Microzonification of Floor with Sliding in Nulti Parish

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Abstract. Cuenca is located on three areas controlled by mega faults and 14 areas at risk from landslides. One of them is the Nulti community that registers landslides in 16 subzones during the past 10 years. The waters presence on the ground is one of the causes of these earth movements, and this generated infrastructural and environmental losses. According to the update of the Development and Land Management plan, the Santa Cecilia Pasto Romero neighborhood is one of the affected areas. This community is located in the Parish Center. This place lost a considerable number of houses and equipment like educational, health, religious, administrative infrastructure, because it has landslides of great magnitude. Given these antecedents, we carried out a study that allows to know the behavior of this soil against seismic waves and identify risk areas, their properties such as soil type, allowable loads, and elasticity modulus. In this way, the risks presented by these landslides were determined from a soil profile by processing the results of the Geophysical analysis like seismic refraction tests, standard penetration, and electrical soundings. This study generates future lines of research like slope stabilization and drainage designs. This study opens the possibility of being replicated in the other parishes of Cuenca. The subsoil presents a cohesive material with high plasticity. The soil has deposits with silty clay matrices of high plasticity. This mass of soil has been removed by the effects of landslides and colluvial deposits. Therefore, its low shear wave velocities expose loose soils of high plasticity and medium to high expansivity.

1. Introduction

Soil instability are problems that occur worldwide. These phenomena are generated by different causes: morphological, geological, climatic characteristics and anthropic factors. Therefore, it is important to monitor these events, understand their behavior, the causes that generate it, and mitigate this problem. Cuenca is located on three areas controlled by mega faults and 14 areas at risk from landslides. One of them is the Nulti community that registers landslides in 16 subzones during the past 10 years. The waters presence on the ground is one of the causes of these earth movements, and this generated infrastructural and environmental losses. According to the update of the Development and Land Management plan, the Santa Cecilia Pasto Romero neighborhood is one of the affected areas [1].

Given the antecedents, the objective of this study is to know the behavior of the soil before seismic waves and identify with indirect methods, its physical and mechanical properties and if this area have risk. This methodology can be replicated in different areas that are considered vulnerable, if indirect
methods such as geophysical and direct methods such as geotechnical field and laboratory tests are applied. Additionally, the results obtained are correlated with the geology of the site.

In the same way, the ground shear wave velocities, the physical, geomechanical and elastic parameters of the materials were determined. These were correlated with the parameters calculated and inferred by the different methods. The bearing capacity of the soil in the area was determined from the geomechanical parameters and their correlation with geophysics.

This article presents the geology of the area, the methodology implemented for seismic refraction and standard penetration tests, the results obtained, and the conclusions and recommendations.

2. Geology of the zone

The area is made up of two geological units, Colluvial Deposits (Qc) that are formed by surface materials and the Loyola Formation (ML).

2.1. Colluvial Deposits (QC)

Deposits located on the slopes and in the lower zone of the mountain. These are heterogeneous materials that have had little transport. They can originate in various ways such as: glide, landslides, flows, etc. In general, they are composed of heterogeneous mixtures of blocks and angular-subangular fragments in a silty clay or sand matrix. The colluvial deposits are associated with latent active scarps, in the study area. This soil erodes and generates V-shaped drainage, as well as reactivations of the land in the presence of runoff. The permeability varies from medium to low depending on the compactness, and water tables can be deep [2].

2.2. Loyola Formation (ML)

Fine-grained clastic sedimentary formation. This has a variation of dark gray shales, siltstones that weather to white and cream clays, locally lenses of silty sandstones finely stratified with calcareous cement, limestones, and mega crevasse. In general, the massif is fragmented with multiple systems of stratification, fracturing and desiccation (on the surface). Fragmented fine granular Loyola sediments appear in undrained conditions and in deep rotational landslides both on low to medium slopes. On the surface predominates: desiccation, fragmentation and laminar erosion. This formation presents the highest number of landslides and the highest intensities [2]. Figure 1. presents the dominant geological formations in the study area: Nulti Parish. In Figure 2, the location of the land is presented.

![Figure 1. Dominant geological formations in the study area: Nulti Parish [3].](image)
3. Applied methodology

3.1. Standard Penetration Test (SPT)
In the present investigation, the SPT has been carried out as a dynamic penetration continuous test. An SPT equipment was used, which includes a "safety" type hammer. For this test, the energy value was adjusted to the standard value of 60%. Whereas, the diameter of the hole has been considered in a range that goes from 65 to 115 mm. Also, the sampler used was of a standard type. Additionally, laboratory tests were carried out from the samples obtained in this test [4], [5], [6].

Four trials were conducted in the study area. The first test was on the lower part of the property. This reached a depth of 6.45 m. The second test was on the middle - lower part, reached a depth of 5.45 m. The third test was carried out on the middle - upper part of the terrain, it reached a depth of 4.45 m. Finally, the fourth test was on the head of the field reached 3.45 m.

