PREDICTION OF SETTLEMENT OF A SIX STORY BUILDING IN KATHMANDU USING FINITE DIFFERENCE ANALYSIS

Handali, S [1], Maharjan, S [2]
[1]Jurusan Teknik Sipil Universitas Kristen Immanuel Yogyakarta
e-mail : safehandali@yahoo.com
[2] Geotechnical Engineer at Macquarie Geotechnical Pty Ltd, NSW, Australia

ABSTRACT

Makalah ini memuat hasil dari program finite difference yang dikembangkan untuk memprediksi penurunan gedung enam tingkat yang dibangun di atas endapan lempung organik di Kalimati, Kathmandu, Nepal. Program tersebut dibuat berdasarkan teori konsolidasi satu dimensi dari Terzaghi dengan mengikut sertakan faktor pembebanan bertahap sesuai dengan sejarah pembebanan gedung yang sesungguhnya dan kondisi tanah yang berlapis. Hasil akhir dari program adalah kurva penurunan dengan waktu. Hasil tersebut kemudian dibandingkan dengan hasil pengukuran penurunan yang dilakukan setiap minggu terhadap gedung tersebut di tujuh lokasi kolom utama bangunan, mulai dari hari ke 136 sampai dengan hari ke 441 dihitung dari awal pembangunan.

Hasil analisa menunjukkan bahwa sampai dengan hari ke 441 penurunan bangunan mencapai 107 mm yang terjadi pada tingkat derajat konsolidasi rata-rata sebesar 42% dari penurunan maximum tanah yang diprediksi akan mencapai 254 mm. Dibandingkan dengan hasil pengukuran penurunan di ketujuh lokasi kolom-kolom terluar bangunan dapat dikatakan bahwa program finite difference yang ditujukan untuk memprediksi penurunan di titik pusat berat bangunan menunjukkan hasil yang konsisten dengan besar penurunan maupun pola penurunan yang diamati pada bangunan tersebut sampai dengan akhir pengamatan penurunan di hari ke 441.

Introduction

Settlement records of a six story building founded on Kalo Mato, an organic soil deposit in Kalimati area in Kathmandu, have been presented in an earlier paper by Handali and Maharjan (2011). Settlement readings from the building were taken since its early stage of construction, namely after the columns at the basement have been cast until the completion of the last floor, covering a period of about ten months. Readings were taken using auto level instrument on markers placed on seven reinforced concrete columns located at the perimeter of the basement. In this paper the result of settlement prediction of the building using finite difference analysis is presented. The result of the prediction is them compared with the settlement records.

Geotechnical properties of the subsoil at the site have been obtained from soil investigation carried out by Central Material Testing Laboratory (CMTL), Institute of Engineering, Tribhuvan University in 2005. Soil investigation work was carried out in conjunction with the planning and construction of the six story building. Three boreholes were drilled to 30 m depth, out of which disturbed and undisturbed samples were retrieved and tested in the lab. Of these boreholes, the borehole located at the center of area of the building was used to provide data for part of the research of Upadhyay (2005) for his Master's thesis and therefore more undisturbed samples were taken from this borehole compared to the other two boreholes. The samples from this borehole were also subjected to tests to find the organic content of the soil.

The finite difference analysis was developed for 1-D consolidation settlement. The program utilized Microsoft Office's Excel program. The analysis included varying load history following the real loading history of the building, multi – layered soil and that the initial pore pressure distribution with depth was equal to that of the vertical stress based on elastic theory. Immediate and secondary settlements were ignored.

Nature of Subsoil

The soil profile at the construction site is presented in Fig 1. The sub-soil was entirely organic clay with organic content found to increase with depth, ranging between 5% at the top part and 13% at the lower part. Slight variations of the colour and index properties of the subsoil could be observed between the top, middle and bottom parts of the borehole. At the top to a depth of 6 m the clay was grayish brown in colour. Underneath it the colour changed to dark gray up to about 15 m. The lower part of the soil up to 30 m depth consisted of an even darker layer. Handali et al (2007) showed that the soil properties were affected by the amount of organic content. Water content, liquid limit, Plasticity Index and void ratio with the increase of organic content while on the contrary the specific gravity, bulk density and dry density decrease with the increase in organic content. The compression index also increases with increasing organic content. Table 1 shows the range of values of the soil properties, roughly showing the properties of the three soil layers.

