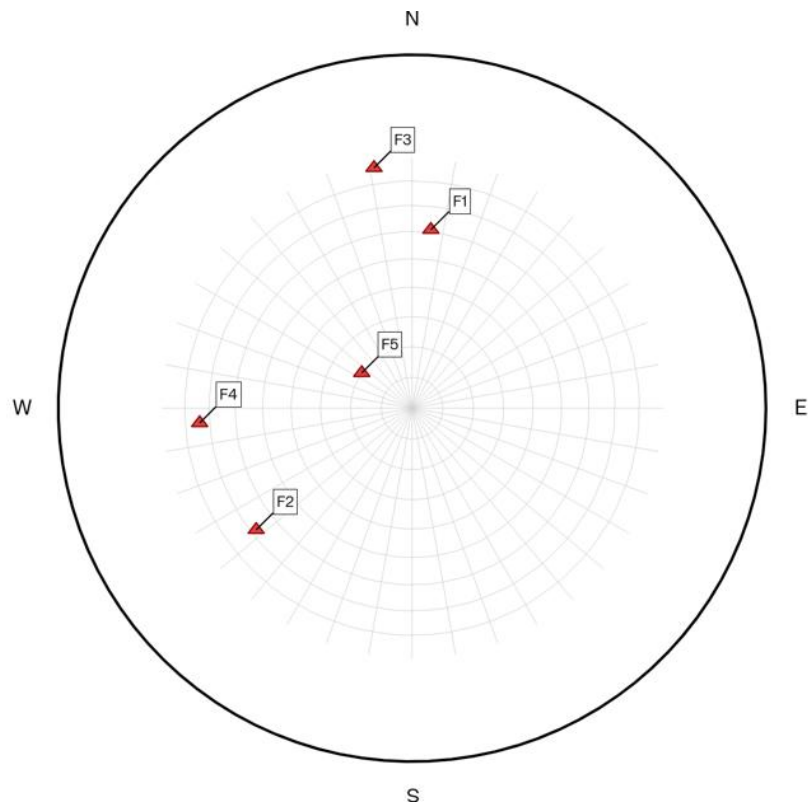


Figure 7.3 - Polar plot of the surveyed discontinuities. The stereogram shows the dip and dip-direction of the main slip surfaces (F1–F5) identified during the field investigation. The scatter of the poles reflects the internal deformation of the landslide mass and the presence of multiple sliding planes within the basal shear zone.

7.4. Geometric configuration and spatial characterization of discontinuities

The first step in 3D numerical modelling with finite-difference methods, such as FLAC3D, is to translate field geological observations into mathematical representations. This is done by analytically converting orientation parameters (Dip/Dip Direction) into plane equations in Euclidean space. While a 2D analysis depicts a discontinuity as a line, in 3D it needs to be defined by the general plane equation:

$$ax + by + cz + d = 0$$

This mathematical transition is essential for precisely representing the rock mass's structural anisotropy within the 3D grid.

7.4.1. Definition of discontinuity orientation parameters

The structural setting of the rock mass is defined by quantifying the spatial orientation parameters of discontinuities, which are essential for slope stability analysis.

These parameters describe the "geometric signature" of weak zones and are uniquely defined by Dip Direction and Dip.

An example of these parameters is shown in Fig. 7.4.

Fault 1				
Dip. Dir.	276			
Dip	29			
Tangent of the apparent inclination in x				-0,55127
Apparent inclination in x				-28,8667
Tangent of the apparent inclination in y				0,057941
Apparent inclination in y				3,316071
Intersection of the plane with X axis				-1,81398
Intersection of the plane with Y axis				17,25891
	x	y	z	
Point A	-1,81398	0	0	
Point B	0	17,25891	0	
Point C	0	0	1	

Figure 7.4 - Calculation spreadsheet for the definition of discontinuity spatial parameters

- Dip Direction:** Indicates the azimuth of the horizontal projection of the line of highest slope on the discontinuity plane. Measured in degrees (0–360°) relative to geographic North, it specifies the dip quadrant and is crucial for evaluating the kinematic compatibility between structural features and the slope profile.

- **Dip:** Indicates the angle of maximum slope of the plane relative to a horizontal reference. Ranging from 0° to 90° , this value affects the shear component of gravity acting on potentially unstable blocks and directly impacts whether the shear resistance threshold along the joint is exceeded.

7.4.2. Analytic formalization of discontinuity planes: intercepts and spatial constraints

Converting geological survey data into FLAC3D's numerical domain requires transforming discontinuity surface orientations, defined by Dip and Dip Direction, into plane equations in Euclidean space. Spatial Intercepts (X, Y) and Control Elevation (Z) serve as geometric coefficients that precisely locate sliding surfaces within the computational mesh.

The values A (X Intercept) and B (Y Intercept) indicate where the fault plane (or discontinuity plane) intersects the Cartesian axes of the model's global reference system. These values are calculated analytically by ensuring the plane passes through a specific control point C at elevation z (commonly $z = 1$).

The typical form of the plane's equation in the model is given by:

$$\frac{x}{A} + \frac{y}{B} + \frac{z}{C} = 1$$

This formulation enables the numerical solver to quickly determine the half-space occupied by the landslide body relative to the separation surface, making it easier to assign different mechanical properties to the grid nodes.

7.4.3. Calculation and distribution of apparent dips

Table 7 presents the calculations for the apparent dip tangent. This step is crucial for the FLAC3D numerical model, because the mesh is typically oriented along the North-South (y) and East-West (x) axes; a fault dipping at 276° (nearly West) does not align perfectly with the grid axes.

The spreadsheet (Fig.7.3) decomposes the plane's orientation into two primary vectors:

- **X-Component:** The apparent slope of the plane along the X-axis.
- **Y-Component:** The apparent slope of the plane along the Y-axis.

This decomposition enables the software to correctly assign interface properties (such as friction and cohesion) to individual mesh nodes. By doing so, it prevents orientation errors that would otherwise lead to an unrealistic structural model.

7.5. Sector analysis and geometric parameter

Table 7 provides a comprehensive geometric overview of the structural discontinuities affecting rock-mass behaviour in the "La Montagna" model. It acts as a crucial link between the geological field survey and the numerical modelling in FLAC3D.

Table 7. Geometric parameters and spatial intercepts of the primary surveyed discontinuities (Reference elevation $z=1$)

Discontinuity	D/DD	Intercept X (A)	Intercept Y (B)	Control Elevation (C)
Fault 1	276/29	-1.814	17.259	$z=1$
Fault 2	142/22	4.020	-3.141	$z=1$
Fault 3	261/87	-0.053	-0.335	$z=1$
Fault 4	176/74	4.111	-0.287	$z=1$
Fault 5	215/20	-4.790	-3.354	$z=1$

Several key observations can be derived from the data shown in Table 7:

- **Fault 1 (276/29)** is characterized by a negative X-intercept (-1.814) and a notably high Y-intercept (17.259). Geometrically, this suggests that the plane dips nearly parallel to the X-axis, oriented westward. The elevated B value (the Y-intercept) indicates a very gentle dip along the Y-axis, representing a low-angle structural surface.
- **Fault 3 (261/87)** is the most geometrically critical plane in the model. Its intercepts are very close to the origin (-0.053 and -0.335). Mathematically, this indicates a sub-vertical plane at about 87° . Because the plane is nearly perpendicular to the horizontal datum, it intersects the horizontal axes almost immediately, indicating a near-vertical structural discontinuity or "vertical drop" in the geomechanical context.

- **Fault 5 (215/20)** is the most horizontal surface identified, dipping only 20° . The negative intercepts on both axes ($X = -4.790$, $Y = -3.354$) indicate a plane dipping toward the third quadrant (South-West). In the numerical model, this orientation indicates a gently inclined bedding plane or a basal sliding surface that could influence the landslide's lateral spread.

Choosing a unit control elevation C ($z = 1$) is a geometric convention that simplifies the calculation of the plane coefficients. In the final modeling phase, these equations are shifted along the Z -axis to align the surfaces with the actual depths from boreholes. This ensures full consistency between the stratigraphic survey data and the digital model.

7.6. From Field Mapping to Computational Input: Establishing the Geomechanical Basis for FLAC3D

The transition from initial surface geostructural assessments to the creation of a detailed numerical model marks a key development in the current research approach. The discretization of the rock mass, required for FLAC3D finite difference simulations, relies on accurately defining contact interfaces that represent real discontinuities.

In this context, Table 6 not only summarizes orientations but also functions as a geometric input protocol for defining "cut" planes within the slope's spatial domain. Each set of three values (Dip, Dip Direction, and spatial location) helps isolate specific rock and soil blocks, transforming the model from a uniform volume into a multi-block system that can replicate the kinematic behaviors observed on-site.

As noted in Section 7.3, the structural dataset examined does not comprise a diffuse joint population. Instead, it consists of a distinct set of discontinuities (F1-F5). This difference is crucial: the choice to avoid using statistical filters or clustering methods like K-means stems from the structures' nature, in which their importance is determined by their kinematic uniqueness rather than by their statistical abundance.

To meet the graphic validation standards required by geostructural literature, Table 6 presents pole data (Pole Trend and Pole Plunge) along with plane parameters. Although the FLAC3D numerical solver works directly with dip and dip direction vectors, projecting poles onto the Schmidt-Lambert net (Figure 7.3) was essential for:

- **Validating internal model consistency:** making sure each plane entered into the code matches exactly the surveyed and stereographically projected trace.
- **Preliminary kinematic check:** examining the pole distribution relative to the "great circle" representing the slope topography to identify potential instability mechanisms, such as planar or wedge sliding, before performing numerical calculations.

A frequently overlooked aspect of landslide numerical modelling in flyschoid rock masses is the code's sensitivity to the precision of intersections. This study defines surface intersections (such as between Fault F1 and Fault F2) as the edges of rock wedges, which then determine the possible directions of kinematic release.

Table 6 shows values rounded to three decimal places based on geometric calculations. This does not indicate highly accurate instrument measurements for compass surveys but is essential for maintaining topological consistency within the software. Even minimal approximation errors in defining the plane can result in 'gaps' or overlaps in the calculation meshes, which can cause numerical instability or, in the worst cases, lead to model convergence failures during stress-strain analyses.

Integrating these data allows the model to overcome the limitations of isotropic approaches. By assigning specific shear strength parameters (such as residual friction angle and zero or reduced cohesion) to each interface listed in Table 6, the FLAC3D model can effectively simulate the rock mass's inherent strength anisotropy.

The slope is now viewed not as a single entity but as a collection of domains, each with movements limited by the preferred sliding directions dictated by structures F1–F5. This approach effectively represents the behavior of flysch, where the tectonic structure often has a greater impact than the geotechnical properties of the intact rock matrix.

Chapter 8. Slope stability analysis

Based on detailed geostructural characterization in Chapter 7, a preliminary kinematic analysis was performed to identify potential instability mechanisms, including planar sliding, wedge failure, and toppling. This step is crucial for verifying the importance of the isolated discontinuities (F1–F5) before moving forward with their numerical discretization in the FLAC3D model (Chapter 9).

The 3D model was developed by integrating geological, geotechnical, and structural data collected during field surveys. Its primary objective was to accurately reconstruct the landslide's geometry, interpret its internal structure, and identify potential mechanisms of movement. This approach seeks to enhance understanding of the slope's overall evolution. Combining diverse datasets provides a cohesive view of the unstable volume and the discontinuity surfaces that affect the landslide's kinematics.

The model's implementation began with processing topographic and digital elevation model (DEM) data, which were carefully filtered and interpolated to establish an accurate reference surface. Structural elements derived from detailed geological surveys were then layered onto this foundation.

Analysing the identified discontinuities uncovered the main fracture systems and weak planes within the rock mass, providing vital insights into the orientation and spread of the key fracture sets. These findings were incorporated into the 3D model to effectively illustrate the tectonic framework and possible zones of deformation.

The results in the following sections are based on an iterative calibration and validation process.

This involved comparing numerical simulations with field evidence and historical monitoring data to develop a consistent and geomechanically plausible model of landslide behaviour. The analyses described here serve as the foundation for future stability evaluations and detailed numerical studies to understand the slope's response under various external loading conditions.

Kinematic analysis is a traditional method in geotechnical engineering used to evaluate potential failure modes of rock and soil slopes, accounting for geometric and frictional constraints.

The following outlines the mathematical framework employed to assess potential failure mechanisms in the "La Montagna" landslide, utilizing discontinuity orientations and strength parameters.

8.1. Wedge Stability Analysis

Input Data:

- Slope: 40°/220°
- Main discontinuities: (Table 8)

Table 8: Orientation parameters of the main identified discontinuity sets

Disc.	Dip (°)	Dip Direction (°)
J1	29	276
J2	22	142
J3	87	261
J4	74	176
J5	20	215

Figure 8.1 illustrates the orientations of the discontinuities.

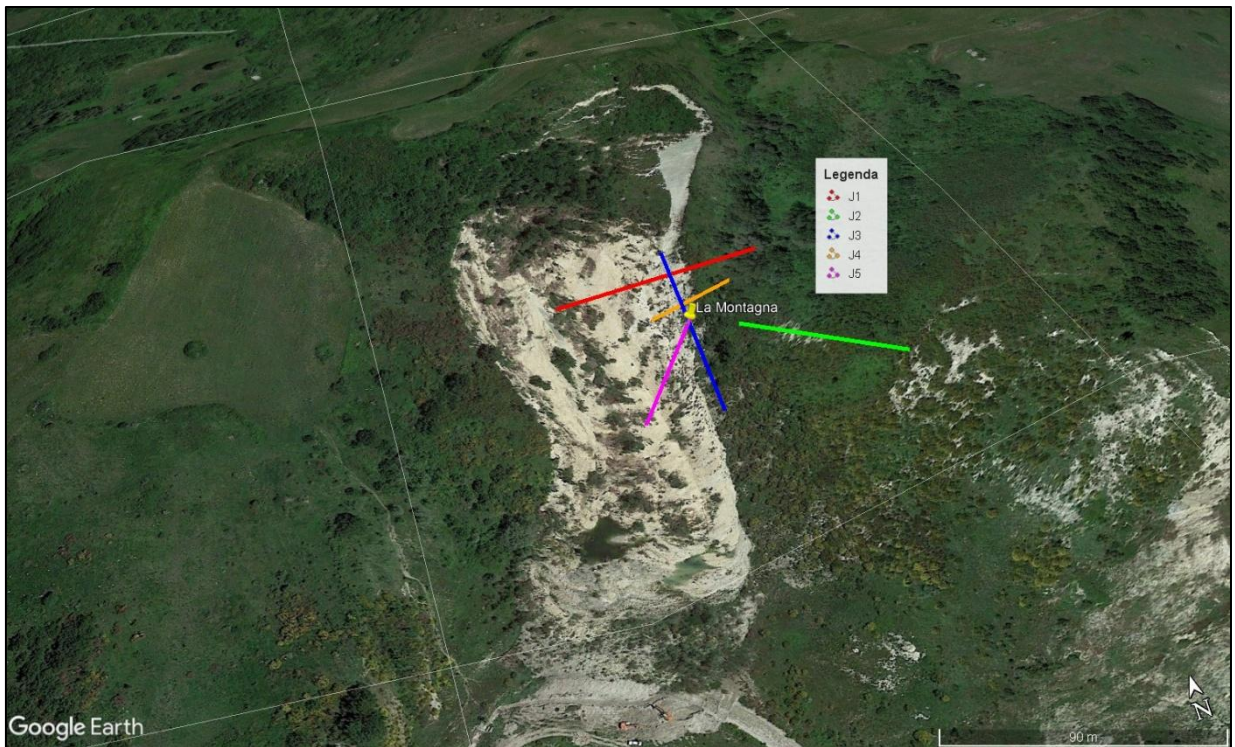


Figure 8.1 - Orientation of the surveyed discontinuities

The geotechnical parameters used for the kinematic analysis are reported below:

- Friction angle (ϕ): 45°
- Cohesion (c): 100 kPa ($= 10^5$ Pa)
- Unit weight (γ): 20 kN/m³ (estimated)
- Reference height (H): 10 m (from FLAC3D model).

