World Vision International Nepal
West Field Office, Dhangadhi

INVESTIGATION REPORT

Geotechnical Soil Investigation Services at the Finalized Sites of Multipurpose Safe Shelters in Kailali and Achham Districts

Project: Community Based Disaster Risk Reduction Project

March, 2024

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World Vision International Nepal
West Field Office, Dhangadhi

Final Report

For

Geotechnical Soil Investigation Services at the Finalized Sites of Multipurpose Safe Shelters in Kailali and Achham Districts

March, 2024

This document is the investigation report prepared for the project ***"Geotechnical Soil Investigation Services at the Finalized Sites of Multipurpose Safe Shelters in Kailali and Achham Districts"*** undertaken by World Vision International Nepal West Field Office, Dhangadhi. This document has been prepared by ***Geotechnical and Associates Pvt. Ltd***. The opinions, findings and conclusions expressed herein are those of the Consultant and do not necessarily reflect those of the World Vision International Nepal West Field Office, Dhangadhi.

Data Sources and Credits

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Authorized Representative

Geotechnical and Associates Pvt. Ltd.

Chyasal, Lalitpur

EXECUTIVE SUMMARY

This report presents the outcomes of a comprehensive Geo-Technical Investigation carried out by Geotechnical and Associates Pvt. Ltd. (GTAA) for the finalized sites of multipurpose safe shelters of in Kailali and Achham Districts, a Community-Based Disaster Risk Reduction (CBDRR) Project in Kailali and Achham districts.

The primary emphasis of the investigation lies in assessing the geotechnical conditions of the sites in which the CBDRR project is going to be implemented. Considering the WVI's commitments to the disaster resilience, thorough investigation including reconnaissance study, in-situ and laboratory testing and comprehensive analysis regarding subsurface conditions and liquefaction potential is performed.

Field investigations include either Standard Penetration Test (SPT) or Dynamic Cone Penetration Test (DCPT), coupled with a series of laboratory tests for soil. These tests encompass Moisture Content, Atterberg's Limits, Particle Size Distribution (Sieve Analysis), Hydrometer Analysis, Bulk Density (Wet & Dry Density), Direct Shear Test (Cohesion & Angle of Friction), and Unconfined Compression Strength Test. The objective is to offer a comprehensive understanding of soil properties, thereby facilitating a robust geotechnical assessment for the project.

The report provides detailed soil compositions from six boreholes (BH-01, BH-02, BH-03, BH-04, BH-05, and BH-06), revealing diverse compositions with varying proportions of silt, sand, and other soil properties in Achham and Kailali sites.

Upon analyzing the collected data from the field and laboratory study, the subsurface conditions in Achham site is found acceptable with medium dense to dense compacted soil for the implementation of proposed project. However, the Kailali site might be troublesome due to high probability of liquefaction in spite of being acceptable regarding bearing capacity.

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1. INTRODUCTION

General

This document has been prepared for the task titled "Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts" within the overarching project named "Community Based Disaster Risk Reduction Project." This aligns with the mutually agreed upon terms between the World Vision International, West Field Office, Dhangadhi (referred to as the 'Client'), and Geotechnical and Associates Pvt. Ltd. (hereafter referred to as 'Consultant'). The objective of this report is to present findings from geotechnical investigations and provide corresponding recommendations for the proposed project.

Figure 1 Location of the Borehole in the site

GTAA has prepared this report as per the reporting schedule for this current assignment. This report provides an overview for this assignment to be conducted for the successful completion of the Geotechnical Investigation and recommendation of the foundation.

Background

Nepal, nestled in the Himalayan region, is a country of remarkable geographic diversity, encompassing the flat plains of the Terai to the towering peaks of the Himalayas. This diverse topography, however, presents unique challenges to infrastructure development and environmental management, particularly given the prevalence of seismic activity and the influence of monsoonal rainfall. In this dynamic landscape, the focus on soil investigations takes center stage as a crucial element in ensuring the stability and sustainability of development projects.

The complex interplay between tectonic forces and diverse soil formations underscores the necessity for precise and comprehensive soil investigations. As Nepal ranks among the 20 most disaster-prone countries globally, understanding the geological intricacies becomes paramount for mitigating risks associated with slope instability and ensuring the resilience of critical infrastructure. Moreover, the seismic vulnerability of the region intensifies the significance of soil investigations in determining foundation designs that can withstand potential earthquakes. The Himalayan seismic belt, in which Nepal is situated, heightens the need for a nuanced understanding of the soil conditions to inform resilient construction practices.

Factors such as geological intricacies, seismic vulnerability, and the need for sustainable development converge to highlight the critical role that comprehensive soil investigations play in shaping the infrastructure landscape of the country.

The vulnerability of Nepal to various natural disasters, including earthquakes, floods, and landslides, has a profound impact on the country's infrastructure. Nepal's critical infrastructure, including buildings, roads, hydropower projects, and agricultural facilities, often faces the challenges of being constructed in demanding terrains. Soil investigation and foundation design take precedence in this initiative, given Nepal's rank as the seventh most earthquake-prone nation. The potential for building collapse during earthquakes, compounded by inadequate preparedness, underscores the critical need for informed interventions to ensure the safety and resilience of public structures. The proposed geotechnical soil investigations at Tikapur and Panchadewal Binayak sites play a crucial role in mitigating the impact of potential disasters on infrastructure. the seismic risks require careful attention to soil conditions and foundation design. These investigations are foundational, aiming to understand the terrain and inform the design parameters for constructing Multipurpose Safe Shelters capable of withstanding seismic events and other environmental challenges. Through comprehensive activities like boreholes, in-situ testing, and laboratory investigations, the project seeks to analyze the geological intricacies of the sites, deciphering factors influencing slope stability and foundation integrity. This understanding guides both the construction phase and the long-term viability of the infrastructure, aligning with the goal of constructing disaster-resilient infrastructure in Nepal's dynamic geological landscape.

General Description of Project Site

The Geotechnical Soil Investigation project is situated in the earthquake-prone regions of Kailali and Achham Districts in Nepal. These districts consistently face the threat of natural disasters, such as earthquakes, floods, and landslides. The initiative forms part of World Vision International Nepal's (WVIN) ongoing commitment to disaster resilience, specifically focusing on the construction of Multipurpose Safe Shelters to mitigate the impact of emergencies in vulnerable communities. The project locations, Tikapur and Panchadewal Binayak, have been identified for the construction of safe shelters under the Community-Based Disaster Risk Reduction Project.

Through geotechnical soil investigations at the designated sites, the project contributes to the overall objective of constructing safe shelters capable of withstanding potential disasters, ensuring the safety and well-being of communities in Kailali and Achham Districts.

The proposed Geotechnical Soil Investigation project focuses on sites in Kailali and Achham Districts, integral to World Vision International Nepal's Community-Based Disaster Risk Reduction (CBDRR) Project. The project is funded by the Ministry of Foreign Affairs, Japan, aiming to enhance disaster resilience in the region.

Kailali and Achham are among the most disaster-prone areas in Nepal, susceptible to earthquakes, floods, and landslides. World Vision International Nepal has been actively involved in constructing disaster-resilient buildings since the fiscal year 2015. The current initiative, under the CBDRR Project, targets the construction of Multipurpose Safe Shelters in Tikapur (28.452N, 81.019E) and Panchadewal Binayak (29.092365N ,81.504337E) to provide communities with emergency shelters during disasters as shown in Figure 1.

The urgency of this project is underscored by the fact that Nepal ranks seventh among nations at risk from earthquakes. The proposed safe shelters will not only serve as a protective haven during emergencies but also contribute to the overall disaster risk reduction efforts in the region. The funding from the Ministry of Foreign Affairs, Japan, highlights the international commitment to supporting initiatives that strengthen disaster resilience in vulnerable communities. The proposed geotechnical soil investigation is a crucial step in ensuring the structural integrity and safety of the Multipurpose Safe Shelters planned for construction in Kailali and Achham Districts.

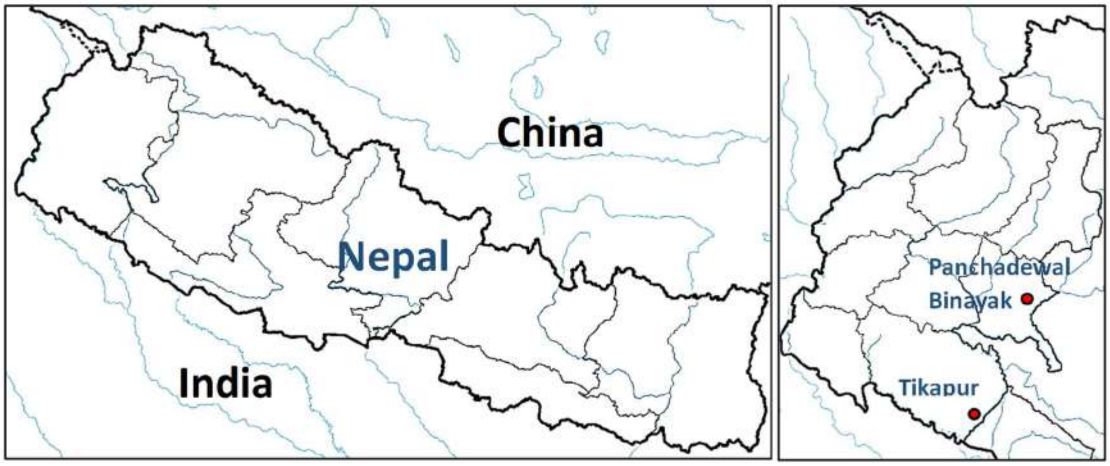


Figure 2. Map Showing the tentative locations of the site (Map Source: Wikipedia)

Objective of the Project

The overall objective is to investigate the soil conditions and the geotechnical properties of the finalized site for multipurpose safe shelter at Tikapur and Panchadewal Binayak.

The specific objectives are as follows: -

1. To do 3 boreholes each in each Site up to 12 meters.
2. To establish the points of the boreholes on sites.
3. To obtain undisturbed samples.
4. To test the soil sample at the laboratory.

Scope of work and investigation

The scope of the work is to study the subsurface conditions of the multipurpose safe shelter sites at proposed sites by conduct six boreholes with 12m depth each, including undisturbed soil sampling for finding soil parameter, and type of foundation.

To ensure the proper design and construction of the structure foundation, we require the following information:

1. Standard Penetration Tests (SPT) at 1.5m intervals
2. Collection of disturbed and undisturbed samples at regular intervals or as needed
3. Laboratory tests and interpretation of data to determine engineering properties
4. Creating Geotechnical Investigation report summarizing the investigation work

2. METHODOLOGY

2.1. Desk Study

The desk study for the Geotechnical Soil Investigation project involved a comprehensive data collection process, undertaken by a team comprising the Team Leader/Senior Geotechnical Engineer, Hydrologist, Geologist, and Surveyor. This study was vital for designing investigation planning and gaining insights into the geological and hydrological conditions at the project sites in Tikapur and Panchadewal Binayak.

In particular, it involved following tasks before field and laboratory testing:

1. Collection of Site Information and Past History:
2. Define Study Objectives
3. Literature Review
4. Review of Hydrological, Geological, Geomorphologic, Seismological Data
5. Collection of Previous Topographic and Geological Maps
6. Collection of Previous Geotechnical Data, if Any

Site Geology

As shown in ***Error! Reference source not found.***, the project area encompasses two distinct geological districts: Achham and Kailali, each characterized by unique geological features and compositions. Achham district is situated amidst significant tectonic boundaries, positioned between the Main Central Thrust (MCT) to the north and the Main Boundary Thrust (MBT) to the south. Nestled within the Lesser Himalaya geological zone of Nepal, Achham lies north of the Siwaliks (foothills) and south of the imposing Higher Himalayas, distinguished by its towering peaks. Comprising predominantly low-grade to medium-grade metamorphic rocks such as schists, phyllites, and quartzites, the Lesser Himalaya terrain in Achham exhibits a rugged topography shaped by geological processes over eons.

Conversely, Kailali district is situated within the Terai (Gangetic Plain) geological region, occupying the southwestern expanse of Nepal along its border with India. Unlike the undulating terrain of the Himalayan foothills or the towering ranges to the north, the Terai region is characterized by vast flat plains. These plains are primarily formed by the accumulation of unconsolidated alluvial sediments, including sand, silt, and clay, meticulously deposited by the intricate network of the Ganges River system over millennia. Renowned for its fertile agricultural land, the Terai offers a stark contrast to the rugged topography of the Himalayan regions. In summary, the project site spans two geologically distinct districts, Achham and Kailali, each offering a fascinating glimpse into Nepal's diverse geological landscape. While Achham showcases the rugged beauty of the Lesser Himalaya with its metamorphic rock formations, Kailali presents the flat plains of the Terai, enriched by the fertile alluvial deposits of the Ganges River system.