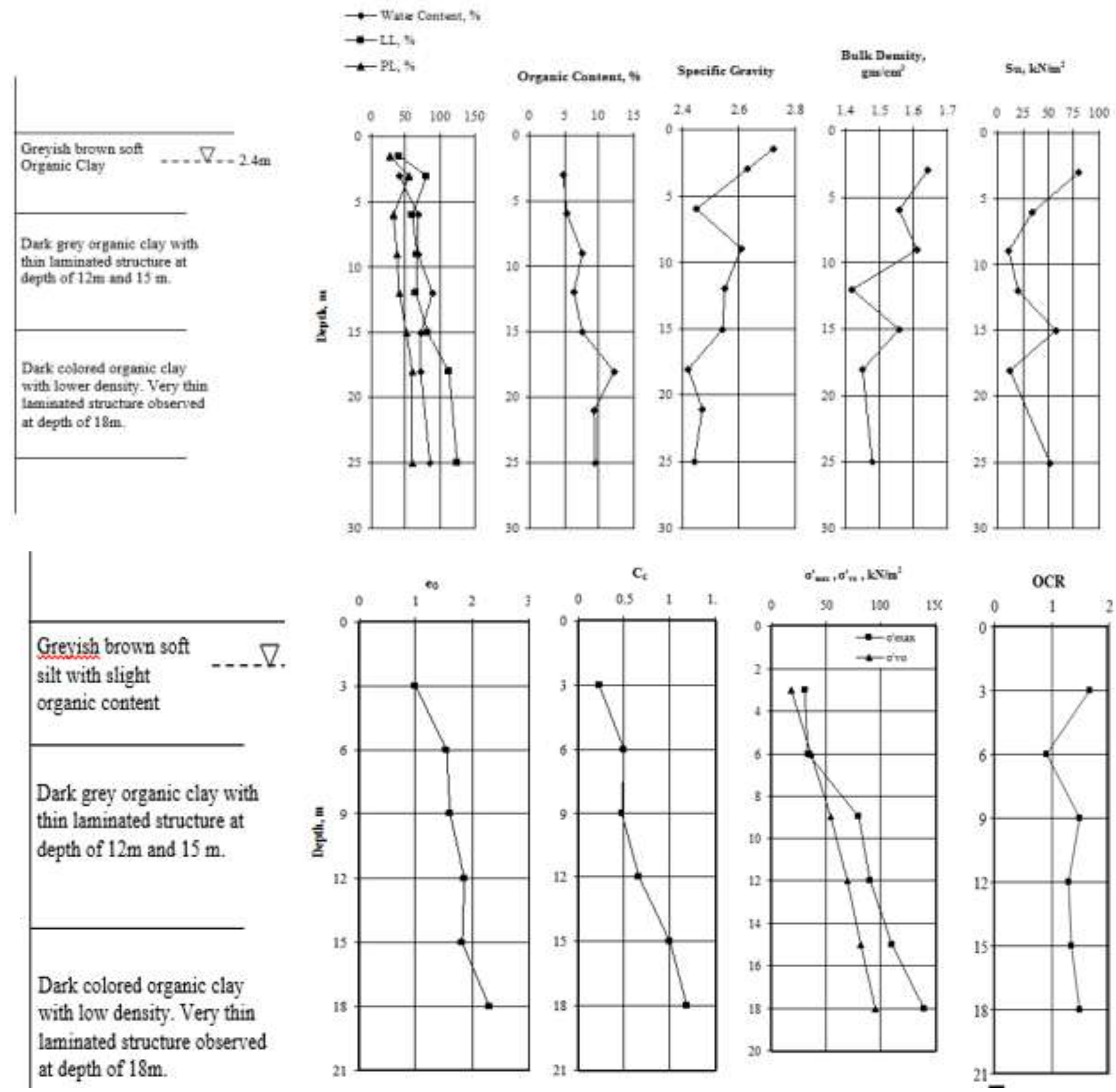

Fig. 1 The Variations of Soil Properties with Depth (Upadhyay, 2005)

Table 1 Range of Soil Properties

Depth, m	w %	LL %	PI %	G_s	γ gr/cc	γ_d gr/cc	e_o	C_c	s_u KN/m^2
0 – 6	40	75	18-25	2.7	1.6	1.12	1.0-1.5	0.25-0.5	79
6 – 15	60	60-80	25	2.5	1.4-1.5	0.75-1	1.5-1.7	0.5-0.7	11-57
15 – 25	75	75-125	> 40	2.45	1.5	< 0.8	> 1.7	> 1	55

The (e, log σ_v') graphs from 1-D consolidation tests conducted on undisturbed samples taken at depth interval of 3 m can be seen in Fig. 3. Fig. 4 shows the relationship between coefficient of consolidation and effective vertical stress from the same tests.

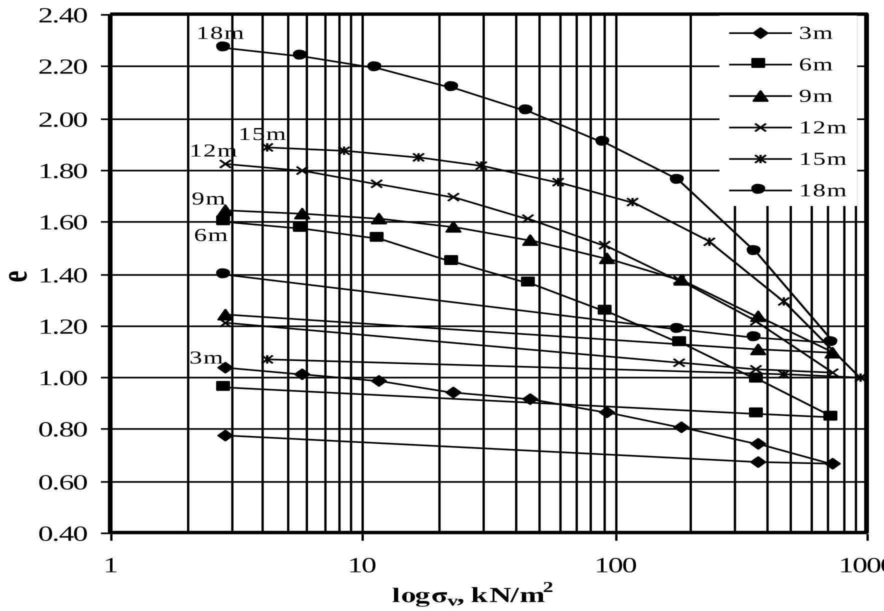

Fig. 3 Relationship between Void Ratio and Effective Stress from the Consolidation Tests (Upadhyay, 2005)