To conduct an accurate analysis, it was essential to identify the intersection lines. Specifically, for each plane, the dip and dip direction were transformed into stereographic polar coordinates using auxiliary angles:

$$\alpha_i = 90^\circ - \text{DD}_i$$

$$\beta_i = 90^\circ - \text{dip}_i$$

where:

- α_i = corrected azimuthal angle (stereographic longitude)
- β_i = co-latitude angle (stereographic latitude)

Once the dip and dip direction parameters were converted into polar coordinates, the unit pole vector was determined for each discontinuity.

$$\vec{P}_i = (\sin \alpha_i \sin \beta_i, \cos \alpha_i \sin \beta_i, \cos \beta_i)$$

In the Cartesian system (X, Y, Z) with Z vertical.

Next, the intersection line between each pair of discontinuity planes was determined by computing the cross product of their respective unit pole vectors.

This line shows the possible sliding direction for wedge failure mechanisms: if it is unfavorably oriented compared to the slope (inside the friction cone), it suggests kinematic instability.

The direction of the line of intersection was thus defined as: $L = P_1 \times P_2$

Expanding:

$$L_x = P_{1y}P_{2z} - P_{1z}P_{2y}$$

$$L_y = P_{1z}P_{2x} - P_{1x}P_{2z}$$

$$L_z = P_{1x}P_{2y} - P_{1y}P_{2x}$$

Normalizing: $\vec{L}_{\text{norm}} = \vec{L}/\|\vec{L}\|$

The results obtained were then translated into Trend and Plunge by applying the reported formulas to all relevant plane pairs.

These results are shown in Table 9, in which the modeled interfaces (J1–J4) directly correspond to the primary tectonic features (F1–F4) described in Chapter 7:

Table 9. Geometric parameters of the intersection lines between discontinuities and the slope face for stability analysis

Disc.	Trend (°)	Plunge (°)	Description
J3-slope	308	-2	Nearly horizontal, facing NNW
J4-slope	321	13	Moderate plunge toward NW
J3-J4	256	-3	Toward SW, low dip
J1-J4	195	-15	Toward S, negative plunge
J5-slope	121	10	Toward SE, low plunge
J2-slope	24	49	Toward N, high plunge

8.2. Kinematic interpretation

The J3-J4 pair is the most critical, with:

- Trend 256°: Aligned toward SW, consistent with the slope's maximum dip direction (220°).
- Plunge -3°: Slightly negative, indicating potential wedge extrusion from the slope face.
- Geometric configuration: The two joints (J3 with 87° dip nearly vertical, J4 with 74° dip) form a wedge.

The J1-J4 and J5-slope pairs show negative plunges, indicating possible extrusion, though their trends are less promising.

A wedge is considered kinematically unstable if it meets the following criteria:

- Trend of the intersection line falls within the daylight (exposure) zone:

$$DD_{\text{slope}} - 20^\circ \leq \text{Trend}_{\text{line}} \leq DD_{\text{slope}} + 60^\circ$$

- Plunge of the line is lower than the slope dip:

$$\text{Plunge}_{\text{line}} < \text{Dip}_{\text{slope}} = 40$$

Based on these criteria, the daylight zone spans from 200° to 280°. For the J3-J4 pair, the intersection line trend is 256°, comfortably within the 200°-280° range, with a plunge below 40°.

Consequently, the J3-J4 pair is considered kinematically critical, indicating a potentially unstable wedge.

Subsequently, the wedge weight was defined using the following formula:

$$W = \frac{1}{2} \gamma H^2 \cot \left(\frac{\alpha}{2} \right)$$

with:

$H = 10$ m and $\gamma = 20$ kN/m³:

$$W = \frac{1}{2} \times 20 \times 10^2 \times \cot \left(\frac{\alpha}{2} \right) = 1000 \cot \left(\frac{\alpha}{2} \right) \text{ kN}$$

The forces were then resolved along the intersection line, approximated by the plunge:

- **Tangential force (parallel to the line):**

$$T = W \sin \psi$$

- **Normal force to the sliding planes:**

$$N = W \cos \psi$$

where ϕ is the exit angle (approximately the plunge of the line).

8.3 Definition of the Factor of Safety (FoS)

The factor of safety was calculated using the following method:

$$\text{FoS} = \frac{c \cdot L + N \tan \phi}{T}$$

Replacing the parameters with Plunge = -3°:

$$\text{FoS} = \frac{100 \times L + W \cos (3^\circ) \times \tan (45^\circ)}{W \sin (3^\circ)}$$

- For J3-J4:

$$\text{FoS} = \frac{0.005 + 1.0 \times \cos (3^\circ)}{\sin (3^\circ)} \approx \frac{1.0}{0.052} \approx 1.81$$

- For J4 – slope:

$$\text{FoS} = \frac{0.005 + 1.0 \times \cos (13^\circ)}{\sin (13^\circ)} \approx \frac{0.98}{0.225} \approx 3.10$$

- For J3 – slope:

$$\text{FoS} = \frac{0.005 + 1.0 \times \cos (2^\circ)}{\sin (2^\circ)} \approx \frac{1.0}{0.035} \approx 1.86$$

The results are summarized in Table 10:

Table 10: Summary of stability parameters and comparison with critical thresholds for the analyzed discontinuity sets

Parameter	J3-J4	J4-slope	J3-slope	Threshold
Trend (°)	256	321	308	200-280
Plunge (°)	-3	13	-2	< 40
Kinematically critical?	Si	No	No	—
FoS	1.81	3.10	1.86	> 1.5
Stability	Partial	Stable	Stable	—

Therefore, the previously discussed geomechanical parameters can be used to identify a critical wedge for the J3-J4 pair.

The same analysis was conducted by lowering the shear strength parameters.

Table 11 highlights the key differences observed:

Table 11. Comparison of results as a function of shear strength parameter variations

Disc.	FoS $\phi=45^\circ$	FoS $\phi=35^\circ$	Reduction	Status
J2-slope	~6.5	0.62	-90%	Unstable
J1-J4	19.3	2.60	-87%	Critical
J3-J4	28.3	13.52	-52%	Marginal
J4-slope	4.28	2.99	-30%	Accettable
J5-slope	5.55	3.86	-30%	Stable

Table 11 shows that the J2-slope combination transitions from stable to unstable when FoS drops below 1.

Meanwhile, J1-J4 becomes the second primary concern because of a smaller safety margin, and J3-J4 remains problematic, even though it was already marginal at $\phi=45^\circ$.

A new critical wedge has been identified: the J2-slope combination, with a Factor of Safety (FoS) of 0.62, indicating instability because it's less than 1.0. Although the plunge of 48.9° exceeds the slope dip of 40° , the reduced friction angle ϕ indicates that the cohesive resistance is insufficient to maintain stability. This setup poses the main danger under reduced-friction conditions.

The second key wedge identified is the J1-J4 combination, with a FoS of 2.60, making it marginal. Its 195° trend sits at the edge of the daylight zone, and the -15.2° negative plunge indicates a definite potential for wedge exit. Positioned southward just beyond the optimal zone, its geometry implies potential movement.

The third significant wedge is the J3-J4 combination, with a Factor of Safety (FoS) of 13.52.

It remains stable but is sensitive to changes. The 256° trend perfectly matches the SW exit direction, and a -3° plunge suggests the wedge might be released.

Following the kinematic analyses that pinpointed critical vulnerabilities in the sliding mechanisms, the next step was detailed numerical modelling of various instability scenarios using FLAC3D.

Chapter 9. Three-Dimensional Geometric Reconstruction of the “La Montagna” Landslide

Overview

Building on the structural data from Chapter 7, particular focus has been placed on developing the three-dimensional numerical model of the landslide.

Initially, the Digital Elevation Model (DEM) was obtained from the official portal of the Ministry of the Environment and Energy Security (MASE), via the National Geoportal, accessible at <https://gn.mase.gov.it/portale/distribuzione-dati-pst>.

This dataset offered a precise, high-resolution topographic foundation vital for accurately anchoring the entire landslide model.

The DEM topographic surface was then imported into the Rhinoceros modelling environment, where the structural discontinuity surfaces (such as bedding planes and slip surfaces) previously identified in Chapter 7 through stereographic analysis and field surveys were incorporated. This step enabled a comprehensive and coherent reconstruction of the three-dimensional geometry of the Montaguto landslide body.

Finally, the geometric model was exported to FLAC3D via the Griddle plugin, ensuring accurate conversion of geometric surfaces into numerical zones and optimal mesh discretization for subsequent kinematic and stability analyses.

9.1. Geomechanical Discretization and Feature Labelling

To ensure consistency throughout the numerical transition, it is important to define the relationship between the geological features identified in the field (Chapter 7) and the numerical interfaces used in FLAC3D. Moving from field-based structural mapping to a three-dimensional numerical model requires a systematic conversion of geological data into mechanical components. While Chapter 7 identified and characterized the primary tectonic structures as Faults (F1–F5), the FLAC3D environment requires that they be discretized as Interfaces (J1–J5).

This shift in nomenclature is not merely a labeling preference but a necessity for geomechanical modeling: it marks the transition from describing a "geological history" to defining "mechanical behavior." Each interface is assigned specific constitutive properties (such as normal and shear stiffness (K_n , K_s) and residual friction) to simulate the rock mass's anisotropic response.

Table 9.1 establishes the direct correlation between the field-mapped features and their corresponding numerical counterparts, ensuring full traceability of the data throughout the modeling process.

Table 9.1 - Correlation between field-mapped structural features (Chapter 7) and numerical interfaces used in the FLAC3D environment, including their specific kinematic and mechanical roles

Geological feature (Ch.7)	FLAC3D Interface (Ch.9)	Mechanical role in the model
Fault F2/F1R	Interface J1/J1R	Primary basal sliding surface (lower/upper)
Fault F2/F2R	Interface J2/J2R	Lateral wedge plane and release surface
Fault F3/F4	Interface J3/J4	Posterior release surface (tension crack)
Fault F5	Interface J5	Internal structural discontinuity

A key distinction must be made between the sub-vertical interfaces J3 and J4. Although these features are incorporated into the 3D geometry, they do not serve as primary sliding planes because of their nearly vertical dip. Instead, they act as posterior release surfaces (tension cracks).

Their inclusion is essential for the model's kinematic accuracy: they simulate the physical separation (representing the loss of tensile strength) between the unstable landslide mass and the stable bedrock. This "release" enables the primary displacement to occur along the basal intersections (J1-J2 and J1R-J2R), accurately representing the multi-block failure mechanism observed in the Montaguto area.

9.2. NURBS-Based Geometric Synthesis and Topo-Structural Modelling in Rhinoceros

The section describes the geometric reconstruction workflow in Rhinoceros. It covers DEM surface import, discontinuity-plane integration, and 3D landslide-body assembly based on Chapter 7 structural data. The process ensures precise geometric coherence before exporting to FLAC3D via Griddle.

The three-dimensional model of the La Montagna landslide was created in Rhinoceros by integrating three key geometric elements: the topographic surface from the MASE DEM, the main landslide volume, and the structural discontinuity surfaces described in Chapter 7.

The external topographic surface, imported from the MASE DEM at 1 m resolution, defines the entire model domain and is represented by a high-density triangular mesh (see Fig. 9.1).

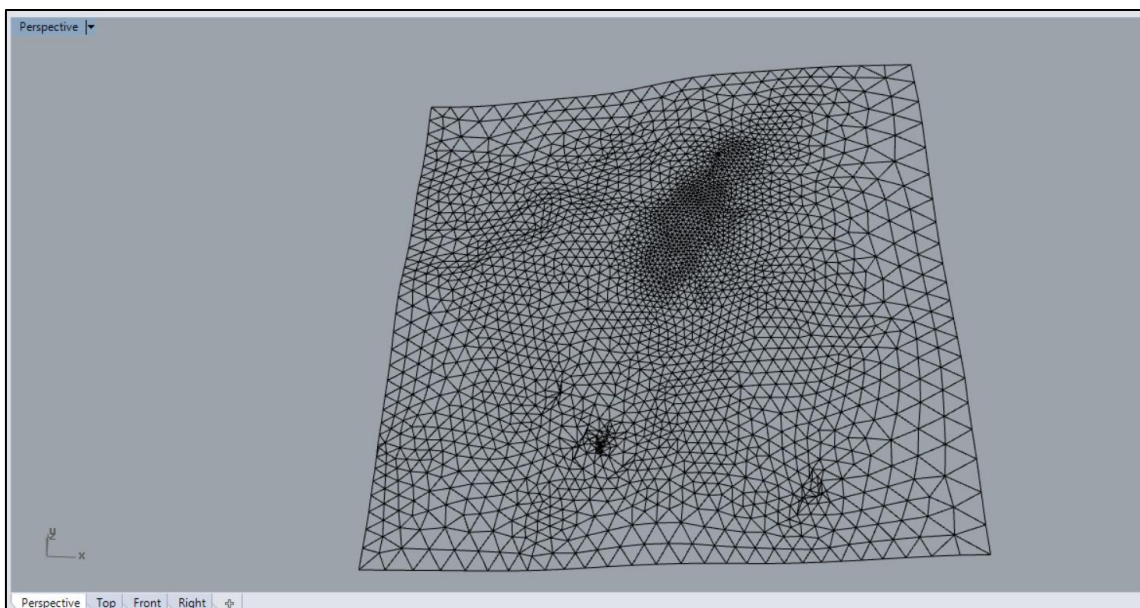


Figure 9.1 - Geometric discretization of the slope surface. Perspective view of the 3D triangular mesh used as the base geometry for the geotechnical simulation. The irregular triangulation allows for a high-fidelity representation of the morphological discontinuities identified during the photogrammetric survey

After reconstructing the topographic surface using a triangular mesh, a prismatic box representing the stable slope prior to the landslide was modelled in Rhinoceros (Fig. 9.2).

Later, the structural discontinuity surfaces identified in Chapter 7 were reconstructed.

The entire landslide, including the slope, was then exported and imported into the Griddle (Itasca) environment to generate the numerical grid, as shown in Figure 9.3.

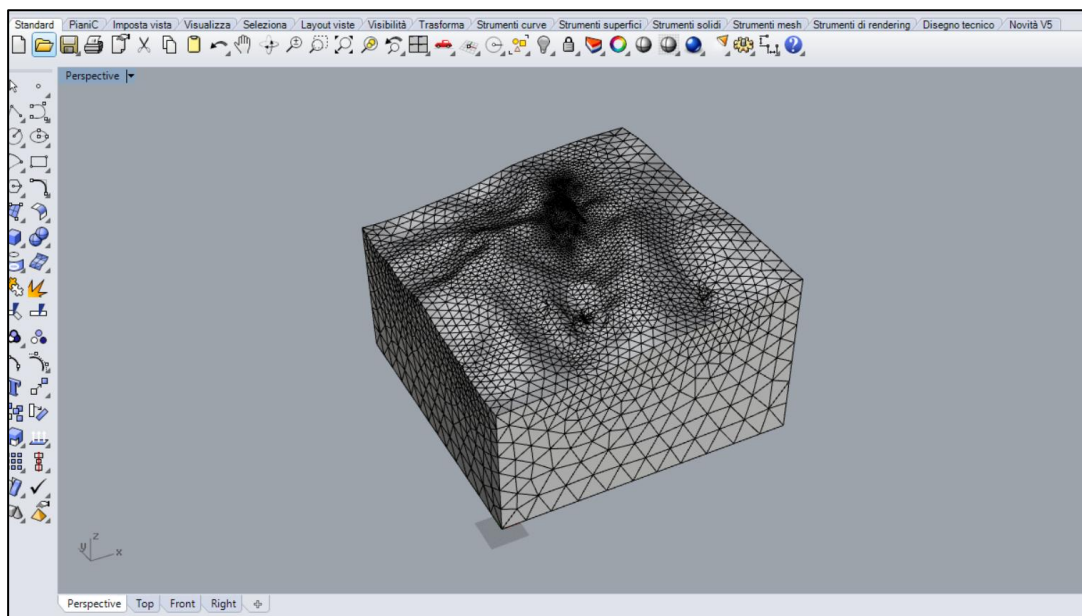


Figure 9.2 - 3D volumetric model of the landslide toe sector. The image illustrates the transition from a surface mesh to a solid domain performed within the Rhinoceros environment, making the model ready for geotechnical numerical simulations. The volumetric discretization allows for the assignment of material properties (e.g., unit weight, cohesion, and friction angle) to the various lithological units identified during the field investigation

Input mesh

Number of surface elements: 21239

Number of nodes: 10515

Output mesh

Number of tetrahedra: 208207

Number of nodes: 39789

Meshing time: 4.40 s

Figure 9.3 - Summary of mesh properties and computational statistics. The table details the transition from the input surface mesh (triangular elements) to the output volumetric mesh composed of tetrahedra. The final domain consists of 208,207 tetrahedral elements and 39,789 nodes, ensuring an appropriate resolution for the geotechnical stability analysis)

9.3. Numerical Implementation in FLAC3D: Mesh Integration and Interface Characterization

The finalized geometric assembly was discretized and imported into FLAC3D version 7.0. The rendering is shown in Figure 9.4.

