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University of G. D'Annunzio
Engineering and Geology Department
Postgraduation Education

Doctoral Dissertation

*Geotechnical Parameters Calibration to Predict Land Sliding
Through Back-Analysis and Sensitivity Analysis Using
Numerical FEM Simulation*

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Pescara July 2025

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1 Introduction

Landslides are hazardous Geotechnical processes that can cause injury, loss of life, environmental degradation, and severe damage to infrastructure. In fact, landslides account for roughly 17% of all fatalities from natural hazards worldwide [51]. The societal and economic toll is immense: between 2004 and 2016 over 55,000 people lost their lives in landslides (excluding those triggered by earthquakes), with global economic losses estimated around \$20 billion annually [52]. Such failures often occur on natural slopes with little warning, as adverse conditions like steep terrain, weak rock or soil, heavy rainfall, or seismic shaking can abruptly destabilize hillsides. These slope collapses not only destroy homes and roads but also disrupt ecosystems. Given the potential for catastrophic consequences, the study of landslides in geotechnical engineering is of paramount importance to protect human safety, preserve the environment, and ensure the resilience of the built environment.

Being able to predict when and where a landslide might occur is a central goal in reducing landslide risk. However, landslide prediction is challenging because slope failures result from complex interactions of factors (geology, hydrology, weather, earthquakes, etc.) and often occur with little advance notice. Despite these difficulties, recent trends underscore the urgency of improving predictive capabilities: climate change has led to increased frequency of intense rainfall and other triggering events, and consequently a rise in landslide occurrence and associated losses, which happened also in the study case of this research area. Geotechnical engineers play a vital role in developing these prediction models by understanding the thresholds at which soil or rock slopes fail and by integrating field monitoring data to ensure that warnings are both timely and reliable. Improving prediction is not only a scientific endeavour but an ethical imperative.

For example, numerical modelling can simulate the effect of infiltration on slope stability. Furthermore, dynamic numerical models allow simulation of landslide triggers, for instance, applying earthquake shaking to a slope model to see if and when it fails, or modelling progressive rainfall infiltration over time. While no model can perfectly predict a complex natural landslide, the insights gained from numerical simulations are invaluable for engineering decision-making. Numerical solutions provide a virtual testing ground to understand failure mechanisms, identify critical factors (like the depth of a weak layer or the impact of a rising water table), and evaluate the performance of various intervention strategies under different scenarios.

In practice, numerical modelling allows engineers to tackle the “what if” questions: *What if the water table rises by another 5 meters? What if we add a row of piles at the slope toe? What is the*

expected runout distance if this slope fails? By answering these questions virtually, engineers can design more effective mitigation measures and early warning thresholds. In summary, modelling is a powerful tool that enhances our predictive understanding of natural slopes, it transforms lessons from past failures and theoretical soil mechanics into practical guidance for future slope design and disaster prevention.

In this regard, during my experience in Geotechnical Engineering in different companies, I noticed there could be a very interesting topic to simulate a real case area with very large dimension on Numerical EFM simulations in Geotechnical Engineering and compare results with the reality of deformations and failure mechanism that happened in the region. There was an advanced Numerical FEM software named *Plaxis FEM code* software, and this software could simulate very large-scale model in both 3D and 2D as well as be eligible to be connected to the Remote Sensors as create “Internet of Things” (IOT) in Geotechnical Engineering by using the language programming like Python. Therefore, I decided to advance my knowledge in Numerical FEM simulation and create simulation in huge dimensions by using *Plaxis FEM code* software and create Back-Analysis in Numerical solution and achieve results comparable with the reality.

In addition, *Plaxis FEM ULTIMATE code* software (*Academic version*) has many advantages and options to open the hand of researchers to work on Numerical FEM solutions and work on different methods and data in Geotechnical Engineering. As Back-Analysis needs a big work to consider all of the data and create Sensitivity Analysis on all of the possible parameters that could be effective on the simulation, so it is a work with precision, elegance and sensitivity to achieve data completely comparable with the reality.

For this reason, we chose the study case of the hamlet of Ponzano (Civitella del Tronto), which an extensive landslide in a large scale occurred on February 12, 2017. This land sliding was triggered by the combination of the effect of intense rainfalls and snowmelt that saturated the slope and its slow infiltration of water due to the increase of temperature in this region. The same of other land sliding disasters, this land sliding also destroyed many buildings and moved out many people from their houses and had a large volume of failure in the region. Therefore, this led us to focus our research on this region through the Numerical FEM simulation and create the model of the region through the Sensitivity-Analysis and Back-Analysis and finally compare the Numerical FEM deformation results with the reality and predict the following failure for the feature.

The 1st phase of the research project was focused on the deformation that happened in region by comparing results with Lidar DEM data and also drone surveying engineering data, which was performed in *G. D' Annunzio University* in Italy under the supervision of professor Nicola Sciarra

and co-supervision *professor Antonio Pasculli*. In the 2nd and 3rd phase focused on creating Back-Analysis and performing Sensitivity Analysis on the big simulation in 1.8 km dimension and compare the results with the reality, whereas these phases was held in the “*Institute of Soil Mechanics, Foundation Engineering and Computational Geotechnics*” of *Technical University of Graz* in *Austria*, with the cooperation of *professor Franz Tschuchnigg* and the co-assistance of *professor Helmut Schweiger*.

2 Study area

Ponzano is one of the hamlets of the Italian municipality of Civitella del Tronto, in the province of Teramo, close to the border with the Marche Region (Figure 1). It is located in a hilly area at an average elevation of 360 m a.s.l. The landslide affects a E-SE facing gentle relief ranging from 400 to 190 m a.s.l. The slope angle ranges from 4 to 12°. The area is extensively covered by crops and pastures with scattered woods and small groups of houses [4].

2.1 Geological and Geomorphological Structure of Ponzano Landslide

2.1.1 Stratigraphy and Lithological Setting

The Ponzano landslide lies on the slopes underlain by the Upper Miocene–Lower Pliocene *Laga Formation*, a foredeep flysch sequence characterized by alternating sandstones, marls, and clays [2]. In the landslide area, this formation is represented by the Fosso Rio and Rapino Members: the Fosso Rio member consists of grey marls with occasional fine-grained sandstones (a pelitic-arenaceous facies up to ~100 m thick), while the Rapino member is an alternating arenite–marly clay–sandstone sequence (~80–100 m thick) [4]. These interbedded lithologies exhibit strong mechanical contrasts, with thick competent sandstone strata (approximately 30–60 m thick) alternating with weaker clayey-marly layers. The sandstone beds form prominent ridges and have significantly higher stiffness and strength, whereas the interlayered pelitic (clay-rich) horizons are relatively weak and prone to deformation [2]. As a result, the contacts between these units often act as planes of weakness where sliding can occur. Indeed, structural and geotechnical analyses indicate that key slip surfaces develop along the clay–marl layers at the interface with the more rigid sandstones, exploiting these mechanically unfavourable horizons. Overlying the bedrock is a substantial cover of unconsolidated deposits – Ponzano’s slope is blanketed by colluvium (up to ~15 m thick) and by older landslide debris (locally up to ~70 m thick) from previous slope failures. This thick superficial cover of clayey–

sandy detritus further contrasts with the underlying bedrock and has its own internal weaknesses, contributing to the complex behaviour of the landslide mass [2].

2.1.2 Structural (Tectonic) Framework

The Ponzano site is situated in the foothills of the Central Apennines and reflects the *fold-and-thrust belt* tectonics of the region. The area is just east of the Montagna dei Fiori thrust front, on the eastern limb of the Civitella–Ponzano ridge, which trends roughly WSW–ENE. The bedrock layers at Ponzano strike about NNW–SSE and dip steeply ($\sim 40\text{--}60^\circ$) toward the ENE as a result of Pliocene folding and thrusting. This steeply inclined homoclinal structure produces a hogback ridge morphology: in fact, two sub-parallel sandstone-capped ridges run NNW–SSE through the upper slope, corresponding to resistant Laga Formation beds that have been tilted up by compressional tectonics. Minor folding is superimposed locally, but the overall structure is a monoclinical east-dipping sequence. Regionally, the stratigraphy is disrupted by high-angle faults related to late Apennine extensional tectonics, and the area remains seismically active with moderate earthquakes. However, no major through-going fault is mapped directly at the Ponzano landslide; instead, the predisposition to slope failure is more due to the *bedding orientation and weak layers* than to a single fault rupture. In summary, the tectonic framework has created a slope where strong beds form a ridge-and-bench topography and weak beds dip out of the slope, a setting highly conducive to landsliding [2].

2.1.3 Slope Morphology and Geomorphological Features

The Ponzano landslide developed on a large east-facing slope approximately 1.5 km in length, dropping from about 400 m at the ridge down to ~ 150 m a.s.l. at the Salinello valley floor. The average gradient of the hillside is gentle ($\sim 10^\circ$), but the slope profile is very irregular, with relatively flat or bench-like areas in the mid-slope and steep scarps at the top and toe of the landslide. The upper slope features a pair of sharp, hogback ridges (aligned NNW–SSE) composed of steeply dipping sandstone strata, which exert a strong structural control on the slope morphology. These ridges effectively form the high ground above the landslide and have guided the development of the main landslide crown scarps. Indeed, two major arcuate scarps opened near the hillcrest along the sandstone ridges, defining a double amphitheatre-shaped crown of the landslide. The southern (up-slope) scarp system is the primary headwall: it consists of two to three coalescing scarps spanning $\sim 350\text{--}400$ m in width, with scarp heights ranging from ~ 2 m up to over 10 m. This large head scarp complex has created local graben-like depressions and counter

slopes (back-tilted benches) immediately downslope of it, indicating rotational slide components in the uppermost part of the failure. To the north, closer to Ponzano village, lies a secondary head scarp system. This northern scarp is smaller (on the order of 100–150 m wide) and its cliffs range from mere decimetres to several meters high. The village of Ponzano sits precariously near this secondary scarp, and indeed its proximity led to severe damage to structures when the slide reactivated. Above and behind the main scarps, the slope has additional tension cracks and small scarplets propagating upslope, evidence of retrogressive failure extending toward the ridge crest. This network of fissures at the head signals that the landslide head is attempting to enlarge itself further uphill (a regressive trend common in rotational slides) [2].

Below the head area, the middle portion of the landslide is characterized by an irregular chaotic topography of displaced blocks and internal scarps. Multiple slide blocks (“slabs” of the colluvium and upper bedrock) have moved differentially, creating a complex assemblage of hummocky terrain. Between these blocks are numerous secondary scarps and depressions. Many of the slide blocks in the mid-slope are rotated, producing local *counter-slopes* (areas that now tilt into the slope instead of down) and uplifted ridges or pressure bulges on their downstream edges. One notable geomorphic feature is a broad landslide terrace in the upper-mid slope – essentially a large bench formed by the settling of a huge slide mass – which is bounded by scarps above and below. This terrace area experienced dramatic bulging and heaving during the slide event, to the extent that the ground cracked extensively and even expelled mud in places (witnessed as small mud-volcano-like vents) due to pressurized pore water in the clayey substrate. The central terrace and bulge zone correspond to the area of greatest damage: several houses that once stood here were completely destroyed as the ground beneath them deformed and lifted [2].

The lateral margins of the landslide are marked by distinct shear lobes and lateral scarps. For example, along the southern flank of the moving mass, an undulating lateral scarp up to ~2 m high formed in 2017. Slickensides and horizontal grooves observed on this flank scarp (striking roughly ENE) indicate significant translational movement – the slide mass sheared past the intact slope laterally, leaving striations that point to the direction of motion. Small “push ridges” of earth have been shoved outward along parts of the lateral boundary as the moving material compressed against stable ground. The northern flank of the landslide consists of a series of lesser slide blocks and step-like scarps, mainly evidencing translational slides that broke away near the village edge. Finally, the lower part of the landslide exhibits features of a flow or earthflow tongue: as the landslide transitioned from a coherent slide above to a flowing mass at the toe, it produced lobate earth mounds, minor flow levees, and additional low-height scarps oriented both across and along the slope. Several small scarps in the toe area face uphill (indicative of compressional thrusting

within the flow) and are accompanied by pressure ridges where material has piled up. The distal toe of the landslide impinged on the lower slope near the valley, and pre-existing gullies in that area were partly filled or rerouted by the deposited debris. Overall, the landslide morphology – from head scarps and rotated blocks in the upper slope to flow-like deformation at the toe – reflects a complex slide mechanism (*sensu* Varnes): initially a translational and rotational slide in the source area that evolved into an earth flow at the downslope end [2].

2.1.4 Indicators of Slope Instability

Prior to and during the 2017 failure, the Ponzano slope displayed numerous geologic and geomorphic indicators of instability:

- **Old Landslide Deposits:** The 2017 event was in fact a reactivation of a known, previously quiescent landslide, meaning the slope had failed in the past and retained a weakened internal structure.
- **Multiple Head Scarps:** The crown of the landslide is defined by large arcuate scarps up to 10 m high, which had developed at the interface of strong sandstone layers and weaker underlying strata. These fresh cliff faces and slump shelves at the ridge line are clear evidence of recent rotational–translational detachment. Smaller scarps just downslope of the main heads carp (forming a tiered scarp system) likewise signal ongoing retrogressive failure.
- **Tension Cracks and Fissures:** Such tension fractures indicate that the head of the landslide was pulling apart and extending back toward the hilltop (a regressive enlargement). During the active sliding, fissures also opened within the moving mass, some hundreds of meters long, delineating the boundaries of blocks and internal shear zones.
- **Back-Tilted Benches (Counter slopes):** In the mid-slope, portions of the displaced mass are tilted backward toward the scarp, forming anomalous nearly flat or inward-sloping surfaces. These counter slope areas, often ponded with water, are characteristic of rotational slide blocks and are a hallmark of deep-seated slope deformation.
- **Bulging and Ground Uplift:** The central part of the landslide exhibited significant bulging of the surface. This was evidenced by uplifted ground (several meters of heave) and the development of pressure ridges and small thrust humps on the landslide surface. The ground deformation was so intense that in places pressurized mud and water were

expelled through fissures (observed as pseudo-*mud volcanoes* on the landslide terrace), dramatically illustrating the build-up of pore pressure and plastic deformation in the substrate. Such bulges and mud ejections are clear signs of an ongoing earthflow-like movement at depth.

- **Lateral Shear Scarps and Striations:** Along the flanks of the landslide, the soil and rock tore away, creating lateral shear surfaces. The southern lateral scarp in particular showed slickensides with ENE-dipping striations, confirming that large sections of the slope moved horizontally relative to adjacent stable ground. Additionally, “push ridges” at the toes of these lateral scarps – small transverse ridges of jumbled earth – mark where the moving mass squeezed outwards. These features all denote significant shear displacement and horizontal spreading of the slide mass.
- **Disrupted Infrastructure:** By the time of the 2017 reactivation, dozens of houses in Ponzano exhibited structural failure (walls cracked, foundations shifted), consistent with the measured 10–40 cm of slow prior movement in the 14 years leading up to the final collapse.

These indicators, taken together, painted a picture of a slope that was already critically stressed and primed for failure even before the catastrophic movement in February 2017 [2].

2.1.5 Kinematic and Mechanical Characteristics

Referring the previous research (Lorenza Solari et al., 2018) the landslide was classified as a complex movement, combining:

- Rotational sliding in the upper and crown portion
- Earth flow-like behaviour in the central and toe portions, indicating ductile movement over a deep slip surface (~15 m depth)

The maximum displacement during the event was measured at ~10 meters, mainly in the crown and middle sectors (e.g., Villa Carosi). Maximum velocities during the failure phase reached 4–5 m/day, qualifying it as moderate (Cruden and Varnes, 1996). Pre-failure data also exhibited a consistent translational motion toward the ESE, in line with slope geometry. Post-failure displacement vectors also derived from rapid motion tracking (RMT) showed dominant southeast-directed motion [4]. A slow but accelerating displacement pattern was detected up to 10 years before the failure.

The landslide developed in Late Miocene–Early Pliocene marine siliciclastic turbidites (Laga Formation), mainly the Fosso Rio and Rapino Members. Slope angles range from 4° to 12°, typical of the eastward-dipping monoclinal relief formed by thrust tectonics. The presence of pre-existing landslide deposits and colluvium facilitated reactivation [4].

Several external factors converged to trigger and exacerbate the Ponzano landslide in 2017, chiefly related to exceptional weather events and seismic shaking. Hydrological forcing was paramount: in January 2017, the region was buried under an unusually heavy snowfall (on the order of 1.5 m of snow even at Ponzano’s hill elevation) followed by a sharp temperature rise in early February. This led to rapid snowmelt, resulting in prolonged infiltration of meltwater into the ground. The colluvial cover and the fractured flysch bedrock beneath Ponzano became fully water saturated as the melting snow percolated downward. The combined effect of snowmelt and heavy rain brought the slope’s water content to critical levels, elevating pore-water pressures in the clay-rich layers and along bedding planes. Essentially, the hillside’s weak layers were lubricated and the effective stress within the slope material was greatly reduced by this sudden influx of water. Field evidence of high pore pressures was seen in the aforementioned mud boils and the fluidized nature of the lower landslide debris. This hydrogeological loading was a key trigger that drove the dormant slide into rapid motion [4].

This is the reason, the local authorities mentioned that three inclinometers as were installed, back in 2003, in the upper portion of the February 2017 landslide and in just few years, the slow motion of the landslide led to the rupture of the inclinometric instrumentation at the depth ranging between 13 and 16 m below surface, in accordance with the hypothesis of the February 2017 landslide surface and the slip surface was inferred at around 15 meters depth [4]. This important information was used during our Sensitivity Analysis and our final Back-Analysis which should be matched with this research data (Lorenzo Solari, et al. 2018).

2.2 Landsliding in Ponzano Region

On February 12, 2017, an extensive landslide occurred in the hamlet of Ponzano (Civitella del Tronto) involved an area of around 60 ha on the SE facing slope of the Ponzano village. The instability of the landslide, with an estimated volume of about $7 \times 10^6 \text{ m}^3$, was triggered resulted by the combined effects of intense rainfalls in January and February 2017 and snowmelt that

saturated the slope and its slow infiltration of water due to the increase of temperature [4]. The maximum thickness of the snow cover in this area, during the month of January was estimated around 150 cm [2]. The snow cover duration was between 17th January and 25th January 2017, whereas on 25th January all of the snow cover was melted. Thirty-two buildings were involved by the landslide movement resulting in the evacuation procedures of the inhabitants. The latitude and Longitude of the landsliding in the Ponzano region was 42:46:30 N and 13:42:25 E respectively [12].

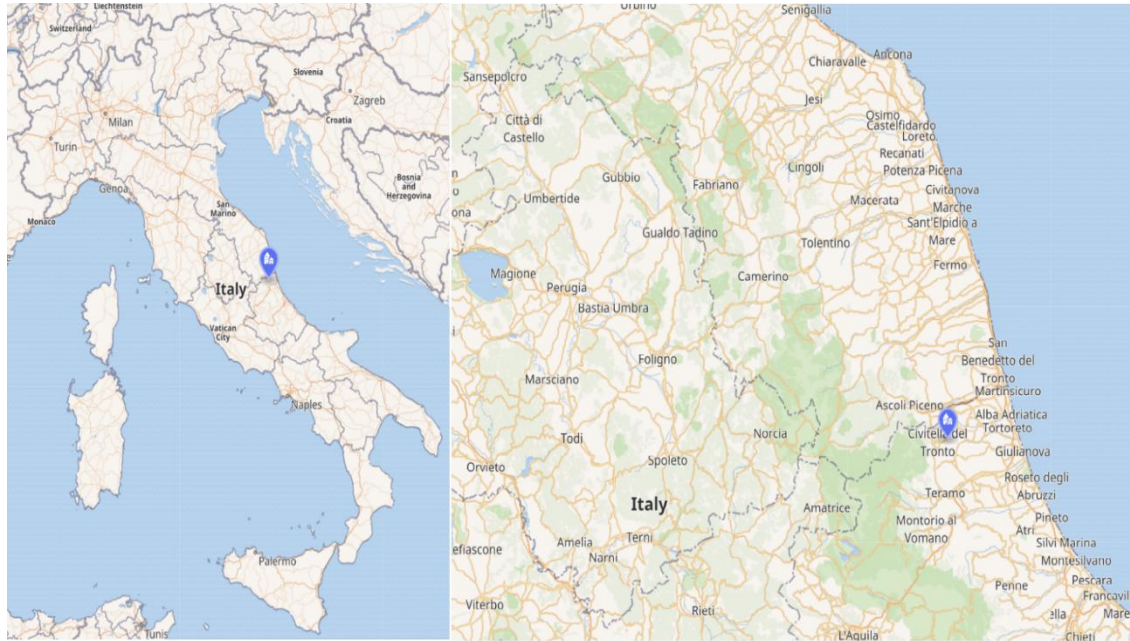


Figure 1 Ponzano location in Italy [36]

2.3 Landsliding Characteristics

Following the landslide failure, the Italian Civil Protection Department defined a restricted area, almost coincident with the landslide perimeter, in which 32 private houses recorded serious damage and about 100 people had to be moved out from their houses. In particular, the restricted area is subdivided into complete residency interdiction because of the extension and severity of the damage and people who lived in this area had to be relocated [4].

The phenomenon caused the interruption of the main road access of the village, that connects Ponzano with Civitella del Tronto, which shows the region labelled 3 and 4 in Figure 2. The road displays severe damages for a length of approximately 300 m. The damage associated with the

ground movement also made unusable the secondary road that connects Ponzano with Villa Carosi (Figure 2). In Figure 2, the main scarp appears as a steep surface, nearly vertical in some points. In this sector of the landslide, other secondary scarps were found [4].

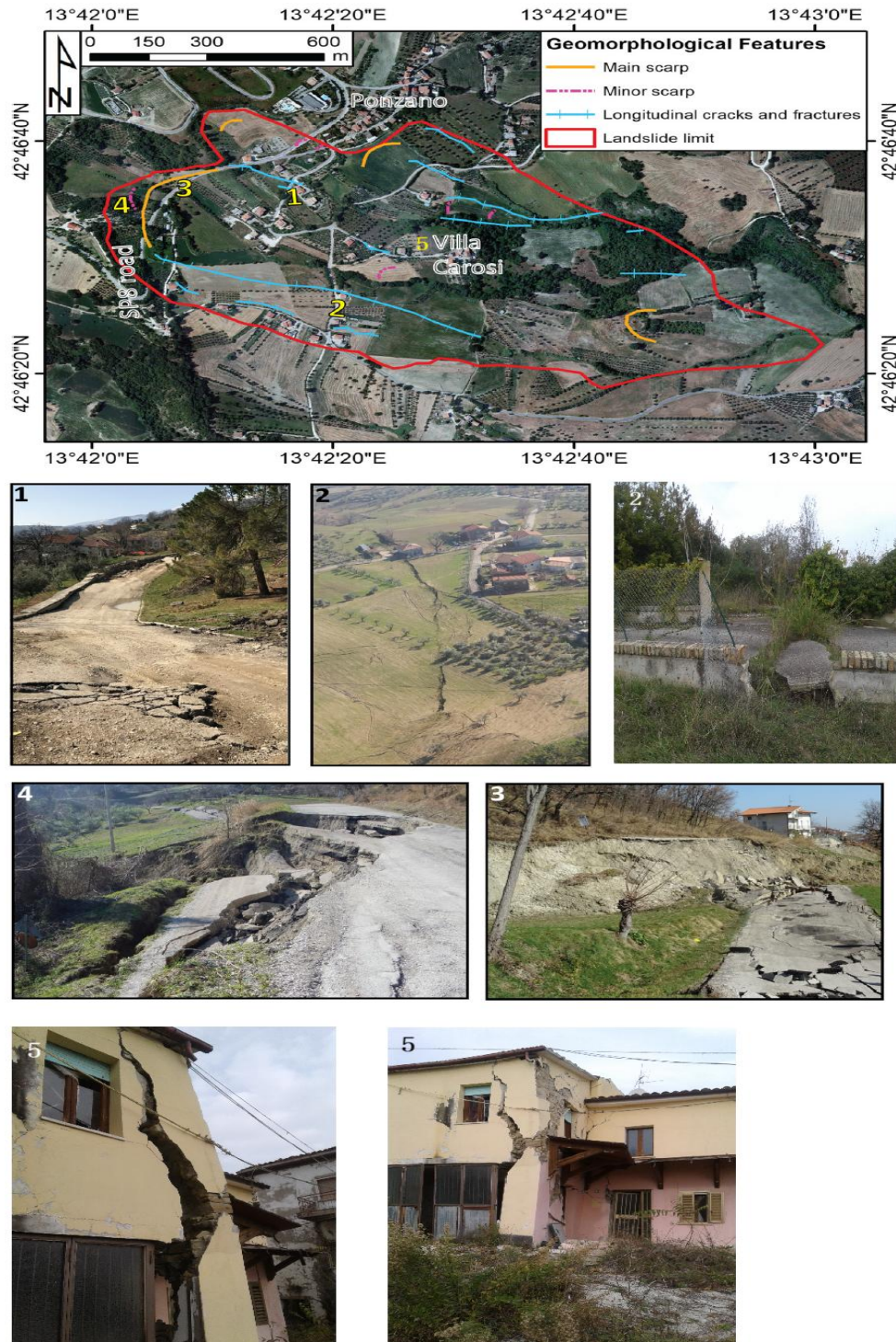


Figure 2 Landsliding characteristic in the Ponzano region in February 2017 [4]

The Ponzano landslide is a large (1.4 km in length, 0.6 km in width, 56 ha surface area) composite landslide with multiple complex movements related to mechanism from translational sliding with local rotational components, in the upper part, to translational sliding/flow in the lower part. The recent movement occurred abruptly on 12 February 2017 and slowly in the next few days, as reactivation of previous large landslides that affected the slope [2]. In the following, the Geomorphology map of the Ponzano region in the plan and section that the main land sliding happened is attached in Figure 3 and Figure 4 respectively [2].

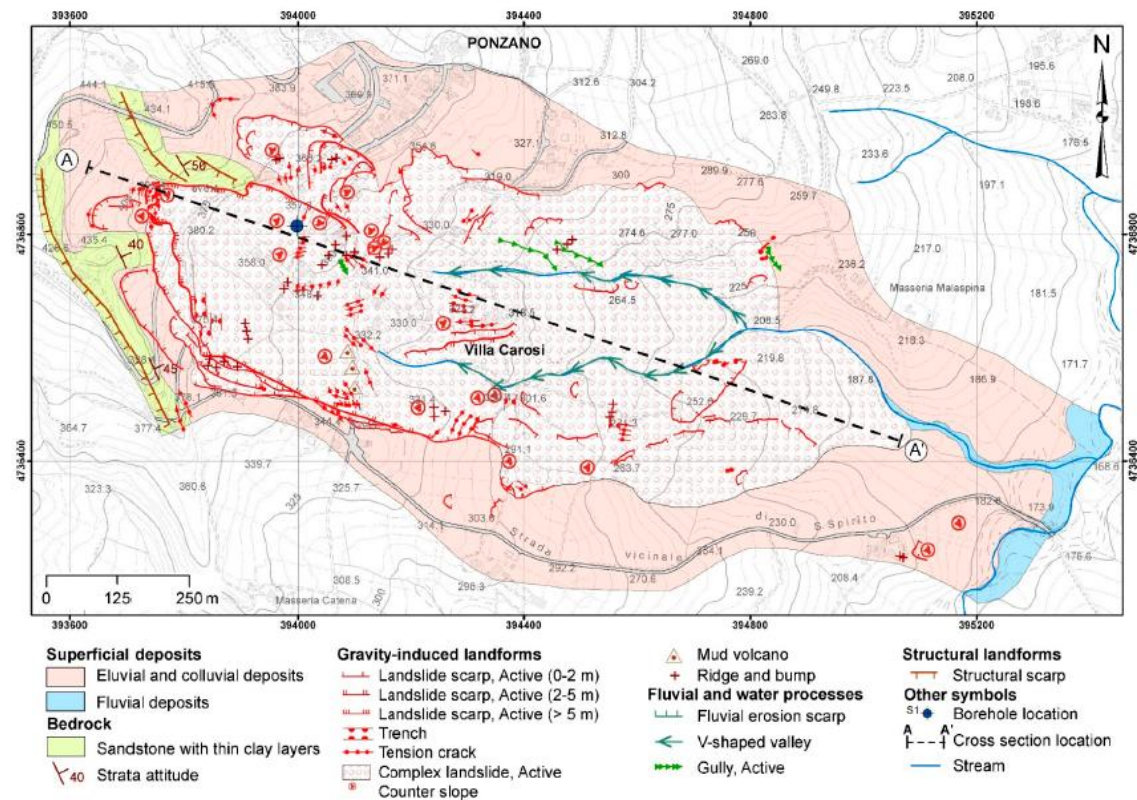


Figure 3 Geomorphological map of the Ponzano area [2]

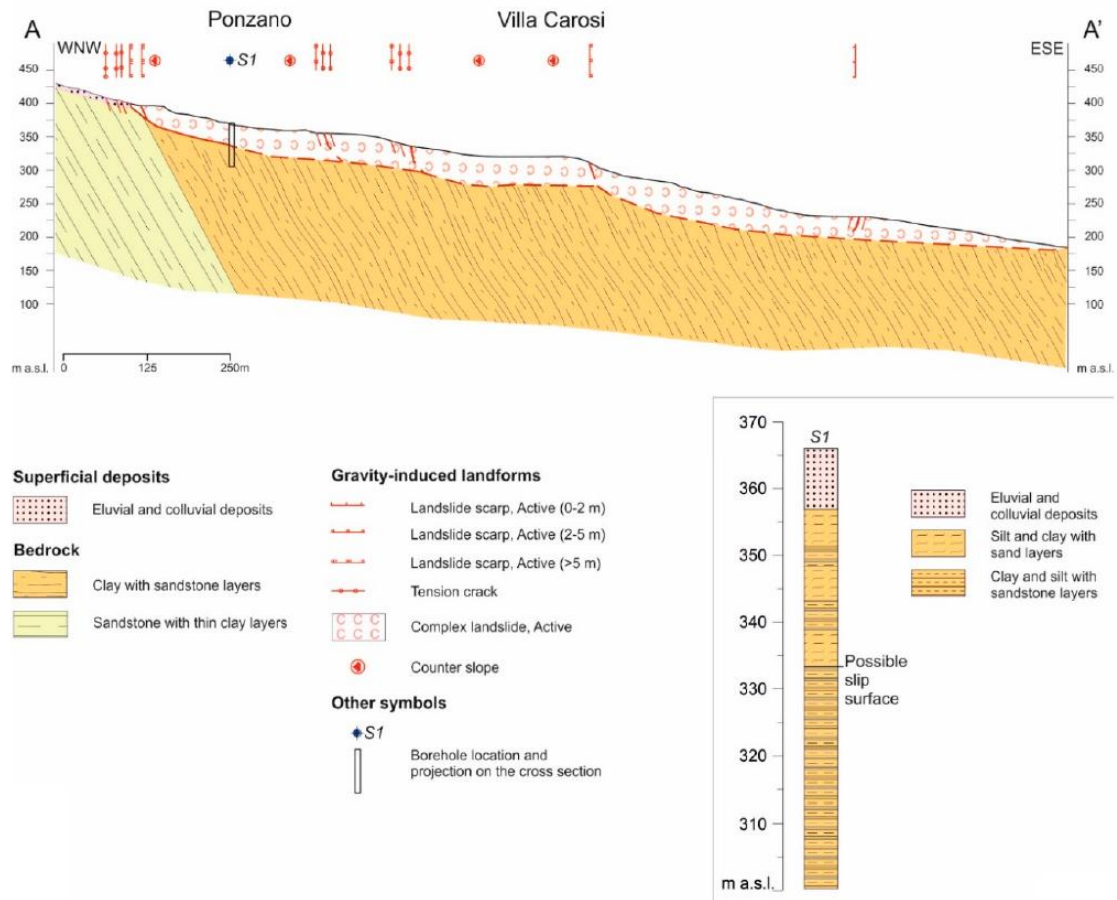


Figure 4 Section A-A from the Geomorphological map of the Ponzano area [2]

3 Data of Study Case Area

3.1 Rainfall Specification

Between 1st February and 12th February 2017 which the landsliding happened in the Ponzano region, the Civitella Fortezza Pluvio rain gauge (3 km westward from Ponzano) registered a cumulated rainfall of 82 mm (Figure 5), higher than the mean monthly precipitation for this part of the Abruzzi region that is equal to 62 mm, representing the 32% more than the total average rainfall of February for this region. In addition, a cumulative rainfall of 99 mm was registered in the January 2017 (Figure 5) again higher than the average monthly precipitation of this region which is equal to 78 mm, representing the 27% more than the total average rainfall of for this region in January [13].

As described in previous sections, there was a snow cover between 17th January and 25th January 2017 and then after that all of the snows cover were melted. This showed that during the snow cover, there were no infiltration into soil layer, although we had precipitation data during these nine days. To consider the effect of snow in our Numerical FEM analysis, the precipitation measured in these nine days between 17th January and 25th January 2017 with snow cover were shifted to the subsequent nine days between 26th January and 3rd February 2017, which is depicted on Figure 6 [5].

The total of rainfall in these 9 days of snow cover, was calculated 72 mm which was divided by its 9 days achieved 8 mm/day. In addition, the snow cover for these 9 days was 1.5 m that total of them melted finally at the end of 9th day, which resulted in 167 mm/day for considering the melting of snow infiltration. Therefore, 175 mm/day was considered for the subsequent 9 days between 26th January and 3rd February 2017 to consider the effect of snow infiltration and rainfall between 17th January and 25th January 2017.

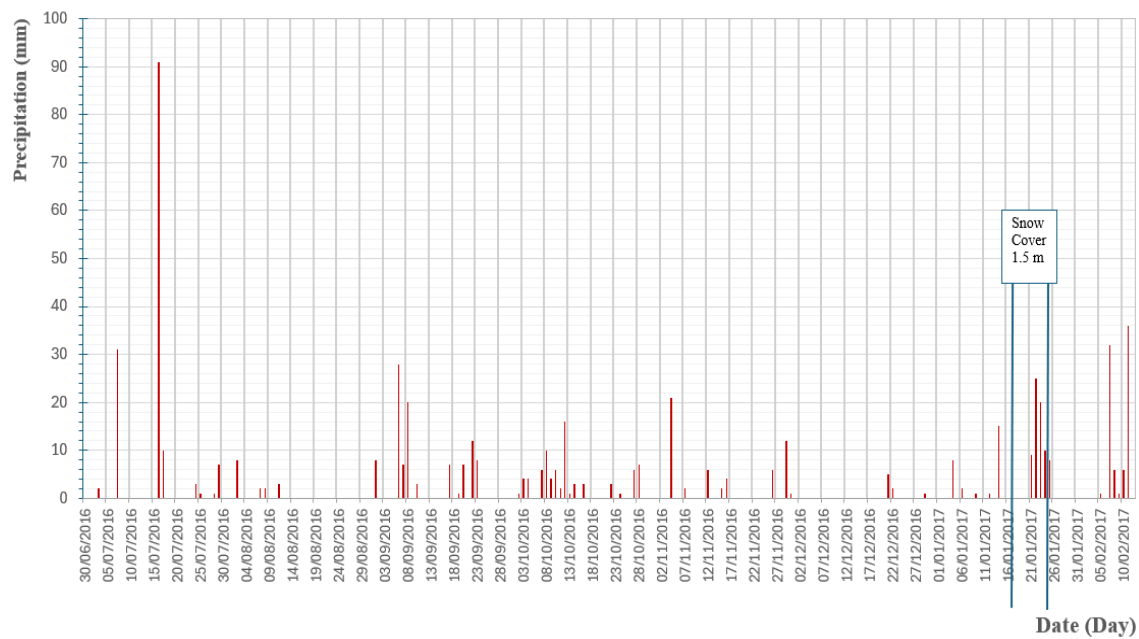


Figure 5 Precipitation records in the Civitella Fortezza Pluvio rain gauge

3 km westward from Ponzano region

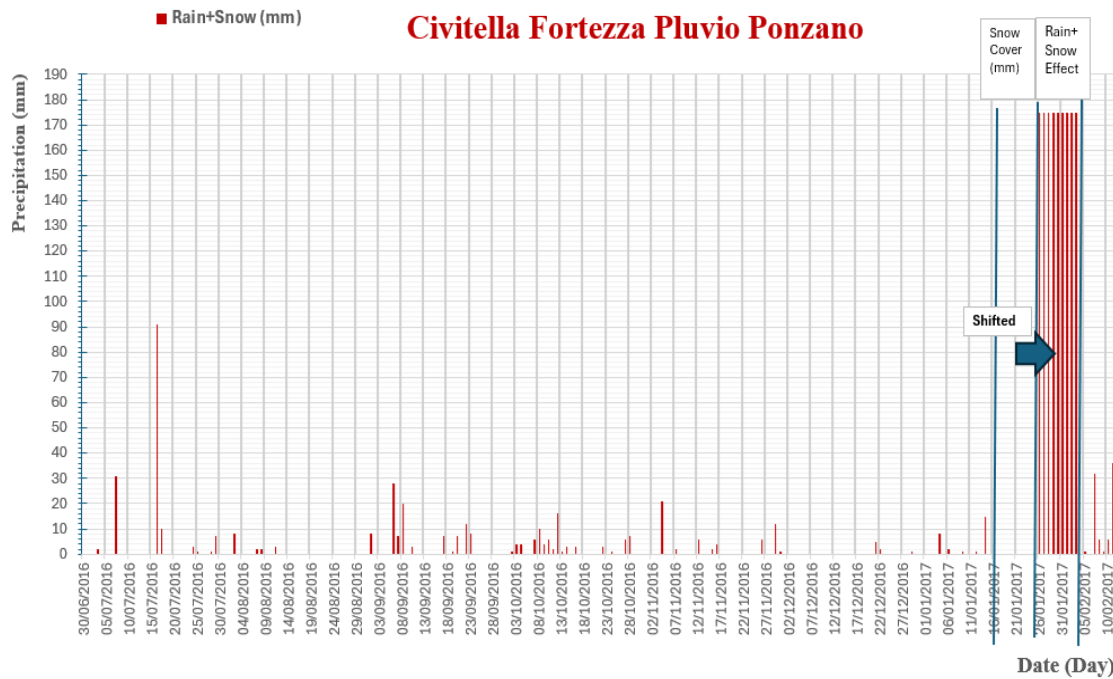


Figure 6 Precipitation records in the Civitella Fortezza Pluvio after shifting the effect of snow cover to the subsequent 9 days

In addition, on Figure 6 it shows that on 17th July 2016 the intensity of rainfall was reached to 91 mm, which showed a high intensity for the precipitation data in Ponzano region, but since the laboratory data showed on Table 2 that the soil classification is between CH or CL and the permeability should be very low, so there would be very instant effect of the change of intensity of precipitation data on the pore water pressure, specifically on deeper soil layer. This procedure will be discussed more on **Section 6.4**, and in this Section it will be showed that the rate of change of pore water pressure will be increased by increasing the intensity of precipitation data on 17th July 2016, but these changes will be mostly limited to the layers near the ground surface and not specifically in deeper layers.

3.2 Laboratory Data from Ponzano Region

The laboratory tests performed in the “*Department of Geotechnologies for the Environment and the Territory*” as the “*Applied Geology Laboratory*” and the results are showed briefly in the following. The results showed that the 2nd layer defined as the bedrock is the Marl and Flysch

layer. In addition, the 1st layer referring the test and the ASTM D2487 was classified as CH (Fat Clay) or in some parts as CL (Lean Clay).