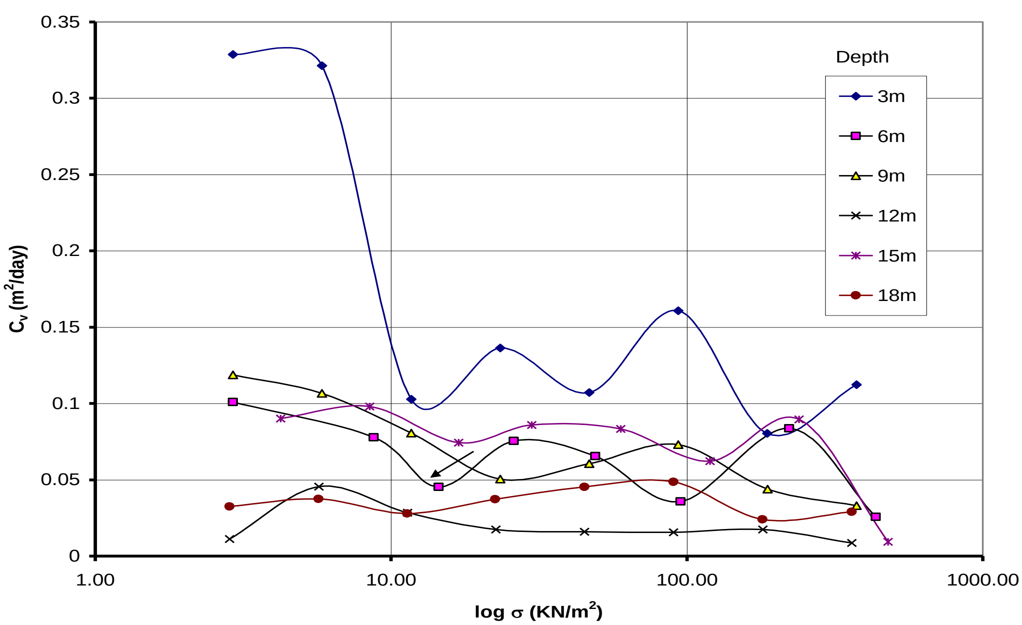

Fig. 4 Variation of c_v with and Effective Vertical Stress at Different Depths

Description of the Building

The floor plan of the building can be seen in Fig. 5 while its longitudinal section is shown in Fig. 6.

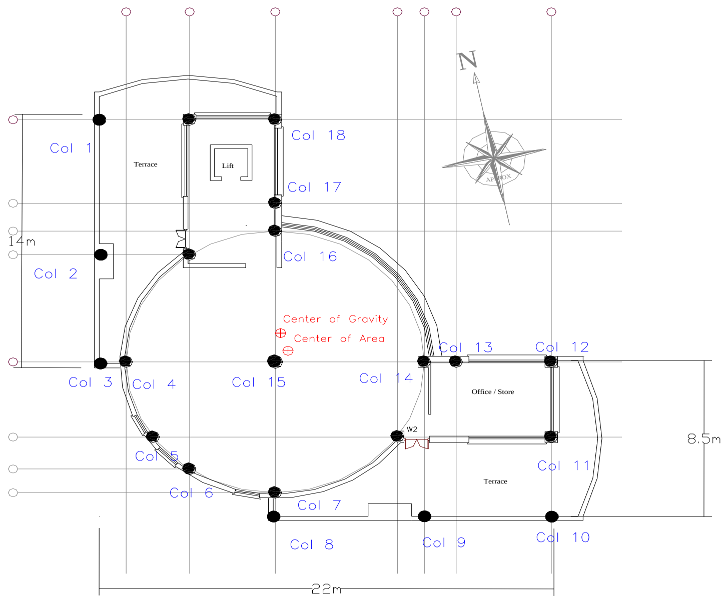

Fig. 5 Floor Plan of the Building and Column Locations

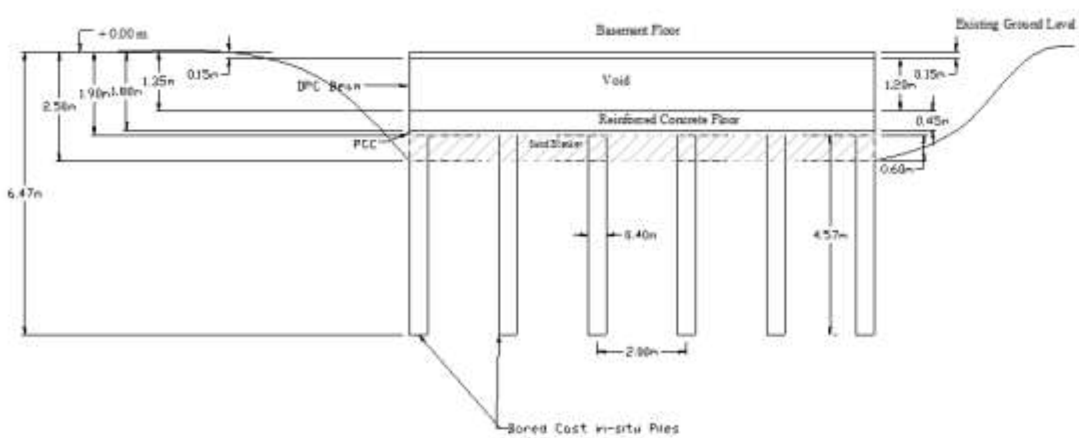

Fig. 6 Longitudinal Section of the Building

The building has a floor area of 310 m^2. Each floor was suspended by 21 columns of 600 mm in diameter. The height of the columns in the basement floor was 3.65 m whereas those of the upper floors were 3.2 m. The building's foundation was a reinforced concrete raft footing of 400 mm thick covering the entire floor plan at a depth of 2.5 m below the original ground surface. The raft was supported by 102 bored piles, placed at a distance of 2 m from other. Each pile has a diameter of 400 mm and length of 5.4 m. Below the raft footing was a concrete slab of 100 mm thick, underlain by 600 mm thick sand layer. Reinforced concrete beams of 1200 mm depth were placed above the raft and a 150 mm thick basement floor was constructed over the beam, creating an empty space of 1.2 m height between the raft footing and the basement floor.

Loading History

The loading history of the building is shown in Fig. 7. The loading chart started with the removal of overburden due to excavation before the laying of the sand blanket, taken as day zero i.e, the beginning of loading. Settlement measurement started on February 21, 2006, 136 days since the laying of the sand blanket. Settlement records were taken until day 400, which means that the recording covered a period of 364 days.