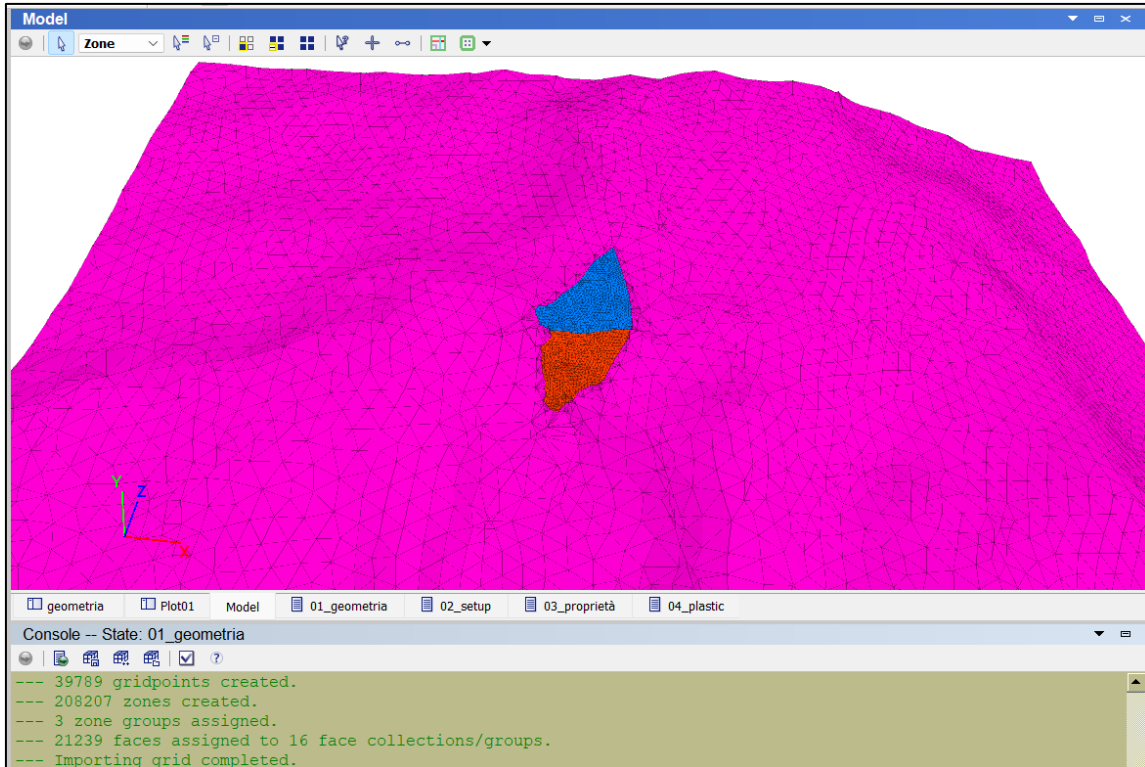


Figure 9.4 - Numerical grid initialization in the simulation environment. The image displays the 3D model after importing the Rhinoceros mesh. The computational domain is partitioned into three main zone groups (coloured magenta, blue, and orange), representing different geomechanical units. The console log confirms the creation of 208,207 zones and 39,789 grid points, ensuring consistency with the previous discretization steps.

Once the meshed geometry was imported into FLAC3D 7.0, the geological and stratigraphic interfaces were identified by assigning specific groups to the zone faces.

In FLAC3D, interfaces are specialized elements representing discontinuity planes. They are vital for simulating weak slip planes and preventing excessive grid distortion, ensuring the model reflects real kinematic behavior.

This approach helps prevent excessive grid distortion and ensures the model accurately reflects real kinematic behaviour.

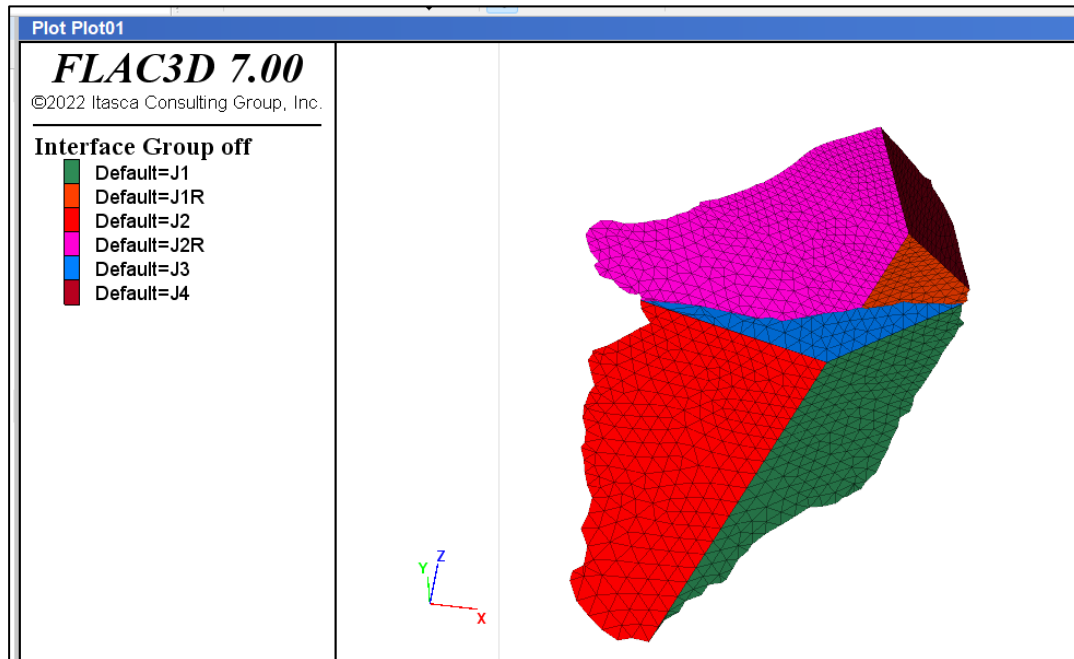


Figure 9.5 - Assignment of numerical interfaces in FLAC3D. The 3D domain is subdivided into several sub-blocks (J1 to J4 and respective contacts) to simulate the mechanical behaviour of internal discontinuities. These interface groups enable modelling of slip, separation, and friction among the geotechnical units identified in the “La Montagna” earthflow.

Following the definition of the geological-stratigraphic interfaces, constitutive properties were assigned to the zones and interface elements based on geotechnical characterizations from laboratory and in situ tests conducted in the Montaguto area. For the zones, grouped by lithotype (e.g., "landslide", "stable"), the Mohr-Coulomb rheological model was applied using the zone property command, specifying parameters such as Young's modulus (E), shear modulus (G), density (ρ), friction angle (ϕ), and cohesion (c).

For the interfaces, normal stiffness (kn) and shear stiffness (ks) were set approximately 10 times higher than those of the adjacent zones, with reduced residual friction (ϕ_{res}) and zero cohesion, to simulate weak slip planes (via the zone interface property command).

Subsequently, initial lithostatic stresses were applied (*zone initialize*), and boundary conditions were applied (*lateral fixities and gravitational model gravity 9.81*), preparing the model for stability analysis.

Chapter 10. Results

10.1. Finite Difference Method (FDM) Simulation of 'La Montagna' Landslide Kinematics: Model Architecture and 3D Computational Strategies

The 3D model was developed by integrating geological, geotechnical, and structural data collected during field surveys. Its primary objective was to accurately reconstruct the landslide's geometry, interpret its internal structure, and identify potential mechanisms of movement. This approach seeks to enhance understanding of the slope's overall evolution. Combining diverse datasets provides a cohesive view of the unstable volume and the discontinuity surfaces that affect the landslide's kinematics.

The model's implementation began with processing topographic and digital elevation model (DEM) data, which were carefully filtered and interpolated to establish an accurate reference surface. Structural elements derived from detailed geological surveys were then layered onto this foundation.

Analysing the identified discontinuities uncovered the main fracture systems and weak planes within the rock mass, providing vital insights into the orientation and spread of the key fracture sets. These findings were incorporated into the 3D model to effectively illustrate the tectonic framework and possible zones of deformation.

The results in the following sections are based on an iterative calibration and validation process.

This involved comparing numerical simulations with field evidence and historical monitoring data to develop a consistent and geomechanically plausible model of landslide behaviour. The analyses described here serve as the foundation for future stability evaluations and detailed numerical studies to understand the slope's response under various external loading conditions.

10.2. Morpho-Structural Modeling and Volumetric Characterization of the Landslide Geometry

The first step in the analysis phase was to build and characterize the geometry, then discretize the computational domain using FLAC3D software, V.7.0. This foundational stage was crucial for determining the volumes involved in the gravitational phenomenon.

As shown in Fig. 10.2, the geometry was divided into zones (mesh) to distinguish the materials' mechanical behaviour.

Notably, the potentially unstable volume is highlighted in purple and blue, while the large red area indicates the stable volume. This zone plays a crucial role as a mechanical and geometric constraint, ensuring that stresses from the moving mass are properly dissipated within the computational domain.

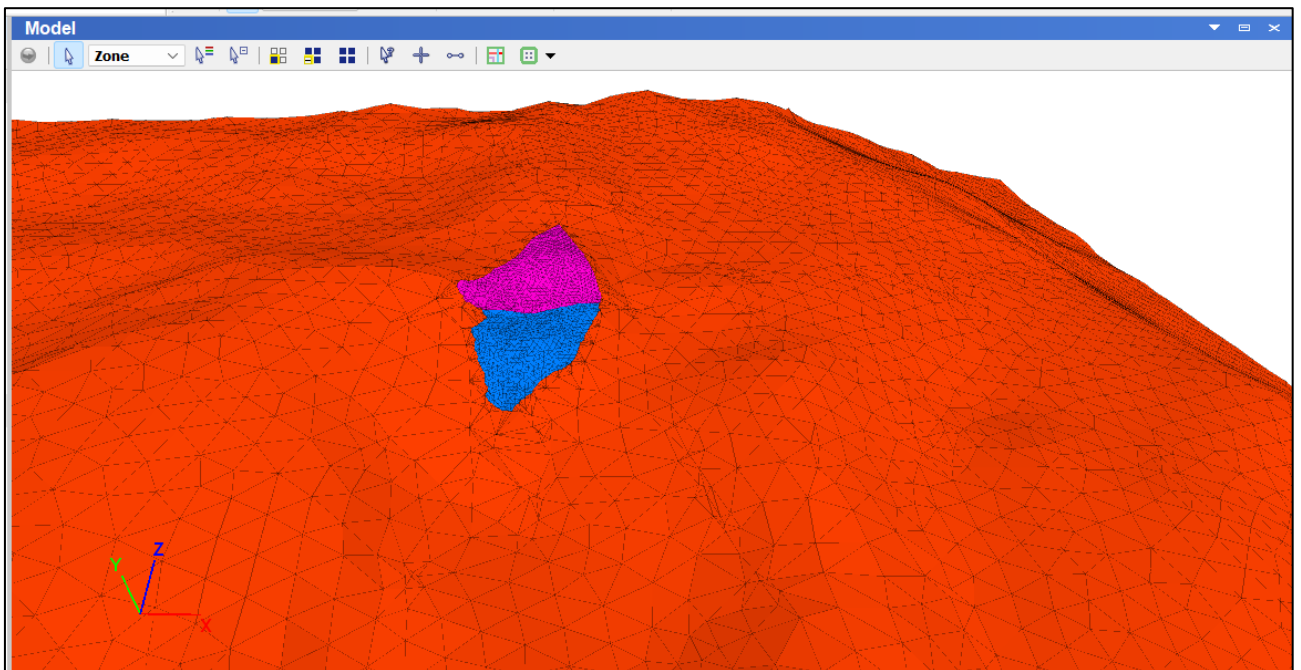


Figure 10.2 - 3D Geometry of the landslide

The geometric reconstruction process in FLAC3D was organized as follows:

- Defined surfaces, including discontinuities and ground data.
- Used the "*zone create*" command to discretize the volume between surfaces into finite elements.
- Applied group logic to manage material properties separately.
- Refined the mesh at contact zones and the landslide toe to improve numerical stability, focusing on areas with expected maximum displacements.

10.2.1 Numerical Model Implementation and Boundary Conditions

Following the geometric reconstruction, the next step was to specify the mechanical properties and boundary conditions in the FLAC3D simulation environment.

A key feature of the model is the use of numerical interfaces, which were included to capture the complex failure geometry observed in the field. These discontinuity surfaces are crucial for accurate simulation, enabling the software to represent internal plastic deformation and the sliding and detachment of the landslide body along kinematically active planes.

Specifically, integrating interfaces addresses the limitations of continuum modelling by precisely capturing the kinematic behaviour of the gravitational phenomenon and the stress distribution along the identified geological weakness surfaces.

The command zone interface, created by the *face-separate range group*, was used to introduce a physical discontinuity in the numerical model. The syntax breakdown is as follows:

- *zone interface create*: initializes the creation of a new interface, uniquely identified by a name or an ID number.
- *by-face*: defines the generation method, guiding the software to align the interface along the faces of existing mesh zones.
- *separate*: indicates the core parameter of the operation. This command directs the software to clone the nodes along the chosen surface. Before execution, the rock blocks share identical nodes, acting as a single continuous mass; afterward, creating duplicate nodes enables the two sides of the surface to move separately, mimicking sliding or detachment
- *range group*: defines the scope of the command, limiting the interface creation exclusively to faces belonging to a specific, previously defined group.