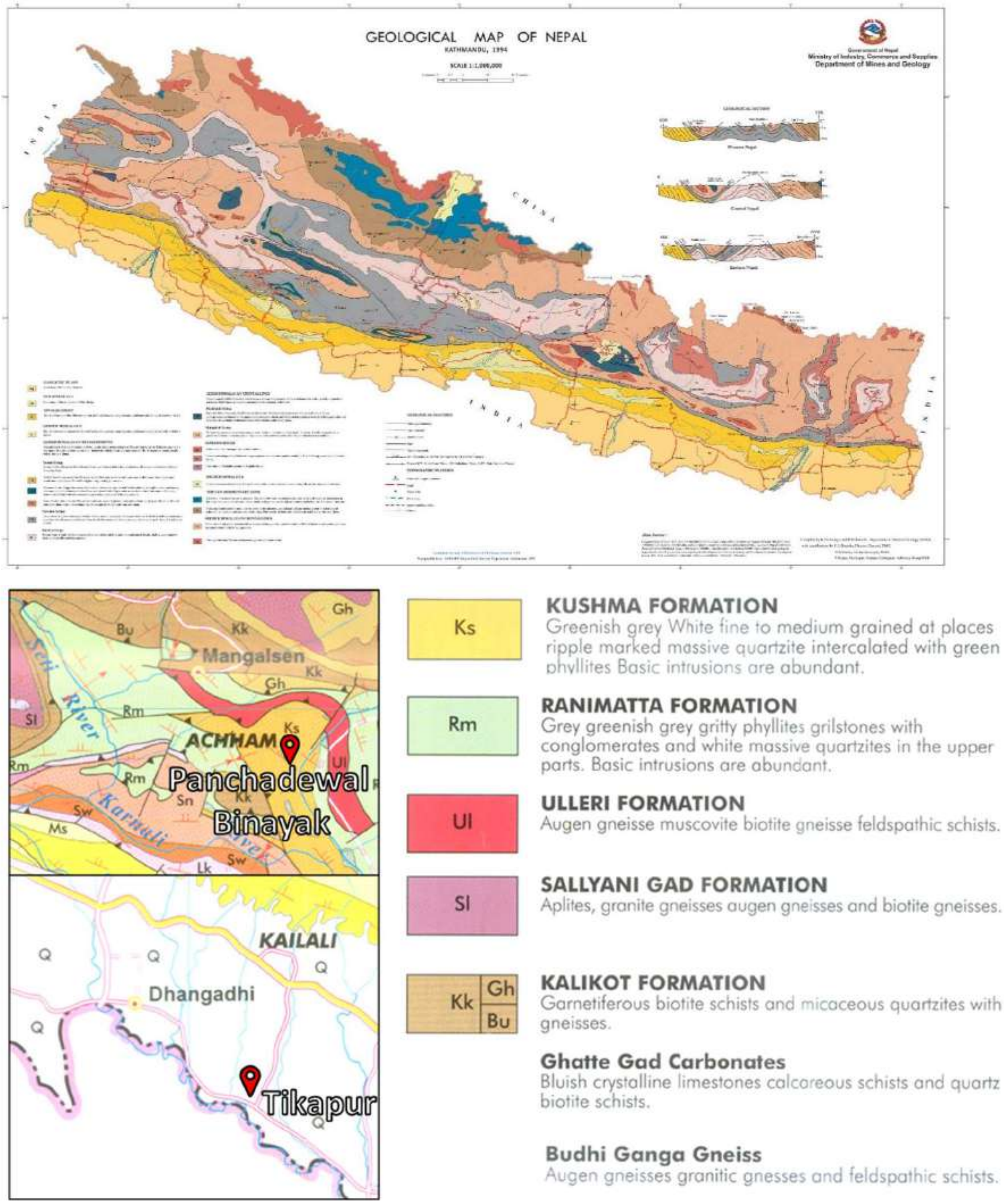


Figure 3 Geological map of Nepal ((Nepal, 1994))

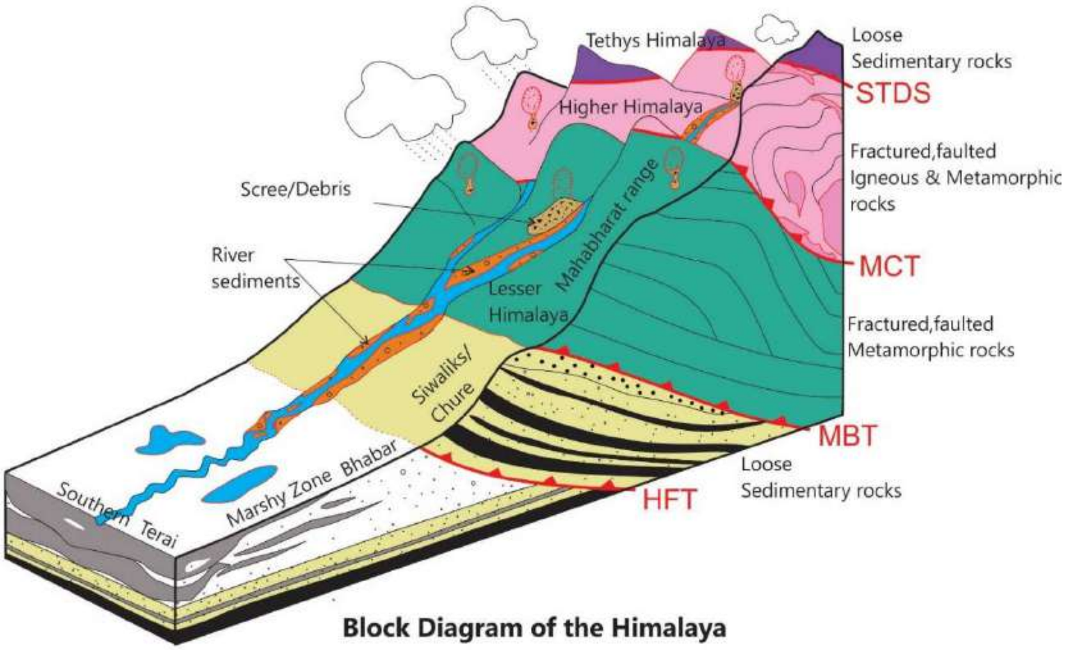


Figure 4 Block Diagram of Nepal Himalaya (Source: Department of Mines and Geology)

2.2. Ground Investigations

The proposed geo-technical investigation to understand the subsurface conditions at the site, assess the foundation soil's bearing capacity, and provide safe bearing capacity recommendations for various structure types. The field investigation took place on, February 21 to March 06, involving drilling with a rotary drilling machine. Boreholes were lined with 150 mm casing pipes to maintain stability. Experienced drillers, civil engineers and geotechnical engineers conducted all field explorations, logging, sampling, and testing, adhering to prescribed technical standards and specifications. At the specified location, borehole drilling was conducted using a Rotary Drilling Rig following the guidelines outlined in IS 2132: 1986. Soil samples were extracted for subsequent laboratory testing. The selection of borehole diameter ensured the production of satisfactory samples for laboratory examinations.

Water was utilized for sludge washing and as a coolant during the drilling process. Immediately after core retrieval, it was organized in core boxes and continuously logged in the field. The borehole logs included visual descriptions and classifications of soil/rock, along with records of SPT/DCPT tests, and other relevant information.

Standard Penetration Test (SPT)

Standard Penetration Tests (SPT) were conducted in the boreholes, with sampling at average depth intervals of 1.5 m. Using a split spoon sampler measuring 35 mm internally and 50 mm externally, equipped with a standard cutting shoe at the base, it was driven into the ground at the borehole's base. This was achieved with a hammer dropped from a height of 760 mm.

The sampler underwent an initial 150 mm seating penetration, followed by two additional 150 mm penetrations to reach the final depth. The sum of the blows required to reach the second-to-last 150 mm depth was recorded as the N-value. (Clayton, 1995)

There is a brief details of field tests that were conducted during this geotechnical investigation work:

- SPT/DCPT SPT/DCPT tests were carried out as per IS 2131-1981 (Reaffirmed 2002).
- A total of 48 Standard Penetration Tests (SPT) / Dynamic Cone Penetration Tests (DCPT) were conducted.
- These field tests provided insights into the soil's resistance to penetration.
- They helped determine the soil's relative density / Consistency, which were critical factors in foundation design.

Figure 5 Instrument setup for Standard Penetration Test

Sample collection

During the sample collection process, a meticulous approach was employed to ensure the integrity of both disturbed and undisturbed samples. Disturbed samples were primarily obtained using the split spoon barrel of the Standard Penetration Test (SPT) tube. Prior to sample collection, thorough cleaning of boreholes took place to eliminate any loose disturbed soil particles that may have accumulated during the boring operation.

For disturbed samples, the split spoon barrel was employed to capture representative soil sections during SPT tests. These disturbed samples were carefully preserved by placing them in air-tight double 0.5 mm thick transparent plastic bags. Each bag was appropriately labeled for identification purposes and then securely sealed to prevent any moisture loss. This meticulous handling aimed to maintain the sample's original condition until laboratory analysis. Undisturbed samples were collected using thin-walled tube samplers. The thin-walled tube sampler was selected for its ability to preserve the soil structure without significant disturbance. These undisturbed samples were similarly secured in air-tight, properly labeled plastic bags, and sealed for transportation to the laboratory.

The entire sampling process, comprising both disturbed and undisturbed samples, adhered to rigorous protocols to ensure the reliability of subsequent laboratory investigations. In total, this systematic approach resulted in the collection of a comprehensive set of samples for in-depth analysis, contributing to a thorough understanding of the subsurface conditions.

Table 1 Location Details of Boreholes drilled during Geotechnical Investigation

Borehole	Location	Co-ordinates
BH-01	Panchadewal-9-Chiltada, Achham	29.09133188°N,81.50596965°E
BH-02	Panchadewal-9-Chiltada, Achham	29.09136755°N,81.50610509°E
BH-03	Panchadewal-9-Chiltada, Achham	29.09133770°N,81.50622485°E
BH-04[a]	Tikapur-7-Kuntipur, Kailali	28.45228172°N,81.01905229°E
BH-05[b]	Tikapur-7-Kuntipur, Kailali	28.45241994°N,81.01898524°E
BH-06[c]	Tikapur-7-Kuntipur, Kailali	28.45247672°N,81.01912818°E

[a]First Borehole for Kailali Site [b]Second Borehole for Kailali Site [c]Third Borehole for Kailali Site

Analysis of In Situ Data

Both Standard Penetration Test (SPT) and Dynamic Cone Penetration Test (DCPT) were employed to determine the soil's strength parameters in its in-situ conditions. The N core values, namely (N1)60 and (N1)70, derived from SPT/DCPT tests, offer valuable insights into the consistency of cohesive soils and the relative density of cohesionless soils. In the analysis of in-situ data following borehole drilling, the Field N values were initially determined and subsequently adjusted for corrections, including Rog length correction and corrections for overburden and dilatancy effects. The resulting corrected N70 value serves as a crucial indicator of soil consistency. According to established standards, an N70 value falling within the range of 0-2 signifies very loose soil, while a range of 4-10 indicates loose soil. A value between 10-30 suggests medium dense soil, 30-50 signifies dense soil, and an N70 value exceeding 50 implies very dense soil. This classification provides valuable insights into the soil's engineering properties.

2.3. Laboratory Investigation

All the necessary laboratory tests were carried out in accordance with IS standard specifications. Standard laboratory test was carried out to characterize the soil strata. The laboratory test includes the following tests: Moisture Content, Grain Size Analysis, Specific Gravity, Atterberg Limits, Bulk & Dry Density test, Unconfined Compression Strength Test, Direct Shear Tests and Consolidation test.

Table 2 Summary of the test conducted

	Achham Site			Kailali Site		
Depth	BH-01	BH-02	BH-03	BH-04	BH-05	BH-06
0						
1.5	PDA, NMC, DST	PDA, NMC,BD, SPG	PDA, NMC,BD, SPG DST	NMC,BD, PDA, SPG	NMC,BD,ATL,PDA, HAD, SPG	NMC,BD, DST, SPG
3	PDA, NMC, DST	PDA, NMC, BD, SPG	PDA, NMC, BD, SPG	NMC,BD,ATL, PDA,UCS,HAD, SPG	NMC,BD,ATL,PDA, HAD, SPG	NMC,BD,ATL,UCS, SPG PDA, HAD DST
4.5	PDA, NMC,BD,DST, SPG	PDA, NMC,	PDA, NMC,BD, SPG DST	NMC,BD, PDA, SPG	NMC,BD,ATL,UCS, SPG PDA, HAD,	NMC,BD,ATL, PDA, HAD, SPG, CT
6	NMC,BD,PDA	PDA, NMC,	PDA, NMC, BD, SPG	NMC,BD,PDA, DST, SPG	NMC,BD,PDA, DST, SPG	NMC,BD,PDA,HAD, SPG DST,

		BD,DST , SPG				
7.5	PDA, NMC	PDA,NMC	PDA, NMC, BD, SPG	NMC,BD, PDA ,HAD ATL,	NMC,BD,DST, SPG PDA,	NMC,BD, SPG PDA, DST
9	NMC, ,PDA	PDA, , SPG NMC, BD	PDA, NMC, BD,DST, SPG	NMC,BD, PDA,HAD, SPG ATL,	NMC,BD,DST, SPG PDA,	NMC,BD, DST, SPG PDA,
10.5	PDA, NMC	PDA, NMC, BD, SPG	PDA, NMC,BD,DST, SPG	NMC, PDA,BD, SPG,DST,	NMC,BD, SPG PDA,	NMC,BD, SPG PDA,
12	NMC	NMC, BD, SPG	PAD,NMC, BD,DST, SPG	NMC, PDA,BD, SPG,DST,	NMC,BD, SPG DST,,PDA,	NMC,BD,DST, SPG PDA,

Table 3 Abbreviation for tests used in

Table 2

Test Type	Abbreviation
Moisture Content	NMC
Atterberg's Limits	ATL
Particle Size Distribution (Sieve Analysis)	PDA
Hydrometer Analysis	HAD
Bulk Density (Wet & Dry Density)	BD
Consolidation Test	**CT**
Specific Gracity	SPG
UCS/DST	UCS/DST

Specific gravity

The specific gravity test is conducted on soil samples passing a 2.0 mm IS sieve. Specific gravity is the ratio of the weight of soil particles in air to the weight of an equal volume of distilled water at 20 °C. It is crucial for computing various soil properties, including void ratio, unit weight, hydrometer-based particle size determination, and degree of saturation. This method utilizes a pycnometer for specific gravity

Figure 6 Samples are preparing for Specific gravity test

determination.

Grain size analysis

Grain size distribution analysis and hydrometer test were conducted to characterize the soil sample. Dry sieving involved using sieves with specified aperture sizes (e.g., 4.75mm to 75 microns) stacked from largest to smallest. The soil was shaken for 10 minutes, and the retained material on each sieve was weighed, expressed as a percentage of the total sample weight. Additionally, a hydrometer test was performed to assess the fine fraction of the soil. This involved dispersing a soil-water suspension, allowing sedimentation, and using a hydrometer to measure particle settling rates. The results from both tests provide comprehensive information about the soil's particle size distribution and help in understanding its engineering properties (Beuselinck, 1998).

Figure 7 Hydrometer analysis conducted at the GTAA Laboratory.

Atterberg limits

The physical properties of fine-grained soils, such as clay and silt, are influenced by their water content, resulting in liquid, plastic, or solid consistency states. The Atterberg Test assesses the consistency of cohesive soils. The Liquid Limit is the water content at which the soil flows to close a groove of standard width when jarred. The Plastic Limit is the water content where soil crumbles when rolled into 3mm threads. Three tests yield the average Plastic Limit. The Plasticity Index is the difference between Liquid Limit and Plastic Limit. Casagrande liquid limit device determines liquid limit, and standard rolling method determines plastic limit. The Casagrande plasticity chart classifies fine-grained soil per the Unified Soil Classification System. (Polidori, 2007)

Figure 8 Atterberg Liquid limit test at GTAA Lab

Bulk and Dry Density

Bulk density and dry density are important soil properties measured in the laboratory to assess soil compaction and engineering properties. Bulk density is the mass of soil per unit volume, including both solids and pores. Dry density is the mass of soil per unit volume when all moisture is removed. In the lab, bulk density is determined by dividing the mass of a soil sample by its total volume, including pores. Dry density is calculated by dividing the mass of the soil sample by its volume after removing all moisture.

Direct shear test

Soil shear strength, vital for determining foundation soil's safe bearing capacity, is its resistance to sliding along internal planes. Direct shear tests on disturbed samples from three boreholes were conducted. Samples, extracted and molded into 6.0 x 6.0 cm^2 cross-sectional areas, were trimmed to 2.5 cm high. Metal plates on both surfaces prevented pore water dissipation during shearing. Mechanically operated direct shear equipment applied shearing at a constant strain rate. Cohesive samples were sheared rapidly (tests under 10 minutes) to maintain undrained conditions. Shearing occurred at three normal stresses (50 kPa, 100 kPa, 150 kPa). Results, depicted as failure envelopes, provide the angle of internal friction (ϕ) and cohesion intercepts (c).

Unconfined Compressive Strength

The Unconfined Compressive Strength (UCS) test assesses the strength of cohesive soils in a laboratory setting. To conduct the test, prepare a saturated soil sample, trim it to desired dimensions, and place it in the unconfined compression apparatus. Record initial dimensions and weight. Apply axial load until failure, noting the maximum load and deformation. Calculate cross-sectional area, stress at failure, and Unconfined Compressive Strength (UCS). This data offers insights into soil stability, aiding geotechnical engineers in foundation design and soil classification. The UCS test is crucial for evaluating a soil's load-bearing capacity and overall engineering properties. (Nazir, 2013)

Figure 9 Unconfined Compressive strength test at GTAA

Consolidation Test

The consolidation test is a crucial geotechnical laboratory test used to determine the settlement behavior of soils under applied loads. It helps engineers understand how soils deform over time due to the expulsion of water from void spaces. The test is particularly important in predicting the settlement of structures such as buildings, dams, and roads.

The coefficient of compression (C_c) is a fundamental parameter used to quantify the compressibility of soils. It was determined from the consolidation test data, typically by fitting a mathematical model to the virgin compression curve. One common approach was to use Terzaghi's one-dimensional consolidation theory, which related the coefficient of consolidation (Cv), the coefficient of volume compressibility (m_v), and the coefficient of compression (Cc) through the equation:

$C_c = C_v * m_v$

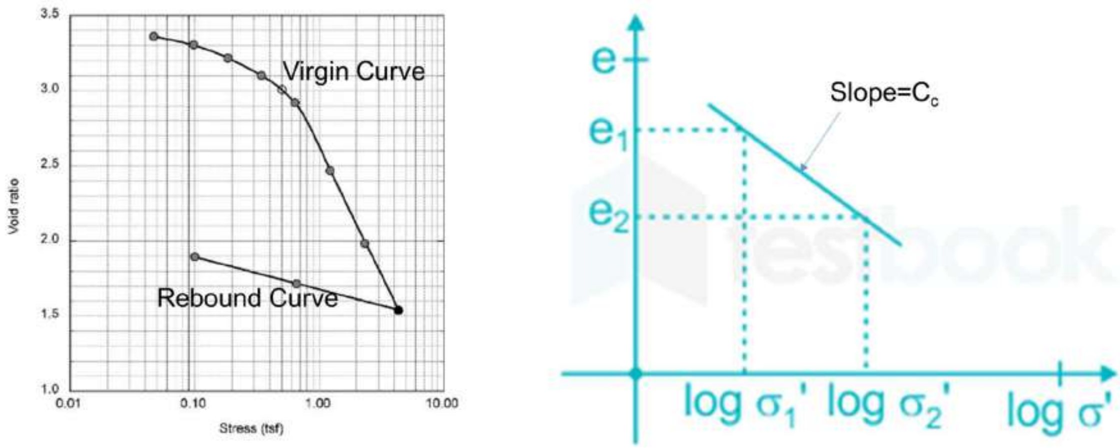


Figure 10 Consolidation Test and Determination of Cc

The coefficient of compression (Cc) was determined separately through the slope of the logarithm of the time versus settlement curve during the primary consolidation phase in this study.

2.4 Liquefaction Analysis

In Nepal, most of the geotechnical investigations are limited to standard penetration tests to a depth of 15 to 20 m, because other in-situ geotechnical investigations such as cone penetration test and shear wave velocity test have been sparsely used. The liquefaction potential assessment in the Kathmandu Valley has relied almost exclusively on SPT-N values and borehole data (Sharma et al. 2019).

A simplified method using SPT-N value suggested by Idriss and Boulanger (2008) was adopted to perform an analysis of the factor of safety (FS) with respect to liquefaction on each layer considering the earthquake scenario of M_w 8.0 with PGA of 0.40g. The scenario earthquake of M_w 8.0 with PGA of 0.40g was chosen based on the probabilistic seismic hazard studies that have been conducted for Kathmandu Valley considering seismic source zone models based on improved earthquake catalogs and modern ground-motion models (soil (Nepal National Building Code 105:2020 (NBC-105 2020). Additionally, the Iwasaki et al. (1982) method was adopted to calculate Liquefaction Potential Index (LPI) of the sites using FS against liquefaction on each layer.

In this method, the FS with respect to liquefaction can be calculated using following equation 1. The property of the soils to resist liquefaction is defined as the cyclic resistance ratio (CRR), and the stress (loading) that results in liquefaction is termed as the cyclic stress ratio (CSR).

$$FS = \frac{CRR_{7.5}}{CSR} MSF \qquad (1)$$

Where $CRR_{7.5}$ is the cyclic resistance ratio calibrated for an earthquake of magnitude 7.5. The $CRR_{7.5}$ can be modified using the magnitude scaling factor (*MSF)* for an earthquake having different magnitudes; MSF that accounts for the effects of the number of cycles during the earthquake or earthquake duration. The value of MSF for the considered scenario earthquake was calculated using Equation 2 (Idriss and Boulanger 2008):

$$MSF = 6.9e^{-\frac{M_w}{4}} - 0.058 \ \ (\leq 1.8) \qquad (2)$$

Equation 3 was used for determining the CRR for a cohesionless soil with any fines content.

$$CRR_{7.5} = \exp\left(\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right) \qquad (3)$$

where $(N_1)_{60cs}$ is an equivalent clean-sand *SPT* blow count. Following equations (Equations 4 and 5) are used to calculate $(N_1)_{60cs}$:

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \qquad (4)$$

$$\Delta(N_1)_{60} = \exp\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right) \qquad (5)$$

where $(N_1)_{60}$ is the corrected SPT-N value*; FC* is the fines content in the soils.

The measured *SPT-N* value was corrected using Equation 6:

$$(N_1)_{60} = NC_NC_EC_BC_RC_S \qquad (6)$$

where $(N_1)_{60}$ is the *SPT* blow count normalized to the atmospheric pressure of 100 kPa, and a hammer efficiency of 60%, *N* is the measured *SPT* blow count, and C_N, C_E, C_B, C_R, and C_S are the correction factors for the overburden stress, hammer energy ratio, borehole diameter, rod length and samplers with or without liners, respectively.

The *CSR* is calculated by Equation 7:

$$CSR = 0.65\frac{\tau_{max}}{\sigma'_{vc}} = 0.65\frac{\sigma_{vc}}{\sigma'_{vc}}\frac{a_{max}}{g} r_d \qquad (7)$$

where: τ_{max} is the earthquake-induced maximum shear stress, a_{max} is the peak horizontal acceleration at the ground surface, *g* is the gravitational acceleration, σ_{vc} and σ'_{vc} are the total overburden stress and effective overburden stress respectively, and r_d is the stress reduction coefficient given by Equation 8:

$$r_d = exp\left[-1.012 - 1.126sin\left(\frac{z}{11.73} + 5.133\right) + M_w\left(0.106 + 0.118sin\left(\frac{z}{11.28} + 5.142\right)\right)\right]$$
(8)

where: z is the depth of the soil layer in meter.

Liquefaction potential index (LPI)

The factor of safety against liquefaction at a given depth does not provide clear information on the severity of the potential ground deformation. For predicting the severity of liquefaction at a site through considering the soil profile in the top 20 m, the LPI was calculated using Equation 9 (Iwasaki et al. 1982):

$$LPI = \int_0^z F(z)W(z)dz \quad (9a)$$

$$F(z) = 1 - FS \qquad \text{For FS} < 1 \quad (9b)$$

$$F(z) = 0 \qquad \text{For FS} \geq 1 \quad (9c)$$

$$W(z) = 10 - 0.5z \qquad \text{For z} < 20 \quad (9d)$$

$$W(z) = 0 \qquad \text{For z} \geq 20 \quad (9e)$$

Based on the LPI value, liquefaction susceptibility of the site can be classified into four categories as (Table): Very Low, Low, High, and Very High (Iwasaki et al. 1982).

Table. Liquefaction potential classification (Iwasaki et al. 1982)

LPI	Susceptibility
0	Very low
0<LPI≤5	Low
5<LPI≤15	High
LPI > 15	Very high

2.5 Bearing Capacity Analysis

Analysis of allowable bearing pressure

The allowable bearing pressure (q_{all}) is the maximum pressure that can be imposed on the foundation soil taking into consideration the ultimate bearing capacity of the soil and the tolerable settlement of the structure. Analysis to determine the ultimate bearing capacity and the pressure corresponding to a specified maximum settlement were performed and the minimum pressure obtained from the two analyses were adopted as the allowable bearing pressure.

Allowable bearing pressure using strength parameter (c and Ø)

Since the soil in the vicinity of the foundation level has been found to be grayish color very dense gravel at greater depth, grey silty clay with high plasticity at intermediate depth, the allowable bearing capacity has been analyzed using the angle of friction and cohesion values from direct shear test results. Empirical formula of Terzaghi applicable for this type of soils has been used to obtain the allowable bearing pressure with safety factor equal to 3.

a. Terzaghi's Method:

$q_{ult} = cN_cs_c + qN_qW_q + 0.5\gamma BN_\gamma s_\gamma W_\gamma$

where,

$N_q = a^2 / a \text{ Cos}^2 (45 + \varphi/2)$, $a = e^{(0.75\pi-\varphi/2)\tan\varphi/2}$

$N_c = (N_q – 1) \text{ Cot}\varphi$

$N_\gamma = \tan\varphi / 2 * (K_{p\gamma} / \cos^2\varphi – 1)$

c. Effect of water table:

i) If water table is likely to permanently remains at or below a depth of (D_f + B) beneath the ground level surrounding the footing then $W_q = 1$.

ii) If the water table is located at depth D_f or likely to rise to the base of the footing or above then the value of W_q shall be taken as 0.5.

iii) If the water table is likely to permanently got located at depth $D_f<D_w<(D_f+B)$, then the value of W_q be obtained by linear interpolation.

3. RESULTS

Field Observations

During the field investigation using the Standard Penetration Test (SPT)/Dynamic Cone Penetration Test(DCPT), six boreholes (BH-01 to BH-06) were drilled in total in two sites, revealing distinct soil profiles. BH-01, BH-02 and BH-03 unveiled fine brownish silt mixed with Boulder. In overall, Dense compacted soil was obtained in Achham Site with no sign of water table up to 12m depth. BH-04,BH-05 and BH-06 displayed Brown colored silt mixed with clay up to 4.50m, transitioning into a white fine clayey fine silt until 9.50m. After 9.50m fine sand with some traces of gravel was encountered in Kailali site. Water table was encountered at depth 7.00m in BH-04 and 6.00m in both borehole BH-05 and BH-06.

Depth (m)	SPT (N-Value) Uncorrected					
	BH-01	BH-02	BH-03	BH-04	BH-05	BH-06
- 0.0	0	0	0	0	0	0
- 1.5	50	50	50	7	9	11
- 3.0	50	50	50	—	14	—
- 4.5	50	32	50	26	—	14
- 6.0	50	50	50	29	30	22
- 7.5	50	44	50	35	50	15
- 9.0	50	50	50	24	50	50
- 10.5	50	50	50	50	50	15
- 12.0	50	50	50	41	50	50

Laboratory Investigation Results

Achham Site

The predominant soil composition of borehole 1 was boulder. The data from borehole 2 and borehole 3 reveals dynamic variations in soil composition with depth. In borehole 2, at shallow depths (1.5m to 4.5m), the soil comprises sand, gravel, and a low proportion of silt and clay. Conversely, at greater depths (6m to 12m), gravel content notably increases, while sand and silt/clay proportions fluctuate. Similarly, at shallow depths (1.5m to 4.5m), it's primarily sandy with minor gravel and silt/clay. Deeper (6m to 12m), gravel content increases significantly, indicating a coarser texture. Overall, the soil is predominantly sandy, with variations in gravel and boulders content at different depths.

The moisture content varies across boreholes, ranging from 0.35% to 4.39% for borehole 1, 14.02% to 54.1% for borehole 2, and 15.6% to 58.08% for borehole 3.

The strength parameters show variations across boreholes, with borehole 1 exhibiting cohesion values 12.92kpa and friction 26.15°, borehole 2 with cohesion varying from 14.35 kPa to 16.94 kPa and friction angles from 24.67° to 25.66°, and borehole 3 displaying cohesion values ranging from 13.78 kPa to 20.09 kPa and friction angles from 21.71° to 25.66°.

The bulk density for boreholes 1 is 1.45 g/cm³, and its ranges for 2, and 3 are 1.52 g/cm³ to 1.93 g/cm³ and 1.49 g/cm³ to 1.6 g/cm³, respectively. Additionally, the specific gravity of borehole 1 is 2.66 and its ranges for 2, and 3 are 2.64 to 2.69, and 2.64 to 2.69, respectively. These variations reflect differences in soil compaction and mineral content across the boreholes, providing insights into soil density and composition for engineering assessments.

Kailali Site

The borehole data reveals dynamic variations in soil composition with depth across all sites. In borehole 1, the soil composition shifts from predominantly sandy with minor gravel at shallow depths (1.5m to 4.5m) to a heterogeneous mixture of sand, gravel, clay, and silt at deeper levels (6m to 12m). This indicates a gradation from sandy to mixed sandy and silty or clayey layers with increasing depth. Similarly, in borehole 2, the soil initially comprises a mix of clay, silt, and sand with higher proportions of slit & clay at shallow depths, transitioning to a predominantly sandy composition at deeper levels. Borehole 3 follows a similar pattern, with a mixture of sand, gravel, and slit & clay at shallow depths giving way to a predominantly sandy composition at greater depths, occasionally interspersed with gravel.

The moisture content varies across boreholes, ranging from 21.04% to 26.73% for borehole 1, 14.02% to 54.1% for borehole 2, and 15.6% to 58.08% for borehole 3.

The strength parameters show variations across boreholes, with borehole 1 exhibiting cohesion values ranging from 7.46 kPa to 10.3 kPa and friction angles from 19.04° to 22.75°, borehole 2 with cohesion varying from 8.66 kPa to 11.56 kPa and friction angles from 20.69° to 22.35°, and borehole 3 displaying cohesion values ranging from 6.5 kPa to 14.69 kPa and friction angles from 18.21° to 20.69°.

The bulk density ranges for boreholes 1, 2, and 3 are 1.68 g/cm³ to 1.82 g/cm³, 1.38 g/cm³ to 1.99 g/cm³, and 1.66 g/cm³ to 2.02 g/cm³, respectively. Additionally, the specific gravity ranges for boreholes 1, 2, and 3 are 2.59 to 2.65, 2.57 to 2.67, and 2.59 to 2.66, respectively. These variations reflect differences in soil compaction and mineral content across the boreholes, providing insights into soil density and composition for engineering assessments.

The unconfined compressive strength (UCS) values for the soil samples from boreholes 1, 2, and 3 are 124.25 kPa, 74.78 kPa, and 68.9 kPa, respectively, indicating varying levels of soil strength across the different boreholes.

The soil from boreholes 1, 2, and 3 exhibits varying degrees of plasticity across different depths. Borehole 1 shows consistent plasticity with LL ranging from 33.57 to 35.21, PL ranging from 22.99 to 26.99, and PI ranging from 7.39 to 8.22 at depths of 3m, 7.5m, and 9m. Borehole 2 also displays consistent plasticity with LL ranging from 32.59 to 34.39, PL ranging from 25.68 to 26.45, and PI ranging from 6.14 to 8.21 at depths of 1.5m, 3m, and 4.5m. Borehole 3 is characterized by medium plasticity, with LL ranging from 32.86 to 34.78, PL ranging from 26.96 to 27, and PI ranging from 5.9 to 7.78 across depths of 4.5m and 13m.

Table 4 Laboratory Test Result of Borehole 1 (Kailali).

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts

Location: Kuntipur-7-Tikapur, Kailali

Client: World Vision International

Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compression Index, Cc	UCS Test, Cu(KN/m^2)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/m^2)			LL	PL	PI	
	1.50	-	-	-	84.03	0.00	15.07	21.04	1.68	2.61	-	-	-	-	-	-	-	
	3.00	ML	1.30	68.28	30.42	0.00	69.58	22.99	1.72	2.60				124.25	33.57	22.99	7.58	Undisturbed Sample
	4.50	-	-	-	80.69	0.00	19.31	21.15	1.70	2.62	-	-		-	-	-	-	
	6.00	-	-	-	92.48	0.00	7.52	22.04	1.75	2.61	24.00	8.00		-	-	-	-	
1	7.50	ML	24.07	38.53	37.40	0.00	62.60	26.73	1.79	2.59	-	-		-	35.21	26.99	8.22	
	9.00	ML	1.52	63.58	34.90	0.00	65.10	21.29	1.73	2.61	-	-		-	35.09	25.71	7.39	
	10.50	-	-	-	80.46	0.00	19.54	24.24	1.82	2.64	28.00	10.00		-	-	-	-	
	12.00	-	-	-	82.96	0.00	17.04	24.24	1.75	2.65	28.00	12.00		-	-	-	-	

Table 5 Laboratory Test Result of Borehole 2(Kailali)

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts

Location: Kuntipur-7-Tikapur, Kailali

Client: World Vision International

Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compression Index, Cc	UCS Test, Cu(KN/m^2)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/m^2)			LL	PL	PI	
	1.50	ML	21.53	42.00	36.47	0.00	63.53	24.19	1.38	2.61	-	-	-	-	32.59	26.45	6.14	
	3.00	ML	1.69	70.19	27.63	0.00	71.87	14.02	1.68	2.57				-	34.39	26.19	8.21	
	4.50	ML	26.84	57.27	15.88	0.00	81.12	54.10	1.89	2.59	-	-		74.78	33.15	25.68	7.47	Undisturbed Sample
	6.00	-	-	-	90.41	0.00	9.35	21.04	1.87	2.63	25.00	13.00		-	-	-	-	
2	7.50	-	-	-	90.22	0.00	9.78	19.90	1.89	2.63	27.00	10.00		-	-	-	-	
	9.00	-	-	-	93.98	0.00	6.02	23.18	1.92	2.67	27.00	10.00		-	-	-	-	
	10.50	-	-	-	74.36	0.00	25.64	21.47	1.99	2.61	-	-		-	-	-	-	
	12.00	-	-	-	58.90	0.00	41.40	25.19	1.75	2.63	24.00	12.00		-	-	-	-	

Table 6 Laboratory Test Result of Borehole 3(Kailali)

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts
Location: Kuntipur-7-Tikapur, Kailali
Client: World Vision International
Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compression Index, Cc	UCS Test, Cu(KN/m²)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/m²)			LL	PL	PI	
	1.50	-	-	-	-	-	-	23.44	2.02	2.59	22.00	17.00	-	-	-	-	-	
	3.00	ML	-	-	19.31	0.00	80.69	58.08	1.66	2.60	25.00	10.00		68.90	32.86	26.96	5.90	UD
	4.50	ML	2.98	78.87	18.15	0.00	81.85	31.49	1.92	2.61	-	-	0.41	-	34.78	27.00	7.78	UD
	6.00	-	-	-	91.57	0.37	8.06	22.00	2.01	2.64	25.00	13.00		-	-	-	-	
	7.50	-	-	-	80.93	0.77	18.30	26.08	1.99	2.66	22.00	17.00		-	-	-	-	
3	9.00	-	-	-	64.01	0.00	35.99	23.49	1.92	2.63	25.00	7.00		-	-	-	-	
	10.50	-	-	-	25.62	57.80	17.30	15.60	1.75	2.62	-	-		-	-	-	-	
	12.00	-	-	-	43.46	0.00	56.54	27.14	1.93	2.62	25.00	10.00		-	-	-	-	

Table 7 Laboratory Test Result of Borehole 1(Achham)

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts

Location: Panchdewal Binayak Na.pa. 09 Chiltade (Achham)

Client: World Vision International Nepal

Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compre ssion Index, Cc	UCS Test, Cu(KN/ m^2)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/ m^2)			LL	PL	PI	
1	0.0-1.5	-	-	-	0.74	99.21	0.05	0.35	-	-	-	-	-	-	-	-	-	
	1.5-3.0	-	-	-	0.00	100.00	0.00	1.32	-	-	-	-		-	-	-	-	
	3.0-4.5	-	-	-	13.07	85.76	1.18	4.39	1.45	2.66	27.00	12.00		-	-	-	-	
	4.5-6.0	-	-	-	0.00	100.00	0.00	0.62	-	-	-	-		-	-	-	-	
	6.0-7.5	-	-	-	0.00	100.00	0.00	0.82	-	-	-	-		-	-	-	-	
	7.5-9.0	-	-	-	0.00	100.00	0.00	0.78	-	-	-	-		-	-	-	-	
	9.0-10.5	-	-	-	0.00	100.00	0.00	0.73	-	-	-	-		-	-	-	-	
	10.5-12.0	-	-	-	0.00	100.00	0.00	0.65	-	-	-	-		-	-	-	-	

Table 8 Laboratory Test Result of Borehole 2(Achham)

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts

Location: Panchdewal Binayak Na.pa. 09 Chiltade (Achham)

Client: World Vision International Nepal

Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compression Index, Cc	UCS Test, Cu(KN/m²)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/m^2)			LL	PL	PI	
2	0.0-1.5	-	-	-	65.38	29.51	5.12	13.70	1.61	2.65	-	-	-	-	-	-	-	
	1.5-3.0	-	-	-	39.99	50.85	9.16	12.52	1.56	2.65	25.00	16.00		-	-	-	-	
	3.0-4.5	-	-	-	36.14	61.45	2.40	8.90	1.52	2.64	26.00	16.00		-	-	-	-	
	4.5-6.0	-	-	-	0.00	100.00	0.00	0.63	-	-	-	-		-	-	-	-	Only Boulder
	6.0-7.5	-	-	-	45.83	49.53	4.64	15.45	1.60	2.69	26.00	16.00		-	-	-	-	
	7.5-9.0	-	-	-	0.00	100.00	0.00	0.61	-	-	-	-		-	-	-	-	Only Boulder
	9.0-10.5	-	-	-	54.26	41.27	4.47	13.91	1.93	2.65	26.00	14.00		-	-	-	-	
	10.5-12.0	-	-	-	55.32	36.32	8.36	16.26	1.64	2.66	26.00	15.00		-	-	-	-	

Table 9 Laboratory Test Result of Borehole 3 (Achham)

Lab Test Summary Sheet

Project : Multipurpose Safe Shelters at the Finalized Sites in Kailali and Achham Districts

Location: Panchdewal Binayak Na.pa. 09 Chiltade (Achham)

Client: World Vision International Nepal

Consultant: Geotechnical and Associates

Bore Hole - No:	Tested Depth,m	USCS Group Name	Particle Size Distribution					NMC(%)	Bulk Density (gm/cc)	Specific Gravity	Direct shear		Compre ssion Index, Cc	UCS Test, Cu(KN/m^2)	Atterberg Limits			Remarks
			Clay(%)	Silt(%)	Sand(%)	Gravel(%)	Slit & Clay(%)				ϕ	C (kN/m^2)			LL	PL	PI	
	0.0-1.5	-	-	-	93.37	0.36	6.27	10.00	1.52	2.65	26.00	15.00	-	-	-	-	-	
	1.5-3.0	-	-	-	85.34	7.88	6.79	12.94	1.56	2.64	24.00	16.00		-	-	-	-	
	3.0-4.5	-	-	-	89.61	2.56	7.83	10.78	1.54	2.66	-	-		-	-	-	-	
	4.5-6.0	-	-	-	24.60	73.84	1.56	7.40	1.49	2.66	26.00	13.00		-	-	-	-	
	6.0-7.5	-	-	-	53.97	41.42	4.61	16.40	1.60	2.68	-	-		-	-	-	-	
3	7.5-9.0	-	-	-	59.70	33.88	6.43	8.82	1.49	2.67	-	-		-	-	-	-	
	9.0-10.5	-	-	-	68.97	24.27	6.76	9.19	1.50	2.69	22.00	20.00		-	-	-	-	
	10.5-12.0	-	-	-	47.37	50.03	2.60	9.88	1.50	2.66	24.00	14.00		-	-	-	-	

4. DESIGN AND ANALYSIS

In the geotechnical site investigation process, the field's N-value is initially determined, which is then corrected for rod length and overburden influences. This corrected N-value is pivotal for subsequent calculations. Employing established methods, undrained cohesion and undrained friction angle are derived from the corrected N-value and soil types. These geotechnical parameters play a crucial role in assessing soil strength. Subsequently, a comparison is made between the calculated undrained cohesion and friction angle with the standard values. This comparative analysis informs about the soil stability and governs the decision-making process for foundation design. The precision in correcting N-values ensures a better foundation design, aligning with safety and performance standards.

4.1 Bearing Capacity Analysis

On the basis of ultimate bearing capacity and allowable settlement, the following allowable bearing pressures for shallow foundation have been recommended. Water table is assumed at ground considering the monsoon season. As the bearing capacity of soil depends on the size of footing and depth of footing, the exact bearing capacity of soil cannot be determined without know footing size and load on footing. The reported allowable bearing pressures are for typical shallow foundation size.

Table 10 Recommendation for Foundation with a Calculation Example.

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi Method

Identification Achham BH-01

INPUT

Units of Measurement
SI SI or E

Foundation Information
Shape SQ SQ, CI, CO, or RE
B = 2.13 m
L = 2.13 m
D = 1.52 m

Soil Information
c = 0 kPa
phi = 38 deg
gamma = 18 kN/m^3
Dw = 0 m

Factor of Safety
F = 3

Terzaghi Results

Bearing Capacity
q ult = 1,342 kPa
q a = 447 kPa

Terzaghi Computations

Unit conversion	1	a_θ =	4.86359
γ_w =	9.8	N_c =	77.50
ϕ (radians)	0.663225	N_q =	61.55
$W_{footing}$	163	N_γ =	82.28
γ conc	23.6	γ' =	8.2
		coefficient #1 =	1.3
		coefficient #3 =	0.4
		σ_{zD}' =	12.464

Sample Calculation (BH-01 Achham)

Soil Related Inputs:

C=0

Φ=38°

Υ_b=18 kN/m^2

Υ_{sub}= Υ_b- Υ_w=8.2kN/m^2

Surcharge,q= $\Upsilon_{sub} \times D_f$

=12.464

Assuming water table on ground (For worst case scenario)

D_w=0m

Foundation Related Inputs:

Assuming Square Footing, L=B= 7'=2.13m

Depth of Foundation, D_f=5'=1.52m

$$a = e^{\left(\frac{3\pi}{4} - \frac{\phi}{2}\right)\tan\phi}$$

=4.86

$$N_q = \frac{a^2}{2\cos^2\left(45^o + \frac{\phi}{2}\right)}$$

=61.55

$$N_c = \cot\phi\left[\frac{a^2}{2\cos^2\left(45^o + \frac{\phi}{2}\right)} - 1\right] = \cot\phi\left(N_q - 1\right)$$

=77.50

$$N_\gamma = \frac{1}{2}\tan\phi\left[\frac{k_P}{\cos^2\phi} - 1\right]$$

=82.28

Correction Factors:

α_1=1.3

α_2=0.4

General Equation for ultimate bearing capacity for any type of foundations is:

$$q_u = \alpha_1 c N_c + q N_q + \alpha_2 \gamma_B N_\gamma$$

From above data,

q_u=1342 kPa

Taking factor of safety=3

q_a=q_u/FoS

=447kPa

Achham				
BH-01				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Boulders	392	420	447
1.83m (6ft)	Boulders	445	472	499
2.13m (7ft)	Boulders	495	523	550
BH-02				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Boulders	392	420	447
1.83m (6ft)	Boulders	445	472	499
2.13m (7ft)	Boulders	495	523	550
BH-03				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Boulders	392	420	447
1.83m (6ft)	Boulders	445	472	499
2.13m (7ft)	Boulders	495	523	550

Kailali				
BH-01				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Sand	175	180	184
1.83m (6ft)	Sand	189	193	197
2.13m (7ft)	Sand	202	206	210
BH-02				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Sandy Silt	171	174	178
1.83m (6ft)	Sandy Silt	183	186	190
2.13m (7ft)	Sandy Silt	194	198	202
BH-03				
Depth (m)	Soil	Allowable Bearing Capacity (kPa)		
		Width (m)		
		1.52 m (5 ft)	1.83m (6ft)	2.13m (7ft)
1.52 m (5 ft)	Sandy Silt	170	173	177
1.83m (6ft)	Sandy Silt	182	185	189
2.13m (7ft)	Sandy Silt	193	197	201

Overall Recommendations:

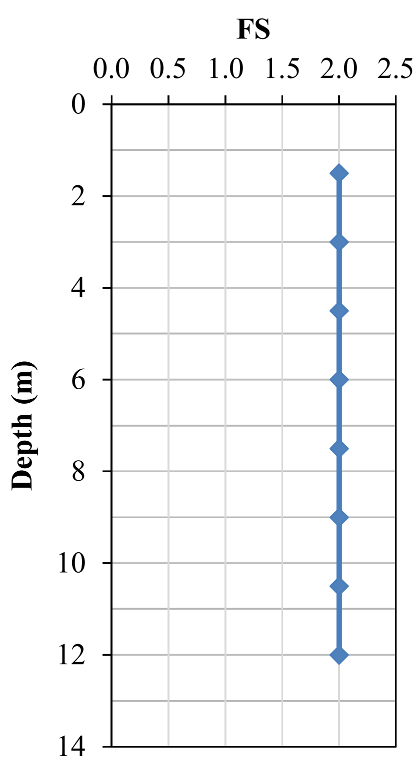
- The design should be correlated with the site conditions.
- Significant variations in soil type observed during excavation should be reported.
- The on-site geotechnical engineer should evaluate and confirm further action in response to observed change.

4.2 Liquefaction Analysis and its Impact on Foundation Recommendation.

The following figure shows typical liquefaction potential analysis based on SPT-N value and the results reveal very high probability of liquefaction. For detailed calculations, please refer to the annex section of this report.

Location:		BH-04	Kailali											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				7										
Average γ above water table (kN/m³):				17										
Average γ Below water table (kN/m³):				17										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	7	Silt	15	75	1.25	1	0.8	1	7	1.70	11.90	0.688	4.3337086
3	3	7	Silt	69	75	1.25	1	0.85	1	7	1.44	10.70	0.723	3.5303813
4	4.5	26	Sand	19	75	1.25	1	0.95	1	31	1.10	33.90	2	0
5	6	29	Sand	7	75	1.25	1	0.95	1	34	1.00	34.32	2	0
6	7.5	35	Silt	62	75	1.25	1	0.95	1	42	0.94	39.19	2	0
7	9	24	Sand	34	75	1.25	1	1	1	30	0.90	26.95	2	0
8	10.5	50	Sand	19	75	1.25	1	1	1	63	0.93	58.25	2	0
9	12	41	Sand	17	75	1.25	1	1	1	51	0.89	45.77	2	0
													LPI	**7.86**

Figure 11 Typical liquefaction potential analysis based on SPT-N value



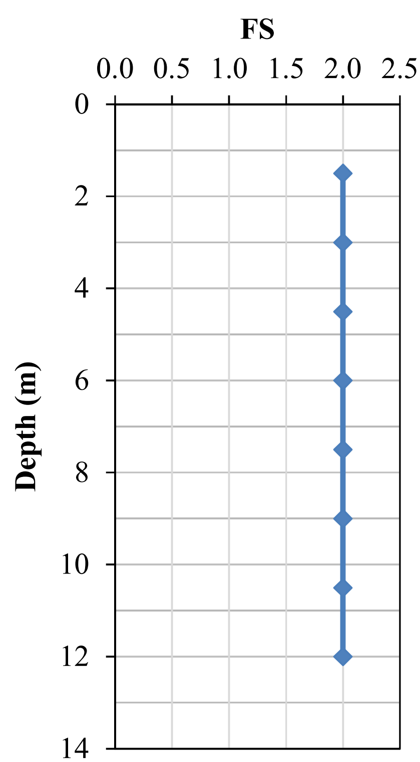


Figure 12 Factor of safety plots of BH-01 and BH-02

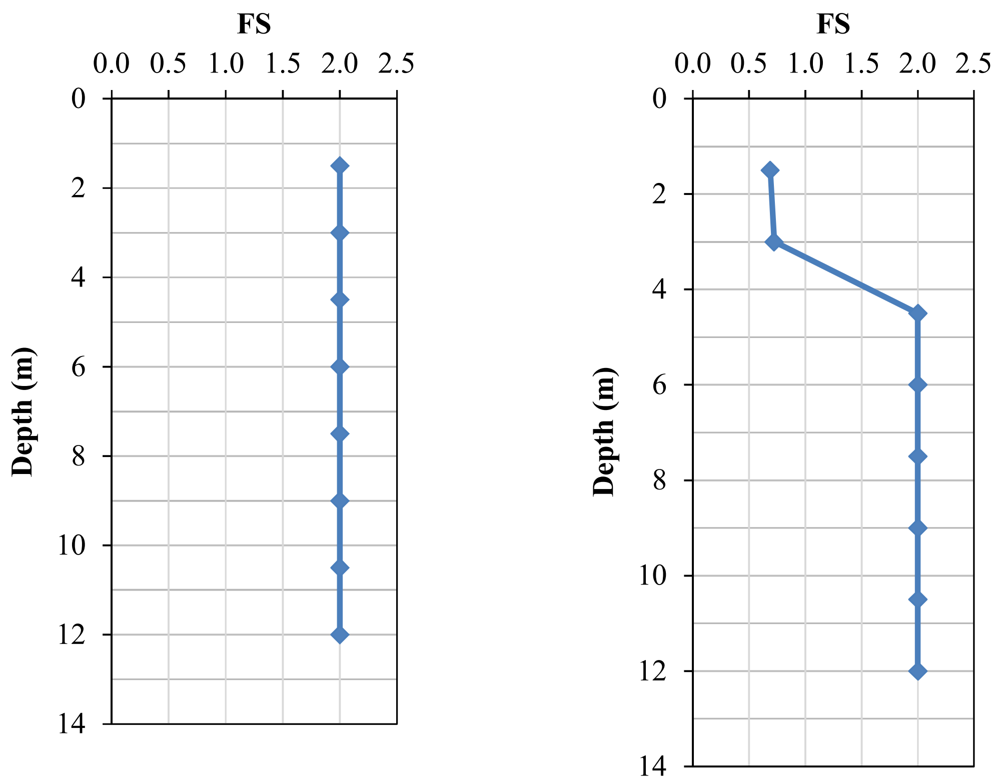


Figure 13 Factor of safety plots of BH-03 and BH-04

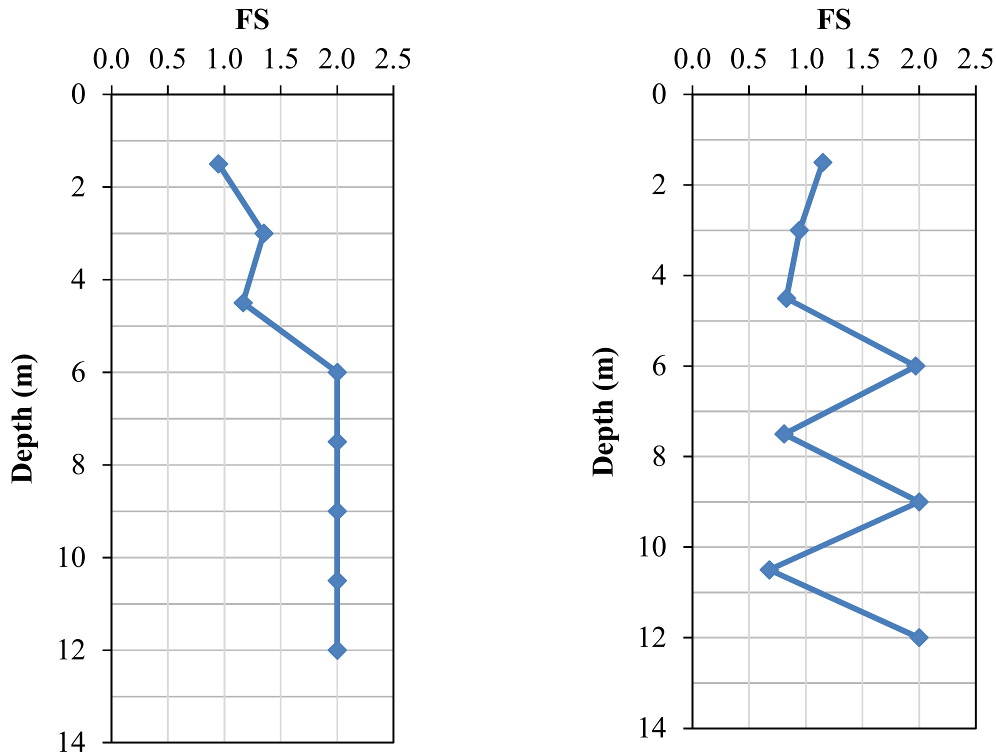


Figure 14 Factor of safety plots of BH-05 and BH-06

Based on the provided liquefaction potential indices (LPI) and the classification criteria by Iwasaki et al. (1982), here are the interpretations of the results for the six boreholes:

1. BH-01: LPI = 0
   - Susceptibility: Very low
   - Interpretation: Borehole BH-01 has a very low liquefaction potential, indicating minimal susceptibility to liquefaction under seismic loading.
2. BH-02: LPI = 0
   - Susceptibility: Very low
   - Interpretation: Borehole BH-02 also has a very low liquefaction potential, suggesting minimal susceptibility to liquefaction under seismic loading, similar to BH-01.
3. BH-03: LPI = 0
   - Susceptibility: Very low
   - Interpretation: Borehole BH-03 demonstrates a very low liquefaction potential, implying minimal

susceptibility to liquefaction under seismic loading, consistent with BH-01 and BH-02.

4. BH-04: LPI = 7.86
   - Susceptibility: High
   - Interpretation: Borehole BH-04 has a high liquefaction potential, indicating significant susceptibility to liquefaction under seismic loading.
5. BH-05: LPI = 0.71
   - Susceptibility: Low
   - Interpretation: Borehole BH-05 exhibits a low liquefaction potential, suggesting relatively low susceptibility to liquefaction under seismic loading.
6. BH-06: LPI = 6.76
   - Susceptibility: High
   - Interpretation: Borehole BH-06 demonstrates a high liquefaction potential, indicating significant susceptibility to liquefaction under seismic loading, albeit slightly lower than BH-04.

These interpretations provide insight into the varying degrees of liquefaction susceptibility across the study area. Boreholes BH-01, BH-02, and BH-03 have very low liquefaction potential, while BH-05 shows low potential. BH-04 and BH-06 exhibit high liquefaction potential, suggesting the need for careful consideration of mitigation measures in these areas to mitigate the risk of liquefaction-induced damage during seismic events.

5. CONCLUSION

Laboratory Investigation Results:

- Borehole-01(Achham):

Predominantly dense boulder with brown silt, moisture content 0.35% to 4.39%. Cohesion (c) and friction angle (φ) are 12.92KN/m^2 and 26.15° of sample in depth 3.0-4.5.

- Borehole 2 (Achham):

Composition of Sand and Gravel, moisture content 0.63% to 16.26%. Cohesion (c) and friction angle (φ) ranging from 14 kPa to 16 kPa and 25° to 26°, respectively.

- Borehole 3 (Achham):

Similar composition to Borehole-02, varying proportions of gravel, sand, silt, and clay, moisture content 7.4% to 16.40%. Cohesion (c) and friction angle (φ) from 14.64 kPa to 20.09 kPa and 21.71° to 25.66°, respectively.

- Borehole 4 (Kailali):

Varying composition of sand and silt, clay (1.52-24.07%), moisture content 21.04% to 26.73%. Cohesion (c) and friction angle (φ) ranging from 8.90 kPa to 12.34 kPa and 23.19° to 27.14°, respectively.

- Borehole 5 (Kailali):

Mainly silty soil with 1.69-26.84% clay and 15.88-93.98% sand, moisture content 14.02% to 54.10. Cohesion (c) and friction angle (φ) range from 10.91 kPa to 13.78 kPa and 23.68° to 26.64°, respectively.

- Borehole 6 (Kailali):

Silt and clay composition with some traces of gravel at about depth 10.5m, moisture content 15.60% to 58.08%. Strength parameters: cohesion (c) and friction angle (φ) varying from 10.33 kPa to 17.53 kPa and 21.71° to 24.67°, respectively.

Foundation Recommendations

- Kailali Site: Allowable Bearing Capacity of 170KPa with FOS 3 for the depth and width of foundation of 5 ft.
- Achham Site: Allowable Bearing Capacity of 392 KPa with FOS 3 for the depth and width of foundation of 5 ft.

Overall Recommendations regarding foundation:

- Design correlation with site conditions is essential.
- Significant soil type variations during excavation must be reported.
- On-site geotechnical engineers should evaluate and confirm further action in response to observed changes.

- Liquefaction Analysis Results

Based on the results of the liquefaction analysis of the six boreholes, it is evident that the study area presents a heterogeneous distribution of liquefaction susceptibility. Boreholes BH-01, BH-02, and BH-03 exhibit very low liquefaction potential, indicating minimal vulnerability to liquefaction under seismic loading. Conversely, BH-04 and BH-06 demonstrate high liquefaction potential, suggesting significant susceptibility to liquefaction-induced damage. BH-05 falls within the low liquefaction potential category, indicating a relatively lower risk compared to BH-04 and BH-06 but still requiring attention specially when constructing the isolated footing. Here are some suggestions:

- Increase Footing Size: Increase the size of the footing to distribute the load over a larger area.
- Deep Foundation: Consider using a deep foundation technique such as piles or drilled piers instead of an isolated footing.
- Ground Improvement Techniques: Implement ground improvement techniques such as soil compaction, vibro-compaction, or soil stabilization to increase the density and strength of the soil.
- Use of Geosynthetics: Employ geosynthetic materials like geotextiles or geogrids to enhance the stability of the soil and provide reinforcement against liquefaction-induced deformation.
- Grouting: Inject grout into the soil to fill voids and improve its stability. Grouting can help densify loose soils and increase their strength, reducing the risk of liquefaction.
- Monitoring and Inspection: Implement a monitoring and inspection program during and after construction to detect any signs of liquefaction-induced damage and take timely corrective measures if needed.

For possible liquefaction mitigation recommendation please refer to the annex A4 of this report.

6. REFERENCES

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7. ANNEXES

A1: Photographs

Figure 15 Drilling Process of BH-03 at Achham Site

Figure 16 SPT test at Borehole 03 in Kailali Site

Figure 17 Field Visited by the representative of World Vision International and WAC Nepal

Figure 18 Samples in Borehole-01 in Achham Site (a) 0-5m (b) 5-10m (c) 10-12m

Figure 19 Samples in Borehole-02 in Achham Site (a) 0-5m (b) 5-10m (c) 10-12m

Figure 20 Samples in Borehole-03 in Achham Site (a) 0-5m (b) 5-10m (c) 10-12m

Figure 21 SPT Samples in Borehole-01 in Kailali Site (a) 1.50m (b) 4.50m (c) 9.0m

Figure 22 SPT Samples in Borehole-02 in Kailali Site (a) 1.50m (b) 3.0m (c) 6.0m

Figure 23 SPT Samples in Borehole-03 in Kailali Site (a) 1.50m (b) 4.50m (c) 7.

A2: In-situ Borehole Log

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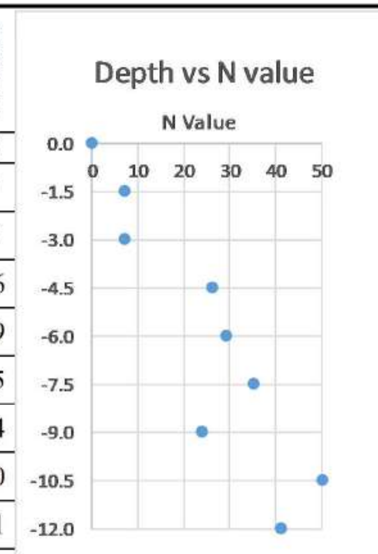
Drilling Log

Project:	Multipurpose Safe Shelter Project	**Date:**	11/19/2080
Location:	Kunitipur-7-Tikapur	**Drilling Method**	Core Drilling
GPS Coordinate:	29.091357N, 81.506116E	**Logged by:**	Santosh Pokharel
Client:	World Vision International, Nepal	**Ground water**	7.0m
Borehole No:	01	**Weather:**	**Sunny/** Rainy/ Normal

Soil Description	Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
From 0 to 0.5m: Light Brown Sandy Silt **From 0.5 to 2.20m**: Light Grey Loose Fine Sand **From 2.20 to 4.0m:** Dark Brown Clay Silt with Fine Sand **From 4.0 to 4.50m:** Light Brown Medium Dense medium to fine sand traces of few silt and clay mix **From 5.00 to 7.00:** White Grey Medium Dense medium to fine sand **From 7.00 to 8.00:** Dark Brown Hard Clay Silt mix few fine sand **From 8.00 to 9.50:** Dark Brown Medium Dense fine sandy silt **From 9.50 to 12.00:** Dark Gray Dense to Dense FIne Sand	0.0							0
	-1.5	SPT		3	4	3		7
	-3.0	UD		-	-	-		7
	-4.5	SPT		4	13	13		26
	-6.0	SPT		13	15	14		29
	-7.5	SPT		12	15	20		35
	-9.0	SPT		8	11	13		24
	-10.5	DCPT		20	35	45		50
	-12.0	SPT		15	18	23		41
End Depth					**Ground:**	Wet/ Dry		



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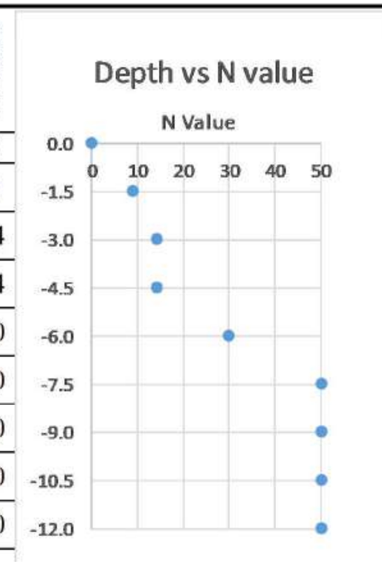
Drilling Log

Project:	Multipurpose Safe Shelter Project	**Date:**	11/20/2080
Location:	Kunitipur-7-Tikapur	**Drilling Method**	Core Drilling
GPS Coordinate:	28.45241994N, 81.01898524E	**Logged by:**	Santosh Pokharel
Client:	World Vision International, Nepal	**Ground wate**	6.0m
Borehole No:	02	**Weather:**	Sunny/ Rainy/ **Normal**

Soil Description	Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
From 0 to 0.3m: Light Brown Loose Sandy Silt	0.0							0
From 0.3 to 2.0m: Dark Brown Loose Fine Sand y Silt	-1.5	SPT		3	4	5		9
	-3.0	SPT		4	6	8		14
From 2.0 to 5.0m: Dark Brown medium Dense Fine Sandy Silt	-4.5	UD		-	-	-		14
From 5.0 to 8.50m: Dark Grey Dense medium to fine sand	-6.0	SPT		8	13	17		30
From 8.50 to 10.00: Dark Grey Dense to Dense fine sand with trances of few silt	-7.5	SPT		16	50/13cm	-		50
	-9.0	SPT		19	50/8cm	-		50
From 10.00 to 15.00: Dark Gray Dense to Dense FIne Sand	-10.5	DCPT		25	32	47		50
	-12.0	SPT		20	50/7cm	-		50
End Depth					Ground:	Wet/ **Dry**		



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Drilling Log

Project: Multipurpose Safe Shelter Project
Location: Kunitipur-7-Tikapur
GPS Coordinate: 28.45241994N, 81.01898524E
Client: World Vision International, Nepal
Borehole No: 03

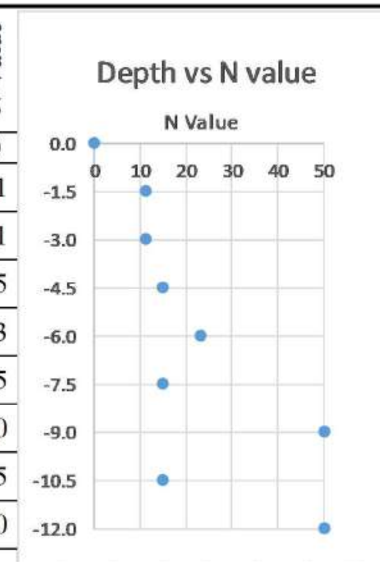
Date: 11/20/2080
Drilling Method Core Drilling
Logged by: Santosh Pokharel
Ground wate 6.0m
Weather: **Sunny**/ Rainy/ Normal

Soil Description

From 0 to 0.4m: Light Brown Loose Sandy Silt
From 0.4 to 2.0m: Light Brown Grey Medium Dense, Medium to Fine Sand, Traces od Few Silt, Few Clay Mix
From 2.0 to 3.8m: Dark Brown Clay Silt with Fine Sand mix
From 3.8 to 5.50m: Dark Brown Medium Dense fine sandy silt
From 5.50 to 8.50: Light Grey Medium Dense medium to fine sand
From 8.50 to 10.00: Light Grey Dense to Dense Fine Sand Taces of Silt
From 10.00 to 11.0: Light Grey Dense fine sandy silt with Small Boulder
From 11.0 to 12.00: Light Gray Dense to Dense FIne Sand,

Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
0.0							0
-1.5	SPT		3	4	7		11
-3.0	UD		-	-	-		11
-4.5	SPT		6	8	7		15
-6.0	SPT		4	12	11		23
-7.5	SPT		6	7	8		15
-9.0	SPT		21	26	38		50
-10.5	DCPT		4	7	8		15
-12.0	SPT		25	50/3cm	-		50
				Ground:	Wet/ Dry		

End Depth



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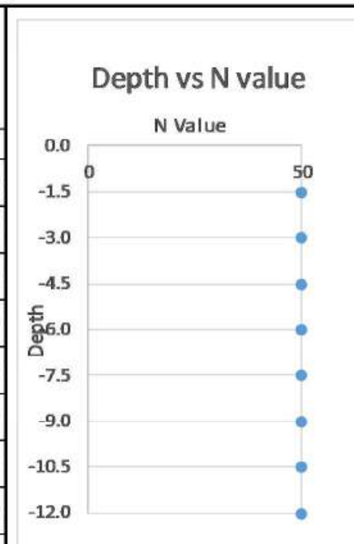
Drilling Log

Project:	Multipurpose Safe Shelter Project	**Date:**	11/14/2080
Location:	Chiltada, Panchadewal, Achham	**Drilling Method**	Core Drilling
GPS Coordinate:	29.0912764N, 81.5059986E	**Logged by:**	Santosh Pokharel
Client:	World Vision International, Nepal	**Ground water:**	Not Found
Borehole No:	01	**Weather:**	Sunny/ Rainy/ Normal

Soil Description	Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
From o.oom to 5.00m: Very Dense Boulders, Gravel and Sand. Mix Color Brown White From 5.00 to 12.00: Very Dense Boulders, Gravel Water Loss was encountered after 5.00m depth DCPT performed due to high presence of boulder	0.0							
	-1.5	DCPT		42	50/2cm	-		50
	-3.0	DCPT		38	50/8cm			50
	-4.5	DCPT		50/4cm	-	-		50
	-6.0	DCPT		50/2cm	-	-		50
	-7.5	DCPT		50/6cm	-	-		50
	-9.0	DCPT		50/8cm	-	-		50
	-10.5	DCPT		50/7cm	-	-		50
	-12.0	DCPT		50/9cm	-	-		50
End Depth					Ground:	Wet/ Dry		



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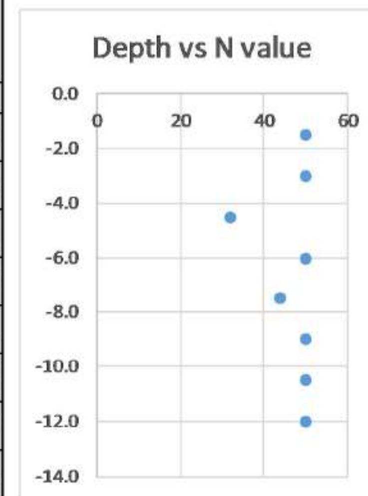
Drilling Log

Project:	Multipurpose Safe Shelter Project	**Date:**	11/15/2080
Location:	Chiltada, Panchadewal, Achham	**Drilling Method**	Core Drilling
GPS Coordinate:	29.091357N, 81.506116E	**Logged by:**	Santosh Pokharel
Client:	World Vision International, Nepal	**Ground wate**	Not Found
Borehole No:	02	**Weather:**	**Sunny**/ Rainy/ Normal

Soil Description	Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
From 0 to 4.5m: Brownish Silty Sand with Boulder From 4.5 to 7.5m: Gravel Boulder From 7.5m to 11.0m : Boulders, Gravel and mix color, Brown White Remark: At 4.5m SPT test conducted as relative soft strata was encountered but no sample obtained	0.0							
	-1.5	DCPT		43	50/7cm	-		50
	-3.0	DCPT		29	43	50/4cm		50
	-4.5	SPT		8	12	20		32
	-6.0	DCPT		27	44	50/2cm		50
	-7.5	DCPT		26	27	17		44
	-9.0	DCPT		37	50/7cm	-		50
	-10.5	DCPT		28	36	42		50
	-12.0	DCPT		36	43	50/6cm		50
End Depth					**Ground:**	Wet/ Dry		



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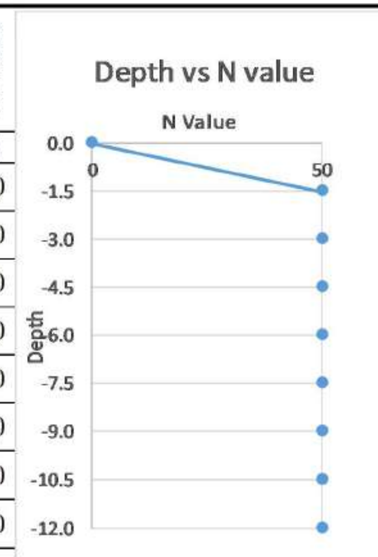
Drilling Log

Project: Multipurpose Safe Shelter Project
Location: Chiltada, Panchadewal, Achham
GPS Coordinate: 29.091337N, 81.506224E
Client: World Vision International, Nepal
Borehole No: 03

Date: 11/15/2080
Drilling Method Core Drilling
Logged by: Santosh Pokharel
Ground wate Not Found
Weather: **Sunny**/ Rainy/ Normal

Soil Description	Depth, m	SPT /DCPT/ UD	Recovery	No. of blows 10/15 cm	10/15 cm	10/15 cm	Nc-Value	N-Value
SPT performed at 3.0m depth. No sample obtained at split spoon sampler Soil Type: Brownish Silt with Boulder No Ground water table encountered during the drilling upto 12m	0.0							0
	-1.5	DCPT		30	35	50/7cm		50
	-3.0	SPT		50/14cm	-	-		50
	-4.5	DCPT		41	50/6cm	-		50
	-6.0	DCPT		50/5cm	-	-		50
	-7.5	DCPT		32	34	50/8cm		50
	-9.0	DCPT		42	45	50/9cm		50
	-10.5	DCPT		27	34	44		50
	-12.0	DCPT		38	40	50/9cm		50
End Depth					**Ground:**	Wet/ Dry		



A3: Liquefaction Data Analysis Sheets:

		N:	E:	BH
Location:				4
Input Parameter:				
Peak ground acc (g):	0.34			
Earthquake Magnitude, M:	8			
Water table depth (m):	19			
Average γ above water table (kN/m³):		19		
Average γ Below water table (kN/m³):		19		
Bore hole diameter (mm):		100		
Requires correction for sample liners (YES/NO):			NO	
Rod lengths assumed equal to the depth plus 1.5 m (for the above ground extension)				

SPT sample number	Depth (0)	Measure d N	Soil type (USCS)	Flag "Clay" "Unsaturated" "Unreliable"	Fines Content (%)	Energy Ratio	C_E	C_B	C_R	C_S	N_{60}	σ_{vc} (kPa)	σ_{vc}' (kPa)	C_N	$(N_1)_{60}$	ΔN for fines content	$(N_1)_{60\text{-}CS}$	Stress reduction Coeff, r_d	CSR	MSF for sand	K_σ	CRR for M=7.5 and σvc' =1 atm	CRR	FS	F(z)	W(z)	F(z)*W(z)*1.5
1	0	0				75	1.25	1	0.75	1	0	0	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
2	1.5	7	MH	Silt	88	75	1.25	1	0.8	1	7	28.5	28.5	1.70	11.90	5.5201	17.420	0.9983	0.2206	0.8758	1.100	0.17793	0.17142	**0.8**	0.22305	9.25	3.09
3	3	6	MH	Silt	91	75	1.25	1	0.85	1	6	57	57	1.37	8.77	5.5114	14.277	0.9895	0.2187	0.8758	1.062	0.15014	0.13961	**0.6**	0.36155	8.5	4.61
4	4.5	13	MH	Silt	91	75	1.25	1	0.95	1	15	85.5	85.5	1.08	16.70	5.5114	22.207	0.9792	0.2164	0.8758	1.024	0.23625	0.21192	**1.0**	0.02071	7.75	0.24
5	6	10	MH	Silt	91	75	1.25	1	0.95	1	12	114	114	0.94	11.14	5.5114	16.651	0.9676	0.2138	0.8758	0.986	0.17065	0.14733	**0.7**	0.311	7	3.27
6	7.5	28	MH	Silt	68	75	1.25	1	0.95	1	33	142.5	142.5	0.88	29.29	5.5808	34.871	0.9547	0.2110	0.8758	0.910	1.07920	0.86049	**2.0**	0	6.25	0.00
7	9	26	MH	Silt	68	75	1.25	1	1	1	33	171	171	0.81	26.48	5.5808	32.065	0.9408	0.2079	0.8758	0.882	0.65077	0.50267	**2.0**	0	5.5	0.00
8	10.5	23	MH	Silt	91	75	1.25	1	1	1	29	199.5	199.5	0.75	21.48	5.5114	26.995	0.9259	0.2046	0.8758	0.880	0.34633	0.26679	**1.3**	0	4.75	0.00
9	12	33	MH	Silt	91	75	1.25	1	1	1	41	228	228	0.75	30.83	5.5114	36.342	0.9101	0.2011	0.8758	0.769	1.49161	1.00483	**2.0**	0	4	0.00
10	13.5	50	MH	Silt	91	75	1.25	1	1	1	63	256.5	256.5	0.80	49.91	5.5114	55.417	0.8938	0.1975	0.8758	0.725	2.00000	1.26991	**2.0**	0	3.25	0.00
11	15	50	ML	Silt	91	75	1.25	1	1	1	63	285	285	0.77	48.18	5.5114	53.687	0.8769	0.1938	0.8758	0.694	2.00000	1.21545	**2.0**	0	2.5	0.00
12	16.5	41	ML	Silt	87	75	1.25	1	1	1	51	313.5	313.5	0.69	35.38	5.5230	40.902	0.8596	0.1900	0.8758	0.666	2.00000	1.16619	**2.0**	0	1.75	0.00
13	18	50	ML	Silt	87	75	1.25	1	1	1	63	342	342	0.72	45.04	5.5230	50.565	0.8422	0.1861	0.8758	0.640	2.00000	1.12121	**2.0**	0	1	0.00
14	19.5	48	MH	Silt	89	75	1.25	1	1	1	60	370.5	365.595	0.69	41.29	5.5172	46.808	0.8248	0.1847	0.8758	0.620	2.00000	1.08673	**2.0**	0	0.25	0.00
																											11.21

Location:		BH-01	Achham											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				30										
Average γ above water table (kN/m^3):				21										
Average γ Below water table (kN/m^3):				21										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	50	Boulder	10	75	1.25	1	0.8	1	50	1.23	61.71	2	0.00
3	3	50	Boulder	10	75	1.25	1	0.85	1	53.13	1.10	58.32	2	0.00
4	4.5	50	Boulder	10	75	1.25	1	0.95	1	59.38	1.01	60.12	2	0.00
5	6	50	Boulder	10	75	1.25	1	0.95	1	59.38	0.96	56.74	2	0.00
6	7.5	50	Boulder	10	75	1.25	1	0.95	1	59.38	0.91	53.83	2	0.00
7	9	50	Boulder	10	75	1.25	1	1	1	62.5	0.87	54.56	2	0.00
8	10.5	50	Boulder	10	75	1.25	1	1	1	62.5	0.84	52.28	2	0.00
9	12	50	Boulder	10	75	1.25	1	1	1	62.5	0.80	50.19	2	0.00
													LPI	**0.00**

Location:		BH-02	Achham											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				30										
Average γ above water table (kN/m^3):				22										
Average γ Below water table (kN/m^3):				22										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	50	Boulder	10	75	1.25	1	0.8	1	50	1.23	61.33	2	0
3	3	50	Boulder	10	75	1.25	1	0.85	1	53	1.09	57.84	2	0
4	4.5	32	Boulder	10	75	1.25	1	0.95	1	38	1.01	38.24	2	0
5	6	50	Boulder	10	75	1.25	1	0.95	1	59	0.95	56.15	2	0
6	7.5	50	Boulder	10	75	1.25	1	0.95	1	59	0.90	53.20	2	0
7	9	50	Boulder	10	75	1.25	1	1	1	63	0.86	53.89	2	0
8	10.5	50	Boulder	10	75	1.25	1	1	1	63	0.83	51.56	2	0
9	12	50	Boulder	10	75	1.25	1	1	1	63	0.79	49.44	2	0
													LPI	**0.00**

Location:		BH-03	Achham											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				30										
Average γ above water table (kN/m^3):				22										
Average γ Below water table (kN/m^3):				22										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	50	Boulder	10	75	1.25	1	0.8	1	50	1.23	61.33	2	0
3	3	50	Boulder	10	75	1.25	1	0.85	1	53	1.09	57.84	2	0
4	4.5	50	Boulder	10	75	1.25	1	0.95	1	59	1.00	59.60	2	0
5	6	50	Boulder	10	75	1.25	1	0.95	1	59	0.95	56.15	2	0
6	7.5	50	Boulder	10	75	1.25	1	0.95	1	59	0.90	53.20	2	0
7	9	50	Boulder	10	75	1.25	1	1	1	63	0.86	53.89	2	0
8	10.5	50	Boulder	10	75	1.25	1	1	1	63	0.83	51.56	2	0
9	12	50	Boulder	10	75	1.25	1	1	1	63	0.79	49.44	2	0
													LPI	**0.00**

Location:	BH-04		Kailali											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				7										
Average γ above water table (kN/m^3):				17										
Average γ Below water table (kN/m^3):				17										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	7	Silt	15	75	1.25	1	0.8	1	7	1.70	11.90	0.688	4.3337086
3	3	7	Silt	69	75	1.25	1	0.85	1	7	1.44	10.70	0.723	3.5303813
4	4.5	26	Sand	19	75	1.25	1	0.95	1	31	1.10	33.90	2	0
5	6	29	Sand	7	75	1.25	1	0.95	1	34	1.00	34.32	2	0
6	7.5	35	Silt	62	75	1.25	1	0.95	1	42	0.94	39.19	2	0
7	9	24	Sand	34	75	1.25	1	1	1	30	0.90	26.95	2	0
8	10.5	50	Sand	19	75	1.25	1	1	1	63	0.93	58.25	2	0
9	12	41	Sand	17	75	1.25	1	1	1	51	0.89	45.77	2	0
													LPI	**7.86**

Location:	BH-05		Kailali											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				6										
Average γ above water table (kN/m^3):				17										
Average γ Below water table (kN/m^3):				17										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	9	Silt	63	75	1.25	1	0.8	1	9	1.70	15.30	0.949	0.7122173
3	3	14	Silt	71	75	1.25	1	0.85	1	15	1.35	20.09	1.351	0
4	4.5	14	Silt	84	75	1.25	1	0.95	1	17	1.13	18.84	1.166	0
5	6	30	Sand	9	75	1.25	1	0.95	1	36	1.00	35.51	2	0
6	7.5	50	Sand	9	75	1.25	1	0.95	1	59	0.98	58.09	2	0
7	9	50	Sand	6	75	1.25	1	1	1	63	0.96	60.17	2	0
8	10.5	50	Sand	25	75	1.25	1	1	1	63	0.95	59.14	2	0
9	12	50	Sand	41	75	1.25	1	1	1	63	0.93	58.16	2	0
													LPI	**0.71**

Location:	BH-06		Kailali											
Peak ground acc (g):				0.34										
Earthquake Magnitude, M:				8										
Water table depth (m):				6										
Average γ above water table (kN/m^3):				19										
Average γ Below water table (kN/m^3):				19										
SPT sample number	Depth (m)	Measured N	Soil type	Fines (%)	Energy Ratio	CE	CB	CR	CS	N60	CN	(N1)60	FS	F(z)*W(z)*1.5
1	0	0	0	0	75	1.25	1	0.75	1	0	-	-	-	-
2	1.5	11	Silt	25	75	1.25	1	0.8	1	11	1.70	18.70	1.151	0
3	3	11	Silt	45	75	1.25	1	0.85	1	12	1.32	15.40	0.946	0.6906443
4	4.5	15	Sand	8	75	1.25	1	0.95	1	18	1.08	19.19	0.83	1.9802012
5	6	23	Sand	18	75	1.25	1	0.95	1	27	0.95	26.05	1.972	0
6	7.5	15	Sand	35	75	1.25	1	0.95	1	18	0.89	15.92	0.808	1.7973864
7	9	50	Sand	17	75	1.25	1	1	1	63	0.94	58.48	2	0
8	10.5	15	Silt	56	75	1.25	1	1	1	19	0.81	15.22	0.679	2.2905898
9	12	50	Silt	81	75	1.25	1	1	1	63	0.90	56.13	2	0
													LPI	**6.76**

A4: Strategies for Mitigating Liquefaction Risks and Remedial Action Protocols

In Section 4.2, it is evident that there is a notable probability of liquefaction occurring at the Kailali site. To mitigate this risk, interventions aimed at altering the soil conditions can be pursued. One viable approach involves enhancing the soil's resistance to liquefaction through methods such as deep compaction or ground reinforcement. By implementing these techniques, the threshold for triggering liquefaction can be raised, thereby reducing the likelihood of its occurrence.

Another strategy entails modifying the subsurface conditions to fundamentally alter the behavior of the ground. For instance, soil cement stabilization can be employed to prevent liquefaction even under the most severe levels of earthquake shaking. Given the unique characteristics of the Kailali site, both of these approaches hold significant promise in mitigating the liquefaction hazard effectively.

To summarize, the following two approaches are particularly relevant for addressing liquefaction risks at the Kailali site:

1. Reduction of Liquefaction Potential through Ground Improvement Strategies
2. Enhancing Structural Resilience through Liquefaction-Resilient Design

Approach 1: Mitigating Liquefaction Potential with Ground Improvement Technique

Considering the soil conditions prevalent at the Kailali site, characterized by a high presence of silty soil with limited cohesion and plasticity, two ground improvement methods are recommended:

1. **Deep Compaction with Vibratory Probe:**
   This method involves the densification of soil through the application of vibration, inducing dynamic loading that leads to seismic compaction. It offers a moderate-cost solution and can achieve relative density levels of up to 80%.
2. **Deep Compaction with Heavy Tamping (Dynamic Consolidation):**
   This approach applies high-intensity impacts at the surface, making it particularly suitable for soils lacking cohesion, similar to those found at the Kailali site. It also presents a moderately costly option, making it feasible for implementation in the planned project at the site.

It is essential to ensure that the compaction reaches a level where the minimum field values of N(corrected) of 15 are attained at the foundation level, as per IS 1893:2016 Part 1 standards.

Approach 2: Building the Liquefaction Resilient Structure

Another alternative for addressing the liquefaction issue observed at the site involves altering the consequences of potential ground movement. Shallow foundations, susceptible to differential ground settlement and weak soil, can be mitigated by providing a robust and rigid building platform. This approach aims to minimize the potential for strength loss and the formation of sand boils beneath footings, while also facilitating the smoother distribution of ground settlement across the building footprint.

In this regard, a Mat foundation emerges as a suitable option. The preliminary dimensions of the Mat foundation are as follows:

Length (L): 51 feet

Breadth (B): 43 feet

Thickness (t): 8 feet 6 inches

Implementing a Mat foundation of these dimensions can enhance the structural stability and resilience against liquefaction-induced ground movement at the site.

Remark: Above Recommendations are made based on the preliminary investigation and detail study should be conducted prior adopting the suitable approach.