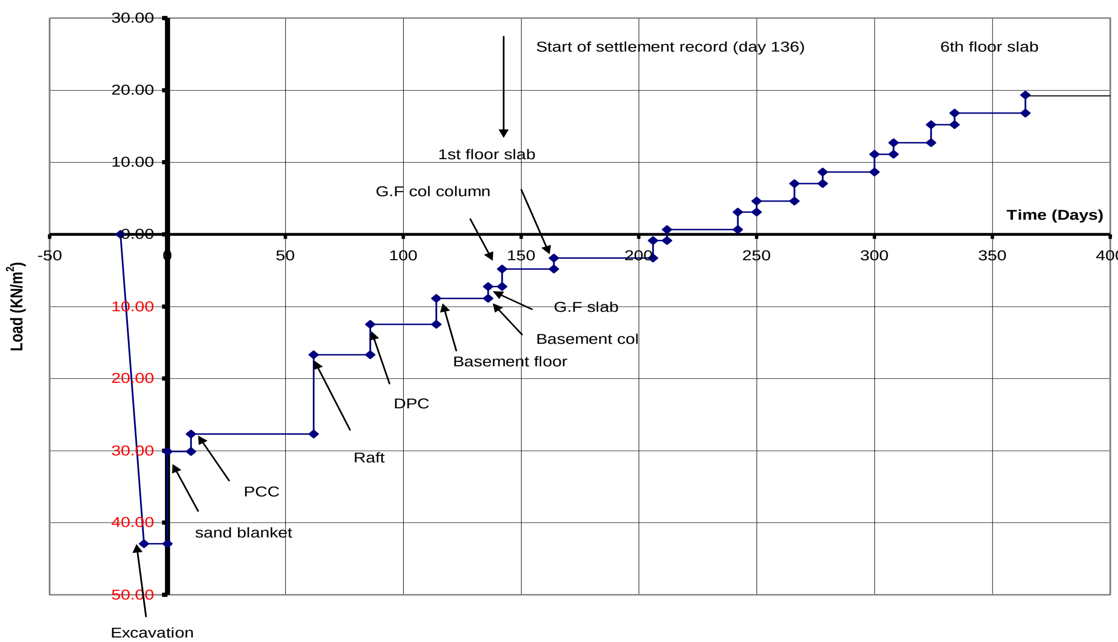

Fig. 7 Loading History of the Building

Ultimate Consolidation Settlement

The lower boundary of the consolidating layer under the raft was taken as the depth under the load where the stress level was 10% of the load. Distribution of the vertical stress under the center of gravity of the building was calculated by means of Newmark's

Chart, from which it was discovered that the lower boundary of the compressed layer was 24 m below the foundation level. The ultimate consolidation settlement was calculated as:

$$s = \sum_{i}^{n} \frac{\Delta e_i}{1 + e_{oi}} * H_i \qquad (1)$$

e_{oi} = void ratio prior to reloading, Δe_i = change in void ratio, H_i = thickness of sub-layer and n = number of sub-layers. Δe_i was determined from the relevant stress change experienced by the soil as revealed by the (e, logσ') graph in Fig. 3.

The correction factor from Skempton and Bjerrum (1957) was assumed to be 1.

Effect of Piles

As mentioned earlier, bored piles of 5.9 m length were constructed under the raft footing at a spacing of 2 m between each other. The piles were meant to provide anchoring for the building against earthquake and to increase the safety factor against bearing capacity failure by nailing the soil beneath the foundation. The presence of the piles, however, introduced some complications to the settlement analysis, particularly with regards to the compressibility of the soil between the piles. The conventional solution to estimate the settlement of pile group is to treat the pile group as an equivalent raft, the size and location of the base being determined by the assumption of load transfer mechanism which depends on the subsoil conditions. Consolidation settlement is then calculated by assuming the equivalent raft resting on the layer underneath it. The pile-soil within the enclosure of the pile group is assumed to act as a rigid block, i.e., no settlement occurs within the pile group. This approach is suitable if the building load is entirely carried by the piles. For the building in this study, however, the 102 piles carried about 30% of the building load only, i.e., 70% of the load was transferred by the raft to the soil right beneath it. In addition to that, the load transferred by the shaft of the piles (which was significantly larger than the load transferred by the tips) to the surrounding soil was carried by the same soil that also sustained the vertical load transferred directly by the raft. This led to the conclusion that the entire load of the building was directly transferred to the subsoil beneath the raft as if no piles were present.

Finite Difference Formulation

When loading on compressed later is applied gradually, the dissipation of excess pore pressure with respect to time under 1-D loading according to Terzaghi is:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial^2 z} + \frac{\partial q}{\partial t} \tag{2}$$

where $\frac{\partial q}{\partial t}$ is the rate of loading. At any node within the soil layer, Eq. 2 can be expressed in finite difference form as follows:

<u>For $0 \leq t \leq t_c$</u>

$$u_{i,j+1} = u_{i,j} + \beta(u_{i-1,j} + u_{i+1} - 2u_{i,j}) + \sigma_{j+1} - \sigma_j \tag{3}$$

Where, σ_j = load intensity at time j.

σ_{j+1} = load intensity at time j +1

t_c = end of loading period

The subsoil consisted of three different layers, with boundary between layers at depths of 6 m and 15 m. The finite difference formula at the boundary nodes:

$$u_{i,j+1} = u_{i,j} + \beta'(C_1 * u_{i-1,j} + C_2 * u_{i+1} - 2u_{i,j}) + \sigma_{j+1} - \sigma_j \tag{4}$$

Where, $$\beta' = \beta_1 * \left\{ \frac{1 + \frac{k_2}{k_1}}{1 + \frac{k_2}{k_1} x \frac{c_{v1}}{c_{v2}}} \right\} \tag{5}$$

$$C_1 = \frac{2k_1}{k_1 + k_2} \tag{6}$$

$$C_2 = \frac{2k_2}{k_1 + k_2} \tag{7}$$

$$\beta_1 = \frac{c_{v1}.\Delta t}{(\Delta z)^2} \tag{8}$$

$$\beta_2 = \frac{c_{v2}.\Delta t}{(\Delta z)^2} \tag{9}$$

The coefficients of consolidation c_{v1} and c_{v2} were those of two adjacent layers with k_1 and k_2 their coefficients of permeability. β_1 and β_2 were operators of the adjacent layers.