The modelling of the interfaces is represented in Figure 10.3

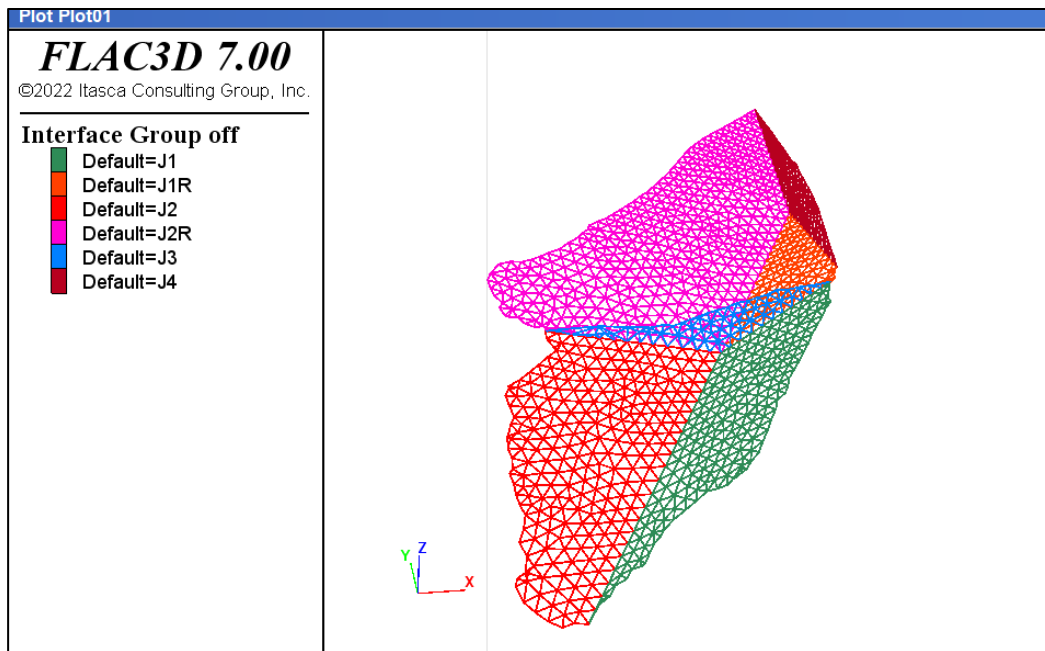


Figure 10.3 - Definition of numerical interfaces in the FLAC3D environment. The 3D model is partitioned into distinct sub-blocks (J1 to J4 and respective contacts) to simulate the mechanical behavior of the landslide discontinuities. These interface groups enable the explicit modeling of slip, separation, and friction among the geotechnical units identified during field surveys.

It is evident that the interfaces are not perfectly flat planes but irregular surfaces that follow the actual failure path.

Furthermore, mesh refinement is applied along the sliding surfaces to ensure precise stress and displacement calculations at the discontinuities. The geometric reconstruction at these interfaces effectively isolates the landslide body from the surrounding rock mass, enabling analysis of the collapse mechanism as a set of interacting blocks.

After defining the interfaces, the constitutive model was assigned to the unstable zones using *the zone cmodel assign mohr-coulomb* command. Meanwhile, an *elastic model* was applied to the stable part of the slope to improve computational efficiency.

Boundary conditions were subsequently applied to the model's base and lateral boundaries. This step is essential because it converts the

geometric coordinates into a mechanically constrained system that can react to forces.

Without fixed boundary conditions, the FLAC3D model would collapse immediately under gravity at the start of the simulation.

Using the defined geometric discretization, the model was characterized both mechanically and physically.

For the initial modelling phase, a density of 2200 kg/m^3 was assigned to the entire domain.

Before starting the dynamic analysis, the in-situ stress state was set using the *initialize-stresses command*, which created a lithostatic pressure distribution appropriate for depth and gravity. This step is essential for assessing the stress distribution along the sliding interfaces established earlier and for later stages.

10.3. Geomechanical Parameterization and Elastic Stress Initialization in FLAC3D Slope Stability Analysis

To define the numerical model in FLAC3D, structural discontinuities were modeled as mechanical interfaces that follow the Mohr-Coulomb failure criterion to replicate the slope's kinematic behavior.

The parameters used are listed in Table 12.

Table 12. Geomechanical properties and shear strength parameters (friction, cohesion, and stiffness) assigned to the discontinuity sets

ID	Type	(ϕ°)	Coesion (c) [Pa]	Stiffness-normal (kn) [Pa/m]	Stiffness-shear (ks) [Pa/m]
J1	Landslide	10	$1 \cdot 10^4$	$1 \cdot 10^8$	$1 \cdot 10^8$
J2	Landslide	10	$1 \cdot 10^4$	$1 \cdot 10^8$	$1 \cdot 10^8$
J1R	Landslide	10	$1 \cdot 10^4$	$1 \cdot 10^8$	$1 \cdot 10^8$
J2R	Landslide	10	$1 \cdot 10^4$	$1 \cdot 10^8$	$1 \cdot 10^8$
J3	Stable	40	$1 \cdot 10^5$	$1 \cdot 10^8$	$1 \cdot 10^8$
J4	Stable	40	$1 \cdot 10^5$	$1 \cdot 10^8$	$1 \cdot 10^8$

It is important to note that the initial modeling phase used the '*model solve elastic only*' command, indicating that the focus was on analyzing the system's elastic response.

In this initial phase, the primary goal is to enable the model to reach a preliminary elastic equilibrium state, alleviating gravity-induced stresses without causing plastic failure.

By temporarily preventing breaches of the cohesion and friction-angle strength thresholds, the software computes a purely elastic response using the normal and shear stiffness modules.

Due to the low strength parameters at the interfaces, an elasto-plastic solution might hinder model convergence.

Therefore, the elastic-only command serves as a preparatory phase before the stability analysis.

The outcomes of this initial elastic modeling phase are shown in Fig. 10.4. These figures show the initial stress distribution and displacement field, confirming that the model has reached numerical equilibrium before the plastic constitutive laws are activated.

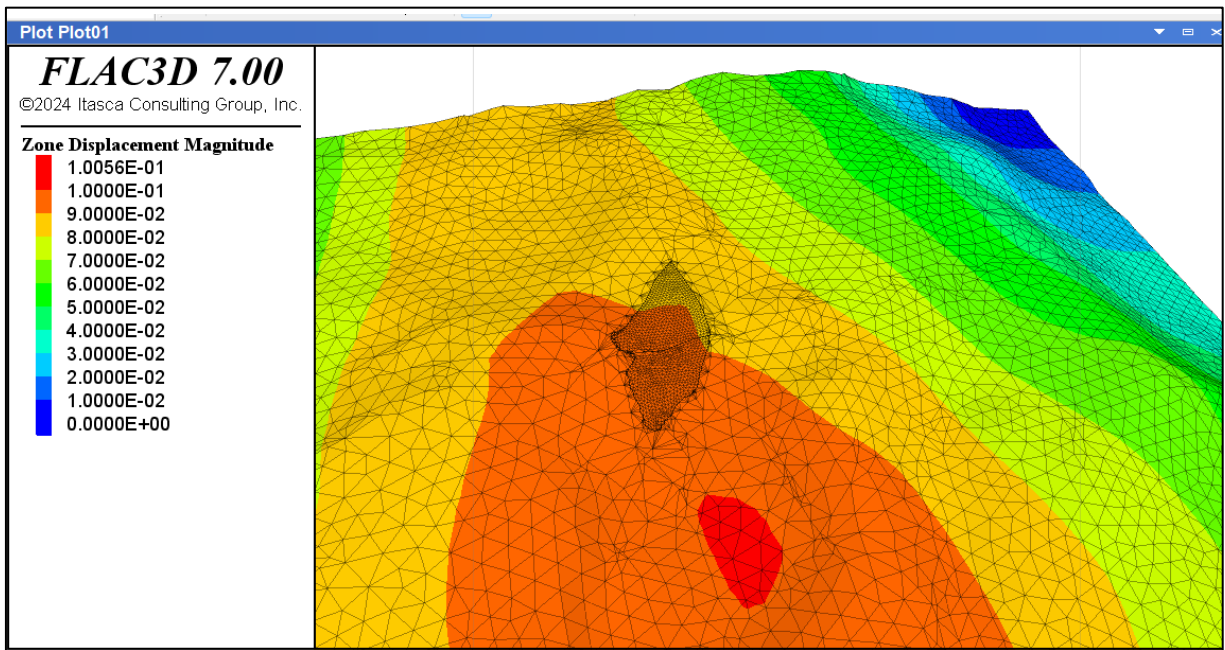


Figure 10.4 - Numerical simulation results: Zone displacement magnitude. The contour plot illustrates the distribution of total cumulative displacements within the 3D domain. The maximum movement (indicated by the red/orange contours) is concentrated in the landslide toe sector, reaching values of approximately 0.1 meters. This spatial distribution reflects the kinematically active portion of the earthflow under the simulated boundary conditions.

The achievement of equilibrium is confirmed by very small maximum displacements, approximately $1,0056 \times 10^{-1}$ m.

The color gradient also shows that zones with reduced Young's modulus (i.e., potentially unstable areas) are more deformable than the stable mass yet remain within the elastic range.

The concentration of movements in the central part of the slope supports the correct geometry of the interfaces and the consistency of the mesh.

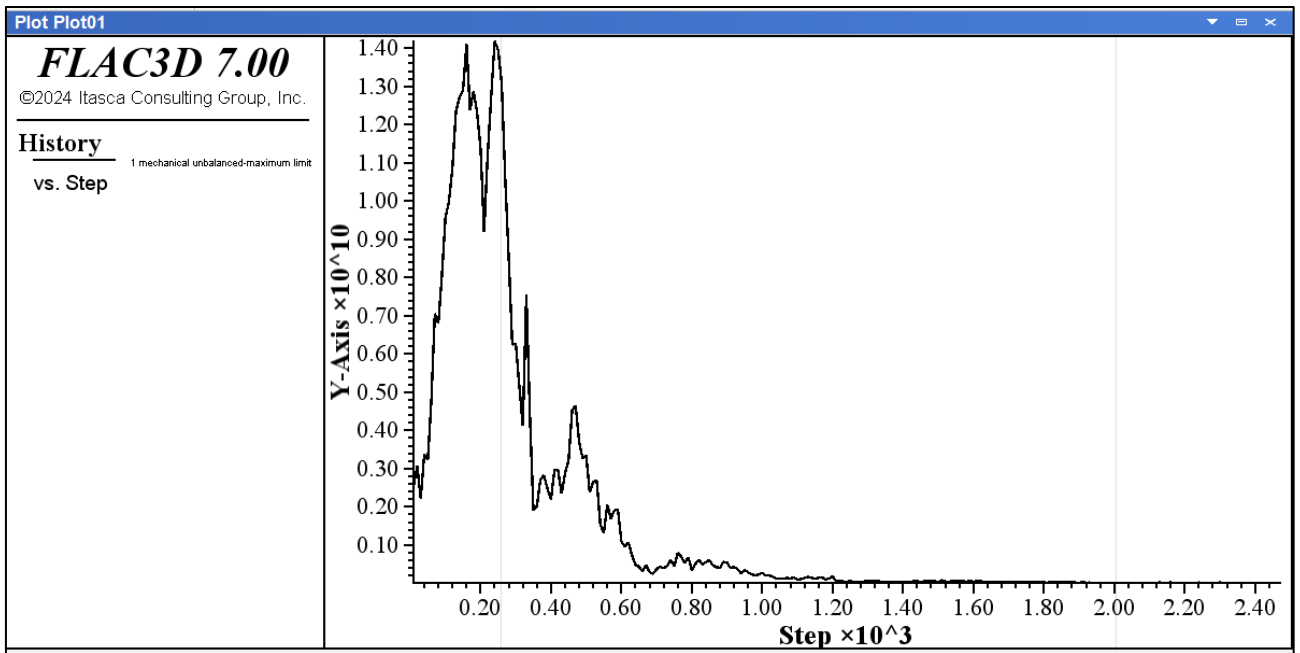


Figure 10.5 - Evolution of the maximum unbalanced force versus calculation steps during the elastic initialization phase

The graph in Fig. 10.5 shows the trend of the Maximum Unbalanced Force across calculation steps.

The curve analysis confirms the model's validity. Notably, in the initial phase (up to about 0.5×10^3 steps), there is a sharp increase in unbalanced force, driven by gravitational loading on the complex geometry.

After reaching a peak, the curve declines asymptotically toward zero, indicating that the calculation algorithm effectively dissipates kinetic energy and approaches mechanical equilibrium. The stable leveling at the end confirms the success of the "model solve elastic only" command.

Numerical convergence ensures that the displacements in Fig. 10.4 are precise and reflect the true slope settlement before plasticity occurs.

Figure 10.6 shows the distribution of vertical stresses with depth.

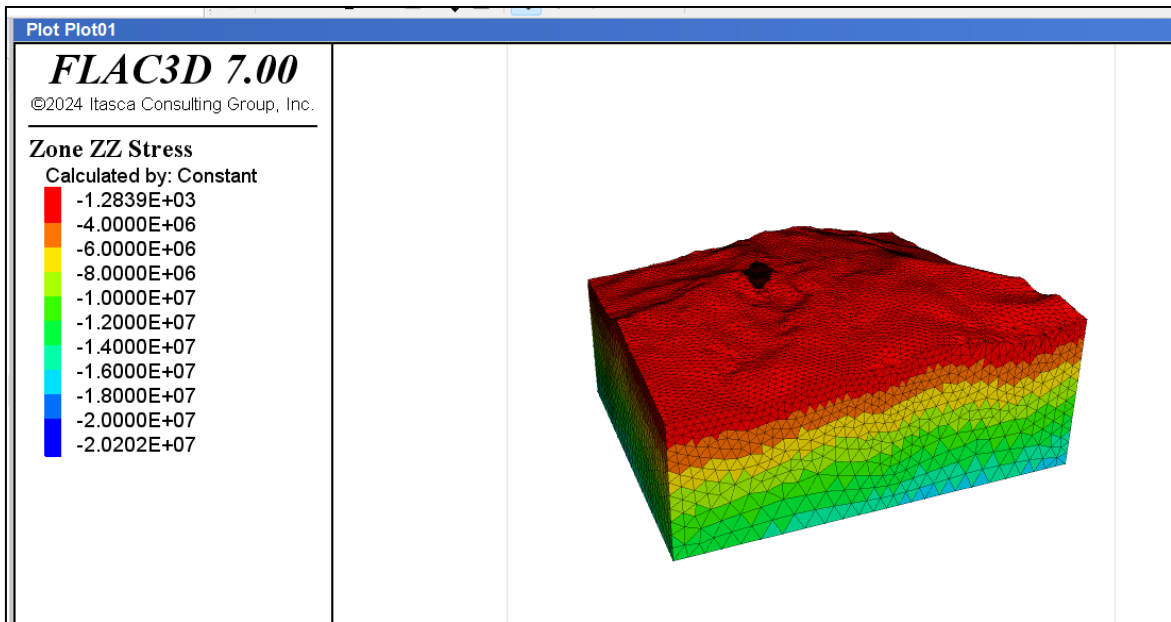


Figure 10.6 - Numerical initialization of the geostatic stress field. Perspective view of the volumetric domain illustrating the vertical stress contours. This initialization step is critical for establishing the initial equilibrium conditions before simulating landslide reactivation or pore-pressure variations.

Fig. 10.6 shows a vertical stress distribution that increases with depth, starting near zero at the surface (red zone) and reaching roughly -2.02×10^7 Pa at the bottom.

These values confirm the proper application of the gravitational field at the base of the slope. This phase is essential because it shows that the stress state has stabilized as the unbalanced force decreases, as depicted in Figure 10.5, and that the displacements shown in Figure 10.4 are purely elastic.

10.4. Numerical Implementation of Elasto-Plastic Constitutive Laws and Shear Failure Mechanisms

After completing the elastic initialization phase and confirming the correct lithostatic stress distribution and the numerical model's stability, the analysis entered the plastic regime.

Consequently, the '*elastic-only*' constraint was removed to enable the application of the Mohr-Coulomb non-linear constitutive model. This change is crucial because it provides a more precise evaluation of the slope's actual stability and helps detect potential failure zones along previously constrained interfaces.

The transition to the plastic range was implemented via a dedicated script. The procedure consisted of the following stages:

- The '*model restore*' command was used to restore the balanced stress state from the elastic phase. Subsequently, displacement vectors were reset using the '*zone gridpoint initialize displacement 0 0 0*' command to isolate the plastic deformations that developed in this phase, distinguishing them from the initial elastic settlements.
- Zone Property definition: the material properties were assigned to the zones.
- Interface parameterization: the interfaces were parameterized.
- Convergence optimization involved setting local damping with the '*zone mechanical damping local 0.5*' command before the final '*model solve*', aiming to improve convergence toward equilibrium and identify failure mechanisms.