3.2. Geophysical Seismic Refraction Test
The objective of the Seismic Refraction test (MASW) is measuring the propagation speed of the terrestrial waves "P" (Vp) and "S" (Vs). This process is realized to indirectly determine the thickness and compactness of the study soil strata. This method can know the stratum up to a maximum depth of 48 m depending on the number of geophones that the team has, and 30 m from the MASW drillings. This depth also depends on the study area and the lines executed on the surface.

The basis of the seismic refraction test consists of the measurement of the travel times of the compression waves (P waves). These waves are generated by an impulsive energy source at points located at predetermined distances along an axis on the surface of the ground (geophones). The energy that propagates in the form of waves is detected, amplified and recorded in such a way that its arrival time at each point can be determined. The start of the recording is generated by a starting device that activates the data acquisition system at the moment of the impact or explosion. The difference between the arrival time and the zero time allows evaluating the propagation time of the waves from the energy source to the place where they are recorded [7].
The Multichannel Analysis of Surface Waves (MAWS) method was implemented for the study. The study was carried out with a Gea24 PASI seismograph, 24 vertical 4.5 Hz geophones, 16-pound hammer, GEOPSY software, GEA-PC software and other inputs, [8], [9], [10].

Four seismic lines were made in the study area. Each line has a scan length of 48 meters. The test generated dispersion curves, and consequently, after data processing, the wave velocities presented by the terrain. The dispersion curve of the surface waves was obtained and it was conjectured, from which the phase velocity curve (m/s) vs the frequency (Hz) was obtained. Finally, the GEOPSY software was used to invert the dispersion curve, in order to determine the velocity profile for the midpoint, of the geophonic arrangement [11], [12].

Figure 3. Seismic line carried out on the study soil in the Nulti parish, together with the velocity Vp and Vs.

3.2.1. Dynamic soil response
An equivalent homogeneous horizontal stratification is considered to determine the dominant period of the soil. This action is carried out because the study soil consists of a single layer of 28.5 m deep. The relationship between these parameters is presented in Equation 1, where $T_s$ is the dominant period of the equivalent stratum, $H_s$ the total thickness of the ground stratum and $V_s$ the effective speed of propagation of shear waves in the stratum [13].

$$T_s = \frac{4 + H_s}{V_s}$$  \hspace{1cm} (1)

4. Results and discussions
4.1. Geomechanical parameters obtained in geotechnical tests
In order to determine the physical and mechanical parameters, the following laboratory tests were performed with the samples obtained: water content [14], granulometry [15], liquid limit [16], plastic limit [17], SUCS method classification [18], direct shear [19], simple compression [20]. The results are presented in Table 1 and Table 2.
Table 1. Physical parameters of the samples analyzed in the laboratory.

| Test | Depth             | W (%) | % LL | % LP | % IP | % Gravel | % Sand | % Fine | SUCS | AASHTO |
|------|-------------------|-------|------|------|------|----------|--------|--------|------|---------|
| SPT - 1 | 2.00m to 2.45m | 26.86 | 77.99 | 33.05 | 44.94 | 0.00 | 8.19 | 91.81 | CH | A – 7 – 6 - (49) |
|       | 5.00m to 5.45m | 41.99 | 69.75 | 33.86 | 35.89 | 4.40 | 34.95 | 60.65 | CH | A – 7 – 6 - (21) |
|       | 6.00m to 6.45m | 23.02 | 77.21 | 30.38 | 46.84 | 0.00 | 10.05 | 89.95 | CH | A – 7 – 6 - (49) |
| SPT - 2 | 1.00m to 1.45m | 35.84 | 78.25 | 32.31 | 45.94 | 0.00 | 10.58 | 89.42 | CH | A – 7 – 6 - (48) |
| SPT - 3 | 4.00m to 4.45m | 37.68 | 73.18 | 37.56 | 35.62 | 0.00 | 11.00 | 89.00 | MH | A – 7 – 6 - (39) |
|       | 1.00m to 1.45m | 17.87 | 43.16 | 25.24 | 17.92 | 0.00 | 54.68 | 45.32 | SC | A – 7 – 5 - (5) |
| SPT - 4 | 4.00m to 4.45m | 27.11 | 45.33 | 32.32 | 13.01 | 0.00 | 57.17 | 42.83 | SM | A – 7 – 5 - (3) |
|       | 2.00m to 2.45m | 45.61 | 53.49 | 31.08 | 22.42 | 0.00 | 45.81 | 54.19 | MH | A – 7 – 6 - (10) |

Table 2. Geomechanical parameters of the samples analyzed in the laboratory.