At the impermeable boundary:

$$u_{i,j+1} = u_{i,j} + \beta_2(2u_{i-1,j} - 2u_{i,j}) + \sigma_{j+1} - \sigma_j \qquad (10)$$

<u>For $t > t_c$</u>

At any node within the soil later:

$$u_{i,j+1} = u_{i,j} + \beta(u_{i-1,j} + u_{i+1} - 2u_{i,j}) \qquad (11)$$

At the boundary of different soil layers:

$$u_{i,j+1} = u_{i,j} + \beta'(C_1 * u_{i-1,j} + C_2 * u_{i+1} - 2u_{i,j}) \qquad (12)$$

At the impermeable boundary:

$$u_{i,j+1} = u_{i,j} + \beta_2(2u_{i-1,j} - 2u_{i,j}) \qquad (13)$$

The finite difference grid involving depth and time variables for the compressed layer can be seen in Fig. 8. The vertical spacing (□z) of 1.0 m and time interval (□t) of 2 days were determined to cause $\beta \leq 0.5$ for the solution to converge.□

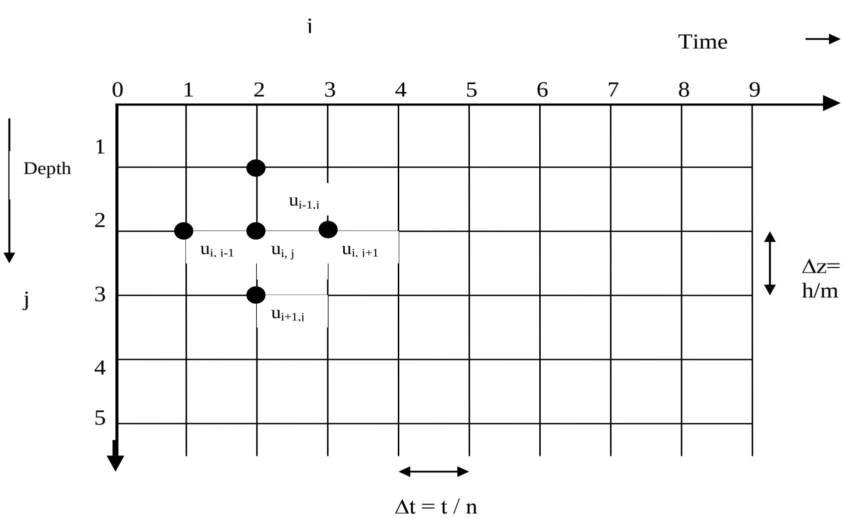

Fig. 8 Finite Difference Grid within a Soil layer

The upper surface of the consolidating layer was assumed to be free draining (the sand layer under the raft) while the lower boundary was impermeable.

Excess pore pressure at each node was calculated for each time interval, resulting in pore pressure profile across the depth of the compressed soil layer at any particular time. The average degree of consolidation across the depth of the compressed layer at the particular time is:

$$\overline{U_i} = 1 - \frac{\overline{u_i}}{\overline{u_o}} \tag{14}$$

$\overline{U_i}$ and $\overline{u_i}$ = average degree of consolidation and average excess pore at the particular time, respectively, $\overline{u_o}$ = average of cumulative initial excess pore pressure from the beginning of consolidation until that particular time. Settlement at any particular time:

$$S_i = \overline{U_i} * S_f \tag{15}$$

S_f = ultimate consolidation settlement for the stage of load at that particular time.

Selection of Consolidation Parameters

The coefficients of consolidation were determined from Fig. 4. The coefficients of permeability were calculated as:

$$k = c_v * m_v * \gamma_w \tag{16}$$

While m_v and c_v vary with stress level, k was assumed to be constant throughout the loading. Each m_v used in the calculation of settlement at each stage of loading was determined from void ratio-pressure graph for the relevant stress levels.

Ultimate Consolidation Settlement

Calculation of ultimate consolidation settlement resulted in a settlement of 254 mm at day 400. The predicted settlement was considerably higher than the limit of maximum settlement stated by IS Code 1902 -1978, where the maximum settlement is 100 mm for raft foundation on clay soils. The same limit has been proposed by MacDonald and Skempton (1955).

Result of Finite Difference Prediction and Comparison with Actual Settlement

Figure 9 depicts graphs showing the variation of excess pore pressure with depth after the application of major construction load as predicted by the finite difference analysis. The initial increase in pore pressure was generated by the pressure imposed by 600 mm thick sand blanket. The profile of the initial excess pore pressure was identical with that of the vertical stress due to the uniformly deposited sand layer. The excess pore pressure dissipated in upward direction towards the sand layer. The subsequent increase in load in the form of 100 mm thick concrete slab resulted in increase in the excess pore pressure through out the depth of the consolidating layer beyond the remaining excess pore pressure generated by the sand blanket. The same process was followed stage by stage until the application of the final construction load, after which the dissipation of excess pore pressure continued uninterrupted until the end of consolidation. The average degree of consolidation reaches 98% at 7000 days, or around 19 years after consolidation started, as indicated by the figure.