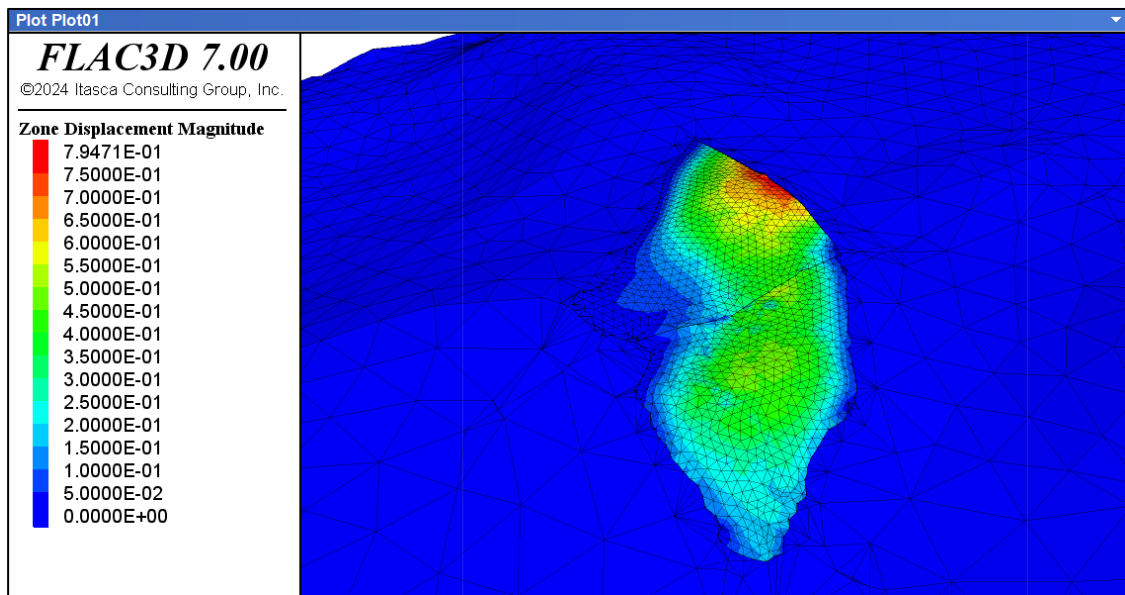


Figure 10.7 - Contour plot of Zone Displacement Magnitude at the end of plastic analysis

Figure 10.7 shows the zone displacement magnitude, indicating the volume of the currently mobilized landslide. The contour map highlights several important features:

- The red area represents the zone of maximum displacement (7.9471×10^{-1} m).
- The green and yellow areas indicate intermediate displacement levels.
- The blue area represents a stable portion of the model, where displacements are close to zero.

10.4.1. Displacement along the x-component

The analysis of the X-component of displacement focuses on the lateral movement of the unstable mass (Fig.10.8).

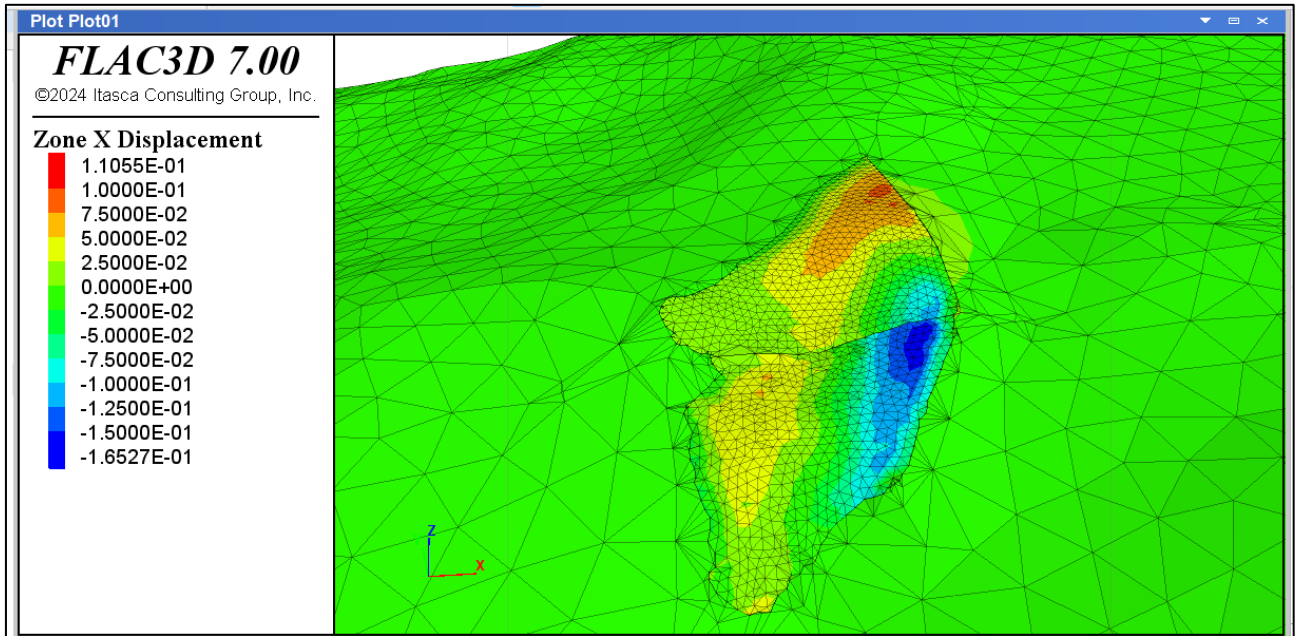


Figure 10.8 - Contour plot of zone x-Displacement

The distribution shows uneven movement, with values ranging from 0.11 m to -0.16 m.

The dominance of negative X-displacement (blue) indicates that the landslide primarily drifts laterally toward the negative X-axis.

This pattern of differential displacement clearly indicates the landslide's asymmetric behavior, shaped by the specific dip and dip-direction of the two intersecting interface planes.

These findings confirm that the failure is not solely translational but is also influenced by the 3D geometric configuration of the discontinuities.

10.4.2. Displacement along the y-component

The displacements along the Y-component are evaluated below (Fig. 10.9).

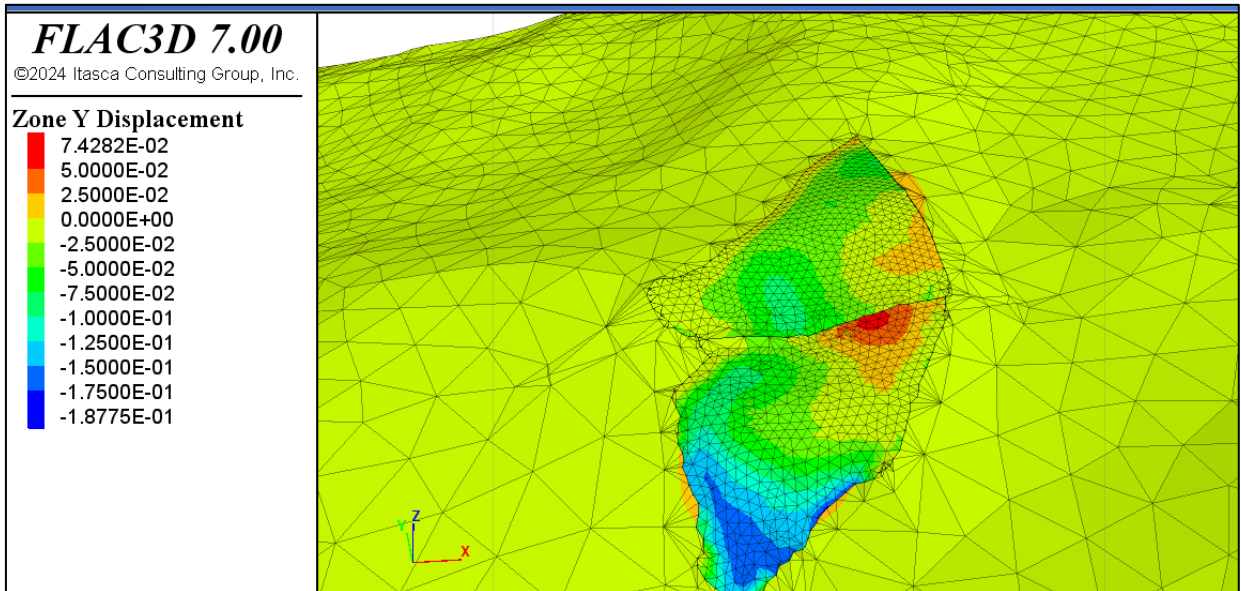


Figure 10.9 - Contour plot of zone y-Displacement

The scale of values shows displacements from roughly +0.007 m (red) to around -0.187 m (blue).

The upper-right part of the block shows positive or near-zero values (green/yellow), whereas the lower-left toe displays stronger negative values. Comparing the displacements along the Y and X components shows that the hypothesized wedge is not sliding directly down the maximum slope line. Instead, its movement is influenced by the intersections of the interfaces, which will be discussed later.

10.4.3. Displacement along the z-component

The displacements along the Z-component are evaluated below (Fig. 10.10).

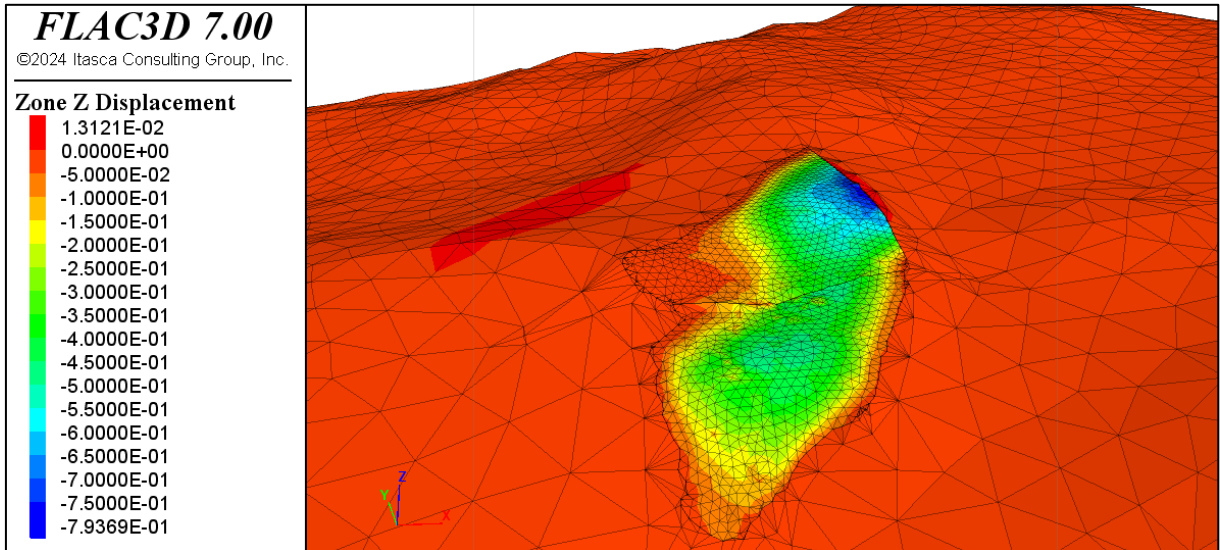


Figure 10.10 - Contour plot of zone z-Displacement

Assessing the vertical component is essential for understanding how gravity impacts the unstable mass and for determining the wedge's settlement along the failure surfaces.

Figure 10.10 shows that nearly all the mobilized mass undergoes negative displacement, indicating downward movement consistent with a gravity-driven landslide.

Additionally, the maximum vertical displacement is approximately -0.794 m (shown in blue).

This closely aligns with the previous total magnitude of approximately 0.795 m, indicating that the vertical component is dominant in this sector.

10.5. Kinematic Behavior and Shear Displacement along Structural Discontinuities

Once the kinematic behavior of the moving mass was defined through displacement analysis, it was necessary to investigate how structural discontinuities influence the failure mechanism.

Specifically, this section evaluates the behavior of the previously defined interfaces, which were assigned reduced strength parameters to simulate natural joint planes.

The objective is to verify how stresses and shear displacements localize along these surfaces and whether the hypothesized wedge movement is effectively governed by sliding along the planes with the lowest friction angle.

The analysis of the Interface Shear Displacement quantifies the magnitude of sliding along the structural discontinuities (Fig. 10.11).

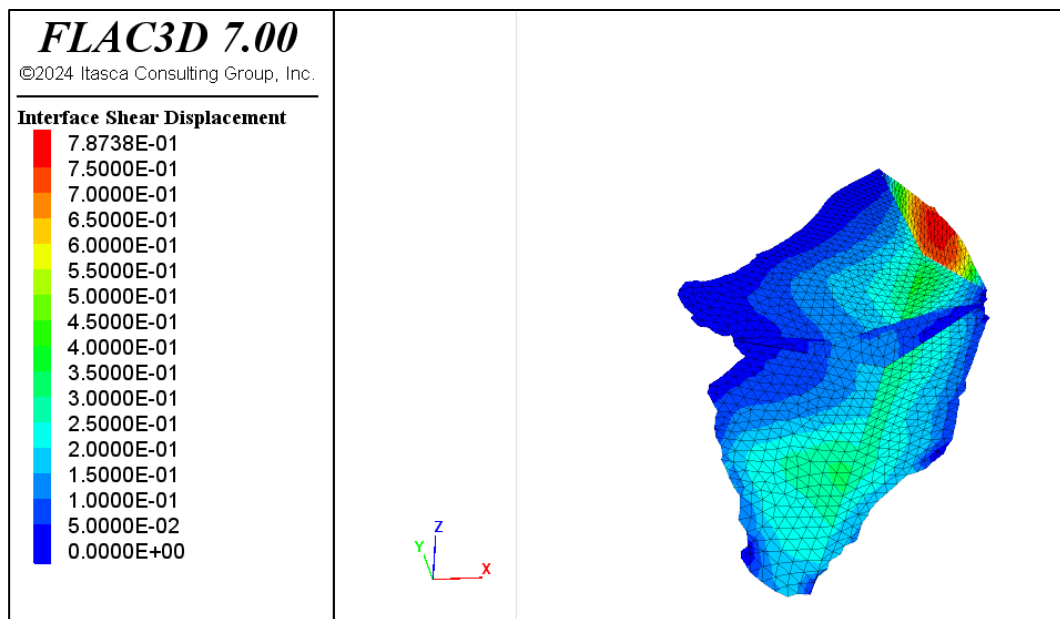


Figure 10.11 - Visualization of the cumulative shear displacement across the landslide sub-blocks. This output validates the mechanical role of the interfaces in accommodating the overall downhill movement of the landslide mass

The maximum value, about 0.787 m, is located on the upper part of the right interface and directly correlates with the previously observed total displacement magnitude in those areas.

The shear distribution is uneven, with the right plane showing notably higher displacement than the left.

Given these results (particularly the intersecting geometry of the planes, the localization of shear along these surfaces, and the asymmetry in the X, Y, and Z displacement components), a wedge-type failure mechanism can be hypothesized.

This kinematic interpretation is supported by the observation that the entire mobilized mass moves as a single block, with its path precisely defined by the intersection of the two main discontinuity planes.

To verify the above description, a displacement graph was generated for the point on the interface with the maximum stress (Fig. 10.12).