In Table 1 the grain size distributions laboratory tests at different depth are attached in the following. More details data from the granular distribution sizes graphs are described in the Appendix Chapter of the Dissertation. Furthermore, referring all of the laboratory tests performed on specimens at different depths, the average test results for the 1st and the only soil layer are attached in the Table 2. The field investigation also revealed that no underground water level was found in subsoil layers.

Table 1 Grain size distribution laboratory tests achieved from “*Department of Geotechnologies for the Environment and the Territory*” referring 1st soil layer at different depths

Depth (m)	Clay < 0.002 mm	Silt 0.002 mm < < 0.05 mm	Sand 0.05 mm < < 4.75 mm	Gravel 4.75 mm <
5.0 – 5.5 m	40	50	10	0
10.0–10.5 m	36	50	14	0
15.0–15.4 m	37	50	13	0
20.0–20.4 m	35	52	15	0
25.0–25.5 m	42	52	8	0
29.0-29.5 m	42	55	8	0
40.0-40.3 m	42	54	8	0

Table 2 Laboratory data achieved from “*Department of Geotechnologies for the Environment and the Territory*” referring the 1st soil layer as an average of soil specimens tested

Description	Parameters	Values	Unit
Unsaturated Unit Weight of Soil	γ_{unsat}	16.0	$\frac{\text{kN}}{\text{m}^3}$
Saturated Unit Weight of Soil	γ_{sat}	20.0	$\frac{\text{kN}}{\text{m}^3}$
Liquid Limit	W_L	52	-
Plastic Limit	W_P	18	-
Index of Plasticity	I_P	34	-
Initial Void Ratio	e_{init}	0.8	-
Over Consolidation Pressure	OCR	1.5	-
Secant Reference Modulus	E_{50}^{ref}	10000 – 20000	kPa

Reference Pressure	P_{ref}	640	kPa
c'	c'	20	kPa
φ'	φ'	23	Degree
Poisson ratio	Poisson ratio	0.33	-
ASTM D2487	Soil Classification	mostly CH (fat clay) and in some cases CL (lean clay)	-

4 1st Phase of Project on Plaxis FEM Code 3D Software

In the 1st Phase of the project, for the project of the land sliding in the Ponzano we collected the field data (e.g., settlements, displacements) from different sources, which were achieved from different sources such as the Drone data on 2019 (after the land sliding happened in the region) and also the DEM data (Digital Elevation Model) from the history the land sliding was not happened yet on 2013 and also from the history the land sliding happened on 2019 and started to work on them and compared them with each other's to achieve the best coloured-deformation map that happened in the region to best show the real condition of the study case area. In the following, we set up a Numerical model (FEM) by using the Plaxis FEM code software, which is one of the most powerful Geotechnical Engineering software for simulation, and assigned the initial (estimated) soil parameters to the model and run the analysis and compared the results with the coloured-deformation map to adjust the results with the reality, and this procedure was done and the results was achieved which were comparable with the coloured-deformation map of surveying data from Lidar DEM data and Drone data.

4.1 Collecting Point Data Before the Land Sliding Happened

4.1.1 Collecting from “Abruzzo Region Open Data Portal”

The “Abruzzo Region Open Data Portal” is an online website which is managed by the regional government of Abruzzo, Italy and provides access to wide array of datasets related to the region, which also include our research area Ponzano region. In this portal we downloaded the D.B.T. (Database Territoriale Regionale) which refers to the regional territorial database and are the collection of geographic information layers standardized and structured for the entire of this

region with different scale. We used the scale 1:5000 with the format SHP (vector “shapefile” SHP) from the year 2007, before the land sliding happened in the region.

These DBTR (Regional Territorial Database) were created by adapting and structuring, with respect to the IntesaGis 2006 standards, the first and second batch of the CTRN; they were subsequently updated using photogrammetric imaging in 2007. This geographic information was organized into hierarchical groups, according to the model defined in the State-Regions agreement on Geographic Information Systems. The hierarchical categories (each divided into layers, themes, classes) are the following: geodetic information; roads, mobility and transport; buildings and human activities; hydrography, orography; vegetation; technological networks; administrative boundaries; areas of relevance [15].

After collecting the data from the portal, we imported them as the “contour line” of the SHP file into the QGIS software (Quantum GIS) and extract vertices into point data including z coordinates from these D.B.T data. QGIS software is a free and open-source Geographic Information System (GIS) software, and analysed the geospatial data downloaded from the portal to create data with z coordinates to be used for the coloured-deformation map [40]. The coordinate reference system used in the software was “WGS 84/UTM Zone 33N” with the Multiscale DBT (Topographic Database).

The extraction data showed the distance between some points were around 100 m achieved from the “Abruzzo Region Open Data Portal” website sources, which showed not so much good accuracy for our Numerical calculations, since our goal was to create a colour-deformation map to compare with FEM calculation results.

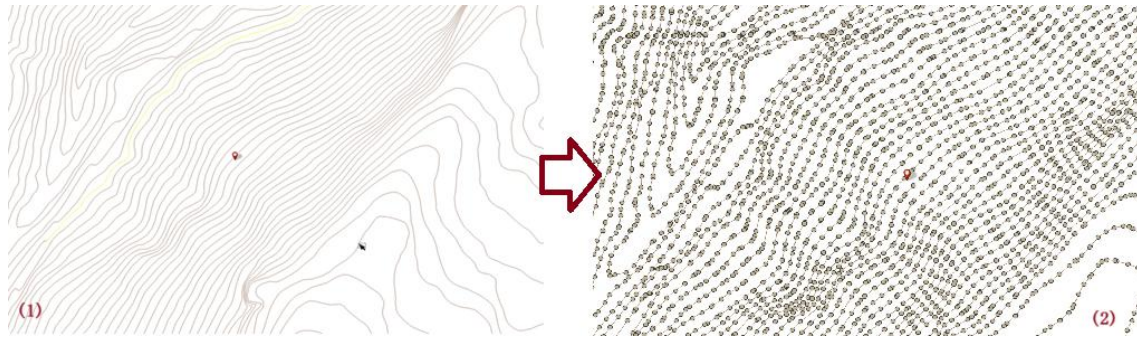


Figure 7 Importing contour lines SHP data from the “Abruzzo open data portal” into the QGIS software (1), and extract vertices into point data with z coordinate in the QGIS software (2) [40]



Figure 8 Extracting vertices contours SHP data into point data including z coordinates through QGIS software for the total of the study region [40]

4.1.2 Collecting from “National Geoportal”

The “National Geoportal” is an online platform managed by Italy's “Ministry of Environment and Energy Security (MASE)” and serves as the access point for environmental information, providing a comprehensive repository of geospatial data and services related to Italy's territory. Through this portal we download the DEM (Digital Elevation Model) data from 2013 with the very high resolution of 1.0 m Pixel size and started to work on it.

DEM (Digital Elevation Model) is a digital representation of ground surface topography or terrain and it's essentially a 3D model of the Earth's surface, derived from survey data and is usually in a gridded or raster format, without any objects like vegetation or buildings, and just the bare earth surface.

These DEM data were resourced from the Lidar data (Light Detection and Ranging), which is a powerful tool in surveying engineering used to create high-resolution, accurate 3D models of the Earth's surface and it's a remote sensing method that measures especially terrain, vegetation, and man-made structures.

4.1.2.1 Creating Point Cloud Using Global Mapper Software

By this specification we achieved the bare earth surface of the year 2013 from the project area, before the land sliding happened in the region, which was very essential in our computational Numerical solutions project. Through downloading the DEM data in the format of the TIF, we used the Global Mapper software to extract point cloud data from the DEM data (Figure 9). Global Mapper is a comprehensive Geographic Information System (GIS) software developed by Blue Marble Geographics and it offers an extensive array of spatial data processing tools and supports different file formats, including vector, raster, and elevation data, which could be used in our project to create point cloud data as it supports LiDAR data with capabilities for point cloud visualization, editing, and analysis [45].

Through the mentioned Global Mapper software, we worked on the data and then exported the data in the point cloud data with the format “XYZ Grid” to be able to work on it and import it into the Numerical FEM solution software to run and analysis.

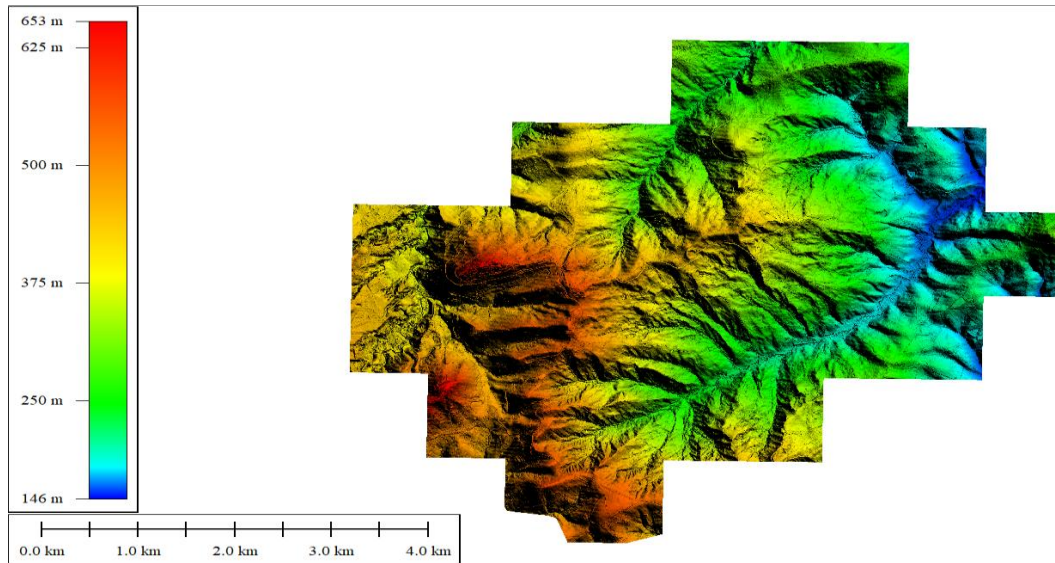


Figure 9 DEM data from the study case area of Ponzano in 2013 with resolution 1.0 m Pixel size [45]

4.1.2.2 Creating Cloud Data Using ReCap Software

After using the Global Mapper software and extracting data from it, we imported them into the ReCap software to work on data and prepare them to be able to be imported into the CAD workflow software to create surface from them (Figure 10).

Autodesk ReCap (short for Reality Capture) is a software in civil engineering, surveying, and construction to process and manage 3D scan data from LiDAR data, which we achieved from the “National Geoportal” [41]. This software is especially valuable when working with point clouds and 3D models of terrains and it manage 3D point clouds from laser scanners (like LiDAR), clean and edit data like remove noise, crop, align them, and prepare them for CAD workflows which is our next step.

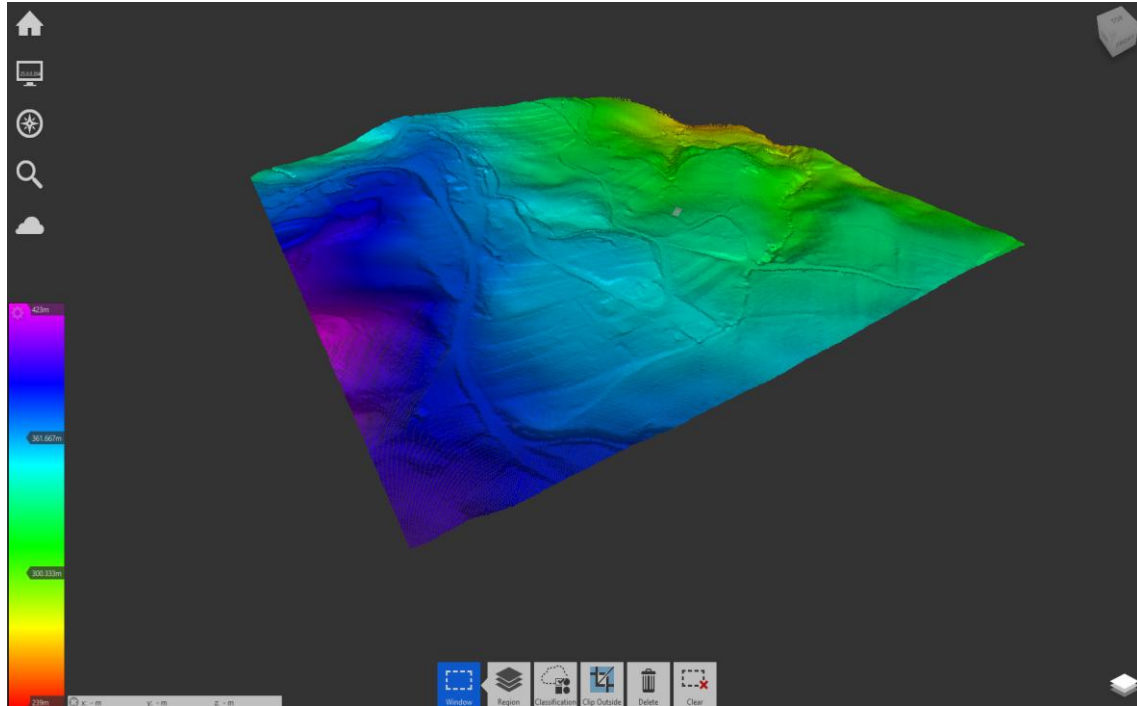


Figure 10 Removing noise, crop, align point cloud data 2013 using ReCap software [41]

4.1.2.3 Creating Solid Surface Using Civil 3D software

Once working on data using the ReCap software, we imported them into the Civil 3D software to create solid surface, which was one of our goals to create coloured-deformation map after the land sliding happened in the Ponzano region. To create the coloured-deformation map which could shows what happened in the Ponzano region, we need the deformed solid surface from the history before the land sliding happened in the region, which is performed in this section, and also the deformed surface from region after the land sliding occurred, which will be performed in the next section. Then after that, these two solid surfaces will be compared with others to detect the deformation that happened in the region as a coloured-deformation map using the Civil 3D software.

Autodesk Civil 3D is a powerful civil engineering design software built on top of AutoCAD, and it is widely used for infrastructure design, terrain modelling, etc [42]. This software could survey data processing and terrain modelling and create and analyse terrain models like TINs from survey data, which we will use this feature in the project.

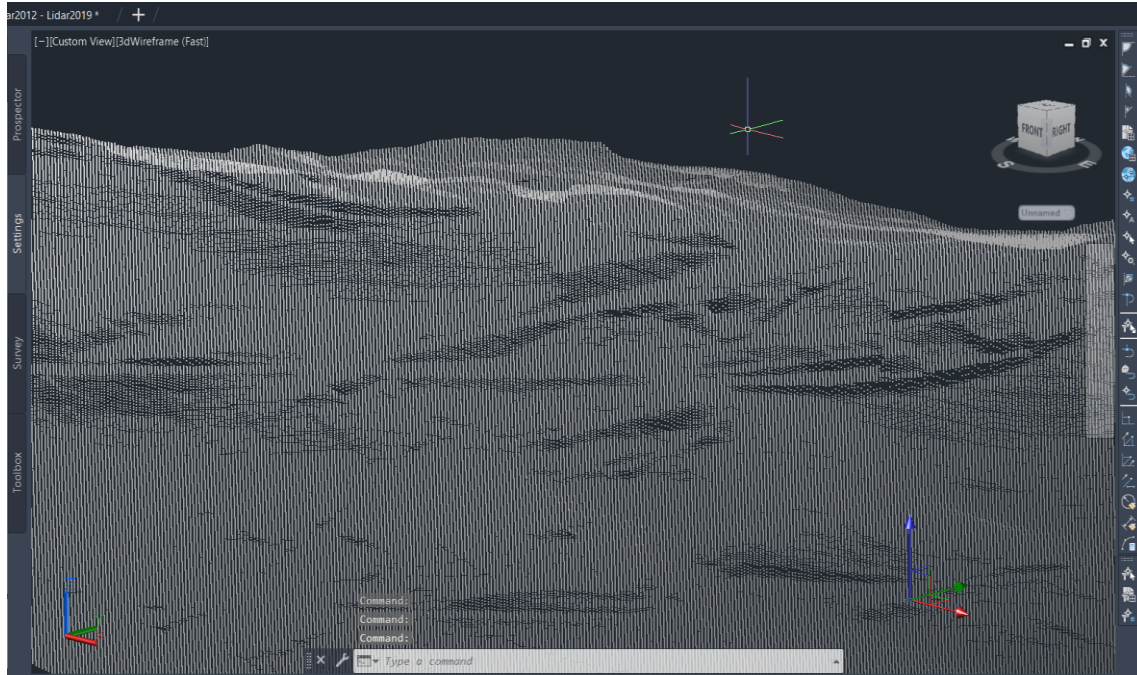


Figure 11 Importing cloud data 2013 from ReCap software into Civil 3D software [42]

As we imported the cloud data from the ReCap software into the Civil 3D software (Figure 11), we started to create the solid surface from the ground surface in 2013 before the land sliding happened in 2017. Therefore, we created solid surface by adjusting the distances between points and using the technique “No Filter” between the other options of “Planer Average” and “Kriging Interpolation” techniques. The technique “Kriging Interpolation” is a statistical technique, considers spatial correlation and “Planer Average” consider surfaces by averaging the elevations of points, but since our points are from Lidar data as the DEM data and they are related to the bare ground surface, so the best choice was the “No Filter” option to create the solid surface between the points data. Therefore, in this regard the surface from the condition before the land sliding happened was created from the source point cloud data of “National Geoportal” related to the history 2013.

4.2 Collecting Point Data After Land Sliding happened

4.2.1 Collecting from Drone Survey

A drone survey was conducted to investigate the deformed ground surface of the study case area in 2019, after the land sliding happened. Since the drone survey data were raw data which were

not clean and did not represent the bare ground surface and they included all of the details such as vegetations, structure, grasses, etc. so cleaning and noise filtering and so on showed necessary to create a good cloud data for the analysis. In this regard, the following procedure was performed to prepare a clean data from the drone survey data.

4.2.1.1 Cleaning Point Cloud Data Using CloudCompare Software

The drone cloud data were imported into the CloudCompare software to clean data and create ground surface. It should be mentioned that CloudCompare is a free and open-source 3D point cloud processing software which is used in surveying, civil engineering, geospatial analysis, and 3D modelling and it is specialized in handling LiDAR data, laser scans, and 3D mesh comparisons, which made it essential for our project as we had a high-density cloud data from drone surveying which needed to be cleaned and filtered [44].

To clean the data, we 1st used the “Cloth Simulation Filter (CSF)” method [16]. In this setting, “cloth resolution” option refers to the grid size of cloth which was used to cover the terrain, and we selected 0.1 (unit in meter), since the smallest cloth resolution we set, the finer DTM we got. In addition, we set the “max iterations” to 500, as it refers to the maximum iteration times of terrain simulation, which was enough for most of the scenes. For the “classification threshold” we set 0.2 as the threshold (unit was in meter) to classify the point clouds into ground and non-ground parts based on the distances between points and the simulated terrain, which resulted the best achievement, and by using the options “segment” and “cross section” we separated noise from the ground surface.

In the following, by using the “Statistical Outlier Filter” (SOR) technique we removed the points far from their neighbours and by performing the option “Noise Filter” we filtered the noise in the cloud data.

In the final step of the CloudCompare software operations, we performed the “Rasterize” technique to convert our 3D point cloud data into the Digital Elevation Model (DEM) by using the minimum elevation to be able to export elevation grids to the next step software “ReCap” for the next operation. Figure 12 and Figure 13 show the condition of the DEM data before and after editing process in CloudCompare software respectively. As it could be seen from the Figure 13, because of the coverage of trees, vegetation, buildings, etc. on ground, many points are removed and cleaned to create the bare ground surface by using the CloudCompare software, and this is the disadvantage of using the drone survey compared with the using of Lidar data ((Light

Detection and Ranging) which could detect the bare ground surface and there is no need to remove and clean point data from the ground surface.

Therefore, by using the drone survey and removing objects points from ground surface, we need special software to interpolate and estimate the empty spaces between removed points and fill out the empty spaces, so this procedure in the following may not be accurate compared with providing Lidar data from the study case area. In this regard, we used the Civil 3D software as one of the best software to interpolate the empty spaces between points and reduce the uncertainties about estimation of empty spaces.

This procedure was done, because at the beginning of the project there were no other data after the history of land sliding happened in the region to create coloured-deformation map and compare the deformation that happened in the region after the land sliding in 2017, but by advancing the project to check out the accuracy of the data we were finally being successful to find also the Lidar data from 2019, which was kindly provide by the “Airbus Defence and Space GmbH” Company from Germany free of charge and they provided this data for the research activity, which will be described in **Section 4.2.2**.



Figure 12 DEM data from drone surveyed data before editing points in CloudCompare software [44]



Figure 13 DEM data from drone surveyed data after editing data in CloudCompare software [44]

4.2.1.2 Cleaning Data Using ReCap Software

After working with the CloudCompare software to prepare the DEM data, they were imported into the ReCap software to work on them (Figure 14) and prepare them to be able to be imported into the CAD workflow software to create surface from them, the same that happened in *Section 4.1.2.2*.

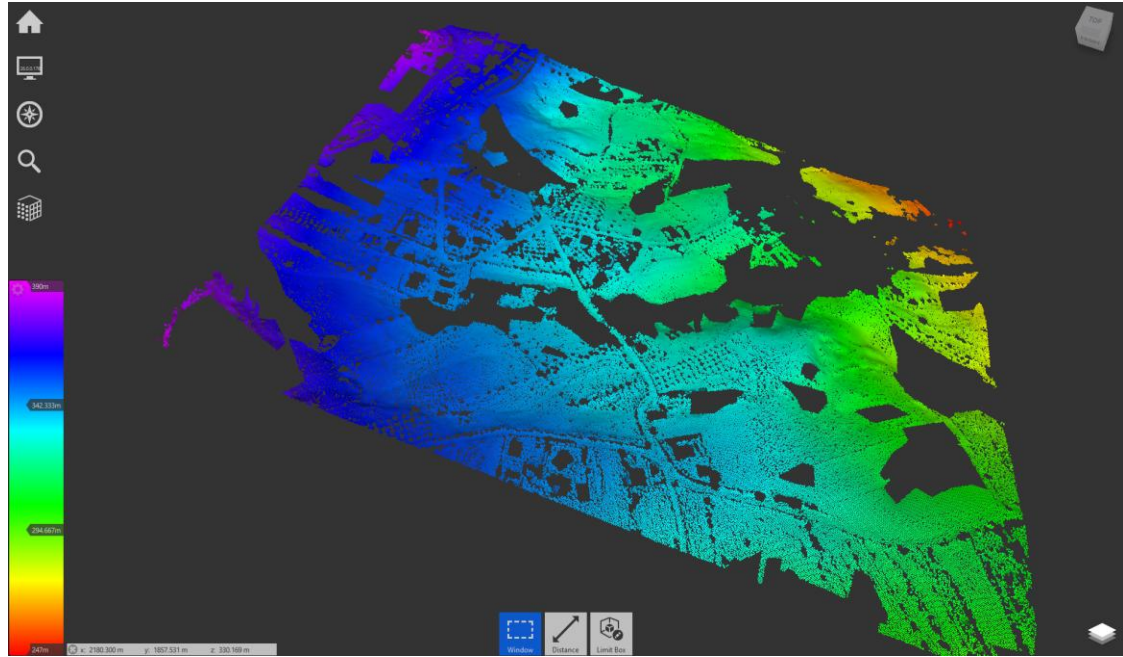


Figure 14 Removing noise, crop, align point cloud data 2019 using ReCap software [41]

4.2.1.3 Creating Solid Surface Using Civil 3D software

After working on drone data from 2019 by using the ReCap software in previous section, we imported them into the Civil 3D software, the same as **Section 4.1.2.3**, to create solid surface, which could be introduced as the deformed ground surface after the land sliding happened in 2017, and lead us to create the coloured-deformation map by comparing the surface from **Section 4.1.2.3** and **Section 4.2.1.3**. To create the coloured-deformation map which could shows what happened in the Ponzano region, we need the deformed solid surface from the history after the land sliding happened in the region, which is performed in this section, and also the deformed surface from region before the land sliding occurred, which was performed in the **Section 4.1.2.3**. Then after that, these two solid surfaces will be compared with each other's to detect the deformation that happened in the region as a coloured-deformation map using the Civil 3D software.

The same as **Section 4.1.2.3** we described before, we imported the cloud data of the history of 2019 drone data from the ReCap software into the Civil 3D software, and started to create the solid surface by adjusting the distances between points and using the technique “No Filter” to interpolate the best results for the empty vegetated spaces between the points, which the result could be seen in Figure 15.

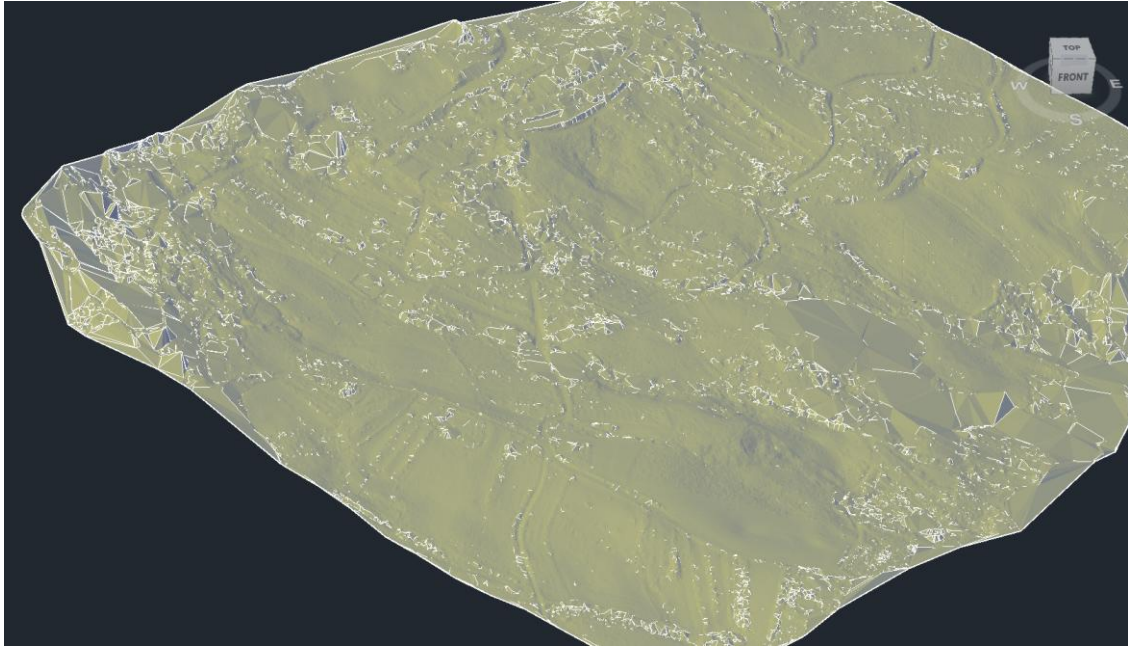


Figure 15 Creating solid surface by interpolating empty spaces using Civil 3D software [42]

In this regard, everything is ready to create the coloured-deformation map to detect the deformation that happened in the study case area after the land sliding on 2017, but as described in **Section 4.2.1.1** by advancing the project and after receiving the Lidar DEM data of 2019 from the “Airbus Defence and Space GmbH” Company, we achieved more high precision data with more accuracy to create our solid surface without the problem of removing the vegetation areas and filling the empty spaces. Thereafter that, in the next section we described the procedure for creating the solid surface using the new Lidar DEM data from 2019 and then we go ahead to create the coloured-deformation map using both drone data 2019 and Lidar DEM data 2019 compared with the Lidar DEM data 2013 and compare their results.

4.2.2 Collecting from “Airbus Defence and Space GmbH” Company

In the following, we achieved the Lidar DEM data from 2019, after the history land sliding happened in the region, with the accuracy 4.6 m Pixel size and started to perform the procedure to create the solid surface and then compare the results with the Lidar DEM data 2013 from **Section 4.1.2.3** to detect the deformation that happened in the region.

4.2.2.1 Creating Point Cloud Using Global Mapper Software

We imported the Lidar DEM data of 2019 in the format of the TIF with the resolution of 4.6 m Pixel size into the Global Mapper software to work on the data for editing (Figure 9) and then exported the data in the point cloud data with the format “XYZ Grid” to be able to work on it and import it into the Numerical FEM solution software to run and analysis.

Since our goal is to compare the data with the data from Lidar DEM data of 2013, before the land sliding happened in the region, and our source Lidar data 2013 were with higher resolution 1.0 m Pixel size (as mentioned in *Section 4.1.2*), so we resampled our source data Lidar DEM 2013 from 1.0 m to 4.6 m Pixel size to be the same as the Lidar data of 2019 from Airbus Company and created the same comparison between them.

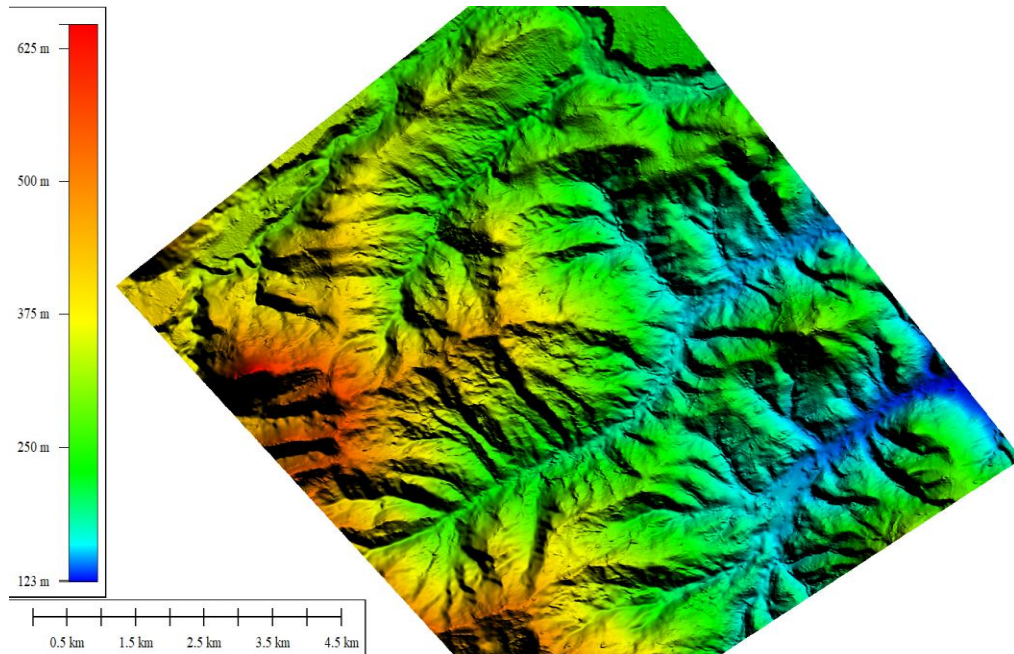


Figure 16 Lidar DEM data 2019 from the study case area with resolution 4.6 m Pixel size [45]

4.2.2.2 Creating Cloud Data Using ReCap Software

After performing by the Global Mapper software and extracting data from it, we imported them into the ReCap software to work on the terrain data and managed 3D points and prepared them to be able to be imported into the Civil 3D software to create our solid surface to be prepared for the coloured map (Figure 17).

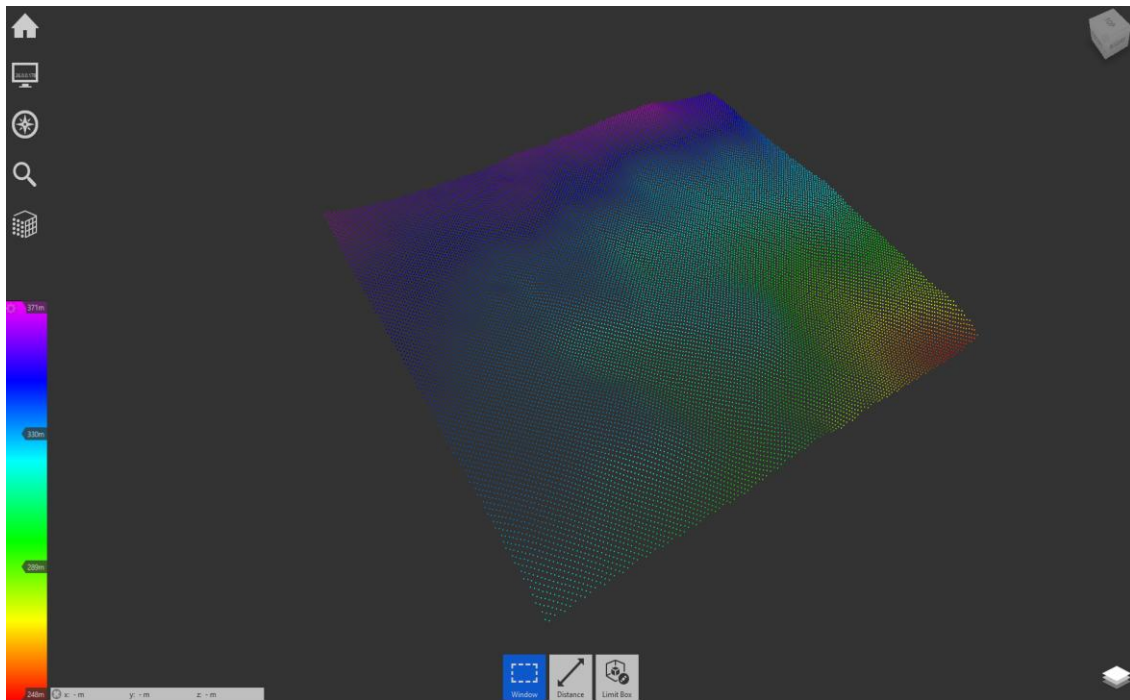


Figure 17 Removing noise, crop, align point cloud data from Lidar DEM data 2019 [41]

4.2.2.3 Creating Solid Surface Using Civil 3D software

After preparing the data through the ReCap software, we imported them into the Civil 3D software to create our solid surface from the source of Airbus Lidar DEM data of 2019. In this regard, we started to create the solid surface by adjusting the distances between points and again the same as previous using the technique “No Filter” between the other options of “Planer Average” and “Kriging Interpolation” techniques, which showed the best solid surface from the bare ground surface (Figure 18).

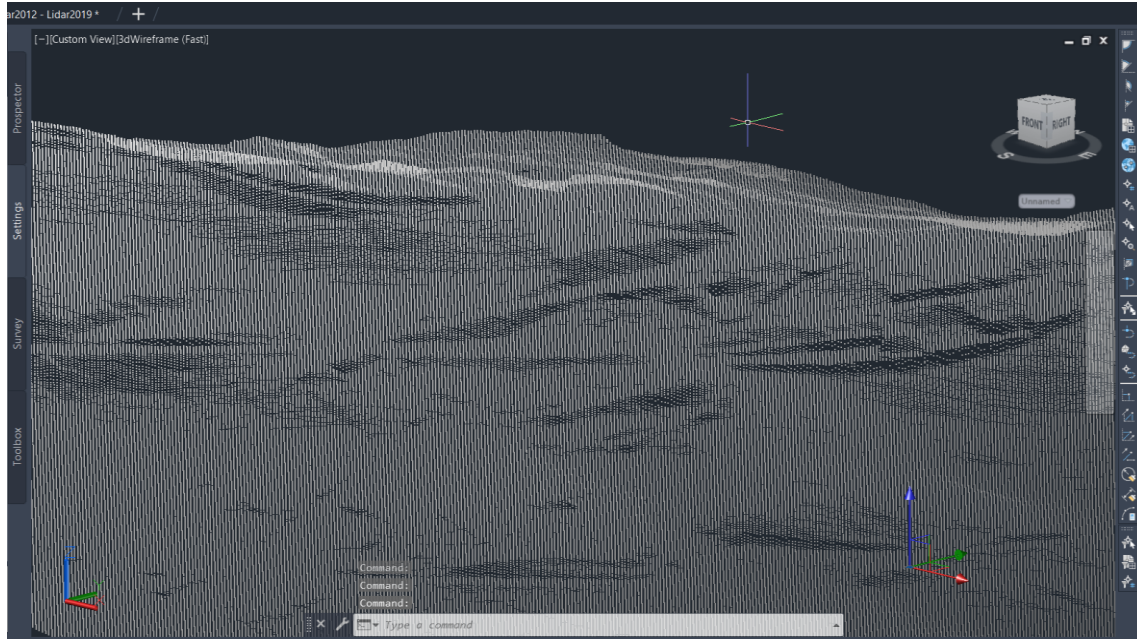


Figure 18 Importing Airbus Lidar DEM data 2019 from ReCap software into Civil 3D software [42]

4.3 Creating Coloured-Deformation Map

After creating our solid surfaces from Lidar DEM data 2013 (before the land sliding) as well as the Lidar DEM data 2019 and drone data 2019 (after the land sliding) in Civil 3D software, we started to create our coloured-deformation map to see what happened in the region and how was the deformation in the study case area, and then after that we go head to create our Numerical FEM code simulation to create the same simulation as a Back-Analysis EFM calculation method. In this section, to create our coloured-deformation we use the technique TIN-Volume surface in the Civil 3D software.

TIN Volume Surface in Civil 3D is a special type of surface used to calculate and display the difference in elevation between two TIN surfaces (solid surface), which is typically an existing ground surface and a design surface. This means TIN Volume Surface comparing two TIN surfaces, one is the “Base Surface” that we defined as the Lidar DEM data 2013 as the 1st base ground surface, and the 2nd was the “Comparison Surface” which was the proposed surface as the Drone data 2019/Lidar DEM data 2019. By this technique and ability of the Civil 3D professional software, we calculated the deformation that happened in the region after the land sliding happened in the region in 2017. In the following we described the procedure, and the results achieved in this section.

4.3.1 Coloured Map from Lidar DEM Data 2013 and Drone Data 2019

As mentioned in previous part, we created our 1st coloured-deformation map using the technique TIN Volume surface by choosing the Lidar data 2013 as the base and the Drone data 2019 as the comparison surface, and the result is attached in Figure 19.