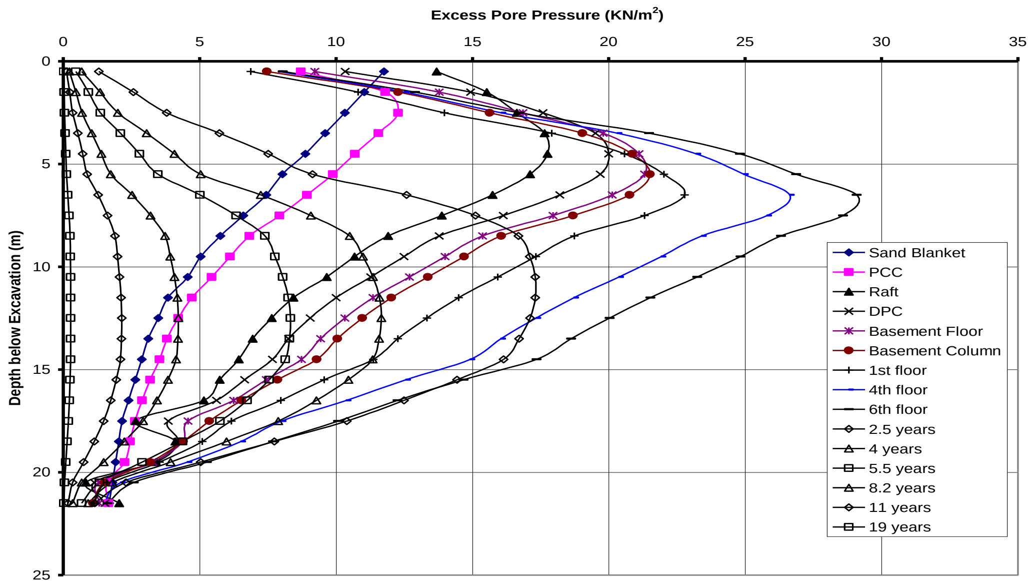

Fig. 9 Pore Pressure Profile at Different Stages of Construction Calculated from Finite Difference Method

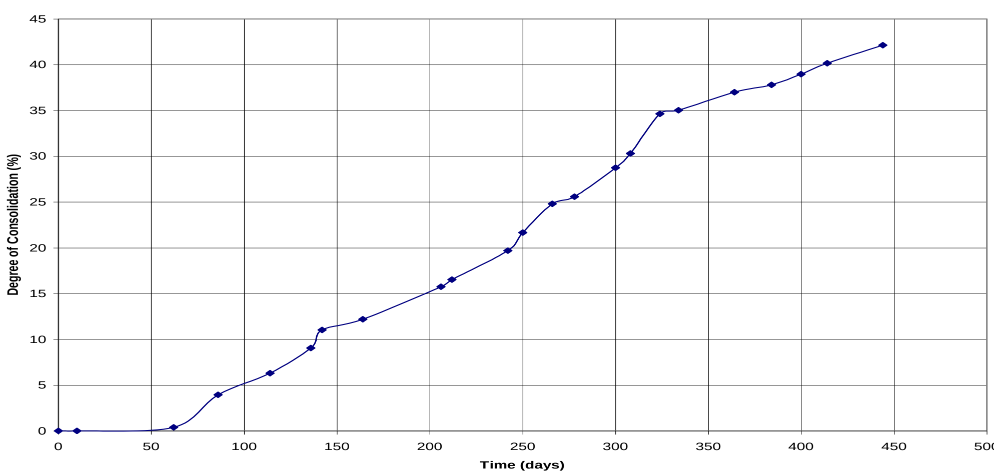

Fig. 10 Average Degree of Consolidation

Figure 10 shows the average degree of consolidation vs. time while Fig. 11 shows the predicted settlement–time curve. The calculation of excess pore pressure and its rate of dissipation started with the application of the sand blanket. The analysis predicted that the sand blanket imposed 6 mm settlement within 10 days, before the 100 mm thick PCC slab was applied. Though the increase of pore pressure due to the sand blanket was included in the finite difference analysis, the resulting settlement was omitted in Fig. 11 because it was not part of the settlement experienced by the structure, which started when the 100 mm PCC slab was laid.

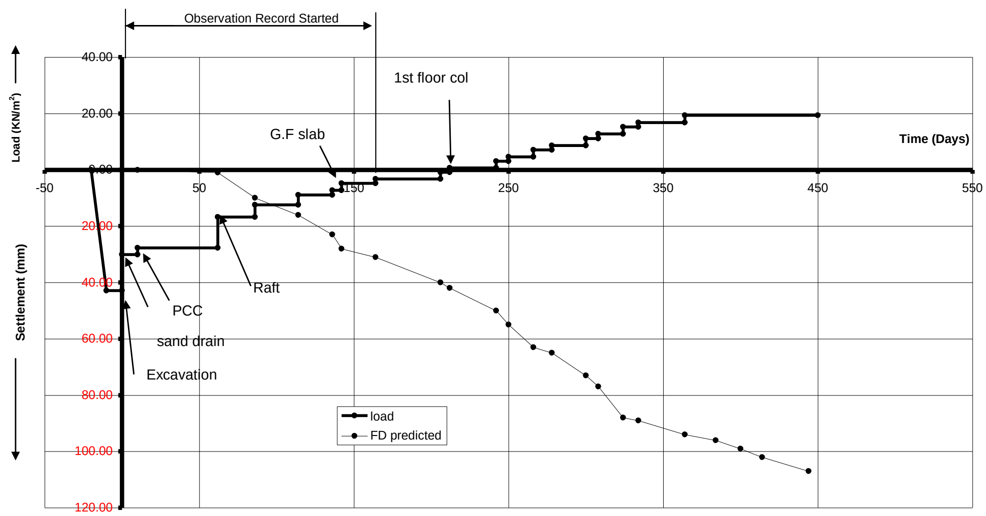

Fig. 11 Settlement vs. Time from Finite Difference Prediction

Figure 12 shows the settlement records of the building, measured at seven reinforced concrete columns located at the perimeter of the building. Each column was identified by a number and the locations of the columns within the building can be seen in Fig. 5.