| Laboratory test performed | Test Number | Type of sample | Material depth | Tested samples of the same specimen | % W | Average humidity of undisturbed sample | Wet density g/cm³ | Friction angle | Cohesion Kg/cm² |
|--------------------------|-------------|----------------|----------------|-------------------------------------|-----|----------------------------------------|------------------|---------------|----------------|
| Direct shear             | SPT – 1     | Altered, reshaped | 2.00m to 2.45m | 3 | 30.83 | 1.65 | 12 | 0.21 |
|                          | SPT – 2     | Unaltered, reshaped | 5.00m to 5.45m | 3 | 35.85 | 1.61 | 2.96 | 0.25 |
|                          | SPT – 3     | Altered, reshaped | 6.00m to 6.45m | 3 | 31.62 | 1.62 | 9.57 | 0.28 |
|                          | SPT – 4     | Altered, reshaped | 4.00m to 4.45m | 3 | 35.81 | 1.63 | 2.61 | 0.3  |
| Simple Compression       | SPT – 1     | Unaltered, reshaped | 2.00m to 2.45m | 1 | 38.31 | 1.79 | -   | 3.11 |
|                          | SPT – 2     | Unaltered, reshaped | 3.00m to 3.45m | 1 | 43.45 | 1.8  | -   | 1.84 |
|                          | SPT – 3     | Unaltered, reshaped | 3.00m to 3.45m | 1 | 23.18 | 1.82 | -   | 8.32 |

Table 3: Elastic parameters calculated as a function of wave velocities. Results of seismic tests.

| Seismic Line | Layers number | Soil stratum | Wave velocity | Specific weight | Allowable capacity | Shear modulus | Poisson coefficient | Young modulus | Oedometric modulus | Ball modulus | Ballast modulus | Dominant period |
|--------------|---------------|--------------|---------------|-----------------|-------------------|---------------|---------------------|----------------|-------------------|--------------|----------------|-----------------|
| LS - 1       | 1             | 0.0 - 1.5    | 280           | 172             | 13.09             | 56.29         | 39475.48            | 0.2            | 94503.07          | 31699.42    | 51979.25       | 32772.7         |
|              | 2             | 1.5 - 3.0    | 315           | 190             | 13.48             | 64.04         | 49690.61            | 0.2            | 120458.03         | 38990.21    | 70211.57       | 42080.88        |
| LS - 2       | 1             | 0.0 - 1.5    | 238           | 146             | 12.57             | 45.88         | 27310.58            | 0.2            | 65453.24          | 28194.54    | 36159.57       | 22710.92        |
|              | 2             | 1.5 - 3.0    | 260           | 160             | 12.85             | 51.4          | 33532.37            | 0.2            | 80158.34          | 26985.58    | 43836.59       | 27778.3         |
| LS - 3       | 1             | 0.0 - 1.5    | 250           | 152             | 12.72             | 48.35         | 29967.68            | 0.2            | 72328.33          | 23771.2     | 41110.44       | 25186.28        |
|              | 2             | 1.5 - 3.0    | 270           | 164             | 12.97             | 53.18         | 35563.92            | 0.2            | 85899.5           | 28178.09    | 48975.32       | 29923.78        |
| LS - 4       | 1             | 0.0 - 1.5    | 258           | 156             | 12.82             | 50.02         | 31815.23            | 0.2            | 77110.58          | 25075.18    | 44600.94       | 26911.32        |
|              | 2             | 1.5 - 3.0    | 270           | 164             | 12.97             | 53.18         | 35563.92            | 0.2            | 85899.5           | 28178.09    | 48975.32       | 29923.78        |

\[ T_s = 0.68 \]
4.2. Elastic parameters and layers determined by geophysical methods

In refractive seismic, the velocities of seismic waves depend on the elastic parameters and the density of the rocks [21]. Additionally, the results obtained from the longitudinal and shear waves allow the determination of different elastic parameters. Young's modulus, shear modulus, specific weight, Poisson's ratio, bearing capacity, Bulk's modulus, oedometric modulus, ballast modulus, were calculated.

In the city of Cuenca, for type "B" soils the dominant periods oscillate between 0.1 to 0.25 sec, for type C soils the dominant soil period is between 0.1 to 0.5 sec [22]. In the present study, a dominant period value of 0.68 sec was obtained.

4.3. Seismic hazard analysis of the study area

The value of the design seismic acceleration was determined from the seismic hazard map. This was prepared in 2011, and incorporated into the Ecuadorian Construction Code NEC 2015 [23]. The maximum acceleration in rock expected for the design earthquake (Z) is equal to 0.25g. This verification applies for a return period of 475 years, in zone II for the Azuay seismic zone and consequently, the zone under study. This value of 0.25 is the minimum recommended by the standard for the analyzed area. This value must be reduced according to the regulations of the minimum shear safety factor, being 60% of the acceleration, in the case of slope stability analysis.