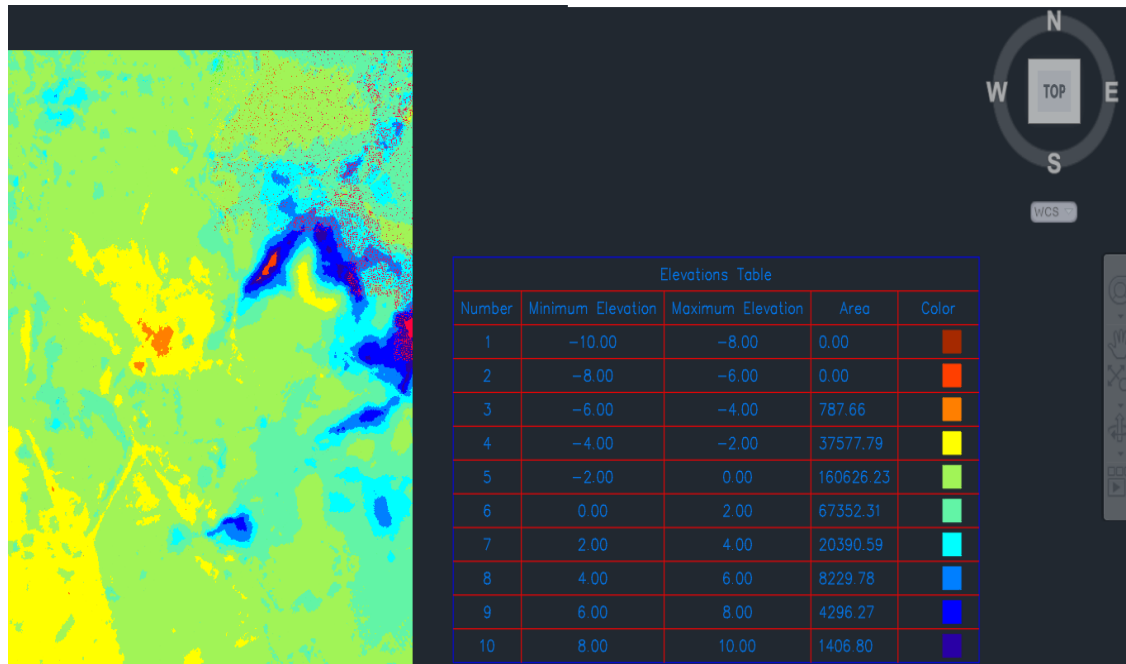


Figure 19 Coloured-deformation map from Lidar DEM data 2013 and Drone data 2019 [42]

4.3.2 Coloured Map from Lidar DEM Data 2013 and Lidar DEM Data 2019

In this section, we created our 2nd coloured-deformation map again by using the technique TIN Volume surface by choosing the Lidar data 2013 as the base, but in this section by choosing the Lidar DEM data 2019 as the comparison surface and the result is attached in Figure 20.

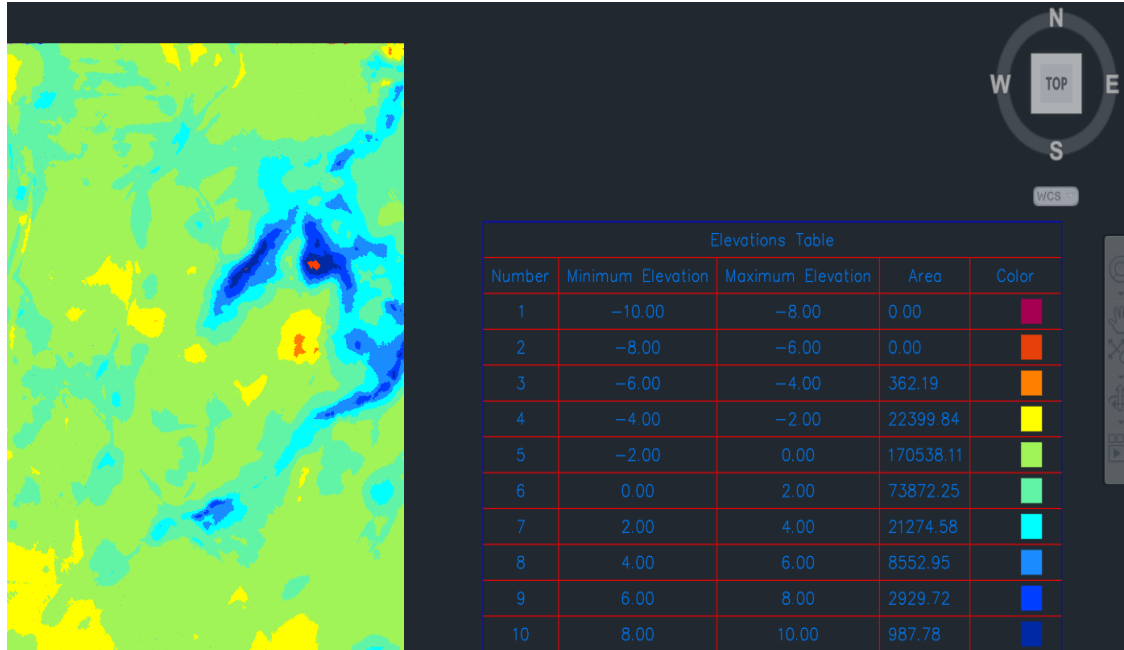


Figure 20 Coloured-deformation map from Lidar DEM data 2013 and Lidar DEM data 2019 [42]

4.3.3 Choosing Best Coloured-Deformation Map

As discussed in previous sections, the coloured-deformation map was computed by using the technique “TIN Volume Surface”. Since described in previous section, at the first of evaluation we had access to surveyed drone data 2019, but since drone data survey also the vegetation, trees, buildings, obstacles, ..., so always there is need of cleaning, rasterizing, noise filtering, etc. to clean the data and prepare the surface and in this procedure always some point would be deleted, as the drone survey does not have the ability to cross the vegetation area and survey the bare ground surface.

Therefore, we did all of the best techniques as described before, using different professional and open-source software such as CloudCompare, QGIS, Global Mapper, ReCap, Civil 3D software to perform the best results and create the minimum uncertainties. In the following of the project, as we achieved the Lidar DEM data from the “Airbus Defence and Space GmbH” Company with good accuracy 4.6 m Pixel size and since the specification of Lidar (Light Detection and Ranging) data is to create high-resolution and accurate 3D models of the earth's bare surface by using remote sensing method that measures distances through laser light and detect surfaces, so the accuracy of the result is higher.

In this section, to compare the results and the accuracy of data and since our calculations in the following is referred to the Numerical FEM code software and we need high accuracy data from coloured-map to compare the results, so we chose the coloured map of Lidar DEM data 2013-2019 (**Section 4.3.2**), although the results of drone was fine, but the Lidar DEM data had more higher accuracy for our Numerical calculations.

4.4 Creating Soil Layers Using Rhinoceros Software

After achieving the coloured-deformation map which depict the deformation happened in the study case area after the land sliding on 2017, we go ahead to the other side of the project, which is creating our Numerical FEM simulation and then performing Back-Analysis by using the “Sensitivity Analysis” to achieve the best results which could be comparable with the coloured-deformation map from previous section.

In this regard, we should at first create our soil layers in the Numerical FEM solution software by referring to the laboratory test data and subsoil layers structures achieved in **Section 3.2** and **Section 2.3** respectively. If these soil surfaces being in the triangulated Irregular Network mesh (TIN surfaces) and imported into the Numerical FEM calculation software, they would contain large number of faces and irrelevant details which could increase the time of Numerical calculations significantly high.

Therefore, we need to simplify and smooth our surfaces, but still accuracy, before importing them into our Numerical FEM calculation software. In this regard, we use the Rhinoceros powerful 3D modelling software that is known for its ability to create, edit, and analyse complex 3D geometries (the same as in this project), and could create NURBS (Non-Uniform Rational B-Splines), which we need in this project [43]. Therefore, by using the Rhinoceros powerful 3D modelling software we created our soil layers of the project (Figure 21) and then simplified and smooth surfaces by the technique NURBS surfaces (Figure 22) using the Rhinoceros professional 3D software and then imported them into our Numerical FEM solution software in the following section.

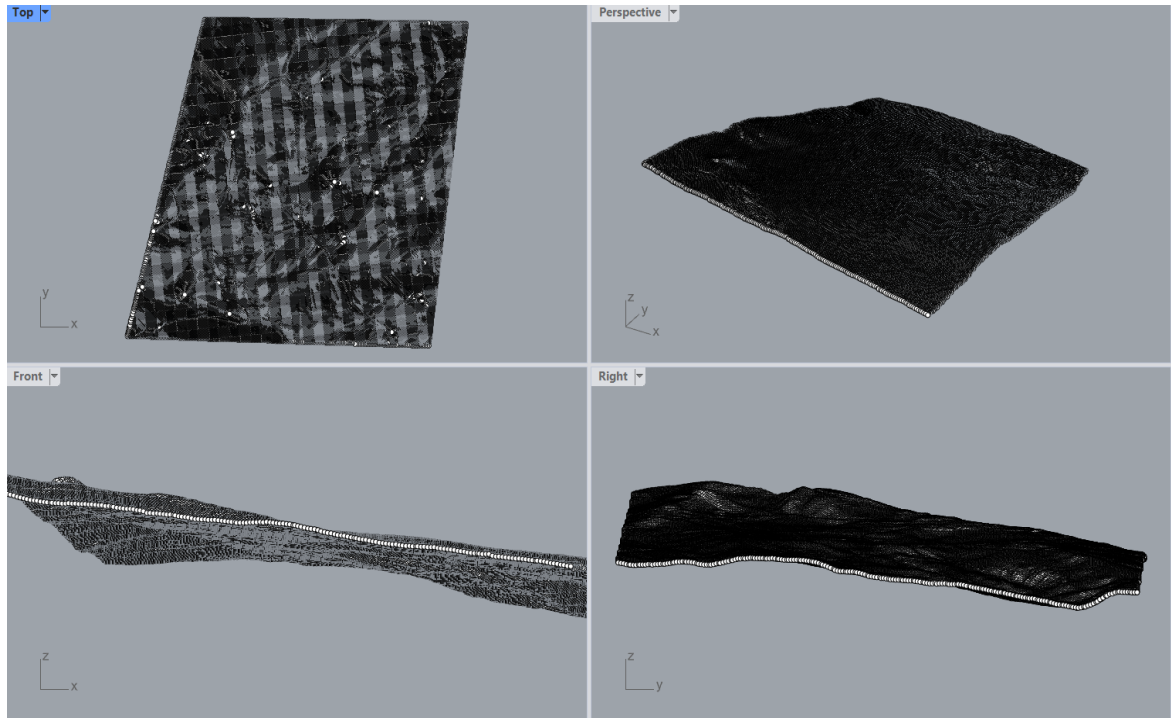


Figure 21 Creating soil layers using Rhinoceros powerful 3D software [43]

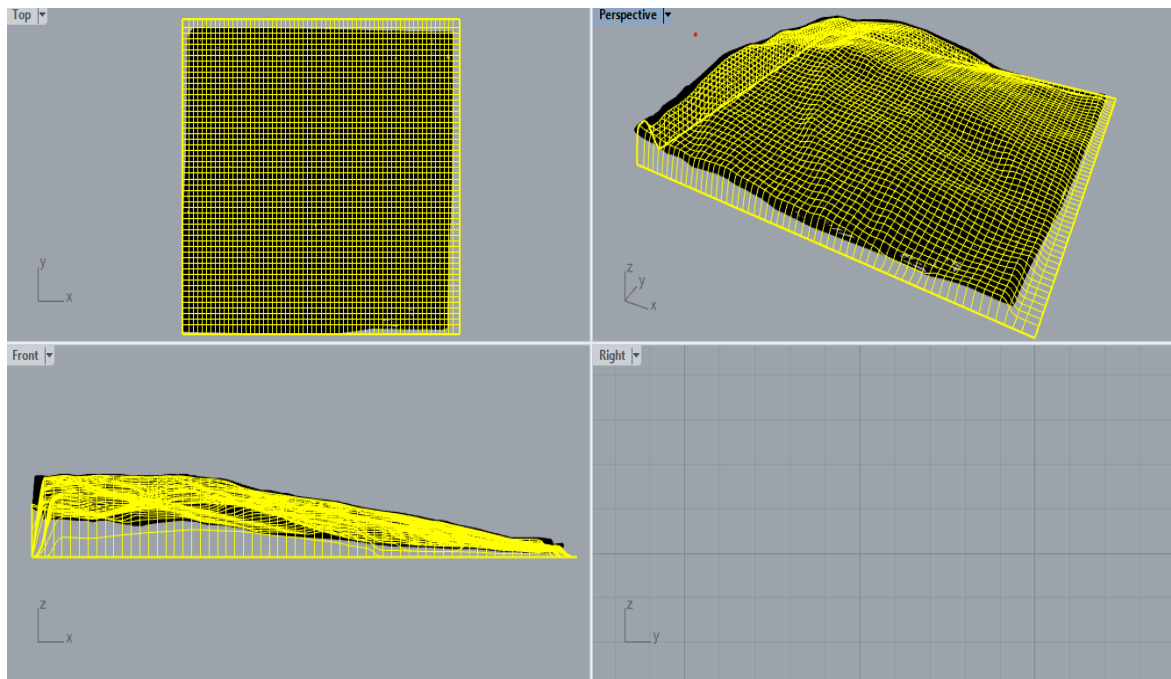


Figure 22 Creating NURBS surface for soil layers using Rhinoceros powerful 3D software [43]

4.5 Simulation in Numerical FEM Code Plaxis 3D Software

After creating the soil layers and turned them into NURBS surfaces to smooth our calculations in Numerical method, we started to work with a Numerical FEM code software that could have high ability and performance in FEM calculations as well as being able to be connected to the remote sensors using language programs like Python through HTTP based platform, which will be described in the following.

Therefore, we chose Plaxis FEM code software, which is one the most powerful software in FEM calculations in Geotechnical Engineering and it covers all of the ability and has the platform to be connected to language program for synchronizing data from the field, and ability to synchronize data as an IoT (Internet of Things), which is the main step to go forward to create Digital Twin Technology in Geotechnical Engineering [46, 47]. Consequently, we start to create the project in FEM calculations and imported our soil layers as the NURBS surfaces from previous sections into Plaxis FEM code software as is shown on Figure 23.

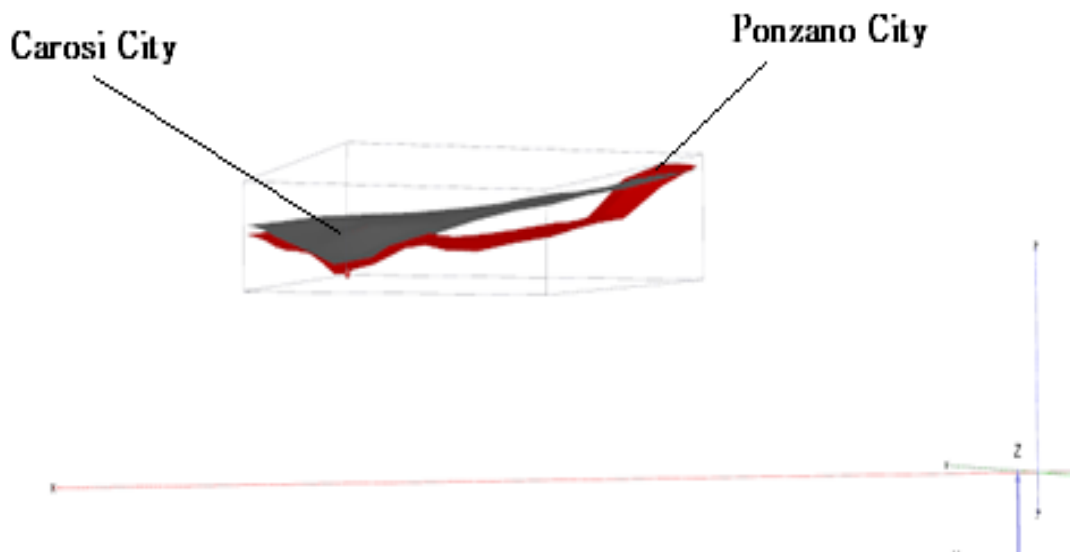


Figure 23 Importing soil layers as NURBS surfaces into Plaxis FEM code 3D software [46]

By importing the soil NURBS surfaces into the Plaxis FEM 3D software, we started to create our simulation and defined the parameters from the laboratory data from **Section 3.2**. Then after that, we created very fine mesh for the simulation and prepared the model for analysing (Figure 24).

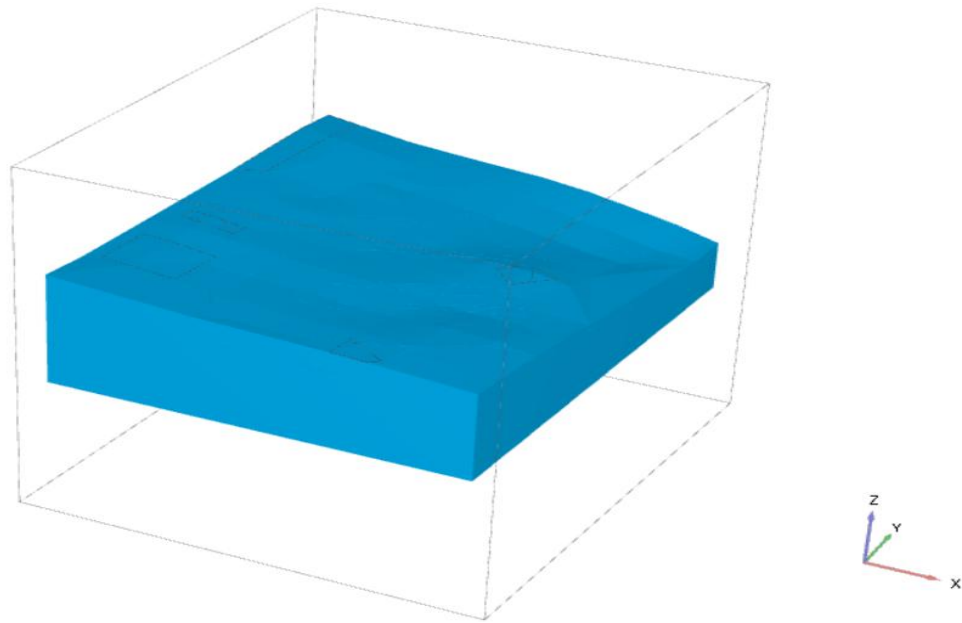


Figure 24 Preparation the simulation for analysing in the Plaxis FEM 3D code software

After analysing the model we achieved the results which was more matched with the coloured-deformation map and showed the results achieved by the Numerical FEM calculation method had a good performance with the coloured-deformation map of the Lidar DEM data 2013-2019 achieved in **Section 4.3.2**. The result of the Plaxis FEM 3D code software is attached in Figure 25. This should be mentioned that Plaxis FEM code software could calculate the analysis until failure mechanism and could not continue the deformation after failure, but by considering the scale for the value, the pattern is the same in both of the Plaxis FEM analysis and the coloured-deformation map of the Lidar DEM data 2013-2019.

The results of this part of the project were first presented in the “**WMESS-World Multidisciplinary Earth Sciences Symposium**” as the Scopus conference in August 2023, and then after that it was published in 2024, and it is now available in the “**Journal of the Polish Mineral Engineering Society**” [1].

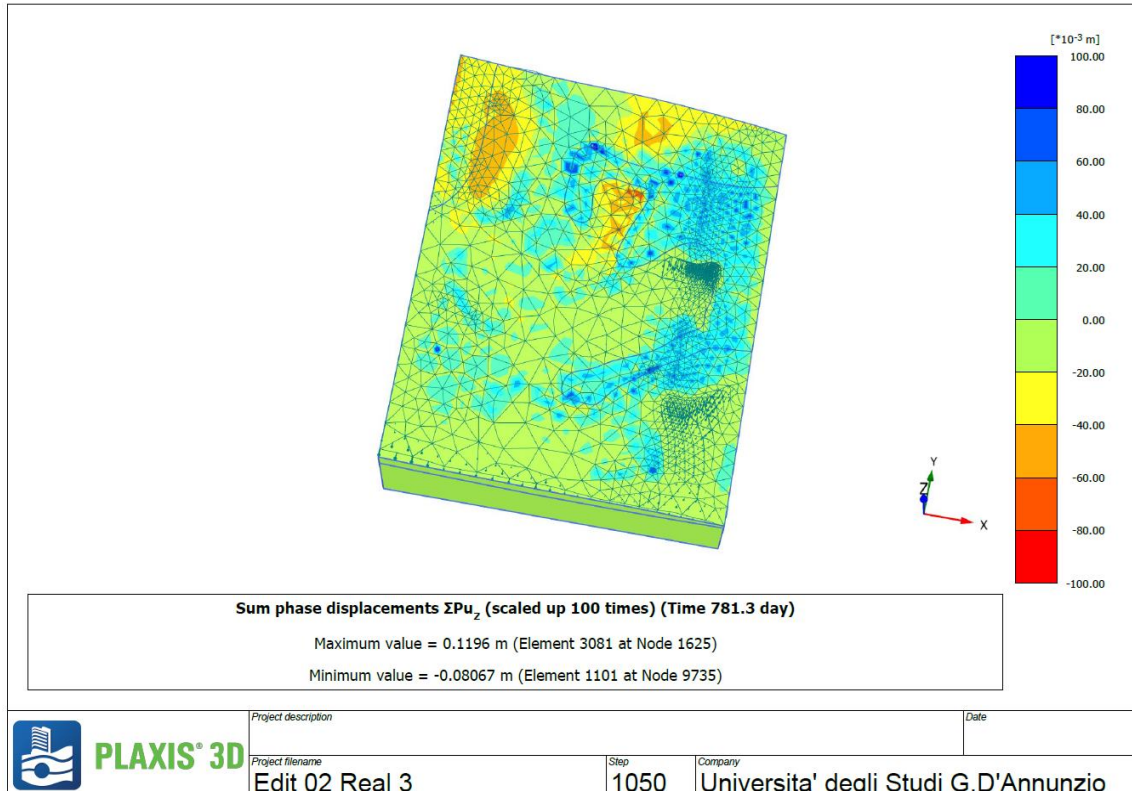


Figure 25 Plaxis FEM code 3D simulation deformation results in Z direction after analysing

5 2nd Phase of Project as Creating Digital Twin Technology for 1st time in Geotechnical Engineering

In the 2nd Phase of the project, by referring to the previous section and by considering my idea when I applied for this university, it was about creating the Digital Twin Technology for the 1st time in Geotechnical Engineering, and it was applicable by using the Plaxis FEM code software. The reason for choosing the Plaxis FEM code software was that it is one of the rare Geotechnical Engineering software that has the ability to be connected to the remote sensors as it has the HTTP based API (REST HTTP API) for Python scripting, known as Plaxis Remote Scripting Interface, which allows us to automate and interact with the software programmatically and we can use this API to input data, modify models, run simulations, and extract results (Figure 26).

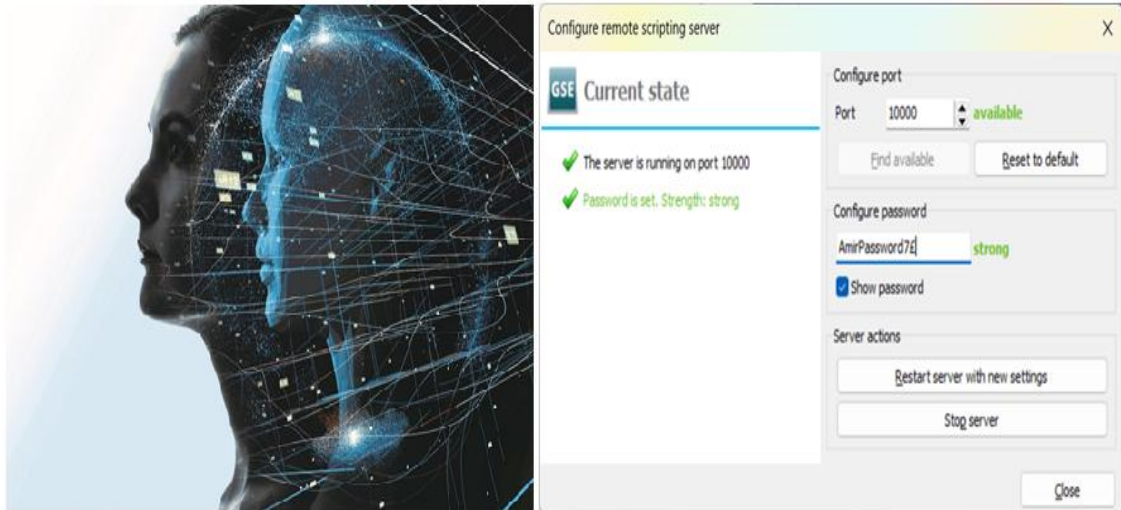


Figure 26 HTTP based API (REST HTTP API) of Plaxis FEM code to connect Python scripting for remote sensors [46]

Plaxis FEM code 3D software has the Boilerplate which could be connected to the remote sensors using the following approach:

`s_i` bound to object represent Plaxis input application (Remote Script Server)

`g_i` bound to global object of current open Plaxis model in input and used when we want to make changes in the model

which could synchronize data with remote sensors automatically. IDE (Integrated Development Environment) is intended for programming in the Python language. In addition, Plaxis FEM code has also the SciTE platform, which is an editor that is shipped with PLAXIS application and is a simple editor that can be used to write Python code and execute it via the PLAXIS Python API and it is free to distribute and light to pack and we can fully run a Python script with SciTE. Therefore, Plaxis FEM code also has simplified and provided some approaches in this direction to create the Digital Twin Technology in Geotechnical Engineering.

Besides that, we can also use the Jupyter Notebook, which is a notebook where we can include code and text and use it to run it with PLAXIS or even we can use also the Spyder, since it is an own Python kernel and is a traditional IDE environment with powerful debugging and data

exploration features, suitable for large projects and more detailed script development, which could be also applicable in this project.

To achieve this goal, we can choose the “HTTP Requests” for the communication Protocol of the remote sensors, that could be connected to our Plaxis FEM code software.

In this regard, we can physically connect remote sensor to PC (for example for Serial communication use USB cable) and we connect remote sensor to the Spyder IDE and then start to synchronize data. We also think about that, as if the physical remote sensors could be very expensive and cost effective for our project, there are *Virtual Sensors* and *APIs Sensors* to Simulate Actual Remote Sensors and could do their effects in the same way. We searched for that about finding *Virtual Sensors* and *APIs Sensors* for the project and there are some sources that provide API Sensors as the following:

- OpenWeatherMap API: It provides weather data that can be used to simulate environmental sensors [48]

In Figure 27, I registered for the “OpenWeatherMap” website which provides API sensor data, just as an example to show the procedure

- Online Mock APIs: It includes some popular mock APIs for testing and development purposes

Mockaroo: Allows to generate realistic test data in JSON, CSV, and other formats

JSONPlaceholder: Provides a simple REST API for testing and prototyping

- ThingSpeak: An IoT analytics platform service that allows to aggregate, visualize, and analyse live data in cloud

- Random Data Generators: Custom Python scripts that generate random or patterned data

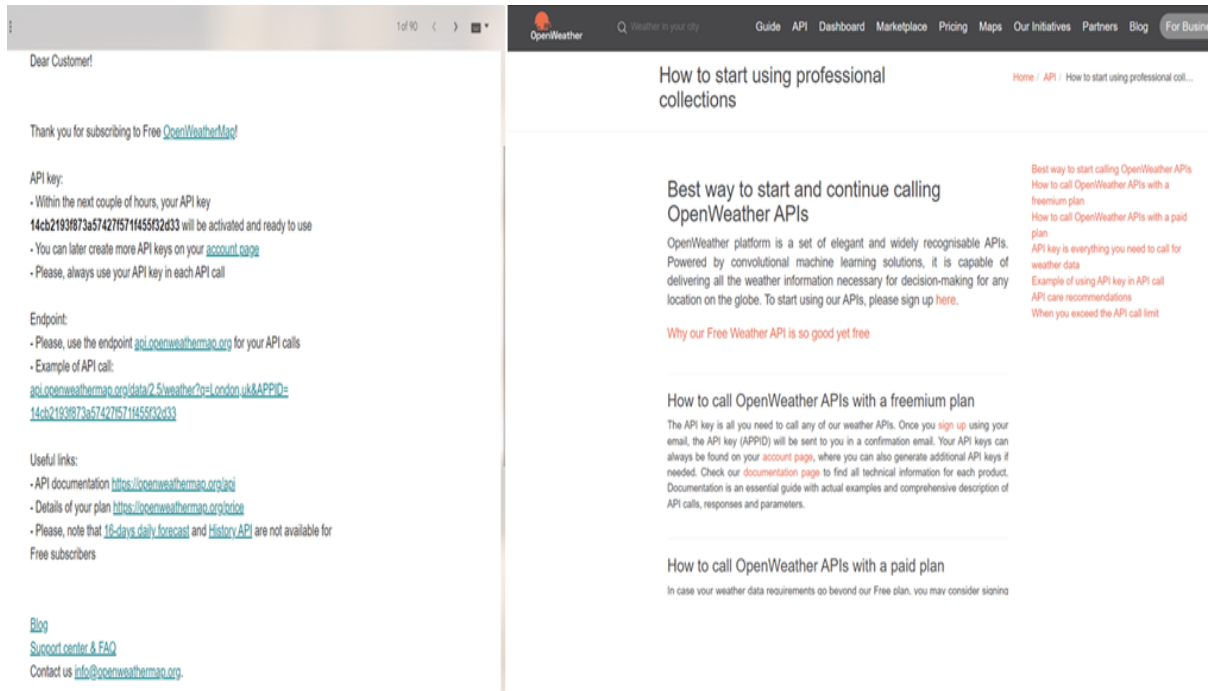


Figure 27 Registered “OpenWeatherMap” website that provided APIs sensor data [48]

As an example in Figure 28, I just showed the template Python language program which could be written to show how the API sensors, which I registered in the “OpenWeatherMap” website, could be connected to Plaxis FEM code software by building a boiler plate platform and using the HTTP based API (REST HTTP API) for connecting and as an intermediate between API sensors and Plaxis FEM code software [49].

```

pip install requests
import requests
from plxscripting.easy import *

def get_weather_data(city, api_key):
    url = f'http://api.openweathermap.org/data/2.5/weather?q={city}&appid={api_key}'
    response = requests.get(url)
    return response.json()

# Step 1: Fetch data from API Sensor
api_key = '14cb2193f873a57427f571f455f32d33' # Replace api_key with OpenWeatherMap API key
city = 'London'
weather_data = get_weather_data(city, api_key)
temperature = weather_data['main']['temp']

```

```

# Step 2:
Connect to Plaxis and feed data and create Boilerplate for Plaxis
s_i, g_i = new_server('localhost', 10000, password='AmirPasscode67')
# new_server establishes a connection to the Plaxis server

# Set the temperature data in Plaxis
g_i.setmaterial("SoilMaterial", {"temperature": temperature}))

print(f"Temperature in {city} ({temperature}K) has been set in Plaxis.")

.....

s_i, g_i = new_server('localhost', 10000, password='AmirPasscode67')
s_i.new()

g_i.SoilContour.initializeRectangular(0, 0, 70, 25)
borehole = g_i.borehole(0)
g_i.soilLayer(25)
g_i.setproperties("ModelType", "Axisymmetry")

material1 = g_i.soilmat()
material1.setproperties(
    "MaterialName", "Soil",
    "Colour", 15262369,
    "SoilModel", 1,
    "gammaUnsat", 20,
    "gammaSat", 20,
    "temperature",
    "Gref", 96150,
    "nu", 0.3,
    "cref", 0,
    .....

```

Figure 28 Template for writing the Python language program to connect API sensors from the “OpenWeatherMap” website to Plaxis FEM code software by creating boiler plate and using the HTTP based API (REST HTTP API) [49]

To gain this goal I started to apply for the Exchange Period to complete and finish the project abroad in one of the high ranked universities with full equipment and data, and I sent my request. In this step, I received the acceptance to follow and complete my idea from the following high ranked universities in July 2024:

- SAALG Geomechanics Company, which was connected with the Politechnique of Barcelona, Spain
- Head of Department at Hamburg University of Technology, from the Institute of Geotechnic and Construction Management, Germany
- Georgetown University in Washington DC, United States America
- Department of Environment and Resource Engineering, Geotechnics & Geology, the Technical University of Denmark (DTU)

- Department of Geotechnical Engineering, Structural & Earthquake Engineering
University of British Columbia Vancouver Campus Canada
- *Institute of Soil Mechanics, Foundation Engineering and Numerical Geotechnics* from
the *Technical University of Graz, Austria*
- Université Grenoble Alpes Lan3R, France

After reviewing the acceptance, since I am Iranian and I needed the visa for the United State America and also Canada and as my three years PhD period was going to be finished by the end of February 2025, so I had a very limited time to decide and accomplish and I could not engaged myself with the visa procedure in that short time period. Therefore, I decided between the other's option and since I liked to go to Technical University of Gras (TU Graz) in Austria as for me as the Geotechnical Engineer, "Karl von Terzaghi" the father of soil mechanics and geotechnical engineering was from TU Graz, so I decided to accomplish my PhD project in this university under the abroad supervision of professor Franz Tschuchnigg and abroad co-supervision professor Helmut Schweiger.

In this regard, I joined this university on 17th July and performed the project, and the 1st meeting was scheduled on 8th August with the presence of 8 PhD candidates, professor Franz Tschuchnigg, professor Helmut Schweiger and two other professors from this university, and also the online presence of my supervisor professor Nicola Sciarra and my co-supervisor professor Antonio Pasculli.

5.1 Critique to Work on Digital Twin Technology

As I mentioned I joined the "Technical University of Graz" from 17th July 2025 under supervision of professor Franz Tschuchnigg and co-supervisor professor Helmut Schweiger to continue the project. According to the meeting on 8th August, it was mentioned that since the land sliding happened in a very large scale around 1.8 km in distance (referring the Section A-A on Figure 3 and Figure 4) and two main land sliding that happened in the region in Section A-A (relevant to

Figure 2), so simulation such a very big scale in 1.8 km in length and in at least 0.5 km in width in 3D Numerical simulations with very fine mesh sizes (around 3-4 m size) by considering the flow of high intensive precipitation in soil layers will take at least around some weeks just to finish 1 simulation and run.

Therefore, as we wanted to connect remote sensors to the simulation also, it would take times to do it and referring my remaining 5 months PhD period, which my PhD 3-years period was going to be finished by the end of February 2025, so they suggested to go deeper in the simulation and focus on the Section A-A (Figure 4) simulation, which two main major land sliding mechanisms was happened, and since there were some limited laboratory data from this specific zone, perform the Back-Analysis and do the Sensitivity Analysis for this specific zone in 2D Numerical simulation to match and comparable with the reality and failure mechanism that happened in this specific path (Section A-A of Figure 3).

Therefore, the following procedures as described in the following, which has been done in “*Technical University of Graz*” under the supervision of *professor Franz Tschuchnigg* and co-supervisor *professor Helmut Schweiger* was done to perform a very complete and deep analysis on what happened in the path of the Section A-A and do the Sensitivity Analysis to match the data with reality by using Plaxis FEM code 2D software.

6 3rd Phase of Project on Back-Analysis in Plaxis FEM 2D

6.1 Prerequisite Definitions

In the Geotechnical Engineering, Back-Analysis (or Back-Calculation) refers to the process of determining unknown Geotechnical parameters or validating a model by comparing observed field behaviour (e.g., displacements, settlements, pore pressures, or failure surfaces) with the results of Numerical simulations. Back-Analysis requires the “Sensitivity Analysis” in the Numerical FEM calculation to calibrate the model behaviour with the reality by considering and changing the parameters and specification to influence the output results and achieve the best results comparable with the reality. There are different types of Sensitivity Analysis like “one-at-a-time (OAT)” analysis or “global sensitivity” analysis. In the “global sensitivity” we vary multiple parameters together, which would be more complex and often probabilistic. For the “one-at-a-time (OAT)” analysis, we vary one parameter while keeping others constant, which was

performed in this project to achieve the best results that could be comparable with the reality through the Numerical FEM Plaxis 2D code software.

As mentioned in **Section 3.2** there are limited laboratory test data (Table 2) from the path of land sliding failure mechanism in the Section A-A (Figure 3) of the study case Ponzano. In addition, as mentioned in **Section 3.2** there were no underground water flow found in the path of Section A-A, but the precipitation data showed high and intensive values of infiltration before the land sliding happened on 12th February 2017 (**Section 3.1**). As a result, the condition is the infiltration of high and intensive precipitation into unsaturated soil layer, so considering the “Initial Degree of Saturation” should be the most determinative parameter in the Numerical FEM calculations and during the Back-Analysis and the Sensitivity Analysis. Therefore, the Hydraulic Model should be specifically taking into consideration accurately.

Referring the data provided in previous sections, we started to simulate the area and performed the Sensitivity Analysis to calibrate the model behaviour with reality, but before that some prerequisite definitions, as pointed out above, are needed to be described to be more familiar that what is going to be happened and then after that the Back-Analysis and Sensitivity Analysis will be started by using the Plaxis FEM Code 2D software.

6.1.1 “Plan Strain” Model with 15-Nodes elements

In our simulation, we used the “Plan Strain” Model with 15-Nodes elements in our Numerical FEM Plaxis 2D simulation. In this model, the strains in the out-of-plane direction (usually z-direction) are assumed to be zero, but the stresses in the z-direction (σ_z) can exist, due to Poisson’s effect in the model. 15-node elements represent triangular elements with 15 nodes (including mid-side and internal nodes), as declared in Figure 29, and use quadratic interpolation for displacement fields, that is much more accurate, especially for receiving stress gradients, our non-linear material behaviour modelling in Ponzano region, Simulating plasticity and deformation zones in our results and is more accurate in our FEM calculation. Therefore, we used this model for our element in our FEM analysis.

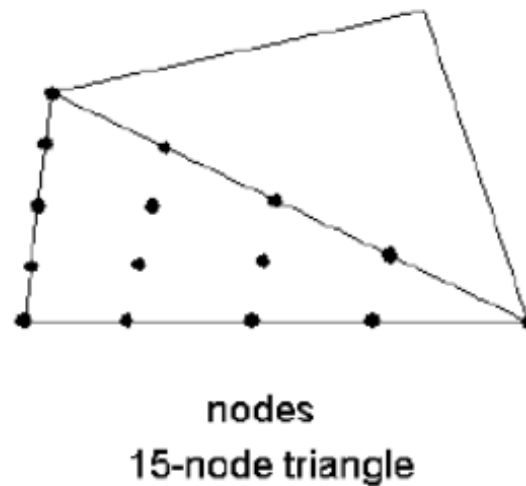


Figure 29 “Plan Strain” Model with 15-Nodes elements in our Numerical FEM analysis

6.1.2 Unsaturated Soils

As mentioned in previous also, there were no underground water flow found in the path of the land sliding failure, and the infiltration is related to the high and intensive precipitation in the region. Therefore, this shows that the soil layer is not completely saturated, and it is unsaturated (semi-saturated) and the water infiltration from the precipitation data should be considered to flow through the unsaturated zones. Accordingly, a partially saturated or unsaturated soil is a multiphase system and, depending on the definition used, consists of multiple phases [5]. The soil solids (particles) phase and the water and air phases could be considered for the phases of the system (Figure 30).