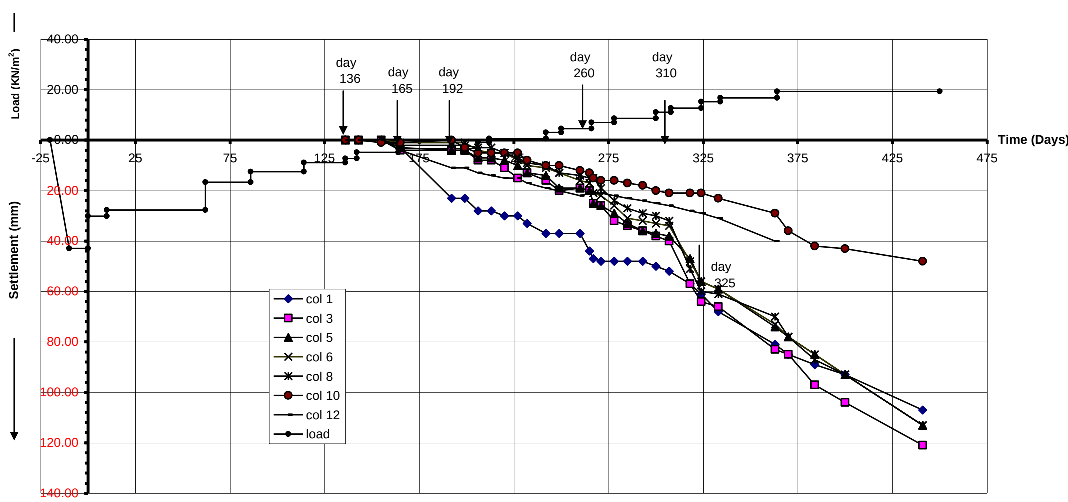

Fig. 12 Actual Settlement Record of the Building at Each Column Location

Settlement observation commenced on day 136 after the laying of the sand blanket, hence no record of settlement was yet available for the period between the casting of 100 mm thick concrete slab at day 10 (the first load on the subsoil which was due to the structure), until day 136. To compensate the absence of settlement readings, the result of the finite difference prediction (shown in Fig. 11) between day 10 and 136 was 'patched' to the initial parts of all the settlement records. The amendment to the initial readings resulted in an addition of 21 mm settlement, assumed to be uniformed for al the settlement points. Figure 13 shows the adjusted settlement curves, along with the entire settlement curve predicted by the finite difference analysis until day 441, the last day of the settlement readings. The prediction shows a settlement of 107 mm at day 441, which corresponded average degree of consolidation of 42% (Fig. 10).

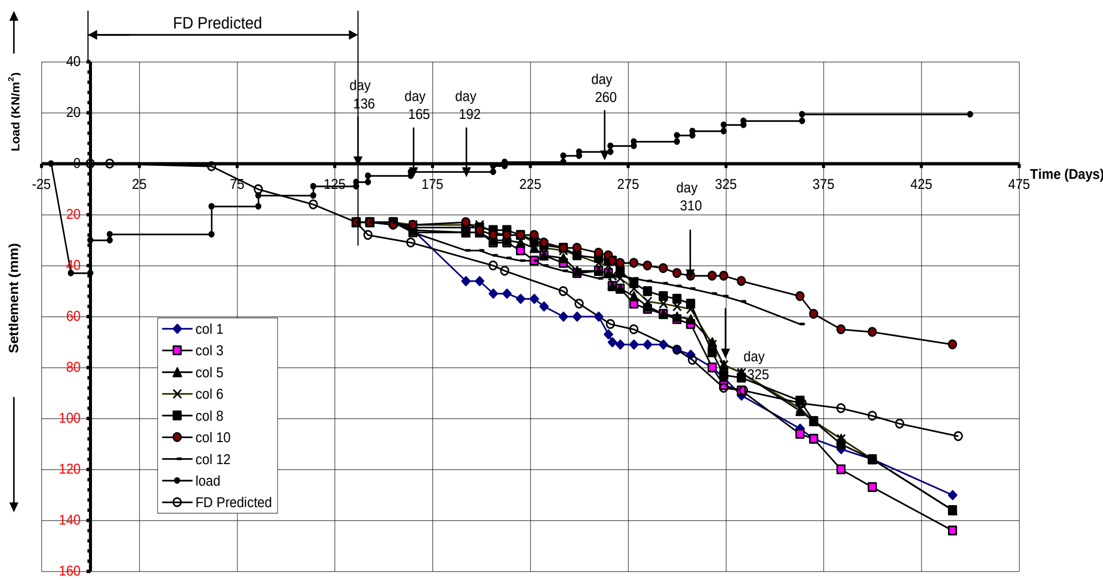

Fig. 13 Modified Actual Settlement and Predicted Settlement

It has been mentioned earlier that the settlement prediction was made for the center of gravity of the building. Since the center of gravity nearly coincided with the center of area of the building (see Fig. 5), it could be assumed the settlement curve represented settlement at the center of area of the building. The settlement prediction therefore, was meant to indicate more or less the average settlement of the building. In Fig. 13 it can be observed that predicted settlement curve was found nested among those of the settlement records obtained from the different locations of the building around its center of area. It could be said that that the result of prediction satisfied the requirement that it should show the average of the settlement measured at the different points of the building, at least qualitatively.

Figure 14 shows the settlement of Columns 1, 10 and that from the finite difference prediction. The positions of Columns 1 and 10 were diametrically opposite from each other and almost at the same distance from the building's center of area. As pointed out by Handali and Maharjan (2011), the settlement readings suggested that during the construction the building tilted more or less along the line of Column 1 – Column 10, with axis of rotation in the northeast-southwest direction passing through the center of gravity (≈ center of area) of the building. The differential settlement between those two columns on day 441 was 50 mm. This requires that the settlement of the center of gravity of the building should be somewhere between those shown by the two measuring points, in other words the curve of the predicted settlement should be found between the curves of the columns. As can be observed in Fig. 14, the predicted curve was found to be between the graphs of Column 1 and Column 10. This indicates that the result of the prediction was at least consistent with the results of settlement observations of the two columns, both in the range of values as well as the pattern of the settlement in the entire building.