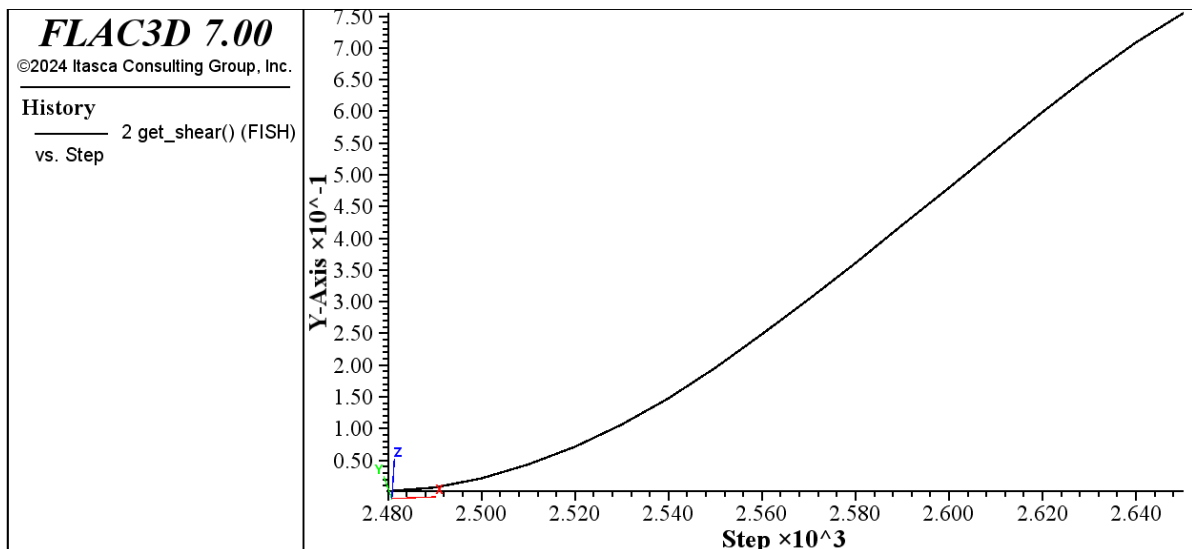


Figure 10.12 - Evolution of shear displacement over computational steps. The history plot, generated via the “get-shear” FISH function, shows the cumulative shear displacement during the numerical analysis. The upward curvature of the plot indicates a transition toward an accelerating creep phase, suggesting that the model is approaching a state of numerical instability or failure under the applied conditions

Between steps 2480 and 2520, the curve shows concavity upward, indicating the block is accelerating.

After step 2540, the trajectory becomes a straight line with a steady slope, indicating an active collapse mechanism in which the wedge moves at a constant speed. The y-axis confirms that displacement exceeds 0.740 m and increases linearly, indicating full mobilization of the mass.

The stress graph has been analyzed (Fig. 10.13).

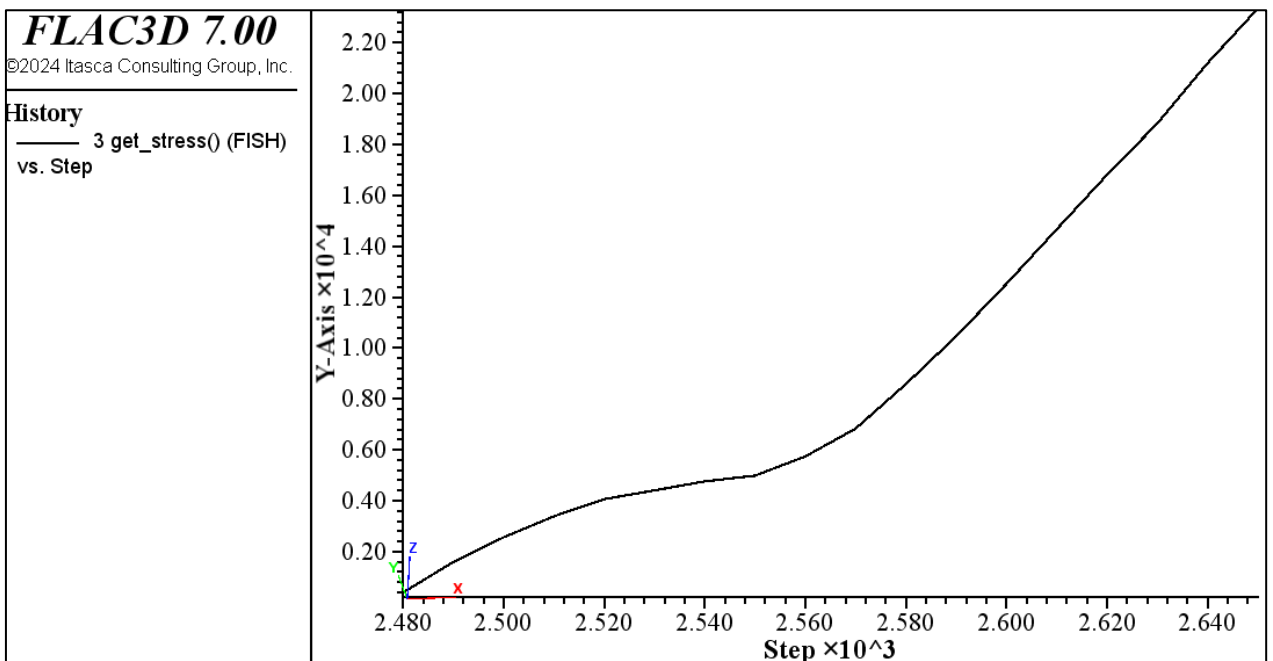


Figure 10.13 - Evolution of mechanical stress during the simulation. The plot displays the stress magnitude (calculated via the `get_stress()` FISH function) on the Y-axis ($\times 10^4$) against the simulation steps on the X-axis ($\times 10^3$)

Notably, the stress component reaches approximately 0.40×10^4 Pa, shows a slight inflection point, and then rises sharply to 2.20×10^4 Pa. This is noteworthy because stress usually levels off during failure, yet it continues to increase here. This can be explained by the wedge's fall, which increases pressure on the lateral walls of the discontinuities.

10.6. Limit State Analysis and Factor of Safety (FoS)

Determination via the Strength Reduction Technique

To assess the stability conditions of "La Montagna" slope, advanced numerical modelling was conducted using FLAC3D 7.00. Given the geometric complexity and the presence of low-friction interfaces, the Strength Reduction Method (SRM) was employed to determine the global Factor of Safety (FoS). A tailored FISH script was developed to reduce shear strength parameters ($\tan\phi$ and c) at interfaces, ensuring numerical convergence.

10.6.1. Numerical results and quantitative assessment

The stability analysis yielded a calculated Factor of Safety (FoS) of 1.02. This near-unitary value is technically significant:

- **Limit Equilibrium:** a FoS of 1.02 shows that the slope is at the verge of failure. This clearly explains the landslide's ongoing sensitivity to small environmental triggers (like rainfall) and its past intermittent movement.
- **Failure Mechanism:** the model reached a state of "active failure" where the reduction of friction triggered a clear kinematic release. The displacement vectors confirm that the movement is entirely governed by the basal interfaces J1-J2 and J1R-J2R.

10.6.2. Interpretation of Displacement and Plasticity

The analysis recorded a maximum displacement of 0.10 m (10 cm) after a 2000-step calculation phase. It is important to interpret this value not as a stabilized settlement but as the beginning of kinematic mobilization.

Kinematic release: recording 10 cm of movement, along with the absence of numerical convergence at lower strength ratios, indicates that the multi-block system has exceeded its critical threshold.

Role of Tension Cracks: displacement begins only after the sub-vertical interfaces J3 and J4 (acting as posterior release surfaces) reach a tensile failure state. This shows that the "vertical" intersections identified by the reviewer are not sliding tracks but disconnection points that enable the blocks to slide along the flatter basal planes.

Plastic Zone Distribution: as shown in Figure 9.14, the model's plastic state confirms that shear failure is strictly concentrated along the pre-defined interfaces. The "shear-now" and "tension-now" indicators (red and magenta zones) align perfectly with the basal sliding planes and the posterior release surfaces, while the internal blocks remain in an elastic state. This provides visual evidence that instability is structurally controlled (anisotropic).

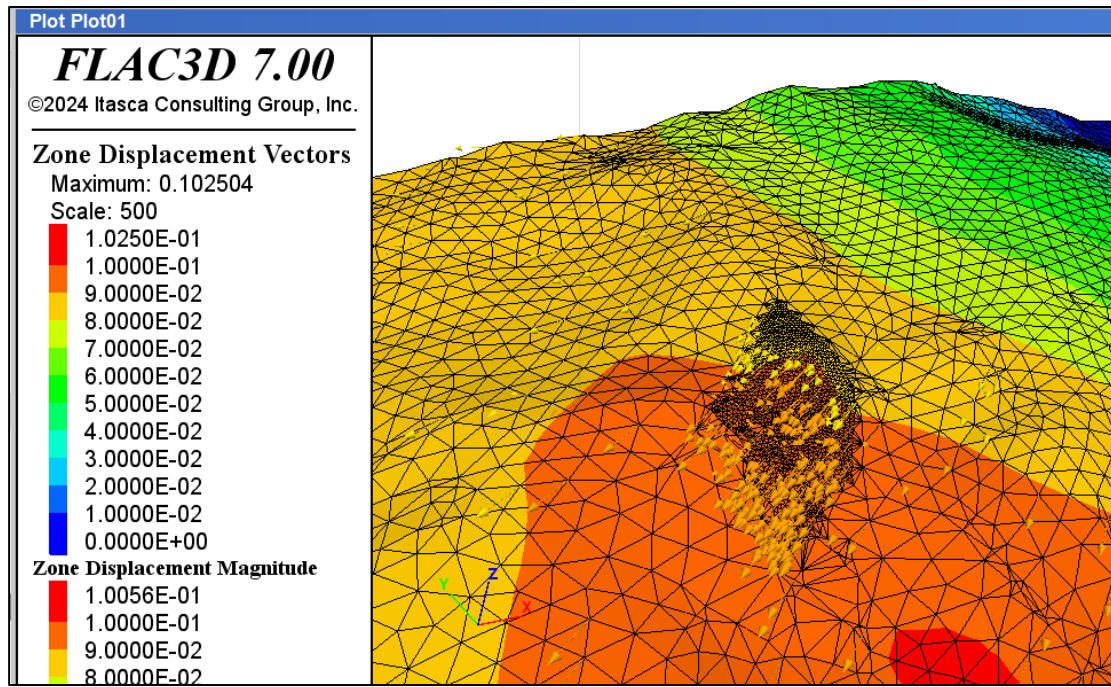


Figure 9.14 - Plasticity indicators and failure zones in the FLAC3D multi-block model. The concentration of shear and tensile failure along the interfaces (J1–J4) validates the structurally controlled kinematic mechanism.

11. Discussion of Results

11.1. Analysis of Numerical Transition and Model Validation

The three-dimensional numerical modelling of the "La Montagna" landslide is an essential advancement from the initial 2D Limit Equilibrium Method (LEM) analyses. While 2D models tend to underestimate stability by neglecting lateral confinement effects, the FLAC3D discretization effectively captured the spatial interaction among different sets of discontinuities.

The consistency between the geological-structural mapping (Chapter 7) and the model's mechanical response (Chapter 9) confirms the validity of this approach. The shift from a continuous medium to a discrete block system accurately represents the nature of the Apennine flysch formations, in which overall behavior is dictated by interface kinematics rather than rock-matrix strength (Eberhardt et al., 2004).

11.2. Mechanical Significance of the Factor of Safety (FoS

1.02)

The calculated FoS value of 1.02 indicates the slope is at the critical limit equilibrium. In engineering terms, a 2% safety margin is effectively negligible given the natural uncertainties in geotechnical analysis and the estimation of strength parameters (ϕ , c).

This value quantitatively explains the "intermittent mobility" observed historically: the system appears stable in dry conditions but is not resilient to external disturbances. According to the literature on flysch (Cascini et al., 2010), these systems continually operate near a failure threshold, where exceeding peak strength along basal interfaces is already underway.

11.3. Fracture Hydrodynamics and Meteorological Triggers

A key aspect of the discussion is the role of the sub-vertical interfaces J3 and J4. In the model, these are not just geometric boundaries; they also function as open tension cracks.

- **Hydraulic Bypass:** During severe weather events, these cracks serve as preferred drainage paths that directly connect the surface to the deep sliding planes (J1-J2).
- **Multiplier Effect:** Water filling the vertical fissures applies a horizontal hydrostatic thrust (water pressure thrust) that supports the reduction of effective pressure on the base.
- **Impulsive Response:** Unlike continuous models, where infiltration occurs slowly (Darcy's law), the response here is sudden. The pore pressure peak on basal surfaces can occur within hours of a storm's onset, instantly dropping the FoS below 1.0 ($\text{FoS} < 1.0$).

11.4. Kinematic Interpretation of the Displacement

The 10 cm displacement observed in the model should not be viewed as a final equilibrium state but as an indication of yielding. In rock mechanics, this value indicates that the system has reached its elastic limit.

Once kinematic mobilization is triggered, the resistance along the basal interfaces undergoes strain-softening, decreasing from peak to residual values. This transition is especially dangerous in flyschoid formations, as the large difference between peak and residual strength can lead to a sudden acceleration of the mass, which the model identifies as "active failure."

11.5 Vulnerability Analysis of Strategic Infrastructure

The 3D model enables overlaying displacement vectors onto precise coordinates of downstream infrastructure, outlining different risk scenarios.

11.5.1. State Road 90 Highway (SS 90 Highway)

The SS90 bis is in a highly vulnerable position, precisely along the zone of maximum expansion of the unstable wedge. The kinetic energy estimated for a mass moving at a FoS of 1.02 suggests that current rockfall barriers or surface containment measures could be bypassed or destroyed. A sudden activation could cause the complete closure of the road, cutting off connections between Campania and Puglia, with economic damage estimated at millions of euros per week of disruption.

11.5.2. Benevento-Foggia Railway Line

The risk to the railway line is more subtle but equally important. Even if a catastrophic collapse doesn't occur, the model shows how plastic deformations can spread toward the railway embankment. Differential settlements due to slope movement can lead to track misalignment, which may exceed safety tolerances for train operations. This requires real-time monitoring because passenger safety depends on detecting millimeter-level displacement before it grows to centimeters.

11.6. Proposed Mitigation Strategies

Based on the numerical results, three intervention strategies are discussed:

- **Deep Drainage:** for effectiveness, sub-horizontal drains must physically intercept the J1-J2 plane at the depth specified by the 3D model. Superficial drainage would have little impact on overall stability.
- **Crack Sealing (J3-J4):** a low-cost, priority intervention involves sealing tension cracks at the crown to minimize water inflow during rainfall.
- **Active Anchors:** the use of tie-backs or large-diameter piles must include an anchoring length greater than the depth of the modeled basal interface to prevent the "floating pile" effect within the unstable mass.

Conclusions

The integrated geomechanical study of the "La Montagna" landslide has successfully bridged the gap between detailed field structural mapping and advanced 3D numerical modeling. By achieving the primary research objectives, this work provides a definitive mechanical framework for a complex multi-block failure that traditional 2D analyses could not fully characterize.

a) Validation of the Multi-Block Kinematic Model

The research confirms that the landslide's behavior is governed by a 3D wedge-failure mechanism, in which stability is determined not by the overall strength of the rock mass but by the anisotropic properties of the identified structural interfaces (J1–J5). The observed asymmetry in displacement components and the concentration of shear stresses (up to 2.20×10^4 Pa) during the failure phase provide quantitative evidence of this structurally controlled movement.

Specifically, the modelling results achieved the following milestones:

- **Mechanical Traceability:** the shift from geological "Faults" to numerical "Interfaces" ensured that field observations were directly translated into mechanical behavior, confirming the J3-J4 interfaces as essential posterior release surfaces (tension cracks).
- **Limit Equilibrium Confirmation:** the calculated Factor of Safety (FoS) of 1.02 acts as a numerical benchmark for the slope's "intermittent mobility." It shows that the system is in a near-unsafe state, which explains why minor environmental triggers can cause sudden reactivation.
- **Yielding Indicators:** the recorded interface shear displacements (~10 cm) are interpreted not as a stabilized state, but as the onset of kinematic mobilization, indicating the point where the system exceeds its elastic threshold and enters failure.

b) Practical Implications for Infrastructure Risk Management

Beyond the theoretical contribution, this study provides significant practical relevance for managing the SS90 bis State Road and the Benevento-Foggia railway line.

