

# VARIATION OF COEFFICIENT OF CONSOLIDATION OF KAMERO, KALO AND RATO MATO OF NEPAL

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**Abstract**— Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water, the process continuing until the excess pore water pressure set up by an increase in total stress has completely dissipated. The application of load to a soil creates a state of excess pore pressure. These excess pore pressures dissipate by the gradual movement of water through the voids of the soil, and this results in a volume change that is time dependent. A soil experiencing such a volume change is said to be consolidating, and the vertical component of the volume change is called consolidation settlement. Laboratory test show that the coefficient of consolidation is varied during the loading and consolidation process. It is known that consolidation process is accompanied by decrease in void ratio which leads to decrease in the coefficient of permeability. The importance of the decrease of the coefficient of consolidation on the time rate of settlement and pore water pressure needs to be investigated. Undisturbed and remolded samples were collected from three different sites. The samples thus collected were preserved and tested in CMTL and Geotech and Associates lab. The lab test includes (Index properties test, Organic content test, Consolidation test and XRD tests). The soils under study are major deposit in Kathmandu, Kaski and Nawalparasi districts, so due to booming construction it is necessary to study its changing consolidation and permeability characteristics and thus predict its settlement rate and life of structure. For kamero and kalo mato it is seen significant change in coefficient of consolidation which is 47.5% and 53% respectively within a period of 3 months at constant consolidation pressure of 57.586 ton/m<sup>2</sup>. Similarly for rato mato the change is 71.79% in a period of 9 months. Hence, it can be concluded that the effect of decrease in void ratio at later time of consolidation leads to decrease in coefficient of consolidation. Variation in  $C_v$  misleads in predicting actual settlement of the structure. A detailed study is must to find change in  $C_v$  of different soil at different consolidation pressure.

**Keywords**—Consolidation, Permeability, XRD

## I. INTRODUCTION

Soil is a natural body of minerals and organic matter occurring on the surface of the earth that is a medium for plant growth and has ever-changing properties in response to many physical, chemical and biological processes going on over a geological period. Most of the soil deposit in Kathmandu valley comprises sand and clay with organic content. Clay is a finely grained natural rock or soil material that combines one or more clay minerals. It is made of a number of fine-grained, earthy materials that become plastic when wet. Chemically, clays are hydrous aluminum silicates, usually containing minor amounts of impurities such as potassium, sodium, calcium, magnesium, or iron. Black soil, widely known as *Kalomato* in Nepali language is one of major soil deposits of Kathmandu valley. It exhibits high rate of swelling and shrinkage when exposed to changes in moisture content and hence has been found to be very troublesome from engineering consideration. The rate of montmorillonite is more in black cotton soil which causes expansiveness and crack in soil without any warning signs, and is therefore dangerous for construction. The reddish residual soil, widely known as *Ratomato* in Nepali language, contains appreciable amount of iron oxide and is often used as a construction material. The white clayey soil commonly known as *Kameromato* is a major soil deposit in Kaski district of Nepal and is also the area of study.

The soils of Nepal are highly variable and are derived mainly from young parent material. The nature and composition of soil depends on the nature and composition of the parent rock, prevailing climatic conditions, topography or physiographic condition, the vegetation it supports and soil formation process. So far, various types of soil found in Nepal are alluvial soil, lacustrine soil, rocky soil and mountain soil. Alluvial soil is usually found in the terai region and in lowland river basins. This alluvial soil is very fertile in nature because it is formed by the materials deposited by rivers. The sandy and gravel soil is found in churiya region where gravel and conglomerate are predominantly found. This is not fertile soil. Lacustrine soil is found in the Kathmandu valley. The mountain soil is formed by

boulders, sands and stone carried down by glacier. It is also not fertile [1]

*Kalomato* soil found in Kathmandu valley is formed by lacustrine deposit. Because of its high swelling and shrinkage characteristics, *Kalomato* soil has been a challenge to the engineers since long. It is very hard when dry, but loses its strength completely as humidity penetrates the soil. *Kalomato* soil is made up of varying properties of minerals like Montmorillonite and kaolinite, chemicals like Iron Oxide and Calcium Carbonate and organic matter like humus. [2] *Kameromato* soil found in Ghattekhola, Kaski is also a residual deposit. Very few researches are done till date on the *Kameromato*, so its detailed geotechnical investigation is yet to be carried out. Chemical compositions of *Kameromato* samples of *Kamerotar* area of Bhaktapur primarily include clay minerals having MgO and K<sub>2</sub>O with SiO<sub>2</sub> and Al<sub>2</sub>O<sub>3</sub>. On the other hand, the amount of Fe<sub>2</sub>O<sub>3</sub>, Na<sub>2</sub>O and CaO in fine clay sample is found to be lower than in bulk. [3] Red soil is a tropically weathered soil with a high concentration of sesquioxides of iron and/or alumina. It has correspondingly low content of alkalis and alkaline earths. It exists in wide range of chemical composition. Silica content varies from low to medium and exists usually as kaolinite, whenever it is found in substantial amount. The presence of iron oxides in various states of hydration gives red soil a range of colors. [4]

Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of some of the pore water, the process continuing until the excess pore water pressure set up by an increase in total stress has completely dissipated. Consolidation theory is required for predicting both magnitude and rate of consolidation settlements to ensure serviceability of structures founded on a compressible soil layer. Terzaghi's theory of consolidation makes certain assumptions about the consolidation process. He asserts that soil is homogeneous and fully saturated, solid particles and pore water are incompressible, flow of water and compression of soil are one dimensional (vertical), and strains are small. Darcy's law is valid at all hydraulic gradients, but the most important assumption is that the coefficient of permeability and the coefficient of volume compressibility remain constant throughout the consolidation process. It is known that consolidation process is accompanied by decrease in void ratio which leads to decrease in coefficient of permeability. Effect of decrease in coefficient of permeability on time rate of settlement and pore water pressure needs to be investigated. The settlement rate and pore water pressure dissipation rate are mainly controlled by permeability of soil. Both laboratory and field tests show that permeability is varied during the loading and consolidation process. The variation of  $C_v$  with pressure has not received much attention. Review of some available data in past shows  $C_v$  is not constant but varies with consolidation pressure. An examination shows that  $C_v$  increases with consolidation pressure for

kaolinite but decreases with consolidation pressure for montmorillonite. This inconsistency in the variation of  $C_v$  with consolidation pressure for different soils needs reconciliation.

This paper takes into account the change in coefficient of permeability with varying consolidation on three different fine grained soils of Nepal.

#### A. Problems and issues

- Life of different structures in Kathmandu valley and other places of Nepal built over fine grained soil are usually underestimated with constant coefficient of consolidation. Decrease in coefficient of consolidation with time decreases the settlement rate and increases the life of structures.
- Decrease in void ratio and subsequent decrease in the coefficient of permeability followed by slower dissipation of pore water pressure accompanies consolidation. This, in turn, makes it difficult to predict bearing capacity of soil.
- Different types of soil have different settlement patterns which largely depend on varying coefficient of consolidation. Hence comparative study of different types of fine grained soil is a must.

## II. METHODOLOGY

#### A. Soil Sampling Technique

Open excavation were carried out. Undisturbed and disturbed samples were collected in thin walled sampler tubes. Sampling tubes were sealed at both ends by using wax and plastic to preserve moisture content. All the sample tubes were kept in a container and covered with wet jute bag followed by a plastic sheet to preserve the natural state of the samples.

Location of sample collection:

*Kalomato* - Chardobato, Bhaktapur Building site

*Ratomato* - Chormara, Nawalparasi

*Kameromato* - Ghatte Khola, Kaski



Fig. 1. Soil Sampling

#### B. Soil Testing Method

Soil testing included various testing methods to determine index, properties and consolidation

characteristics of soil. Coefficient of consolidation was determined from consolidation test.

#### a) Consolidation Test

This test was performed to determine magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures.



Fig. 2. Consolidometer

Undisturbed Samples were extruded from sampling tube and cut with trimming tools to a cylindrical shape with 5.9 cm diameter and 2 cm height. Specimen was weighted and recorded, which was further used for determination of water content of the soil. Three different samples of undisturbed specimen were prepared and tested using consolidation test.



Fig. 3. Undisturbed soil sample

Extensive consolidation test procedure was deployed. Firstly, soil specimen was collected using consolidation metal ring. The ring was confirmed to be clean and dry and its weight, inner diameter and height were measured using weighing balance and calipers respectively. The metal ring was then pressed into the soil sample using hand and it was then taken out with soil specimen. As the soil specimen should project about 10 mm on either side of metal ring, excess soil content was trimmed on top and bottom of the rings using knife, spatula and fine metal wires. This excess soil was later used to measure the water content of soil sample.

For measuring the water content of the soil sample, firstly it was confirmed that the ring did not contain any soil on its outer part and weight of the metal ring with soil specimen was recorded. Then two porous stones were taken and saturated by boiling them for 15 minutes and submerging between 4 to 8 hours in distilled water. Consolidometer was then assembled by placing the parts of consolidometer from bottom to top in the following order: bottom porous stone, filter paper, specimen ring, filter paper and top porous stone. Loading pad was placed on top of the porous stone and the consolidometer was eventually locked using metal screws provided.

The whole assembly was mounted on a loading frame and centered such that the load applied was axial. Then initial trail load was applied which did not allow any swelling in the soil. In general, 5 kn/m<sup>2</sup> initial load was applied. The load was left until there was no change in dial gauge reading for at least 24 hours and final reading of dial gauge for initial load was recorded.

First load increment of double load was applied and stop watch was started immediately and readings from the dial gauge were recorded at various time intervals. In general, readings were taken at 0.25, 1, 2.5, 4, 6.25, 9, 12.25, 16, 25, 30 minutes, 1, 2, 4, 8, 24 hours.

In general, primary consolidation of soil (90% of consolidation) was reached within 24 hours. Hence, readings were recorded up to 24 hours.



Fig. 4. Dial gauge

Second load increment which was double the first load was applied and the same procedure was repeated as mentioned for the first load. Similarly double load increments was applied till 7 days and the same procedure was repeated. After the seventh day, increment of load was stopped and the reading was taken after every 24 hour for 3 to 9 months.

Finally, the assembly was removed from the loading frame and dismantled. The specimen ring was taken out and excess water was wiped out, after which the specimen ring weighted and recorded. The specimen was eventually placed in an oven and its dry weight was specified and recorded.

Calculation for Consolidation Test of Soil

$$\text{Height of solids, } H_s = W_s / (G \cdot \gamma_w \cdot A)$$

$$\text{Height of Voids, } H_v = H - H_s$$

$$\text{Void ratio, } e = H_v / H_s$$

Graphs of dial gauge reading vs. logarithm of Time were plotted to determine the coefficient of consolidation.

III. RESULTS AND DISCUSSION

A. Grain Size Distribution

Figure 5 shows the detail grain size pattern of various soil samples tested. Hydrometer analysis done for finer particles of *Ratomato*, *Kalomato* and *Kameromato* shows 58 %,16 % and 8.5 % of clay particles (i.e., finer than 0.002 mm) respectively.

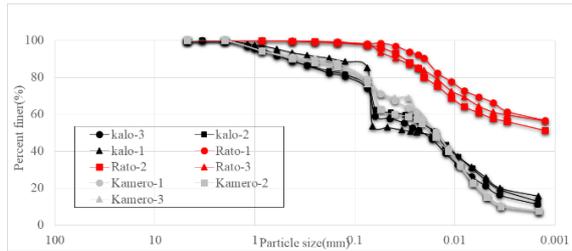


Fig. 5. Grain Size Distribution

B. Position of soil in Plasticity Chart

Figure 6 reveals the relationship between liquid limit and plasticity index (Casagrande's Plasticity Chart). Soil samples of *Ratomato* and *Kalomato* lie below A-line, whereas that of *Kameromato* lies above A-line.

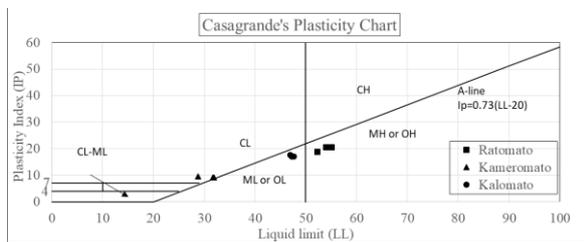


Fig. 6. Plasticity Chart

C. Activity

Figure 7 shows the activity of different samples tested based on plasticity index and percentage of clay fraction. Lines having slope of 0.75 and 1.4 (i.e.,  $A=0.75$  and  $A=1.4$ ) and passing through origin are reference lines for classifying the soil. Based on this, *Ratomato* was classified as inactive and *Kalomato* and *Kameromato* were classified as normal active soil.

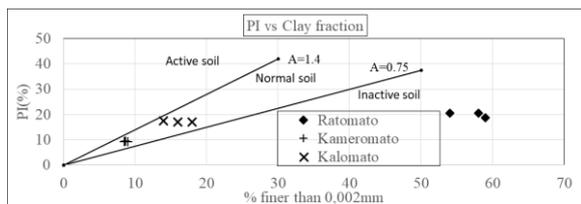


Fig. 7. Relationship between Plasticity index and Percentage of Clay Fraction

D. Basic Geotechnical properties

Detail description of various soil samples tested are presented in Table 1.

Table.1. Geotechnical Properties of three soils

Description	Ratomato	Kalomato	Kameromato
Liquid Limit	55.08	47.23	31.89
Plastic Limit	34.51	30.09	22.70
Plasticity Index	20.57	17.14	9.19
Shrinkage Limit (Lab)	22.55	19.01	20.55
SL(Casagrande'chart)	24	19	20
Sp.gravity	2.55	2.45	2.66
Clay fraction(%)	58	16	8.5
Organic content(%)	0.67	1.95	0.35
Soil type	MH	ML	CL
Natural(w%)	30.33	35.53	20.69
Liquidity Index(IL)	-0.18	0.78	-0.16
IL(Classification)	Hard	V.soft to soft	Hard
Activity	0.35	1.07	1.08

E. Consolidation Test

Figure 8 shows the coefficients of consolidation ( $C_v$ ) of *Kameromato*, *Kalomato* and *Ratomato*. The value of  $C_v$  for *Kameromato* varied from  $2.46 \times 10^{-7}$  m<sup>2</sup>/min to  $9.16 \times 10^{-5}$  m<sup>2</sup>/min. For *Kalomato* the value varied from  $5.55 \times 10^{-7}$  m<sup>2</sup>/min to  $9.13 \times 10^{-6}$  m<sup>2</sup>/min. Likewise, for *Ratomato* the value of  $C_v$  varied from  $4.92 \times 10^{-7}$  m<sup>2</sup>/min to  $3.28 \times 10^{-6}$  m<sup>2</sup>/min. Consolidation test was performed for 7 days where the load varied from 0.904 ton/m<sup>2</sup> to 57.856 ton/m<sup>2</sup> while maintaining one load for 24 hours.

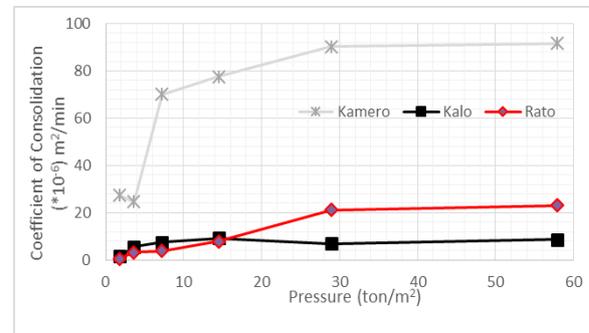


Fig.8. Coefficient of consolidation of three soil

F. Variation of Coefficient of Consolidation

Figure 9 shows the change in coefficient of consolidation of *Kameromato* and *Kalomato* within a period of 90 days at constant load intensity of 57.586 ton/m<sup>2</sup>. The graph shows that there is significant decrease in  $C_v$  for both *Kameromato* and *Kalomato*. The value of  $C_v$  for *Kameromato* decreased from  $9.15 \times 10^{-5}$  m<sup>2</sup>/min to  $4.81 \times 10^{-5}$  m<sup>2</sup>/min which was 47.5%. Similarly for *Kalomato*,  $C_v$  decreased from  $8.73 \times 10^{-6}$  m<sup>2</sup>/min to  $3.4 \times 10^{-6}$  m<sup>2</sup>/min which was 53%. Similarly, variation of coefficient of consolidation of *Ratomato* in a period of 270 days. The value of  $C_v$  varied from  $2.31 \times 10^{-5}$  m<sup>2</sup>/min to  $6.52 \times 10^{-6}$  m<sup>2</sup>/min which was 71.79 %. At the end of 3 months final water content of *kamero mato* and *kalo mato* was 20.34% and

32.4% respectively. For rato mato at the end of 9 months final water content was 23.16%.

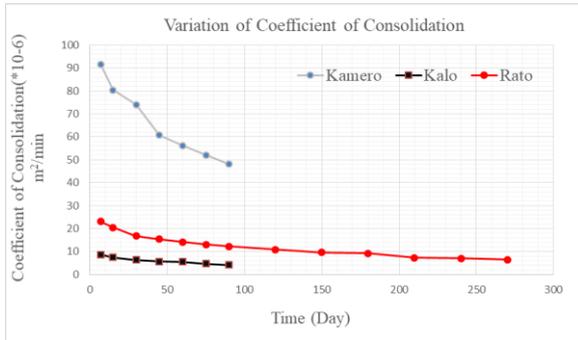


Fig. 9. Variation of coefficient of consolidation of Kamermato, Kalomato and Ratomato

G. Mineralogical composition

Minerals found in *Kameromato*, *Kalomato* and *Ratomato* did not possess pure chemical composition. Detailed mineralogical identification and quantification showed that chlorite was the major constituent of *Kameromato* (60%), also shown in Figure 10. Figure 11 shows quartz as the major element for *Kalomato* (60%) and protoenstatite as the second major element (26%). Similarly, Figure 12 shows quartz as the major mineral in *Ratomato* (80%). Mohs value of quartz is 6 which makes *Ratomato* and *Kalomato* hard. Mohs value of chlorite is 2.5 which is comparatively lower than that of quartz. This makes *Kameromato* soft in comparison to *Kalomato* and *Ratomato*. Percentage of quartz was found to be more in *Ratomato* than in *Kalomato*. This makes coefficient of consolidation of *Ratomato* more than that of *Kalomato*. Figure 11, Figure 13 and Figure 15 shows the mineral peak identification of kamero mato, kalo mato and rato mato respectively

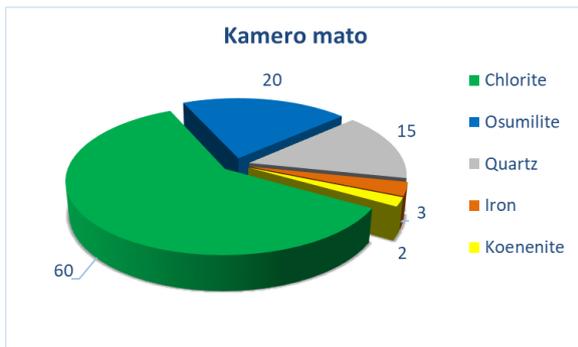


Fig.10. Kamermato Minerals

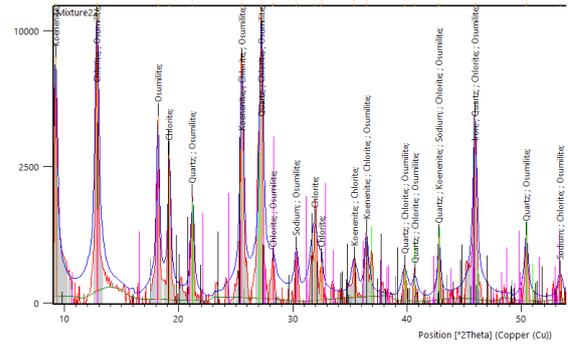


Fig. 11. Kamero mato peak identification

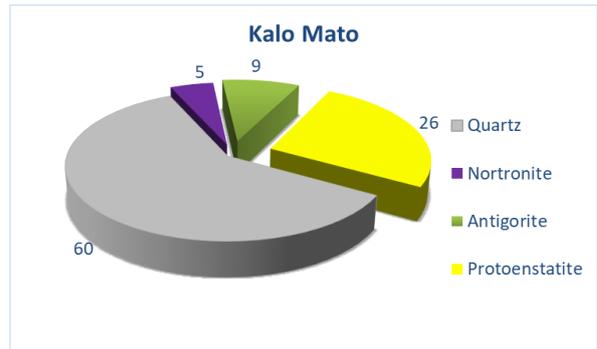


Fig. 12. Kalomato minerals

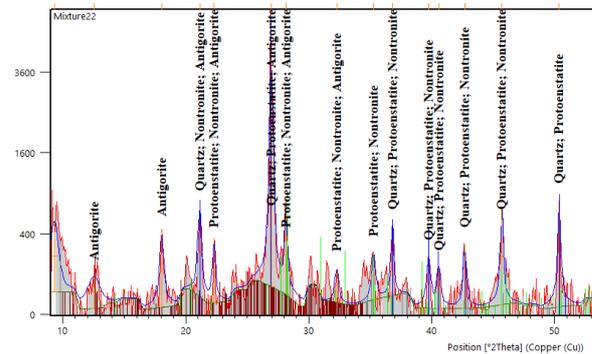


Fig. 13. Kalo mato peak identification

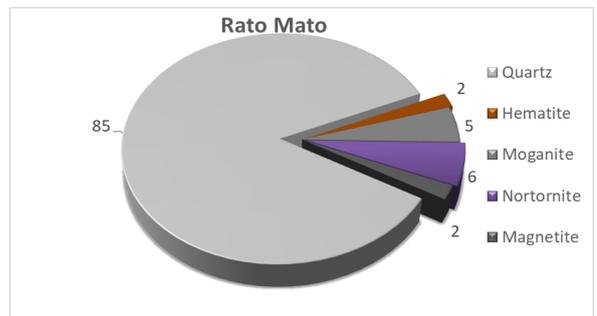


Fig.14. Ratomato Minerals

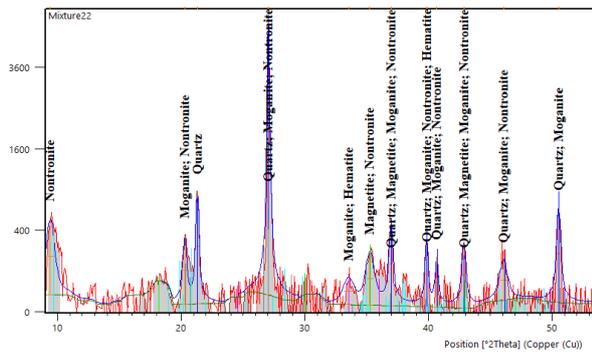


Fig. 15. Rato mato peak identification

#### IV. CONCLUSION

Results of the study indicate that the coefficient of consolidation is not constant but varies with the change in consolidation pressure and also with time. Significant change in coefficient of consolidation of *Kameromato* and *Kalomato* was witnessed which was 47.5% and 53% respectively within a period of 3 months at a constant consolidation pressure of 57.586 ton/m<sup>2</sup>. Similarly for *Ratomato*, the change was 71.79% in a period of 9 months. Hence, it can be concluded that the effect of decrease in void ratio at later time of consolidation leads to decrease in coefficient of consolidation. The variation in  $C_v$  thus may mislead in predicting the actual settlement of the structure. A detailed study is required to find the change in  $C_v$  of different soil types at different consolidation pressures.

Based on the detail mineralogical identification, quantification and Mohs values, quartz content is more in *Ratomato* and least in *Kameromato*. Major minerals found in *Ratomato* are hard, while those found in *Kalomato* are moderately hard and those found in *Kameromato* are soft, as its major mineral (chlorite) has low Mohs value. The reason behind higher value of  $C_v$  for *Ratomato* in comparison to *Kalomato* is the availability of higher concentration of quartz in it.

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