4.3.1. Soil profile as a function of shear wave (Vs)

The shear wave velocity (Vs) was obtained from the geophysical tests. This was used to determine the type of profile based on the type of soil. The terrain oscillates between 150 m/s and 190 m/s. In addition, the results show materials with high humidity and plasticity indexes. These values are greater than 20. Therefore, in the area, the soil profile is type E, as shown in Table 4.

| Profile Type | Description | Definition |
|--------------|-------------|------------|
| A            | Competent rock profile | Vs ≥ 1500 m/s |
| B            | Medium stiffness rock profile | 1500 m/s > Vs ≥ 760 m/s |
| C            | Very dense soil profiles or soft rock, which meet the shear wave velocity criteria, or | 760 m/s > Vs ≥ 360 m/s |
|              | Very dense soil profiles or soft rock that meet either of the two criteria | N ≥ 50,0 Su ≥ 100 Kpa |
| D            | Rigid soil profiles that meet the shear wave velocity criteria | 360 m/s > Vs ≥ 180 m/s |
|              | Rigid floor profiles that meet either of the two conditions | 50,0 > N ≥ 15,0 100 Kpa > Su ≥ 50 Kpa |
| E            | Profile that meets the shear speed criteria, or Profile containing a total thickness H greater than 3m of soft clays | Vs < 180 m/s IP > 20 w ≥ 40% Su < 50 Kpa |
| F            | Soil profiles type F that require an evaluation carried out explicitly on site by a geotechnical engineer. | |

4.3.1. Elastic seismic spectrum of acceleration

The elastic seismic acceleration spectrum for the study soil is obtained from the soil profile and the maximum acceleration in rock expected for the design earthquake (Z). This spectrum was calculated for a rare and very rare earthquake [23]. These results are presented in Figure 4.
Figure 4. Elastic seismic spectrum of accelerations representing the design earthquake.

5. Conclusions and recommendations

5.1. Conclusions
The area is dominated by a heterogeneous deposit known as the Loyola Formation. This formation generates mass movements due to its unstable lithological characteristics. The geomechanical results corroborate this fact, since, the tests expose cohesion and low friction angles. If we add high humidity, the soil notably decreases its cohesion, as well as its resistance to shear. The physical and mechanical characteristics of the analyzed strata correspond to dominant fine soils, clays and silts of high plasticity. The material is generally working in a plastic range, not favorable for the transmission of loads.

The shear wave velocities $V_s$ was determined by refractive seismic calculations. These speeds range on average between 150 m/s and 190 m/s. The dominant period for a 30m stratum was calculated from these speeds. $T_s$ is equal to 0.68 s. The maximum acceleration calculated in the elastic seismic spectrum that represents the design earthquake for a rare earthquake and very rare earthquake is 0.87g and 0.58g respectively. This analysis is for soil type E.

The entire study area is highly vulnerable to mass movements because its topographic and geological conditions. The tests made it possible to qualitatively determine clay and silty soils as highly expansive. These characteristics indicate that the soil has global instability. Slope modeling, analysis of drainage systems, containment structures and expansive potential of the land, must be carried out before implementing any type of construction work.

5.2. Recommendations
The control of surface and deep runoff is recommended, as well as the implementation of drainage systems that allow to mitigate the movement of masses, control the expansion of the material and promote global stability of the area.

A stability analysis of the zone is recommended. This is done by modeling the slope in critical conditions such as saturated materials, with possible seismic events and loads on the headwater. This will allow taking into consideration possible mitigation measures to generate stability. Likewise, slope maps and risk maps must be generated. These will help define possible areas for the implantation of buildings within the zone.
One solution to the instability of the area is the control of crop irrigation of short-cycle species. Since, the entrance of the water to the subsoil by infiltration, generates increase of weight in the mass, increase of the pore pressure (impervious soils) and decrease of the resistance to the ground shear; and consequently, mass movements are generated. Additionally, revegetation systems should be implemented with species that help stability due to the effect of water absorption and the depth of their roots, in vulnerable areas such as the one in the present study.

It is advisable to reduce the weight to the mass, in certain areas. The general stability of the area improves if the materials transported by creeping effect remove. Another option is to improve the soil with materials such as hydrated lime. This chemically treatment transforms the clay particles, generating an impermeable soil with a higher load capacity. It is important to mention that these options represent a high financial cost.

A solution to the problem of foundations in cohesive soils is the point pile foundation system. This foundation transmits the load to harder strata, through its base. These systems can also be used as containment walls, since they absorb the shear forces of the ground, in the case of slope stability. It is proposed for future lines of research.

It is advisable to consider the soil and structure interaction. If the foundation soil has a soft material, it is possible that movements in the foundation can generate wear of the weak material, consequently affecting the building. Therefore, it is essential to replace the foundation soil material. It is proposed for future lines of research.

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