- **Predictive Tool:** the 3D model locates the exact failure surfaces (J1-J2) and the "trigger" zones (J3-J4). This enables infrastructure managers to shift from reactive to proactive maintenance by focusing on sensors that measure the dilation rates of the rear tension cracks.
- **Design Support:** The results offer a clear pathway for mitigation. Any future stabilization efforts (such as deep sub-horizontal drainage or active anchoring) must focus on the specific depths and orientations of the basal interfaces identified in this 3D discretization to be effective.

c) Future Developments

To further refine this predictive tool, future research should include transient pore water pressure analysis:

1. **Dynamic Pore Water Pressure Analysis.** The research will focus on dynamic analysis of pore-water pressure, moving beyond steady-state evaluations. A transient 3D coupled hydro-mechanical model will be implemented to investigate the generation and evolution of pore water pressure during extreme rainfall events. This approach will enable precise simulation of the temporal degradation of the Factor of Safety (FoS).
2. **Complex Hydro-Kinematic Bypass Modelling.** The structural complexity of the subsurface will be addressed through advanced hydro-kinematic modelling. Specifically, discrete fracture modelling of the J2-J4 joint intersections will be performed, as illustrated in the conceptual diagram. The integration of depth-

dependent permeability is planned to accurately reflect the rock mass's hydraulic behaviour.

3. **Process-Based Early Warning Thresholds.** These analytical results will be operationalized through the development of process-based early warning thresholds. Rather than relying exclusively on cumulative rainfall, the system will integrate thresholds based on displacement velocity and pore pressure, cross-referenced with rainfall intensity-duration curves. This methodology aims to significantly reduce false alarms and enhance the reliability of the warning system.
4. **Multi-Sensor Data Assimilation and Validation.** Lastly, data assimilation and validation will be carried out to ensure model accuracy. Real-time field data from monitoring networks (specifically inclinometers and piezometers) will be integrated into the workflow. This continuous feedback loop will enable ongoing model calibration and ensure a rapid, data-driven response to any detected instability.

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Appendix

Title: “Multi-temporal evolution analysis of Maronti Cliff (Ischia Island, Italy) derived from multi-perspective photographic datasets”

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INTRODUCTION

Past research reveals that instability occurrences of natural and engineered slopes are influenced by a range of factors including extensive human activities (*Sidle et alii, 2004*), improper land use (e.g. unauthorized construction works within marginally stable slope) (*Parisi & Sabella, 2017; Luo et alii, 2019*) and climate and its variations (*Gariano & Guzzetti, 2016*). Coastal areas are naturally dynamic and subject to critical variation over time due to a range of preparatory and triggering factors. In particular, sea cliff environments are often prone to several erosional processes able to constantly shape the coastal landscape. Natural phenomenon coupled with human-induced effects has significant consequences on subvertical coastal environments. *Sunamura (1992)*, reports that problems of average cliff recession rates in excess of 1 m/year are experienced at coastal sites in Denmark, Germany, Russia, Japan, New Zealand, Canada, the UK and the USA. Quantifying volumes and revealing patterns of cliff edge and face evolution, can provide critical information to understand the processes triggering cliff failures. Traditional field surveys for in situ direct measurements in sub-vertical coastal contexts are challenging for the operators and require time-consuming efforts to capture numerous data to ensure spatial representativity. In the last decades, remote sensing techniques have proved to be efficient methods to analyze change along coastal areas over time (*Parente et alii, 2015; Apostolopoulos & Nikolakopoulos, 2021*).

The use of long-range remote sensing techniques (e.g., satellite data, airborne SAR interferometry and airborne laser ranging) can be adopted to successfully provide large-scale monitoring such as the estimation of annual retreat rate of long stretches of coastline (*Sun et alii, 2023*). For the assessment of the activity patterns of a specific cliff sector the adoption of close-range solutions is a more suitable solution (*Lim et alii, 2005*).

Such techniques including TLS and photogrammetry have proved their potential to understand and quantify evolution of the entire cliff section (*Young et alii, 2010; Letortu et alii, 2018; Parente et alii, 2019*). For highly dynamic environments such as coastal cliffs, increasing the frequency of surveys can facilitate the estimation of change. Also, monitoring cliff change over a range of spatial and temporal scales requires utilizing a suite of historical and modern data. However, datasets for poorly studied cliff sections are often unavailable. A solution may be considering photogrammetry coupled to crowd-sourced photos and videos captured by amateurs (*e.g., Alsadik, 2020; Rihani, 2023*). The availability of photos for touristic sites is a considerable resource for research purposes although such photographic material often lacks accurate location information and can vary in quality depending on the acquisition sensors and platform used (*e.g., smartphones, digital cameras, drones, etc.*). However, the use of low-cost sensors such as smartphones can successfully perform valid SfM-based ('Structure from Motion') photogrammetric restitutions (*Micheletti et alii, 2014; Jaud et alii, 2019*). SfMphotogrammetry allows to generate 3D models (such as point clouds) that can be used to quantify topographic and volumetric changes over time to better understand multi-temporal geomorphic variations in a range of environments including seacliff (*Yu et alii, 2022*).

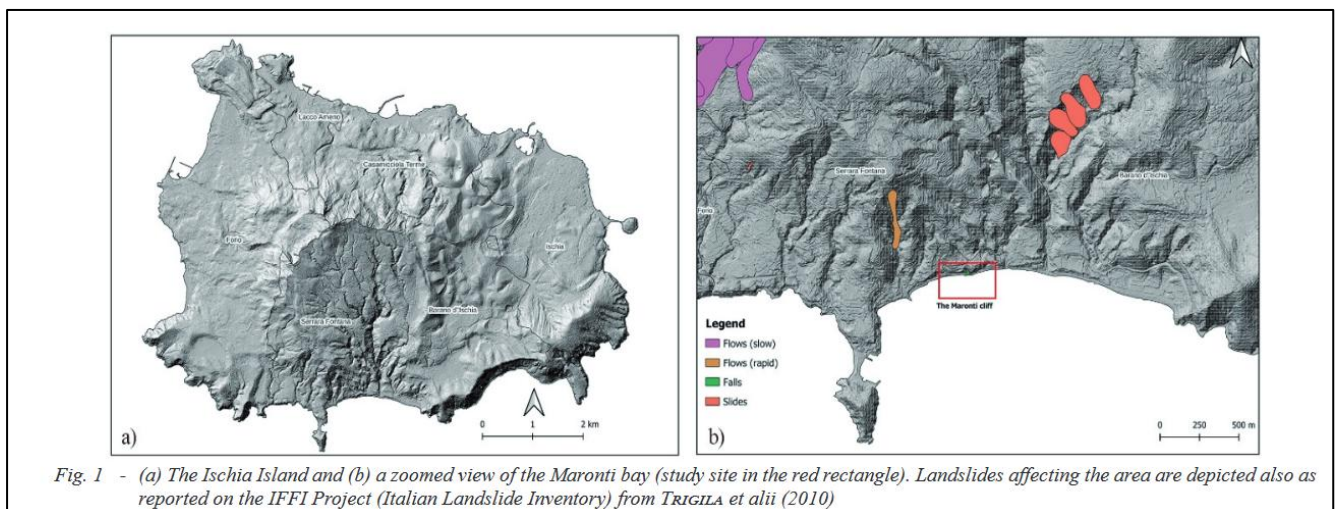
Furthermore, manual and automated pipeline available in modern photogrammetric software can be adopted to automatically generate aligned multitemporal 2D and 3D models (*Turner et alii, 2015; Feurer &Vinatier, 2018; Cucchiaro et alii, 2020; Parente et alii, 2021*). The site considered for this study is a portion of cliff situated along the Maronti cliff on the Ischia Island (Italy). Information regarding erosional events at Maronti cliffs are limited. To the best of the authors knowledge the only scientific work at Maronti consists in the estimation of the bay erosion modelling considering the sediment transport dynamics (*Giordano et alii, 2006*).

Scientific information regarding the multitemporal morphological evolution of the cliff is not available. The purpose of the present work is to use multitemporal and multiperspective photographic datasets acquired from both, amateurs (*crowdsourced photos - Guerin et alii, 2020*) and experienced operators (topographic survey), to estimate preliminary data about the evolution of a highly unstable portion of the

Maronti cliff. Furthermore, the work evaluated the ability of two different 3D models co-registration procedures to quantify the erosion rate and discuss opportunities for future work.

STUDY AREA

Ischia Island (Fig. 1a) is located in the Gulf of Naples (eastern Tyrrhenian Sea margin, Italy), with an area of 46.3 Km² and is dominated by the Monte Epomeo (787 m a.s.l.), in its central portion and by the NE-SW Monte Vezzi - Monte Cotto alignment of peaks, in the SE corner. The investigated site is situated along the Maronti bay (Fig.24b), a two kilometres long pocket beach extending between two small peninsulas namely Sant'Angelo and Capo Grosso (from west to east). The study site is a small portion of cliff located in the central part of the Maronti bay affected by evident recent erosional processes including rock falls and rock topple (Fig.1b).



GEOMORPHOLOGICAL SETTINGS

The island represents the merged portion of a large volcanic field, covering at least 300 Km² in which eruptive mouths, calderic structures and resurgent structures have developed (*ISPRA, 2009*). It lies at the end of the continental shelf that delimits the northern and the central sectors of the Apennine chain. From north to east, the platform is connected to the coastline and connects Ischia, Procida and the Phlegraean volcanic district. To the northwest it descends toward the Gulf of Gaeta basin while to the south it deepens toward the deep basin of the Tyrrhenian Sea (*Vezzoli, 1988*). The analysis of historical sources and the results of archaeological studies not only reveal the fragility of the area, but also show that recent geological development, characterized by eruptions, earthquakes and landslides, has influenced the distribution of human settlements (*Guadagno et alii, 1995*). From a geological point of view, the island is mainly composed of volcanic rocks and landslide deposits and subordinately of sedimentary terrigenous rocks, whose presence is to be linked to the alternating between complex eruptive phases and slope instability (*De Vita et alii, 2006, 2010; Della Seta et alii, 2012*). From the oldest to the youngest, the main geological units are represented by the Ancient Ischia, by the San Nicola synthem, by the Buceto synthem. The coastline of the Ischia Island has undergone significant changes in the past due to the important volcanic and tectonic evolution which have been associated phenomena of slope instability that have caused the retreat of the coastline on the order of hundreds of meters (*Del Prete & Mele, 1999*). From a geological point of view the investigated site is characterized by the presence of the Maronti Formation. Over the last years this coastal sector has experienced multiple natural and anthropogenic events that have shaped the general morphology and bathymetry of the area (e.g., extreme storms, nourishments, cliff cuts, installation of mitigation measures, etc.).

National newspapers have reported on the various events that have affected the cliff since 1970 (*Popoli et alii, 1978*). In particular, the studied cliff portion appears to have undergone multiple phenomena affecting its stability. The first evidence of instability was reported on 7th June 1978, which caused the death of five people and

three injured. Newspaper reports that the 20-meter-high cliff suddenly collapsed shortly after 12 noon (Fig.2).



Fig. 2 - The Maronti landslide, June 8, 1978 (extracted from: "Il Mattino")

Another significant event is the one in August 1983, again documented by National newspaper when several hundred cubic meters of debris rock collapsed, permanently burying a factory that had already been closed due to previously occurring phenomena. Recently, two main instability events occurred on December 15, 2020, and November 22, 2022. The 2020 event caused the edge of the cliff to approach the road and homes in the immediate vicinity while the 2022 collapse completely destroyed a long stretch of the road and seriously damaged neighbouring properties. The current condition of this portion of the cliff is well represented in Figure 4.

MATERIALS AND METHODS

DATASETS

The set of photos used in this study were collected in 2011, 2021 and 2023 from a range of different sensors and for different purposes. A detailed description of the used datasets is reported in Table 1.

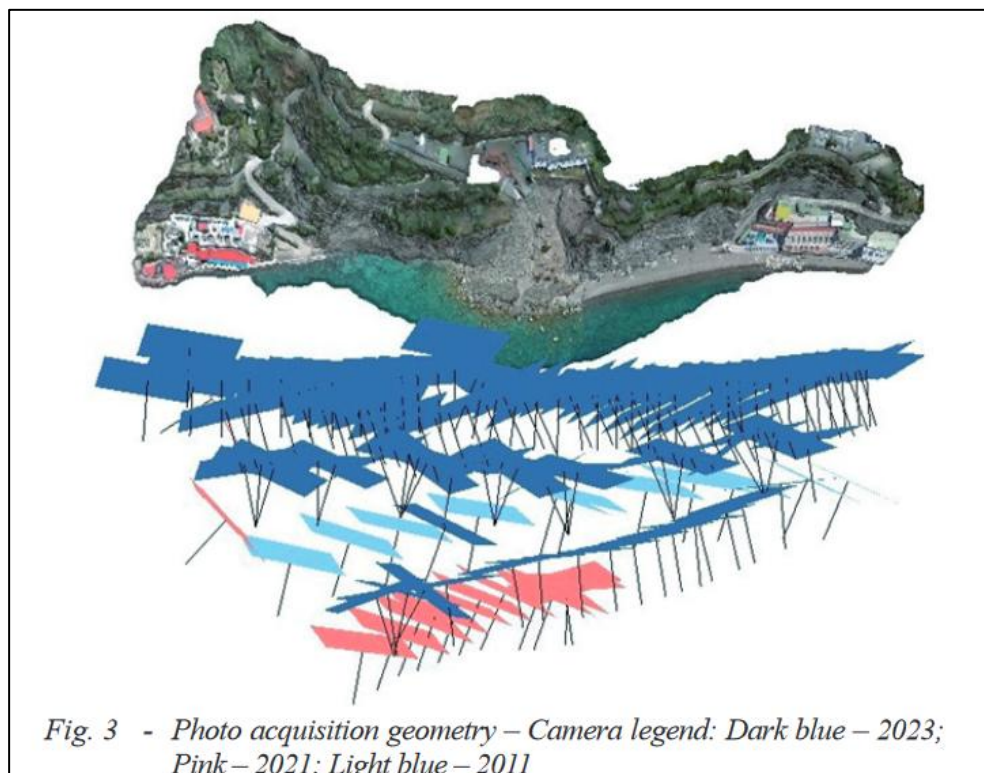
Year	September 2011	July 2021	May 2023
Acquisition platform	DSLR	Smartphone	Drone
Camera	Nikon Coolpix P100	Huawei FIG-LX1	Autel Evo 2 Pro
Number of photos	11	12	92
MP	10.3	8	
Distance from the object	Range: ~50-100m	Range: ~50-150m	Range: ~40-100m
Purpose of acquisition	Tourism	Tourism	Monitoring and inspection
Topographic surveys	-	-	GNSS station (6 GCPs)
Weather conditions	Sunny	Sunny	Sunny

Tab. 1 - Details about the acquisition of each photographic dataset

Summarizing, sets of photos acquired on 17th September 2011 and 20th July 2021 were collected from a boat at a range of distances and from different angles of view. A total of 11 and 15 photos formed the 2011 and 2021 datasets, respectively. Although the previous set of photos were not intended for photogrammetric processing, the acquisition geometry, spatial coverage and quality of images were suitable for the photogrammetric Structure-fromMotion workflow adopted in this study. The drone-based survey was carried out on 23rd May 2023 with the Autel Evo 2 Pro Enterprise RTK equipped with a RGB camera (note: the drone was not equipped with the RTK module at the time of acquisition).

The drone-based data collection was conducted manually by an expert operator that captured a total of 92 images of the site from a range of perspectives flying approximately from 40 to 100 m from the centre of the investigated cliff. Flight height and distance from the cliff were not constant during the acquisition, thus a GSD (Ground Sample Distance) value cannot be estimated. Image acquisition geometry of each dataset was reconstructed as part of the SfM camera alignment process and is reported in Figure 3 further dataset collected before flying the drone, includes the position of six photogrammetric targets well distributed on the study site (Fig.26). The targets were used as ground points to ensure the correct georeferencing of the photogrammetric outputs. The position of their centres was estimated using the GNSS positioning technique (Fig.4).