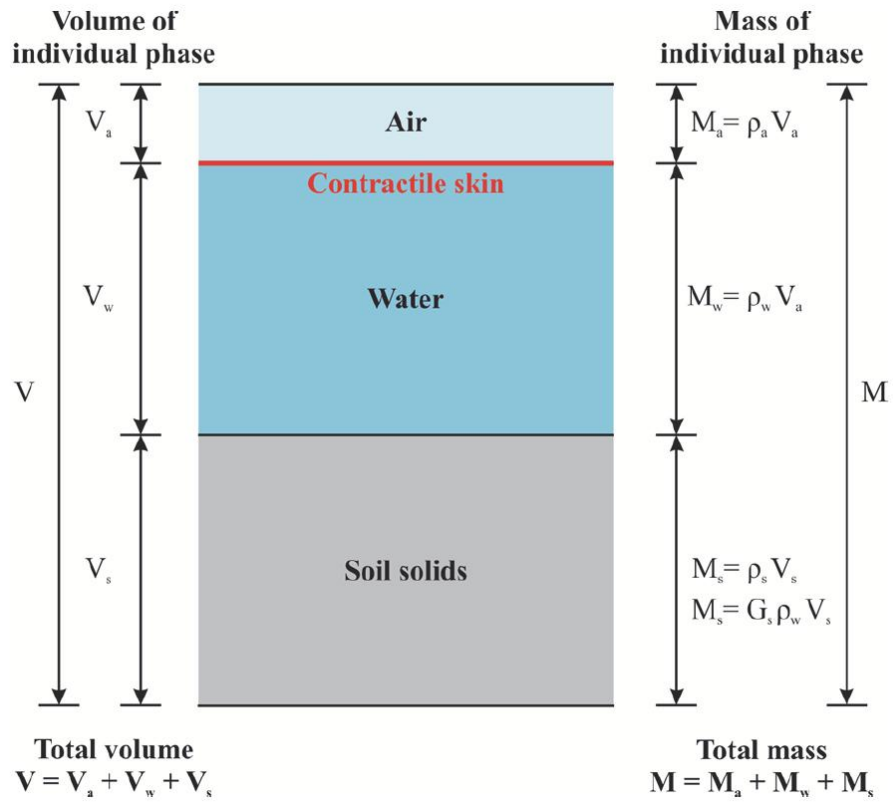


Figure 30 Phase diagram for unsaturated soil [17]

6.1.3 Continuity Equation

Hydraulic conductivity regulates the rate of water flow through a porous medium. When performing steady-state or transient flow analysis of an incompressible fluid (e.g. water) through a porous medium such as soil, the coefficient of permeability of the soil is the main soil property required. In order to describe transient fluid flow in unsaturated soils under isothermal conditions, the principle of mass conservation can be applied. This principle, which is also called the continuity principle, states that the rate of water loss or gain is equal to the net flow into and out of a given soil volume [5].

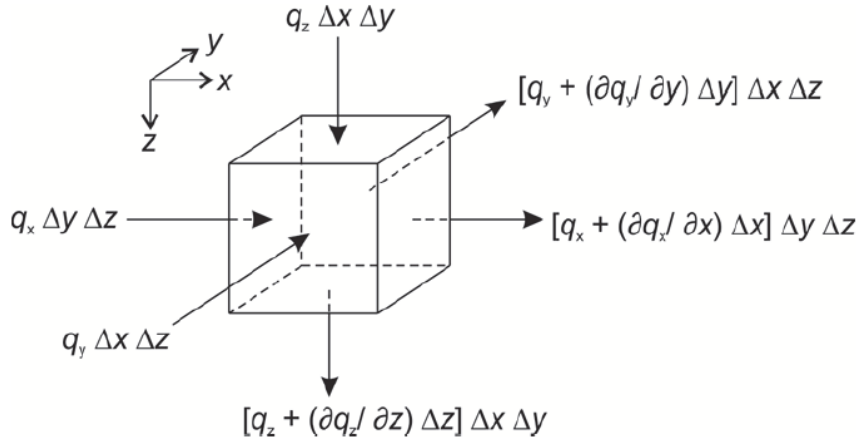


Figure 31 Principle of flow through the elemental volume of soil [18] (original work of Richards (1931b) and Terzaghi (1943))

Therefore, under the assumption that there are no changes in water density, the governing equation of transient or unsteady fluid that flows through an elemental soil volume, either saturated or unsaturated, can be written as follows:

$$-\rho \left(\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} + \frac{\partial q_z}{\partial z} \right) = \frac{\partial (\rho \theta)}{\partial t} \quad (1)$$

where ρ is the density of water (kg/m^3) and q_x , q_y , and q_z are specific discharge (or Darcy fluxes) in the x, y, and z directions, respectively (m/s) and θ is the volumetric water content and t is the time (s). θ is the volumetric water content, which represents the volume of water per unit volume of soil. Therefore, the term $\rho \theta$ represents the mass of water per unit volume of porous medium, and its time derivative $\frac{\partial \rho \theta}{\partial t}$ represents the rate of change of water mass over time stored in the element (specific volume). Since, this equation essentially states the net inflow of water mass into a soil element equals the rate of change of water mass stored in the element.

6.1.4 Darcy's Law for Unsaturated Soil in Numerical calculations

In saturated soils, water flow is usually described using Darcy's law, which states that the rate of water flow through a soil mass is proportional to the hydraulic head gradient of the water in that

soil mass. The equilibrium equation for groundwater flow can be expressed using the following equation:

$$\nabla p_w + \rho_w \underline{g} + \underline{\varphi} = 0 \quad (2)$$

where ∇p_w is referred to as the gradient of the pore water pressure which causes the groundwater flow, ρ_w is the unit weight of water, $\underline{g}$ is the vector of gravitational acceleration $(0, -g, 0)^T$ and $\underline{\varphi}$ is the vector of the friction force, per unit volume, between the flowing fluid and the soil skeleton (solid particles). This friction force depends linearly on the velocity of the fluid and acts in the opposite direction. This can be expressed by the following relation:

$$\underline{\varphi} = -\underline{\underline{m}}^{int} \underline{q} \quad (3)$$

where $\underline{q}$ is the velocity of the fluid (specific discharge) and $\underline{\underline{m}}$ can be written as:

$$\underline{\underline{m}}^{int} = \begin{bmatrix} \frac{\mu}{K_x} & 0 & 0 \\ 0 & \frac{\mu}{K_y} & 0 \\ 0 & 0 & \frac{\mu}{K_z} \end{bmatrix} \quad (4)$$

where μ is the dynamic viscosity of the fluid and k_i is the intrinsic permeability of the porous medium. As hydrostatic conditions are assumed, the term $\rho_w \underline{g}$ is not affected by the gradient of the water pore pressure in the vertical direction, so:

$$\underline{q} = \frac{1}{\underline{\underline{m}}^{int}} (\nabla p_w + \rho_w \underline{g}) \quad (5)$$

Instead of requiring both intrinsic permeability and viscosity, the coefficient of permeability k_{sat} is commonly used in soil mechanics, and can be expressed as follows:

$$\underline{\underline{k}}^{sat} = \rho_w g \frac{k_i}{\mu} \quad i=x,y,z \quad (6)$$

However, the coefficient of permeability in an unsaturated state depends on the degree of saturation of the soil. Thus, for partially saturated soils the permeability can be written as:

$$\underline{k} = k_{rel} \underline{k}^{sat} \quad (7)$$

Thus, to have a form of Darcy's Law which can be used for partially saturated soils, the coefficient of permeability could be modified from the saturated term by including k_{rel} , as shown in equation 7, resulting in the following, which could be used in our Numerical calculation using Plaxis FEM code software:

$$\underline{q} = \frac{k_{rel}}{\rho_w g} \underline{k}^{sat} (\nabla p_w + \rho_w \underline{g}) \quad (8)$$

6.1.5 Compressibility of Pore Fluid and Pure Water

There are certain loading cases in engineering practice (e.g. undrained loading) where the compressibility of the pore fluids affects the stress state of the soil. The magnitude of these induced changes in pore air and pore water pressures in the soil volume is highly dependent on the compressibility of the water-air mixture in the pores, which is derived from the compressibility of the individual water and air components, in consideration of their proportion of the total volume [19]. Various authors have described the fundamental physical laws to determine the compressibility of an air-water mixture (e.g. Schuurmann 1966, Fredlund 1976). In general, the isothermal compressibility of a fluid can be written as [Fredlund et al. 2012]:

$$C_{fluid} = -\frac{1}{V} \cdot \frac{dV}{dp_{fluid}} \quad (9)$$

C_{fluid} is the compressibility of the fluid, V is its volume and dV is the volume change relating to a change in the pore fluid pressure dp_{fluid} . Under the assumption of constant atmospheric air pressure p_{atm} , the compressibility of air C_{air} is inversely proportional to the absolute air pressure, which is defined as the sum of the air pressure in the soil P_{pair} and the atmospheric air pressure p_{atm} [17]:

$$C_{air} = \frac{1}{p_{air} + p_{atm}} \quad (10)$$

According to Dorsey (1940), who presented measurement results of the compressibility of water, pure water is nearly incompressible (Figure 32). According to Fredlund (1976), the compressibility of the air-water mixture can be derived from the compressibility of the individual constituents, considering their proportion of the total volume. For standard applications in soil mechanics, the compressibility of water in soils is assumed to be incompressible [5].

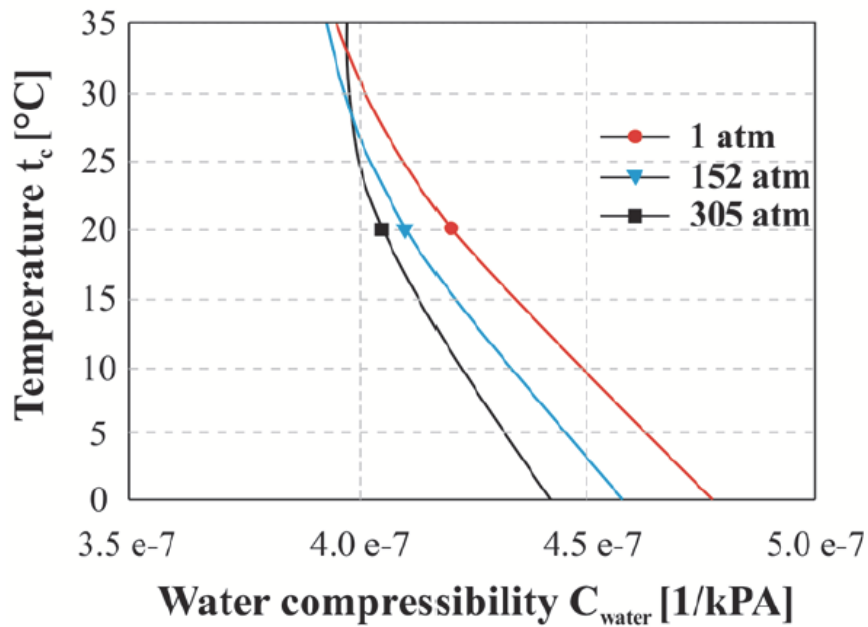


Figure 32 Compressibility of pure water [21]

6.1.6 Hydraulic Models

The relationship between the water content and suction (negative pore water pressure) of a soil can be described by the Soil Water Characteristic Curve (SWCC), which is the most important and sensitive part in this project. Generally, SWCCs are s-shaped curves and plotted in a semi logarithmic scale (suction is plotted in logarithmic scale).

6.1.6.1 Soil Water Characteristic Curve (SWCC)

In general, the shape of SWCCs depends on several soil characteristics. In coarse-grained soils, the SWCC can be related to the grain size distribution. Due to the presence of smaller pores, well graded sands retain less water than poorly graded sands at the same suction levels, so in the same saturation degree, poorly graded sands show high suction stresses compared with the well graded sands.

In fine-grained soils, the liquid limit and the plasticity index quantify the ability of the clay fraction to attract water [22], and thus of the soil to retain water under suction. Finer grained, more plastic soils, which commonly contain a larger clay fraction, and are often denser soils, can retain larger water volumes than coarse-grained soils at same suction levels. Therefore, at the same saturation degree for fine graded soil, higher plastic soils show higher suction stresses than low plastic fine soil [5].

The Figure 33 illustrates the differences in the degree of saturation for various soil types at a same suction values (Barbour 1998), and shows that the curves show different shapes depending on whether the soil is coarse- or fine-grained and at different degree of compaction or plasticity index they show different values for suction stresses. The shapes show that in coarse-grained soils, it is capillarity that dictates the water held under suction, whereas in fine-grained soils, it is the larger plasticity index (I_p) and the clay water interaction that holds the water [5].

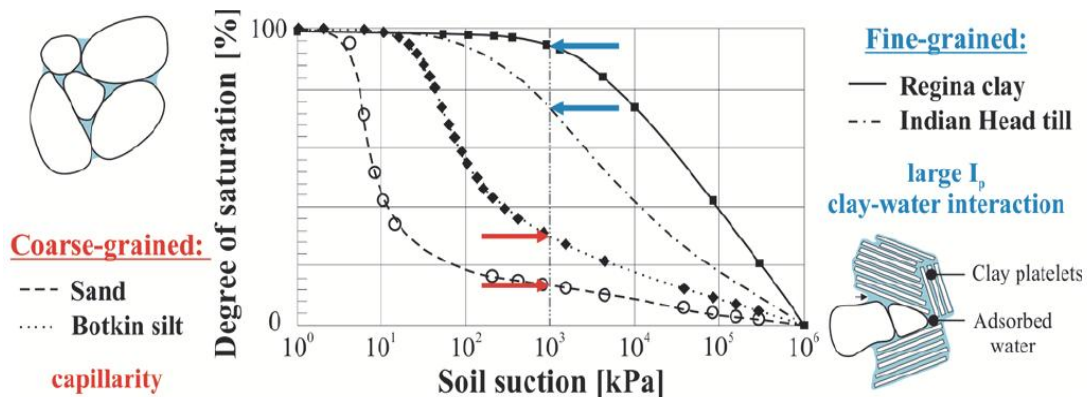


Figure 33 Influence of soil type on the SWCC [23]

6.1.6.2 Hysteresis in Soil Water Characteristic Curve

The relationship between the water content and suction in the soil is hysteretic. This means that the SWCCs of the wetting and drying paths are different and each SWCC equation can be best fitted to either the wetting (adsorption) or drying path (desorption). In general, the drying path shows higher suction values at a similar water content compared to the wetting path (Figure 34). It must be noted, however, that there is no unique or single SWCC for any one kind of soil or even for one specific soil sample and also it is not possible to determine whether a soil is on the wetting or the drying path at the moment of sampling in the field [5].

Therefore, this is the reason why in this project the SWCC behaviour regarding the “initial degree of saturation” is one of the most important parameters in performing the Sensitivity Analysis related to the Back-Analysis calculation in our geotechnical engineering problem. Figure 34 also describe that due to the entrapment of air bubbles in soil layer, full saturation θ_s may not be reached during the wetting process.

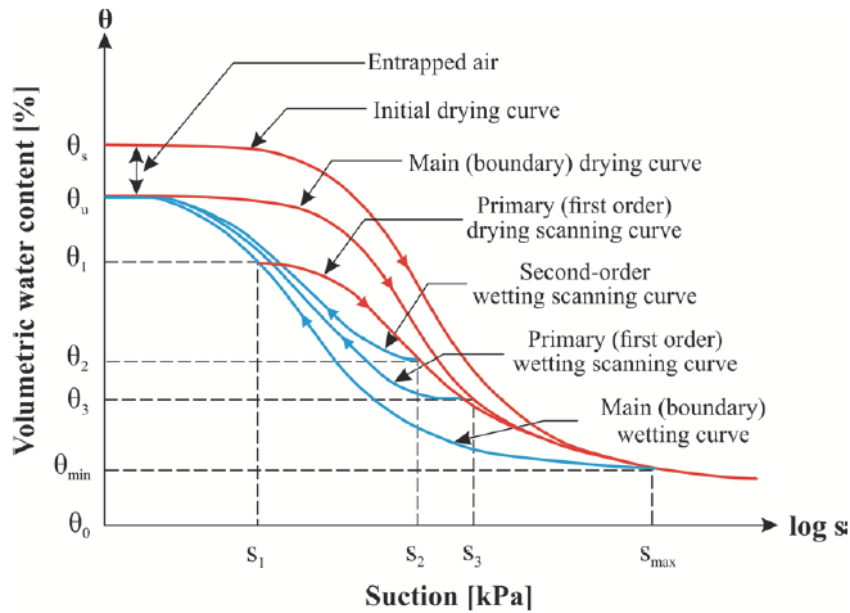


Figure 34 Conceptual boundary to define drying and wetting of unsaturated soils [24]

The water retention hysteresis of soils has been investigated by several researchers (e.g. Mualem 1984), who attributed their observations to several microscopic (particle size) properties and relative macroscopic scales. One of the explanations for this special hysteretic behaviour is

commonly known as the “inkbottle” effect [26], which describes that the hysteretic behaviour is based on the non-uniform distribution of interconnected pores’ size in the structure of the soil. This hypothesis can be schematically illustrated using the analogy of a non-uniform capillary tube. The capillary rise during upwards flow (wetting) is controlled by the larger radius of the tube, whereas the capillary height upon downward flow (drying) is governed by the smaller pore radius (Figure 35).

Another mechanism in order to explain the hysteretic behaviour in soil water retention was suggested by Hillel (1998). This mechanism is generally known as the contact angle or “raindrop” effect and is described as the difference in the liquid solid contact angles for advancing and receding water menisci which is described in Figure 36.

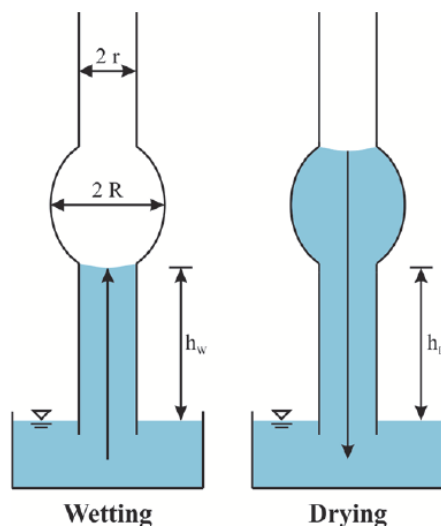


Figure 35 “Ink-bottle” effect illustrated by non-uniform capillary tube [26]

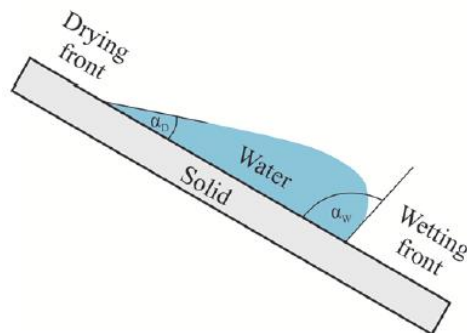


Figure 36 “Raindrop” effect with differing soil-water contact angle during flow

6.1.6.3 Van Genuchten Model

There are numerous models to describe the SWCC mathematically, but in the thesis, the commonly used and widely accepted Van Genuchten model was used in the following numerical analyses [28]. The Van Genuchten function is a three-parameter equation (equation 12), which relates the saturation to the pressure head ϕ_p (equation 11).

$$\phi_p = \frac{P_w}{\rho_w g} \quad (11)$$

$$S(\phi_p) = S_{res} + (S_{sat} - S_{res})(1 + (g_a |\phi_p|)^{g_n})^{g_c} \quad (12)$$

The shape of this function is dependent on the grain-size and pore-size distribution and the mineralogy of the material. S_{res} is defined as residual saturation, which remains even at high suction levels, whereas S_{sat} is the degree of saturation under saturated conditions. There could be some air-bubbles in the water even at saturated conditions which causes S_{sat} generally to be less than one. There are three measurable curve fitting parameters in Equation 12: g_a is related to the air entry value of the soil, g_n is a function of the water extraction rate of the soil once the air entry value has been exceeded and g_c is a function of the residual water content and therefore related to the shape of the curve in high suction ranges.

In order to calculate the relative permeability $k_{rel}(S)$ of the soil in our Numerical calculations, which is in relation to Mualem (1976) and Van Genuchten (1980), first the effective degree of saturation S_e is obtained as equation 13.

$$S_e = \frac{S - S_{res}}{S_{sat} - S_{res}} \quad (13)$$

$$k_{rel}(S) = (S_e)^{g_l} \left[1 - \left(1 - S_e^{\frac{g_n}{g_n-1}} \right)^{\frac{g_n-1}{g_n}} \right]^2 \quad (14)$$

Whereas g_l is a material specific, empirical parameter. The derivative of the degree of saturation with respect to the suction pore pressure can be written as equation 15.

$$\frac{\partial S(p_w)}{\partial p_w} = (S_{sat} - S_{res}) \left[\frac{1 - g_n}{g_n} \right] \left[g_n \left(\frac{g_a}{\gamma_w} \right)^{g_n} p_w^{(g_n-1)} \right] \left[1 + \left(g_a \frac{p_w}{\gamma_w} \right)^{g_n} \right]^{\frac{1-2g_n}{g_n}} \quad (15)$$

6.1.6.4 Datasets for Soil Water Characteristic Curves

To do the best representative of SWCCs relating to the unsaturated soils analyses in a project, the best and idealistic approach for determining the SWCC for soil layers should be determined by performing the laboratory or field tests, which are time-consuming and very expensive, and in this project it was not possible to do that. In this regard, there are several famous different SWCC data sets available for using without considering laboratory or field tests. These datasets distinguish between varieties of different soil types, that could correctly model the flow in unsaturated soils, which is one of the most important parts in this project.

There are two well-known and widely accepted datasets as the HYPRES (HYdraulic PProperties of European Soils) series, which is an international soil classification system developed by Wösten et al. (1999) and the USDA (United States Department of Agriculture) that provides an alternative soil classification series and was developed for agricultural purposes. These two well-known datasets are completely covered by Numerical FEM Plaxis code software, which were used in this project.

By considering the laboratory tests performed for the grain size distributions in different depths (Table 1), the “fine” soil class was defined for the HYPRES model and the “silty clay loam” and “silty clay” was adjusted for our USDA soil classes model. Therefore, the Van Genuchten parameters for the HYPRES series for top soil of “fine” soil class is shown in Table 3, and the lists of the Van Genuchten parameters for the USDA (United States Department of Agriculture) series referring top soil of “silty clay loam” and “silty clay” has been presented in Table 4.

Table 3 Van Genuchten parameters for HYPRES topsoil series [34]

Soil class	k_{sat} (m/day)	g_a (1/m)	g_l (-)	g_n (-)
Topsoil				
coarse	0.600	3.83	1.2500	1.3774
medium	0.121	3.14	-2.3421	1.1804
medium fine	0.023	0.83	-0.5884	1.2539
fine	0.248	3.67	-1.9772	1.1012
very fine	0.150	2.65	2.5000	1.1033

Table 4 Van Genuchten parameters for USDA soil series and $g_1=0.5$ [34]

Soil texture	$\theta_r (-)$	$\theta_s (-)$	$k_{sat} (m/day)$	$g_a (1/m)$	$g_n (-)$
sand	0.045	0.43	7.13	14.5	2.68
loamy sand	0.057	0.41	3.50	12.4	2.28
sandy loam	0.065	0.41	1.06	7.5	1.89
loam	0.078	0.43	0.25	3.6	1.56
silt	0.034	0.46	0.60	1.6	1.37
silty loam	0.067	0.45	0.108	2.0	1.41
sandy clay loam	0.100	0.39	0.314	5.9	1.48
clayey loam	0.095	0.41	0.624	1.9	1.31
silty clayey loam	0.089	0.43	0.168	1.0	1.23
sandy clay	0.100	0.38	0.288	2.7	1.23
silty clay	0.070	0.36	0.00475	0.5	1.09
clay	0.068	0.38	0.0475	0.8	1.09

6.1.7 Soil Models

6.1.7.1 Hardening Soil Model

In contrast to an elastic-perfectly plastic model, the yield surface of a hardening plasticity model is not fixed in principal stress space, but it can expand due to plastic straining [31]. The Hardening Soil model is an advanced model for simulating the behaviour of different types of soil, both soft soils and stiff soils. When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. In the special case of a drained triaxial test, the observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola. The Hardening Soil model, however, supersedes the hyperbolic model by far: Firstly, by using the theory of plasticity rather than the theory of elasticity, secondly by including soil dilatancy and thirdly by introducing a yield cap. Some basic characteristics of the model are:

- Stress dependent stiffness according to a power law: Input parameter m
- Plastic straining due to primary deviatoric loading: Input parameter E_{50}^{ref} (is used for shear strain (triaxial))

- Plastic straining due to primary compression: Input parameter E_{oed}^{ref} (is used for 1D compression)
- Elastic unloading / reloading: Input parameters E_{ur}^{ref} , ν_{ur} (is used for elastic unloading/reloading calculations)
- Failure according to the Mohr-Coulomb failure criterion: Parameters c , ϕ and ϕ

In real soils, stiffness increases with confining pressure. This means the more pressure a soil is under, the stiffer it behaves. The Hardening Soil model (HS model) captures this nonlinear stress-dependent behaviour by scaling the stiffness moduli with current stress and using the option “stress dependency of soil stiffness as the parameter (m)”.

- If $m=0$ that means stiffness is constant (like in linear elastic models)
- If $m>0$ that means stiffness increases with stress

Therefore, this is more realistic than constant-stiffness models like Mohr-Coulomb, especially for modelling settlement under pressure in complex simulations. This is the reason HS model is more accurate settlement predictions (especially under varying loading) and captures nonlinear deformation behaviour.

A basic idea for the formulation of the Hardening Soil model is the hyperbolic relationship between the vertical strain, ε_1 , and the deviatoric stress, q , in primary triaxial loading. In the following, the standard drained triaxial tests tend to yield curves that can be described in the following [31]:

$$-\varepsilon_1 = \frac{1}{E_i} \frac{q}{1 - q/q_a} \quad \text{for : } q < q_f \quad (16)$$

In this formulation q_a is the asymptotic value of the shear strength, and E_i is the initial stiffness that can be calculated as the following:

$$E_i = \frac{2E_{50}}{2 - R_f} \quad (17)$$

E_{50} is the secant stiffness modulus at 50% of the failure stress in a drained triaxial test and R_f is the failure ratio which can be described as the following:

$$R_f = \frac{q_f}{q_a} \quad (18)$$

And q_f is the ultimate deviatoric failure value of the shear strength. In addition, E_{50} can be described in the standard drained triaxial test as the following:

$$E_{50} = E_{50}^{ref} \left(\frac{c \cos(\varphi) - \sigma_3' \sin(\varphi)}{c \cos(\varphi) + p^{ref} \sin(\varphi)} \right)^m \quad (19)$$

Furthermore, the q_f can be calculated as:

$$q_f = (c \cot(\varphi) - \sigma_3') \frac{2 \sin(\varphi)}{1 - \sin(\varphi)} \quad (20)$$

Referring these equations, the hyperbolic stress-strain relation in Hardening Soil model for the standard drained triaxial test can be shown in the Figure 37 [31]. In many practical cases it is appropriate to set E_{ur}^{ref} equal to $3E_{50}^{ref}$ [31], which is the default setting used in our Numerical FEM Plaxis simulation.

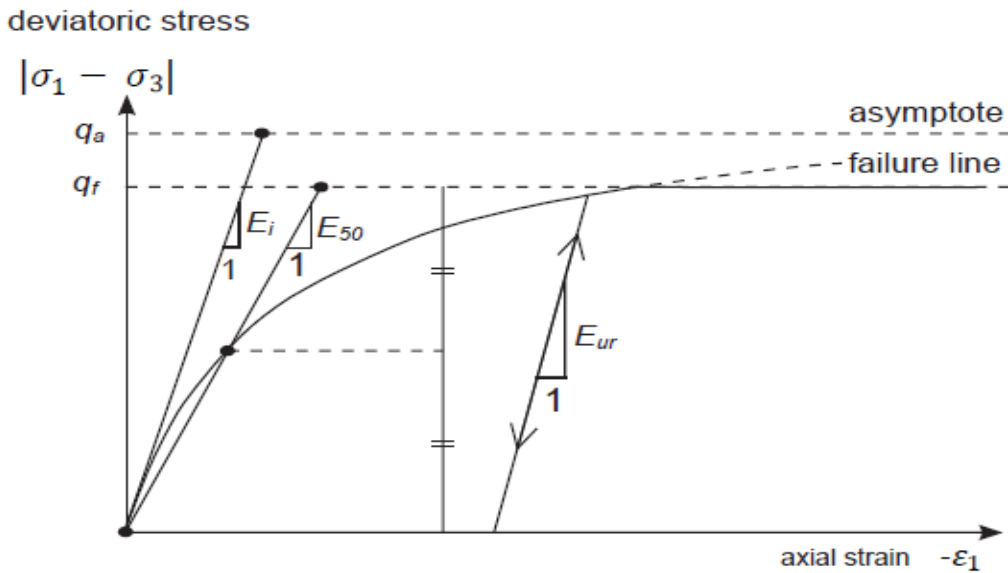


Figure 37 Hyperbolic stress-strain relation in primary loading for the
standard drained triaxial test in HS model

In addition, for 1D compression using the oedometer conditions of stress and strain, which could be used for compression and settlement under the pressure, the HS model implies the relationship:

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cos(\varphi) - \frac{\sigma'_3}{K_0^{nc}} \sin(\varphi)}{c \cos(\varphi) + p^{ref} \sin(\varphi)} \right)^m \quad (21)$$

E_{oed} is the current oedometer modulus at the current stress level and the tangent stiffness modulus obtained from an oedometer test, which can be seen on Figure 38. E_{oed}^{ref} is the reference oedometer modulus (measured at reference pressure), k_0^{nc} can be calculated as the $-\sigma'_1 = \frac{\sigma'_3}{k_0^{nc}} = p^{ref}$, p^{ref} is the reference pressure, and m is the power for stress dependency stiffness (dimensionless parameter).

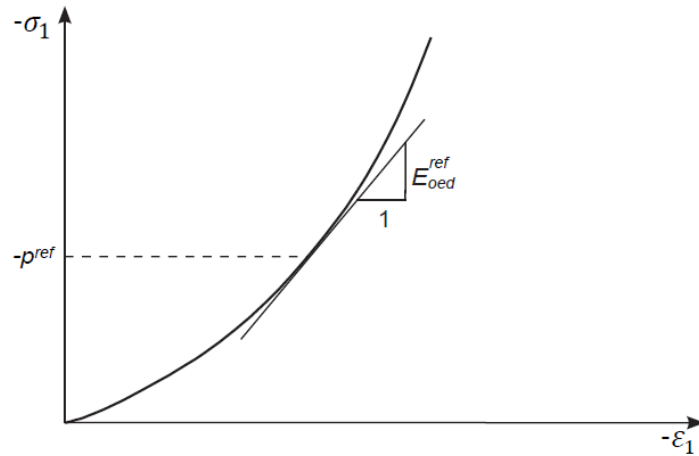


Figure 38 Definition of E_{oed}^{ref} in oedometer test results

And in the following, on Figure 39 representation of total yield contour of the Hardening Soil model in principal stress space for cohesionless soil is shown. This is the 3D yield surface plot in principal stress space for principles stress in HS model and show how the boundary going beyond, as the soil yield plastically under different combinations of stresses. Each point on the surface represents a yield point and is the combination of principal stresses at which plastic deformation begins.

The asymmetric/rounded shape reflects stress dependency of stiffness and strength, and the dome (cone-like expansion) shape indicates non-associated plasticity and the ability to model different stiffnesses in different loading modes (triaxial compression, triaxial extension, oedometer, etc.). It also reflects how the shear yield envelope grows with increasing mean effective stress and if the soil is subjected to higher mean stresses, it can withstand higher deviatoric stresses before yielding as the nonlinear, stress-dependent hardening. These all are the core improvement of HS model over other simpler models like Mohr-Coulomb.

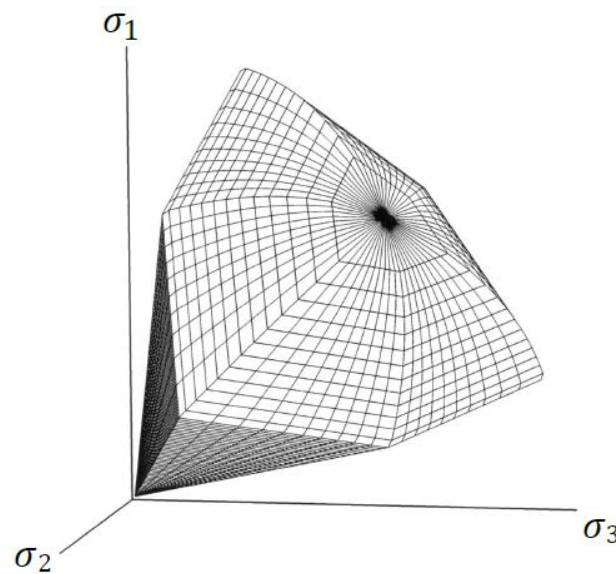


Figure 39 Representation of total yield contour of the Hardening Soil model in principal stress space (cohesionless soil)

6.1.7.2 Stress dependant stiffness according to a power law (m):

As mentioned in previous section, the power “ m ” in geotechnical engineering, refers to the relationship between the soil's stiffness or permeability and the effective stress in the soil and is commonly used in constitutive models and permeability functions to account for how soil properties change under varying stress conditions. m is often determined based on laboratory tests or field measurements that correlate stiffness or permeability with stress and its typical values are around 0.5 for sand soil layers and 0.8 to 1.0 for clay and silt layers. Therefore, as the 1st soil layer is considered as the silt layer, so the 0.8 was assumed for the calculation to provide more accurate

predictions of soil behaviour for stress-sensitive processes like consolidation, seepage, and deformation in the analysis.

6.1.7.3 Hoek-Brown Model

The material behaviour of rock differs from the behaviour of soils in the sense that it is generally stiffer and stronger. The dependency of the stiffness on the stress level is almost negligible, so stiffness of rocks can be considered constant. On the other hand, the dependency of the (shear) strength on the stress level is significant. In this respect, heavily jointed or weathered rock can be regarded a frictional material. A first approach is to model the shear strength of rock by means of the Mohr-Coulomb failure criterion. However, considering the large range of stress levels where rock may be subjected to, a linear stress-dependency, as obtained from the Mohr-Coulomb model, is generally not sufficient. Furthermore, rock may also show a significant tensile strength. The “Hoek-Brown” failure criterion is a better non-linear approximation of the strength of rocks which was developed by Evert Hoek and Edwin Brown. It involves shear strength as well as tensile strength in a continuous formulation. Together with Hooke's law of isotropic linear elastic behaviour it forms the “Hoek-Brown” model for rock behaviour. The generalised “Hoek-Brown” failure criterion can be formulated as a non-linear relationship between the major and minor effective principal stresses [31]:

$$\sigma_1' = \sigma_3' + \sigma_{ci} \left[m_b \frac{\sigma_3'}{\sigma_{ci}} + s \right]^a \quad (22)$$

Where σ_{ci} is the uniaxial compressive strength of the intact rock material. m_b is a reduced value of the intact rock parameter m_i , which also depends on the Geological Strength Index (GSI) and the Disturbance Factor (D):

$$m_b = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) \quad (23)$$

s and a are auxiliary material constants for the rock mass, that can be expressed as:

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \quad (24)$$

$$\alpha = \frac{1}{2} + \frac{1}{6} \left[\exp\left(\frac{-GSI}{15}\right) - \exp\left(\frac{-20}{3}\right) \right] \quad (25)$$

The Hoek-Brown failure criterion is illustrated in Figure 40. All of the described parameters will be evaluated for the simulation in our Numerical FEM Plaxis software in θ .

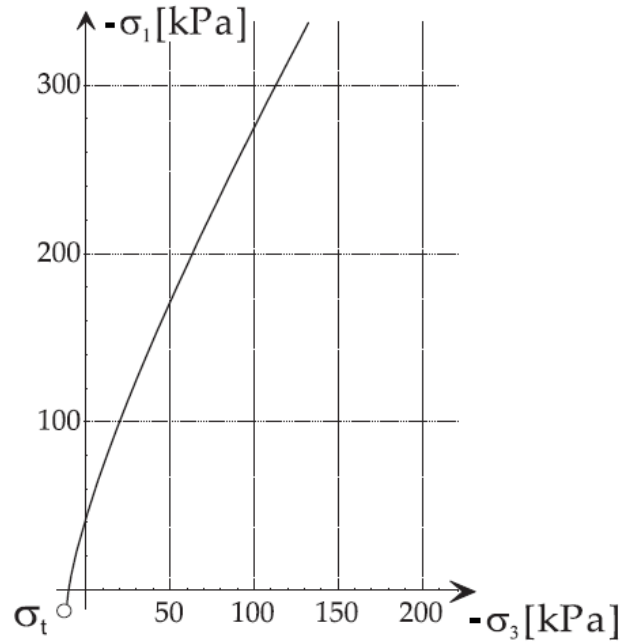


Figure 40 Hoek-Brown failure criterion in principal stresses

The Hoek–Brown model is particularly suited for rock slope stability, tunnel and cavern design, excavation stability, deep foundation and abutment simulations and it is used to simulate the mechanical behaviour of intact rock and rock masses, considering their degree of fracturing, weathering, and excavation disturbance. Its features are that it is a non-linear failure envelope (unlike the straight line of Mohr–Coulomb), considers both intact rock properties and rock mass quality, and allows adjustment for disturbance due to blasting or excavation.

6.1.8 Pore Water Pressure

Numerical FEM Plaxis 2D is generally used for effective stress analysis in which total stresses are divided into effective stresses, σ' , and active pore pressures, p_{active} :

$$\sigma = \sigma' + P_{active} \quad (26)$$

Active pore pressure (P_{active}) is defined as the effective saturation (S_{eff}), times the pore water pressure (p_{water}):

$$P_{active} = S_{eff} \cdot P_{water} \quad (27)$$

$$S_{eff} = \frac{S_r - S_{res}}{S_{sat} - S_{res}} \quad (28)$$

As described in previous section, S_r is the “degree of saturation” and S_{sat} is the “saturation” at full water content in soil (100 %). Another parameter in the mentioned formula is the “residual saturation” (S_{res}), which describe the minimum water content the soil retains even when "dry" and soils always retain a small amount of water even when they are considered "dry". This shows that Plaxis FEM software deal with saturated soil below the phreatic level, as well as with partially saturated soil above the phreatic level, which is very important in our project.

In pore water pressure P_{water} further division is made between “steady state pore pressure” (p_{steady}) and “excess pore pressure” (p_{excess}):

$$P_{water} = P_{steady} + P_{excess} \quad (29)$$

P_{steady} is the steady-state pore pressure (hydrostatic pressure) depending on groundwater as the initial condition. P_{excess} (excess pore pressures) are pore pressures that occur as a result of stress changes in undrained materials (in this project “undrained A”). This means that “excess pore pressure” is the extra water pressure inside the soil pores above the normal and equilibrium pore water pressure.

In this project, changes in stress were a result of change in hydraulic conditions. Hence, excess pore pressures were the result of a deformation analysis. In our “fully coupled flow-deformation” analysis, excess pore pressures occurred soil layer (except non-porous bedrock), by considering the defined permeability. In this case excess pore pressure was calculated by subtracting the steady-state pore pressure (p_{steady}) from the pore water pressure (p_{water}):

$$P_{excess} = P_{water} - P_{steady} \quad (30)$$

6.1.9 Safety Calculation (Phi/C Reduction)

Griffiths & Lane (1999) showed that the finite element method represents an accurate, adaptable and powerful alternative method for slope stability analysis, as it requires less a priori assumptions, especially concerning possible failure mechanisms. In the finite element method, the development of the failure mechanism takes place “naturally” as an outcome of the analysis.

In the Finite Element Method, simple self-weight calculations, and the influence on the factor of safety of single construction stages of more complex slopes, or of the installation of particular support measures can be quantified at any step of the calculation. There is no need to make assumptions about the shape or location of the failure surface when performing safety calculations using the finite element method, although in the Limited Equilibrium Method it is needed to define the shape and location of around failure in the simulation. In FEM Method failure is found automatically as slope failure occurs naturally through the zones due to insufficient shear strength to resist shear stresses, not where it is assumed, which can be considered a big advantage when compared to limit equilibrium methods [5].