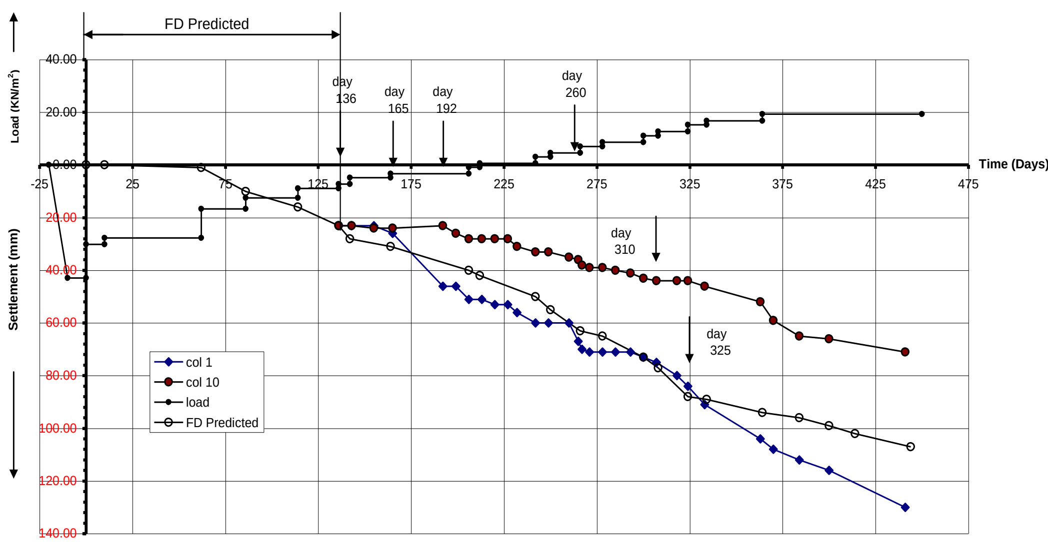

Fig. 14 Comparison of Settlement Curves of Columns 1 and 10 with the Settlement Curve from the Prediction

The same consistency between the result of the prediction and observation can be seen in Figure 15, showing the settlements of Columns 5 and 6. These two columns were located more or less along the axis of rotation of the tilt which passed through the center of gravity (or center of area) of the building. It should be expected that those two columns would have settlements which were more or less similar to that of the center of the building. This figure shows that the predicated curve was reasonably close to those measured at the two columns. This reinforces the earlier finding that the result of the prediction was consistent in terms of the amount of settlement as well as the pattern of the settlement observed on the entire building.

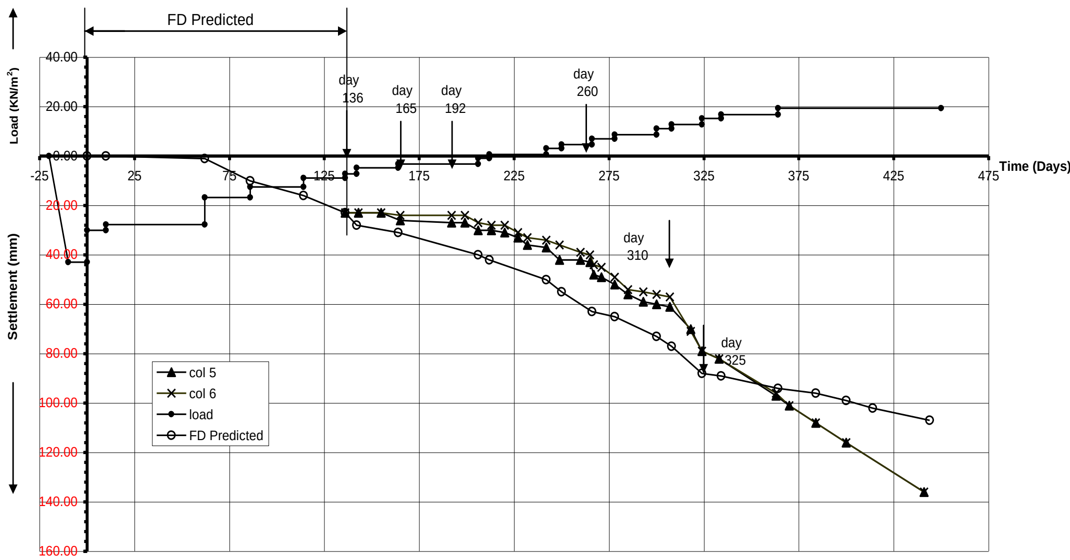

Fig. 15 Comparison of Settlement Curves of Columns 5 and 6 with the Settlement Curve from the Prediction

Acknowledgement

The authors want to thank Civil Homes Pvt. Ltd for allowing settlement measurements to be conducted on the Civil Saving and Credit Co-operative Limited Head Office in Kalimati for this research. Note of thanks was also given to Mr. Him Bandhu Upadhyay and to Central Material Testing Laboratory, Institute of Engineering, Pulchowk Campus, Tribhuvan University, for providing the soil investigation data used for the development of the finite difference analysis.

References

Central Material Testing Laboratory, IOE, TU, 2005, "Final Report on Soil Investigation Work for Proposed Civil Saving and Credit Co-operative Limited Head Office, Soaltee Mode, Kalimati, Kathmandu," Unpublished Report.

Handali, S., Maharjan, S, 2011, Settlement Observation of a Six-Story Building in Kathmandu, Majalah Ilmiah UKRIM, Edisi 1/th XVI/2011

Handali, S., Khadka, P., Neupane, R, Upadhayay, H.B., Acharya, G., 2007, Pengaruh Kandungan Organik pada Besaran Indeks Tanah Kalo Mato, di Kathmandu, Nepal, Majalah Ilmiah UKRIM, Edisi 2/th XII, 2007

MacDonald, D.H., and A.W. Skempton, 1955, “A Survey of Compression between Calculated and Observed Settlements of Structures on Clay,” Conf. on Correlation of Calculated and Observed Stresses and Displacements, ICE London, pp. 318-337.

Skempton, A.W., and Bjerrum, L., 1957, A Contribution to the Settlement Analysis of Foundations on Clay, Geotechnique, 7, 1968-78.

Upadhyay, H.B. 2005, “Geotechnical Properties of Some Organic Deposits in Kathmandu Valley,” M.Sc Thesis, Institute of Engineering, Pulchowk, Laitpur.