A South Galaxy G7 GNSS receiver allowed determining the spatial position of a total of 6 targets using the RTK (Real Time Kinematic) method, with an accuracy of up to 2 cm, depending on the number of visible satellites, GSM signal and corrections from a base (Topcon NETG3 receiver) located on Ischia Island (municipality of Barano d'Ischia) a few km away from the study site.



PROCESSING AND 3D MODELS COREGISTRATION

The imagery captured at the sea-cliff site was processed using a classic SfM-MVS pipeline. The SfM-MVS photogrammetric processing was implemented entirely in Agisoft Metashape v.1.7.4 (*AgisoftMetashape, 2023*).

For each set of images, a five-step procedure was used to generate a dense point cloud. These steps included: loading the four images into the software; estimating camera pose and photos orientation via the ‘SmartMatch’ tool; georeferencing the sparse point cloud in a global coordinates system using the photogrammetric targets coordinates previously measured with a GNSS receiver (only possible with the 2023 dataset); optimizing the model and minimizing the error in all marked points, 3D points and camera station positions and angles; and increasing the point cloud density via the MVS algorithms and rendering the model with RGB values.

All processing parameters were kept constant for the generation of each 3D model. The absence of photogrammetric targets on the imagery collected in 2011 and 2021 lead to the generation of dense point clouds in an arbitrary coordinate system. Also, the models were not correctly scaled. Therefore, in order to conduct a proper multitemporal cloud-to-cloud analysis, it is necessary to adopt a solution for transforming all 3D models and 2021 as ‘compared’ ones.

The purpose of this registration is to estimate a 3D similarity transformation between the reference and the compared models. This was achieved by the manual selection of well- distributed correspondent points (also indicated as point-pairs) between the reference model and the compared one. When using this solution, it is normal practice to identify natural and/or anthropological features considered stable over the monitoring period (*Bakker & Lane, 2017*). The second registration procedure followed instructions furnished in *Parente et alii (2021)* for the so-called Time-SIFT solution.

This procedure allowed to co-register multitemporal SfM-MVS-based models through the identification of tie-points over multitemporal images. An advantage of the abovementioned approach is the ability to co-register 3D models even when using

multisource and multi-perspective data, captured across widely varying spatial and temporal scales.



Fig. 4 - Photogrammetric targets locations (above) and example of targets measurement (point 2 – bottom right; point 6 – bottom left)

MULTI-TEMPORAL ANALYSIS

After aligning the multitemporal 3D datasets, analysis of change was conducted using the multiscale model-to-model cloud comparison (M3C2) plugin (*Lague et alii, 2013*). The M3C2 is a sophisticated cloud-to-cloud comparison method implemented in CloudCompare (2023) which computes signed distances along surface normals. It does not imply the interpolation of surfaces and it is particularly advantageous for applications on complex 3D objects typical of landscape environments. This approach has been employed for many studies investigating geomorphic change (*Cook & Dietze, 2019; Di Francesco et alii, 2020; Mazza et alii, 2023*). The M3C2 algorithm is a two-step pipeline which compares point to point cloud data using surface normals that are consistent with surface roughness and measure the change that occurs along the normal direction.

This approach provides more reliable accuracy estimates when compared to other comparison approaches as it accounts for sources of uncertainty due to surface roughness and models registration (*Stumpf et alii, 2015*). Considering the surface roughness of the site of interest, the M3C2 plugin was preferred for the comparison between multi-temporal point clouds. The parameters for the above algorithm were set based on suggestions found in Lague et alii (2013) and are stated in Table 2:

Normals (D) (m)	Projection (d) (m)	Max depth (m)	Registration Error (m)	Core points	Normals (calculation mode)	Preferred orientation
0.5	0.3	10- 25	0.5	Cou ld #1	Default	-Y

Tab. 2 - M3C2 parameters used for this study

For this study analysis of change on the cliff face was carried out for the following multitemporal comparisons: 2011-2021 and 2021-2023. This allowed to characterize the two main rockfall events occurred in December 15, 2020 and November 22, 2022. Both co-registration procedures were used, allowing to generate a total of four maps of change. Note that the same M3C2 parameters were used for all 3D comparisons.

RESULTS

A total of three 3D point clouds were generated through the photogrammetric processing of the 2011, 2021 and 2023 datasets. Figure 5 allow to evaluate the reconstruction completeness for each point cloud. As expected, point cloud density is different for each reconstruction. Specifically, the 2011 and 2021 3D models are characterized by lower quality in terms of spatial coverage when compared to the 2023 counterpart. Interestingly the 2011 dataset produced a 3D reconstruction with a much higher number of points compared to the 2021 set of photos. Missing points in 2011 and 2021 3D models are not preventing comparisons with the 2023 model and the generation of change of maps through the M3C2 plugin.

Considering the high erodibility of the cliff, Figure 6(a, c) depicts a more reasonable alignment for the point-pairs co-registration where the older reconstruction dated back to 2011 (point cloud coloured in red in Figure 6a, c) is positioned ahead the other reconstructed models on most of the investigated cliff. However, an abnormal behaviour is observed on the right side of the reconstructions where the 2023 point cloud stands above the other two. This is observed in Figure 6 (b, d) for the SIFT registration also. The SIFT registration appears to be less accurate with most of the 2023 reconstruction covering the other two point clouds. Such alignment differences tend inevitably to affect the accuracy of the multitemporal change analysis.

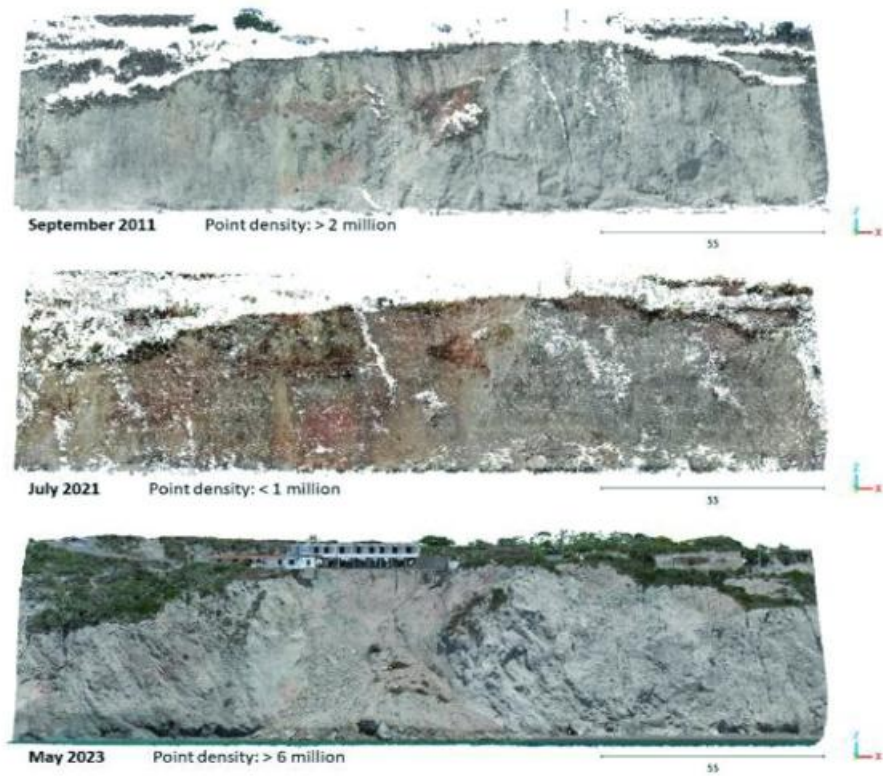


Fig. 5 - An overview of the point clouds reconstruction quality

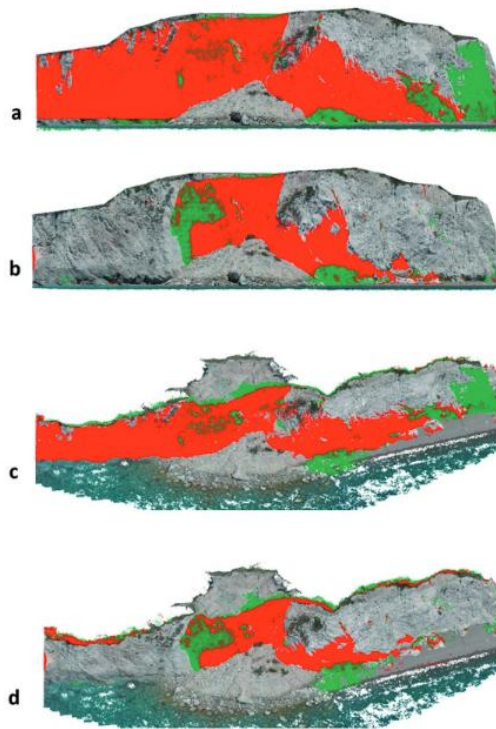


Fig. 6 - Point clouds alignment differences between the two co-registration approaches. Differences are shown in two perspectives: frontal view: (a) point-pairs and (b) SIFT; top-oblique: (c) point-pairs and (d) SIFT. Point cloud legend: red (2011)

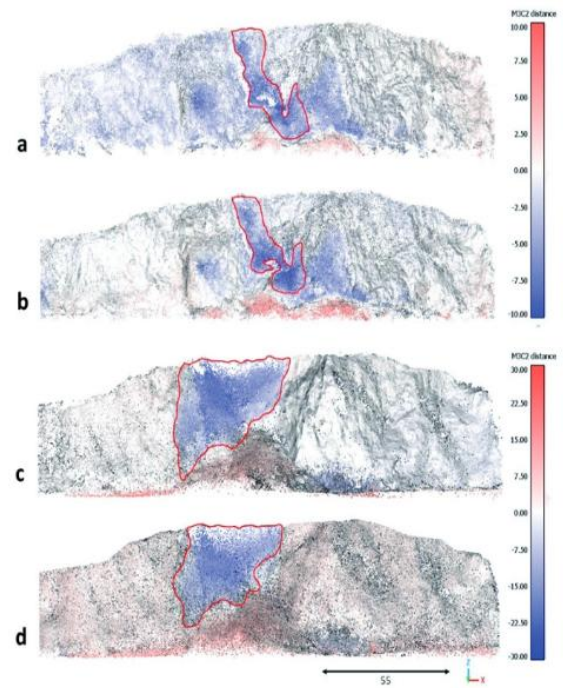


Fig. 7 - M3C2 maps of change obtained for a range of comparisons combinations: (a) 2011-2021 point-pairs; (b) 2011-2021 SIFT; (c) 2021-2023 point-pairs; and (d) 2021-2023 SIFT. The red polygon shows the area considered for the average retreat rate and volumetric estimation

The map of change for the 2011-2021 shows differences of max +10m and min -10 m. The period 2021-2023 that includes a larger rockfall event depicts variations of the cliff face in the range +30 m and -30 m. Observing the spatial distribution of change illustrated in Figure 7(a, b) it is evident that the point-pairs and SIFT registration produces comparable results with erosive events well defined in the center portion of the cliff (in blue) and area of accumulation at the bottom (in red). The left side of the studied cliff in the change of map of Figure 7a produced well distributed negative values that indicates probable minor erosional events.

A distinct pattern of erosion characterizes Figure 7(c,d) where the main collapse of the cliff dated back to December 15, 2020 and November 22, 2022 is well represented. For the period 2021-2023 the main potential erroneous data in terms of estimated changes are observable in Figure 7d (SIFT alignment) where the presence of area of accretion (in reddish) on the left and right side of the cliff indicates potential

misalignment. The previous observation is valid for the pointpairs registration only on the left side of the cliff.

Approximate ranges of retreat values estimated for the central portion of the cliff indicated with the red polygons in Figure 7 (sectors affected by the major instability events) are as follows: for the period 2011-2023: 7-12 m (point-pairs) and 5.5-11.5 m (SIFT); for the period 2021-2023: 5-25 m (point-pairs) and 3-24.5 m (SIFT). Furthermore, preliminary analysis carried out with the “2.5 Volume” tool in CloudCompare allowed to define the material loss for the two investigated periods. Following results in terms of removed material were estimated: $\approx 7500 \text{ m}^3$ (2011-2021) and $\approx 14000 \text{ m}^3$ (2021-2023).

DISCUSSION AND CONCLUSION

The availability of state-of-the-art software allows for the generation of high-accuracy and high-resolution results even when using images acquired with low-cost sensors and also to use algorithms to automatically register multitemporal datasets (*Feurer & Vinatier, 2018; Jaud et alii, 2019*). The 2011 and 2021 sets of photos used in this study were captured by amateur operators with a good overlap percentage even if a rigorous acquisition geometry was not carried out. This allowed for the adoption of a SfM-MVS photogrammetric workflow and the generation of dense point clouds. The 2011 datasets allowed for the generation of a much denser point cloud when compared to the 2021 counterpart. Considering the similar number of photos processed for both datasets and same weather conditions, this result was probably influenced by the better quality of the DSLR camera sensor and the lower acquisition distance (increased average GSD) of the 2011 dataset. As expected, point cloud reconstruction for 2011 and 2021 present noticeable differences when compared to the 2023 model.

According to *Jaud et alii (2019)* a number of factors can cause such differences including camera geometry during acquisition, number of photos processed and effectiveness of tiepoints detection that is influenced by GSD value, photo overlapping and variation in site illumination. The effectiveness in tiepoints detection is critical not only for the generation of accurate results but also for the adoption of automatic co-registration procedures (*Parente et alii, 2021*).

Results of change detection of this study are unavoidably affected by multiple sources of errors. Many aspects may influence the quality of a 3D reconstruction including camera system selection, calibration, network configuration, acquisition procedure and survey scale (*Mosbrucker et alii, 2017*), software used (*Niederheiser et alii, 2016*) and external factors (e.g., site characteristics, weather and light conditions, etc.).

A quantitative accuracy assessment between point clouds obtained with amateurs set of photos (*2011 and 2021*) and the drone-based dataset (*2023*) was not possible.

Estimation of the root mean square error (RMSE) between the multi-temporal constructed 3D results requires the definition of unchanged features (e.g., *Yu et alii,*

2022). However, identifying multiple stable points is complex and highly subjective considering the dynamicity of the study site and the lower coverage and spatial resolution of the reconstructions obtained for 2011 and 2021. Similarly, a quantitative assessment to evaluate the two co-registration approaches proved to be difficult for the same reasons. Future work will be addressed to define proper evaluation of the above quantitative assessment.

As shown in Figure 6 an initial qualitative assessment was conducted to preliminary define the co-registration approaches. From the graphical result it is clear that the two approaches present significant differences that are reflected on the maps of change generated (Figure 28). This highlights the need to conduct further scientific observations to accurately define difference values between the two co-registration approaches.

Adoption of an Iterative Closest Point (ICP) algorithm can provide an improvement in terms of alignment of portions of point clouds. This approach minimizes the square errors between correspondences when a reasonable alignment is already estimated between the two multitemporal datasets. However, this approach was not adopted because of the already mentioned difficulty in identifying and removing the active zones which have potentially changed between surveys. In fact, isolating areas of the 3D model where change may have occurred is critical to maximize the quality of the ICP registration and obtain reliable ICP transformation parameters (*Turner et alii, 2015*).

Although results shown in this research need further analysis, this is the first study investigating the instability events along a portion of the Maronti cliff that has caused significant recession and critical loss of infrastructure over the last years. It is recognised the need to carry out a robust workflow to estimate uncertainties inherent in data co-registration and subsequent topographic and volumetric change analysis.

Furthermore, testing further coregistration methods (*Cucchiaro et alii, 2020*) and volumetric calculation methods (*Štroner et alii, 2019*) can refine results. The previously described work will be integrated with further research approaches including the use of satellite data. Quantifying volumes and revealing patterns of cliff edge and face evolution, will provide critical information to understand the processes

triggering cliff failures at Maronti bay. Also, using satellite-based analysis can reveal essential information to understand the influence of development and engineering structures on coastal erosional patterns (*e.g., Parente et alii, 2015*).