In Plaxis FEM Code software, the safety approach the shear strength parameters $\tan \phi$ and c of the soil as well as the tensile strength are successively reduced until failure of the structure occurs. The dilatancy angle ψ is, in principle, not affected by the phi/c reduction procedure. However, the dilatancy angle can never be larger than the friction angle. When the friction angle ϕ has reduced so much that it becomes equal to the (given) dilatancy angle, any further reduction of the friction angle will lead to the same reduction of the dilatancy angle. The total multiplier ΣM_{sf} is used to define the value of the soil strength parameters at a given stage in the Plaxis FEM Code analysis [34]:

$$\Sigma M_{sf} = \frac{\tan \varphi_{input}}{\tan \varphi_{reduced}} = \frac{c_{input}}{c_{reduced}} = \frac{s_{u,input}}{s_{u,reduced}} = \frac{\text{Tensile strength}_{input}}{\text{Tensile strength}_{reduced}} \quad (31)$$

$$SF = \frac{\text{available strength}}{\text{strength at failure}} = \text{value of } \Sigma M_{sf} \text{ at failure} \quad (32)$$

Where the strength parameters with the subscript 'input' refer to the properties entered in the material sets and parameters with the subscript 'reduced' refer to the reduced values used in the analysis. ΣM_{sf} is set to 1.0 at the start of a calculation to set all material strengths to their input values. The strength parameters are successively reduced automatically until all additional steps have been performed and when the final step has resulted in a fully developed failure mechanism the Factor of Safety is given by ΣM_{sf} in our FEM Code software calculations, which will be used in this project for the study case of Ponzano region.

6.2 Creating Simulation in Plaxis FEM Code 2D Software

6.2.1 Materials Definitions

6.2.1.1 Soil Layer

In the 3rd phase we want to focus on the Section A-A (Figure 4) and perform a deep calculation on the failure mechanism that happened in this section in 2D and as there are some laboratory data from this section achieved from the “*Department of Geotechnologies for the Environment and the Territory*” (**Section 3.2**) and some data are missed and needed to be determined in the Numerical FEM simulation, so Back-Analysis is needed in details to apply the calculation to be comparable with the reality. In this regard, the Sensitivity Analysis in details for missed data is needed to perform the Numerical FEM calculations to be the same that happened in the region in Section A-A. As mentioned in **Section 3.2** the laboratory data showed with details, but for more convenience in reading the dissertation we attached the performed laboratory data again as Table 5. Therefore, these data are adjusted for the soil layer specification in the Numerical FEM 2D simulation of Plaxis.

Table 5 Laboratory data achieved from “*Department of Geotechnologies for the Environment and the Territory*” referring the 1st soil layer as an average of the tested soil specimens

Description	Parameters	Values	Unit
Unsaturated Unit Weight of Soil	γ_{unsat}	16.0	$\frac{\text{kN}}{\text{m}^3}$
Saturated Unit Weight of Soil	γ_{sat}	20.0	$\frac{\text{kN}}{\text{m}^3}$
Liquid Limit	W_L	52	-
Plastic Limit	W_P	18	-
Index of Plasticity	I_P	34	-
Initial Void Ratio	e_{init}	0.8	-
Over Consolidation Pressure	OCR	1.5	-
Secant Reference Modulus	E_{50}^{ref}	10000 – 20000	kPa
Reference Pressure	P_{ref}	640	kPa
c'	c'	20	kPa
ϕ'	ϕ'	23	Degree
Poisson ratio	Poisson ratio	0.33	-
ASTM D2487	Soil Classification	mostly CH (fat clay) and in some cases CL (lean clay)	-

6.2.1.2 Bedrock Layer

The bedrock material in the Ponzano (Civitella del Tronto) area is described as Miocene–Pliocene pelitic-arenaceous bedrock sequence and composed of interbedded pelitic (**clay-lime**) and arenaceous (**sandstone**) layers often referred to as the Laga Formation flysch (**sandstone with marl/claystone** interbeds), which implies **weak to moderately strong cemented sandstone rock** [2].

6.2.1.2.1 MR (Modulus Ratio)

To simulate the bedrock in the simulation, Hoek-Brown model participants are needed to be determined. Given that the Ponzano region has sandstone as a major component, the best representative choice for the MR (Modulus Ratio) ratio of the bedrock is 275 [31, 32].

6.2.1.2.2 Uni-axial compressive strength of intact rock (σ_{ci})

The uniaxial compressive strength (σ_{ci}) of intact rock can be determined as the most appropriate match for Laga Formation flysch (sandstone with marl/claystone interbeds) is likely to be 37.5 MPa [31].

6.2.1.2.3 Intact rock parameter (m_i)

The intact rock parameter (m_i) is an empirical model parameter, which reflects the brittleness or frictional strength of the intact rock, and it depends on mineralogy, texture, and type of rock. Since the Ponzano bedrock is the mix of weakly cemented sandstone and marl (Flysch), a reasonable value could be 17 for this parameter in the simulation [31].

6.2.1.2.4 Geological Strength Index (GSI)

GSI can be chosen taking the structure and the surface conditions of the rock mass into account. Referring the flysch with poor surface conditions and laminated structure it could be GSI=40-50. Therefore, as an average we used the GSI=45 for the calculation in the bedrock [31].

6.2.1.2.5 Disturbance Factor (D)

The Disturbance factor (D) is a parameter that depends on the amount of disturbance of the rock as the result of mechanical processes in open excavations, tunnels or mines, such as blasting, tunnel boring, machine driven or manual excavation. The Laga Formation is described as intensely fractured, folded, and affected by landslide activity. The area has undergone progressive deformation due to hydrological triggers, and clear evidence of weathering [3]. These conditions indicate that the rock mass is highly disturbed and structurally weakened, especially in the landslide-affected zone. Therefore, it is reasonable to assume D between 0.8 and 1.0, but for conservative considering in our simulation, we considered it as 1.0 in the calculations.

6.2.1.2.6 Rock Mass Young's Modulus (E_{rm})

In principle, Young's modulus can be measured from axial compression tests or direct shear tests on rock samples. However, this modulus is more applicable to the intact rock material and should be reduced to obtain a representative stiffness of the in-situ rock mass. The rock mass Young's modulus E_{rm} is assumed to be a constant value for the considered rock layer. The rock mass stiffness E_{rm} can be estimated through the Hoek & Diederichs (2006).

$$E_{rm} = E_i \left(0.02 + \frac{1 - D/2}{1 + e^{(60 + 15D - GSI)/11}} \right) \quad (33)$$

$$E_i = MR \times \sigma_{ci} \quad (34)$$

As all of the parameters has been described in previous sections, so referring the described parameters the Rock Mass Young Modulus is determined as 522756 kN/m².

6.2.2 Geometry

The geometry was adopted from Figure 4 to create the simulation with the focus of the mechanism of failure of the land sliding that happened in this region in 2D dimensions in Numerical FEM Plaxis software. This profile was considered as the base for defining the geometry in the following of the Sensitivity Analysis (Figure 41). As described in **Section 3.2**, we do not have underground water flow in subsoil layers and the only infiltration in subsoil layer is related to the precipitation. Therefore, to save time in calculations and increase the speed of calculations in Numerical FEM software we created two different boundary line firstly in our 1st soil layer and secondly in our bedrock layer to define finer mesh distribution size with optimum (smaller) width in the summation and save time in our Numerical FEM calculations. This also happened in the bedrock layer and the part of the bedrock layer that is more near the soil layer has been divided with more mesh distribution size to save time of calculation in our Numerical analysis. This prevent of create very fine mesh for all part of the bedrock and soil layer and let us to choose the most important part of our simulation for more fine mesh distribution size in our Numerical calculations. The definitions for the mesh size distribution by considering the boundary lines are described in the following section.

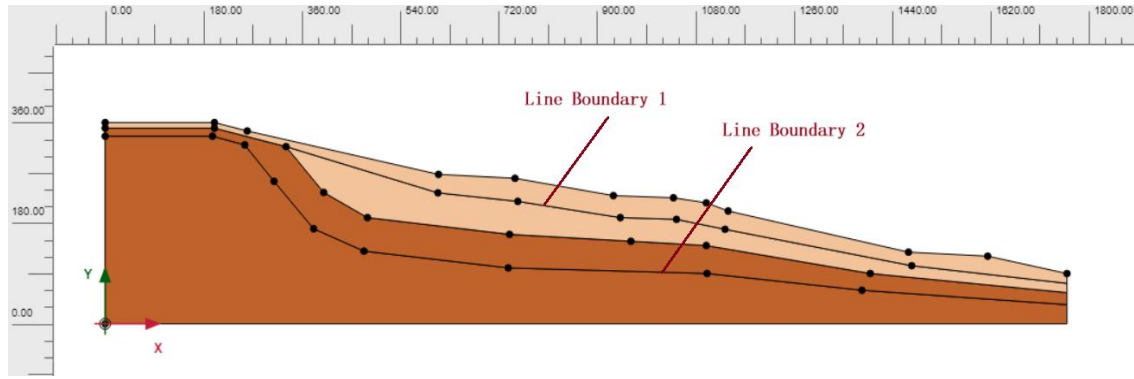


Figure 41 Creating geometry of the simulation referring the reference [2]

6.2.3 Mesh

Since the calculation were performed in the 2D dimensions with around 1.8 km to more focus on the project and have a better understanding of the mechanism of failure in the area, so we created very fine mesh dimensions to increase the accuracy and have a better understanding of the mechanism of failure (Figure 42). As described in previous section, the coarseness factor that is defined for different soil layers has been defined as depicted in Figure 42.

It can be seen that by creating the boundary line 1 (Figure 41), the coarseness factor for the top of section was defined smaller and equal to 0.07 and for the down section was determined 0.075. In addition, by creating the boundary line 2, the coarseness factor for its top section defined smaller and equal to 0.1 and for the down section specified 0.3. This helped us to increase the speed of our calculation much more during FEM analysing and more focus on sections that are more determinative and critical in our final FEM analysis.

Coarseness Factor in PLAXIS FEM affects the mesh quality and integration accuracy during the finite element calculations and particularly during plasticity and stiffness matrix integration over each element and scaling the parameter used to control the accuracy of numerical integration over elements, especially for nonlinear material models, which is used in this Numerical simulation. This factor influences the “stress point integration across elements”, the “smoothness of stress distribution”, the “fidelity of strain localization” and the “plastic zone development during loading”. The Coarseness Factor determines how many of integration points are used or how detailed the calculation mesh is treated during analysis and this value less than 1.0, it creates finer integration and more accurate with higher precision, although the calculation time will be slower.

The detailed information about the general condition of the mesh distribution and mesh sizes and also the mesh dimensions for the special 1st soil layer which is determinative for our calculation has been attached in the Figure 43.

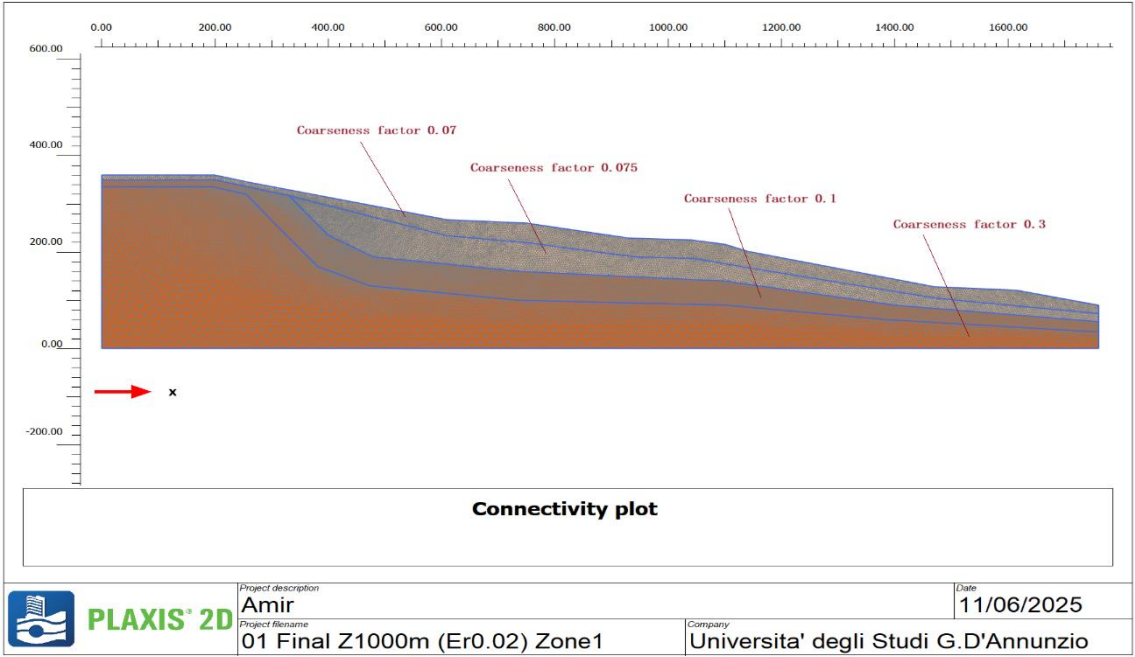


Figure 42 Definition of the mesh size distribution in FEM Plaxis Code software for different sections

General		General		Original
Model		Node 144565		
Plane strain		X		937.738 m
Elements		Y		226.789 m
15-Noded		Node 144575		
Acceleration		X		939.888 m
Gravity angle		Y		228.949 m
-90.00 °		Δx		2.150 m
x-acceleration		Δy		2.161 m
0.000 m/s ²		Distance		3.048 m
y-acceleration		Orientation		45.136 °
0.000 m/s ²				
Earth gravity				
9.810 m/s ²				
Mesh				
Nr of soil elements				
35089				
Nr of nodes				
282143				
Average element size				
4.634 m				
Maximum element size				
22.89 m				
Minimum element size				
0.9802 m				

Figure 43 General information referring mesh size distribution in total simulation (left)

and mesh size distribution specifically for 1st layer (right)

6.2.4 Flow Condition

As described before there is no underground water flow in the simulation. In the flow condition, we defined the left boundary conditions “closed” and the right boundary conditions defined as “open”, but this procedure was changed in the Sensitivity Analysis as this definition was not so much clear in the region, so this was one of the parameters that was going to be changed during the Sensitivity Analysis (Figure 44).

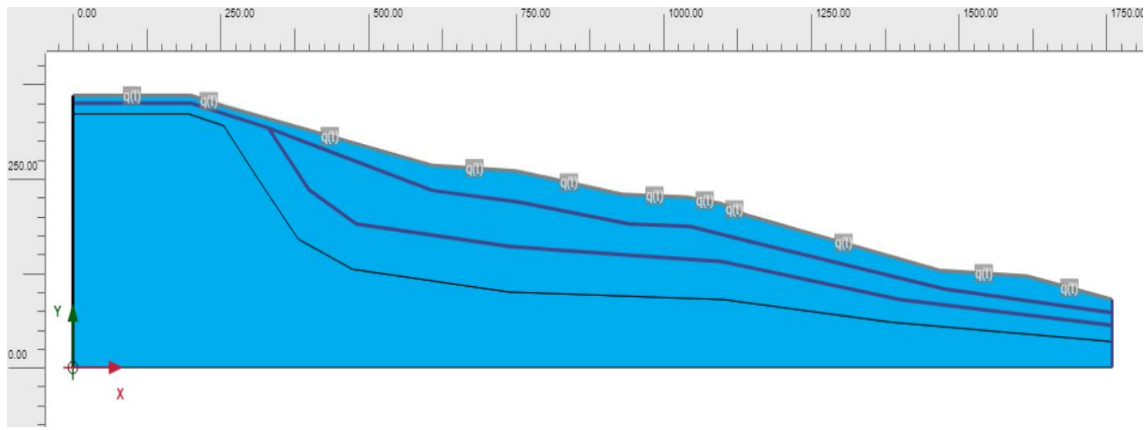


Figure 44 Definition of the flow condition in the simulation

6.2.5 Phase Definitions

In PLAXIS FEM software, a “phase” is a core concept used in the simulation process to define the distinct step or stage in the construction sequence, loading, or boundary condition changes in our Numerical FEM simulation. In PLAXIS FEM phase refers to the calculation stage whereas:

- Geometry maybe changed during the excavation, embankment and in our case referring the land sliding that is going to be happened.
- Loads and the boundary conditions during the project
- Material properties and the behaviour models will be considered
- Time progresses, which is important in our project to consider the infiltration of the precipitation

In this process, each phase builds upon the results of the previous phase, which is essential for modelling in Numerical FEM simulation.

6.2.5.1 Initial Phase

In Plaxis FEM software, the Initial Phase is a foundational step in our project, whereas it represents the in-situ condition of the soil mass before any construction, loading, or other interventions begin in the study case area and it's the baseline from which all other calculation phases evolve in the following and everything else will be done in the phases that follow. In the Initial Phase we performed the following procedure:

- We established the initial stress state of the soil in the project
- We set up the initial pore water pressures in soil layer
- Gravity loading was applied in this project to settle the soil under its own weight
- Initial geometry and the boundary conditions were considered
- For the “pore pressure calculation type”, the “steady state groundwater flow” was considered to determine the hydraulic conditions by using the permeability in the definitions

6.2.5.2 Full Coupled Flow-Deformation Phase

In the following phases, we considered the “full coupled flow-deformation” phase to perform the influence of infiltration of the precipitation data in soil layer and on the deformation that is going to be happened in the region. This phase simultaneously solves the mechanical deformation and pore water flow in unsaturated also as was defined in our project. This phase solves the equilibrium of stresses and deformations, and also the transient flow equations in unsaturated soil layer, which is related to the infiltration of precipitation triggering the deformation and failure in the simulation, simultaneously. Therefore, this phase is vital for our accurate time-dependent simulation.

In the “numerical control parameters” we defined the “max steps” on 10000, to be sure the calculations will be converged. Referring the “time interval” we defined the number of precipitation days by considering the history of the precipitation of our data in the Sensitivity Analysis.

6.2.5.3 Safety Phase

After calculation of the flow and deformation simultaneously, to achieve our goal for the failure mechanism during the land sliding that happened in the region, we defined the “safety factor”, which is called also as Phi-c Reduction Phase and is specialized to evaluate the stability of the sloped simulation and more realistic and robust for complex failure mechanisms. It helps determine the Factor of Safety (FOS) against failure using a numerical method rather than traditional limit equilibrium methods and shows if slope is stable under self-weight or surcharge.

In this phase, we considered 250 for the “max steps” which create more accurate and stable safety factor in the final analysis. In addition, we did not consider undrained behaviour A in the safety analysis, as this could decrease the safety factor in our Numerical simulation, which would create the worst condition in the simulation.

6.3 Sensitivity Analysis

As described before, more data are needed for our Numerical FEM calculation in the Section A-A to create the simulation to have the same failure mechanics that happened, and this led us in our back-Analysis to go toward the Sensitivity Analysis. As mentioned, the laboratory data that are tested are described in Table 5, so the other necessary data that the Sensitivity Analysis should be performed for, and their description will be discussed more in the following:

- Effect of Drainage Type: drained/undrained A
- Effect of Run-off Water (pore pressure head limits)
- Effect of Boundary Conditions
- Effect of Considering Suctions in Safety Factor calculations Phase
- Effect of Evaporation
- Effect of History of Precipitation
- Effect of Permeability Coefficients
- Effect of Unsaturated Zone
- Effect of SWCC behaviour, which is the one of the most critical parameters in this project
- Effect of Van Genuchten equation fitting parameters, which is the one of the most critical parameters in this project

In this section, for each Sensitivity Analysis, one parameter is changed, and the other parameters are going to be fixed and this procedure will be performed for all of the Sensitivity Parameters that are described above to achieve the best results in the final Numerical FEM calculations in Plaxis Code software and in the final by considering the worst results from each parameter, the last simulation by performing all of the worst parameters will be calculated to create the same conditions that happened in Ponzano and create the land sliding failure in the results and achieving the minimum safety factor in the FEM code analysis.

As mentioned before, the mesh size distribution was each run was considered with the coarseness factor for the main layer 0.07, which is a very fine mesh around 3.0 m for the dimension of the elements in a 1.8 km dimension model. Thus, each run and analysis during the Sensitivity Analysis *was taken several days just for one run and analysis*, which led our Numerical FEM Plaxis code analysis exhausting but worthwhile in this dissertation.

6.3.1 Effect of “Drainage Type”

The drainage type determines the generation of excess pore pressures in our Safety calculation type. In this section, the drained material behaviour of stiffness and strength are defined in terms of effective values, but for the “drained” behaviour no excess pore pressures are generated whereas in the “undrained A” behaviour a large bulk stiffness for water is automatically applied to make the soil as a whole incompressible, and excess pore pressures are calculated, even above the phreatic surface.

Therefore, in this section by changing the “drainage type” from “undrained A” to “drained” conditions, the safety factor increased from 2.09 to 2.11 (Figure 45). Therefore, by considering the Sensitivity Analysis referring the **“drainage type” of undrained A** condition, it showed the worst and lowest S.F in our Numerical FEM evaluation and it was considered for our final FEM analysis.

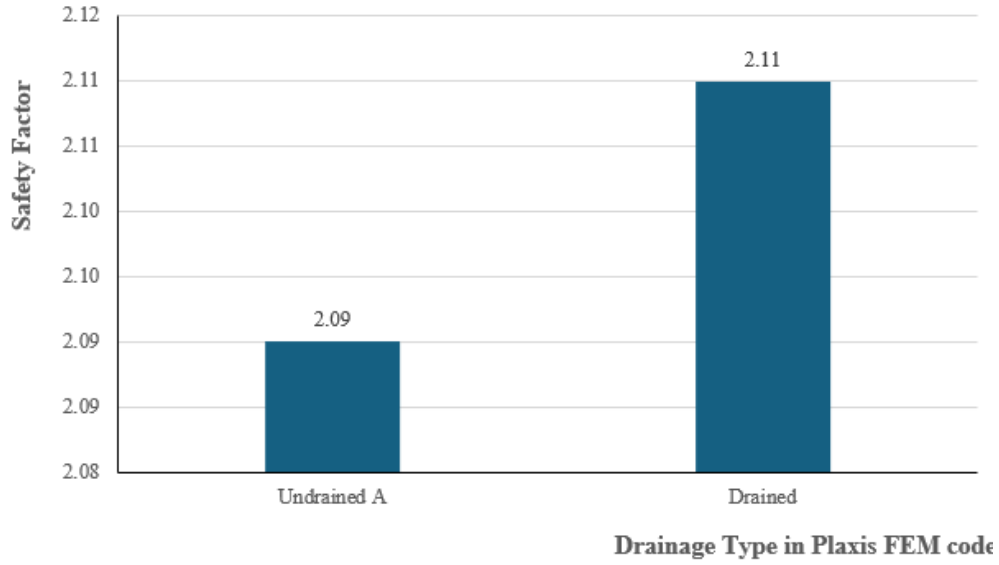


Figure 45 Effect of drainage type in Plaxis FEM analysis

6.3.2 Effect of Run-Off Water (Pore Pressure Head Limits)

In Plaxis FEM code software infiltration conditions could be specified manually for geometry boundaries as a conditional inflow. This means by defining the “maximum pore pressure head” (ψ_{max}), relative to the elevation of the boundary, if the groundwater head from the precipitation increases above this level, the infiltration discharge changes into the corresponding head to simulate run off and the water is supposed as the run-off and a constant head boundary condition equal to $y + \psi_{max}$ is applied like a surcharge on the soil layer instead.

In this regard, by changing the run of water (ψ_{max}) from 0.1 to 0.001 m above the ground surface, the safety factor decreased from 2.12 to 2.09 respectively (Figure 46). Therefore the **“run of water (ψ_{max})” with 0.001 m** condition showed the worst and lowest S.F in our FEM code analysis and considered for the final analysis.

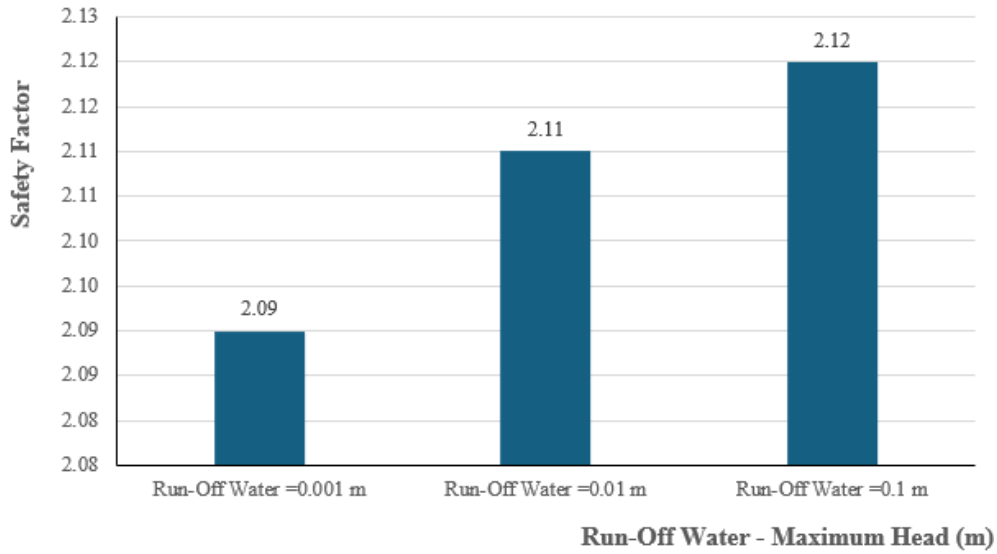


Figure 46 Effect of “maximum pore pressure head” (ψ_{max}) on safety factor analysis in FEM model

6.3.3 Effect of Boundary Conditions

In this Sensitivity Analysis we considered the boundary conditions “open” and “closed” for both of the left and right boundaries of the simulation. Of course, in the “open” condition we let the water to be discharged from the simulation or inflow into the simulation, and by considering the “closed” boundary condition, there is no outflow or inflow in our FEM simulation. In this regard, by changing the condition of the boundaries, the results are depicted in Figure 47. In this figure the symbol “C” is related to the “close” condition and the symbol “O” is related to the “open” BC of our FEM simulation.

The results showed that by fixing the right BC and changing the BC on the left side, the results did not significantly change, but by fixing the left BC and changing the right BC the safety factor changed significantly. Figure 47 showed that considering the “close” condition on the right side of the FEM simulation, increased the safety factor from around 2.09 to 2.16. This showed that by blocking the flow from the right side of BC, the safety factor increased, so for our final FEM simulation we considered the **“open” BC of the right side** of the model to apply the worst condition.

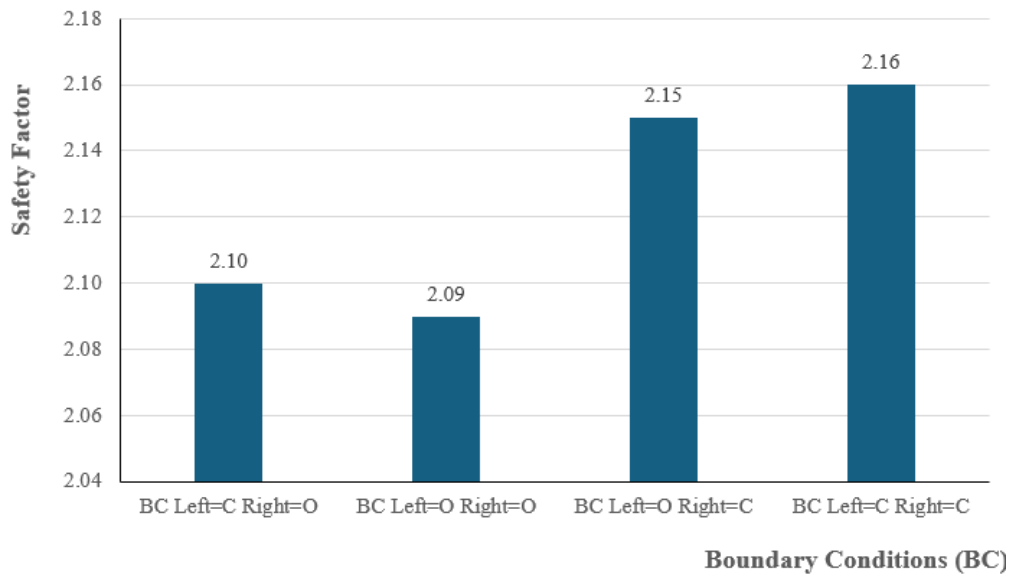


Figure 47 Changing safety factor in FEM simulation by considering boundary conditions (BC)

6.3.4 Effect of Suctions in Safety Factor Calculations Phase

In Plaxis FEM code software, it is possible to choose the option “ignore suction” in safety factor analysis, which could result in neglecting the suction effect in our constitutive model, and soil behaves like a saturated soil. In this case, excess pore pressure is only governed by the compressibility of water in a similar way as saturated soil and is generally much higher than the case of considering suction. This might probably result in a stiffer undrained volume change behaviour, but a lower shear strength due to a higher pore pressure build-up. In this regard, to consider the Sensitivity Analysis we select/deselect the option “ignore suction” in our safety factor phase analysis. The Sensitivity Analysis results showed that by selecting the “ignore suction” in the safety phase, the S.F decreased significantly from 3.32 to 2.09 in our FEM analysis (Figure 48).

Therefore, in our final FEM simulation we will **select the “ignore suction” option in the safety factor phase** analysis, to perform the worst conditions for the region.

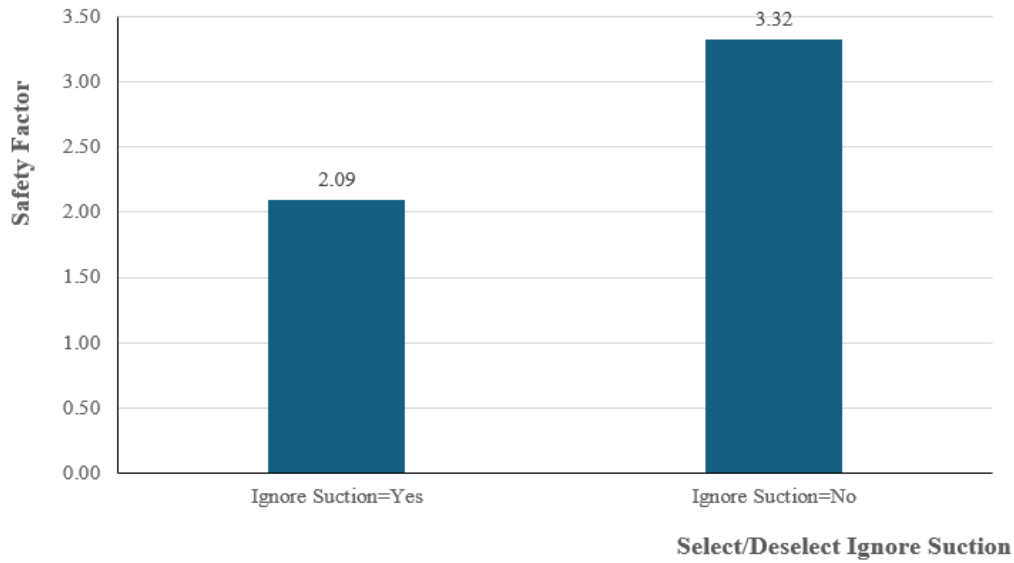


Figure 48 Select/deselect “ignore suction” option in safety phase

6.3.5 Effect of Evaporation

Since evaporation and evapotranspiration have not been measured on site where the soil material and general hydrological information comes from, the evapotranspiration part was assumed for the “Temperate Climate Zones”, as per the hydrological standard for this region [2, 35]. Therefore, referring the precipitation data in Figure 5, the evaporation also calculated for the Ponzano locality of Civitella del Tronto in the Teramo province of Italy, which is located in the “Temperate Climate Zones”, and the evapotranspiration for these regions are typically accounts for about 40% to 60% of the total annual precipitation. Then after that, for evaluating the precipitation data, also by considering 40% and 60% of the total annual precipitation the precipitation data was evaluated and subsequently applied to those dates that had no precipitation data recorded as the “negative precipitation”. Consequently, it was evaluated besides considering also no evaporation (pure precipitation) to perform the Sensitivity Analysis for this parameter in our Numerical FEM simulation.

As described above, we evaluated the effect of considering 40 % and 60 % of evaporation on duration which was divided into the days without precipitation data recorded, and the results showed that the safety factor was not affected significantly, and it mostly changed between 2.09 to 2.10. Therefore the **“no evaporation of the history of precipitation”** was considered for the final analysis.

6.3.6 Effect of History of Precipitation Data

In addition, the last 227-days and 531-days precipitation before the land sliding happened on 12th February 2017, was evaluated as well as the 43-days to determine the sensitivity analysis for the impact of the precipitation history on the safety analysis in the Ponzano region. In this regard, the results showed by increasing the history of precipitation data from 43-days to 531-days, the safety factor was not changed significantly, and it was between 2.09 and 2.10.

Therefore the **“history of precipitation” with 531 days before landsliding day (the big history of precipitation data)** condition was considered for the final analysis.

6.3.7 Effect of Permeability Ratio

From the Ponzano region it was referenced that the saturated permeability is around 10^{-6} m/s, but this was just an assumption and there were no laboratory data for that, so to perform the Sensitivity Analysis we considered the saturated permeability ratio between 0.01 and 1 m/days in our Numerical FEM calculations. The results showed that by changing the saturated permeability ratio from 0.01 m/day to 1 m/day, the safety factor slightly did not change significantly, and it was between 2.09 and 2.10 in our FEM calculations.

Therefore the **“saturated permeability ratio” with 1 m/day** was considered for the final analysis.

6.3.8 Effect of Unsaturated Zone (ψ_{unsat})

ψ_{unsat} is in the unit of length relative to the phreatic level and it sets the maximum pressure head until which is used for calculation of relative permeability and degree of saturation in FEM calculations. Above the level of ψ_{unsat} , the value of K_r and S remains constant, and, in this way, a minimum degree of saturation (S_{min}) is guaranteed in the FEM simulation. It is used to limit the relative permeability K_r and degree of saturation for high unsaturated zones. By changing the unsaturated zone from 1 to 40 m, the safety factor increases from 2.09 to 2.16 (Figure 49).

We can keep the “unsaturated zone” on 1.0 m to create the worst state of the region, but because this is very theoretically to keep a minimum in “degree of saturation” is all of soil layer, and as we want to understand what happened specifically during snow period that land sliding happened after the snow period in the region, we considered the **“Unsaturated Zone (ψ_{unsat})” with 1000 m** condition in our FEM calculations and was considered for the final analysis to create unlimited area for areas to change the saturation degree dynamically during precipitation history.

This would help us to see and control the variations of “degree of saturation” and specifically “suction pore water pressure”, during the intensive values of infiltration after melting snow period and lead us to receive a more realistic simulation in our Back-Analysis comparable with the reality. This section just showed how much the “unsaturated zone” could be effective and influential on safety factor analysis in our FEM analysis.

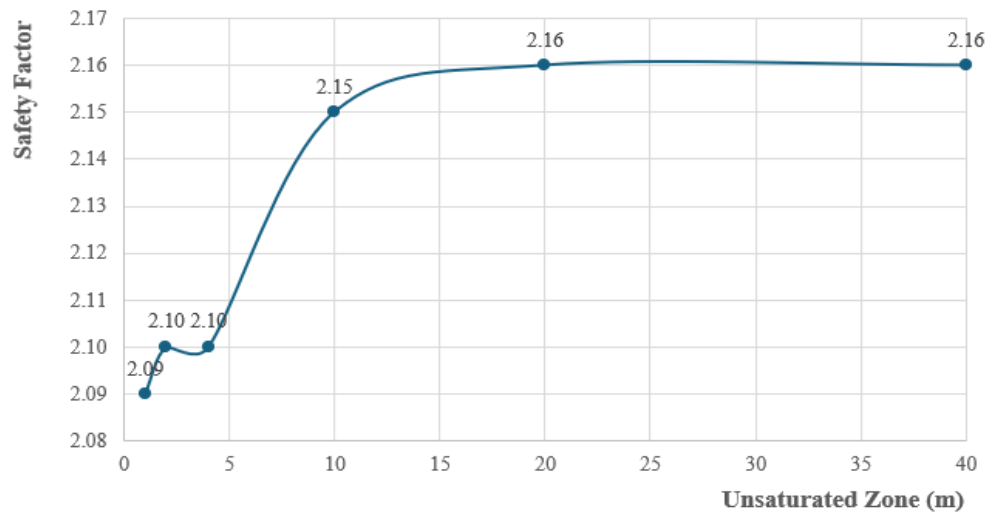


Figure 49 Effect of unsaturated zone (ψ_{unsat}) on safety analysis in Plaxis FEM code

6.3.9 Effect of SWCC Behaviour

Referring the **Section 6.1.6** the SWCC behaviour is the most important and critical parameter in our Numerical EFM code simulation. As mentioned in **Section 6.1.6** we used the two well-known dataset *HYPRES* (*HYdraulic PProperties of European Soils*) series and *USDA* (*United States Department of Agriculture*) in our simulation. As mentioned before, referring the laboratory tests for the grain size distributions in different depths (Table 1), the “fine” soil class was considered

for the HYPRES model and the “silty clay loam” and “silty clay” was performed for our USDA soil classes model in our Sensitivity Analysis in the Numerical Plaxis FEM code simulation.

Figure 50 shows the effect of SWCC behaviour on safety factor analysis in our Numerical FEM code calculation and the results showed that the **well-known dataset HYPRES** represented the lowest safety factor and the worst condition, which could be used in our final simulation.

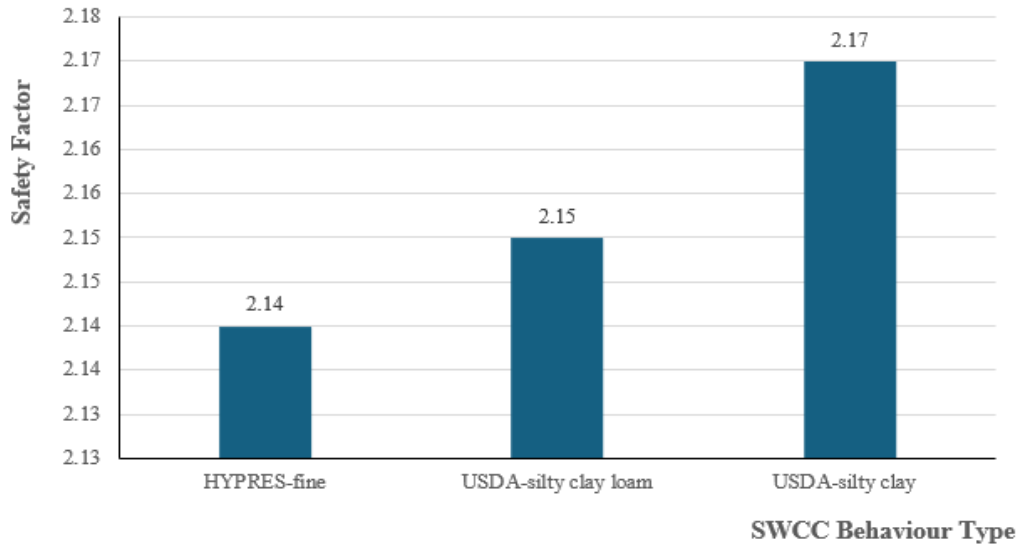


Figure 50 Effect of SWCC behaviour type on safety factor analysis

6.3.10 Effect of Van Genuchten Equation Fitting Parameters

This section we also focused on the influence of each van Genuchten fitting parameter, which was described theoretically in ***Section 6.1.6.3***, on the shape of selected SWCCs **out of the HYPRES databases** described in the previous section. Numerical **Plaxis FEM code software with the version “ULTIMATE”** has this ability for Numerical calculation to do the Sensitivity Analysis by considering the values of parameters out of the Van Genuchten Equation Fitting Parameters, which is its advanced ability in Numerical FEM simulation.

Therefore, this section shows the influence of the g_a factor, which is related to the air entry value, on the safety factor analysis. In the following, the curves relating the out of SWCC HYPRES dataset has been represented on Figure 51.

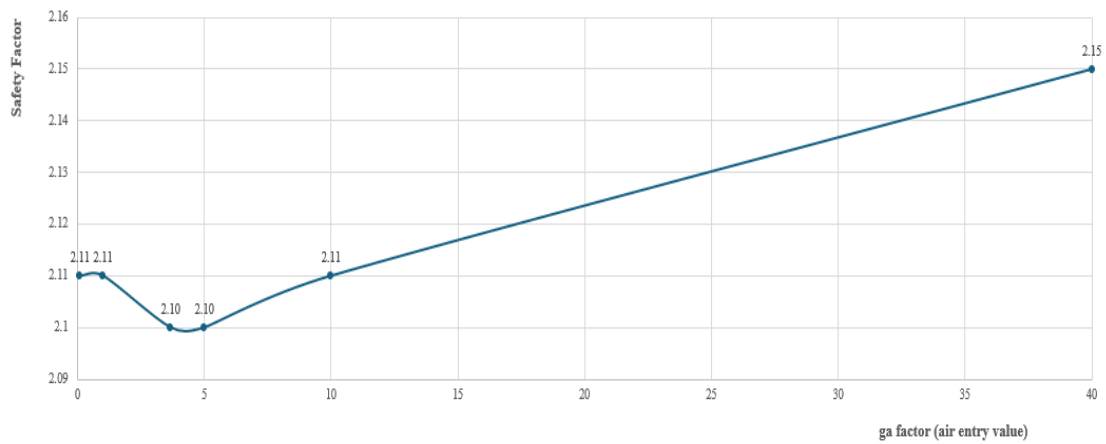


Figure 51 Influence of the g_a factor on “safety factor” analysis for HPRES model

Figure 52 shows the influence of the g_n factor, which is a function of the water extraction rate of the soil once the air entry value has been exceeded, on the shape of HYPRES’ “fine” SWCC. The $g_n = 1.1012$ for HYPRES’ “fine” SWCC is always indicated with a thicker line.

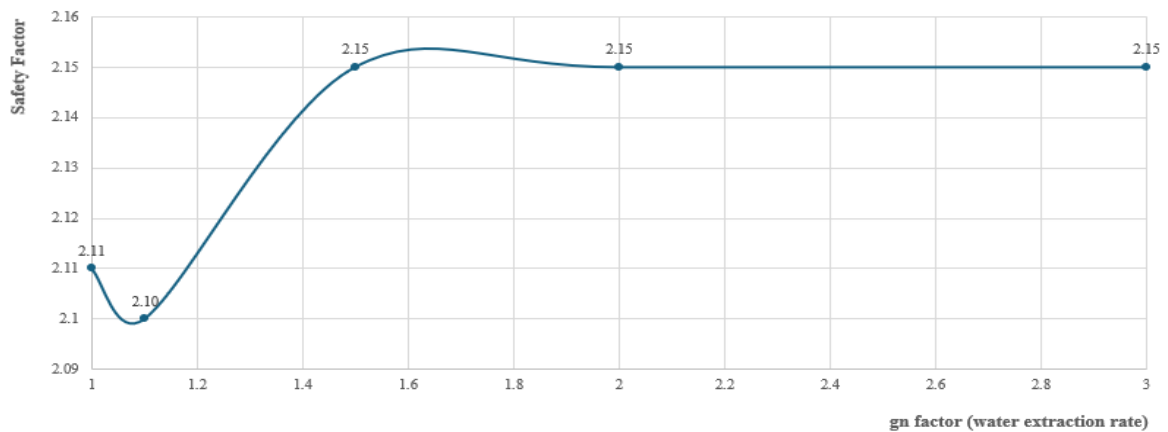


Figure 52 Influence of the g_n factor on “safety factor” analysis for HPRES model

As the influence of the g_l parameter on the hydraulic permeability has been investigated [29, 36], it was suggested that $g_l = -1.9772$ may be used with sufficient correctness for the HYPRES-series of “fine” soil class. Figure 53 illustrates the impact of g_l on the safety factor analysis when using HYPRES’ “fine” SWCC and various g_l -factor values. The $g_l = -1.9772$ for HYPRES’ “fine” SWCC is always indicated with a thicker line.

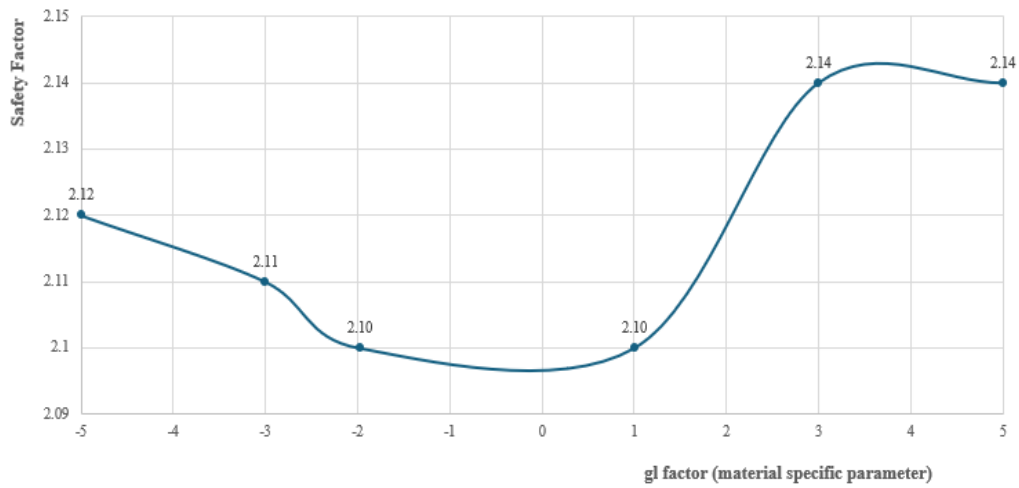


Figure 53 Influence of the g_1 factor on “safety factor” analysis for HPRES model

6.4 Final Analysis by Considering Worst Parameters

Referring the Sensitivity Analysis performed in **Section 6.3**, the parameters attached in Table 6 was considered for the final analysis of our Numerical FEM code simulation in Plaxis software to create the worst condition in the region and create the same land sliding and failure that happened in the region.

Table 6 Parameters values in the Ponzano region by considering
worst parameters achieved in previous sections

Conditions	Description	Parameters	Values	Unit
Known Parameters from Lab Tests	Unsaturated Unit Weight of Soil	γ_{unsat}	16.0	$\frac{\text{kN}}{\text{m}^3}$
	Saturated Unit Weight of Soil	γ_{sat}	20.0	$\frac{\text{kN}}{\text{m}^3}$
	Liquid Limit	W_L	52	-
	Plastic Limit	W_P	18	-
	Index of Plasticity	I_P	34	-
	Initial Void Ratio	e_{init}	0.8	-

	Over Consolidation Pressure	OCR	1.5	-
	Secant Reference Modulus	E_{50}^{ref}	10000 – 20000	kPa
	Reference Pressure	P_{ref}	640	kPa
	c'	c'	20	kPa
	ϕ'	ϕ'	23	Degree
	Poisson ratio	Poisson ratio	0.33	-
	ASTM D2487	Soil Classification	mostly CH (fat clay) and in some cases CL (lean clay)	-
Unknown Parameters achieved from “Sensitivity Analysis”	Drainage Type	-	Undrained A	
	Run-off Water	ψ_{max}	0.001	m
	Boundary Condition	BC	Left=close Right=Open	-
	Suction in “safety factor” phase	-	Ignored	-
	Evaporation	-	Not Considered	-
	History of Precipitation	-	531	days
	Saturated Permeability Ratio	K_{sat}	1	m/day
	Unsaturated Zone	ψ_{unsat}	1000	m
	SWCC Fitting Method	-	Van Genuchten Method	-
	SWCC Dataset	-	HYPRES dataset “fine” soil class	-

As described in **Section 2.3** there are two major landsliding happened in the Ponzano region in Section A-A, which has been described more clearly in Figure 54. To perform the Back-Analysis to achieve the same land sliding mechanism that happened in the region, we performed the Sensitivity Analysis on all of the possible parameters that could be effective on the mechanism of the failure in the region. Through calculating the worst scenarios of the parameters (Table 6), we will go ahead in this section to perform our final Back-Analysis in this section to achieve the best results that could be comparable with the real condition happened in the region in Ponzano in Section A-A by using our Numerical FEM Plaxis code analysis.

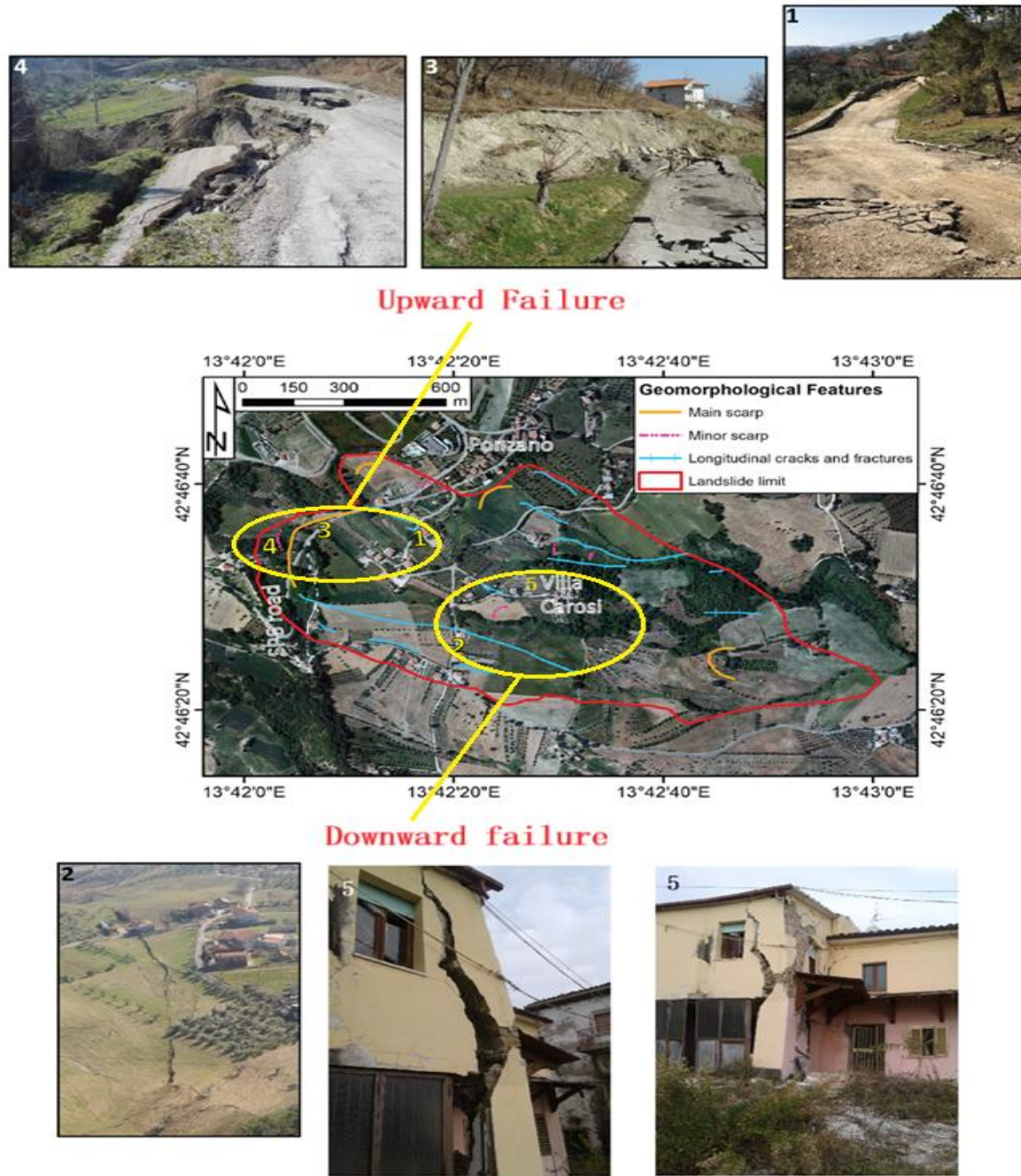


Figure 54 Two major land sliding mechanisms that happened in Section A-A of Ponzano

As described before, the simulation has been created by considering all of the achieved data and calculations and in a very fine mesh size (around 3.0 m), although the dimensions of the simulation was around 1.8 km in our Numerical FEM code analysis (*Section 6.2* and *Section 6.3*), which would take several days just for each run and analysis. After performing the analysis for our final model by considering all of the worst scenarios from previous sections, the following

results have been achieved, which showed a very clear failure mechanism that is comparable with the reality happened in the region from our Numerical FEM analysis in Plaxis ULTIMATE version software.

6.4.1 Results for Failure in Downward of Ponzano Region

6.4.1.1 Incremental Deviatoric Strain

In Plaxis FEM code software, the “incremental deviatoric strain” could be described as the following [34]:

$$\varepsilon_q = \sqrt{\frac{2}{3} \left[\left(\varepsilon_{xx} - \frac{\varepsilon_v}{3} \right)^2 + \left(\varepsilon_{yy} - \frac{\varepsilon_v}{3} \right)^2 + \left(\varepsilon_{zz} - \frac{\varepsilon_v}{3} \right)^2 + \frac{1}{2} (\gamma_{xy}^2 + \gamma_{yz}^2 + \gamma_{zx}^2) \right]} \quad (35)$$

Deviatoric strain is described as ε_q ,

ε_{xx} , ε_{yy} and ε_{zz} are described as normal strains in the x, y and z direction respectively,

ε_v is considered as the “volumetric strain” which could be calculated as the following:

$$\varepsilon_v = \varepsilon_{xx} + \varepsilon_{yy} + \varepsilon_{zz} \quad (36)$$

And γ_{xy} , γ_{yz} and γ_{zx} are described as the shearing strain parameters in the x, y and z direction respectively.

“Incremental deviatoric strain” represents the change in shape and distortion of the soil or material, without any change in its volume, during a single calculation step. “Incremental” means it’s not the total strain, but only the amount added during the current step of the analysis, and we can track how this shear deformation evolves step-by-step. Therefore, by using the option “incremental deviatoric strain” we can detect the shearing zone, plastic zone and potential failure surface, which are very critical in our Numerical FEM analysis to achieved results comparable

with the reality. By considering these conditions, Figure 55 shows the behaviour of failure in Section A-A which exactly happened in the downward of the Ponzano (Carosi village) the same as the reality happened by referring to Figure 54. Consequently, the 1st failure revealed in the simulation the same as reality in the downward of the model with a very clear failure mechanism.

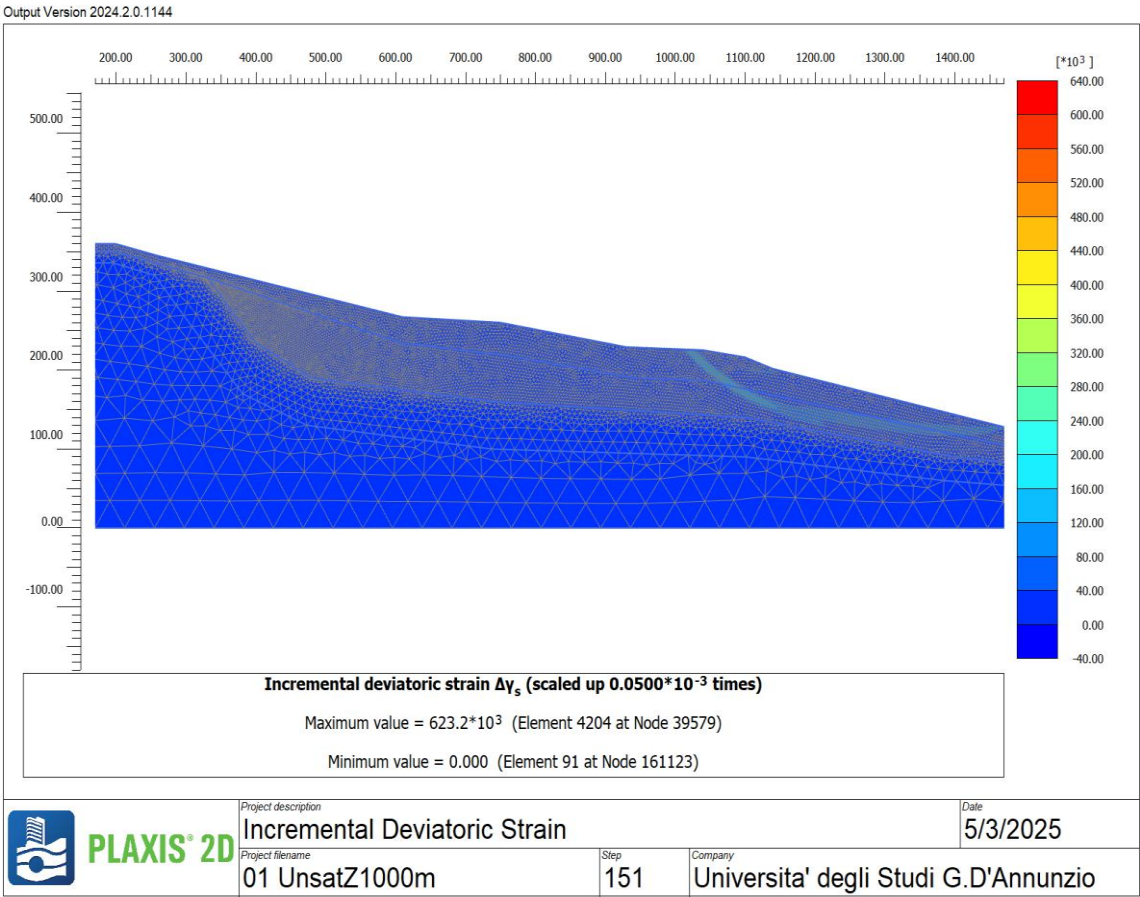


Figure 55 Incremental deviatoric strain in our “safety factor” phase analysis

6.4.1.2 Plastic Points

Referring the previous section, we should see the "plastic points" in the region, which showed high values of “incremental deviatoric strain”. These “plastic points” show that the soil has yielded in this region and meaning that stress state reached our material’s yield surface (Figure 56).

“Plastic points” indicated that there was permanent and irreversible deformation in our simulation, and they are no longer purely elastic, and they became plastic (permanent) straining. Figure 56 revealed the failure zone and mechanism of failure that is no longer stable, and declared the location is completely the same that the land sliding happened in Section A-A in the Ponzano region in the downward of the area in Carosi village as declared in Figure 54.

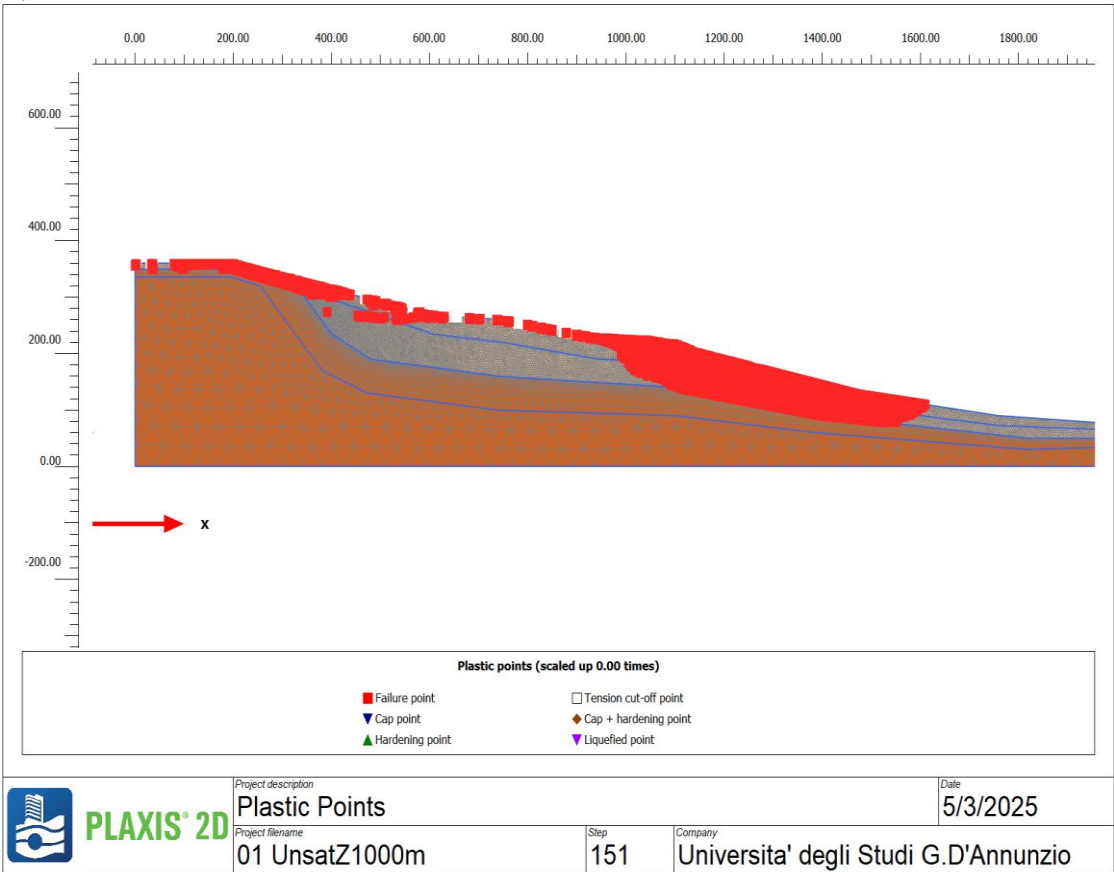


Figure 56 Plastic points in “safety factor” phase analysis

6.4.1.3 Safety Factor Analysis

As mentioned in **Section 6.1.8** the safety factor was calculated in accordance with “Strength Reduction Method (Phi/C Reduction)” in our Numerical FEM simulation. In this analysis, the method reduced soil strength parameters until our model no longer found an equilibrium and

collapse happened, which is comparable with our slides in the Ponzano region. Referring that, the results of this method for the simulated area is described in Figure 57.

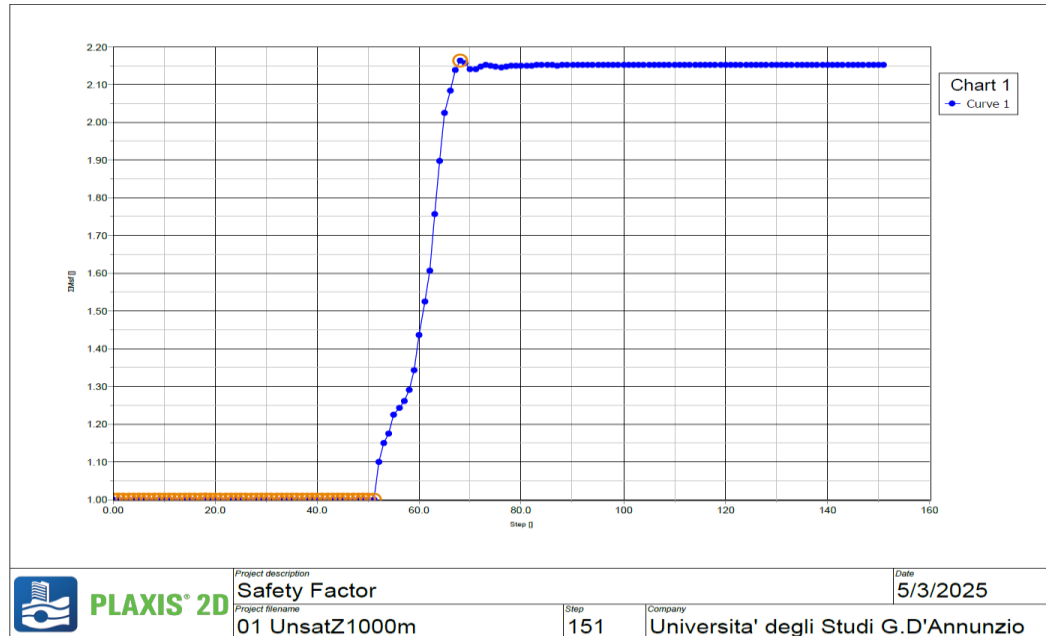


Figure 57 Safety factor analysis which showed failure in downward of Section A-A

6.4.1.4 Displacement Results

The “Sum Phase Displacements” option contains the different components of the accumulated displacement increments in the whole calculation of the phases as calculated at the end of the current phase and displayed on a plot of the geometry. In the following the total displacement in X and Y direction in “full coupled flow-deformation” and “safety factor” phases have been presented respectively, although care should be taken that the values for the “safety factor” analysis are meaningless and the output for the “safety factor” phase displayed as to clarify about the performance of deformation in the final simulation before failure happened in Ponzano (from Figure 58 to Figure 61).

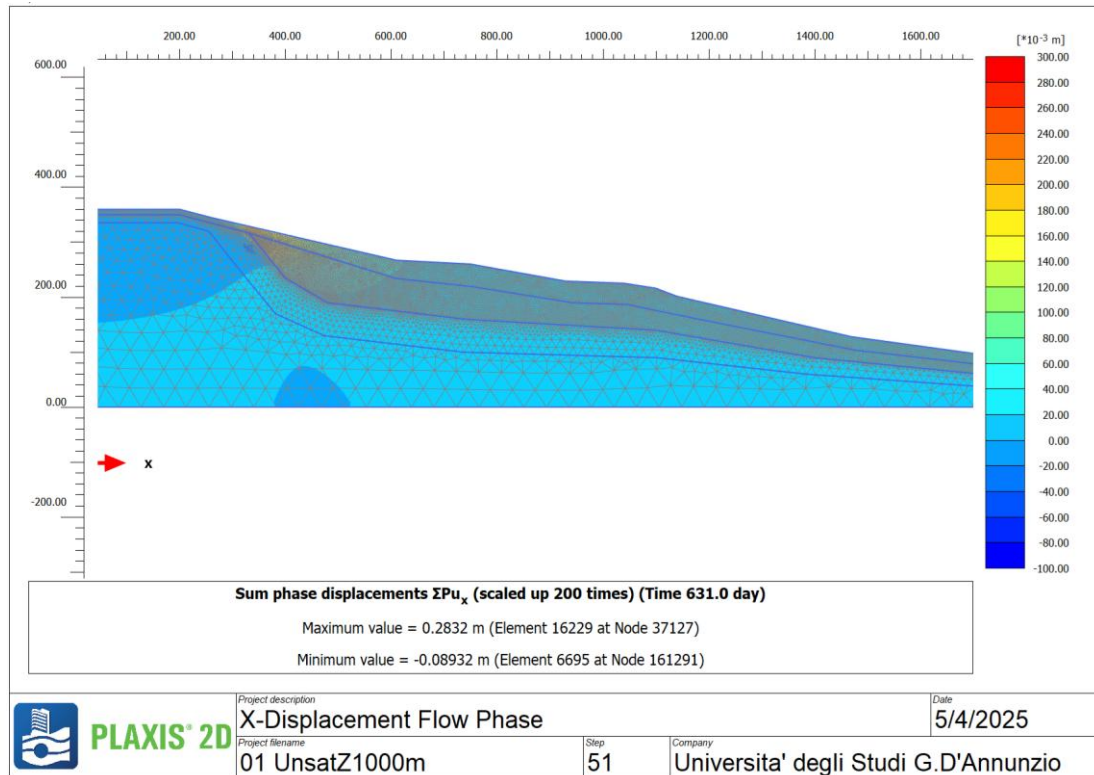


Figure 58 Sum phase displacement in the X direction for the “full coupled flow-deformation” phase

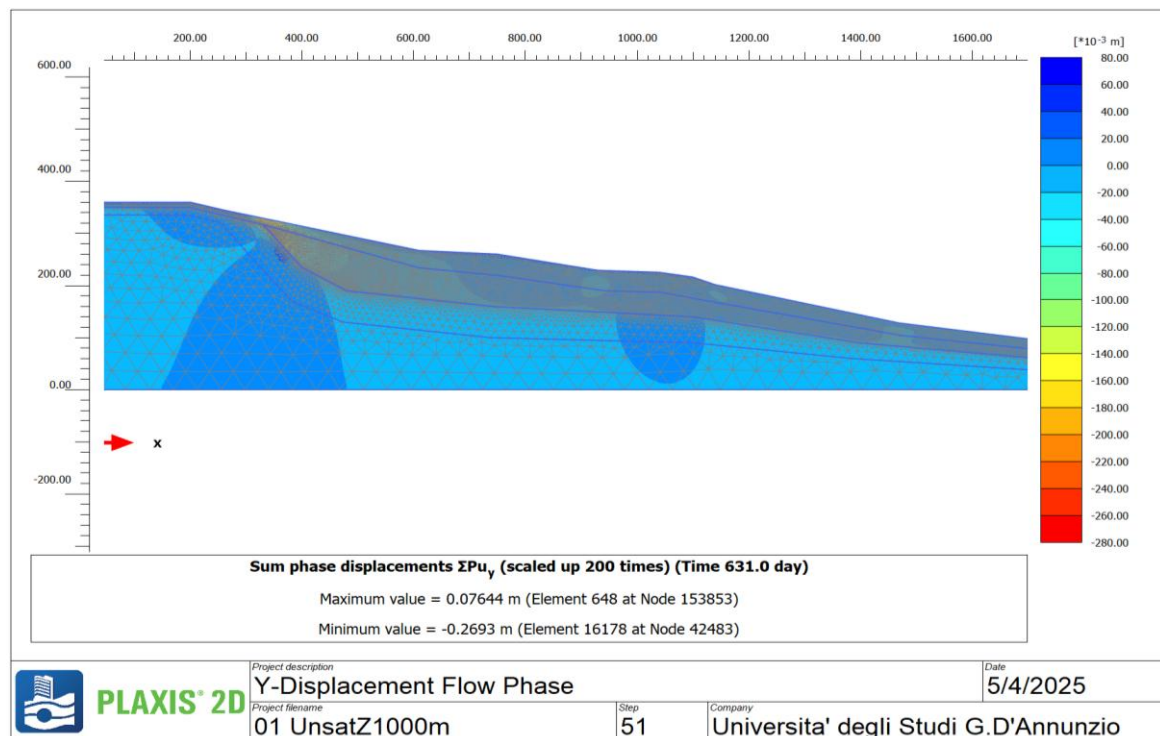


Figure 59 Sum phase displacement in the Y direction for the “full coupled flow-deformation” phase

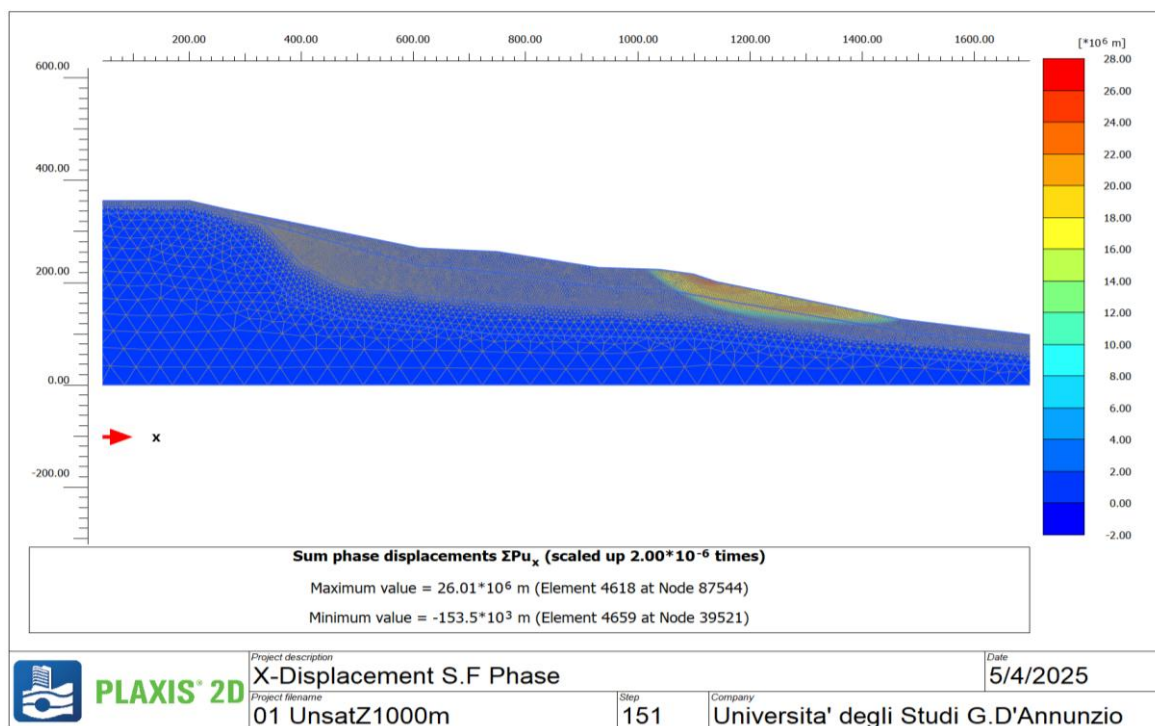


Figure 60 Sum phase displacement in the X direction for the “safety factor” phase

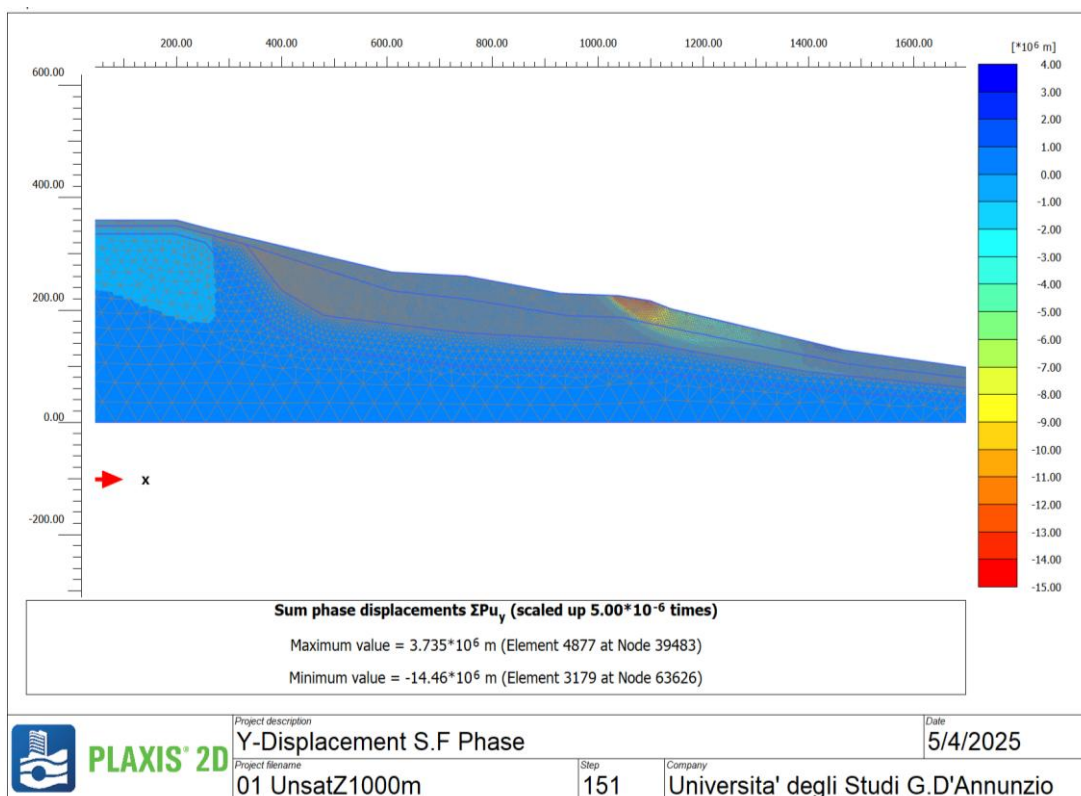


Figure 61 Sum phase displacement in the Y direction for the “safety factor” phase

6.4.1.5 Relative Shear Stress

“Relative Shear Stress” parameter is a dimensionless parameter which shows how close the shear stress in soil is to its shear strength and if it equal to be 1, it shows the soil is at failure. Therefore, by this parameter the critical zones can be identified in slope simulation in our project [16].

Figure 62 shows that the “Relative Shear Stress” is critical in most of the FEM simulation during the “safety” analysis. Therefore, it shows that although the 1st failure happened in the middle of the region (Carosi village), but results of “Relative Shear Stress” shows that probably there would another land sliding failure mechanism in another region of model as well as the middle.

Since the Plaxis FEM code software has the ability to deselect some part of the model from applying the “strength reduction method”, therefore in **Section 6.4.2** we will evaluate other part of the region as if another land sliding is going to be happened in the area or not. This pattern has been performed in the following and the final achievement have approved of this hypothesis that the 2nd failure mechanism happened in the upward of the model the same as the reality in the region.

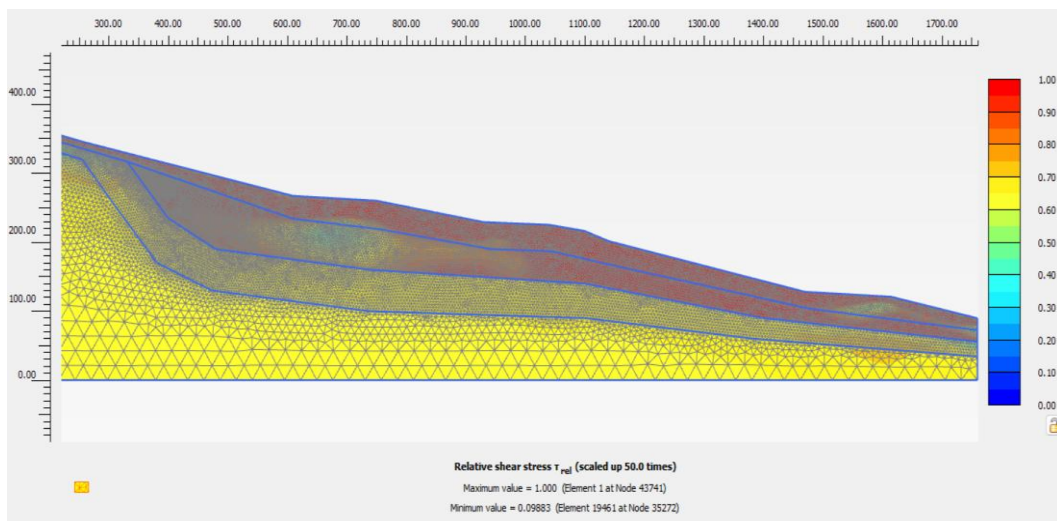


Figure 62 Relative shear strain for the “safety” phase

6.4.1.6 Groundwater Flow (q)

As mentioned before, there were no underground water flow found in the region, so evaluating the flow by considering just the precipitation history could lead us to understand how much flow affected the model.

The results in Figure 63 showed that since there was no underground water flow in subsoil layers, the precipitation history did not create deep underground water flow in the Ponzano region and just influenced the area near the ground surface and the results showed that mostly the first 5 meter below the ground surface has been affected by the precipitation data, although the precipitation was very intensive during the history before the land sliding.

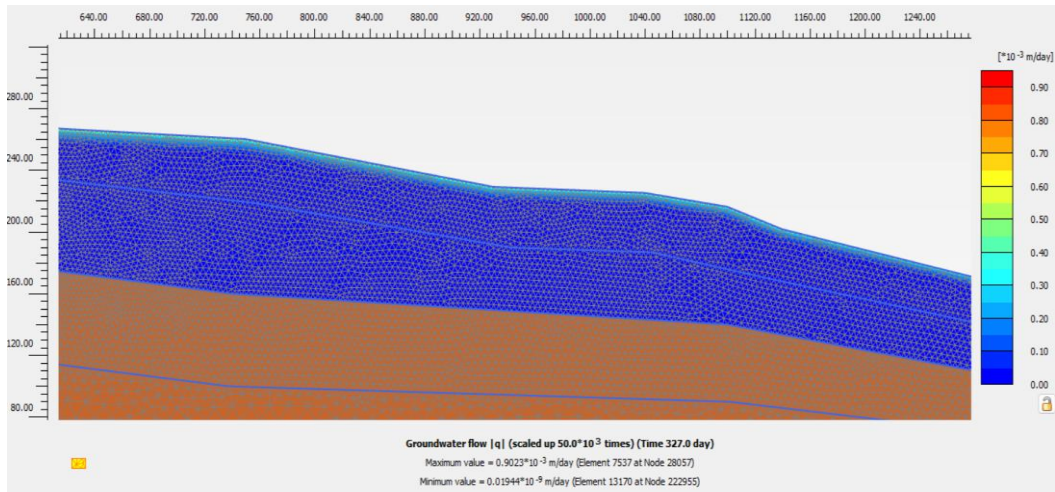


Figure 63 Groundwater flow condition (q) in the “full coupled flow-deformation” phase

6.4.1.7 Degree of Saturation (Sr)

“Degree of Saturation” is one of the important parameters in hydro-mechanical part of modelling in PLAXIS FEM code software. The “Degree of Saturation” measures how much of the pore space in our soil model is filled with water and it is defined as:

$$Sr = \frac{V_w}{V_v} \quad (37)$$

In this formula, S_r is the “Degree of Saturation”, V_w is the volume of water in pores and V_v is the total volume of the pores in the soil specimen. If all the pores are filled with water, the soil is fully saturated, and $S_r=100\%$ and if some pores are filled with air, the soil is partially saturated, and S_r is less than 100% .

In this model this parameter is very important because we are simulating the precipitation data and their infiltration into the model, so the degree of saturation changes dynamically. In this regard, the strength of our Numerical FEM soil simulation can be changed as the partially saturated soils often have higher apparent strength due to suction, but if heavy rain happened in the Ponzano region, the unsaturated soils could suddenly lose its strength as they become saturated and suction strength will be disappeared (refer the **Section 6.4.1.4**). Thus, considering this parameter in our Numerical simulation is one of the most important parameters. As we described in **Section 6.4.1.4**, we considered the “Unsaturated Zone” equal to 1000 m , so we created unsaturated zone unlimited, which would let the suction pore water changed dynamically and automatically during infiltration periods changes.

In this regard, as the degree of saturation decreased during dry period in Ponzano region, the suction pore pressure would be increased and if the saturation increased during wet period, the suction dropped down, which could be very important and is one of the most influential parameters caused the failure happened in the Ponzano region after melting the snow period in 25th January.

Because the mesh dimension is very small around 3.0 m and the dimension of the model is very large in 1.8 km , so to create a better view of conditions of the “degree of saturation”, we selected different “stress points” at different depths in the upward, middle and downward of the simulation, to focus on the parameter’s changes during the precipitation history whereas the 1st failure and 2nd failure happened in the region (Figure 64). It should also be noted that the “stress points” in Numerical FEM Plaxis code software, also known as Gauss points or integration points, are points inside the “15-Nodes element”, which are used to show the variation of stress, strain and plasticity calculations with high accuracy. In the following, to shorten the “stress point” in the results of the following graphs we use the abbreviate phrase of “SP” [34, 53].

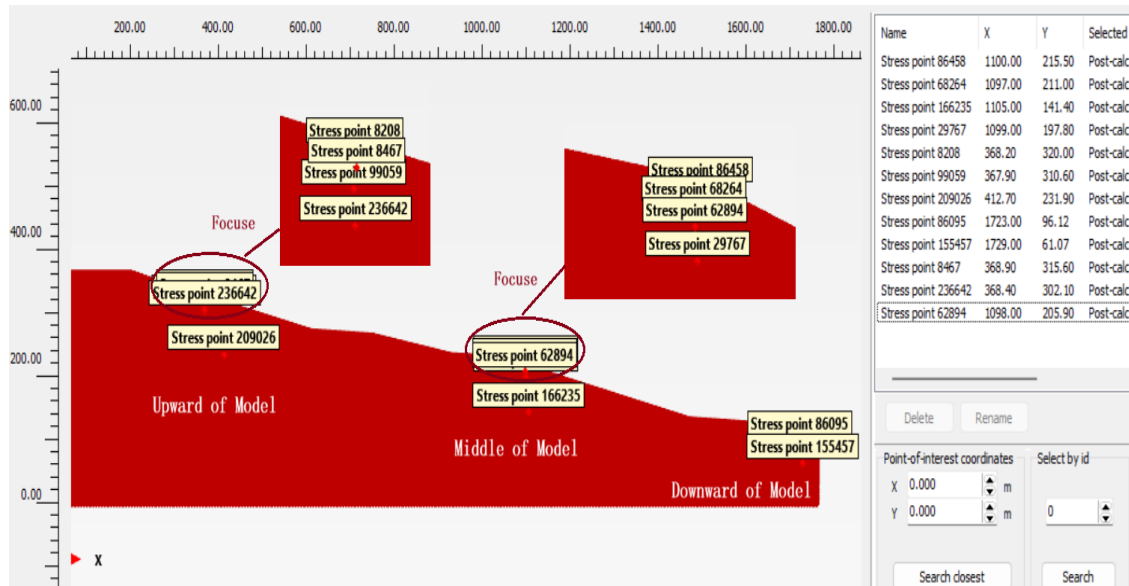


Figure 64 Selected “stress points” in the FEM model to describe the “Degree of Saturation”

At first, we focused on the middle and upward of the model, where the 1st and 2nd land sliding failure happened to analysis the reason the failure happened in these study case region (Figure 54).

In previous section we mentioned that the precipitation history with its intensive values did not create deep underground water flow in the Ponzano region and just influenced mostly the first 5 meters below the ground surface. The dynamic pattern of changes in the “degree of saturation” also confirmed the pattern of flow in our Numerical FEM simulation. The results show that the “degree of saturation” is influenced mostly near the ground surface up to around 20 m below the ground surface, and the “degree of saturation” after the depths of 20 m, was mostly keep constant and keep its initial saturation from its previous phase (Initial phase) for the middle and upward of the region.

The outputs specifically show that “degree of saturation” near the ground surface in depth 1.0 m, is completely in the harmony with the variation of precipitation intensity for the middle and upward of the simulation. As the intensity of precipitation decreased, the “degree of saturation” near the ground surface also reduced and when the intensity increased, the saturation as well increased.

In this depth (1.0 m below the ground surface), as the intensity of precipitation decreased after day 153, the rate of saturation started to be decreased as well and this reduction in saturation intensified during the snow period (days between 201 and 209). In the following, as the snow

started to be melted (days between 210 and 218) and the rate of infiltration increased in subsoil layer, the “degree of saturation” also started to be increased significantly in both of the middle and up part of the region, where the 1st and 2nd land sliding happened in these regions.

Figure 65 and Figure 66 also show that by moving deeper to the soil layer at depths 10 m and 20 m below the ground surface, the saturation was affected by the precipitation history, but it was increased smoothly by applying the precipitation history, and there were no harmony with the variation of precipitation intensity anymore. In these depths, we could not see any sudden drop or increase of “degree of saturation” during the changes of the intensity of precipitation for the middle and upward of the region, but the results showed these depths are still affected through the infiltration of the precipitation history.

The results at depths 65 m and 80 m below the ground surface for middle and upward of the region respectively show that by moving deeper to the under layer of our soil model, there is not any changes in the degree of saturation and soil layer keeps its initial saturation from its previous phase (Initial Phase) in the these depths and were not affected by the intensity of precipitation history.

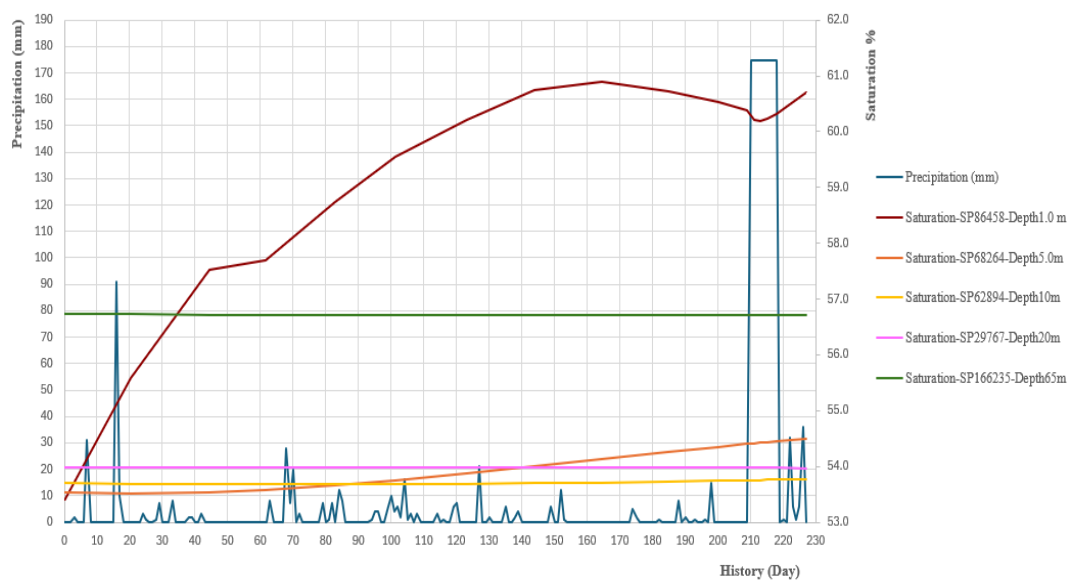


Figure 65 Dynamic changes of “degree of saturation” versus precipitation history in different depths for middle of the Numerical FEM model

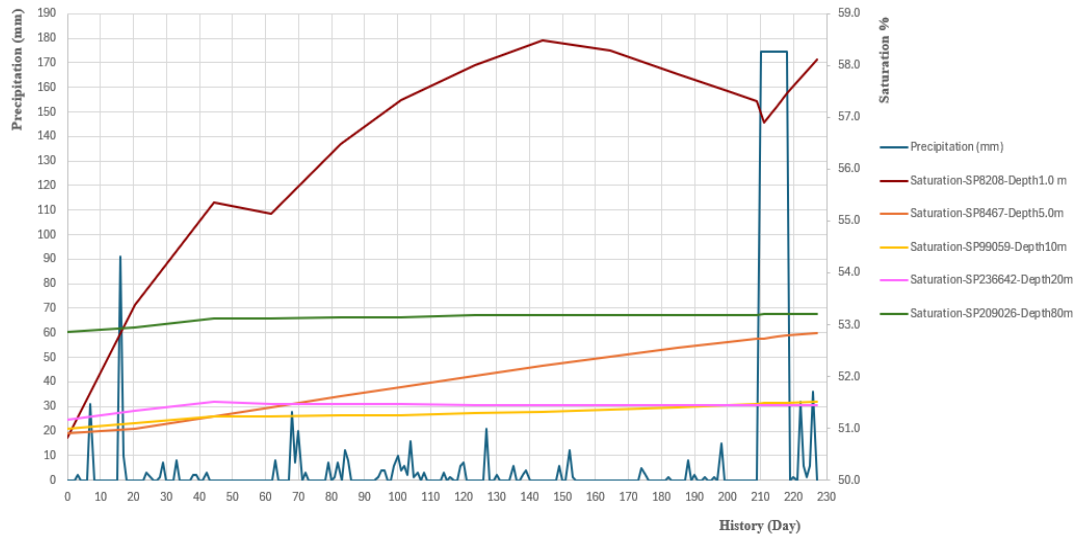


Figure 66 Dynamic changes of “degree of saturation” versus precipitation history in different depths for upward of the Numerical FEM model

In contrast, the result for the down part of our Numerical FEM simulation, where no land sliding failure reported in this region, showed that the “degree of saturation” was not changed significantly with the variation of precipitation intensity, even for points near the ground surface (Figure 67). The results showed that there was no harmony or even any significant increase of degree of saturation with the variation of precipitation intensity, even for area near the ground surface (Figure 67).

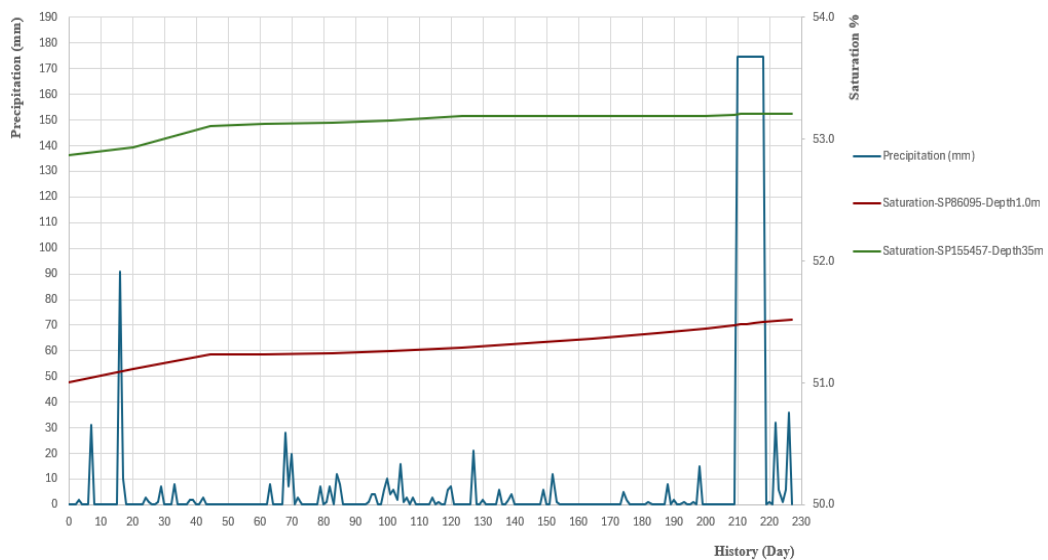


Figure 67 Dynamic changes of “degree of saturation” versus precipitation history in different depths for downward of the Numerical FEM model

6.4.1.8 Suction Pore Water Pressure

As described in **Section 6.1.8**, plotting the suction pore pressure is one of the important parameters as we can predict failure conditions in our Numerical FEM project. In this regard, we can follow the condition of suction pore pressure in the simulation, as if the suction pore pressure decreased during intensive precipitation history, the probability of failure mechanism increased significantly. As described in **Section 6.1.6** the relation between suction pore water pressure and degree of saturation is calculated by *SWCC* as the *hydraulic models* named *Van Genuchten Model*. This model described how suction (capillary pressure) changes with water content or degree of saturation in the soil. When the soil is fully saturated, so all pores are filled with water and suction pore water pressure decreased. As the soil dries, degree of saturation decreases and capillary forces increase, leading to more negative pore pressure (suction), which is one of the important parts in this Numerical FEM Plaxis 2D code simulation.

The same as previous section in the beginning we focused on the middle and upward of the model, where our 1st and 2nd land sliding failure happened to analysis the reason in the study case region (Figure 68 and Figure 69).

In this regard, the strength of our Numerical FEM soil simulation can be changed as the partially saturated soils often have higher apparent strength due to suction, but if heavy rain happened, the unsaturated soils could suddenly lose its strength as they become saturated and suction strength will be disappeared. Therefore, as the degree of saturation decreased during dry period in Ponzano region, the suction pore pressure would be increased and if the saturation increased during wet period, the suction dropped down, which could be one of the most influential parameters caused the failure happened in the Ponzano region after melting the snow period on 25th January (wet period).

As discussed in previous section the “degree of saturation” is influenced mostly near the ground surface up to 20 m below the ground surface, and the “degree of saturation” after the depths of 20 m, was mostly keep constant and keep its initial saturation from its previous phase (Initial phase) for the middle and upward of the region. In the following, the suction results also confirmed this achievement.

The outputs specifically show that the suction near the ground surface in depth 1.0 m, is also completely in the harmony with the variation of precipitation intensity for the middle and upward of the simulation. As the intensity of precipitation decreased, the suction near the ground surface started to be increased and when the intensity increased, the suction as well increased.

This is very clearly that as the intensity of precipitation decreased after day 153, the rate of suction started to be increased as well and this increase in suction intensified during the snow period (days between 201 and 209) and during the intensive infiltration of snow melting in the subsequent days as the wet period (days between 210 and 218), there is significant decrease in the suction for both of the middle and upward of the region where the 1st and 2nd land sliding happened in these region.

Figure 68 and Figure 69 also shows that for depth 5.0 m, although the suction was affected by the precipitation history and it was decreased smoothly by applying the precipitation history around 17% and 31% for the middle and upward of the region respectively, but there was no harmony with the variation of precipitation intensity, and we could not see any sudden drop or increase of suction during the changes of the intensity of precipitation for the middle and upward of the region.

By moving deeper to the soil layer at depths 10 m and 20 m below the ground surface, we can see small decrease of suction compared with the suction at depth 5 m, but in any way it shows still the decreasing in suction pore water pressure in soil layer.

The results at depths 65 m and 80 m below the ground surface for middle and upward of the region respectively show that by moving deeper to the under layer of our soil model, there is not any significant changes in the suction pore water pressure and soil layer keeps its initial suction from its previous phase (Initial Phase) in the these depths and was not affected by the intensity of precipitation history.

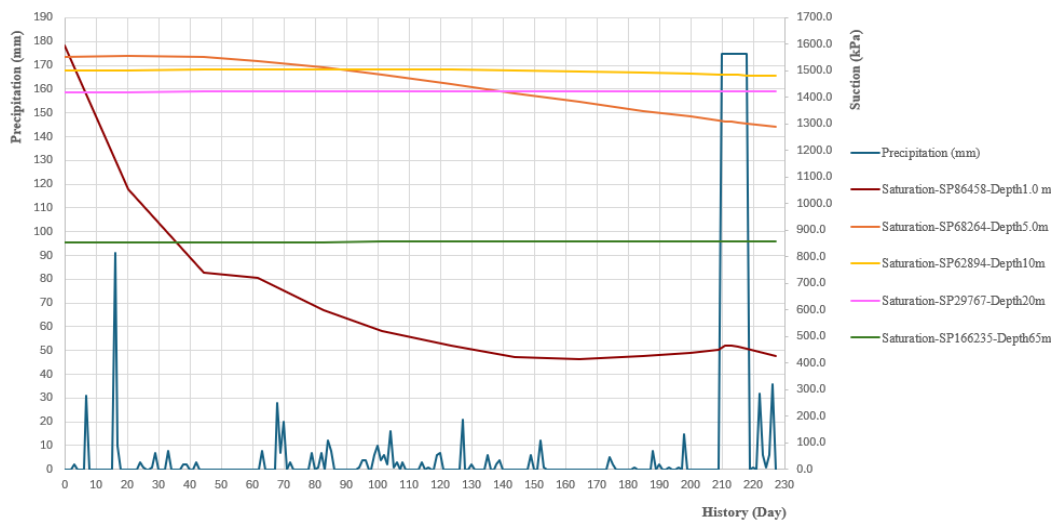


Figure 68 Dynamic changes of suction versus precipitation history in different depths for middle of the Numerical FEM model

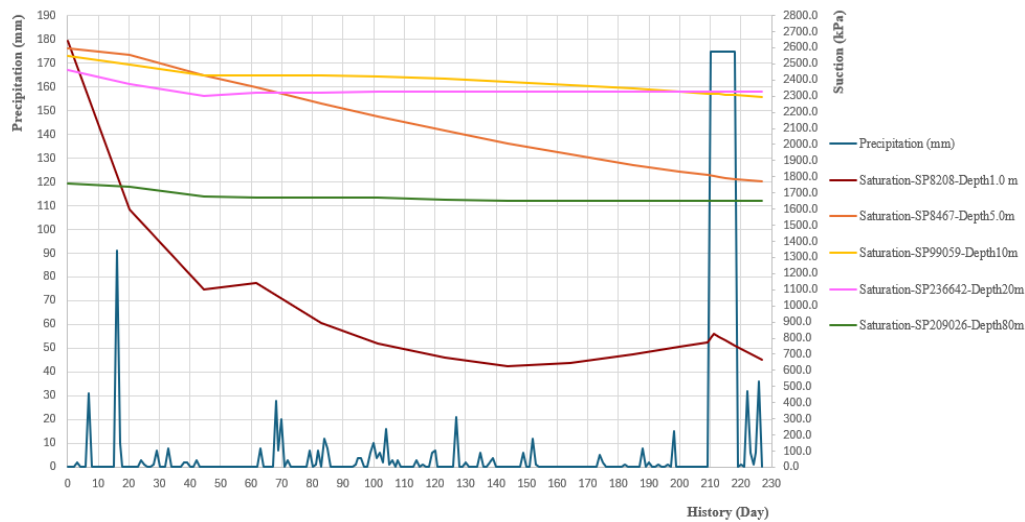


Figure 69 Dynamic changes of suction versus precipitation history in different depths for upward of the Numerical FEM model

In contrast, the result for the dynamic change of suction versus precipitation history for the downward of our Numerical FEM simulation, where no land sliding failure reported in this region, showed the suction pore water pressure was not changed significantly with the variation of precipitation intensity, even for points near the ground surface (Figure 70).

Figure 70 shows that for both of the depths 1 m and 35 m, although the suction was affected by the precipitation history and it was decreased smoothly by applying the precipitation history, but there was no harmony with the variation of precipitation intensity, and we could not see any sudden drop or increase of suction pressure during the changes of the intensity of precipitation even for area near the ground surface.

This shows that because of the slop geometry of the area, the flow infiltration from precipitation history was moved from upward of the region to downward of the region, so the downward of the region had mostly stable condition in the saturation (Section 6.4.1.7) and suction during the variation of precipitation intensity and the drying and wetting period of the infiltration history did not affect the downward of the model. This could be one of the reasons why no land sliding failure mechanism was reported from this region.

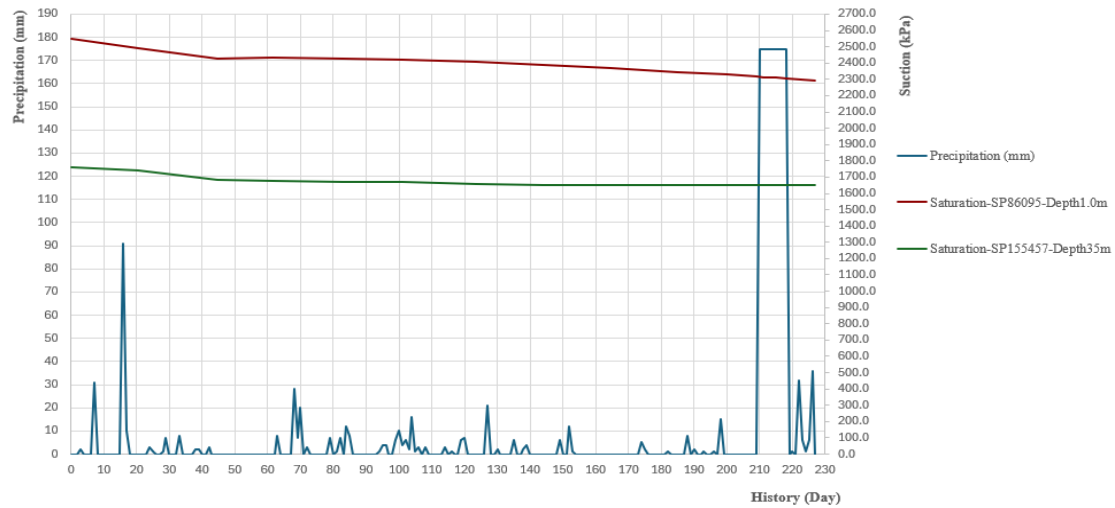


Figure 70 Dynamic changes of suction versus precipitation history in different depths for downward of the Numerical FEM model

6.4.1.9 Excess Pore Water Pressure

Plotting the excess pore pressure also could be one of the important parameters as by increasing the excess pore water pressure significantly, the effective shear resistance of soil layer could be dropped down, and land sliding failure mechanism could be happened in the selected region. In this regard, we can follow the condition of excess pore pressure in the simulation, as if the excess pore pressure increased during the precipitation history, the probability of failure mechanism increased significantly. Therefore, we should follow all of these steps in our Numerical FEM analysis during our final Back-Analysis calculation to achieve the best results.

The graph of Figure 71 shows that the excess pressure just near the ground surface at depth 1 m below the ground surface, is in harmony with the variation of precipitation intensity and we do not have significant excess pore water pressures after the depths of 5 m, and it is mostly near zero or in a small value.

Referring to the points near ground surface at depth 1 m, as the intensity of infiltration decreased, the excess pressure near the ground surface also decreased and when the intensity increased, the excess pore water pressure also increased.

Figure 71 shows that for depth 5.0 m, the excess pore water pressure increased smoothly from zero to -262 kPa and -827 kPa for the middle and upward of the region respectively during precipitation history.

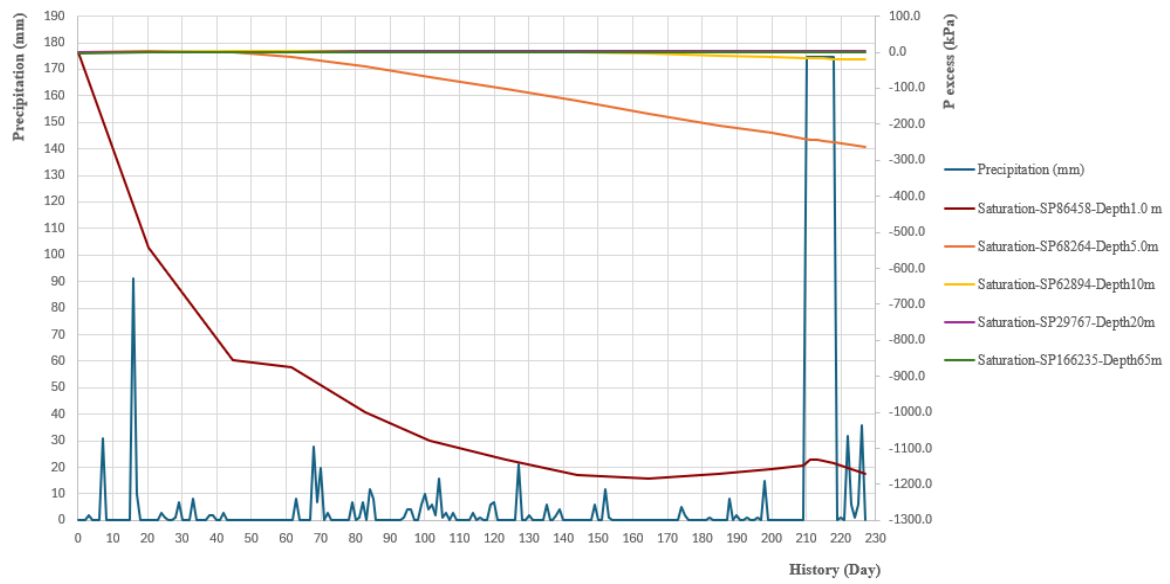


Figure 71 Dynamic changes of excess pore water pressure versus precipitation history in different depths for middle of the Numerical FEM model

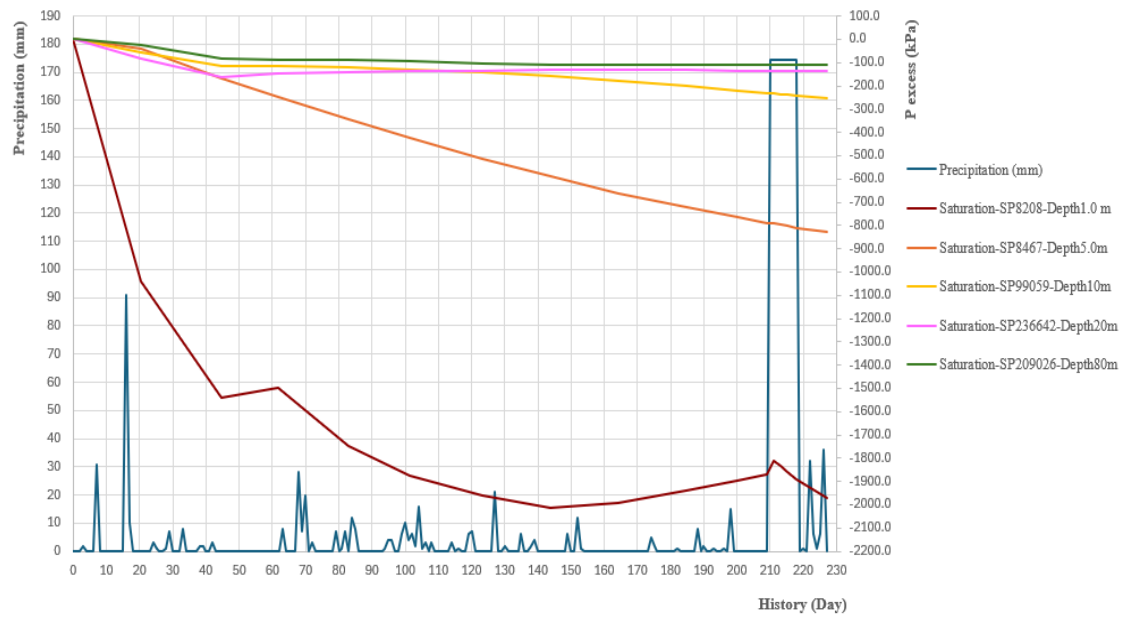


Figure 72 Dynamic changes of excess pore water pressure versus precipitation history in different depths for upward of the Numerical FEM model

6.4.2 Results for Failure in Upward of Ponzano Region

Plaxis FEM ULTIMATE has the ability to deselect a specific region from considering the “strength reduction method”. As described in previous section the 1st land sliding failure mechanism happened in the downward of the region and now, we want to analysis the 2nd land sliding failure mechanism and find the location of failure mechanism and compare with the reality, as if they are the same or not. Thus, we created two different zone in our Numerical FEM calculations and separate the downward of the model (right-side of Figure 73) from the upward of the model (left-side Figure 73), to see if any land sliding failure mechanism maybe happened in the upward region also, but of course with higher safety factor compared with the S.F of downward of the simulation in **Section 6.4.1.3**, same as the reality that two clear land sliding failure mechanism happened in the region.

Since the Numerical FEM Plaxis simulation is the same and we just created the 2nd zone in the upward of the region and focused on the failure mechanism in this section, so we interpret the safety factor phase analysis in this section, as the other parameters should be the same of previous section.

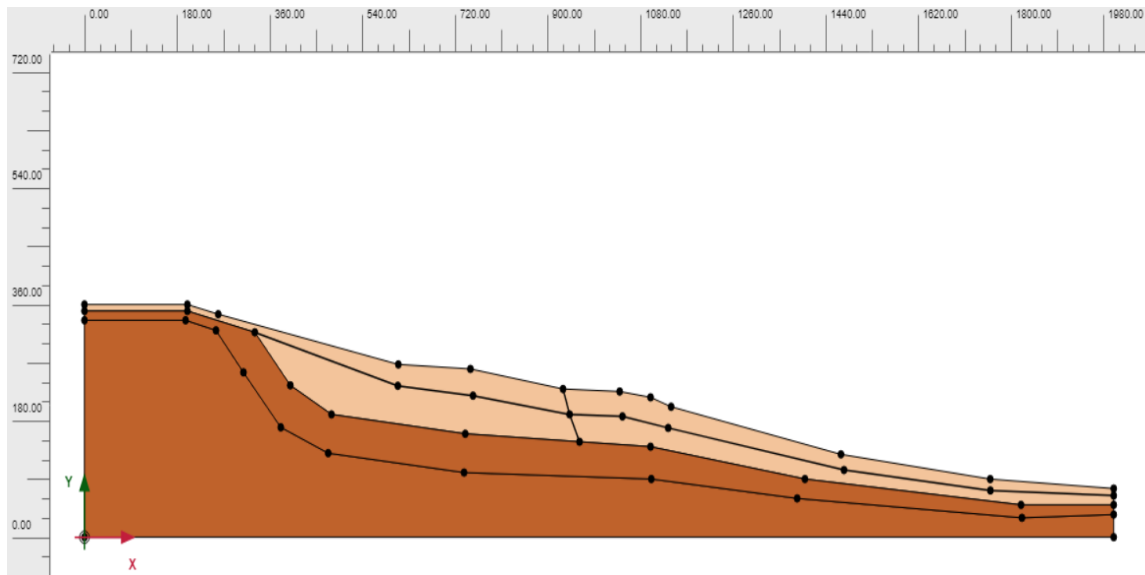


Figure 73 Creating two different zones in Section A-A and deselecting middle part of region

from “strength reduction method”

6.4.2.1 Incremental Deviatoric Strain

As mentioned in *Section 6.4.1.1* “Incremental deviatoric strain” represents the change in shape and distortion of the soil or material, without any change in its volume, during a single calculation step and we can track how this shear deformation evolves step-by-step. Thus, by this option in this section we can detect the shearing zone, plastic zone and potential failure surface, and understand if the same land sliding failure mechanism happened in the region, which could be comparable with the reality.

By considering these conditions, Figure 74 shows the behaviour of failure in Section A-A which exactly happened in the upward of the Ponzano region the same as the reality happened by referring to Figure 54. Therefore, by applying the worst conditions from the Sensitivity Analysis in *Section 6.3*, the same 2nd clear land sliding failure mechanism, the same as the reality happened in the region of Ponzano, happened in our Numerical Back-Analysis in FEM Plaxis code calculations.

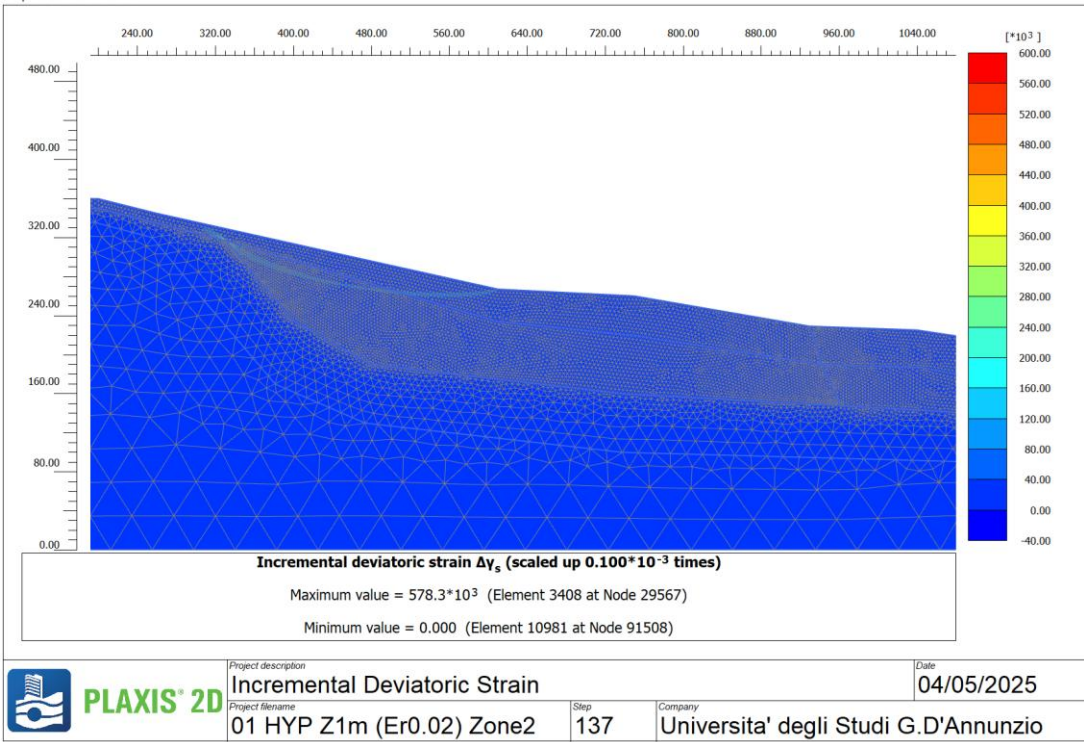


Figure 74 Deviatoric strain for the “safety factor” phase analysis in upward region of Ponzano the same as reality

6.4.2.2 Plastic Points

Referring the previous section, we checked the "plastic points" in the upward of the region Ponzano region, which showed high values of “incremental deviatoric strain”. These “plastic points” showed that the soil has yielded in this region also and meaning that stress state reached our material’s yield surface and permanent and irreversible deformation achieved in our simulation (Figure 75). Thus, Figure 75 revealed the clear failure zone and mechanism of failure that is no longer stable, and declared the location is completely the same that the land sliding happened in Section A-A in the Ponzano region as declared in Figure 54.

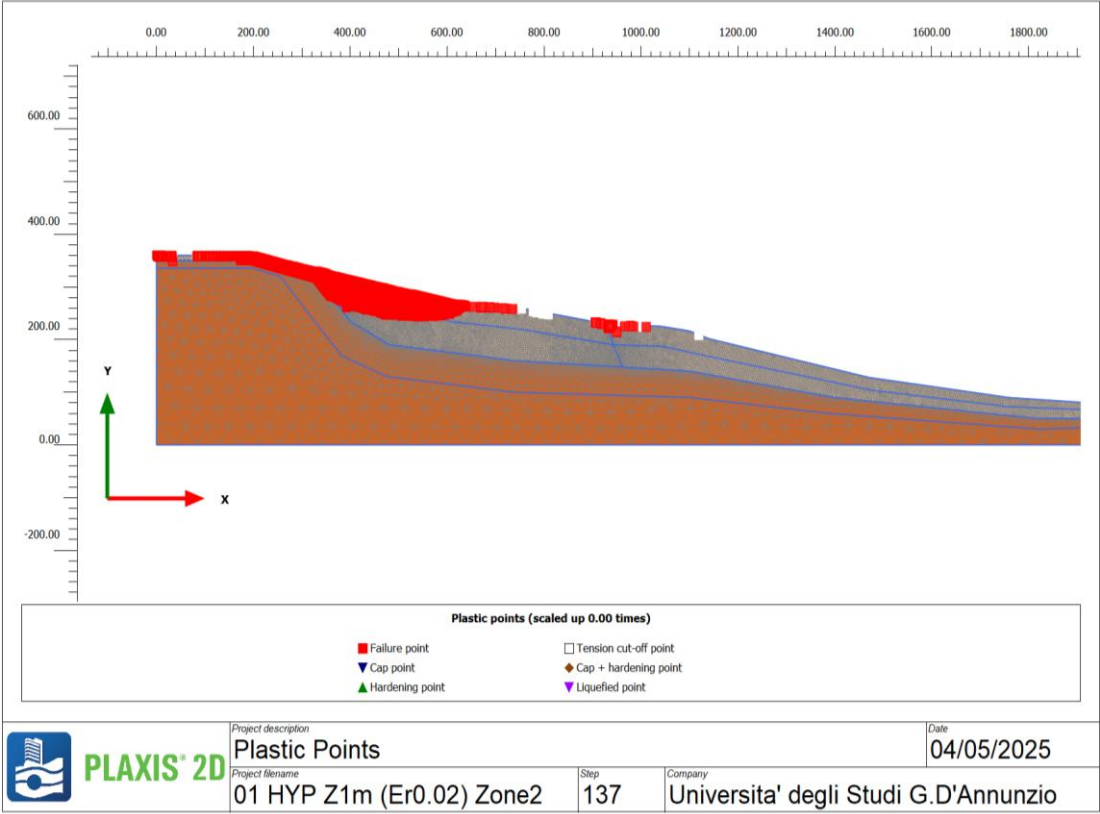


Figure 75 Plastic points of the “safety factor” phase analysis in Section A-A in upward of Ponzano area

6.4.2.3 Safety Factor Analysis

As described in *Section 6.1.8* the safety factor was calculated in accordance with “Strength Reduction Method (Phi/C Reduction)” in our Numerical FEM simulation. In this analysis, the method reduced soil strength parameters until our model no longer found an equilibrium and collapse happened, which is comparable with our land slides in the Ponzano region. Referring that, the results of this method focused on the upward region of Ponzano is described in Figure 76, which showed the 2nd failure, but of course with higher S.F value compared to S.F of *Section 6.4.1.3*. The suction pore water pressure value also approved this pattern as described in *Section 6.4.1.8*.

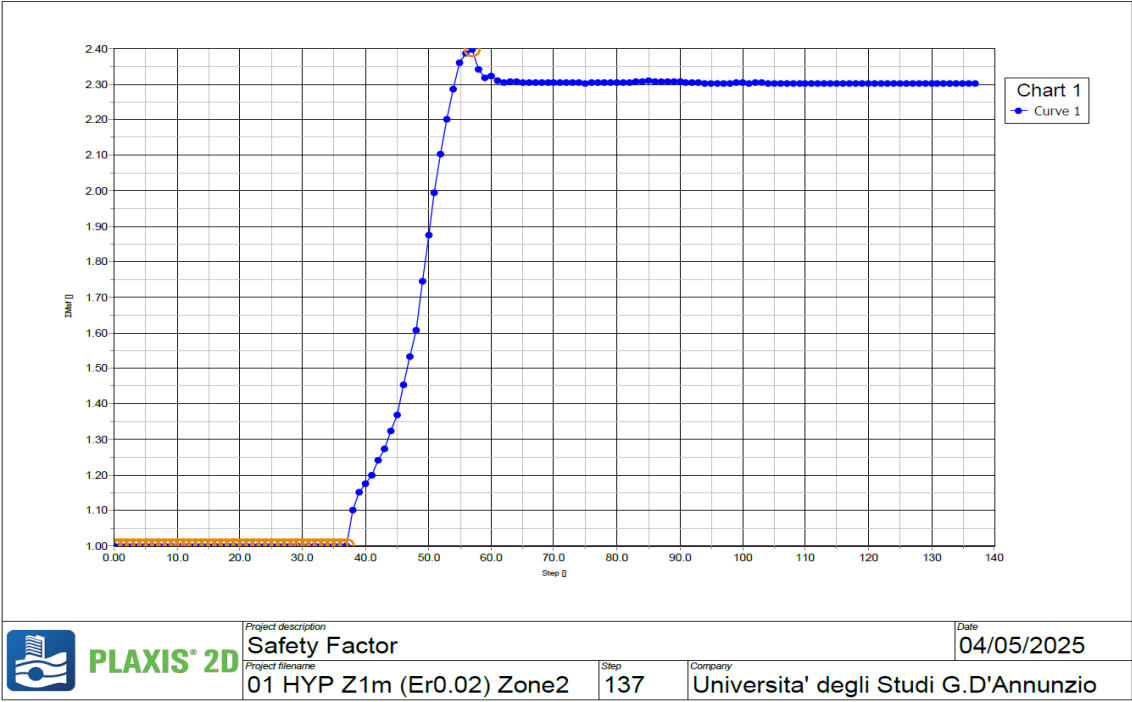


Figure 76 “Safety Factor” chart referring the upward of Ponzano region

6.4.2.4 Displacement Results

The “Sum Phase Displacements” option contains the different components of the accumulated displacement increments in the whole calculation of the phases as calculated at the end of the current phase and displayed on a plot of the geometry. In the following the total displacement in

X and Y direction in the “safety factor” phases have been presented respectively, although care should be taken that the values for the “safety factor” analysis are meaningless and the output for the “safety factor” phase displayed as to clarify about the performance of deformation in the final simulation before failure happened in Ponzano (from Figure 58 to Figure 61).

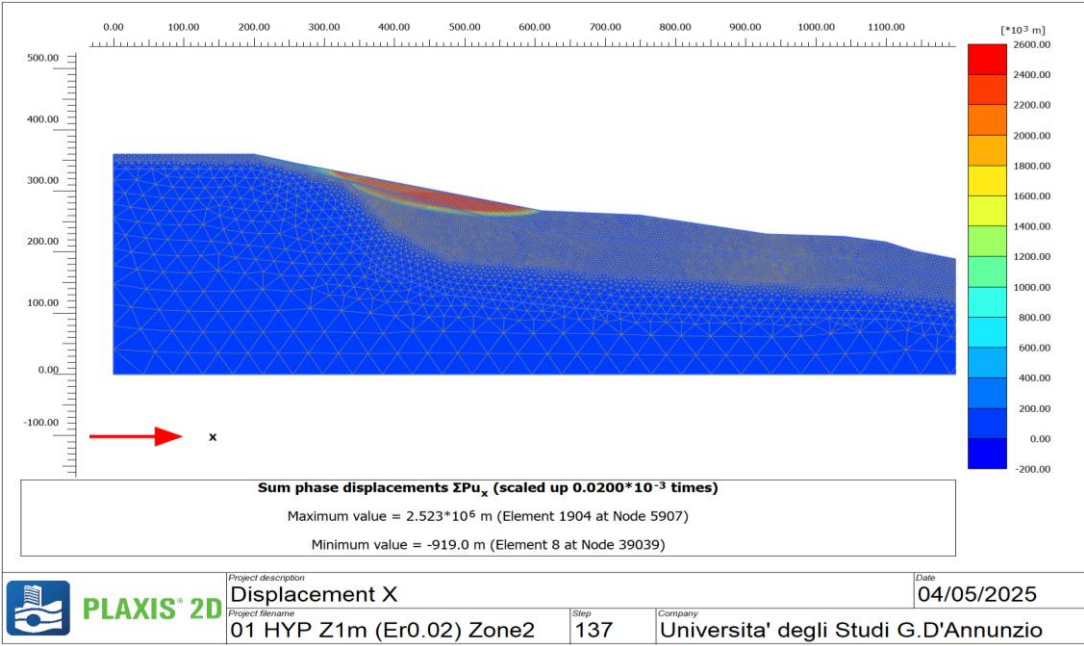


Figure 77 Sum phase displacement in the X direction for the “safety factor” phase

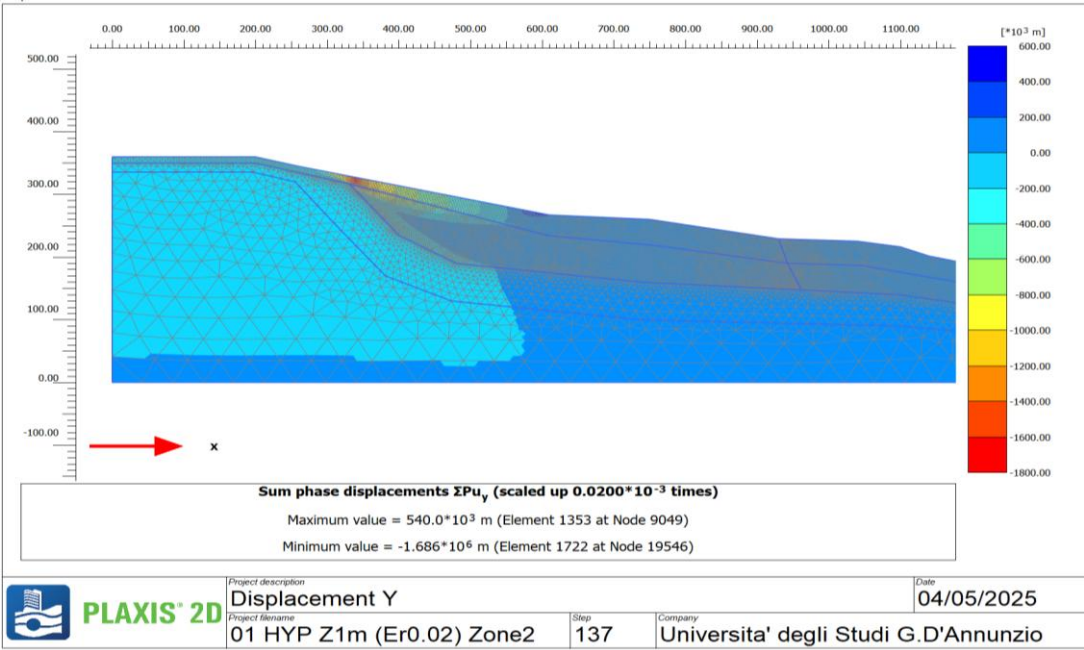


Figure 78 Sum phase displacement in the Y direction for the “safety factor” phase

7 Discussion

The 1st phase of the project showed that how much the Plaxis FEM analysis could achieve the best results for the deformation happened in the region the same as the coloured-deformation map of the Lidar DEM data 2013-2019 achieved from the calculations with different software such as CloudCompare, ReCap, Civil 3D and Rhinoceros software. Although one of the disadvantages of Plaxis FEM code software is that it calculates until the failure and does not calculate the deformation after the failure happened, but by considering the scale the pattern is the same for the deformation happened in the region for both Plaxis FEM 3D code analysis and the coloured-deformation map of the Lidar DEM data 2013-2019.

In the following, as we focused on the Section A-A of the Ponzano model to create a very accurate model and achieved the same two main land sliding failure that happened in this section. By applying the Sensitivity Analysis on the Numerical FEM Plaxis code 2D simulation, the results showed that the most important parameter which triggered the land sliding failure mechanism in the Ponzano region was related to the SWCC model and the Van Genuchten model for the hydraulic model in the FEM simulation. The Van Genuchten model correlated the suction pore water pressure to the water content and degree of saturation in unsaturated soil layer, which is related to the condition of the Ponzano region.

The results showed that how much changing the Van Genuchten parameters g_a , g_n and g_i could change the safety factor in the analysis. In addition, choosing the Van Genuchten Equation Fitting parameters could affect the safety factor as well. In this Sensitivity Analysis we finally chose the HYPRES dataset for our fitting parameters, which showed the minimum values for the safety factor analysis. Of course, considering other parameters like, run-off water, boundary conditions, history of precipitation and unsaturated zone could also affect the safety factor considerably, as we performed for them also in our Sensitivity Analysis, and the worst output results from each evaluation was defined and the final model to create the worst state in the region was created.

After performing the Sensitivity Analysis, we go ahead to our final step, which was creating the final simulation of the Ponzano region by applying the Back Analysis in our Numerical simulation. Referring the previous researches, they showed that there are two main land sliding happened in the Ponzano region, one in the top part of the region and the other in the middle part of the area (villa Carosi) and these consequences were achieved through the site surveying and observations in the filed [2, 3, 4].

In our Numerical FEM simulation, as we did not have many different Geotechnical engineering parameters to simulate the area, we started to perform the Sensitivity Analysis to have the best estimate of the unknown parameters to be considered in our simulation and then after that perform our final Back-Analysis in our Numerical Plaxis simulation to simulate the study case of Ponzano region and achieved the same results that happened in the region. After performing all of unknown parameters through Sensitivity Analysis, our final Numerical FEM Plaxis also achieved the same two main big land sliding, first in the middle part of the simulation and the second in the upward of the region, exactly the same of the reality and previous research articles [2, 3, 4], although in our simulation we did not have many different and critical Geotechnical engineering parameters, but by using the Sensitivity Analysis technique this goal was achieved in our Numerical FEM simulation successfully.

In addition, in previous research (Lorenza Solari et al., 2018) it was mentioned that an earth flow-like with rotational behaviour with a deep slip surface around 15 m depth happened in the region. They also mentioned that in just few years, the slow motion of the landslide led to the rupture of the inclinometric instrumentation at the depth ranging between 13 and 16 m below surface and the slip surface admitted the rupture at around 15 meters depth [4].

In addition, as mentioned before, degree of saturation and pore water pressure (specifically the suction pore pressure), play the key factors on the soil shearing resistance and in our Numerical FEM simulation. Suction increases the soil strength and if the infiltration happened and intensive rainfall applied, the degree of saturation increased, which resulted on dropped down of suction pore water pressure and resulted to decrease the soil strength.

In the study case area as there were a high and intensive precipitation history and there were different of wet and dry period in the region, so the degree of saturation was changed repeatedly and this completely in reverse affected the suction pore pressure, which changed the soil shearing strength and finally caused failure in upward and middle of the region.

After performing the Sensitivity Analysis and applying the final back Analysis in our Numerical FEM model, the first failure happened in the middle of the region (Carosi village) and the second failure happened in the upward of the study region, of course with higher safety factor compared with safety value achieved in the middle of region. For both of the top and middle of the region that the two main land sliding happened, the degree of saturation and suction pore water pressure were completely in harmony with the variation of the precipitation history at depth 1.0 m below the ground surface (near the surface). As the intensity of rainfall increased, the saturation increased simultaneously and suction pressure dropped down, and during the time precipitation intensity decreased, the saturation also decreased, and the suction pressure increased in reverse.

Our Numerical FEM results showed that the rate of reduction in the degree of saturation and increase in the suction pressure intensified during the snow period (between days 201 and 209), whereas no infiltration happened in this period, and as the snow melting started (between days 210 and 218) the saturation increased and suction pressure decreased significantly, and all of these wet and drying cycles finally results on failure on 12th February 2017 (day 217 in our time Numerical period division).

By moving to deep soil layer at depths 10.0 m in the middle of the region, our Numerical FEM results showed smooth increase of saturation and smooth decrease of suction pore water pressure despite of the precipitation variations. The reduction in the suction pore water pressure at depth 10.0 m, was one of the main reasons that resulted the failure mechanism in the middle of the study case region. This reduction of suction pore pressure, as one of the key factor in shearing resistance, also repeated at depth 20.0 m, but with slighter rate, which confirmed the shearing failure mechanism that happened between depth 10.0 and 20.0, the same as previous research (Lorenza Solari et al., 2018) depicted of rupture of the inclinometric instrumentation at the depth ranging between 13 and 16 m below surface and also the slip surface at around 15 meters depth.

By moving to the down of our Numerical FEM simulation the results showed no failure mechanism the same of the filed observations and satellite SAR data from previous researchers approved no land sliding and failure in the down of the region [2, 3, 4]. Our Numerical FEM showed very slight and smooth increase in degree of saturation and slight decrease of suction, even for depth 1.0 m below the ground surface, where we found harmony and significant changes in the up and middle part of the simulation. Therefore, our final Back Analysis approved also the condition of the down part of the region by creating Numerical FEM simulations and calibrating parameters through the Sensitivity Analysis.

Performing Back-Analysis and Sensitivity Analysis always create challenges as many parameters are unknow and analysis all the possible and affective parameters could be a very tough and time-consuming challenges. In this analysis, to increase the accuracy of the simulation and since two major big land sliding happened with very near distance with each other's, this led us to create a huge 1.8 km dimension simulation in the Numerical FEM analysis with the mesh size distribution of around 4 meters. Each Sensitive Analysis taken **several days** considered **just for one run**, and this challenged the project much more, but it was worth it, and this showed the power of Numerical FEM calculations that under the conditions the calculation performed in the right way, the results could be trustable and comparable with the reality.

8 Conclusion

Plaxis FEM software solution showed that it could achieve the deformed shape of the study region the same as the coloured-deformation map of Lidar DEM data created through CloudCompare, ReCap, Civil 3D and Rhinoceros software.

In the following, we moved deeper in the project and focused on Section A-A, to simulate very accurate model of the region and achieve the same failure mechanism that happened in the region through the Numerical FEM simulation. Since, many different Geotechnical engineering parameters were unknown and undetermined, we used the Sensitivity Analysis at the first step to calibrate our Geotechnical engineering parameters and then after that we go ahead to create our final model by the Back-Analysis to achieve the same results happened in the study case area.

In the first step, the Sensitivity Analysis showed the most important parameters that triggered the big land sliding in the Ponzano region was the SWCC model and the Van Genuchten model for the hydraulic model in the FEM simulation as they correlate the suction pore pressure to the water content and degree of saturation in unsaturated soil layers.

The Sensitivity Analysis showed that changing the Van Genuchten parameters g_a , g_n and g_l could change the safety factor in the analysis. In addition, choosing the Van Genuchten Equation Fitting parameters and choosing the HYPRES dataset for our fitting parameters, revealed the minimum values for the safety factor analysis. Of course, considering other parameters like, run-off water, boundary conditions, history of precipitation and unsaturated zone could also affect the safety factor considerably, as we performed for them also in our Sensitivity Analysis, and the worst state in the study case region was created in our Numerical FEM simulation.

After applying the Sensitivity Analysis and calibrating the Geotechnical engineering parameters, we moved to create our final FEM simulation through the Back Analysis in Plaxis software. After applying the Back Analysis, the results showed two main and big land sliding and failure mechanism in the up and middle part of the region, the same of field and satellite SAR data from previous researched and confirmed with the real condition of the Ponzano region. The results showed two main land sliding happened between depth 10 m and 20 m below the ground surface. Our Numerical FEM simulations showed by moving toward the surface, the variations and changes for suction pore water pressure and degree of saturation (two major factors that affected the land sliding) were clearer and more significant, as we observed harmony in changing the suction pore pressure and saturation at depth 1.0 m near the surface. At this depth, the saturation increased, and suction pressure decreased as the intensity of precipitation data increased, and vice

versa. These wet and drying periods, specifically during the snow period (between days 201 and 209), that no infiltration happened in the region and after the snow melting happened in the region (between days 210 and 218), resulted the two big land sliding happened in the region on 12th February 2017 (day 227 in our precipitation history).

These variation of saturation and suction pressure were not observed in the down part of the model in our Numerical EFM simulation, even at depth 1.0 m near the ground surface. The results showed very slight variations in suction pore pressure and saturations during the precipitation history, and this revealed that during wet and drying periods the down part of the region was not affected by, so no failure mechanism happened in this region, the same as the reality condition in the region.

Therefore, this was approved that by applying the Sensitivity Analysis in a region that many different parameters are unknown and calibrating the parameters, this could be possible to evaluate the condition of the region in Numerical FEM solutions, and this showed the power of the Numerical FEM calculations. In addition, by applying the final Back-Analysis calculations in the region through the Numerical FEM solution, it is possible to achieve the same failure mechanism that happened in the region the same as the reality.

All of the performed achievement was considered to evaluate the power of Numerical FEM simulation through powerful FEM software like Plaxis FEM code software, that how they could behave and simulate a real case region with huge dimension, like 1.8 km, and applying different techniques like Sensitivity Analysis and Back Analysis to achieve the exact and same results of a real case study region.

9 Financial Research Funding

The License of Numerical FEM Plaxis code ULTIMATE version software (Academic license) was bought from the *G,D'Annunzio University* from *professor Nicola Sciarra* and *professor Antonio Pasculli* in 2022 an 2023 and the related paper was published on Scopus: [2-s2.0-85204775120](#). In the following by moving to the *Technical University of Graz* to complete the project, *professor Franz Tschuchnigg* delivered me the full access to the lincense of both *3D and 2D Numerical FEM Plaxis code ULTIMATE version software* on 2024 and 2025 and the related paper is going to be published in the in the “Geoscience” journal of MDPI publisher with the cooperation of *Technical University of Graz* in *Austria*.

10 Acknowledgement

As I moved to *Technical University of Graz* in Austria to complete my final phase of the project on 2024 as the visiting period and in cooperation with *professor Franz Tschuchnigg* and *professor Helmut Schweiger*, thanks so much from them for all of their support during these period, even after returning from *TU Graz*, I am still in connection with *professor Franz Tschuchnigg* and giving the report to *professor Franz Tschuchnigg* to publish the paper for the final and last phase of the project in Ponzano region using the Numerical FEM Plaxis code software, which is under the process to be published in the “Geoscience” journal of MDPI publisher.

In addition, I acknowledge from the “*Airbus Defence and Space GmbH*” Company in Germany, which provided the Lidar DEM data of the history 2019 with high resolution 4.16 m Pixel size free of charge for the project to be imported as the point cloud data into the Civil 3D software and improve the comparison deformation quality that happened in the area with the final Numerical Plaxis FEM 3D simulation, which increased the accuracy of the final results as described in *Section 4.3.2*.

11 Attending Conferences and Courses

- 1- My 2nd Doctoral paper in cooperation with professor Franz Tschuchnigg from the “Institute of Soil Mechanics, Foundation Engineering and Computational Geotechnics” of Technical University of Graz in Austria with title “***Geotechnical Parameters Calibration to Predict Land Sliding in a Test Case Area Through Back-Analysis and Sensitivity Analysis Using Plaxis FEM 2D Code software***” is under the process to be published in “***Geoscience***” Journal of MDPI publisher.
- 2- Attending the conference “*World Multidisciplinary Earth Sciences Symposium*” for the presentation of “***Digital Modelling to Predict the Land Sliding Hazard in a Selected Area; Ponzano’s (Italy) Test Case***”, in Prague, Czech Republic 28.08.2023.
- 3- Attending the conference “***Presentation of the Guide to landslides in Italy***” by the Italian of Applied and Environmental geology, L’Aquila 26.10.2023
- 4- Attending the course “***INTRODUCTION TO COMPUTATIONAL MODELLING AND FLUIDODYNAMICS***” with Professor Antonio Pasculli, May 2024, 20h di lezioni teoriche e casi di studio

12 Appendix

12.1 Granular Distribution Size Graphs

12.1.1 Granular Distribution Size Depth 5.0 – 5.5 m

SETACCIATURA

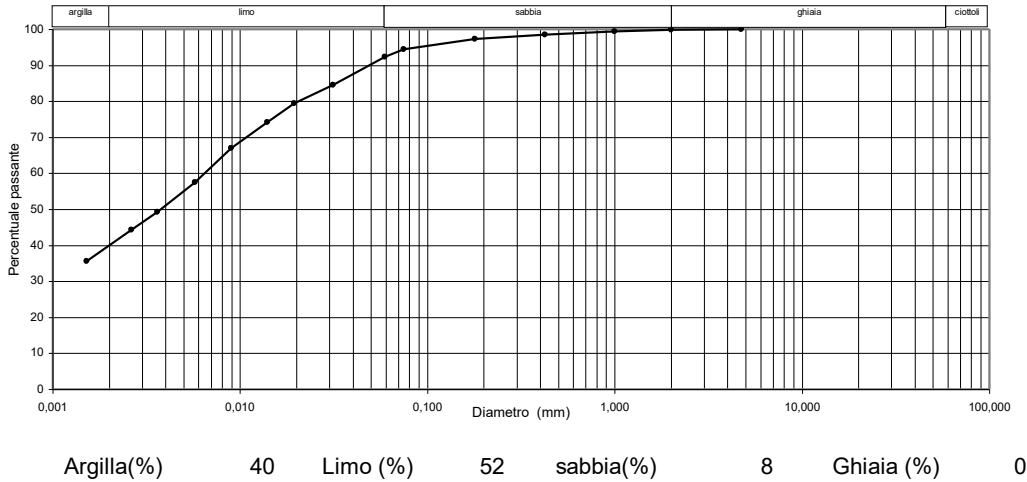
m= 157,40 g (peso iniziale)

Setaccio	trattenuto parziale	passante
(mm)	(g)	%
4,75	0,00	100,0
2,0	0,31	99,8
1,0	0,57	99,4
0,425	1,39	98,6
0,180	1,91	97,3
0,075	4,54	94,5
fondo		

SEDIMENTAZIONE

m_w = 62,97 g (peso iniziale umido)
w = 3,2 % (contenuto d'acqua)
m = 61,02 g (peso iniziale)
ρ_s = 2,7 Mg/m³ *
* = dato presupposto

Diametro equivalente	passante
(mm)	%
0,0593	92,4
0,0313	84,6
0,0195	79,4
0,0139	74,2
0,0090	66,9
0,0057	57,5
0,0036	49,2
0,0026	44,2
0,0015	35,7



Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla debolmente sabbioso

12.1.2 Granular Distribution Size Depth 10.0 – 10.5 m

SETACCIATURA

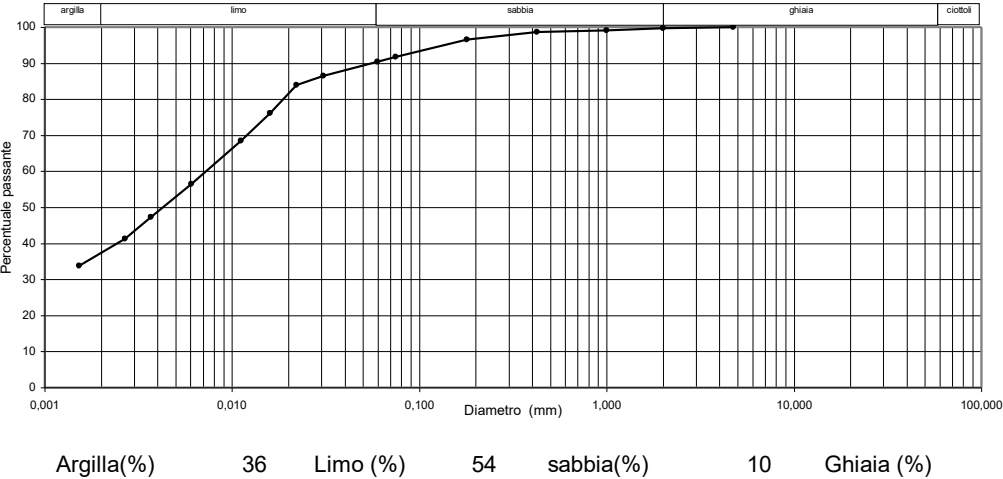
m= 144,62 g (peso iniziale)

Setaccio (mm)	trattenuto parziale (g)	passante %
4.75	0.00	100.0
2.0	0.52	99.6
1.0	0.80	99.1
0.425	0.71	98.6
0.180	2.91	96.6
0.075	7.10	91.7
fondo		

SEDIMENTAZIONE

$m_w =$ 63,28 g (peso iniziale umido)
 $w =$ 2,8 % (contenuto d'acqua)
 $m =$ 61,53 g (peso iniziale)
 $\rho_s =$ 2,7 Mg/m³ *
* = dato presupposto

Diametro equivalente (mm)	passante %
0,0598	90,3
0,0308	86,5
0,0221	83,9
0,0160	76,1
0,0112	68,4
0,0061	56,5
0,0037	47,2
0,0027	41,3
0,0015	33,8



Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla debolmente sabbioso

12.1.3 Granular Distribution Size Depth 15.0 – 15.4 m

SETACCIATURA

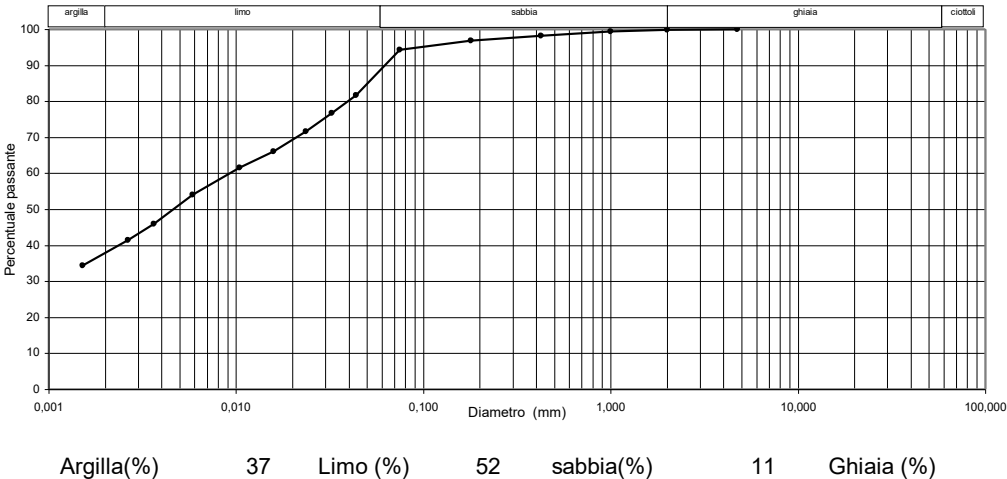
m= 193,42 g (peso iniziale)

Setaccio (mm)	trattenuto parziale (g)	passante %
4,75	0,00	100,0
2,0	0,37	99,8
1,0	0,77	99,4
0,425	2,30	98,2
0,180	2,70	96,8
0,075	4,90	94,3
fondo		

SEDIMENTAZIONE

$m_w =$ 64,81 g (peso iniziale umido)
 $w =$ 2,6 % (contenuto d'acqua)
 $m =$ 63,16 g (peso iniziale)
 $\rho_s =$ 2,7 Mg/m³ *
 * = dato presupposto

Diametro equivalente (mm)	passante %
0,0438	81,7
0,0324	76,7
0,0236	71,7
0,0158	66,1
0,0104	61,6
0,0059	54,1
0,0036	46,0
0,0026	41,5
0,0015	34,5



Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione
 delle indagini geotecniche 1977)

limo con argilla sabbioso

12.1.4 Granular Distribution Size Depth 20.0 – 20.4 m

SETACCIATURA

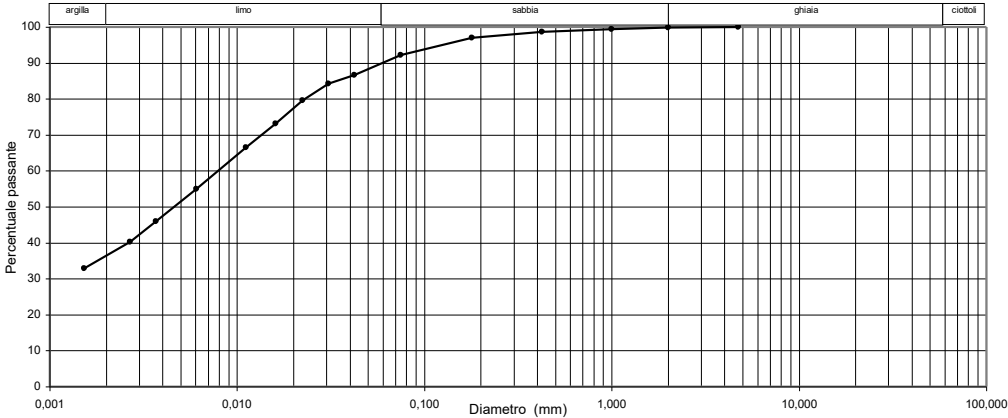
m= 168,90 g (peso iniziale)

Setaccio	trattenuto parziale	passante
(mm)	(g)	%
4,75	0,00	100,0
2,0	0,32	99,8
1,0	0,59	99,5
0,425	1,44	98,6
0,180	2,62	97,1
0,075	8,34	92,1
fondo		

SEDIMENTAZIONE

m_w = 65,01 g (peso iniziale umido)
w = 2,8 % (contenuto d'acqua)
m = 63,22 g (peso iniziale)
ρ_s = 2,7 Mg/m³ *
* = dato presupposto

Diametro equivalente	passante
(mm)	%
0,0422	86,7
0,0308	84,2
0,0224	79,6
0,0161	73,1
0,0112	66,6
0,0061	55,0
0,0037	46,0
0,0027	40,2
0,0015	32,9



Argilla(%) 35 Limo (%) 53 sabbia(%) 12 Ghiaia (%) 0

Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla debolmente sabbioso

12.1.5 Granular Distribution Size Depth 25.0 – 25.5 m

SETACCIATURA

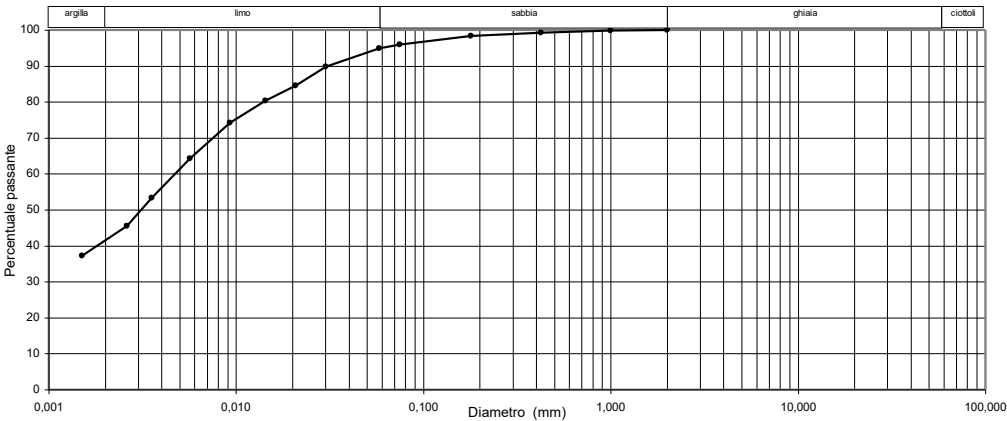
m= 204,65 g (peso iniziale)

Setaccio (mm)	trattenuto parziale (g)	passante %
2.0	0.00	100.0
1.0	0.40	99.8
0.425	1.24	99.2
0.180	1.84	98.3
0.075	4.69	96.0
fondo		

SEDIMENTAZIONE

$m_w =$ 62,88 g (peso iniziale umido)
 $w =$ 3,0 % (contenuto d'acqua)
 $m =$ 61,07 g (peso iniziale)
 $\rho_s =$ 2,7 Mg/m³ *
* = dato presupposto

Diametro equivalente (mm)	passante %
0.0581	94.9
0.0302	89.7
0.0209	84.5
0.0144	80.4
0.0093	74.1
0.0057	64.2
0.0036	53.3
0.0026	45.5
0.0015	37.2



Argilla(%) 42 Limo (%) 53 sabbia(%) 5 Ghiaia (%) 0

Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla

12.1.6 Granular Distribution Size Depth 29.0 – 29.5 m

SETACCIATURA

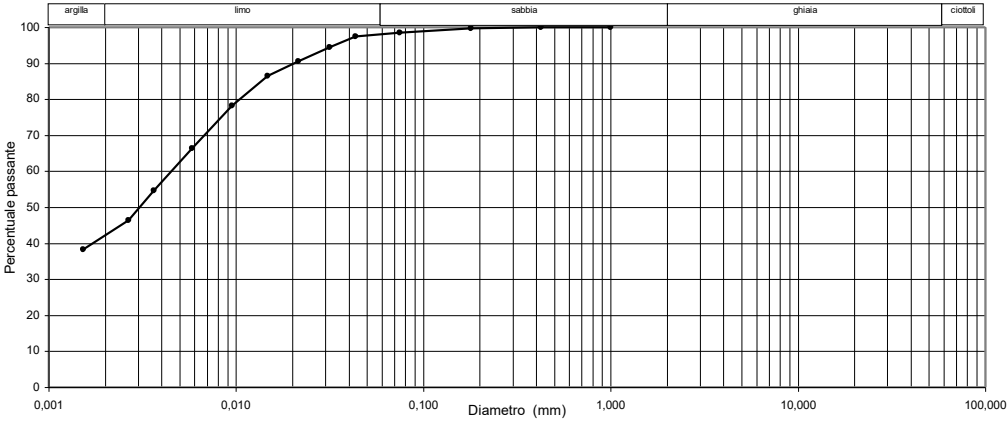
m= 134,72 g (peso iniziale)

Setaccio (mm)	trattenuto parziale (g)	passante %
0,425	0,00	100,0
0,180	0,50	99,6
0,075	1,50	98,5
fondo		

SEDIMENTAZIONE

m_w = 63,89 g (peso iniziale umido)
w = 18,7 % (contenuto d'acqua)
m = 53,82 g (peso iniziale)
 $\rho_s = 2,7 \text{ Mg/m}^3$ *
* = dato presupposto

Diametro equivalente (mm)	passante %
0,0434	97,4
0,0316	94,4
0,0215	90,6
0,0148	86,5
0,0096	78,2
0,0058	66,4
0,0036	54,6
0,0027	46,3
0,0015	38,4



Argilla(%) 42 Limo (%) 56 sabbia(%) 2 Ghiaia (%) 0

Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla

12.1.7 Granular Distribution Size Depth 40.0 – 40.3 m

SETACCIATURA

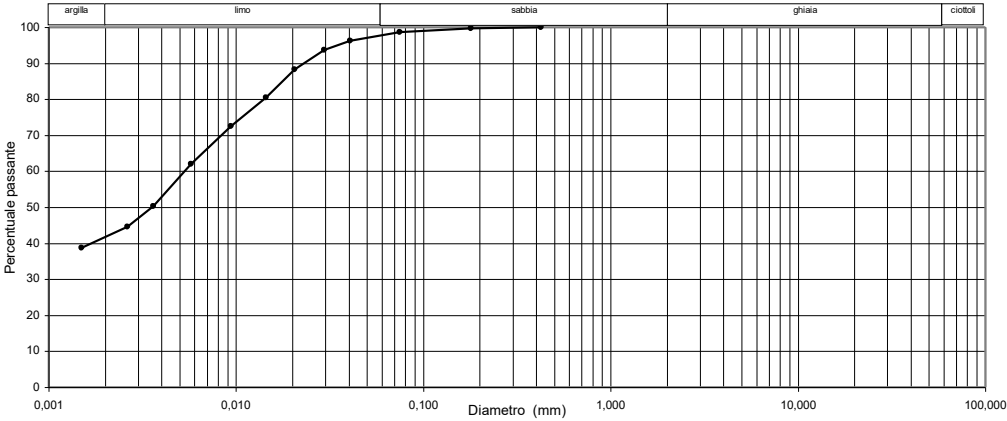
m= 164,50 g (peso iniziale)

Setaccio (mm)	trattenuto parziale (g)	passante %
0,425	0,00	100,0
0,180	0,55	99,7
0,075	1,64	98,7
fondo		

SEDIMENTAZIONE

m_w = 68,77 g (peso iniziale umido)
w = 14,2 % (contenuto d'acqua)
m = 60,22 g (peso iniziale)
ρ_s = 2,7 Mg/m³ *
* = dato presupposto

Diametro equivalente (mm)	passante %
0,0406	96,3
0,0296	93,6
0,0205	88,4
0,0145	80,4
0,0094	72,5
0,0058	62,0
0,0036	50,4
0,0026	44,6
0,0015	38,8



Argilla(%) 42 Limo (%) 56 sabbia(%) 2 Ghiaia (%) 0

Classifica granulometrica (AGI - Raccomandazioni sulla programmazione ed esecuzione delle indagini geotecniche 1977)

limo con argilla

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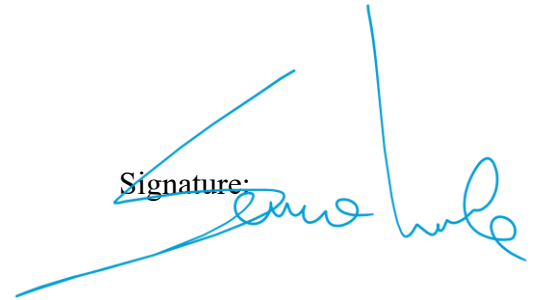
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Coordinator:

Prof. Ing. Luigi Berardi

Signature:

