STUDIES ON GROUND DEFORMATION BEHAVIOUR AROUND BRACED EXCAVATION IN SOFT CLAY USING NUMERICAL METHODS AND GEOTECHNICAL CENTRIFUGE MODEL

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STATEMENT OF ORIGINALITY

I, Saptarshi Roy registered on 26.10.2016 do hereby declare that this thesis entitled "Studies on Ground Deformation Behavior Around Braced Excavation in Soft Clay Using Numerical Methods and Geotechnical Centrifuge Model" contains literature survey and original research work done by the undersigned candidate as part of Doctoral studies.

All information in this thesis have been obtained and presented in accordance with existing academic rules and ethical conduct. I declare that, as required by these rules and conduct, I have fully cited and referred all materials and results that are not original to this work.

I also declare that I have checked this thesis as per the "Policy on Anti Plagiarism, Jadavpur University, 2019" and the level of similarity as checked by iThenticate software is **7** %.

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Place: Kolkata

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Symbols and Notations

FEM – Finite element method

FS – Factor of safety against basal heave

AIR = Apparent influence zone

2D = Two dimensional

L1 = Layer 1

L2 = Layer 2

L3 = Layer 3

L4 = Layer 4

SSCM = Soft soil creep model

SSM = Soft soil model

M-C = Mohr-Coulomb

γ_{unsat} = Unsaturated unit weight

γ_{sat} = Saturated unit weight

c_u = Undrained cohesion

e_0 = Initial void ratio

ν_s = Poisson's ratio

K_0 = Coefficient of earth pressure at rest

λ^* =Modified compression index

κ^* = Modified swelling index

μ^* =Modified creep index

E_w = Young's modulus of wall

A_w = Cross sectional area of wall

I_w = Second moment of inertia of the wall section

E_{st} = Young's modulus of strut

A_{st}= Cross sectional area of strut

s_{st} = Average spacing between consecutive struts

ε_c^e = Elastic deformation

ε_c^c = Consolidation deformation

τ_c = Parameter which indicates consolidation time

t_c = Consolidation time

t' = Effective creep time

H_{exc} = Excavation depth

B_{exc} = Excavation width

D = Embedded depth

T_{wall} = Wall thickness

$\delta_{g,max}$ = Maximum ground displacement

$\delta_{w,max}$ = Maximum wall displacement

NGD = Normalized ground deformation

NWD = Normalized wall deflection

s_u = Average shear strength of soil

σ_v' = Effective vertical stress

h_{avg} = Average strut spacing

L = Distance from wall

d = Depth from ground surface

δ_{vl} = Ground deformation at a distance '*l*' from edge of wall

δ_{hd} = Horizontal wall displacement at depth 'd' from ground surface

RMSE = Root mean square of error

NRMSE = Normalized root mean square of error

S_t = Ground settlement for different cases

W_t = Wall displacement for different cases

S_0 = Ground settlement values obtained when no delaying/ stopping occurs

W_0 = Wall displacement values obtained when no delaying/ stopping occurs

E_R = Excavation rate

S_T = Pause time

L= Length

W= Width

H =Height

MS = Mild steel

CI = Cast iron

VFD = Variable frequency drive

ω = Angular velocity

r_e = Nominal radius

N = Geotechnical centrifuge constant

g =Acceleration due to gravity

RPM = Revolutions per minute

EI = Equivalent bending stiffness.

E_D = Excavation Depth

γ = Unit weight

w = Water content

% = Percentage

$\delta max(0.25)$ = Ground deformation corresponding to 0.25 day

$\delta max(1)$ = Ground deformation corresponding to 1 day

1D = One dimensional

δ_{Rate} = Settlement rate

$\delta_{max(7)}$ = Ground deformation at pause time 7 days

$\delta_{max(0)}$ = Ground deformation corresponding to 0 day or no pause

ABSTRACT

During Kolkata metro railway construction in the late 1980s using braced excavation systems, it was observed that the excavation sequence and the rate of excavation including delay during construction had a major impact on the deformation behavior of the excavation system. In order to access the effect of these and other associated factors like excavation depth, excavation width, diaphragm wall thickness, wall embedment depth, strut locations, and soil properties, finite element analysis of typical braced excavations in soft clayey deposits of Kolkata is performed. In the present study, a thorough, parametric study has been conducted using finite element analysis to address the influence of various parameters on deformation characteristics of braced excavation in soft clayey deposits. The importance of correct estimation of soil parameters for braced excavation design is also documented. The analysis of typical braced excavations in soft clay is carried out using Plaxis 2D software where the soft soil creep constitutive model is used. On the basis of numerical study, a handy design guideline is recommended. Further multi-variate regression models are developed incorporating various important excavation parameters for adequate prediction of maximum wall and ground displacement along with wall and ground surface deformation profile. Here large numbers of data reported in case histories and generated artificially from FE analysis are used for formation of regression equations. The proposed model is validated comparing results from literatures not used for development of the model. Further, extensive study has been conducted to understand the effects of excavation rate and construction stoppages on ground and braced wall displacement. Fitted equations are developed for the maximum ground settlement and maximum lateral wall deflection conducting multi-variable regression analysis where time parameters like rate of excavation, pause in construction and depth of excavation are used as independent parameters. Further, a systematic study of the layer wise ground deformation behind a braced excavation in soft clayey soil similar to that available in Kolkata is performed using physical model study using a geotechnical centrifuge for different depths of excavation, number of struts and also considering construction delay or construction stoppage after reaching the final cut level. The mechanism of this deformation was also assessed by evaluating the contribution of undrained, consolidation and creep deformation to the total ground deformation. The tests results were also used to predict the effect of construction delay on various important factors like rate of settlement, change of zone of influence behind a braced wall etc. The experimental results are also validated with the observed values obtained from reported case studies.

Keywords: Finite element analysis; Braced Excavation; Ground Movement; Wall Deflection; Soft Soil Creep Model; Multi-variate Regression Models; Rate of excavation; Pause Time; Geotechnical Centrifuge; Undrained Deformation; Consolidation Deformation; Creep Deformation.

CHAPTER 1 - INTRODUCTION

1.1 GENERAL

Over the recent decades, braced excavation has become a widely used method for constructing underground structures in various types of soils. Its popularity stems from being a cost-effective and relatively straightforward construction technique that requires less specialized equipment. Braced excavation finds application in heavily built-up areas for building underground basements, parking lots, shopping centers, pumping stations, and metro railways, among others. Despite its advantages, braced excavation can lead to ground movement, which can affect nearby structures. Excessive ground movement may impact the appearance, efficiency, and durability of structures in the vicinity. As urban development progresses with high-rise buildings and towers, excavations for foundations have become deeper, sometimes beyond 15 meters compared to the typical depth of 5 to 8 meters. To ensure excavation stability and reduce the effects on neighboring structures and underground utilities, continuous wall structures are often employed. Multi-strutted structural systems, like strut, wale, and diaphragm wall arrangements, are preferred in such cases to minimize ground movements, settlements, and achieve economic benefits. Predicting horizontal wall displacements and ground settlements poses challenges due to the complex and non-linear behavior of soil. Three common techniques are used for prediction: interpolation from published databases, numerical analysis employing finite element or finite difference methods, and physical model tests conducted using geotechnical centrifuges. While numerical models attempt to incorporate various aspects of soil behavior, they can be complex, lack clear physical meanings for some parameters, and demand substantial computational resources. The parameters often require specialized testing techniques and laboratory skills, making them less appealing to practicing engineers. As a result, many engineers' resorts to design charts that rely on factors of safety against basal heave to estimate wall deflections relative to soil properties. In summary, braced excavation is a widely utilized construction method for underground structures, but it comes with challenges related to ground movement prediction. Engineers seek a balance between accuracy and practicality, opting for design charts as a more straightforward approach to account for soil behavior and wall deflections in braced excavation projects.

Early studies on braced excavations were based on field observations, and those studies focused on excavation-base instability caused by bottom heave, lateral movement of support systems, ground settlement adjacent to excavations, effects of soil type and excavation geometry on the performance of the excavation system, and earth pressure on braced walls (Terzaghi 1943, Bjerrum and Eide 1956, Peck 1969, Lambe 1970, Goldberg et al. 1976). Lambe (1970) concluded that the state of the art for design and analysis of braced excavations was far from satisfactory, and suggested the use of finite element method in conjunction with field studies as the way forward for gaining proper understanding of deep excavation performance. O'Rourke (1981) pointed out the importance of site preparation in ground excavation work and related the lateral movement of excavations to ground settlements, based on field observations. Clough and O'Rourke (1990) categorized movements in a braced cut into two types: movement related to excavation and support process, and movement related to auxiliary construction activities. Finno and Harahap (1991) simulated the construction of a 40-ft-deep braced excavation in saturated clays in Chicago by using a coupled finite element (FE) analysis. Tefera et al. (2006) studied the ground settlement and wall deformation of a sheet pile wall during different stages of excavation using a large-scale model test in dry sand bed and compared the results with those of FE analysis. Finno et al. (2007) used the FE software Plaxis for conducting a parametric study to show the effects of excavation geometry on the deformation behavior of soil around braced excavations. They observed that when the ratio of the excavated length to excavated depth of a wall is greater than 6, plane strain simulations yield the same displacements in the center of the wall as those obtained from three-dimensional FE analysis. Hsiung (2009) investigated the deformation characteristics of several excavations in Kaohsiung, Taiwan, and found that the maximum lateral wall displacement $\delta_{wall, max}$ is approximately 0.03-0.3% of maximum excavation depth and that the ratio of maximum surface settlement $\delta_{ground, max}$ to maximum wall displacement $\delta_{wall, max}$ varies over 0.5-0.7 for the excavations constructed by bottom-up method and over 1.3-1.8 for the excavations using a semi-top-down method. Hsiung (2009) observed that the subsidence of the ground surface behind the diaphragm wall extended to a distance of up to three times the maximum excavation depth. Whittle et al. (1993) performed coupled FE analysis, combining flow and deformation, of a top-down construction for a seven-story, underground parking garage in Boston. De Lyra Nogueira et al. (2009) also conducted coupled

FE analysis on braced excavations with different constitutive models and different excavation rates and showed that the choice of the constitutive model affects the magnitude and distribution of excess pore pressure. Hashash and Whittle (2002) presented a detailed interpretation of the evolution of stresses around a braced excavation in a deep layer of soft clay considering anisotropic stress-strain-strength relationships, small strain nonlinearity, and hysteretic response upon load reversal. Babu et al. (2011) used the finite difference software FLAC to perform a two-dimensional (2D) analysis of a vibration isolated system using open trenches. Nogueira et al. (2011) performed FE analysis of an instrumented, unsupported excavation constructed in a soft clay deposit using a non-associated elastoplastic constitutive model. Chowdhury et al. (2013) performed numerical analysis of a braced excavation and provided design guidelines. These apart, empirical and semi-empirical methods have been used for estimating the ground surface settlement induced by braced excavations (Bowles 1988; Ou et al. 1993; Hsieh and Ou 1998).

From the observation of above study some factors which influence the magnitude and pattern of ground movements are as follows

1. Profile of the subsoil layer and engineering properties of different strata.
2. Size and depth of the excavation.
3. Degree of wall embedment and degree of preloading.
4. Stiffness of support system.
5. Surcharge load.
6. Sequence, method of excavation and work speed etc.

Time effect is another important factor which has considerable bearing on the deformation behavior of excavation especially for soft clayey soil. If the excavation work proceeds slowly or excavation is left open for long periods then secondary and creep effects become significant. Studies related to time-dependent behavior of braced excavations are rather limited. Lien et al. (1993) studied a 6.4 m wide and 8.5 m deep braced excavation in Detroit soft clay in the U.S.A. that was constructed in twelve stages, and it was found that, for excavation up to a depth of 5.18 m, the rate of ground deformation was low at about 0.164 mm/day, but, beyond that depth, there was a significant increase in ground deformation rate to about 0.677 mm/day.

Som and Gupta (1994) reported the effect of time delay on deformations of braced cuts in soft clay based on extensive measurements of ground movement and building settlement at different sections of Kolkata underground metro rail excavation performed during the late 1980s. Additional ground settlement was observed when an excavation was kept open at the final cut level or at any intermediate level for long periods of time Som and Gupta (1994). Ou et al. (1998) investigated a case study at the Taipei National Enterprise Centre (TNEC) site in republic of China where the excavation to a depth of 19.7 m was done in thirteen construction stages, and an increase of 18.2 mm of lateral displacement was observed because the construction was halted for 60 days at stage 6 at a depth of 7.1 m. Liu (2005) monitored a 15.5 m deep multi strutted soft clay excavation of a metro station in Shanghai. Shanghai is located at the exit of the Yangtze River facing the East China Sea and the reported excavation site is in the Yishan Road station, which is located southwest of the city, at the Pearl II Metro Line. The excavation was executed in 5 stages namely at 4.7 m, 6.9 m, 9.7 m, 12.5 m and 15.5 m respectively. Prior to the excavation to stage 5, a 0.6 m thick reinforced concrete middle slab was constructed after the struts at stage 4 had been installed. About 60 days of curing period were allowed (i.e., Stage 4) before the excavation to stage 5 commenced. The average excavation rate followed to reach the final excavation depth was around 6.55 days/m. He reported an additional creep deformation during 60 days curing period of middle slab constructed before fifth stage of excavation.

Further, model test using geotechnical centrifuge become popular now a days. Centrifuge testing involves investigating geotechnical phenomena by employing scaled-down models that experience acceleration forces several times stronger than Earth's gravity. Through this method, the effects of self-weight stresses and gravity-driven processes are accurately replicated. As a result, observations drawn from experiments with reduced-scale models can be extrapolated to real-world situations using established scaling principles. This is of great significance since soil is an immensely nonlinear and historically influenced material and so modelling using geotechnical centrifuge provides good control on the overall behavior of the construction. This level of precision is challenging to achieve through numerical analyses, as they rely on the soil models employed, chosen material parameters, construction sequences, and primarily, uncertainties stemming from the incomplete understanding of the actual soil profile and conditions. As a result, data derived from centrifuge tests offer a valuable supplement to meticulous field studies with instrumentation and corresponding numerical assessments. Development of geotechnical centrifuge facilities and research projects since 1970 has been rapid and well documented (Craig, 1984, Kimura, 1994 and Takemura et al. (1999)). Kimura et al. (1994) reported centrifuge experiments on unsupported excavations, and excavations with sheet pile walls, with or without ties, in NC and OC clays. An in-flight excavator was used to simulate the excavation process. Deformations of the clay, pore water pressures and earth pressures on the wall, were measured. Takemura et al. (1999) investigated a vertical excavation in normally consolidated soft clay in which the construction sequence of a doubly tied wall for an open excavation was simulated properly in-flight with an excavator. Settlement of the ground surface, earth pressure on the wall strains along the wall and pore water pressure in the ground were measured during the test. Konkol. J (2014) presented a derivation of scaling laws by dimensional analysis for the centrifugal modelling. Basic principles of centrifuge modelling are also described.

1.2 MOTIVATION OF PRESENT STUDY

The interaction of braced wall with the adjacent ground is an important aspect of design because this soil-structure interaction influences the overall behavior and effectiveness of the excavation support system. Though deformation behaviours of braced excavations were studied depending upon variations of excavation parameters in literatures, still limited works have been done on systematic parametric analysis of braced excavation presenting design guidelines to achieve optimized deformation values. It is necessary to conduct parametric analysis to assess the response of soil and excavation parameters in order to control soil movements especially in congested urban areas. Further it is observed that estimation of ground or wall deformation values considering various design parameters, such as, excavation depth, excavation width, diaphragm wall thickness, wall embedment depth, shear strength of soil and strut spacing, are not well addressed in literature. However, a particular feature of the soil-structure interaction of braced excavations in soft clayey deposits is its time-dependent nature that is mostly neglected in analysis and design. It is important to note that a major part of Kolkata strata is covered with soft clay down to a depth of 10 m to 15 m below the existing ground level. A significant amount of excavation work has been done in the past during the construction of Kolkata metro. Further, in various infrastructure projects, underground excavation is in progress. During these works, sometimes it is seen that the work is delayed or slowed down because of different reasons beyond the control of the engineer-in-charge. It has been observed in the construction sites in the city of Kolkata in India that delays in construction result in increased ground movements because of creep deformation of Kolkata clay (Som and Gupta 1994). The magnitude of additional ground deformation caused by construction delay may seriously damage the neighboring structures if this excess time-dependent deformation is not considered in design.

Further, a substantial volume of work has been expended for estimation of ground deformation and wall movement in braced excavation by using empirical formula or using finite element method. But a limited amount of work has been done by incorporating the time effect in braced excavation required for the estimation of ground deformation and wall deflection. Most studies are based on field observations, and not much work has been done by using geotechnical centrifuge testing.

1.3 OBJECTIVE OF THE PRESENT STUDY

The objectives of the present study are,

(i) To study ground movement and wall deflection behaviour due to braced excavation in soft clay by varying several time independent parameters such as excavation geometry, wall thickness, wall embedment depth, locations and combinations of strut, soil properties, soil constitutive model etc.

(ii) To study ground movement and wall deflection behaviour due to braced excavation in soft clay by varying several time dependent parameters such as rate of excavation, pause time during construction.

1.4 SCOPE OF THE PRESENT STUDY

In the present study both numerical analysis and physical model tests were carried out in order to study the undrained as well as time dependent deformation behaviour of ground around braced excavation in soft clay. So, scope of the present study has been divided into two parts (A) Scope of the numerical study and (B) Scope of the experimental study.

(A) Scope of the numerical study

(i) In the numerical analysis, a parametric study utilizing finite element analysis was undertaken to evaluate the impact of different controlling factors (depth of excavation, width of excavation, thickness of wall embedment depth, soil strength, average strut spacing), including time-dependent variables like excavation rate and pause duration, on the deformation behavior of braced excavations situated in soft clay deposits.

(ii) For the analysis of typical braced excavations in soft clay, the study employed the PLAXIS 2D software, incorporating a soft soil creep constitutive model.

(iii) On the basis of numerical study, a handy design guideline is recommended.

(iv)Additionally, multi-variate regression models are developed incorporating various important excavation parameters for adequate prediction of maximum wall and ground displacement along with wall and ground surface deformation profile.

(v) Proposed model is validated comparing results from literatures not used for development of the model.

(B) Scope of the experimental study

(i) In the experimental study segment, a methodical examination of layer-by-layer ground deformation occurring behind a braced excavation in soft clayey soil, akin to the conditions found in Kolkata, was undertaken through 14 numbers of physical model experimentation.

(ii) Various excavation depths and numbers of struts were considered, as well as scenarios involving construction delays or stoppages subsequent to reaching the final cut level.

(iii) The fundamental mechanism driving this deformation was scrutinized by analyzing the contributions of undrained, consolidation, and creep deformations to the overall ground deformation.

(iv) The results from these tests were employed to forecast the implications of construction delays on critical factors such as settlement rate and alterations in the zone of influence behind a braced wall.

(v) Finally, the experimental results are validated with the observed values obtained from reported case studies (Som, 2000; Ou, 1998 and Liu, 2005).

1.5 OUTLINE AND ORGANISATION OF THESIS

The thesis has been organized in five different chapters as follows

Chapter 1 briefly discusses the basic theory and brief description of design consideration of braced excavation along with the necessity of parametric study by incorporating with or without time effect for determining adjacent ground deformation behavior around braced excavation in soft clay and further the section also focusses the need for model study using geotechnical centrifuge.

Chapter 2 discusses about the extensive review of literature regarding some general theory, case studies, empirical studies, numerical/analytical studies and centrifuge model studies related to braced excavation and further the lesson learned from each section of the studies have been also discussed.

Chapter 3 focusses on the deformation behavior for design of braced excavation in soft clay with or with incorporating time effect. After extensive finite element method studies design guidelines, simplified model, charts and figures are proposed for estimation of ground and braced wall displacements using the data obtained from the study.

Chapter 4 emphasizes on experimental study on time dependent ground settlement behavior behind a braced excavation in soft clay using geotechnical centrifuge. In the study a systematic analysis of the layer wise ground deformation behind a braced excavation in soft clayey soil similar to that available in Kolkata is performed using physical model study for different depth of excavation, number of struts by incorporating time effect, which leads to some valuable conclusion.

Chapter 5 brings out the summary, conclusions, findings and suggestions for future research.

An **Abstract** of the dissertation has been presented at the beginning of this dissertation.

CHAPTER 2 - REVIEW OF LITERATURE

2.1 GENERAL

Since the early 1940s and throughout the previous decade, researchers and engineers have dedicated their efforts to the field of braced cut design and ground deformation control. Their primary objective has been to bridge the gap between theoretical evaluations and actual field measurements. By doing so, they aim to enhance the accuracy and reliability of their predictions and better understand the behavior of braced excavations and the control of ground deformation. Over the years, advancements in geotechnical engineering, numerical modeling, and laboratory testing have contributed to the ongoing progress in this field. Through continuous research and learning from practical experiences, the goal is to refine design methods and construction practices for braced cuts, leading to more efficient and reliable solutions for deformation control. Ultimately, this work seeks to enhance the sustainability and success of excavation projects while mitigating potential adverse effects on surrounding structures. Researchers and engineers have been working since early forties for the previous decade in the field of design of braced cut and ground deformation. Their main aim is to reduce the disparity between theoretical evaluation and field measurements and to develop more rational construction techniques for deformation control.

In this section, a compilation of data from field studies, outcomes derived from numerical analysis, empirical approaches, and results obtained from centrifuge tests related to braced excavation issues have presented to highlight the present state of art research in this area of work.

2.2 CLASSICAL STUDIES ON BRACED EXCAVATION

Early studies on braced excavations were primarily based on field observations, and focused on some key areas related to their performance during construction as given below:

1. Base instability caused by bottom heave: Researchers studied the stability of braced excavations in clay, particularly the potential for bottom heave, where the bottom of the excavation experiences upward movement due to excessive lateral pressures.

Terzaghi (1943) conducted early analyses on the factor of safety of long braced excavations against bottom heave. The failure surface for such cases in a homogeneous soil was examined and is shown in figure 2.1, considering the width of the cut (B), the depth of the cut (H), and the thickness of the clay layer below the base of the excavation (T).

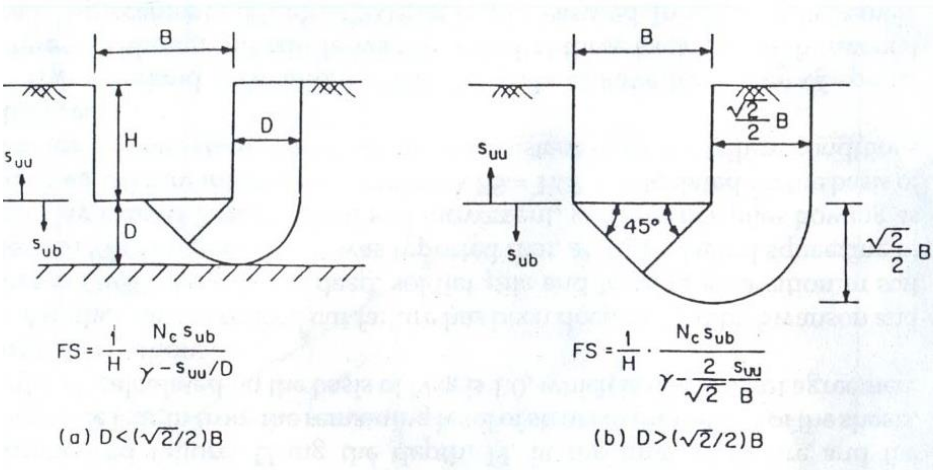


Figure 2.1 – Geometry of excavation and parameter of soil strength for safety factors. (Terzaghi (1943))

2. Earth pressure on braced walls: The lateral earth pressure exerted on the braced walls was a critical parameter in evaluating the stability of the excavation system. **Terzaghi and Peck (1967)** presented the semi-empirical apparent pressure envelope for different types of soil as a function of earth pressure co-efficient, unit weight and depth of excavation. This envelope is used to predict the maximum strut load that can be expected in the bracing of a specific excavation cut. It is essential to note that this envelope does not represent the actual distribution of earth pressure in the excavation, but rather provides an upper bound for the strut loads that may be approached but not exceeded during the excavation process. The method proposed by Terzaghi and Peck has been subject to evaluation by various researchers, including **Wong et al. (1997) and Ng (1998)**. By evaluating the semi-empirical apparent pressure envelope, researchers can refine and improve the understanding of braced excavation behavior and the accuracy of predicting strut loads in different soil conditions and excavation geometries. Such studies contribute to advancements in geotechnical engineering and the design of safe and efficient braced excavations.

3. Ground settlement adjacent to excavations and lateral movement of support systems for different soil type and excavation geometry: The impact of excavations on the surrounding ground, particularly in terms of settlement, was closely monitored to assess potential effects on nearby structures and utilities and the lateral movement of the support systems, such as sheet piles, soldier piles, or diaphragm walls, was a crucial factor in understanding the overall behavior and stability of the excavation.

Peck (1969) summarized a large number of field observations on ground surface settlement around braced excavations in soft to medium / stiff clay and also sand in a graphical form. The method described in the analysis is applicable for predicting settlement profiles in braced excavations, particularly for spandrel type settlement profiles. The settlement curve is categorized into zones I, II, and III based on the soil type and the construction quality. The data used to develop this figure are from case histories prior to 1969, where excavations were supported by sheet piles or soldier piles with lagging. The figure proposes that for very soft to soft clay, the maximum ground settlement is approximately 1% of the maximum depth of excavation. Additionally, the sideways zone of influence extends up to two times the maximum depth of excavation. It is important to note that with the advancement in technology and the use of newer construction methods, such as diaphragm walls, generally, the maximum settlements are of smaller magnitudes than those explained in the analysis. However, the method introduced by Peck is considered as a pioneering work in predicting ground settlement behavior in braced excavations.

Lambe (1970) conducted research on braced excavations, the primary focus was on the design and analysis of deep excavations and the support systems used to ensure their stability. The study explored various factors influencing soil movements during excavation and delved into the engineering aspects of deep excavations. Lambe included three case histories of excavations for the MBTA subway in Boston. For each case, he applied the state-of-the-art design and analysis techniques and compared the predictions to the actual measured performance during construction. The study concluded that, at that time, the state of the art for the design and analysis of braced excavations was not satisfactory, as there were difficulties in accurately predicting support system loads and ground movements. This highlighted the need for improved methodologies and understanding in the field of deep excavation design. Lambe suggested that two promising approaches for gaining a better understanding of deep excavation performance were (a) finite element method and published case histories.

Goldberg et al. (1976) written a report for the FHWA (Federal Highway Administration) consists of three volumes and serves as a comprehensive source of information on lateral support systems for deep excavations. The three volumes cover design recommendations, design considerations, and construction techniques related to braced excavations. To compile their recommendations and guidelines, the authors relied on information gathered from the measurements and performance of 63 case histories. These case histories provided valuable data on the behavior of lateral support systems and ground movements during excavation. Based on this data, the authors made estimations for maximum horizontal wall movements, maximum ground settlements, and the shape of the settlement profile of the ground surface adjacent to excavations. Through their analysis, Goldberg et al. established correlations between the magnitude of deformations and settlement profiles, and excavation depth and soil type. This correlation allowed them to provide valuable insights into the behavior of deep excavations under different geological conditions and construction scenarios.

O'Rourke (1981) conducted a study focused on examining ground movements associated with braced excavations and the construction activities related to them. He emphasized the significance of site preparation activities in influencing ground movements during the excavation process. O'Rourke identified several site preparation activities that could cause ground movements, such as relocating and underpinning utilities, dewatering, constructing support walls, and installing deep foundations. These activities can have an impact on the behavior of the surrounding soil and affect the stability of the excavation. The study also investigated the relationship between the deflected shape of the excavation support wall and the ratio of horizontal to vertical movement of the ground surface. O'Rourke analyzed performance data from seven case histories to draw conclusions about this relationship. He found that the ratio of horizontal to vertical movements of the ground surface is approximately 1.6 for pure cantilever deformation of the wall and 0.6 for pure bulging deformation of the wall. Furthermore, O'Rourke examined the effects of brace stiffness, pre-stressing of braces, and the timing of brace placement. He observed that the effective stiffness of braces could be significantly lower (as low as two percent) than the ideal stiffness (AE/L) due to factors like compression in connections and bending of braces. These findings underscore the importance of considering the real-world behavior of support systems and braces in deep excavation design.

Clough and O'Rourke (1990) investigated movements resulting from deep excavations by analyzing data from various case histories and previous studies. They categorized the movements into two types: movements directly related to the excavation and support process and movements caused by auxiliary construction activities. By examining and summarizing movement data from case histories, Clough and O'Rourke proposed that the settling profile for an excavation in sandy soil or stiff clay is triangular and at the wall will the maximum ground surface settlement will occur and they further observed that the associated movements extend to around 2 to 3 times of excavation depth. Further they proposed that for soft to medium clay the settling profile is trapezoidal and the maximum settlement in normally occurs few distance away from the wall and the influence zone extends up to 2 times the maximum excavation depth. The settlement at various locations can be estimated if the maximum ground settlement is known. Their analysis and findings contributed to a better understanding of the behavior of deep excavations and the factors influencing ground movements during the construction process. One of the key conclusions drawn from their study was that movements due to deep excavations could be reasonably predicted if the significant sources of movement were properly considered. This highlights the importance of taking into account various factors, such as excavation methods, support systems, and auxiliary construction activities, when designing and managing deep excavation projects to mitigate potential adverse effects on the surrounding soil and structures.

Peck (1969), Lambe (1970), Goldberg et al. (1976), O'Rourke (1981), and Clough and O'Rourke (1990) all contributed significantly to the geotechnical engineering profession's understanding of deep excavations. The following are some main outcomes from these reviews:

(i) Soil type is a major factor in deep excavation performance.

(ii) Prompt installation of support is crucial for reducing neighboring movement around braced excavation.

(iii) Prestressing supports is useful for reducing movement.

(iv) Deep excavation dewatering is a common cause of settlement.

(v) Deep excavation performance is influenced by construction sequencing.

(vi) Workmanship is a key aspect in deep excavation performance.

(vii) Support spacing is a key consideration in deep excavation performance.

(viii) Deep excavations can be affected by high initial lateral soil stresses.

(ix) Wall type can have a significant impact on deep excavation performance.

2.3 CASE STUDIES OF DEEP EXCAVATION

In this section, field performance studies of deep excavations have been investigated. Measured field performance studies of deep excavations are significant for two reasons. The first is that field performance studies provide insight obtained from real-world excavation design and construction experiences. Unlike analytical investigations, the performance seen in a deep excavation is not dependent on assumptions, approximations, constitutive models, or formulations. When careful observations of construction operations, site conditions, and other factors are combined with careful measurements of performance, a great deal may be learnt about the behavior of deep excavations. Another reason to examine field performance studies is to learn about variables that are not typically modeled analytically or cannot be modelled properly. Deep excavation finite element calculations, for example, do not often model the construction of support walls or the installation of tieback anchors and also workmanship, for example, is difficult to model analytically. A detailed review of case study has been tabulated in table 1

Table 1 – Field Measurement Studies

Author	**Nature of Study**	**Findings**
Som and Raju (1989)	Studied the ground settlement, wall deflection, earth pressure of braced cuts in soft clay from extensive measurement during Calcutta Metro Construction.	Major settlement of ground was limited within a distance of three times of cut depth. Increase in wall penetration depth provided fixity of the wall at toe and there would be no effect on excavation above lowest strut level due to increase in embedded depth of wall.

Gill and Lukas (1990)	Observed ground movements are presented in deep excavations at eight building sites in downtown Chicago. These excavations were supported by various support systems such as steel sheeting, soldier piles, or diaphragm walls. During the study, the researchers compared the measured displacements with the predicted displacements.	Measured movements closely matched the movements predicted by Clough and Mana's procedure for systems with rakers. The measured movements were approximately half of the predicted movements when cross-lot braces were employed. The stiffness of the support system and the factor of safety against basal heave were the most crucial parameters for minimizing ground movement.
Houghton and Dietz (1990)	Measured performance of a large deep excavation carried out for the 125 High Street project in Boston was presented. To support the excavation, a combination of soldier piles with lagging, bracket pile underpinning, and tangent piles with tieback anchors was utilized.	Global stability issues that led to significant wall deflections and reductions in tieback loads on one side of the excavation. Suggestion has been provided for conducting borings in the area of tieback anchorage during the design phase of deep excavations to address potential issues proactively.
Swanson and Larson (1990)	Failure investigation of a braced excavation in the Washington Metro construction endeavor was studied.	According to their findings, the failure occurred because the undrained shear strengths (S_u) were overestimated.
Som (1991)	The research was based on extensive measurements of ground movement and building settlement at different sections of the Kolkata underground	Additional ground settlements occurred when excavations were kept open at the final cut level or at any intermediate level for extended periods.

	metro rail excavation focusing on the effect of time delay on deformations of braced cuts in soft clay., which took place during the late 1980s.	
Finno and Harahap (1991)	Field investigation to measure ground movements, pore water pressures, wall deflections and impact of the construction sequence on the excavation's performance of 40 feet deep HDR-4 excavation, in saturated Chicago clays has been studied.	Deformations of the sheet pile closely matched the observed deformations during different stages of the excavation. When evaluating potential ground movements for excavations in soft to medium clays anisotropy should be considered.
Finno and Chung (1992)	Field movement of the ground near extensive diggings by investigating the impact of various construction activities, such as installation of wall and deep foundation, repetitive phases of excavation, reinforcement installation, and subsequent removal of support struts in soft Chicago clay has been studied.	Construction activities that alter the stresses in the ground should be taken into account when estimating ground movements around deep excavations. The use of finite element analyses to predict excavation performance has limitations, primarily related to the ability to accurately model construction activities.
Lien et al. (1993)	Conducted a study focusing on a 6.4-meter-wide and 8.5-meter-deep braced excavation in Detroit soft clay in the United States. The excavation was constructed in twelve stages.	During the excavation up to a depth of 5.18 meters, the rate of ground deformation was relatively low, measured at approximately 0.164 millimeters per day. However, beyond that depth, there was a significant

		increase in the rate of ground deformation, reaching about 0.677 millimeters per day.
Reinfurt et al. (1994)	Conducted a study, wherein they detailed the design, construction, and monitoring of temporary shoring systems for 30 feet deep excavation of two stations of the Metro Link St. Louis Light Rail System in soft to medium stiff soils.	Actual ground movements were less than those predicted using the methodology by Clough and O'Rourke (1990). Maximum movements and settlements recorded were essentially less than 0.2% of the excavation height (H). Some consolidation settlements during the construction process, which was expected in soft to medium stiff soils.
Ng (1998)	Conducted a study, reporting field measurements by investigating earth pressure and water pressure within the pores on either side of the structure, load on the support props, sideways shifting of the wall, sinking of the ground behind the wall, and upward movement at the excavation's base (basal heave) from a 10-meter-deep excavation in over-consolidated stiff-fissured gault clay in Lion Yard, Cambridge.	According to the observed value, construction of the diaphragm wall led to a significant reduction in the lateral total earth pressure at the interface between the soil and the wall. The minimal sideways stress within the ground prior to the primary excavation contributed to comparatively low support strut loads and limited sideways wall movement throughout the excavating procedure.
Thasnanipan et al. (1998)	Presented a study, focusing on the effect of different parameters on the inclination of nearby structures and the lateral movement of a diaphragm wall	The ground settlement observed after the conclusion of excavation activities was up to 16 mm. The largest lateral movement was observed at the top of the wall. The delay in

	in a braced excavation project in Bangkok.	installing struts and completing the construction had an impact on the movement of the wall, particularly in the initial excavation phase prior to the installation of the initial level of bracing.
Ou et al. (1998)	Conducted a case study of 19.7 m deep excavation at the Taipei National Enterprise Centre (TNEC) site in the Republic of China by investigating earth pressure and pore water pressure on both sides of the wall, prop load, lateral wall displacement, ground settlement behind the wall, and basal heave inside the excavation. Thirteen construction stages were required for executing the excavation.	The measured results indicated that the prop load corresponded to Peck's apparent earth pressure diaphragm. The lateral ground movement observed at a distance of 2 meters away from the wall was similar to that of the wall itself. The major influence zone of settlement trough behind the wall extended to about $2.5H_e$. There was an increase of 18.2 mm of lateral displacement due to the construction being halted for 60 days at stage 6, which occurred at a depth of 7.1 meters.
Hou (2002)	Presented Zhongsheng shopping Mall excavation project. The project was constructed using the method of central part by bottom-up method and peripheral part by top-down method. In this case excavation had been halted for 64 days at stage 4.	During no excavation stage increase of lateral displacement was around 18.2 mm. No significant bottom heave was observed when there was no excavation carried out.

Sen et al. (2004)	Presented a study, focusing on the excavation system used for constructing an underground garage in the South Boston area, adjacent to the Central Artery/Third Harbor (CA/T) Ted Williams Tunnel. The measured values were then compared with the results obtained from finite element analysis.	Observed deflection values at the final stage of excavation were less (maximum 38 mm) than the analytical values (maximum 89 mm). The stiffness of the support of the excavation system, rather than its strength, had a significant impact on the movements induced in the soil mass during excavation. Finite element analysis provided an economical bracing and wall design and proved useful in evaluating its impact on adjacent structures.
Liu et al. (2005)	Conducted a monitoring study on a 15.5-meter-deep multistrutted soft clay excavation for a metro station in Shanghai. The excavation had dimensions of 17.4 meters in width and 335 meters in length and was supported by a concrete diaphragm wall. The monitoring data collected included wall deflections, surface and sub-surface ground settlements, total pressures, and pore water pressure.	The measured ground deflections were found to be consistent with the small wall deflections. Over a 60-day concrete curing period, no significant "creep" deflection of the diaphragm wall was identified. Settlements during the curing of the middle slab were mainly attributed to primary consolidation settlements rather than secondary consolidation settlements, indicating that creep effects were not significant for the excavation.
Richards et al. (2007)	Conducted a study, presenting recorded alterations in pore water pressure and lateral soil pressure on both sides of a wall while excavating in stiff clay.	During the excavation process, there were significant reductions in pore water pressure and lateral soil pressure on either side of the wall. As anticipated, these

		declines in lateral soil pressure and pore pressure diminished as the distance from the wall increased.
Leung et al. (2009)	Discussed the design of the foundation and 11 m braced excavation for the Sheung Wan storm water pumping station at a reclaimed site on the waterfront of Victoria Harbour, Hong Kong Island. Finite Element Method (FEM) analysis was also performed.	The predicted results in the eastern side matched well with the field values of both ground deformation and wall deflection. However, in the western side, the predicted results overestimated the actual values due to the failure to consider ground improvement.

The reviews in section 2.3 have provided valuable lessons about the performance of deep excavations. Some of the key lessons learned are as follows:

(i) Consolidation plays a significant role in the performance of deep excavations. The process of consolidation, where excess pore water pressures gradually diminish and the soil settles, can greatly influence the behavior of the excavation. Understanding and accounting for consolidation effects are crucial in ensuring the stability and safety of the excavation.

(ii) Accurate characterization of soil and site conditions is of utmost importance in the design of deep excavations. Different soil types and site conditions can lead to varying behaviors and responses during excavation. Properly identifying and understanding the geotechnical properties of the soil and site will enable more accurate design and prediction of the excavation performance.

(iii) Time effect is another critical factor in deep excavation behavior, especially for soft clayey soil. Deformations and settlement in soft soils can occur over time due to time-dependent soil properties and consolidation processes. Considering time effects is vital in predicting long-term behavior and avoiding potential issues during and after excavation.

(iv) Soil improvement techniques can significantly enhance the performance of deep excavations in soft soils. Employing various ground improvement methods, such as compaction grouting, jet grouting, or soil mixing, can strengthen the soil and reduce settlement, improving the overall stability and safety of the excavation.

2.4 EMPIRICAL STUDIES OF DEEP EXCAVATION

Ground motions at the back of a supported wall are caused by imbalanced pressure caused by the removal of mass of soil within the site of excavation. Many factors influence the amplitude and distribution of ground motion, including quality of construction, Condition of groundwater and soil, geometry of excavation, sequences of excavation, excavation duration, condition of surcharge, presence of neighbouring buildings, retaining wall construction method, depth of penetration, stiffness of wall, installation and type of sideways support, and strut spacing and rigidity. Developing an approach based solely on theoretical foundations would be extremely complex. As a result, the majority of extant prediction approaches are based on field observations and local experiences. The following are few regularly used empirical methodologies in engineering practice:

Mana A. I. and Clough G. W. (1981) conducted an investigation involving 11 case studies. They presented a chart related to the maximum measured wall movements from these case histories, which were normalized by depth of excavation with associated safety factor against basal heave (**Terzaghi's (1943)**). Chart displayed the steady non-dimensional movement at high safety factors, indicating a predominantly elastic response. On the other hand, rapid increases in movements were observed at lower safety factors, which were attributed to subsoil yielding. Based on their research findings, the authors proposed upper and lower bounds for predicting the expected movement level, aiming to provide a practical way of estimating ground movements during deep excavation projects.

Bowles J. E. (1988) proposed a method to estimate the spandrel-type settlement profile induced by excavation. The steps of this method are outlined below:

1. Estimate the lateral wall displacement.

2. Calculate the volume of sideways movement of the soil mass.

3. Determine the influence zone (D) using the method suggested by Caspe (1966):

$D = (H_e + H_d) * \tan(45 - \varphi/2)$

where H_e is the final depth of excavation, and φ is the angle of internal frictional of the soil. For cohesive soil, B = H_d, where B is the excavation width; for cohesionless soil, $H_d = 0.5 * B * \tan(45 + \varphi/2)$.

4. Assume that the maximum ground settlement occurs at the wall, and estimate the maximum ground settlement (δ_{vm}) as follows:

$\delta_{vm} = 4 * V_s / D$

where V_s is the volume of lateral movement of the soil mass.

5. Assume a parabolic settlement curve. Calculate the settlement (δ_v) at a distance (d) from the supported wall as follows:

$\delta_v = \{\delta_{vm} * (d/D)^2\}$

Clough, G. W. et al. (1989) proposed a semi-empirical method for predicting movements at clay excavations by evaluating the maximum lateral wall movement (δ_{hm}) in relation to the safety factor (FS) and system stiffness (η). System stiffness (η), defined as ($EI/\gamma_w h^4$) where, EI is the flexure rigidity per unit width of the retaining wall. γ_w is the water unit weight. h is the average strut spacing. The safety factor (FS), as defined by Terzaghi (1943), is used as an index parameter. A chart presented shows δ_{hm} against system stiffness for various FS values, based on average conditions, good workmanship, and the assumption that cantilever deformation of the wall contributes minimally to total movement.

Ou C. Y. et al. (1993) studied soil movement behind retaining walls in Taipei and found that vertical soil movements could extend over a significant distance, increasing with excavation depth. They identified this area as the "apparent influence range" (AIR), within which settlement was notable and could potentially damage nearby structures. Beyond the AIR, settlement was minimal. Their research indicated that the AIR roughly aligns with the active zone, with its upper limit corresponding to the depth of the retaining wall. This suggests that soil settlement diminishes with distance from the wall and becomes insignificant beyond the wall's depth.

$AIR = (H_e+H_p)\tan(45-\Phi/2) < (H_e+H_p)$

Where He is the final excavation depth and Hp is the wall penetration depth.

Hsieh P. G. and Ou C.Y. (1998) setup a procedure for predicting ground deformation. The predicting procedures are listed as follows:

1. Perform lateral deformation analysis, such as finite element methods or beam on elastic foundation methods, to predict the maximum lateral wall deflection (hm).

2. Calculate the cantilever area and deep inward area of expected wall deflection to determine the type of settlement profile. If $A_s \geq 1.6A_c$, a concave settlement profile is used, where A_s and A_c represent areas of deep inward movement and cantilever movement in the graph of wall horizontal displacement against depth, respectively.

3. Using empirical data, calculate the maximum ground settlement. (For example, consider the link between maximum horizontal displacement and maximum ground settlement.)

4. Using the profile proposed by **Ou et al. (1993)**, calculate the surface settlement at various distances behind the wall.

Long M. (2001) analyzed 296 case histories of deep excavations to validate the findings of **Clough and O'Rourke (1990)** in the context of stiff soils. His study focused on specific soil parameters, finding that horizontal and vertical ground displacements ranged from 0.05% to 0.25% and 0% to 0.2% of excavation depth, respectively. In soft clay with poor base stability, larger movements up to 3.2% were observed. Long’s research confirmed general trends consistent with Clough’s work but noted that, in non-cohesive soils and stiff clay, excavation deformations were less dependent on wall stiffness, support systems, and types of support. However, in soft clay with low safety factors against base heave, support system stiffness had a greater impact. Long also assessed the use of Addenbrooke's flexibility number for evaluating support system stiffness, finding similar trends to Clough's but with some data variability. His research contributes to understanding deep excavation behavior across different soil types and support systems, refining engineering approaches for these projects.

Moormann C. (2004) conducted an extensive empirical study involving the analysis of 530 case histories related to retaining walls and ground movement resulting from excavations in soft soils (c_u<75kPa). Based on the gathered data, Moormann drew several conclusions:

(1) The maximum horizontal wall displacement (δ_{hm}) was typically found to lie within the range of 0.5% H to 1.0% H, with an average of 0.87% H. The location of the maximum horizontal displacement occurred at a depth of around 0.5 H to 1.0 H below the ground surface.

(2) The maximum vertical settlement (δ_{vm}) at the ground surface behind a retaining wall ranged from 0.1% H to 10% H, with an average of 1.1% H. The settlement δ_{vm} typically occurred at a distance of less than 0.5% H behind the wall. In some cases, involving soft soils, this distance could extend up to 2.0 H.

(3) The ratio of maximum vertical settlement to maximum horizontal displacement (δ_{vm}/δ_{hm}) mostly varied between 0.5 and 1.0.

(4) Ground conditions and the excavation depth (H) were identified as the most influential parameters affecting deformation due to excavation. The study found that retaining wall and ground movements were generally independent of the system stiffness of the retaining system.

(5) The results were compared to the predictions made by **O'Rourke (1993)**, revealing a significant scatter. It was noted that calculated safety factors around 1 could lead to observed maximum wall displacements (w_{max}/H) as low as 0.1%. This was in contrast to the value of about 1% expected by Clough et al. even for the stiffest support system.

The study by Moormann contributes valuable insights into the behaviour of retaining walls and ground movement during excavations in soft soils. By analysing a wide range of case histories, Moormann provided a more comprehensive understanding of the factors influencing deformation and settlement in such conditions. This information is beneficial for engineering practices and the design of excavation projects to ensure stability and mitigate potential risks.

The reviews summarized from section 2.4 show that the empirical method given by several researchers for investigating the behavior of deep excavations has been very useful. The amount of ground settlement and wall deflection with influence zone due to deep excavations can be easily computed for initial understanding by following the procedure given by different researchers. Though obtained results are not reliable due to complex nature of soil.

2.5 ANALYTICAL AND NUMERICAL STUDIES OF DEEP EXCAVATION

The purpose of this section is to examine published analytical or numerical studies of deep excavations. A detailed review of analytical study has been tabulated in table 2

Table 2 – Analytical and Numerical Studies

Author	Nature of Study	Findings
Clough and Tsui (1974)	Used finite element modeling to compare the performance of tied back walls with braced walls. Parametric finite element modeling used to investigate the effects of pre-stressing, anchor stiffness, anchor spacing, wall rigidity, and excavation depth on tied-back walls and further used model testing and analytical investigations to evaluate the implications of the plane strain assumption and the use of planar walls on the outcomes of 2-D finite element analyses.	Tied-back walls are not necessarily superior to braced walls. Excessive excavation might easily result in multiple motions of neighboring ground and wall. Widely spaced tiebacks can result in high concentrations of ground pressure at anchor levels. Further, presented a design chart that enables for the estimation of inaccuracy in planar strain assumption for tied back excavations.
Clough and Hansen (1981)	Used limit equilibrium calculations of basal stability and finite element analyses of deformation to investigate the effect of anisotropy on the performance of braced walls.	Factors of safety calculated for basal heave without taking anisotropy into consideration can be 50% too high. Parametric analyses revealed that lateral wall motions and ground settlements could be greater due to anisotropy.
Mana and Clough (1981)	Used field performance and parametric finite element research to establish a	Amplitude of movements could be related to the factor of safety against basal heave. Soil modulus

	simplified method for predicting movements in braced excavations in clay.	is a critical element in the performance of braced excavations in clay.
Kaiser and Hewitt (1982)	Used numerical and simplified flow models to demonstrate the impacts of seepage on resulting water pressure, active and passive earth pressures, and piping and bulk heave potential.	Seepage greatly reduces the available passive resistance in front of a wall's toe. Assumption of flow pattern for homogenous isotropic environments made in many designs methodologies' may result in risky designs.
Milligan (1983)	Conducted an analytical study of the deformations behind flexible retaining walls anchored at the top. They also studied the deformations using model experiments with thick sand.	Settlement profile behind a flexible wall in soft clay should be roughly the same as the shape of the deflected wall. A velocity field method was suggested for anticipating deformations behind flexible walls anchored at the top by the wall's deflected shape.
Potts and Fourie (1984)	Conducted finite element analysis (FEA) of single propped excavations. In the FEA, an elasto-plastic model with Mohr-Coulomb failure criteria was used.	K_o of soil has just a minor influence on wall deformation for backfilled walls. On the contrary, K_o has a substantial effect on excavated wall deformation. The soil behind the wall had active failure at small excavation depths before the soil in front of the wall experienced passive failure.
Broms et al. (1986)	Used finite element studies to investigate three strategies for stabilizing braced excavations in soft ground. The investigated methods included, jet grouting between the support walls,	All three strategies increased the excavation's stability. Excavation under water proved to be quite effective. Transverse diaphragm wall members beneath the excavation and between the support walls could be another feasible way for braced excavation stabilization.

	driving timber piles in front of the support walls' toes, and excavating under water.	
Clough et al. (1989)	Proposed a way for designing excavation support systems with movement control as a key focus. Processes for estimating excavation support system movements has been addressed. Case studies were also provided to demonstrate the benefits of the proposed strategy.	Proposed method provides a method of developing support systems based on deformation criteria rather than solely on stability requirements. A chart has also been demonstrated for determining settlements in loose sands caused by sheet pile drive.
Wong and Broms (1989)	Used the finite element method to investigate the effect of undrained shear strength, excavation breadth, excavation depth, wall stiffness, wall penetration, and depth to hard stratum on lateral deflections of braced or anchored sheet pile walls in clay. Straightforward method for estimating lateral deflections was also proposed.	Factor of safety against basal heave governs settlements and lateral deflections using finite element calculations. Wall stiffness and strut spacing are critical factors. Lowering the gap between struts helps to limit movement.
Borja (1990)	Investigated issues related to accurate incremental excavation modeling. A new finite element program for soil-structure problems that used the Modified Cam Clay model to perform plane strain, axisymmetric, and 3-D drained and undrained assessments of trial excavation problems in clay was proposed. Finite	In order to analyze the impact of numerous parameters on excavation performance, analytically correct and accurate algorithms are required.

	element program was used to examine a San Francisco braced excavation case history.	
Goh (1990)	Used the finite element method to conduct parametric investigations on the impacts of wall characteristics, depth to competent soil, excavation width, and wall embedment on deep excavation stability in clay. A method for forecasting the basal stability of braced excavations in soft clay has been demonstrated.	Thickness of the clay layer beneath the excavation, wall embedment depth, and wall stiffness are all critical elements influencing basal stability.
Athanasiu et al. (1991)	Based on Bjerrum's work for clays, provided an anisotropic kinematic strain hardening model. Analyzation of the top-down construction of the Old Viking hotel and an anchored sheet pile wall for the excavation of the Bank of Norway in Oslo, Norway, using the finite element program SSFEAX and the novel model has been done.	Finite element analysis produced satisfactory findings.
Finno et al. (1991)	Evaluated the current state of the art through performing parametric analyses of the behavior of the HDR-4 braced excavation with a coupled finite element formulation. The effects of the constitutive model, boundary conditions, construction	The behavior is controlled by soil response on the passive side of the support wall, anisotropy is a crucial feature to consider, and sheet-pile deflections may be reliably estimated during the excavation process. Predicted settlements differ from observed movements due to substantial

	method, and over-excavation were all considered.	stresses and soil models' failure to describe strain localization.
Ho and Smith (1991)	Investigated the impact of construction methods on braced excavation performance using a finite element program and an elastic-plastic soil model on the Vaterland I and HDR-4 excavations.	Finite element calculations can accurately represent the majority of the physical features observed in deep excavations.
Powrie and Li (1991)	Used an elastic-plastic soil model to perform finite element assessments of four braced excavation construction scenarios. The sequencing and timing of strut installation varied between instances. Comparison of settlement profiles to those of Peck (1969) has also been done.	Effect of increasing wall stiffness can be nearly negligible depending on construction technique, and they concluded that knowing construction technique is at least as significant as soil characteristics when assessing deep excavations.
Whittle (1993)	Describes the use of finite element analysis to model the top-down construction of a seven-story underground parking garage at Boston's Post Office Square.	While there was a reasonable level of agreement, it was emphasized that a thorough characterization of engineering properties throughout the entire soil profile was crucial.
Balasubramaniam et al. (1994)	Investigated the performance of six deep excavations in Bangkok subsoils using various support systems and construction methods. This study paper also includes parametric finite element simulations of the effects of pre-loading, barrette pile and	Analytical results were found to be generally consistent with observed behavior. Deformations are controlled by the rigidity of the retaining wall and bracing devices. Diaphragm walls performed better (with less movements) than sheet-pile walls, and that wall embedment depth

	foundation pile installation, embedment depth, and surcharge.	was a more significant performance component for sheet pile walls than for diaphragm walls.
Fourie (1994)	Investigated the influence of strut stiffness, wall stiffness, pre-stressing, depth of soft clay below excavation, and excavation breadth on the Vaterland, Norway braced excavation in soft clay using the finite element method. In the investigation, the Modified Cam Clay model was used.	Peck's apparent earth pressure diagrams were not always conservative and should be used with caution. Depth of the soft clay layer beneath the excavation had an impact on the pattern of ground movements.
Goh (1994)	Used repeated finite element studies with factored strength and a constant modulus to strength ratio to explore the basal stability of braced excavations in soft clay. Large displacements have been observed to define instability.	When the thickness to hard stratum is smaller than the excavation width, Terzaghi's method for factor of safety may be cautious. Increasing wall stiffness raised the factor of safety against basal heave marginally.
Ou and Lai (1994)	Investigated a rational technique for calculating soil parameters for deep excavation finite element analyses. Further, how consolidation affected wall deformation and settlement has been also studied.	For excavations with long construction periods, consolidation analysis findings agree better with observed behavior than undrained analysis results. Pore water pressures beneath excavation diminish rapidly and subsequently rebound over time.
Rodriguez-Ortiz (1994)	Investigated tied-back and braced excavations using a 1-D beam on subgrade program.	Stiff walls with tie back anchors restrict deflections and that wall motions can double if support stiffness is inadequate and pre-loading is zero. Rigid walls exhibit cantilever type

		displacements when compared to flexible walls, which tend to bulge near the excavation's bottom.
Goh et al. (1995)	Created a neural network for predicting wall displacements in braced excavations. Performance data and parametric finite element studies were used to train the neural network.	After learning from finite element study results, the neural network produced reasonably accurate wall displacements. Neural networks are a beneficial tool because of their capacity to absorb fresh data and make predictions fast.
Hashash and Whittle (1996)	Used nonlinear coupled undrained finite element studies to explore the impact of wall embedment depth, support conditions, and stress history profile on the performance of a braced diaphragm wall in Boston Blue Clay.	Limit equilibrium methods for evaluating the factor of safety against basal heave overestimate the stable depth because they do not account for strength anisotropy and the rise in strength with depth. Deep-seated movements under the excavation grade govern wall deformation and ground settlements.
Ou et al. (1996)	Investigated the effect of corners on excavation deflection behavior by evaluating a typical excavation in a soft to medium stiff clayey subsurface stratum. Presented a relationship based on 2-D finite element results for determining 3-D maximum wall deflection.	Corners restrained wall deformations in 3-D analyses, and that the restraining impact of corners reduced as the distance from a corner increased.
Hsieh and Ou (1998)	Investigated an excavation case study utilizing finite element analysis and the modified hyperbolic model.	Employing a modified hyperbolic model, finite element analysis produced reasonable agreement with observed wall and soil behavior.

Jen (1998)	Conducted extensive parametric simulations to examine the impact of various crucial factors, including excavation geometry, support system, and soil mass stress history profile, on the prediction of ground movements caused by excavations.	Depth of bedrock played the most significant role in influencing the distribution of ground movements. Additionally, excavation dimensions, uncertainties in the stress history profile, and support stiffness were critical factors contributing to the magnitude of displacements.
Addenbrooke et al. (2000)	Conducted 30 nonlinear finite element simulations of undrained deep excavations in stiff clay. The study focused on examining the influence of varying initial stress regimes and prop stiffness values on the internal supports of the excavation.	Support systems with the same displacement flexibility number, under a given initial stress regime and prop stiffness, produced nearly identical maximum lateral wall deflections and ground surface displacement profiles after completing an undrained excavation in stiff clay.
Hashash and Whittle (2002)	Investigated effective stress routes around a multi-propped excavation using a finite element program by adopting the MIT-E3 soil model. The MIT-E3 model used can represent soil strain dependency, stress path dependency, and strength anisotropy.	Soil in front of the wall (1E) follows the typical route of plane strain passive shearing. The major principal stresses are observed to be oriented towards the lowest level of the strut, and an underlying compressive arch distributes loads onto the embedded section of the wall. Subsequently, during the subsequent excavation stages, a deeper arching mechanism develops after installing the lowest strut.
Tefera et al. (2006)	Studied the ground settlement and wall deformation of a sheet pile wall during different stages of excavation using a large-scale model test in dry sand bed and	The simulated and measured results compared reasonably well, but some discrepancy found, could be corrected by using soil parameters

	compared the results with those of FE analysis, based on an elastoplastic soil model calibrated by conventional triaxial and oedometer test.	obtained from triaxial and oedometer test at appropriate stress level.
Finno et al. (2007)	Presents the results of 150 finite-element simulations conducted to define the effects of excavation geometry, i.e., length, width, and depth of excavation, wall system stiffness, and factor of safety against basal heave on the three-dimensional ground movements caused by excavation through clays.	Wall deflection values from 3D simulation are smaller than that of plane strain simulation when length of wall to excavation depth ratio, length of wall to excavation width ratio values are small or wall system is stiff.
Kung et al. (2007)	Proposed a simplified semi empirical model for predicting maximum surface settlement, maximum wall deflection and surface settlement profile due to excavation in soft to medium clay.	Predicted results were compared well with measured values for validation.
Hsiung (2009)	Presented a complete case record of excavation in sand in Kaohsiung, Taiwan. The excavation was approximately 20 m deep, supported by propped diaphragm wall 1 m thick and 36 m long and excavated in medium dense silty sand. Numerical analyses were conducted to evaluate the influences of soil elasticity, creep and soil–wall interface.	Subsidence of the ground surface behind the diaphragm wall extended to a distance approximately three times the maximum depth of excavation and significant time dependent deformation (creep) was observed though excavation induced seepage had limited effect on ground deformation.

De Lyra Nogueira (2009)	Conducted coupled FE analysis on braced excavations with different constitutive models and different excavation rates	Choice of the constitutive model affects the magnitude and distribution of excess pore pressure.
Dan and Sahu (2012)	Proposed a theoretical approach to predict wall deflection and ground surface deformation assuming the plastic deformation mechanism of soil.	An equation was developed to estimate maximum ground deformation by equating total loss of potential energy with virtual work done of soil within influence zone.
Chowdhury et al. (2013)	Undertook a study focused on the development of a numerical model for analysing braced excavations. The aim was to estimate crucial design parameters that significantly affect the behaviour of excavations. The results obtained from the numerical model were then compared with those from a documented case study of a braced excavation in sand.	Among the different combinations they examined, a specific type of strut arrangement paired with a particular ratio of embedded depth to excavation depth produced the most favourable outcomes in terms of excavation behaviour. As a result of their findings, presented a design guideline derived from the outcomes of the numerical study.
Chowdhury et al. (2016)	Investigated both performed numerical and experimental analysis to study the effect of fine content in sandy soil on the behaviour of braced excavation in terms of four design factors: strut force, bending moment developed in wall, wall displacement and vertical ground deformation.	With increase of fine grains content up to 40% in sand all four design factors increase considerably but thereafter rate of increase is negligible.
Goh et al. (2017)	Performed a series of 2D and 3D finite element analyses using hardening soil	A simple equation to estimate maximum wall deflection has been developed considering

	model to investigate the influences of wall stiffness, soil properties, excavation length and depth, wall embedment depth on the maximum wall deflection induced by braced excavation.	different ratios of excavation length over excavation depth.
Xiang et al. (2018)	Performed finite element analysis accounting excavation geometrical parameter, soil parameters, strut stiffness and other related parameters. Further, parametric analyses were also performed to investigate the influence of the various design variables on wall deflections.	Based on these results, for estimating the maximum wall deflection a multivariate adaptive regression splines model was developed.
Dan and Sahu (2018)	Proposed a theoretical method to estimate ground and wall deformation at any location around braced excavation using principle of minimum potential energy approach. Important soil parameter (strength and elastic parameter) and excavation parameter (excavation depth, width, wall embedment length, rigidity of wall) were incorporated for characterization of soil deformation.	The suggested technique performs effectively under varying field circumstances. It can serve as a simplified method to estimate ground and wall deformations at different depths and horizontal distances.
Imtiaz et al. (2019)	Studied the effect of strut arrangement and strut stiffness on soil deformation behaviour during excavation.	As the spacing between two consecutive struts increased, strut force in top strut also increased. An optimum strut arrangement to obtain least deformation values has been proposed.

Teparaksa and Teparaksa (2019)	Performed FEM analysis to predict diaphragm wall deflection for two projects and compared the results with observed values.	FEM analysis using Mohr Coulomb model predicts well but in case of one project where construction was fast, the measured values were less than predicted values.
Dong and Jia (2020)	Optimized critical parameters (like pile diameter, embedment ratio etc) of supported retaining piles in granular soils using finite element method.	Equations were presented to calculate maximum vertical displacement of surrounding soil and horizontal displacement of pile.
Dan and Sahu (2022)	Proposed formulations for prediction of earth pressure and ground deformation analysing equilibrium condition of soil with influence zone of braced excavation. A parametric study was presented to assess the influence of different soil parameters (cohesion, angle of shearing resistance and angle of wall friction) on earth pressure distribution and ground deformation.	The proposed formulations are validated with existing case studies and found satisfactory results
Guo et al. (2022)	Examined the studies conducted on time-dependent deformation of deep braced excavations caused by soil creep.	The significance of accounting for soil creep effects in deep excavations involving soft clays has been highlighted, as such deformations could account for as much as 30% of the total displacements. The impact of soil creep on wall deflection varies based on several factors, including ground conditions, construction methods, strut stiffness, and workmanship.

Hong et al. (2022)	Conducted an investigation on the influence of spatially variable soil layers on the deformation behaviour of deep excavations. Employed a reliability analysis using a multidimensional log-normal distribution model to assess the impact.	Spatial variability of soil parameters has an adverse effect on the safety of deep excavations, and basal heave is particularly susceptible to the spatial variability of the soil.

The reviews summarized in section 2.5 show that the finite element method has been widely used to investigate the behavior of deep excavations, and that finite element analysis of deep excavations is a useful way to investigate many of the factors that control the performance of deep excavations via parametric studies. It is easier and less expensive to use finite element analysis to understand deep excavation behavior than it is to instrument and monitor deep excavations.

The following points summarize some important lessons learned from analytical studies.

(i) Consolidation has a significant impact on the performance of deep clay excavations.

(ii) Anisotropy is a crucial aspect in deep excavation performance.

(iii) Wall construction can produce large motions prior to excavation and can be a major element in deep excavation performance.

(iv) Finite element analysis is quite beneficial for examining deep excavations.

(v) Proper constitutive model selection is critical in finite element analysis of deep excavations.

(vi) Three-dimensional finite element analyses are unusual. This appears to be due to the fact that 3-D analyses are far more difficult than 2-D analysis. Moreover, 2D analysis considering unit wall length produce satisfactory results when compared to the field values.

2.6 GEOTECHNICAL CENTRIFUGE STUDIES OF DEEP EXCAVATION

To acquire dependable and regulated data for a more comprehensive comprehension of soil responses during the excavation process, it is crucial to have authentic and repeatable simulations. Employing field instrumentation during excavation proves to be the most efficient approach, but its major drawback is the limited level of replicability. Soil conditions and construction sequences vary from site to site, making direct correlation and comparison difficult. Additionally, understanding the deformation mechanisms of soils involved in field measurements is often challenging. However, field measurements remain important for calibration and verification of physical and numerical models. The finite element method is a convenient way to analyse soil-structure interaction problems and has proven to be a powerful tool for modelling complex construction processes and site-specific structural properties. However, the reliability of predicting ground movement depends heavily on the input parameters related to material properties. Sensitivity analysis can provide optimal conditions, but it may not generate the necessary database for studies unless combined with other modelling results. As an alternative method to simulate excavation behaviour, small-scale centrifuge modelling has been used. In centrifuge modelling, an artificial acceleration field is created using a centrifuge to simulate gravitational stress, allowing correct scaling of the model. This enables the simulation of excavation behaviour on a small scale, providing insights into methods of soil distortion throughout the procedure. The advantage of this method is the ability to repeat tests and continue testing until failure, which is not always feasible in the field or even with most finite element programs. Given these benefits, the use of physical modelling in a centrifuge has garnered global recognition and is selected as the main methodology for this research. Several geotechnical centrifuge studies conducted by researchers in engineering practice are presented for reference. In conclusion, a combination of field measurements, finite element modelling, and centrifuge modelling provides a comprehensive approach to understanding soil behaviour during excavation. Each method has its strengths and limitations, and together they contribute to a more complete understanding of the complex processes involved in deep excavations. Several used geotechnical centrifuge studies by researchers in engineering application are presented in table 3

Table 3 – Geotechnical Centrifuge Studies

Author	**Nature of Study**	**Findings**

Bolton and Stewart (1994)	Examines the stability and functional performance of in-situ constructed propped diaphragm walls in firm clay.	Centrifuge model tests enabled the extended-term performance of a propped diaphragm wall in firm clay to be observed within a matter of hours, a task that would require numerous years at full-scale. The observation of centrifuge model tests played a pivotal role in developing simplified behavioral patterns for analytical purposes.
Kimura et al. (1994)	Performed centrifuge experiments to examine the performance of unbraced excavations and excavations with sheet pile walls, with and without cross braces, in both normally consolidated (NC) and over-consolidated (OC) clay soils. The excavation process was simulated using an in-flight excavator, and various parameters were measured, including clay deformations, pore water pressures, and earth pressures on the wall.	State of active failure was reached with a lower level of strain compared to the passive side. Moreover, due to anisotropy, a reduced activation of lateral soil pressure on the passive side was observed. Near the retaining wall, negative pore water pressures were induced, although these were partially offset by positive pore pressures arising from the shear deformations of the clay in that region.
Richards and Powrie (1998)	Conducted a series of centrifuge model tests aimed at investigating doubly propped embedded retaining walls in over-consolidated kaolin clay. The study focused on examining the influence of several factors, including groundwater	Maximum bending moment heightened with increased wall embedment depth. Reducing the groundwater level behind the retaining wall led to a notable reduction in both bending moments and prop loads.

	conditions, pre-excavation earth pressure coefficient, depth of embedment, and propping sequence.	
Takemura et al. (1999)	Carried out a study involving a vertical excavation within normally consolidated soft clay. They replicated the construction sequence of a doubly tied wall for an open excavation using an excavator. During the experiment, they monitored ground surface settlement, earth pressure on the wall, strains along the wall, and pore water pressure in the ground to analyse the behaviour of the system.	Even a modest 1-meter embedment into the bottom sand layer considerably enhanced the stability of the excavation. Propping was found to reduce settlement; however, the study revealed that once settlement had occurred, it became challenging to recover it by increasing the strutting force.
McNamara and Taylor (2002)	Undertook a series of centrifuge experiments to examine the efficacy of piles in alleviating both vertical and horizontal ground movements behind an embedded retaining wall within soft clay. They replicated top-down basement construction scenarios using different configurations of piles.	Inclusion of piles behind the retaining wall resulted in a reduction of vertical settlement. Specifically, the use of one row of piles achieved about a 40% reduction, while two rows of piles achieved a greater reduction of approximately 60% in the short term. In the long term, the effectiveness of two rows of piles further improved and resulted in more efficient settlement reduction.
Lim et al. (2003)	Conducted an array of centrifuge experiments to explore the collective	Stiffer enhanced soil layer might lead to greater bending moments within the

	ground resistance provided to a retaining wall when various configurations of soil improvement were employed on the excavated side beneath the formation level.	retaining wall. The existence of a gap between the retaining wall and the improved soil layer was found to significantly raise ground movement during the initial excavation phase. Soil berms were demonstrated to be effective in sustaining a diaphragm wall during excavation, particularly for wider excavations.
Tan et al. (2003)	Discussed the outcomes of comprehensive centrifuge studies focused on soil improvement techniques beneath excavations. The model study involved the careful execution of various soil improvement methods.	By studying the results from the centrifuge tests, they could draw conclusions about the performance of each soil improvement method under specific conditions.
Kongsomboon et al. (2004)	Introduced a procedure for performing a centrifuge test on a stabilized excavation utilizing an embedded enhanced soil berm. To execute the tests, they employed an excavator within the centrifuge, replicating in-flight excavation, and an image processing system under a 100g gravitational environment within the centrifuge	The research aimed to provide valuable insights into the effectiveness and behaviour of the embedded soil berm as a soil improvement technique for stabilizing excavations.

The lessons learned from reviewing the references in section 2.6 are as follows:

(i) Centrifugal modeling has proven to be highly valuable in understanding the underlying mechanisms of geotechnical behavior. Additionally, the data obtained from these centrifuge tests can be used to validate numerical analyses, enhancing the accuracy and reliability of such simulations.

(ii) By applying modeling laws and scaling principles, centrifuge models can replicate correct full-size prototype stress conditions. This enables researchers to accurately simulate in-situ stresses and realistic stress histories, providing a more comprehensive understanding of geotechnical responses.

(iii) The duration needed for consolidation of the centrifuge model is significantly less than that of the prototype using diffusion modeling relationships. Due to the increased gravitational forces in the centrifuge, the entire process occurs at a faster rate, expediting the simulation of time-dependent geotechnical phenomena.

(iv) Centrifuge modeling offers significant cost advantages compared to constructing trial excavation sites at full scale. This cost-effectiveness allows for greater repeatability in testing and provides researchers with much greater control over the experimental conditions, leading to more precise and reliable results.

In conclusion, centrifuge modeling proves to be a powerful tool in geotechnical research, providing valuable insights, cost-efficiency, and enhanced control over experimental conditions compared to traditional full-scale prototypes.

2.7 SUMMARY OF THE LITERATURE REVIEW

The comprehensive literature review highlights that ground settlement, wall deflection, and strut load are strongly affected by factors such as soil characteristics, excavation depth, the support system used, and the construction sequence implemented on-site. While a substantial amount of research exists, much of it involves field studies. Numerical methods like finite element analysis have also been applied but often fail to account for time-related effects. Moreover, only a limited amount of centrifuge testing has been conducted, and these tests have primarily focused on examining how different parameters affect the deformation behavior of braced excavations, with little emphasis on time-dependent factors.

CHAPTER 3 - NUMERICAL STUDY

It has been observed from summarizing section 2.5 that the finite element method has been widely used to investigate the behavior of deep excavations, and that finite element analysis of deep excavations is a useful way to investigate many of the factors that control the performance of deep excavations via parametric studies. It is easier and less expensive to use finite element analysis to understand deep excavation behavior than it is to instrument and monitor deep excavations. Therefore, for the present study PLAXIS 2D software is used as the platform for finite element method. The above section has been categorized into two sections namely 3.1 and 3.2. In the first section i.e., 3.1 the extensive analytical study has been conducted without implementing any time dependent factor for braced excavation and in the next section extensive analytical study has been conducted with incorporating time dependent factor for braced excavation i.e., pause time during construction or construction delays, rate of excavation etc.

3.1 DEFORMATION BEHAVIOUR OF BRACED EXCAVATION IN SOFT CLAY

3.1.1 GENERAL

In present study a thorough, parametric study has been conducted using finite element analysis to address influence of various parameters on deformation characteristic of braced excavation in soft clayey deposits. The importance of correct estimation of soil parameters for braced excavation design is also documented. The analysis of typical braced excavations in soft clay is carried out using PLAXIS 2D software where soft soil creep constitutive model is used. On the basis of numerical study, a handy design guideline is recommended. Further multi-variate regression models are developed incorporating various important excavation parameters for adequate prediction of maximum wall and ground displacement along with wall and ground surface deformation profile. Here large numbers of data reported in case histories and generated artificially from FE analysis are used for formation of regression equations. Proposed model is validated comparing results from literatures not used for development of the model.

3.1.2 NUMERICAL STUDY

3.1.2.1 Site condition and sequence of analysis

The Kolkata metro excavation was performed using the cut and cover method, and vertical excavations were stabilized with the help of diaphragm walls and horizontal struts (Som et al. (2000)). Typically, the following construction sequence was followed: (1) construction of diaphragm walls, (2) excavation of soil between the diaphragm walls to a desired depth, and (3) installation of struts to support the diaphragm wall. Steps 2 and 3 are sequentially repeated until the final depth of excavation is reached.

In order to connect the study to real field conditions, a particular metro railway construction site in central Kolkata (Som et al. (2000)) is selected where a braced excavation was constructed. The soil profile at the site consists of four layers with a top 3-m thick desiccated brownish-grey clayey silt layer (L1) underlain by an 11-m thick soft dark-grey silty clay layer with decomposed wood (L2). This is underlain by a 6 m thick third layer (L3) consisting of a stiff bluish-grey silty clay with cobble. Underlying the silty clay layer is a 10 m thick fourth layer (L4) consisting of yellowish-brown sandy clayey silt. The water table lies at a depth of 2.5 m below the ground surface. The details of the soil profile and the soil properties are described in Table 4. Input parameters of SSCM are shown from column 9 to column 11 of Table 4 and the values of these parameters are obtained from Dan and Sahu (2018). The site condition and soil profiles are typical of those observed in the Kolkata municipal area.

Table 4 - Typical Kolkata soil profile (Dan and Sahu (2018))

Soil layers	Depth from ground surface(m)	γ_{unsat} (kN/m^3)	γ_{sat} (kN/m^3)	c_u (kN/m^2)	e_0	ν_s	K_0	λ^*	κ^*	μ^*
L1 - Light brown/ brownish grey silty clay/ clayey silt	0-3	19	21	40	0.55	0.3	0.79	0.080	0.016	5.30×10^{-3}

L2 - Grey/dark grey silty clay/ Clayey silt with semi-decomposed timber pieces	3-14	20	20	25	0.65	0.3	0.74	0.142	0.028	9.53×10^{-3}
L3 - Bluish grey silty clay with calcareous modules	14-20	20	20	60	0.8	0.3	0.83	0.064	0.013	4.26×10^{-3}
L4 -Yellowish brown clayey silt with sand	20-30	21	21	45	0.5	0.3	0.43	0.060	0.012	4×10^{-3}

Here, γ_{unsat} = Unsaturated Unit Weight of soil, γ_{sat} = Saturated Unit Weight, c_u = Undrained Cohesion, e_0 = Initial void ratio, ν_s = Poisson's ratio, K_0 = Coefficient of earth pressure at rest, λ^* =Modified compression index, κ^* = Modified swelling index, μ^* =Modified creep index

The constructed excavation was 30 m wide and 14 m deep. Diaphragm walls 17 m deep and 0.6 m thick were used to retain the earth. The wall was supported by horizontal struts placed at a horizontal interval of 4.25 m. The construction sequence of the excavation is shown in Figure 3.1 and described in Table 5. First the diaphragm walls were installed, and then excavation was carried out to a depth of 3 m followed by the placement of first row of struts at a depth of 2 m. Subsequently, the second stage of excavation was done to a depth of 8 m, and the second layer of strut was placed at a depth of 4.5 m from the ground surface. Thereafter, excavations were performed in multiple stages, along with the placement of the third (bottom) layer of struts at a depth of 10.6 m from the ground surface. The final depth of excavation reached was 14 m below the ground surface. The diaphragm wall is assumed to behave as a precast-concrete, elastic plate with normal stiffness $E_wA_w = 2.0 \times 10^7$ kN, flexural rigidity $E_wI_w = 6.14 \times 10^5$ kNm^2/m (E_w = Young's modulus of wall, A_w = cross sectional area of wall, and I_w = second moment of inertia of the wall section), and Poisson's ratio 0.15. The struts are assumed to be compression elements with stiffness $E_{st}A_{st} = 3.36 \times 10^6$ KN (E_{st} = Young's modulus of strut, and A_{st}= cross sectional area of strut), and the average spacing between consecutive struts s_{st} = 4.25 m.

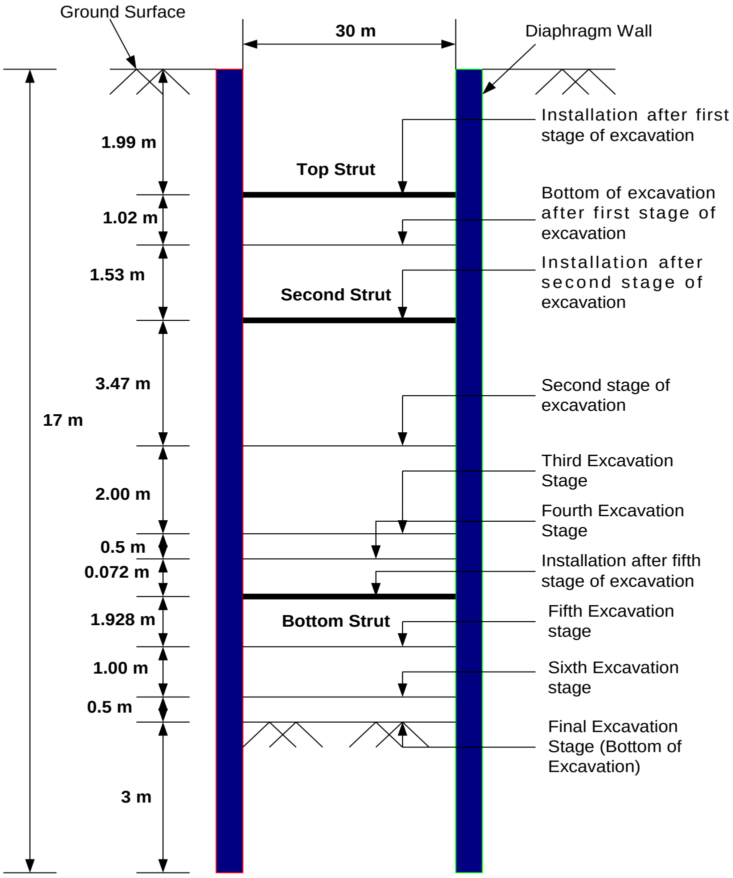


Figure 3.1 - Construction sequence and a cross section of a typical braced excavation at a site in central Kolkata

The braced excavation described in the preceding paragraphs is used as the reference braced excavation and the ensuing parametric study is performed by varying the different dimensions of this reference excavation. In the parametric study, the soil profile and properties are maintained the same for most of the simulations as that of the reference excavation (except for one set of study in which the properties were varied to study the effect of sensitivity of the soil properties on the response of the braced wall) as these represent the typical Kolkata soil profile with typical properties.

Table 5 - Staged construction sequence of a braced excavation at a site in central Kolkata

Construction stage sequence	Description of Construction	Depth of excavation/ construction from the ground surface (m)	Construction time (days)
1	Installation of diaphragm wall	0-17	7
2	First excavation stage up to a depth of 3 m	0-3	30
3	Installation of the first level of struts at a depth of 2 m	2	7
4	Second excavation stage up to a depth of 8 m	3-8	113
5	Installation of the second level of struts at a depth of 4.5 m	4.5	7
6	Third excavation stage up to a depth of 10 m	8-10	41
7	Fourth excavation stage up to a depth of 12.5 m	10-12.5	239
8	Installation of the third level of struts at a depth of 10.6 m	10.6	7
9	Fifth excavation stage up to a depth of 13.5 m	12.5-13.5	27
10	Sixth excavation stage up to a depth of 14 m	13.5-14	18

3.1.2.2 Soil constitutive model

Finite element (FE) analysis using soft soil creep model is being performed and results are validated with deformation values measured during excavation at a particular site in Central Kolkata of Kolkata metro construction during late eighties (Som et. al. 2000). To account the consolidation and creep behavior of soft clays which add significant deformation specially for those practical projects, like, Kolkata metro construction (Som et al. 2000) or excavation in Taiwan (Dang et. al. 2012) where the performance of deep braced excavation was observed to be time dependent, the Soft-Soil-Creep model (SSCM) available in the Plaxis software is used. The soft soil creep model (SSCM) available in Plaxis 2D is used in the FE analysis. SSCM simulates the monotonic creep compression of clays and possesses the following features (Ashrafi et al. (2015)) : (1) stress-dependent stiffness, (2) distinction between primary loading, and unloading-reloading, (3) time-dependent compression, (4) memory of preconsolidation pressure, (5) soil strength following the Mohr-Coulomb failure criteria, (6) yield surface adapted from the Modified Cam-Clay model, and (7) associated flow rule for plastic strains (Plaxis 2D Manual (2012) Version 8). More details on SSCM are available in Stolle et al. (1999), Brinkgreve et al. (2002) and Vermeer and Neher (1999). Further, the SSCM model simulates the monotonic deformation of clays and can evaluate ground displacement which comprises elastic as well as time dependent displacement including creep and consolidation (Figure 3.2) attributed to dissipation of pore water pressure developed due to release of lateral pressure during excavation.

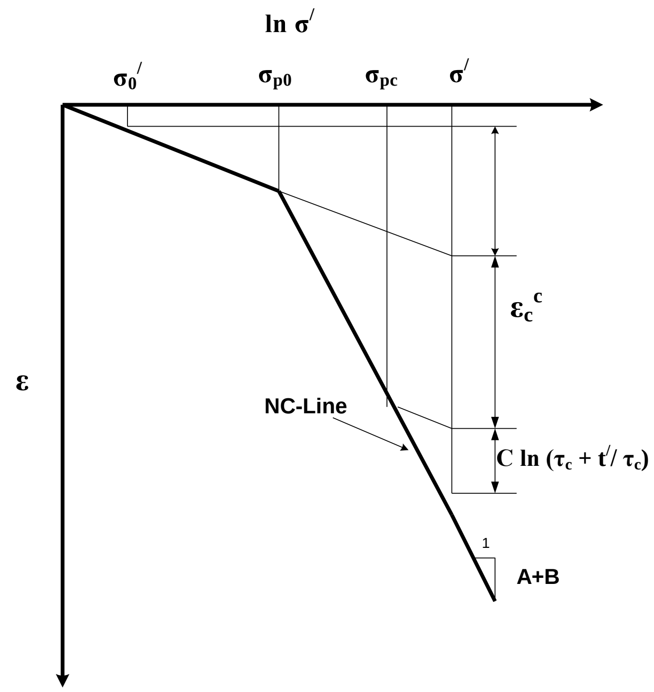


Figure 3.2 - Stress strain curve considering elastic and creep component from oedometer test

In figure 3.2, ε_c^e, ε_c^c and C ln (τ_c+t′/τ_c) are the elastic deformation independent of rate of loading, consolidation and creep after the end of consolidation. τ_c is a parameter which indicates consolidation time t_c and t′ represents effective creep time. Here A and B are used instead of swelling index and compression index respectively. Further, the parameters A, B and C of 1D model may be converted into material parameters κ^*, $\lambda^*$and μ^* to fit into the framework of the proposed model.

The relation between material parameters κ^*, λ^* and μ^* with parameters A, B and C in 1D model can be established using following equations

$$\kappa^* = \frac{3(1-\nu_{ur})}{1+\nu_{ur}} A \text{ , } B = \lambda^* - \kappa^* \text{ , } \mu^* = C$$

where, ν_{ur} is the Poisson’s ratio for unloading- reloading

3.1.2.3 Finite element analysis

Plane strain FE analysis of the braced excavation is performed using Plaxis 2D (Brinkgreve RBJ, Vermeer PA (2002)) in which one-half of the physical domain is used for the analysis considering the vertical symmetry in the problem and is shown in figure 3.3. The horizontal distance to the left vertical boundary is maintained at 65 m from the face of the diaphragm wall. The vertical distance to the bottom horizontal boundary from the base of the diaphragm wall is maintained at 13 m. These distances to the FE domain boundaries are chosen by trial and error to ensure that the boundary effects are absent. Fifteen-nodded triangular elements are used and a typical FE mesh consists of 1144 elements with 9485 nodes (mesh size is varied through a convergence study to obtain the final mesh topology ensuring that the results are independent of mesh). The ground water table is maintained at 2.5 m below the ground surface. Drainage is allowed at the ground surface along the top horizontal boundary of the FE domain and at the bottom of the mesh along the bottom horizontal boundary of the FE domain because there is a presence of sandy layer beneath the bottom soil layer considered in the FE domain (the excess pore pressure at the nodes along the drainage boundaries are set to zero). The vertical boundaries on the two sides are assumed to be sealed with no flow across these boundaries. The staged construction sequence of braced excavations is simulated in the FE analysis although the effects of stress change and disturbance on the soil properties caused by the construction (which are expected to be minimal as no driving or jacked penetration is generally involved in braced wall construction) are not considered in the analysis (i.e., in situ soil properties are used as inputs).

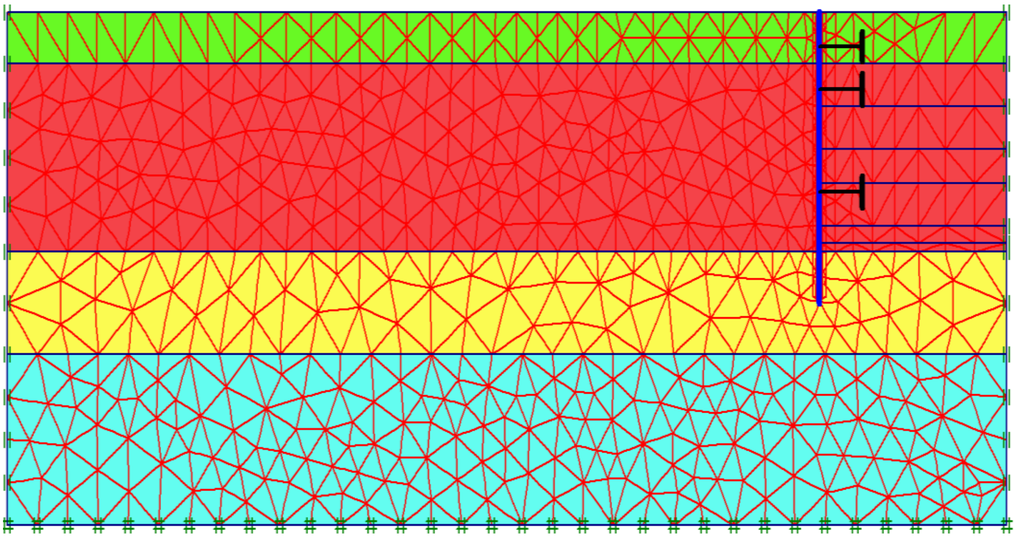
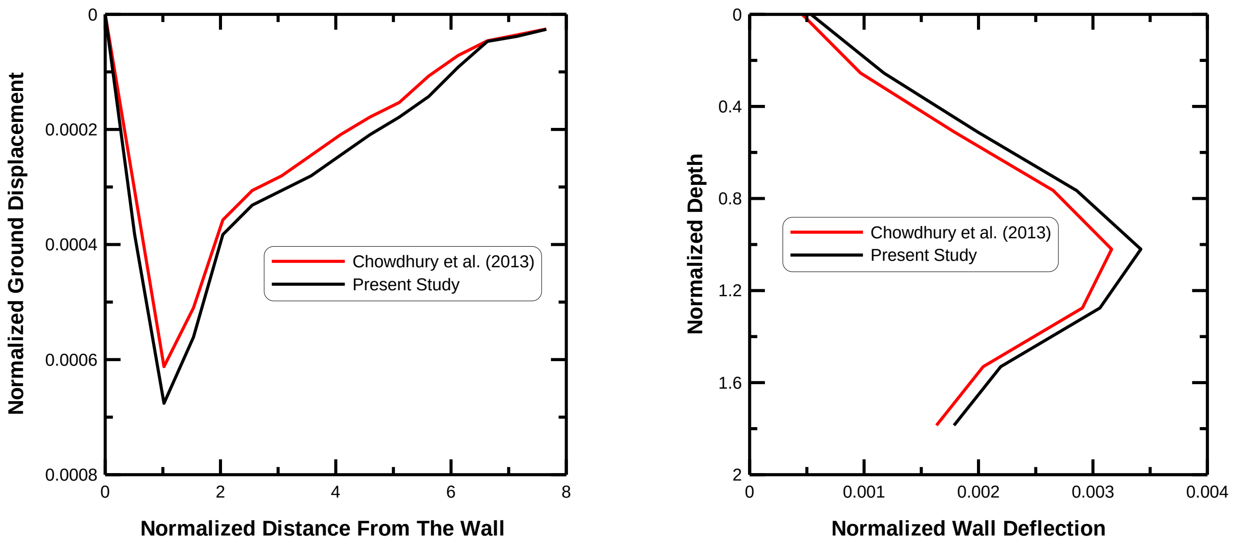


Figure 3.3 - Finite Element Model of a typical braced excavation at a site in central Kolkata

3.1.3 RESULTS AND DISCUSSIONS

3.1.3.1 Verification and Validation of the present study

The accuracy of the present FE analysis is verified by comparing the response of a braced excavation analyzed by Chowdhury et al. (2013) using the present FE model (PLAXIS 2D) with that obtained by them. The inputs to the FE analysis are based on the site conditions and geometry reported in the literature (Hsiung BCB (2009), Chowdhury et al. (2013)) and analysis was performed using the Mohr-Coulomb soil constitutive model because Chowdhury et al. (2013) also used the Mohr-Coulomb model. The predicted settlements of the adjacent ground for the final stage of excavation and the corresponding wall deflections are shown in normalized form in terms of depth of excavations in Figures 3.4(a)-(b), respectively. The deformation profiles obtained from the present FE analysis match reasonably well with those of Chowdhury et al. (2013).



(a) (b)

Figure 3.4 - Verification of accuracy of present FE analysis (a) ground displacement profile ut and (b) braced wall deflection profile (Chowdhury et al. (2013)).

The appropriateness of the present FE analysis in predicting the field behavior of braced excavations is validated by comparing the analysis results with observed ground deformation for the reference braced excavation at a metro construction site in central Kolkata (Figure 3.1 and Table 1). Reported sequence of excavation for different levels has been followed in the analysis. The predicted settlements of the adjacent ground for the third and eighth stages of excavation, along with the actual observed settlement (Som (2000)) are shown in Figure 3.5, and the match is reasonably good. Also plotted in the figure are the FE simulations performed using the Mohr-Coulomb (M-C) constitutive model. It is evident that the Mohr-Coulomb model cannot capture the field behavior and the soft soil creep model (SSCM) used in this study is more appropriate.

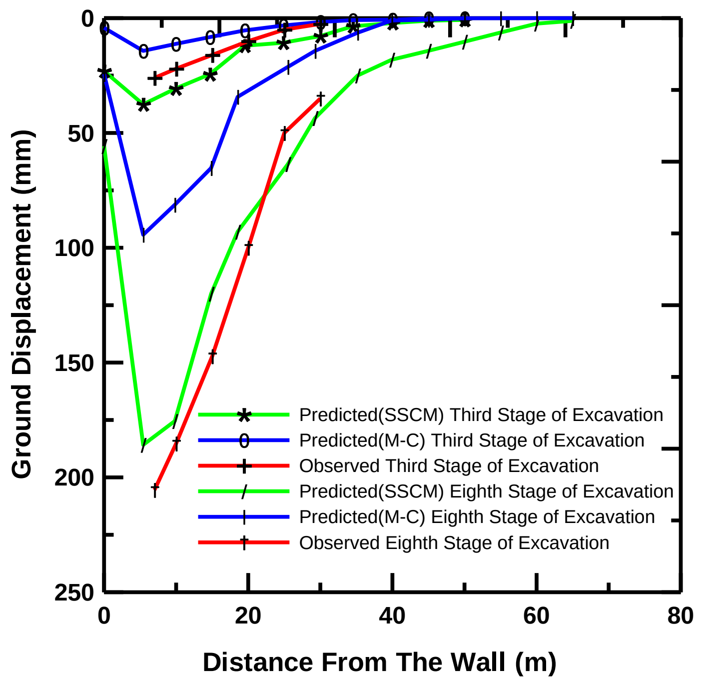


Figure 3.5 - Predicted and observed ground settlement for a braced excavation at a metro railway construction site in central Kolkata

3.1.3.2 Deformation mechanism

Ground settlement adjacent to a braced excavation is the surface manifestation of the subsoil deformation caused by the braced cut. It is found that, as the excavation proceeds, the volume of ground loss adjacent to the wall increases. The mechanism of deformation of both soil and wall for the central Kolkata site, used earlier for validation, is shown in Figure 3.6 by plotting the vector diagram of the displacement. Settlement occurs in the ground surface behind the wall because of stress release in the lateral direction and the corresponding soil deformations manifest as lateral bulging of the wall and heaving inside the excavation.

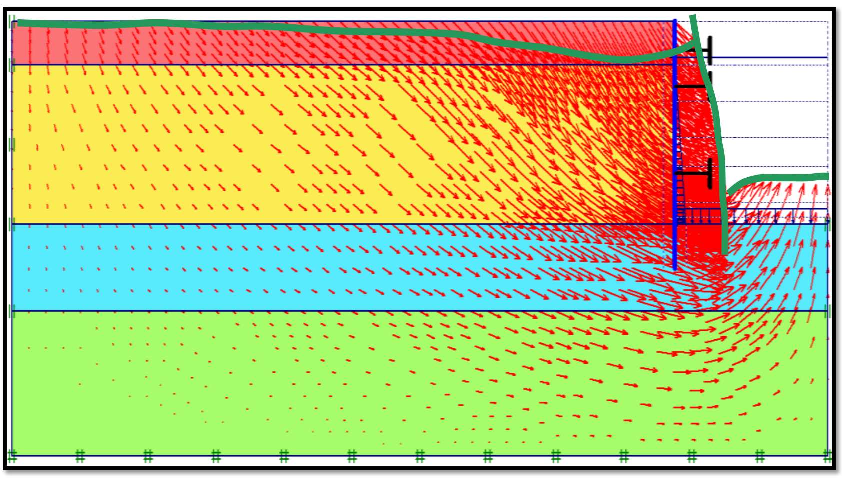


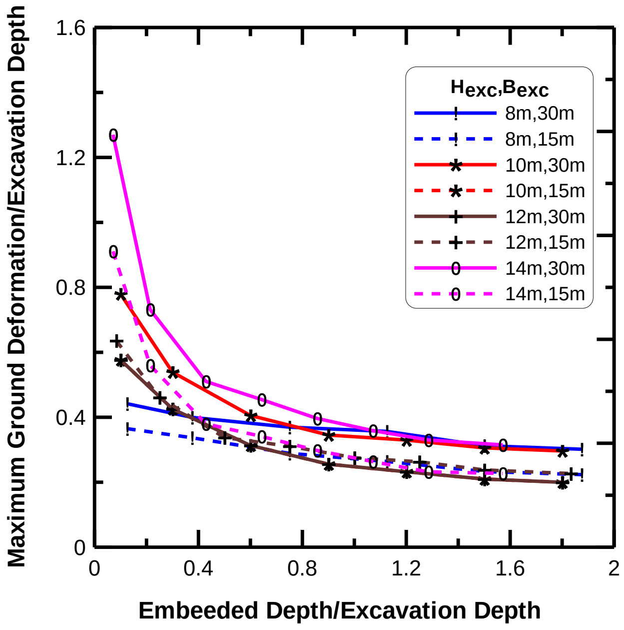
Figure 3.6 - Deformation mechanism of a typical braced excavation in central Kolkata – vectors with directed arrows show the displacements.

3.1.3.3 Parametric study

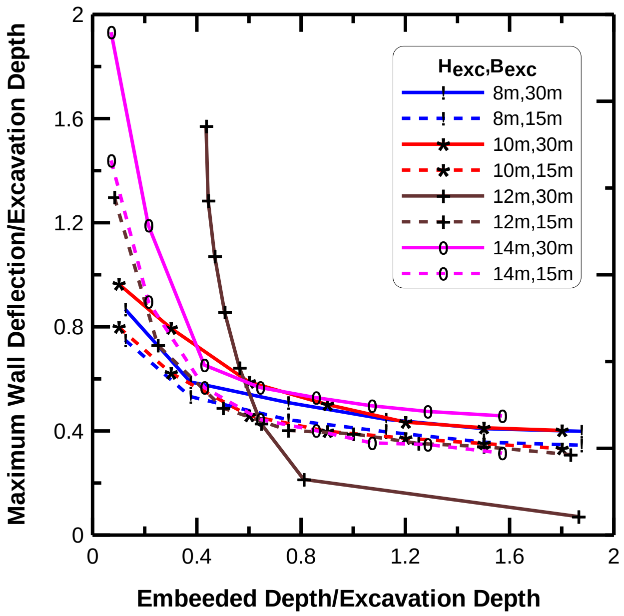
Parametric study has been conducted to investigate influence of design parameters on ground and wall deformation. Four excavation depths, 8 m, 10 m, 12 m and 14 m are considered for analysis. For excavation depths 8 m and 10 m two levels of struts are assumed to be placed at 2 m and 6 m below the ground surface. When excavation depth is beyond 10 m (for excavation depth 12 m and 14 m) three levels of struts are considered which are located at 2 m, 6 m and 9 m (for excavation depth 12 m) and at 2 m, 6 m and 11 m (for excavation depth 14 m). Three excavation widths, 15 m, 30 m and 45 m are taken in present study. For different excavation depths and widths, wall embedment depth, location of struts and diaphragm wall thickness are varied to study their impact on soil surface displacement and wall deformation. The soil profile and properties are maintained the same as that of the reference excavation used for validation of FE model. It is further assumed that the excavation is done at a standard rate of 10 days/m (0.1 m/day) and the time taken for installation of struts is 7 days.

3.1.3.3.1 Effect of wall embedment depth

In present analysis maximum ground displacement ($\delta_{g,max}$) and maximum wall displacement ($\delta_{w,max}$) are normalized with respect to the excavation depth (H_{exc}) and expressed in percentage. Wall embedded depth (D) is expressed normalizing it with respect to excavation depth (H_{exc}) and excavation width (B_{exc}). The effect of normalized wall embedment depth (D) on deformation behavior during excavation is investigated. The variations of normalized maximum ground displacement and maximum wall displacement with non-dimensional parameter ‘D/ H_{exc}’ are presented in Figures 3.7 (3.7(a) and 3.7(b)). Similar figures are plotted between normalized maximum ground and wall deformation and parameter ‘D/ B_{exc}’ and these are shown in Figures 3.8 (3.8(a) and 3.8(b)).

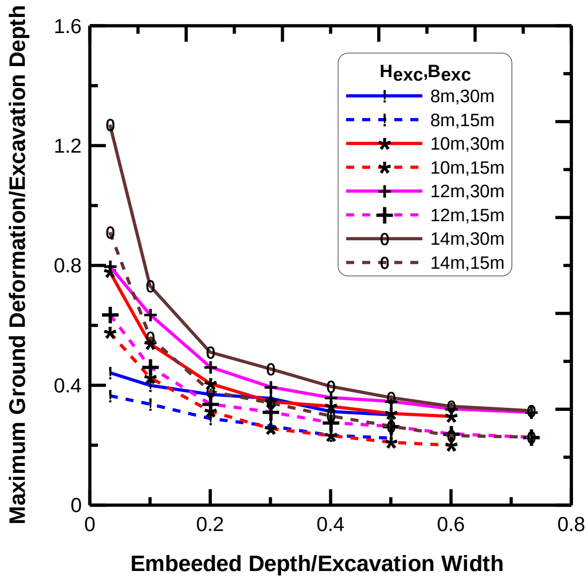


(a)

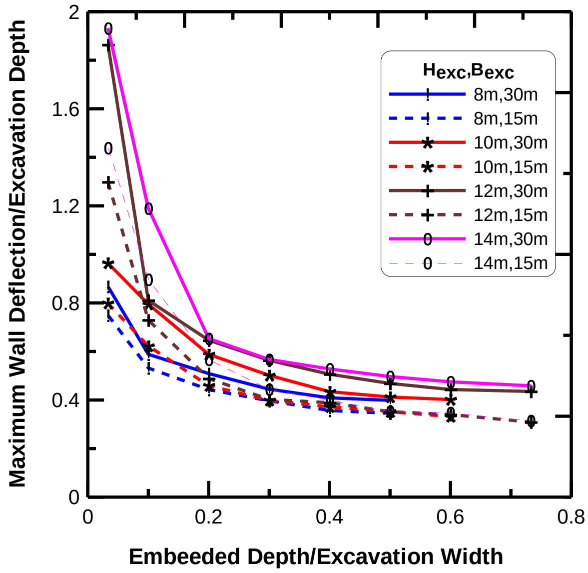


(b)

Figure 3.7 - Effect of normalized embedment depth with respect to excavation depth on (a) maximum ground deformation and (b) maximum wall deflection.



(a)



(b)

Figure 3.8 - Effect of normalized embedment depth with respect to excavation width on (a) normalized maximum ground deformation (b) normalized maximum wall deflection.

From figure 3.7(a) it is clearly seen that with increase of ‘D/ H_{exc}’, normalized maximum ground displacement decreases and then after D/ H_{exc} equal to 1.0, displacement values remain almost constant. For higher excavation depth maximum variation (between largest and smallest normalized ground deformation at different normalized embedment depth) of normalized maximum ground displacement is quite large. When excavation depth is 14 m these variations are nearly 75% for both excavation width 30 m and 15 m. While for excavation depth 8 m variations are around 30%. So, change in embedment depth significantly affects the maximum ground deformation especially for higher excavation depth. Figure 3.7(b) reveals similar kind of results. Here normalized maximum wall displacements attain minimum values at ‘D/ H_{exc}’ equal to 0.7 and then remain almost constant. It appears that wall displacement is very sensitive with respect to ‘D/ H_{exc}’ value and small increase of ‘D/ H_{exc}’ can reduce wall deformation considerably for ‘D/ H_{exc}’ value ranges from 0.125 to 0.6. From figure 3.8(a) it is observed that in all cases normalized maximum ground displacements decrease with increase of ‘D/ B_{exc}’ up to ‘D/ B_{exc}’ equal to 0.3 to 0.4 but then rate of decrement is very gradual. Figure 3.8(b) depicts that normalized maximum wall displacements reach minimum values at ‘D/ B_{exc}’ equal to 0.2 to 0.4 and thereafter no substantial variations are observed. So, from observations it can be said that for braced excavation in soft to medium clay if embedded depth of the wall is kept between 0.7 to 1.0 of excavation depth and 0.2 to 0.4 of excavation width then ground and wall deformation will be in control and damage due to deformation will be minimum. In soft to medium clay as maximum wall displacement occurs at or near the excavation depth, embedment depth is nearly equal to the excavation depth. When excavation is done in stiff soil maximum wall displacement takes place at well above excavation level. In this case required wall embedment depth for deformation control will be less.

3.1.3.3.2 Effect of strut location

Here effects of different strut locations on normalized maximum ground and wall deformations are investigated. Excavation width and wall thickness are kept at 30 m and 0.6 m respectively. Normalized wall embedment depth (D/ H_{exc}) is set at 1.0 as from previous study it is found that optimum deformation behavior is achieved at ‘D/ H_{exc}’ equal to 1.0 for excavation in soft to medium clay.

At first, study has been carried out for excavation depth 8 m and 10 m where two levels of struts are assumed to be placed. For case 1 excavation depth is taken as 8 m with 2nd strut level is fixed at 5 m and 1st strut position is varied as 0.5 m, 1 m, 1.5 m, 2.0 m and 2.5 m respectively. Similarly for case 2 same kind of strut arrangements are considered with excavation depth taken as 10 m. For these two cases variation of normalized maximum ground deformation and normalized maximum wall deflection with change of strut positions are plotted and shown in Figure 3.9 [Where NGD = Normalized Ground Deformation and NWD = Normalized Wall Deflection.]. From this figure, it is observed that minimum normalized deformation values are obtained when 1st strut is placed at 2 m level which means if 1st strut is placed at 0.2-0.25 times the depth of excavation optimum values can be obtained.

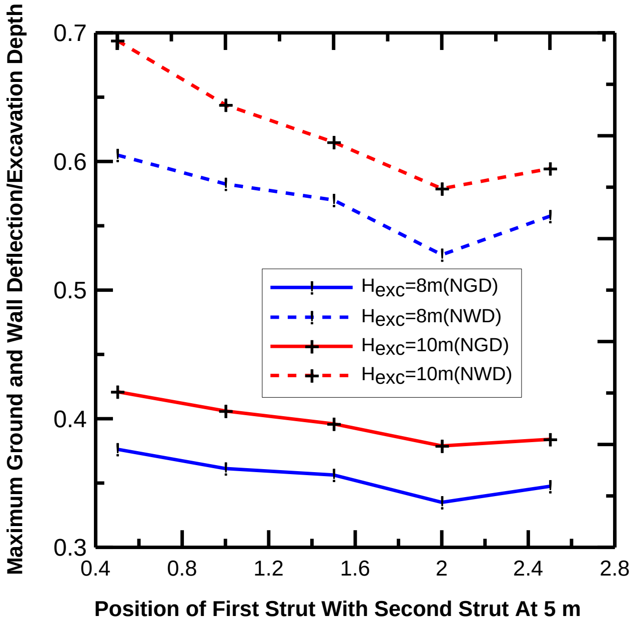


Figure 3.9 - Effect of first strut location with second strut position fixed at 5 m on maximum ground deformation and maximum wall displacement.

In cases 3 and 4, 1st strut is fixed at 2 m level and 2nd strut location is varied at 4 m, 5 m, 6 m and 7 m respectively for excavation depth 8 m (case 3) and 10 m (case 4) to study the effect on deformation characteristic (Figure 3.10).

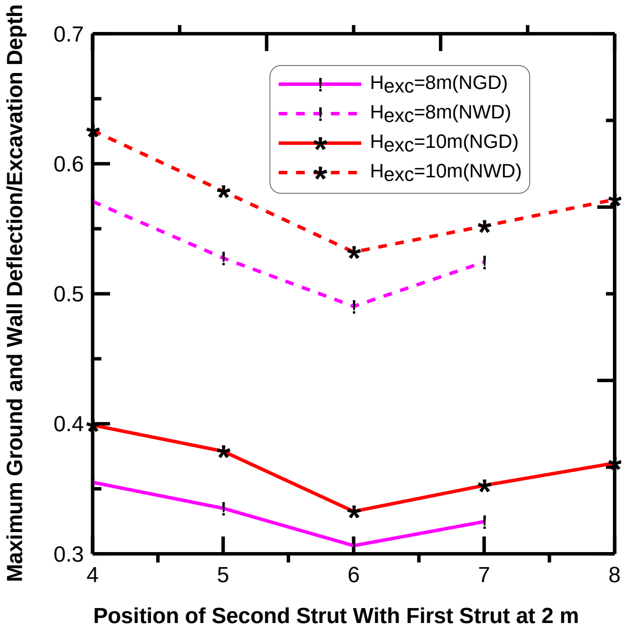


Figure 3.10 - Effect of second strut location with first strut position fixed at 2 m on maximum ground deformation, maximum wall displacement.

Figure 3.10 show that normalized maximum ground and wall displacement is decreased as location of 2nd strut is lowering from 4 m to 6 m. Further lowering of 2nd strut causes increment of normalized maximum ground and wall displacement. This is because if 2nd strut is close to 1st strut or very distant from 1st strut unsupported length is increased resulting greater deformation. So, it is suggested that level of 2nd strut is to be fixed at 0.6-0.7 times the depth of excavation so that minimum deformation can be obtained. This is when two level of strut is installed to stable excavation.

Next greater excavation depth is taken into account and three level of strut is installed. 1st and 2nd strut are placed at 2 m and 6 m. The location of third strut is changed at different positions like at 8 m, 9 m, 10 m, 11 m when excavation depth is 12 m (case 5) and at 8 m, 9 m, 10 m, 11 m, 12 m, 13 m (case 6) when excavation depth is 14 m. Variations of normalized maximum ground deformation and normalized maximum wall deflection with position of third strut are presented in figure 3.11.

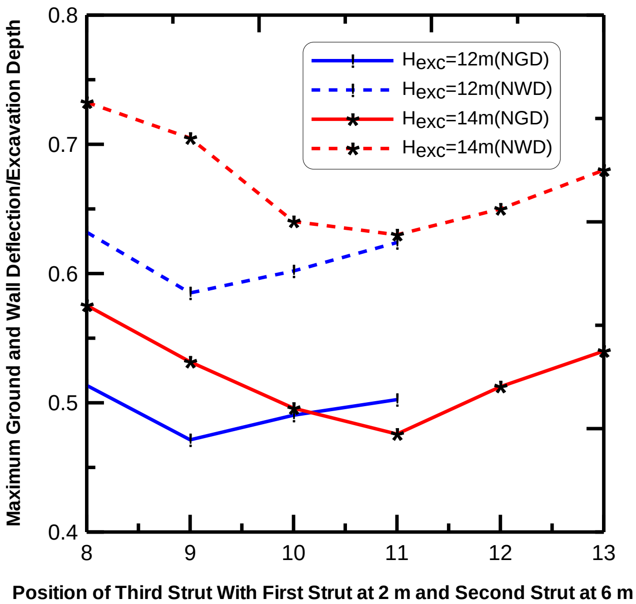


Figure 3.11 - Effect of third strut location with first strut and second strut position fixed at 2 m and 6 m respectively on maximum ground deformation and maximum wall displacement.

From figure 3.11 it can be seen that minimum normalized deformation values (both ground and wall deformation) are obtained when third strut is placed at 9 m in case of excavation depth 12 m and at 11 m when excavation depth is 14m. Thus, if third strut is installed at 0.75-0.80 times the depth of excavation then lower range of normalized maximum ground and wall deformations are achieved. So, when excavation depth is more than 10 m then 1st, 2nd and 3rd levels of struts are to be placed at 0.14-0.17, 0.42-0.5 and 0.75-0.80 times of excavation depth to obtain optimum deformation values. If excavation depth is less or equal to 10 m two levels of struts may be used which are required to be placed at 0.2-0.25, 0.6-0.7 times the depth of excavation to get minimum deformation values.

3.1.3.3.3 Effect of wall thickness

It is attempted to study the effect of diaphragm wall thickness (T_{wall}) on normalized maximum ground displacement ($\delta_{g,}$max/Hexc) and normalized maximum wall deflection ($\delta_{w,}$max/Hexc). Normalized wall embedment depth (D/ H_{exc}) is set at 1.0. For excavation depth 8 m and 10 m two stages of struts are considered (at 2 m and 6 m) and when excavation depth beyond 10 m three stages of struts (at 2 m, 6 m and 10 m for H_{exc} equal to 12 m and at 2 m, 6 m and 12 m for H_{exc} equal to 14 m) are taken. Four different values of wall thickness 0.5 m, 0.6 m, 0.8 m and 1.0 m are employed for present study and variations of 'δ_{vm}/H_{exc}' in percentage with wall thickness (T_{wall}) are plotted (Figure 3.12). With increase of wall thickness 'δ_{vm}/H_{exc} (%)' decreases steadily.

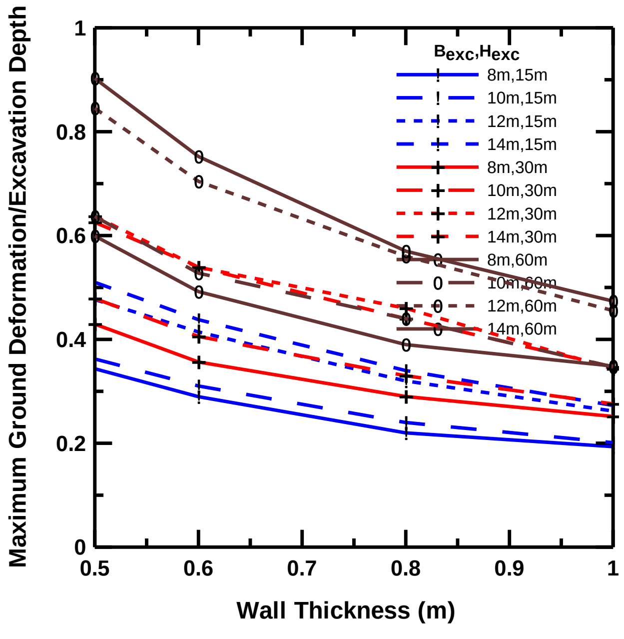


Figure 3.12 - Effect of wall thickness on the normalized ground deformation for different width and depth of excavation

From figure 3.12 the maximum variations of maximum normalized ground displacement are 45% - 47% for excavation depth of 14 m with various excavation width. In case of excavation depth of 8 m maximum variations are within 38% – 44%.

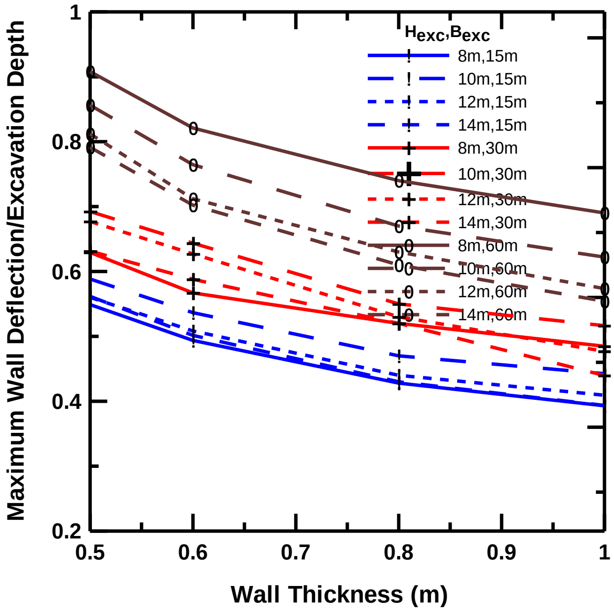


Figure 3.13 - Effect of wall thickness on the normalized wall deflection for different width and depth of excavation.

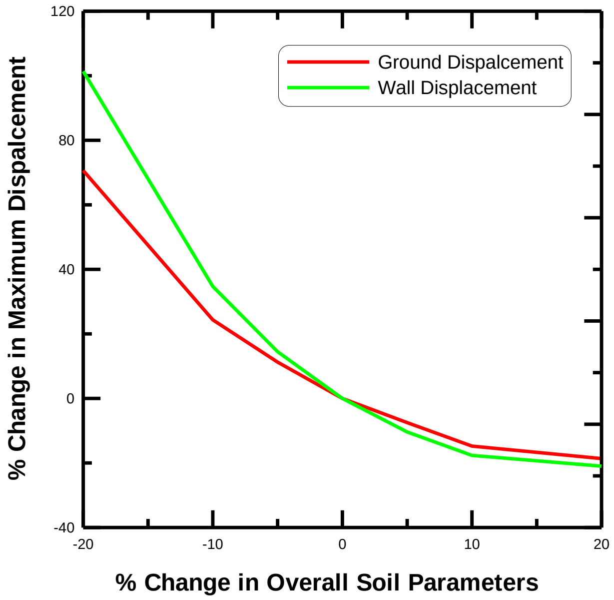
From figure 3.13 variations of normalized maximum wall displacement (δ_{hm}/H_{exc}) with different wall thickness are observed. Maximum changes of 'δ_{hm}/H_{exc}' due to increase of wall thickness are 29.8 – 30.3% for excavation depth 14 m. While for excavation depth 10 m and various excavation width as mentioned the maximum variations of 'δ_{hm}/H_{exc}' are in between 24.7 – 27.3%. From results it is evident that significant change of ground and wall deformations observed when wall thickness is increased. Though no clear optimum value of wall thickness can be obtained from results, still rate of decrement of deformation decreased with increase of wall thickness. It is suggested that wall thickness may be kept between 0.8 m to 1 m for excavation depth beyond 10 m (i.e., 12 or 14 m) which implies that wall thickness of 0.06 to 0.08 times of excavation depth can be provided. If excavation is within 10 m then wall thickness may be restricted to 0.70 m as after 0.70 m, rate of reduction of normalized ground deformation is gradual.

3.1.3.3.4 Importance of estimating correct soil parameters values

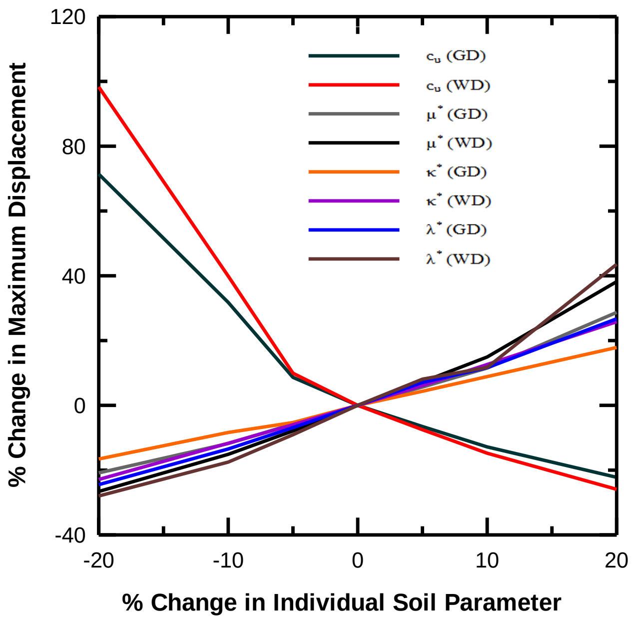
The accuracy of ground and wall deformation prediction depends upon proper modelling of soil behavior in FE analysis. It is difficult to model soil behavior correctly as behavior of soil behind the wall is difficult to characterize and special apparatus is required to measure soil parameters at low strain level. In present analysis impact on soil deformation during braced excavation due to slight variations of soil parameters is observed.

At first the values of all the parameters are simultaneously increased and decreased by 5%, 10%, and 20%, and the impact of such changes on the braced wall system is studied. The study was performed for the braced wall described in figure 3.1. Figure 3.14 (a) shows that an increase in the values of all the parameters has an almost linear impact on the braced wall system — 20% increase leads to about 20% reduction in δ_{hm} (maximum horizontal deflection of wall) and δ_{vm} (maximum vertical ground surface settlement). However, a simultaneous decrease in the values of the soil parameters has an amplified impact on the braced wall — 20% decrease leads to about 100% increase in δ_{hm} and 70% increase in δ_{vm}. Obviously, these are artificially exaggerated values because simultaneous increase or decrease in all the parameters is a remote possibility. Nevertheless, the engineer must be mindful of the impacts of possible variability of soil properties, particularly if the braced wall is constructed in a soft patch within the Kolkata area.

The impact of soil variability is further investigated by altering, one at a time, the four most important soil parameters — c_u, λ^*, κ^*, and μ^* — by ±5%, ±10%, and ±20% for the same braced excavation. Figure 3.14 (b) shows that δ_{hm} and δ_{vm} decrease with an increase in c_u, which appears to be the most sensitive parameter, and increase with increase in λ^*, κ^*, and μ^*. The sensitivities of λ^*, κ^* and μ^* on the braced wall system responses are more or less the same and are less than that of c_u.



(a)



(b)

Figure 3.14 - Effect of variability of soil parameters on the wall and ground displacements: (a) overall variation of soil parameters and (b) individual variation of soil parameters (GD = Ground Deformation, WD = Wall Deflection)

3.1.3.3.5 Design guidelines

The following design guidelines are proposed here.

1. The wall embedment depth should be kept between 0.7 to 1.0 times of excavation depth and 0.2 to 0.4 times of excavation width when construction of braced excavation is done in soft to medium clay soil. For excavation depth greater than 10 m these should be strictly followed to resist excessive ground movement.
2. When excavation is less than 10 m two level of struts can be installed but for greater excavation depth three or more struts should be fixed.
3. The position of first strut should be at 0.2-0.25 times the depth of excavation to get optimum deflection values. If first strut is placed below that specified level, cantilever height of wall is increased producing larger deformation. When strut is very close to ground level unsupported wall length increases.
4. The second strut should be placed in such a way that spacing between struts are not large. For excavation depth up to 10 m, 2^{nd} strut may be installed at 0.6-0.7 times the depth of excavation.

5. When excavation depth is more than 10 m three levels of struts are needed to be installed. In this situation location of 1st, 2nd and 3rd levels of struts are as follows; 1st strut: at 0.14 to 0.17 times the depth of excavation; 2nd strut: at 0.42 to 0.5 times the depth of excavation; 3rd strut: at 0.75 to 0.80 times the depth of excavation
6. Though it is hard to obtain optimum wall thickness still it is recommended that wall thickness can be provided such that it is around 0.06 to 0.09 times of the depth of excavation. There is not much change of displacement values when wall thickness is approximately 0.08 times of depth of excavation.
7. Correct estimation of soil parameters is very critical as little variability can change estimated results largely using FE analysis. Special care should be taken to measure soil parameters using tri-axial apparatus at low strain (10^{-5} to 10^{-3}). Undrained shear strength of clay has greater influence than consolidation parameters on soil deformation during braced excavation. An increase of 20% in undrained cohesion the magnitude of maximum ground settlement and wall displacement values would decrease by 22% and 26% respectively, while a decrease of 20% of same would result an increase in maximum ground settlement and wall displacement values up to 70% and 98 % respectively.
8. To consider time dependent ground movement which is very common in soft clay it is suggested to use soft soil creep model in PLAXIS analysis.

3.1.4 PREDICTION OF GROUND DEFORMATION AND WALL DISPLACEMENT

Estimation of ground deformations during braced excavation is important to control surrounding ground movements as excessive deformations damage adjacent properties especially in urban areas. A good amount of works has been done using numerical analysis and field studies. Though, theoretical studies and mathematical modelling have not been well addressed in the reported literature. Here a simplified model is proposed for estimation of ground and braced wall displacements using data obtained from FE analysis of hypothetical excavation cases.

3.1.4.1 Development of Regression Model

For prediction of maximum ground surface settlement, maximum wall deflection, ground surface and wall deflection profile multivariable regression analyses are performed and simplified equations are proposed.

Equations are formed accounting various parameters like depth of excavation, width of excavation, diaphragm wall thickness, wall embedment depth, soil strength, strut spacing etc. For this purpose, excavation data have been collected from reported case studies and some excavation data have been generated artificially performing FEM analysis. For validation purpose three reported cases are taken into consideration. Finally executing student's t -test and calculating Root mean square error it has been established that proposed model can efficiently predict deformation characteristics of braced excavation.

The following steps are followed to execute proposed model.

1. Total fifty-four cases of braced excavation mainly in soft to medium clay are used for proposed model. Six input parameters are employed for carrying out regression analysis. These are depth of excavation (H_{exc}), width of excavation (B_{exc}), diaphragm wall thickness (T_{wall}), ratio of wall embedment depth and width of excavation (D/B_{exc}), ratio of average shear strength of soil and effective vertical stress (s_u/σ_v'), average strut spacing (h_{avg}).
2. Multivariable regression analysis is performed to form equation for maximum ground surface and wall deflection. The developed equations are expressed as

$$\delta_{vm} = [\alpha 0 + \alpha 1 \times H_{exc} + \alpha 2 \times B_{exc} + \alpha 3 \times T_{wall} + \alpha 4 \times (\frac{D}{B_{exc}}) + \alpha 5 \times (\frac{S_u}{\sigma_{v'}}) + \alpha 6 \times h_{avg}] \quad (1)$$

$$\delta_{hm} = [\beta 0 + \beta 1 \times H_{exc} + \beta 2 \times B_{exc} + \beta 3 \times T_{wall} + \beta 4 \times (\frac{D}{B_{exc}}) + \beta 5 \times (\frac{S_u}{\sigma_{v'}}) + \beta 6 \times h_{avg}] \quad (2)$$

Here, δ_{vm} and δ_{hm} are the maximum ground surface settlement and maximum horizontal displacement of wall respectively.

3. Now to obtain ground and wall deformation profile twenty case histories taken for further analysis. In case of ground displacement, observing reported data two zones are identified i.e., $L/H_{exc} \leq 0.8$ and $L/H_{exc} > 0.8$. Now relations between L/H_{exc} and δ_{vl}/δ_{vm} are separately plotted for those two zones obtained from case histories and linear regression analysis is performed. Similar analysis is done for wall deformation but in this case, zones are categorized as $d/H_{exc} \leq 1.0$ and $d/H_{exc} > 1.0$. Here L is the

distance from wall while d indicates depth from ground surface. δ_{vl} is the ground deformation at a distance '*l*' from edge of wall.

4. Now ground surface settlement at any point can be calculated after computing maximum ground deformation as stated in step 2 and then calculating 'δ_{vl}' from equation between L/H_{exc} and δ_{vl}/δ_{vm} obtained in step 3.
5. Finally, three cases are taken for validation. Predicted values from proposed model and expected values as reported in literature are compared. Student's t – test is used to observe variation between estimated and measured values. Root mean square error (RMSE) and Normalized root mean square error are also calculated.

In present study data are taken from various locations of Kolkata Metro Construction, excavation of a metro station in Shanghai, excavation in Kaohsiung, Taiwan and Taiwan National Enterprise Centre excavation project. A wide range of parameters are considered so that model equations can be applied effectively. The parameters are chosen on the basis of reported literatures and measured values in Kolkata Metro Construction. From analysis it is observed that values of R square (r^2 = 0.866 and 0.880) and adjusted R square (adjusted r^2 = 0.850 and 0.865) are quite high which establishes the acceptability of proposed equations. Coefficients used in above equations are found out as given below.

α_0 = 89.8974, α_1 = 2.7479, α_2 = 0.3952, α_3 = -47.9848, α_4 = -27.0943, α_5 = -112.139, α_6 = -0.5725

β_0 = 44.0947, β_1 = 2.7663, β_2 = 0.5153, β_3 = -37.175, β_4 = -11.3929, β_5 = -55.4557, β_6 = 6.65923

Now focus is given on to predict ground surface settlement at different locations and wall displacement at various depths. Thus, it is attempted to get a relation between maximum displacement and displacement at any other point. From studying case histories, it is found that 'δ_{vl}/δ_{vm}' value is generally increased till L/H_{exc} value reached 0.8 and then decreased. It is similar to concave type of ground deformation profile where maximum ground deformation is observed at some distance behind wall when friction between wall and soil is not fully mobilized. This type of deformation profile occurs when strut is near ground surface and stiffness of wall is sufficient to resist soil movement behind wall. To get trend of this relation between 'δ_{vl}/δ_{vm}' and 'L/H_{exc}' linear regression analysis is performed (Fig 3.15(a) and Fig 3.15(b)). Similar figures are obtained for wall displacement (Fig 3.16(a) and Fig 3.16(b)) but in this case 'δ_{hd}/ δ_{hm}' is increased for d/ H_{exc} value up to 1.0. This is because maximum wall deflection occurs near excavation depth for braced cut in soft to medium clay. Here 'δ_{hd}' indicates horizontal wall displacement at depth 'd' from ground surface. The regression equations can be expressed as follows:

$$\frac{\delta_{vl}}{\delta_{vm}} = -0.83\times\left(\frac{L}{H_{exc}}\right)^2 + 1.452\times\left(\frac{L}{H_{exc}}\right) + 0.311 \quad \text{for } L/H_{exc} \le 0.8 \ (r^2=0.829) \qquad (3)$$

$$\frac{\delta_{vl}}{\delta_{vm}} = 0.032\times\left(\frac{L}{H_{exc}}\right)^2 - 0.364\times\left(\frac{L}{H_{exc}}\right) + 1.063 \quad \text{for } L/H_{exc} > 0.8 \ (r^2 = 0.708) \qquad (4)$$

$$\frac{\delta_{hd}}{\delta_{hm}} = -0.48\times\left(\frac{d}{H_{exc}}\right)^2 + 1.35\times\left(\frac{d}{H_{exc}}\right) + 0.129 \quad \text{for } d/H_{exc} \le 1.0 \ (r^2 = 0.914) \qquad (5)$$

$$\frac{\delta_{hd}}{\delta_{hm}} = 0.501\times\left(\frac{d}{H_{exc}}\right)^2 - 2.101\times\left(\frac{d}{H_{exc}}\right) + 2.596 \quad \text{for } d/H_{exc} > 1.0 \ (r^2 = 0.808) \qquad (6)$$

Now three different case studies are considered to validate proposed model. The details of each case are tabulated in Table 6. Maximum ground and wall deformations are calculated using equation 1 and equation 2. After obtaining maximum values deformation values at other locations are calculated using from equation 3 to equation 6. The estimated values are plotted with reported values and these are shown from figure 3.17 to figure 3.19. From figures it can be concluded that proposed equations can be used for effective prediction for excavation especially in soft to medium clay. Root Mean square of Error (RMSE) and Normalized RMSE for both ground surface deformation and wall deflection for those three cases are conducted. The percentage NRMSE values are within satisfactory range. Table 7 reveals the values of RMSE and NRMSE for above mentioned three studies. Finally, student's t-test is carried out to relate predicted and measured values of deformation for each case. From Table 7 it is also clearly seen that calculated t values are less than $t_{0.05}$ and p values are greater than 0.05. So, it can be said there are no significant differences between the measured and calculated data series.

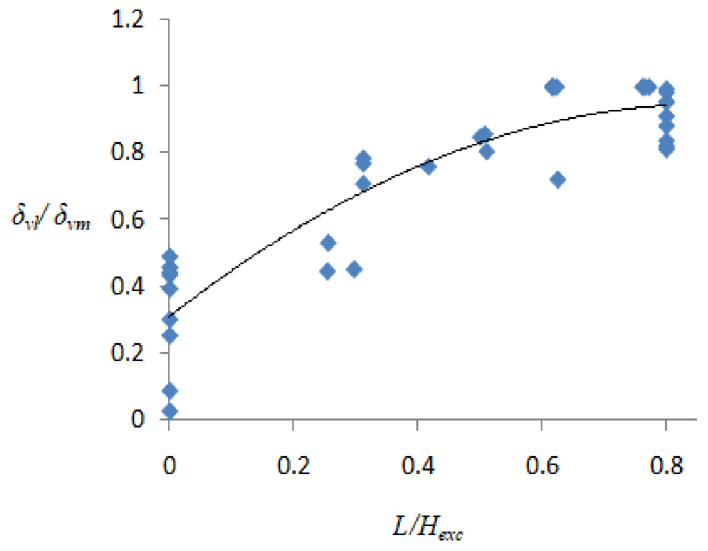


Figure 3.15(a) - Relationship between $\delta_{vl}/\,\delta_{vm}$ and $L/\,H_{exc}$ when ($L/\,H_{exc} \leq 0.8$)

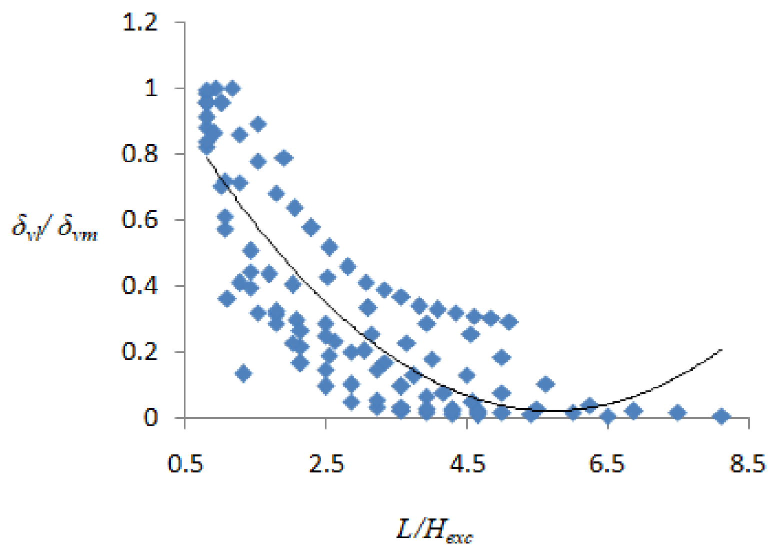


Figure 3.15(b) - Relationship between $\delta_{vl}/\,\delta_{vm}$ and $L/\,H_{exc}$ when ($L/\,H_{exc} > 0.8$)

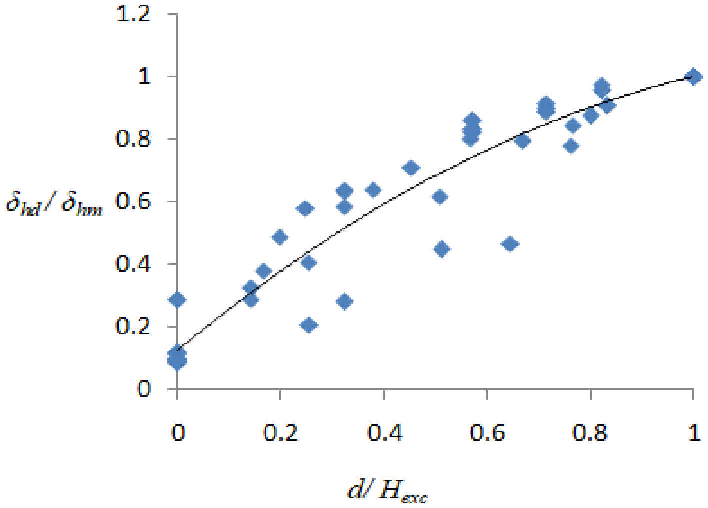


Figure 3.16(a) - Relationship between δ_{hd}/δ_{hm} and d/H_{exc} when ($d/H_{exc} \leq 1.0$)

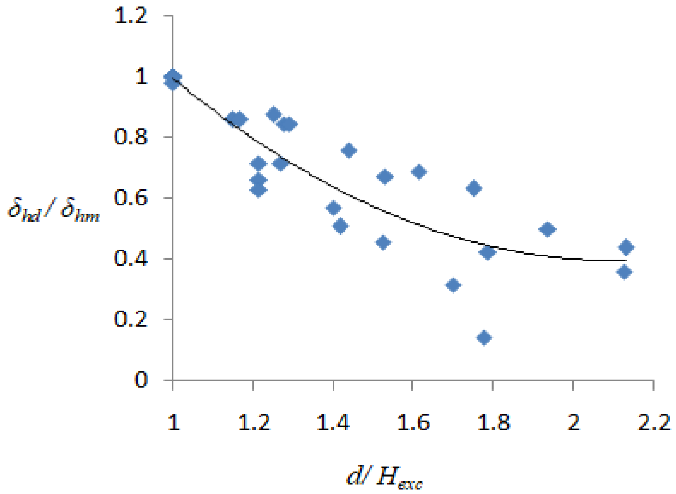


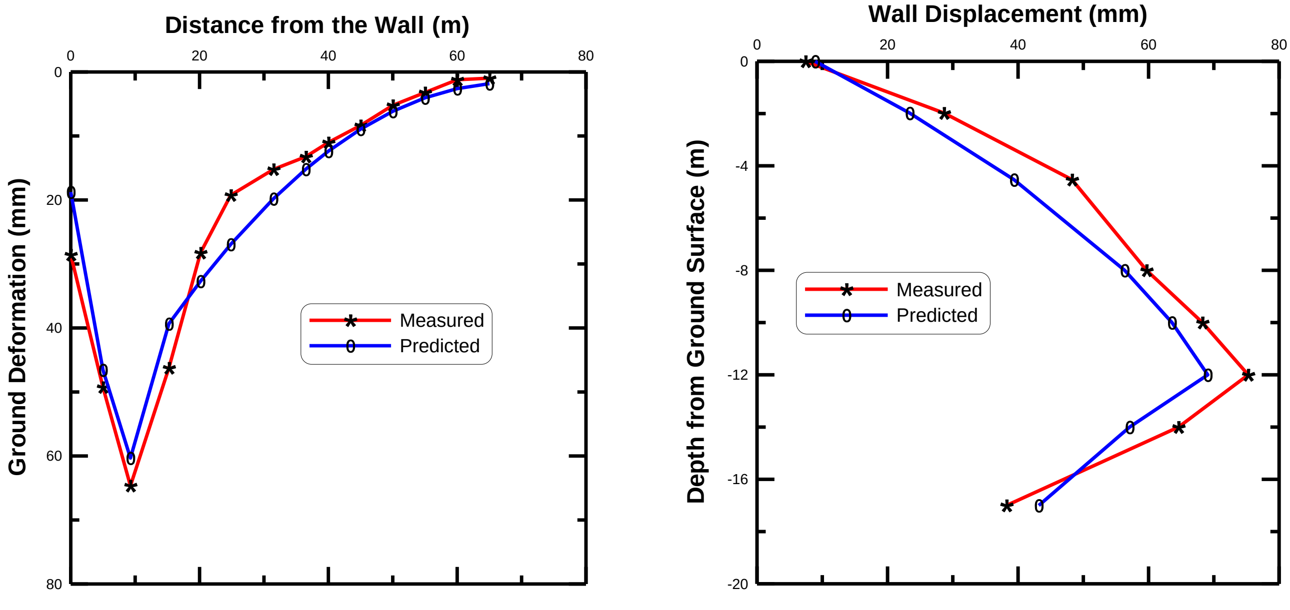
Figure 3.16(b) - Relationship between δ_{hd}/δ_{hm} and d/H_{exc} when ($d/H_{exc} > 1.0$)

Table 6 - Excavation details of three reported cases

Case Study / Parameters	Kolkata (180A C.R. Avenue) (Som et al) (2000)	Bangkok (Thasnanipan et al) (1998)	Taiwan (Dang et al) (2012)
H_{exc} (m)	12	12.7	19.2
B_{exc} (m)	30	12	20
T_{wall} (m)	0.6	0.8	1
D/B_{exc}	0.1667	1.275	1.04
s_u/σ_v'	0.3512	0.6068	0.28

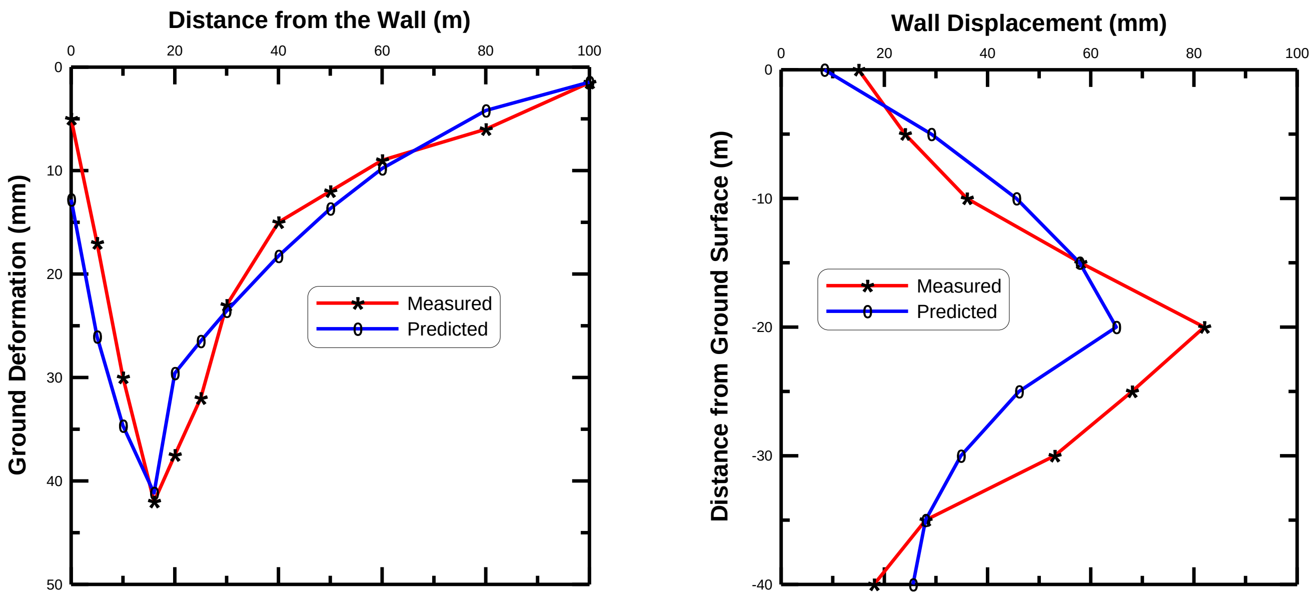
h_{avg} (m)	3	3.5	3.3

[H_{exc} = excavation depth, B_{exc} = excavation width, T_{wall} = diaphragm wall thickness, D = wall embedment depth, s_u = average shear strength of soil , σ_v' = effective vertical stress, h_{avg} = average strut spacing]



(a) (b)

Figure 3.17 - Predicted and measured (a) ground deformation and (b) wall deflection at Kolkata Metro Construction near 180A C.R. Avenue (Som (2000)).



(a) (b)

Figure 3.18 - Predicted and measured (a) ground deformation and (b) wall displacement at an Excavation in Taiwan reported by Dang et al (2012)

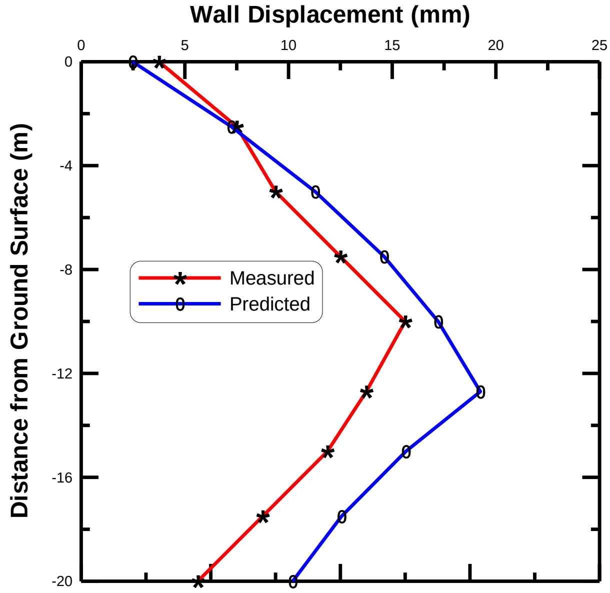


Figure 3.19 - Predicted and measured wall deflection at Excavation in Bangkok reported by Thasnanipan et al (1998)

Table 7 - Details of RMSE and NRMSE values and student's t test results of three reported cases

Cases	RMSE	% NRMSE	Calculated t value	Critical t value at 5% level of significance	p or probability value
Ground deformation at Kolkata metro construction (near 180A C.R.Avenue) (Som (2000))	0.44233	6.947	0.0113	2.055	0.991
Wall deflection at Kolkata metro construction (near 180A C.R.Avenue)(Som (2000))	0.57079	8.4146	0.3368	2.1447	0.741

Ground deformation at Excavation in Taiwan reported by Dang et al (2012)	0.47964	11.8429	0.185	2.074	0.855
Wall deflection at Excavation in Taiwan reported by Dang et al (2012)	1.21076	18.071	0.4703	2.1314	0.645
Wall deflection at Excavation in Bangkok reported by Thasnanipan et al (1998)	0.51223	19.0939	1.114	2.1314	0.283

3.1.5 CONCLUSION FROM SECTION 3.1

In the study prescribed in section 3.1, firstly a parametric study has been done to investigate the influence of different design parameters on deformation behavior. Based on the results design guidelines are proposed such that optimum values of various parameters like wall embedment depth, wall thickness and strut arrangements are determined and from this extensive study it can be concluded that

(i) Wall embedment depth should be at least 0.7 to 1.0 times of depth of excavation to avoid excessive ground movement when excavation is done in soft clay.

(ii) Strut arrangement should be such that unsupported wall length is not large.

(iii) With greater wall thickness small deformation values are obtained.

(iv) Values of soil strength and consolidation parameters should be chosen or estimated carefully as little variation produce marked different deformation results using FE analysis.

Secondly a simplified model is proposed to predict ground and wall movements effectively. Using data obtained from case histories and generated artificially from FE analysis a simplified model is presented performing multi variable regression method.

3.2 TIME DEPENDENT DEFORMATION BEHAVIOUR OF BRACED EXCAVATION

3.2.1 GENERAL

Ground settlement is a major consequence of any excavation work and prediction of surrounding soil movement considering various factors during braced cut is crucial. Apart from some well-known factors like diaphragm wall thickness, embedment depth and strut locations it is observed that the rate of excavation including delay during construction has a major impact on the deformation behaviour of the excavation system. Magnitude and rate of settlement are found to increase very rapidly with slow excavation rate and pause in construction. Present study presents a detailed analysis on the time dependent soil deformations which includes consolidation and creep settlement by performing finite element analysis using PLAXIS 2D considering the soft soil and soft soil creep constitutive model to simulate typical braced excavations in soft clay. Extensive study has been conducted to understand the effects of excavation rate and construction stoppages on ground and braced wall displacement. Further fitted equations are developed for the maximum ground settlement and maximum lateral wall deflection conducting multi-variable regression analysis where time parameters like rate of excavation, pause in construction and depth of excavation are used as independent parameters.

3.2.2 NEED AND SCOPE OF THE PRESENT STUDY

Som (1994) reported settlement at a number of excavation sites of Calcutta metro alignment where the duration of excavation were 150-430 days for depth of excavation varying from 9 – 14 m (giving rate of excavation 15.3–34.2 days/m) for which maximum settlement recorded during excavation was found to be 25–128 mm. Similar observation were also reported by Ou et al (1998) and Liu et al (2005) on braced excavation of 19.7m and 15.5m deep excavation in Taipei and Sanghai soil with rate of excavation about 10days/m and 6.55 days/m respectively. The results reported by them are plotted in Figure 29 to show the increase in maximum settlement with rate of excavation. Figure 29 has been drawn for different rate of excavation used in Kolkata, Taipei and Shanghai with a dimensionless parameter, i.e. normalized maximum ground settlement (S_{max}/H), where S_{max} is the maximum ground settlement and H is the final excavation depth.

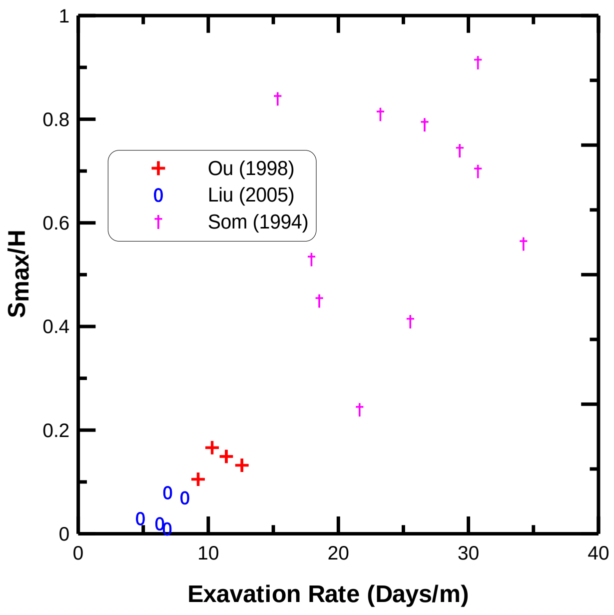


Figure 3.20 - Normalized Maximum ground settlement with different excavation rate.

This (Figure 3.20) indicates that for Taipei and Sanghai soil deformation is much less as rate of excavation was less than or close to 10 days/m, while for Kolkata soil it is much higher as the rate of excavation was 15-34 days/m.

Further, Som (1994) also reported the effect of time delay on deformations of braced cuts in soft clay based on extensive measurements of ground movement and building settlement at different sections of Kolkata underground metro rail excavation performed during the late 1980s. Additional ground settlements were observed when excavations were kept open at the final cut level or at any intermediate level for long periods of time (Som 1994) and recorded to be 24-106 % for a delay of 39-730 days after reaching the final excavation level. Ou et al (1998) also observed an increase of 18.2 mm of lateral displacement as the construction was halted for 60 days at stage 6 at a depth of 7.1 m. Liu et al (2005) reported an additional creep deformation during 60 days curing period of middle slab constructed before fifth stage of excavation.

This necessitates the need for a thorough study of time dependent deformation behaviour of braced excavation in soft clay deposit to understand the mechanism of deformation as well as effect of various parameters related to time likely rate of excavation and hault during and after excavation.

In the study given in section 3.2, a systematic investigation of the mechanism of time dependent deformation characteristics of a typical braced excavations in soft clay of Kolkata is performed using two-dimensional (2D) finite element (using PLAXIS 2D) analysis, and further parametric study focusing on the effect of two major time dependent parameters like rate of excavation and pause in construction has been done. The elasto-plastic creep behavior of soft clay is incorporated in the FE analysis (performed using PLAXIS 2D) by using the soft soil model and soft soil creep constitutive model. At first the accuracy of the present analysis is verified against the results of FE analysis performed on braced excavation cases reported in the literature. Next it is attempted to understand soil settlement mechanism which includes undrained deformation due to excavation, consolidation and creep deformation. Details of soil settlement behavior during various stages of excavation and after excavation is discussed performing soft soil model (SSM) and soft soil creep model (SSCM) where both undrained and drained analysis are carried out. Both SSM and SSCM models have been used in order to identify consolidation and creep settlement separately. Finally, some parametric studies have been done to assess effects of time dependent parameters on deformation characteristics of braced excavation. Further multi-variate regression analysis is performed to develop equations for maximum ground and wall deformations as a function of parameters like depth of excavation, rate of excavation, pause time based on the results of FE analyses of hypothetical excavation cases in soft clay as found in Kolkata. The validity of the proposed equations in simulating the field behaviour is checked by comparing the results obtained from the proposed equations with those from field monitoring of the braced excavations at various sites in Kolkata.

The numerical study part describe in section 3.1.2 remains same for this study also i.e., study which is given in section 3.2

3.2.3 ANALYSIS OF TIME DEPENDENT DEFORMATION

To understand time effect on ground deformation behaviour of soil at first a typical excavation is taken into consideration in soft clay as observed in normal Kolkata deposit. FE analysis has been done using SSCM and SSM model to differentiate between consolidation and creep settlement of soil. Pore water pressure distributions at various excavation stages are also studied to observe change of pore water pressure with time and compare these values with consolidation settlement. The model excavation used in the study is of excavation depth 14 m with 3 struts placed at depth of 2 m, 6 m and 11 m respectively. The properties of soil, diaphragm wall and struts remain the same as discussed earlier. The sequence of model braced excavation has been done in 9 stages. Different excavation rates i.e., 10 days/m, 20 days/m, 35 day/m, 50 days/m, 75 days/m and 100 days/m have been followed in this particular excavation model. Time required for installation of wall as well as each strut was taken to be 7 days. The details of the excavation stages have been tabulated in table 8 for excavation rate 10 days/m.

Table 8 - Description of the excavation system

Sequence of Stages	Description of the Stages	Construction Time (Days)
1	Installation of wall	7
2	Excavation up to 3m	7-37
3	Installation of first strut at 2m	37-44
4	Excavation 3-7m	44-84
5	Installation of second strut at 6m	84-91
6	Excavation 7-10m	91-121
7	Excavation 10-12m	121-141
8	Installation of third strut at 11m	141-148
9	Excavation 12-14m	148-168

In Tables (9a-9d) results have been shown for maximum ground deformation at different distances from the edge of the wall using SSCM and SSM models with drained and undrained analysis. From obtained values attempt has been made to identify consolidation and creep settlement separately.

Table 9a - Values of maximum ground deformations at 5 m from the diaphragm wall

Rate of Excavation (Days/m)	Maximum Deformation using SSM/Undrained Model (mm)	Maximum Deformation using SSM/Drained Model (mm)	Maximum Deformation using SSCM/Drained Model (mm)	Undrained/Immediate Settlement (mm)	Consolidation Settlement (mm)	Creep Settlement (mm)

1	2	3	4	5 = 2	6 = 3-2	7 = 4-3
10	35.43	47.1219	57.89	35.43	11.69	10.77
20	36.045	49.382	79.077	36.045	13.337	29.695
35	37.489	52.4846	108.26	37.489	14.9956	55.7754
50	38.21	55.4045	127.912	38.21	17.1945	72.5075
75	38.45	57.675	128.447	38.45	19.225	70.772
100	38.87	58.1652	128.64	38.87	19.29	70.4748

Table 9b - Values of maximum ground deformations at 9 m from the diaphragm wall

Rate of Excavation (Days/m)	Maximum Deformation using SSM/Undrained Model (mm)	Maximum Deformation using SSM/Drained Model (mm)	Maximum Deformation using SSCM/Drained Model (mm)	Undrained/Immediate Settlement (mm)	Consolidation Settlement (mm)	Creep Settlement (mm)
1	2	3	4	5 = 2	6 = 3-2	7 = 4-3
10	50.57	67.258	74.51	50.57	16.69	7.25
20	51.88	72.45	111.4	51.88	20.57	38.95
35	52.56	75.0608	153.714	52.56	22.5008	78.6532
50	52.86	77.1756	182.107	52.86	24.3156	104.9314
75	53.01	78.9849	188.4138	53.01	25.9749	109.4286
100	53.25	80.94	191.7004	53.25	27.69	110.7604

Table 9c - Values of maximum ground deformations at 15 m from the diaphragm wall

Rate of Excavation (Days/m)	Maximum Deformation using SSM/Undrained Model (mm)	Maximum Deformation using SSM/Drained Model (mm)	Maximum Deformation using SSCM/Drained Model (mm)	Undrained/Immediate Settlement (mm)	Consolidation Settlement (mm)	Creep Settlement (mm)
1	2	3	4	5 = 2	6 = 3-2	7 = 4-3
10	32.571	43.48	48.033	32.571	10.909	4.553
20	33.68	45.80	74.51	33.68	12.12	22.86
35	34.78	48.3442	102.396	34.78	13.5642	45.72
50	35.12	50.853	118.13	35.12	15.733	57.53
75	35.65	52.762	121.123	35.65	17.112	59.2
100	35.98	54.3298	122.82	35.98	18.3498	60.12

Table 9d - Values of maximum ground deformations at 20 m from the diaphragm wall

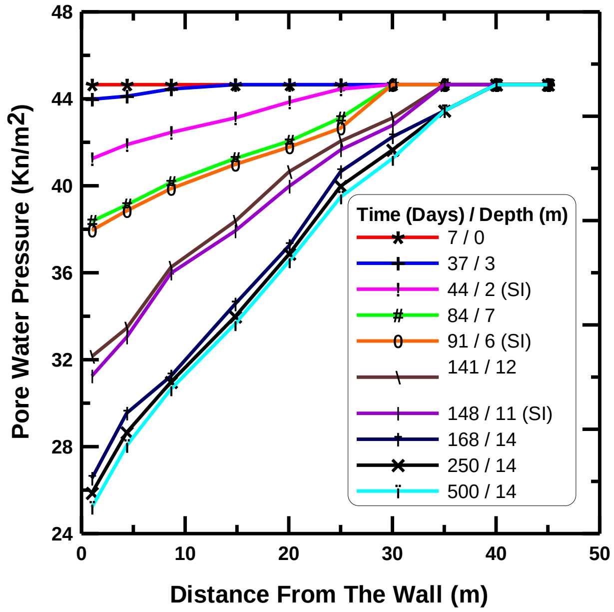
Rate of Excavation (Days/m)	Maximum Deformation using SSM/Undrained Model (mm)	Maximum Deformation using SSM/Drained Model (mm)	Maximum Deformation using SSCM/Drained Model (mm)	Undrained /Immediate Settlement (mm)	Consolidation Settlement (mm)	Creep Settlement (mm)
1	2	3	4	5 = 2	6 = 3-2	7 = 4-3
10	22.60	30.284	31.6	22.60	7.684	1.316
20	23.45	31.892	48.44	23.45	8.442	16.548
35	24.12	33.719	73.24	24.12	9.599	39.521
50	25.25	36.31	86.964	25.25	11.06	50.654

75	25.85	38.26	89.66	25.85	12.41	51.4
100	26.1	39.411	90.75	26.1	13.311	51.339

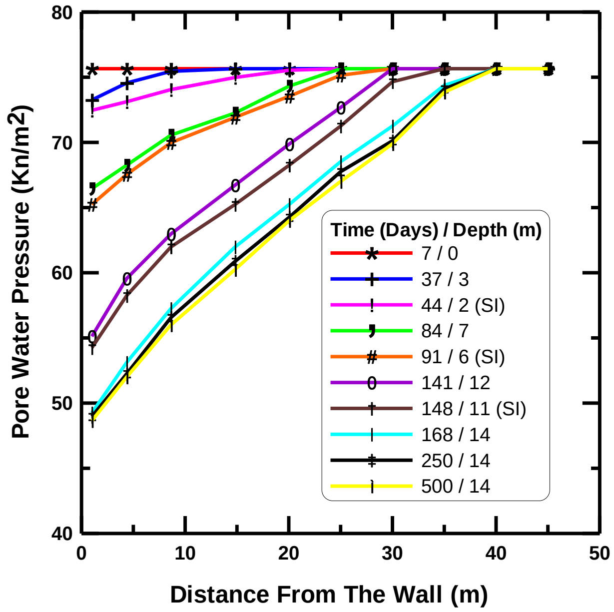
In this context it may be stated that total ground settlement consists of 3 parts (i) Settlement occurring due to stress release during excavation process which is obtained conducting undrained analysis using SSM model (ii) Consolidation Settlement due to pore water pressure dissipation obtained from drained analysis of SSM model and (iii) Creep Settlement or secondary settlement part which occurs after completion of primary consolidation settlement obtained from drained analysis of SSCM model. It is observed from SSM undrained analysis that immediate settlement part is generally same irrespective of rate of excavation. The increase in settlement with the decrease in rate of excavation may be due to mobilization of lower shear strength. Further, more primary consolidation values are obtained for lower rate of excavation though these increases due to decrease in rate of excavation is gradual. Creep settlement is calculated by subtracting values of drained SSM analysis from values of drained SSCM analysis as it is mentioned previously that SSCM model considers creep behaviour of soil. It is seen that creep settlement is increased considerably with the reduction in rate of excavation which establishes the fact that after primary consolidation significant creep settlement is viewed for excavation in Kolkata soil.

3.2.3.1 Pore Water Pressure Data

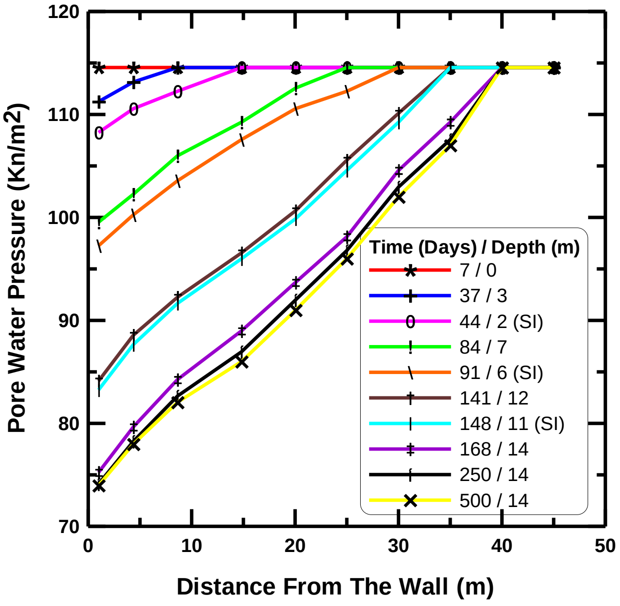
In order to study the change of pore water pressure with progression of excavation, same model excavation is considered as previous section. Rate of excavation is followed as 10 days/ m and variations of pore water pressures with time at different depths are shown in fig 3.21(a-c). Pore water pressure is estimated at 250 days and 500 days to observe variation due to pause in construction after reaching final excavation stage. From figures it can be seen that initially prior to excavation pore water pressures are hydrostatic. As excavation stages are progressed pore water pressures are reduced due to horizontal stress relief. Substantial reduction of pore water pressure is noticed up to final stage of excavation. Thereafter no significant change is observed which means after completion of excavation, soil settlement occurs due to creep effect. As discussed in previous section considerable creep settlement is obtained due to high compressible nature of soft Kolkata soil.



(a)



(b)



(c)

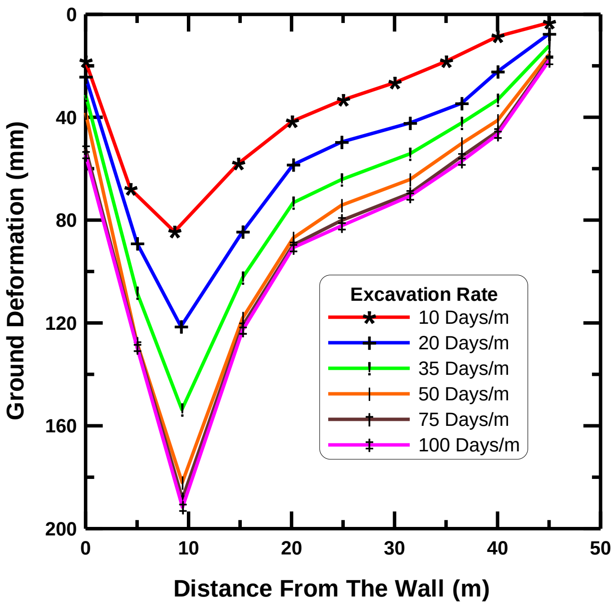
Figure 3.21 - Effect of pore water pressure with distance from the wall, measured at (a) 7 m, (b) 10 m and (c) 14 m below the Ground Surface. (SI = Strut Installation)

3.2.4 PARAMETRIC STUDY

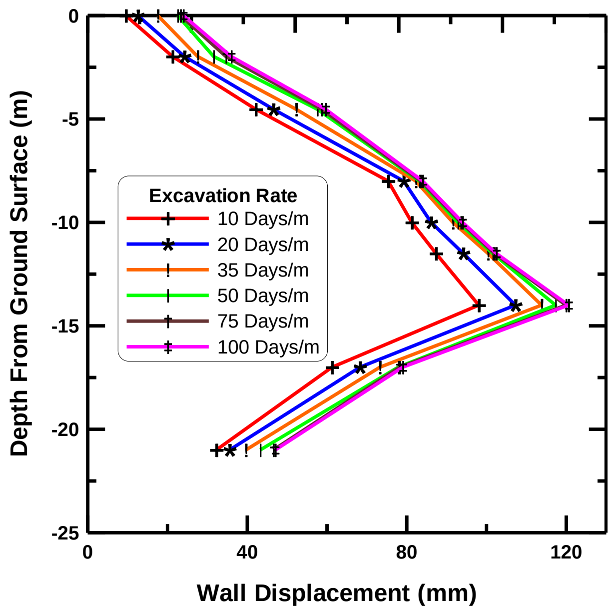
Further some parametric study been done to study the effect of delay in construction work on ground deformation behaviour. SSCM drained model has been used to capture the total settlement for all cases. Two different excavation depths, 14 m and 20 m are considered for investigation. For excavation depth 14 m, three levels of struts are assumed to be installed at 2 m, 6 m and 11 m. In case of 20 m excavation depth, struts are considered to be placed at 2 m, 6 m, 11 m and 16 m. Soil profile and various properties are similar to that of reference excavation in Kolkata soil as discussed earlier. As maximum wall displacement occurs at or near excavation depth for medium to soft clayey soil, wall embedment depth is kept at 0.5 times of excavation depth. Wall thickness and width of excavation are considered as 0.6 m and 30 m respectively.

3.2.4.1 Effect of slow excavation rate

In the foregoing analysis standard excavation rate is maintained at a value of 10 days/ metre excavation (0.1 m/ day). This is because from figure 3.20 it has been observed that for excavation rate close to 10days/m settlement was minimum. In order to investigate the effect of slow excavation rate several reduced excavation rates like 20 days/ m (0.05 m/ day), 35 days/ m (0.0285 m/ day), 50 days/ m (0.02 m/ day), 75 days/ m (0.0133 m/ day) and 100 days/ m (0.01 m/ day) are taken into consideration. In first case standard excavation rate (10 days/ m) is simulated up to depth 5.5 m and thereafter excavation is assumed to be performed at different reduced rates down to excavation depth 14 m. Similarly for case 2 standard excavation rate is followed down to depth 8 m and subsequent excavations are done with slower rate as mentioned before. Similar investigation is carried out for excavation depth of 20 m. Additionally, analyses are performed with the standard excavation rate of 10 days/ m for 14 m deep and 20 m deep excavations for which no alteration of excavation rate is assumed. In figure 3.22(a) and 3.22(b) ground settlement profile behind the wall and wall deformation along the wall with various excavation rates are plotted for case 1. Similar plots are shown in figures 3.23 (3.23(a) and 3.23(b)) for case 2.

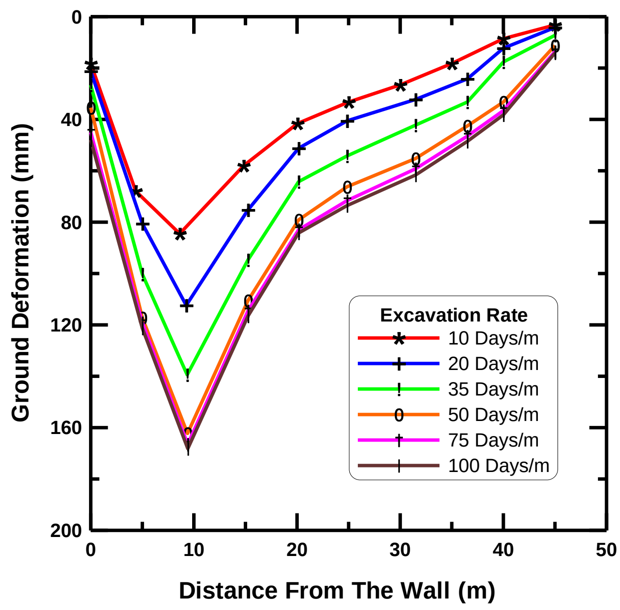


(a)

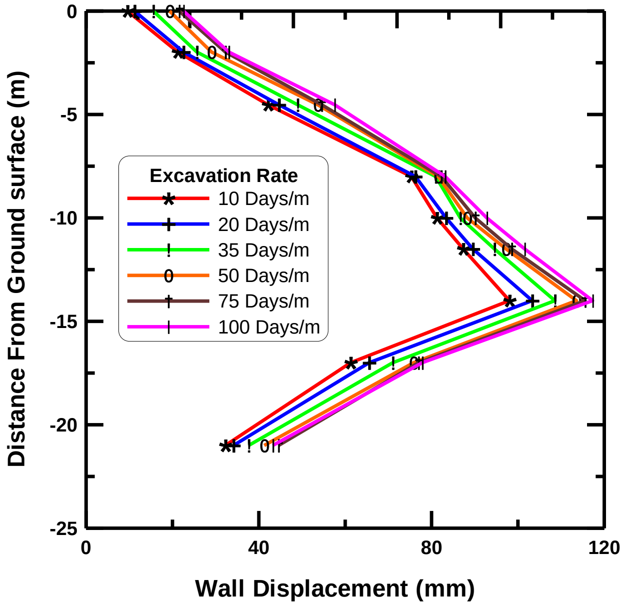


(b)

Figure 3.22 - Effect of excavation rate on (a) ground deformation profile and (b) wall displacement profile when slow excavation rate follows beyond 5.5 m.



(a)

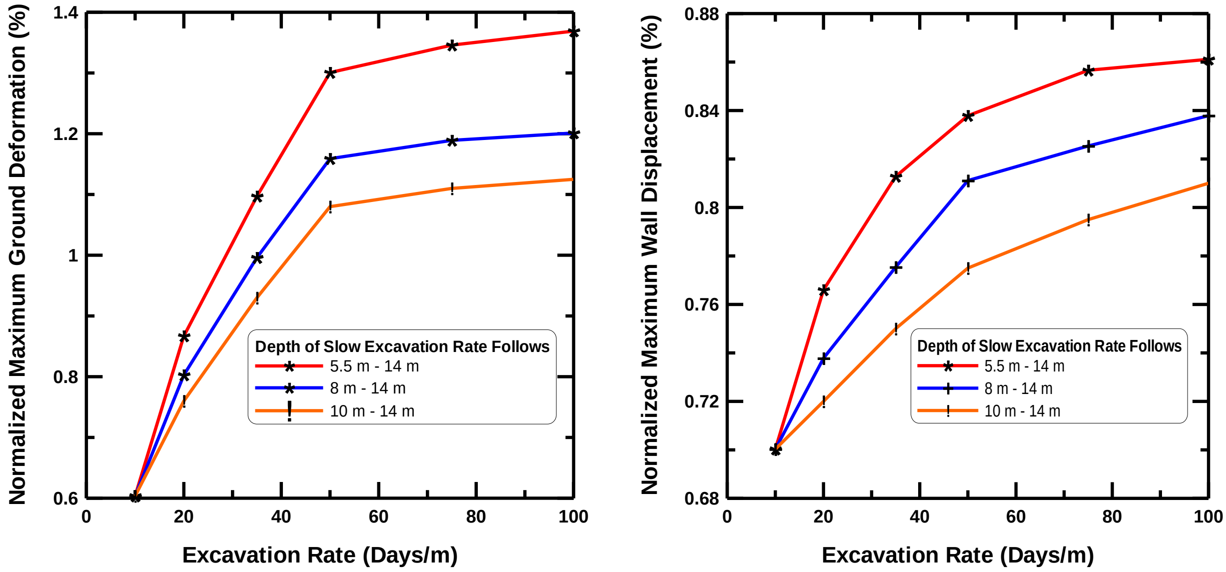


(b)

Figure 3.23 - Effect of excavation rate on (a) ground deformation profile and (b) wall displacement profile when slow excavation rate follows beyond 8 m.

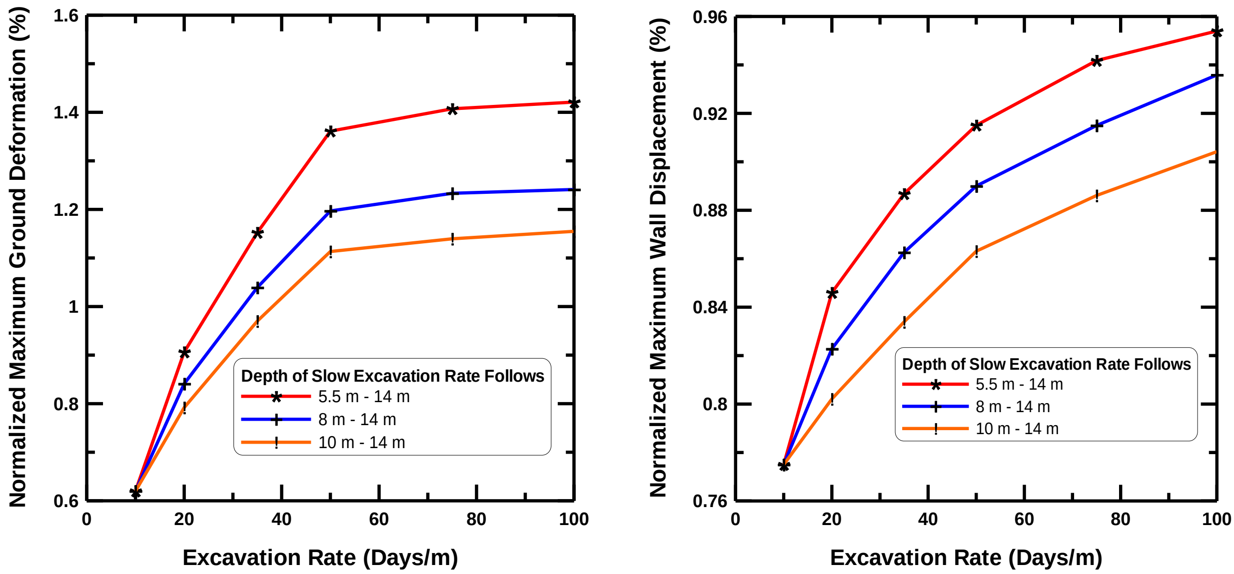
From figures it is quite evident that with reduction of excavation rate deformation values are increased. From figure 3.22(a) it can be said that difference of ground settlement values with decrement of rate of excavation are more prominent in some distance behind the excavation and these differences are not significant adjacent to wall and far behind (beyond 35 m) the wall. The variation of maximum ground settlement is nearly 127% and 44% when excavation rate is 100 days/ m and 20 days/ m respectively compared to no delay or maintaining standard excavation rate of 10 days/ m throughout. Figure 3.22(b) depicts that deviations of wall deformation values are more with slower excavation rate from 10 days/ m to 35 days/ m but further reduction of excavation rate doesn't affect much on deformation values. Figure 3.23 illustrates that braced wall and ground surface exhibits lesser deformation if delayed excavation rate is followed for smaller depth of excavation as slower excavation is employed from 8 m to 14 m. Here difference of maximum ground settlement is about 99% and 33% for excavation rate of 100 days/ m and 20 days/ m when comparing to excavation rate of 10 days/ m.

In figures 3.24 (3.24(a) and 3.24(b)) variations of normalized maximum ground and wall displacements are shown with change of excavation rate for different situations when excavation depth is assumed as 14 m. Maximum deformations are normalized with respect to excavation depth and expressed in percentage. The following three situations are taken. Excavation done at standard rate of 10 days/ m down to different depths of 5.5 m, 8 m and 10 m beyond which the excavation is continued up to excavation depth of 14 m at the different delayed rates discussed above. Similar figures are drawn for excavation depth 20 m and these are shown in figures 3.25 (3.25(a) and 3.25(b)).



(a) (b)

Figure 3.24 - Effect of excavation rate on (a) normalized maximum ground deformation and (b) normalized maximum wall displacement when slow excavation rate follows beyond 5.5 m, 8 m and 10 m respectively for excavation depth 14 m.

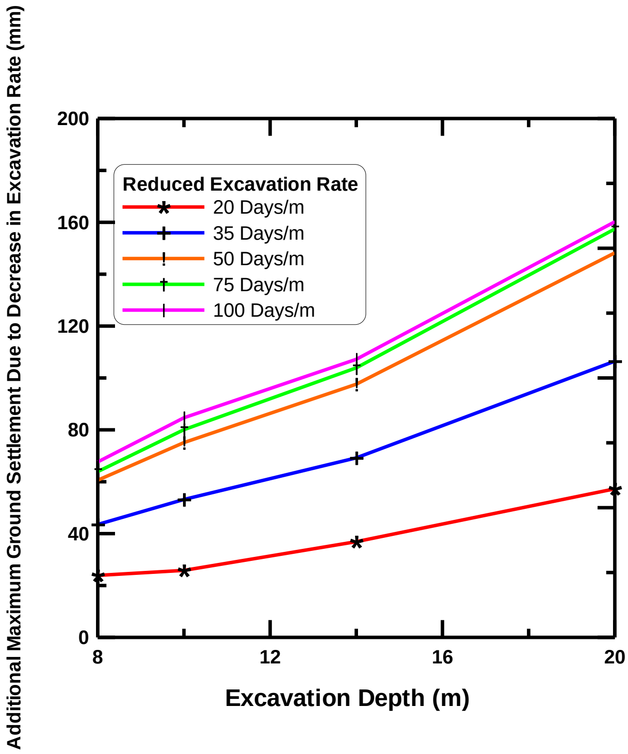


(a) (b)

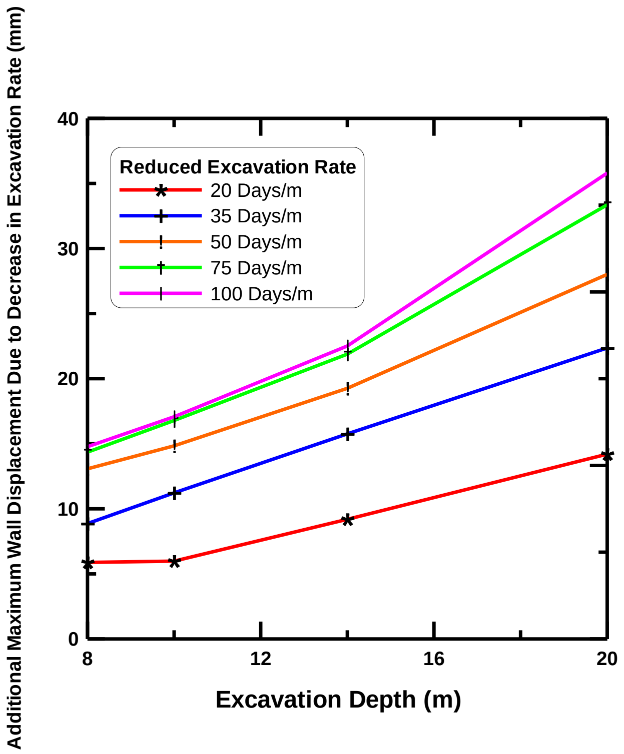
Figure 3.25 - Effect of excavation rate on (a) normalized maximum ground deformation and (b) normalized maximum wall displacement, when slow excavation rate follows beyond 5.5 m, 8 m and 10 m respectively for excavation depth 20 m.

Figure 3.24(a) and 3.24(b) reveal that with reduction of excavation rate from 10 day/ m to 50 days/ m normalized maximum displacements increase rapidly but with further reduction of rate of excavation, rate of increment of displacement values are not considerably high. Identical results are obtained for excavation depth 20 m as observed from figures 3.25 (3.25(a) and 3.25(b)). Here maximum deformation values are greater than those obtained with excavation depth of 14 m. It is further monitored that for three situations mentioned above, deviation of normalized maximum deformation at any certain excavation rate is almost same and lesser values obtained for third condition (when standard rate followed up to 10 m).

Further effect of excavation depth with various excavation rates is assessed. Here, the excavation is performed up to a depth of 5.5 m with a rate of 10 days/m beyond which the excavation is performed at different reduced rates (mentioned above) down to depths of 8 m, 10 m, 14 m and 20 m. Additional maximum ground settlement is calculated by computing difference of maximum settlement obtained from using delayed excavation rate and maximum settlement acquired when standard excavation rate is considered. It is observed from figures 3.26 (3.26(a) and 3.26(b)) that additional maximum ground settlement and wall displacement increase with increase of excavation depth. For greater excavation depth with similar reduction of excavation rate maximum deformations increases rapidly. Thus, it is advisable to avoid any kind of delay when excavation depth is more.



(a)

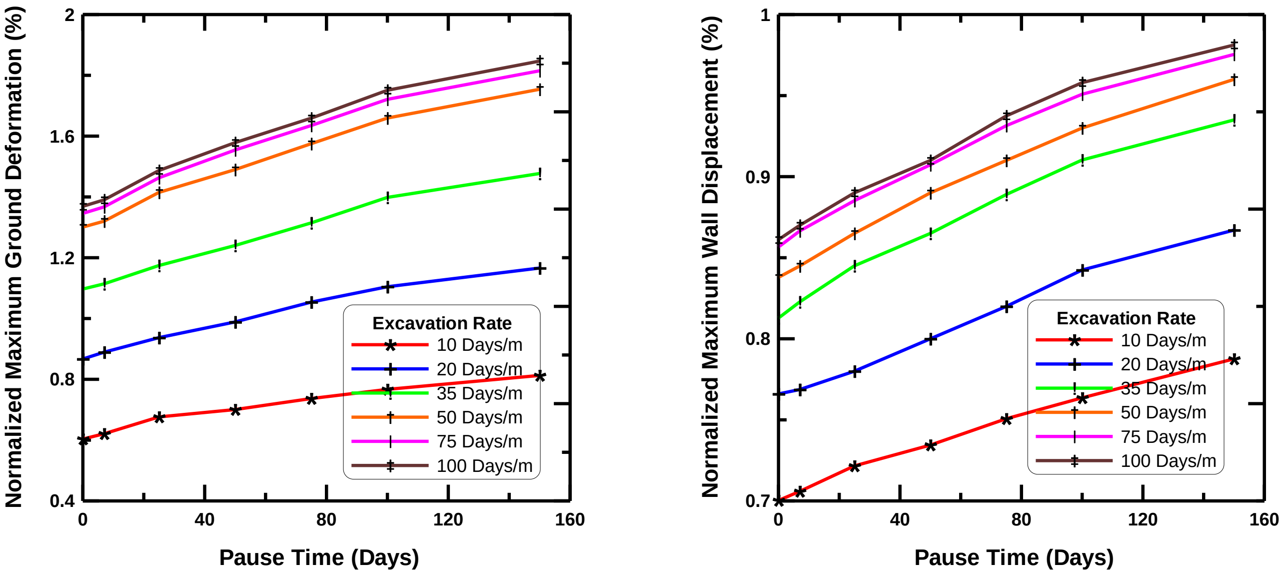


(b)

Figure 3.26 - Effect of reduced excavation rate on (a) additional maximum ground deformation and (b) additional maximum wall displacement for different excavation depth.

3.2.4.2 Effect of construction stoppage

Excavation work for metro railway construction in Kolkata has experienced significant delays in the past and it was observed that deformations in the braced system increased with time (Som 1994). Thus, the effect of construction delay (pause time) when it is left open after get to final excavation depth, should be systematically evaluated. Various cases are investigated with construction stoppages (pause times) of 7 days, 25 days, 50 days, 75 days, 100 days and 150 days. Excavation is assumed to be conducted with a standard excavation rate of 0.1 m/day (100 days/ m) down to 5.5 m depth and then excavation rate is altered. From 5.5 m depth to final excavation depth reduced rates (10 days/ m, 20 days/ m, 35 days/ m, 50 days/ m, 75 days/ m, 100 days/ m) are used for study and the construction stoppage occurs after reaching final excavation depth. Here normalized maximum ground and wall displacement are drawn against pause time for different rate of excavation for excavation depth of 14 m and 20 m (figure 3.27 and 3.28).



(a) (b)

Figure 3.27 - Effect of pause time on (a) normalized maximum ground deformation and (b) normalized maximum wall displacement with different excavation rate for excavation depth 14 m.

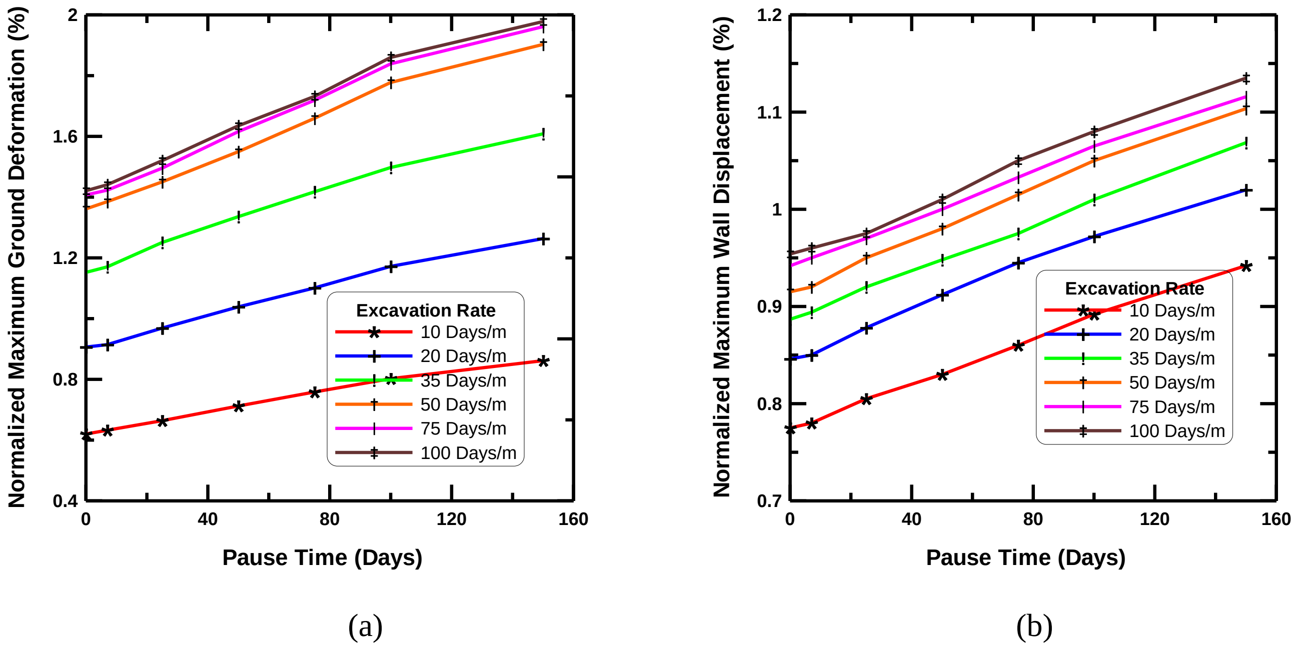
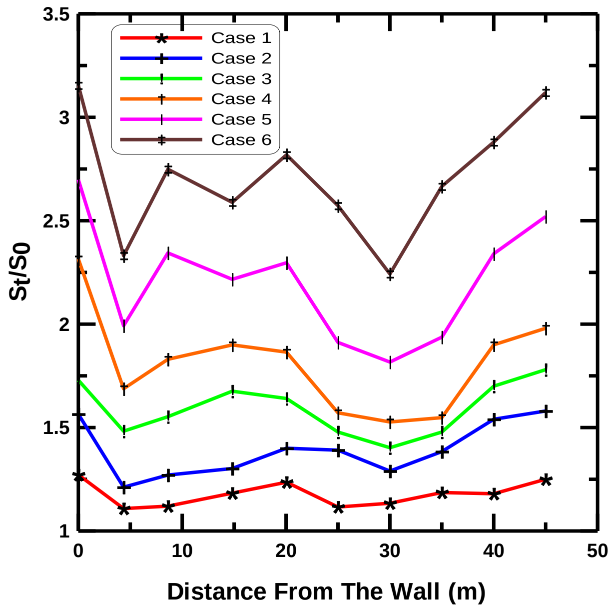


Figure 3.28 - Effect of pause time on (a) normalized maximum ground deformation and (b) normalized maximum wall displacement with different excavation rate for excavation depth 20 m.

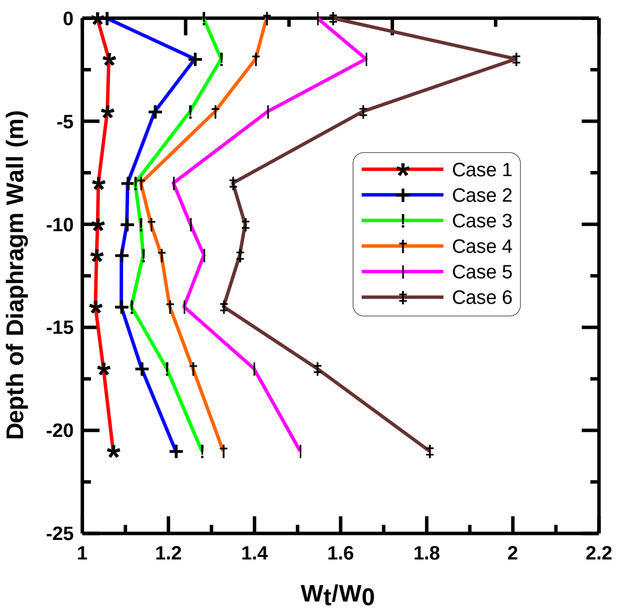
Thus, combined effect of rate of excavation and stoppage time is assessed. From both figures 3.27 and 3.28 it is quite evident that due to pause in construction ground and wall deformation increase uniformly. Though in case of ground deformation, for both excavation depth 14 m and 20 m (fig 3.27 (a) and fig 3.28 (a)) rate of increase is slightly more than wall displacement when slower rate of excavation is adopted. From the plot of wall deformation against stoppage time (Fig 3.27 (b) and 3.28 (b)) irrespective of rate of excavation rates of increase of displacements are similar. An example can be taken from figure 3.27 (a) to realize how pause time effects on deformation characteristics of braced excavation separately and combined with rate of excavation. If standard excavation rate is followed throughout the depth of excavation without any pause time maximum normalized displacement is computed as 0.6036% or displacement value of 84.51 mm. Due to 50-days and 100-days halt after attaining full excavation depth normalized displacement goes up to 0.70% (or 98 mm) and 0.767% (or 107.38 mm) respectively. If excavation after 5.5 m is conducted with slower rate of 35 days/ m with same pause time of 50 days and 100 days cause normalized maximum deformation values of 1.24% (173.6 mm) and 1.39% (194.6 mm) and without pause time cause normalized maximum deformation values of 1.097% (153.58 mm). So, maximum displacement raises nearly 13% and 27% for 50 days and 100 days halt and when combined with slow excavation rate of 35 days/ m displacement values raise 105% and 130%

respectively. So, if construction stoppage and delayed excavation are taken place simultaneously deformation values increase alarmingly which poses serious threat to stability of excavation and adjacent structures.

Figure 3.29(a) and 3.29(b) are drawn to study the combined effect of construction pause and excavation rate on ground surface deformations at various distances behind wall and wall deformations at different points along depth respectively. Figures are drawn for excavation depth of 14 m and different cases are used as mentioned in Table 10. S_t and W_t are the ground settlement and wall displacement for different cases whereas S_0 and W_0 represent displacement values obtained when no delaying/ stopping occurs (Case 0).



(a)



(b)

Figure 3.29 - Effect of excavation rate and pause time on (a) normalized ground deformation and (b) normalized wall displacement.

Table 10 - Various cases used for investigation

Case	Excavation depth	Excavation rate up to 5.5 m	Excavation rate beyond 5.5 m	Pause time after reaching excavation depth
0	14 m	10 days/ m	10 days/ m	0 days
1	14 m	10 days/ m	10 days/ m	25 days
2		10 days/ m	10 days/ m	100 days
3		10 days/ m	20 days/ m	25 days
4		10 days/ m	20 days/ m	100 days
5		10 days/ m	50 days/ m	25 days
6		10 days/ m	50 days/ m	100 days

From figure 3.29 (a) it is seen that at various locations increase in ground surface settlement are quite large especially when excavation is done with slower rate followed by stoppages. The values of ground settlement can be as high as 3 to 3.15 times (increase of 200% to 215%) of values obtained when neither slower excavation rate is followed nor stoppage occurs. The increment is smaller in the region close to the wall and higher at some distance from wall. As the values of settlement beyond 30 m are small, even if ratios of settlement with standard condition are producing large values, it is not the matter of concern. In case of wall displacement (figure 3.29(b)) the ratio of displacement values for the cases considered to the values for standard condition is limited to 1.05 to 2.0 (increase of 5% to 100%).

3.2.4.3 Discussions

From the above study following observations are observed.

1. If slower excavation rate is combined with construction stoppage it causes excessive ground movement. This situation should be avoided to restrict damage on adjacent structures

2. When excavation depth is greater (20 m or more), displacement values change very rapidly with reduced excavation rate. Designer should be careful so that this situation will not arise.
3. Increases of soil movements are less significant near the wall and far behind the wall due to construction stoppage and/ or slower rate of excavation. Though variations of deformations are considerably high at some distance behind wall (generally 0.5 to 1.8 times depth of excavation).
4. When excavation rate is reduced up to 50 days/ m soil deformation increases steadily but if further delayed excavation rate is followed small additional displacement is observed.
5. Time effect has lesser impact on wall movement than ground surface movement. Ground settlement can increase by about 215% or even more while wall movement can increase close to 100% at certain locations. Though increases of maximum ground and maximum wall deformations for delayed construction are about 184% and 40 % respectively.
6. Due to only pause in construction maximum ground settlement increases around 35 to 40% while values of maximum wall displacement go up in the region of 12 to 20%.

3.2.5 REGRESSION ANALYSIS

Multi-variable regression analysis is conducted to generate fitted equations for prediction of maximum ground and wall displacement. Time dependent parameters like excavation rate (E_R), pause time (S_T) and excavation depth (H_{exc}) are considered as input variables to form equations. The model is developed on the basis of data produced artificially from the results of FE analysis of total sixty numbers of hypothetical excavations in soft clayey soil. Proposed equations can be employed to estimate deformation values readily when values of other excavation related parameters and soil parameters are identical with that of cases used for developing model. To check the correctness of proposed equations field data are collected from specific sites of Kolkata Metro Construction where time plays a major role to enhance displacement values due to slower rate of excavation and construction stoppages for various reasons. However, it should be kept in mind that these equations should be used for quick preliminary estimation of deflection values without conducting any kind of FE analysis.

Performing multi-variable regression analysis following simplified equations are proposed

$$\delta_{vm} = \alpha_0 + \alpha_1 H_{exc} + \alpha_2 E_R + \alpha_3 S_T \tag{7}$$

$$\delta_{hm} = \beta_0 + \beta_1 H_{exc} + \beta_2 E_R + \beta_3 S_T \tag{8}$$

δ_{vm} and δ_{hm} represent maximum vertical movement of ground surface and maximum horizontal displacement of wall respectively. Here wide ranges of values are taken for parameters such that proposed equations can be applied effectively to assess effects of delay in construction on deformation behaviour of excavation. From analysis coefficients of above equations are found out as follows

α_0 = -75.01798, α_1 = 8.00306, α_2 = 2.341014, α_3 = 0.200569

β_0 = -60.7666, β_1 = 10.2111, β_2 = 0.889056, β_3 = 0.145895

The values of R square and adjusted R square for equation 7 are 0.9224 and 0.9207 while for equation 8 these values are obtained as 0.9791 and 0.9787. The high values of R square and adjusted R square give an indication that proposed equations can produce satisfactory results.

To validate model eleven different sites of Kolkata Metro Construction are taken into account. Details of time of excavation and settlement data are tabulated in Table 11. Measured settlement data on reaching excavation depth without pause and after pause in construction are obtained for comparison.

Table 11 - Time of excavation and Settlement data at different sections of Kolkata Metro Construction (Som (1994))

Section	Depth of excavation (m)	Average rate of excavation (days/ m)	Pause in Construction (days)	Settlement on reaching excavation depth (mm)	Settlement after pause in construction (mm)
2	9.6	18.5	152	44	64
7/161	10	25.5	90	41	58
7/170	11	23.2	540	90	152
7/170A	9	26.6	450	71	145
7/162	14	30.7	730	97	172
7/164	14	30.7	720	128	264

8	14	29.3	80	104	136
10	9.5	15.3	39	80	90
12	11.6	34.2	158	65	90
15	12.5	17.9	65	45	61
17	10	28.9	232	94	125

Predicted values of ground settlement are calculated using equation 7 for both cases of without stoppage and stoppage after construction. The estimated values are plotted with reported values and it is shown in figure 3.30. From figure it is observed that most of the points are very close to 45^0 line which establishes the fact that proposed equation can produce satisfactory results. There are little variations in result for some cases which may be due to effects of parameters like strut location, depth of embedment of wall etc. Root means square error (RMSE) and normalized root mean square error (NRMSE) for maximum ground settlement values of above-mentioned sites are computed. The table 12 reveals that NRMSE values are below 13% which means prediction is quite good for preliminary estimation of time dependent deformation values especially when excavation is done in soft clayey soil. Further from student's t-test (Table 12) which is performed to relate predicted and observed values it is seen that calculated t values are less than $t_{0.05}$ and p values are greater than 0.05.

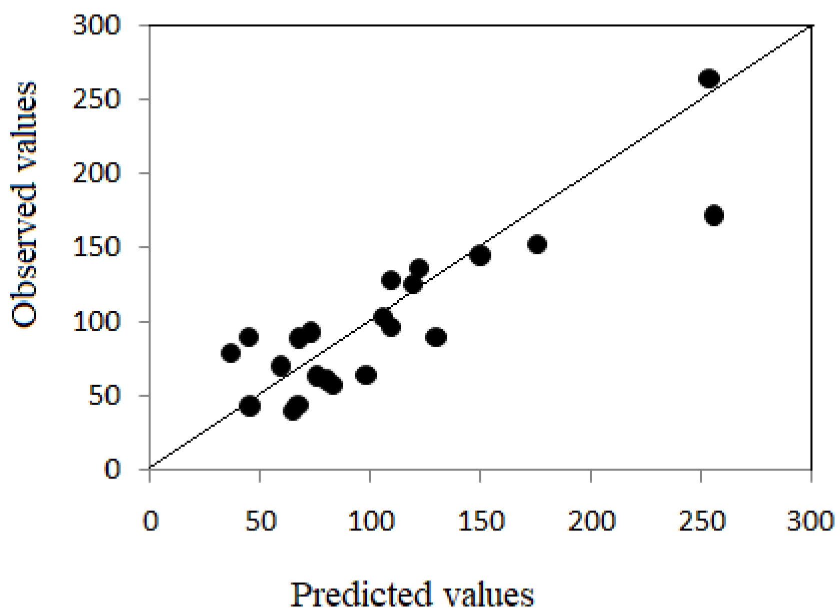


Figure 3.30 - Comparison between observed (Som 1994) and predicted values at various sections of Kolkata Metro Construction.

Table 12 - Results of NRMSE computation and student's t-test

Computed parameter	Values
RMSE	2.8767
% NRMSE	12.9
Calculated t value	0.28452
Critical t value at 5% level of significance	2.01954
p or probability value	0.77745

3.2.6 CONCLUSION FROM SECTION 3.2

In the above study described in section 3.2, systematic FE analysis of typical braced excavation systems are performed considering soft clayey soil where considerable secondary and creep settlements occur. The mechanism of time dependent deformation behaviour of braced excavation is investigated in detail to identify undrained settlement, consolidation settlement and creep settlement separately using soft soil creep model (SSCM) and soft soil model (SSM). Consolidation settlement increases as excavation proceeds but after reaching final stage of excavation no dissipation of pore water pressure is observed. Considerable creep settlement is observed in soft Kolkata soil due to its high compressible nature. As excavation rate and construction stoppage plays a major role to determine magnitude of soil movement around excavation the impact of these two parameters is studied. It is observed from the different cases investigated with different rates of excavation that slower excavation rate causes significant increase of ground deformations especially for higher excavation depth. So, the need of construction control in braced excavation is critically important especially in soft clayey soil.

Further simplified equations are provided for quick estimation of maximum ground and lateral wall displacement performing multi-variable regression analysis. Time dependent parameters like rate of excavation and pause times along with depth of excavation are used as input parameters for predictions and model equations are developed based on the results of FE analyses of hypothetical excavation cases in soft clay. The applicability of proposed model is established comparing results with measured deformation values obtained at different sections of Kolkata Metro Construction where it is computed that normalized root mean square error (NRMSE) of predicted value is satisfactory (12.9%).

CHAPTER 4.0 - EXPERIMENTAL STUDY – TIME DEPENDENT GROUND SETTLEMENT USING GEOTECHNICAL CENTRIFUGE

4.1 General

Centrifuge testing concerns the study of geotechnical events using small-scale models which are subjected to acceleration fields many times Earth's gravity. Using this technique, self-weight stresses and gravity dependent processes are correctly modeled and thus observations from small-scale model experiments can be related to the full-scale prototype situation using well established scaling laws. This is extremely important as soil is a highly nonlinear and history dependent material and so modelling using geotechnical centrifuge provides good control on the overall behavior of the construction. This is not possible with the numerical analyses which depend on soil models used and the associated material parameters chosen, construction sequence and most importantly, uncertainties due to imperfect knowledge of the true soil profile and conditions. Thus, data from centrifuge tests provide a good compliment to careful instrumented field case studies and associated numerical analyses.

It is important to note that a major part of Kolkata strata is covered with soft clay down to a depth of 15-16 m below the existing ground level. During various excavation work for different infrastructure projects in Kolkata, sometimes it is seen that the work is delayed or slowed down because of different reasons beyond the control of the engineer-in-charge. These kinds of delays result in increase in the rate as well as the magnitude of deformation of the soil within the influence zone of the excavation. From literatures cited earlier it is quite clear that delay during construction significantly increase the magnitude of soil displacement within influence zone of braced excavation in soft clay. As limited works have been done on this important issue on time dependent deformation behavior, it is attempted to analyze the parametric behavior of braced excavation by conducting model tests using geotechnical centrifuge at Jadavpur University, Kolkata.

In the present investigation, a systematic study of the layer wise ground deformation behind a braced excavation in soft clayey soil similar to that available in Kolkata is performed using physical model study for different depth of excavation, number of struts and also considering construction delay or construction stoppage after reaching the final cut level. The mechanism of this deformation was also assessed by evaluating the contribution of undrained, consolidation and creep deformation to the total ground deformation. The tests results were also used to predict the effect of construction delay on various important factors like rate of settlement, change of zone of influence behind a braced wall etc. Further, the experimental results are also validated with the observed values obtained from reported case studies (Som, 2000; Ou, 1998 and Liu, 2005).

4.2 EQUIPMENT AND METHODOLOGY

4.2.1 Geotechnical centrifuge

The centrifuge experiment presented in this study was carried out on the fixed beam geotechnical centrifuge facility installed in Soil Mechanics laboratory of Civil Engineering Department at Jadavpur University, Kolkata (Figure 4.1). The 1m diameter centrifuge is designed for a payload capacity of 15 kg with maximum 255g capacity. The centrifuge has two buckets of size 315 mm (W), 315 mm (L) and 225 mm (H) fabricated from MS plates. The assembly consisting of CI bearing housing, EN24 spindle shaft, top and bottom bearing covers and three bearings rests on the Inertia Plate resting on 4 numbers of Anti-Vibration-Mounts. Outer body is fabricated from 8 mm thick MS plate having approximately of 1250 mm and height of 650 mm. The 15 kW 3 phase VFD duty motor is mounted vertically with shaft downwards having V-Belt drives with a pulley on motor shaft drive pulley on spindle shaft. The variable frequency drive provides smooth acceleration to the buckets and the maximum speed of the spindle shaft is 550 RPM for 155g.

Figure 4.1 – Geotechnical Centrifuge at Jadavpur University

In the present study specimen container of size 300 mm (L), 300 mm (W), 300 mm (H) made of acrylic sheet with mild steel stiffener was used and have been provided with scales in the front sheet for measurement of deformation (Figure 4.2). All the tests during the present investigation have been done at 100g and to maintain this the centrifuge machine was rotated at a frequency given as, $\omega^2 r_e = Ng$, where, ω = Angular velocity, r_e = Nominal Radius of the geotechnical centrifuge (0.4626 m) [Madabhushi (2015)], N = Geotechnical centrifuge constant (100) and g = Acceleration due to gravity (9.81m/sec), which gives ω = 46.05 rad/sec. the corresponding frequency = Frequency = $\omega/2\pi$ = 439.74 RPM = 440 RPM

Figure 4.2 - Container used in Geotechnical centrifuge

The scaling law has been applied for fixing the dimensions of the model by multiplying the prototype dimensions by 1/N. 14 m excavation depth in prototype scale has become 140 mm in model. Again, using scaling law, given in table 13 for testing at Ng condition, the model time may be obtained by multiplying the prototype time by $1/N^2$ [Madabhushi (2015)]. For example, model timing in case of 100g experiment for prototype time of 180 days, i.e., 6 months, is $(180*24*60)/100^2$ = 25.92 minutes.

Table 13 - Centrifuge Scaling Law

	Parameter	Scaling Law Model/Prototype	Units
General Scaling Laws	Length	1/N	m
	Area	$1/N^2$	m^2
	Volume	$1/N^3$	m^3
	Mass	$1/N^3$	$Nm^{-1}s^2$
	Stress	1	Nm^{-2}
	Strain	1	-
	Force	$1/N^2$	N

	Bending Moment	$1/N^3$	Nm
	Time (Consolidation)	$1/N^2$	sec

4.2.2 Experimental setup and model configuration

The centrifuge container which has been used for the tests has an internal width of 300 mm, length of 300 mm and a depth of 300 mm, which can produce a 280 mm to 290 mm thick soil layer. This box is designed for plane strain excavation tests. This box is designed for plane strain excavation tests by making it sufficiently rigid with 20 mm thick perspex sheets encased in welded mild steel cage having a viewing window on the front side. The box is made of mild steel strips and perspex plates with scaling on the viewing face in order to facilitate determination of the deformation of kaolin layers provided at different levels in the test bed by capturing the video photograph. The tests described in this research were carried out at a scale of 1:100.

The model diaphragm wall was made from stainless steel with thickness of 3 mm, having an equivalent bending stiffness (EI) at prototype scale of approximately $4.92*10^6$ Nmm^2 representing a concrete diaphragm wall with thickness of about 0.6 m.

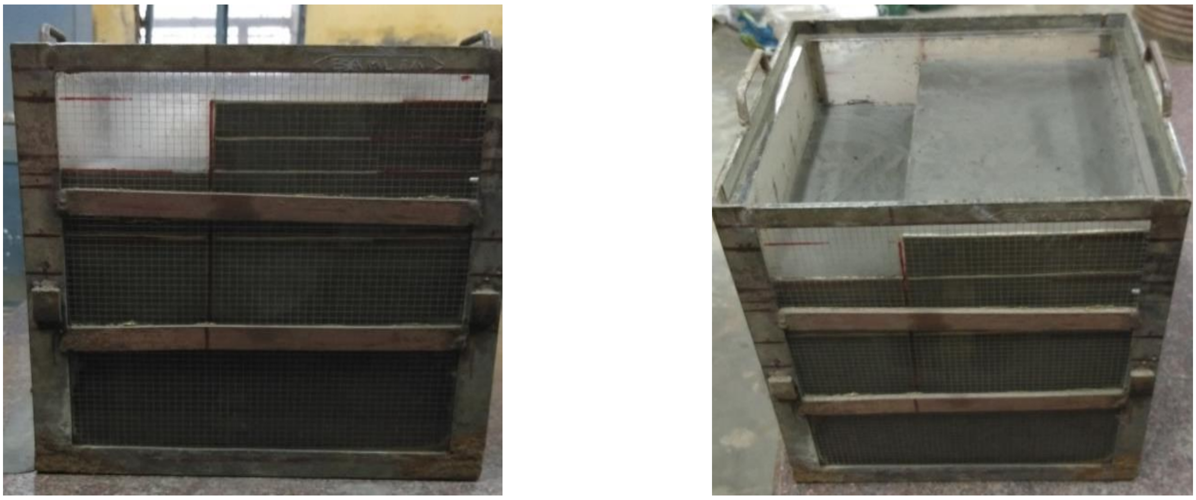
This model wall was embedded 170 mm into the original ground level, representing 17 m at prototype scale. Rubber flippers were attached to both sides of the model wall to prevent water seepage through the excavation side during excavation. The flippers were greased to ensure free sliding movement of the wall. In the present study there was no provision of in-flight automatic excavator. After preparing the soil bed excavation was made down to required depth along with installation of struts at the specified positions, as the case may be.

The model strut was made from stainless steel of size 3mm x 3 mm, having an equivalent bending stiffness (EI) at prototype scale of approximately $1.836*10^6$ Nmm^2.

In the present study four sets of centrifuge tests (14 numbers) have been conducted (excluding the repeated test) under 100g conditions to evaluate the settlement behavior of behind a braced excavation. The details of the experimental program have been tabulated in table 14. Some typical photographs of the model test bed for cases 4, 6, 11 and 14 with different layers of struts has been given in Figure 4.3 (a-d) respectively.

Table 14 – Experimental Program for the Study

Set Number	Case Number	Excavation Depth (mm)	Number of Strut	Strut Position (Below G.L.)
1	1	20	0	--
	2	30		
	3	40		
	4 (figure 4.3a)	50		
2	5	40	1	20 mm
	6 (figure 4.3b)	50		
	7	60		
	8	70		
3	9	80	2	20 mm and 45 mm
	10	100		
	11(figure 4.3c)	120		
4	12	120	3	20 mm, 45 mm and 105 mm
	13	130		
	14(figure 4.3d)	140		



(a)

(b)

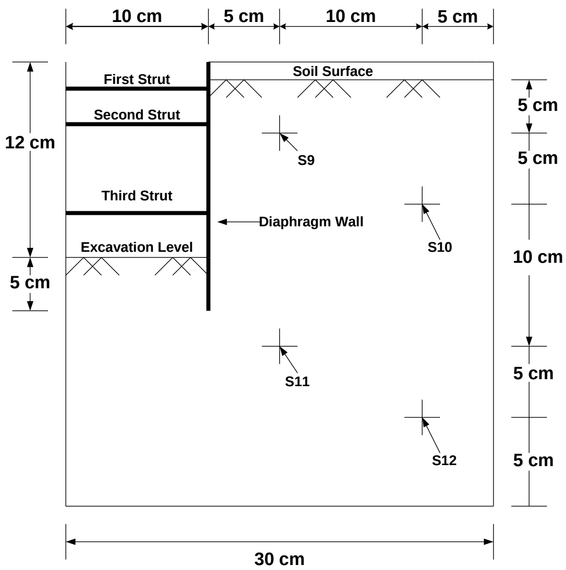
(c)

(d)

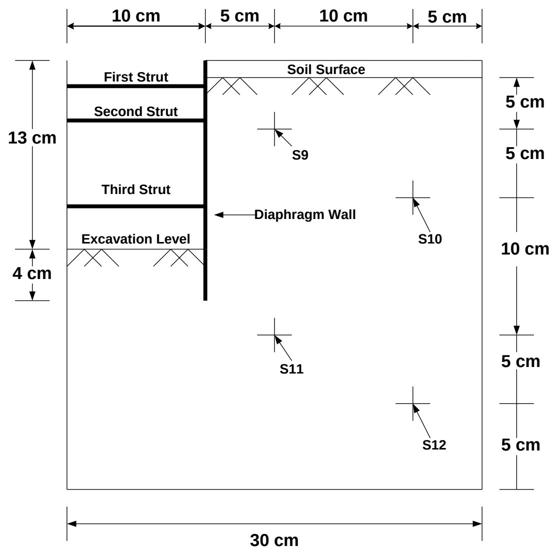
Figure 4.3 - Model test bed in the test box for (a) Case 4, (b) Case 6, (c) Case 11 and (d) Case 14.

4.2.3 Model bed preparation

Soft dark grey silty clay / clayey silt collected from a site near Jadavpur University was used in this study to prepare the test bed. The basic tests of this local soils have been conducted by collected the soil sample from four different positions and depths of each model so that uniformity of test results can be maintained, for case 12 the samples names are S1, S2, S3, S4 respectively, for case 13 the samples names are S5, S6, S7, S8 respectively and for case 14 the samples names are S9, S10, S11, S12 respectively is shown in figure 4.4 (a, b, c). Similar pattern has been followed for soil sample collection for other 11 cases also.



(a)



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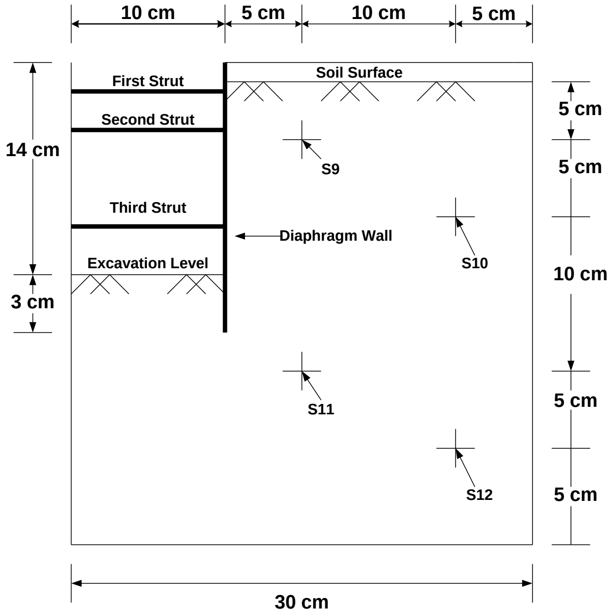


(c)

Figure 4.4 - Soil samples collected positions for (a) case 12, (b) case 13 and (c) case 14.

Liquid limit and plastic limit and percentages of sand, silt and clay of the soil was determined and are given in Table 15. For preparation of test bed dry soil was grinded and mixed with water thoroughly at a predetermined moisture content of 36%. This soil was then taken in the test container in layers of thickness not more than 120 mm and then compacted by 25 numbers of strokes were given by a tamping rod in order to maintain the suitable density of 1.72 gm/c.c. The compaction procedure was fixed by a number of trials in order to achieve required density and undrained cohesion of 2.1 t/m^2. Before preparing the test bed a thin layer of silicon grease was coated on the inner wall of the container, to reduce the impact of side friction [Tan (2003), Kongsomboon (2004), Ong (2006) and Sonnenberg (2010)]. This lubrication facilitates free movement of soil bed adjacent to the rigid container walls, so there will be no out of plane deformation and thereby plane strain condition will prevail throughout the tests. After filling of soil sample up to 120 mm the centrifuge machine is rotated for 10 minutes. After that a diaphragm wall of 170 mm height is fixed and then the rest 170 mm soil layers is laid on both sides of diaphragm wall in 4 layers. Kaolin layers are given in between each layer in order to get the photograph of deformation pattern of soil bed with depths. After filling the sample container, the centrifuge machine is again run for 20 minutes to get the in-situ condition before excavation. After that the excavation to the required depth along with installation of struts has been done. The same procedure has been maintained for preparation of sample in the counterweight container also. Soils were collected from four different positions and depths of each test bed, in some cases before the tests and in some cases after the tests, for determining moisture content, bulk density and undrained shear strength of soils in the test bed in order to check the uniformity of test bed. Average properties of the soil bed which is obtained by obtaining and averaging results from figure 4.5 (a, b, c, d) is given in Table 15.

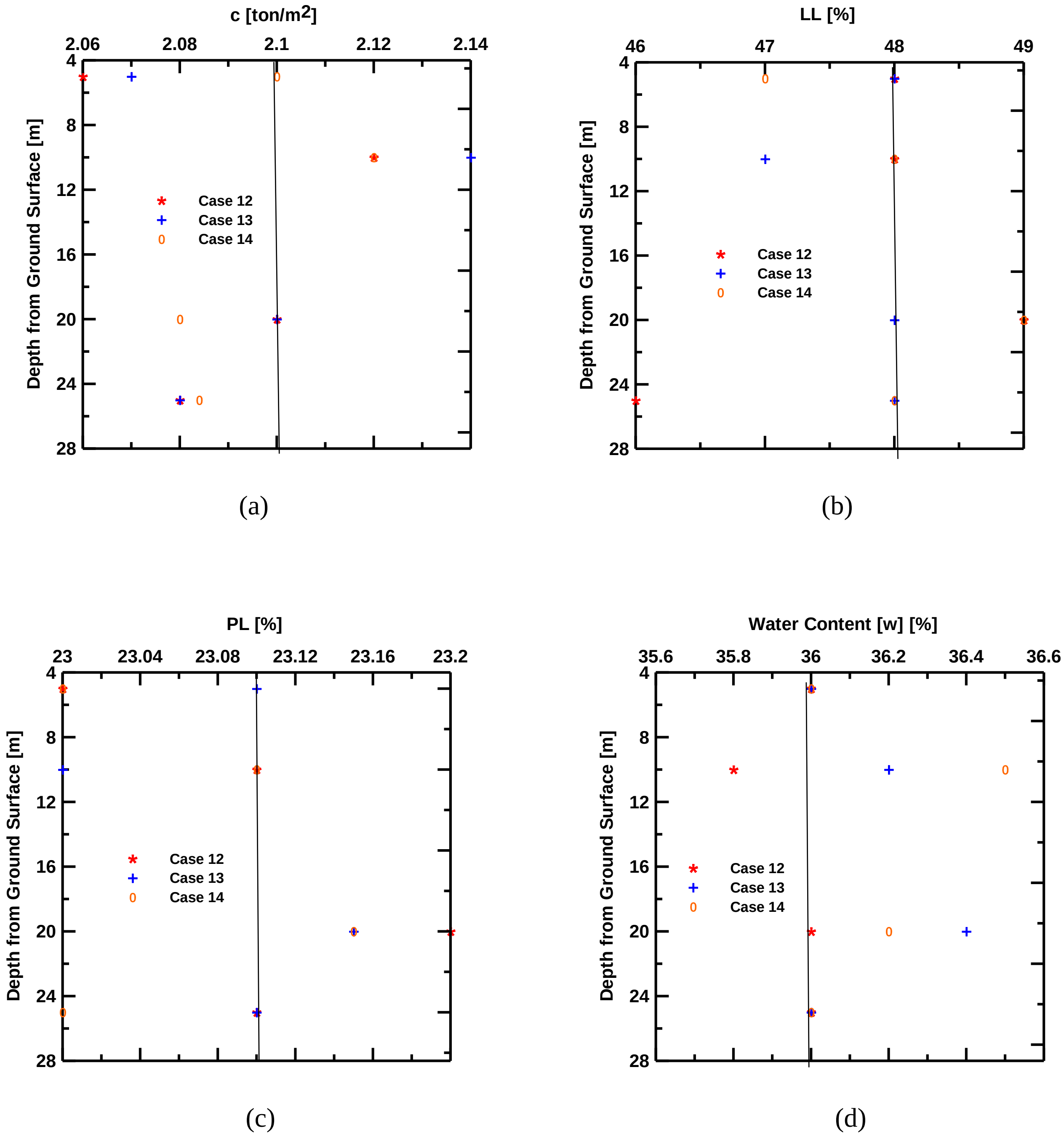


Figure 4.5 - Properties of soils for different depths from ground surface (a) cohesion(c) (b) Liquid Limit (LL), (c) Plastic Limit (PL), (d) Water Content (w).

Table 15 - Average soil properties obtained from test

Soil layer	Liquid Limit (%)	Plastic Limit (%)	Unit weight, γ (gm/cm^3)	Undrained shear strength (ton/m^2)	Water Content (w) (%)
Dark grey silty clay/ clayey silt	48	23.1	1.72	2.1	36

4.2.4 Test Methodology

After the test bed is prepared hand excavation down to required depth has been made and struts at required depth, as specified in Table 11, has been installed. Then the test box is placed at the specified position in the centrifuge. After that the geotechnical centrifuge machine was started along with the synchronized stroboscope which lighted up as the main test box came to a fixed position below the viewing window. This facilitates capturing of the image of the deformed soil bed after a fixed time interval during the test. During the test it was observed that the rate of deformation of the test bed was initially high and became slow with time. The tests were continued for a sufficiently longer period of time of 35 – 40 minutes which is equivalent to about 250 – 280 days when the change in deformation is practically negligible.

4.2.5 Image processing

Image processing techniques have become more widely applied in centrifuge laboratories around the world [Garnier (1991), Allersma (1991), Davies (1998) and Taylor (1998)]. In the present study a video camera is mounted in front of the perspex window to capture the videos of the deformation pattern of kaolin layers at different depths in the test bed with respect to the scales provided on the viewing surface of the container. Image processing of the recorded video was then done using video editor program in Matlab to obtain still image at different time interval. After getting the still image Matlab programming is again used to determine the magnitude of deformation.

4.3 Results and Discussions

During the test it took around 150 seconds or 2.5 minutes to arrive 100g environment after which the speed of the centrifuge became constant as shown in figure 4.6. In this figure increase in settlement just behind the wall is also plotted with time which reveals that deformation increases at uniform rate up to 150 seconds and thereafter no significant increase in settlement is observed for a period of 60 seconds which corresponds to about 7 days in prototype. After the speed became constant the tests were run for further about 40 minutes to assess the influence of pause time on the deformation behaviour of the soil behind the wall. A number of typical plots of time versus settlement for a typical case corresponding to 140 mm excavation depth with 3 struts are given in figure 4.7 in which deformation at zero day corresponds to that of 150 seconds in figure 4.6. This has been considered because the settlement for all the cases were more or less constant up to about 60 seconds after the speed of centrifuge became constant and after that it increased steadily with time. This phenomenon indicates that the ground settlement behind the braced wall was elastic during the initial startup time of 150 seconds and also during further 60 seconds (equivalent to 7 days) after the speed became constant and beyond this soil deformation was time dependent.

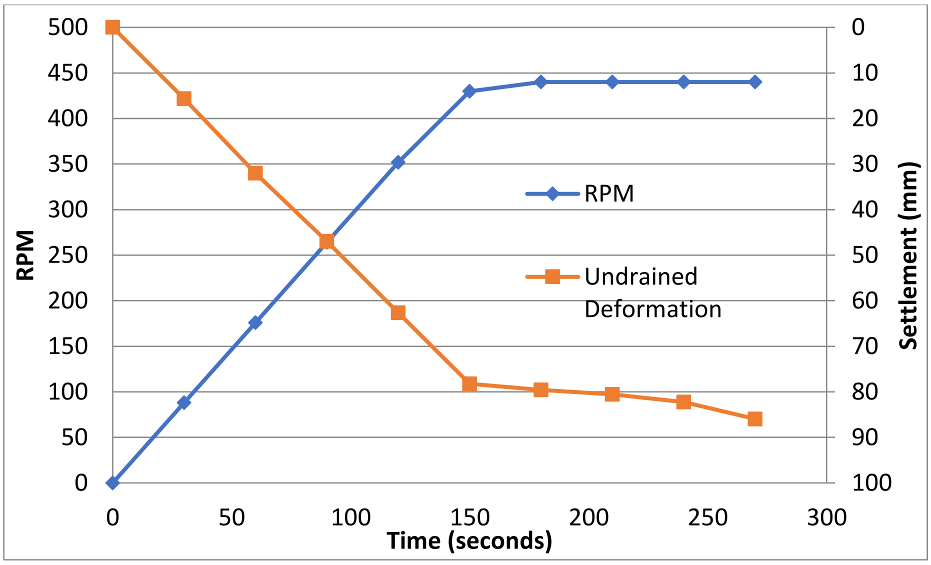


Figure 4.6 - Variation of frequency and ground settlement just behind the wall with time

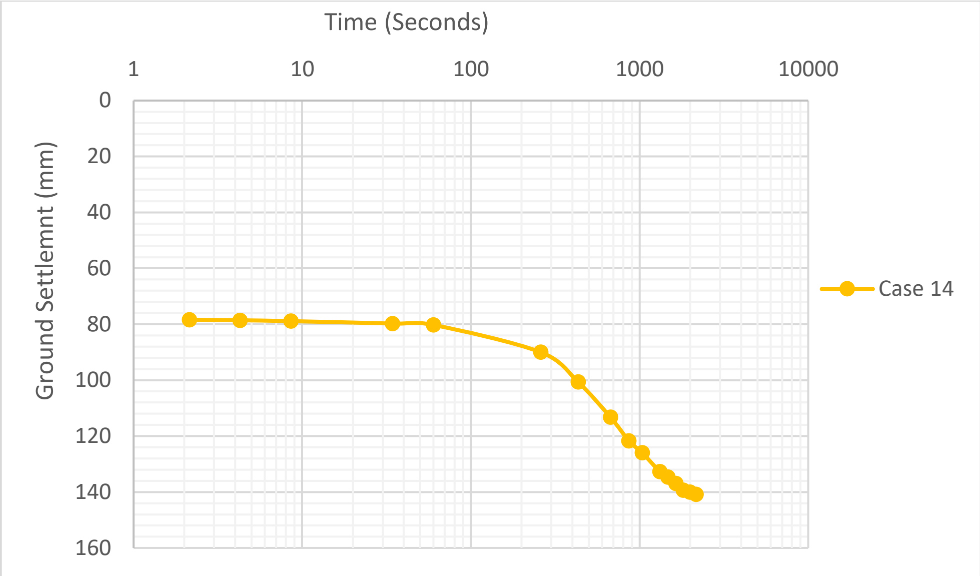


Figure 4.7 - Variation of ground settlement just behind the wall with time 14 m excavation depth (case 14)

4.3.1 Settlement profile due to elastic deformation

In this section ground settlement profile due to elastic deformation corresponding to no delay or pause during construction due to excavation of 4 m (Case 3), 6 m (Case 7), 10 m (Case 10) and 14 m (Case 14) at different depths below ground surface i.e., 0 m, 3 m, 7 m and 14 m has been presented. Photographs of ground deformations around the diaphragm wall as captured by camera before rotation and after 2.5 minutes of rotation (required to achieve 100g) for 14 m excavation depth with 3 struts are shown in figure 4.8. The variation of ground deformations, estimated by image processing using matlab software and obtained after 150 seconds (2.5 minutes) (time to reach 100g environment), on horizontal planes at different depths behind the diaphragm wall, have been plotted in figure 4.9. Similar pattern has also been obtained for other cases also. From figures 4.8 and 4.9 it has been observed that magnitude of vertical deformation was maximum just behind the wall as the soil in the test bed is of soft consistency and the model diaphragm was smooth in nature. The settlement behind the wall was shown to decrease considerably with distance away from the wall and with depth below the ground surface. Further, it is seen that the change in deformation with distance is steeper up to a distance of about 8 m away from the wall, beyond which it is comparatively flatter.


Figure 4.8 - Photographs of ground deformations before test and after rotation for 150 sec for case 14 (Excavation depth 14 m and 3 struts)

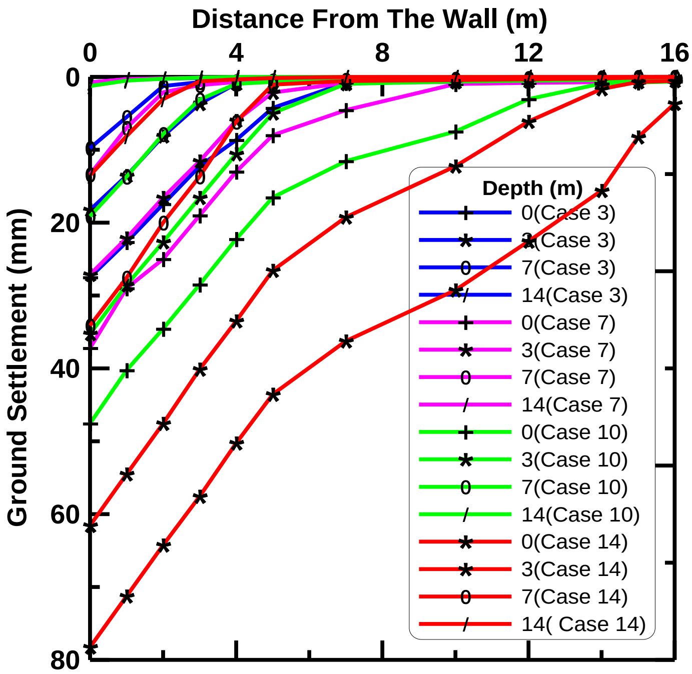
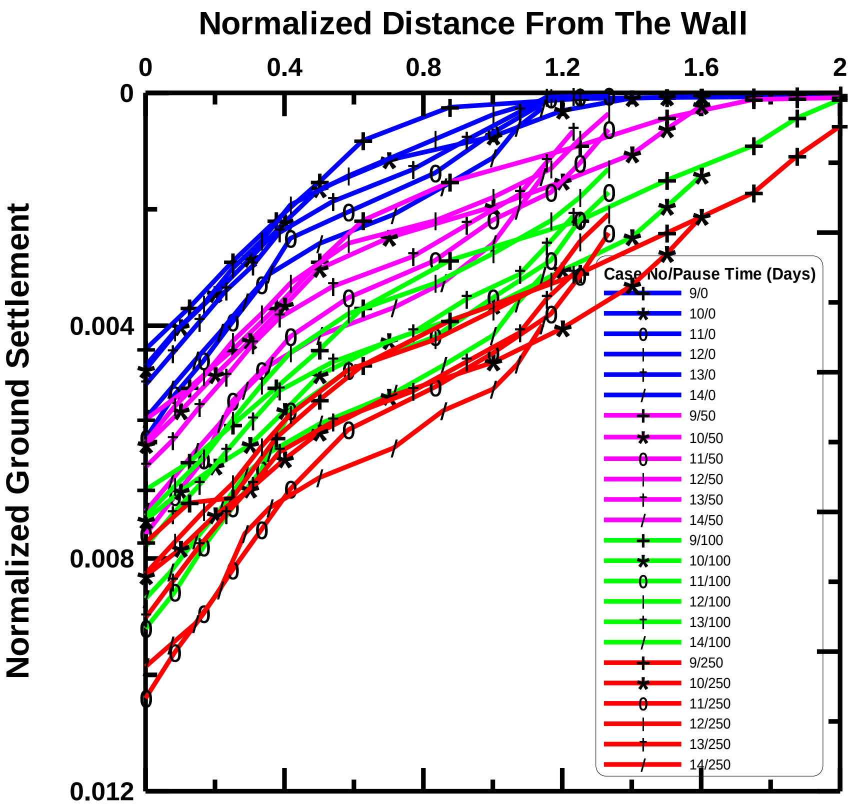


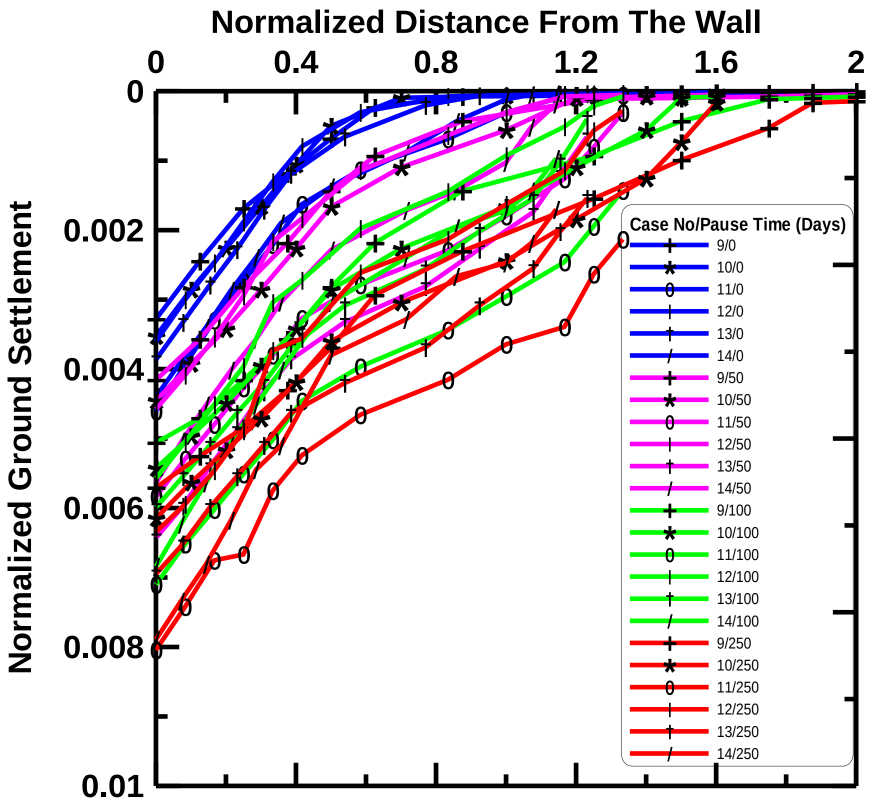
Figure 4.9 – Variation of ground deformations with distance behind the diaphragm wall at different depths below ground surface for (a) case 3, (b) case 7, (c) case 10 and (d) case 14.

4.3.2 Effect of pause time on settlement profile

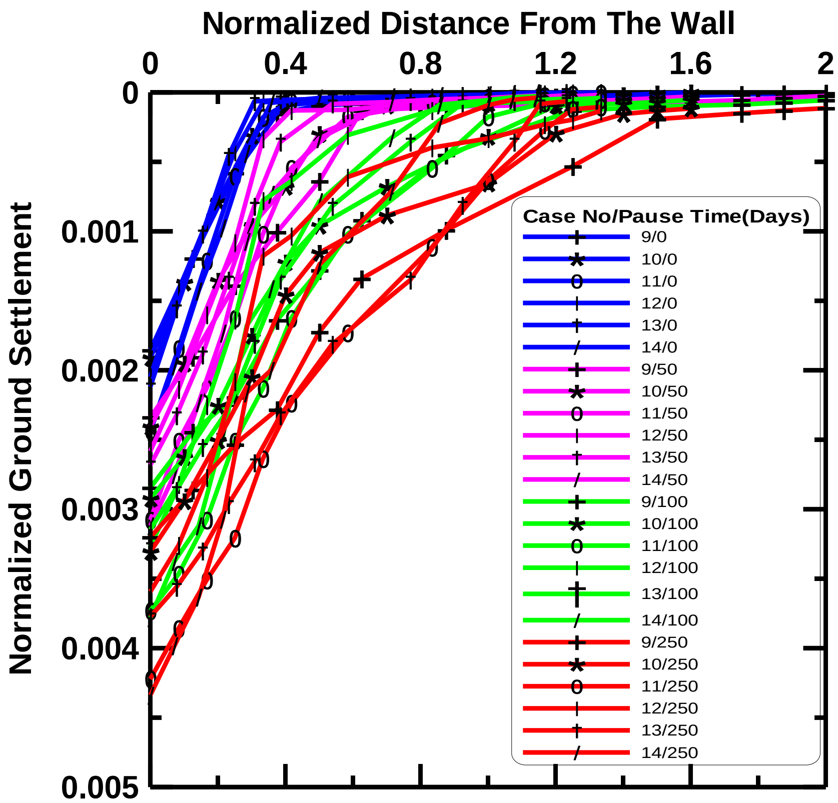
In this section normalized ground settlement at different normalized distance from the wall below ground surface i.e., 0 m, 3 m, 7 m and 14 m behind the diaphragm wall due to construction delay or pause in days during construction after reaching excavation depth of cases 1 to 14 has been studied using the model test results. The delay (in days) was achieved by rotating the centrifuge for about 35-40 minutes after the it reached uniform speed of 440 rpm corresponding to 100g condition and the photographs were taken at regular time interval for the deformed profile when the model bed came below the stroboscope. Using these data, change in normalized ground settlement profile of the soil bed with time have been derived by processing the image using Matlab software and plotted in figures 4.10 (a) – (d). In these figures 0-day normalized settlement profile is corresponding to the ground deformation after initial rotation of 2.5 minute in centrifuge, i.e., achieving 100g environment. Other profiles were corresponding to 7.2 minutes (50 Days), 14.40 minutes (100 Days) and 36 minutes (250 days) respectively. In figures 4.10 (a-d) measured values have been plotted up to a distance of 16m, corresponding to 160 mm in model in tank, due to the limited size of the test tank Ground deformation for different pause time and different distance from the wall are normalized with excavation depths respectively. From these figures it has been observed that as pause time increases normalized ground deformation increases and the rate of increment near ground surface is more prominent. However, the magnitude is found to reduce with depth below the ground. Further it has been also clearly observed that as pause time increases the zone of ground deformation profile shifts significantly towards higher values and the zone of deformation increases rapidly. So, it can be an alarming point to note that as pause time increases during construction the extent or area of the deformation zone also increases rapidly. Therefore, during any excavation project it is suggested to minimize the pause time between construction work so that the neighboring structure would not any effect of this deformation.


Normalized Distance From The Wall
0
0.4
0.8
1.2
1.6
2
0
0.004
0.008
0.012
Normalized Ground Settlement
Case No/Pause Time (Days)
9/0
10/0
11/0
12/0
13/0
14/0
9/50
10/50
11/50
12/50
13/50
14/50
9/100
10/100
11/100
12/100
13/100
14/100
9/250
10/250
11/250
12/250
13/250
14/250

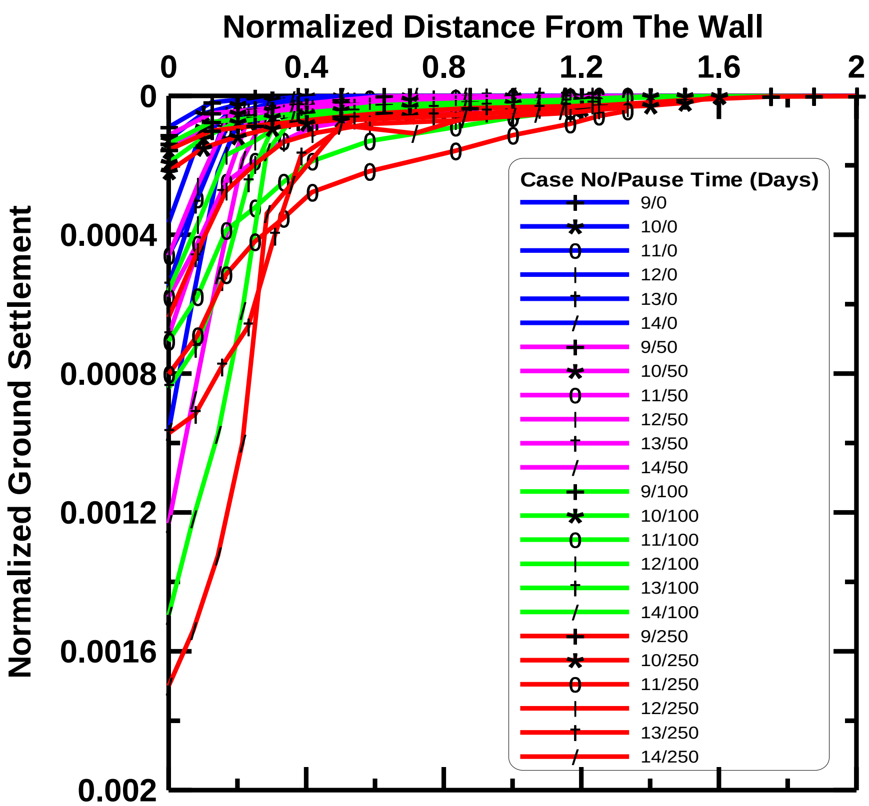
(a)


Normalized Distance From The Wall
0
0.4
0.8
1.2
1.6
2
0
0.002
0.004
0.006
0.008
0.01
Normalized Ground Settlement
Case No/Pause Time (Days)
9/0
10/0
11/0
12/0
13/0
14/0
9/50
10/50
11/50
12/50
13/50
14/50
9/100
10/100
11/100
12/100
13/100
14/100
9/250
10/250
11/250
12/250
13/250
14/250

(b)


Normalized Distance From The Wall
0
0.4
0.8
1.2
1.6
2
0
0.001
0.002
0.003
0.004
0.005
Normalized Ground Settlement
Case No/Pause Time(Days)
9/0
10/0
11/0
12/0
13/0
14/0
9/50
10/50
11/50
12/50
13/50
14/50
9/100
10/100
11/100
12/100
13/100
14/100
9/250
10/250
11/250
12/250
13/250
14/250

(c)


Normalized Distance From The Wall
0
0.4
0.8
1.2
1.6
2
0
0.0004
0.0008
0.0012
0.0016
0.002
Normalized Ground Settlement
Case No/Pause Time (Days)
9/0
10/0
11/0
12/0
13/0
14/0
9/50
10/50
11/50
12/50
13/50
14/50
9/100
10/100
11/100
12/100
13/100
14/100
9/250
10/250
11/250
12/250
13/250
14/250

(d)

Figure 4.10 – Normalized settlement profile at (a) ground surface (b) 3 m (c) 7 m and (d) 14 m depth for different cases [case 9 (E_D=8m-2 strut), 10(E_D=10m-2 strut), 11(E_D=12m-2 strut), 12 (E_D=12m-3 strut), 13(E_D=13m-3 strut) and 14(E_D=14m-3 strut)] and different pause time.

4.3.3 Variation of Undrained, Consolidation and Creep Deformation of Soil with Pause Time

In this section the ground deformation considering construction delay / pause after excavation of 14 m at different depths, 0 m, 3m, 7m and 14 m below ground surface and at a distance of 0 m, 4m, 7m, 10m and 15m away from the wall has been plotted in figures 4.11 (a) – (e) respectively. Similar study has been conducted for other cases also. From the figures it is clearly seen that the log(time) vs. ground settlement curve, in general, follows the pattern of one dimensional consolidation curve which indicates that the total ground settlement consists of 3 parts (i) settlement due to stress release during excavation which is obtained by taking maximum ground deformation reading while rotating geotechnical centrifuge for initial 2.5 minutes when the centrifuge achieved 100 g environment, and may be defined as undrained deformation. This is also taken as the difference between the original zero and R_0. [Where, $R_o = \{(\delta_{max(0.25)}) + (\delta_{max(0.25)} - \delta_{max(1)})\}$, $\delta_{max(0.25)}$ = ground deformation corresponding to 0.25 day and $\delta_{max(1)}$ = ground deformation corresponding to 1 day] (ii) Consolidation Settlement due to pore water pressure dissipation obtained by double tangent method as is done to separate primary consolidation part in one dimensional consolidation test ($R_0 – R_{100}$) as shown in log(time) – settlement plot in figures 4.11 (a)-(e) and (iii) Creep Settlement part which occurs after excavation and were evaluated using double tangent method as done to separate secondary consolidation part of 1D consolidation test plot (after R_{100}). A typical calculation for different components of settlement for ground surface (0 m) just behind the wall is given in table 16 and calculations for different components of settlement for ground surface (0 m), 3 m, 7 m and 14 m below ground surface and at a distance 4 m, 7 m, 10 m and 15 m away from the wall are calculated in the similar manner from figures 4.11 (b)-(e) respectively.

Table 16 – A typical calculation for different components of settlement for ground surface just behind the wall

Location – At the edge of the wall.

Depth from Ground Surface (m)	0	3	7	14
Undrained Deformation (mm)	77.88 (55.29%)	61.27 (55.44%)	33.9 (55.82%)	13.44 (56.48%)
Consolidation Deformation (mm)	(134.59-77.88) = 56.71 (40.26%)	(107.422-61.77) = 45.652 (41.31%)	(59.3-33.9) = 25.4 (41.82%)	(22.184-13.44) = 8.744 (36.75%)
Creep Deformation (mm)	(140.85-134.59) = 6.26 (4.44%)	(110.5-107.422) = 3.078 (2.79%)	(60.73-59.3) = 1.43 (2.35%)	(23.795-22.184) = 1.611 (6.77%)
Total Deformation after 250 days pause (mm)	140.85	110.5	60.73	23.8

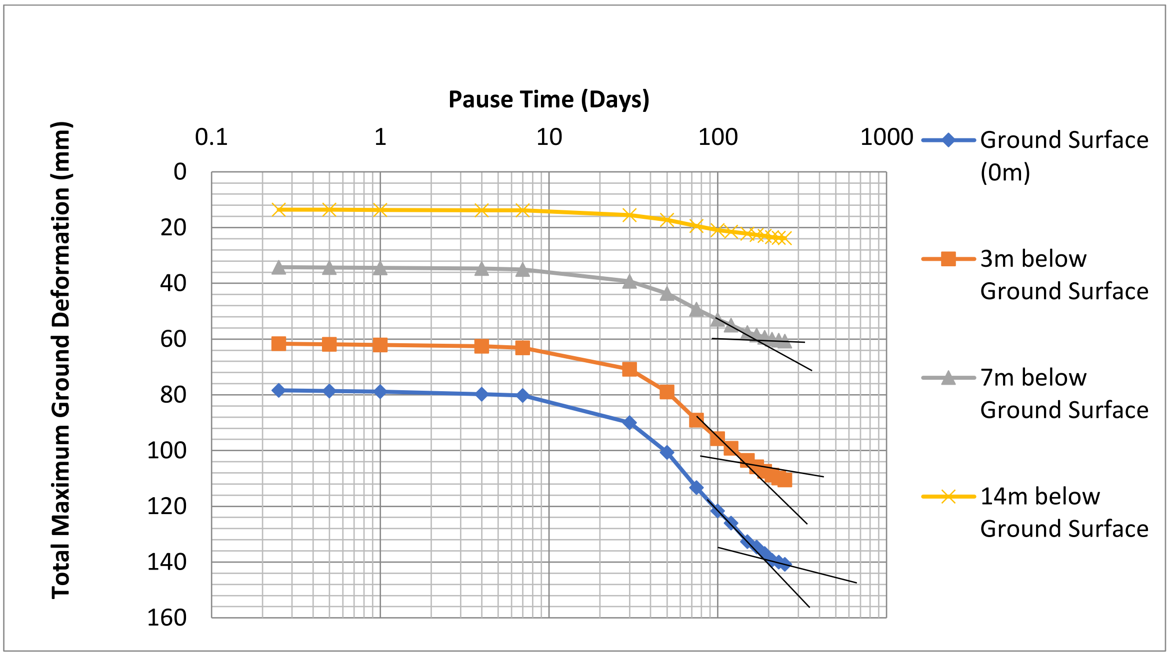


Figure 4.11 a - Variation of ground deformation with pause time at different depths below the ground surface behind the wall (0 m from the wall).

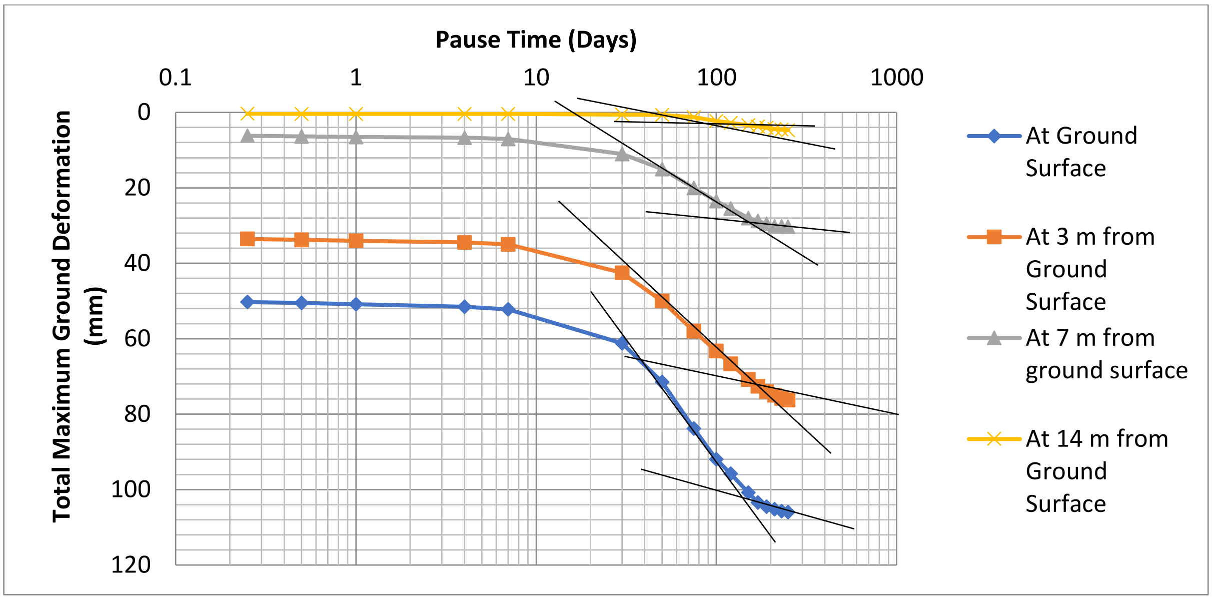


Figure 4.11 b - Variation of ground deformation with pause time at different depths at 4 m away from the wall.

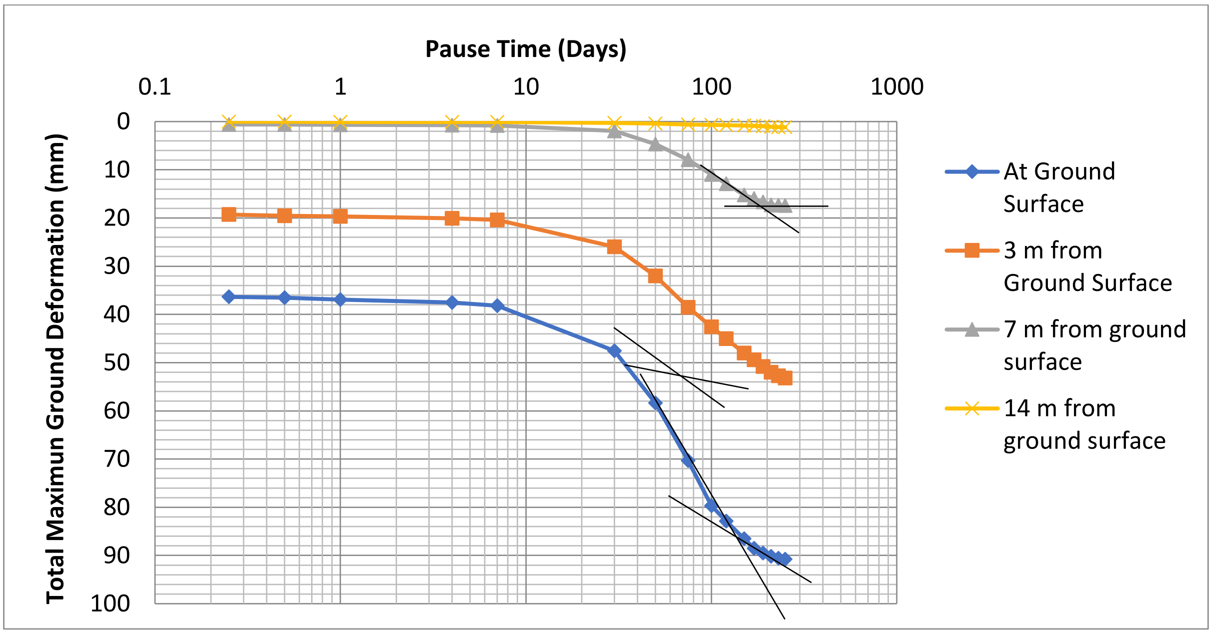


Figure 4.11 c - Variation of ground deformation with pause time at different depths at 7 m away from the wall.

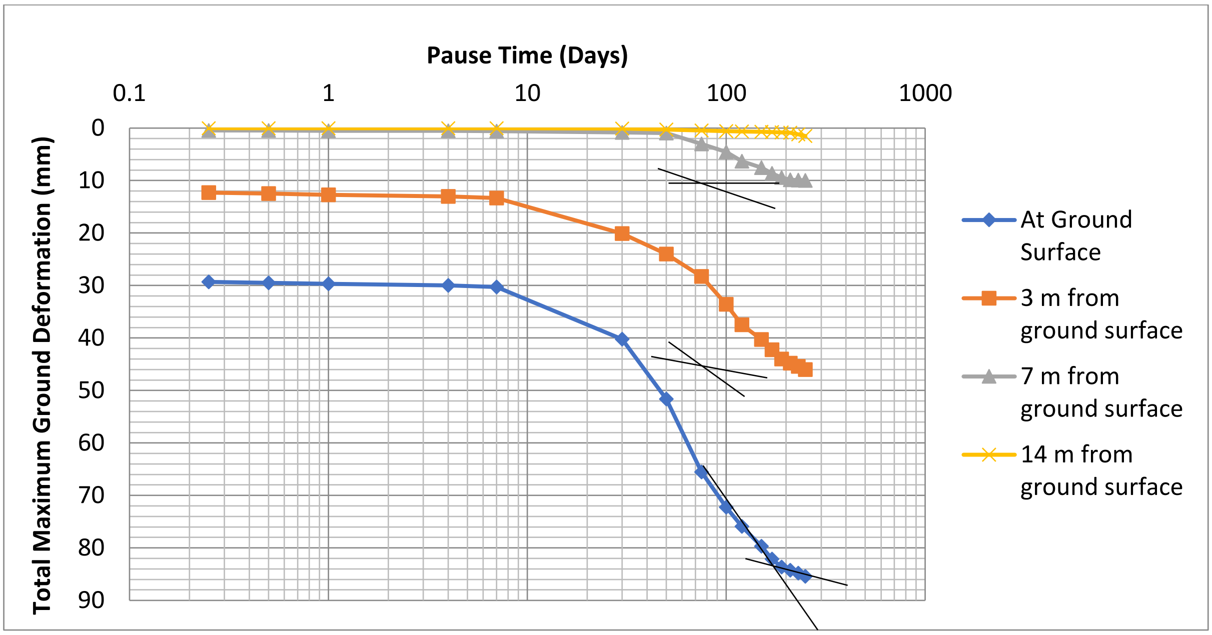


Figure 4.11 d - Variation of ground deformation with pause time at different depths at 10 m away from the wall.

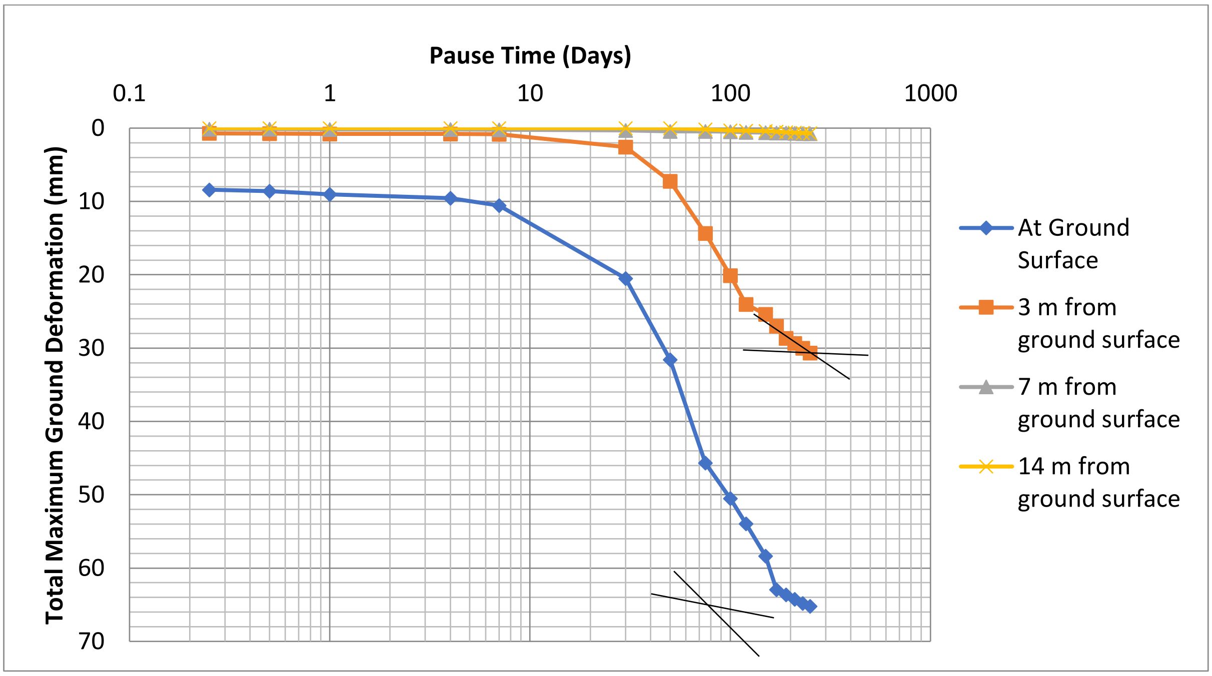


Figure 4.11 e. Variation of ground deformation with pause time at different depths at 15 m away from the wall.

The variation of undrained deformation, sum of undrained and consolidation, total maximum ground deformation (sum of undrained, consolidation and creep deformation components) with distance from the wall at the ground surface and 3m, 7m and 14m below the ground surface for 4 typical cases i.e., case number 14, 10, 6 and 3 are presented in figures 4.12-4.15 respectively. From these figures it is observed that at the near the wall the major part of ground settlement is due to undrained deformation and consolidation deformation while negligible deformation is contributed by creep. It is further observed that at the ground surface and at 3 m depth, beyond 10 m horizontal distance from the wall, contribution of undrained deformation is comparatively lesser with respect to that of consolidation. Similarly, at depth 7m or below, beyond 5m away from the wall there is practically no contribution of undrained deformation. This is because due to pressure release reduction in horizontal stress is maximum giving maximum deviator stresses just adjacent to the wall and it reduces with distance from the wall. However, with time drainage condition of the soil behind the wall changes from undrained to drained which may cause some redistribution of stresses in the soil due to the reduction in modulus and poisson's ratio leading to increase in deformation due to consolidation. For all cases time dependent deformation is found to be predominantly due to consolidation with minimum contribution of the creep.

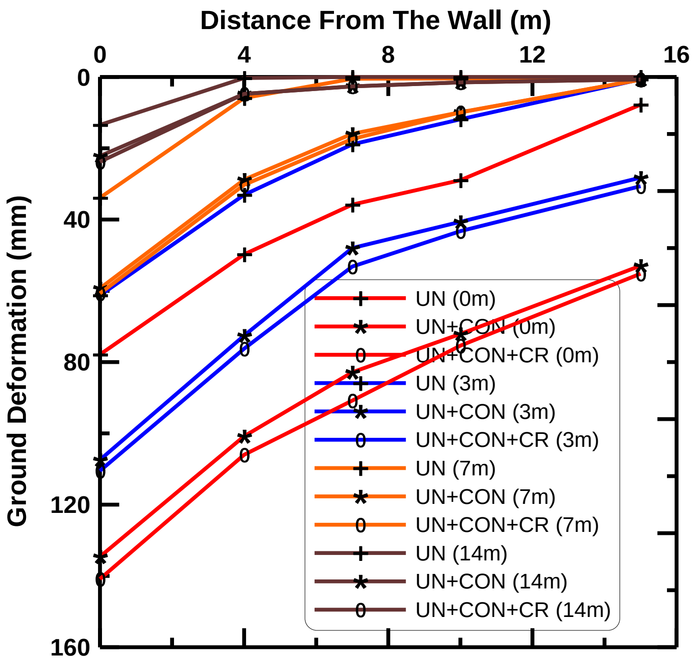


Figure 4.12 - Variation of various components of ground deformation with distance behind the wall at the ground surface and 3, 7 and 14m below ground surface for 14 m depth of excavation (case 14).

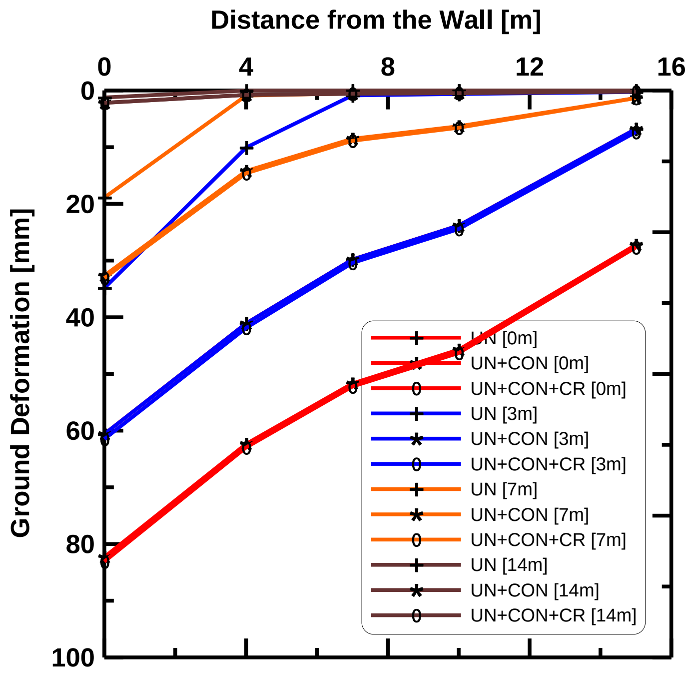


Figure 4.13 - Variation of various components of ground deformation with distance behind the wall at the ground surface and 3, 7 and 14m below ground surface for 10 m depth of excavation (case 10).

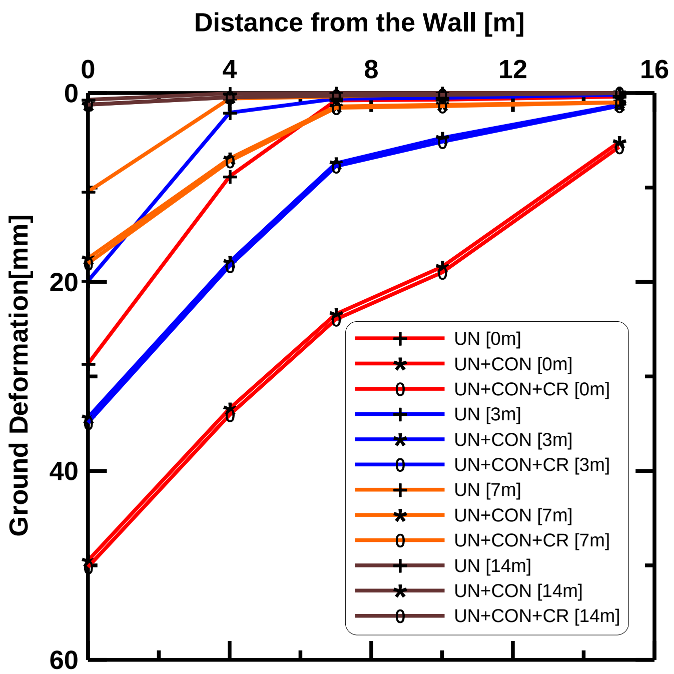


Figure 4.14 - Variation of various components of ground deformation with distance behind the wall at the ground surface and 3, 7 and 14m below ground surface for 5 m depth of excavation (case 6).

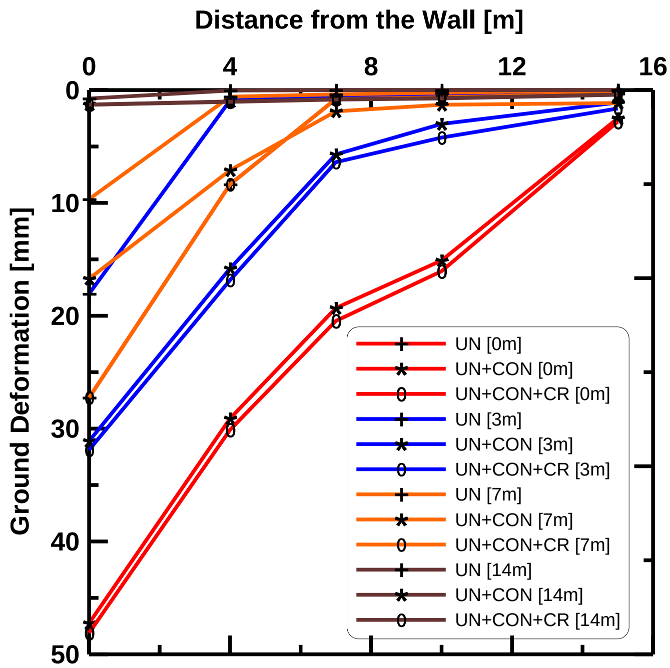


Figure 4.15 - Variation of various components of ground deformation with distance behind the wall at the ground surface and 3, 7 and 14m below ground surface for 4 m depth of excavation (case 3).

4.3.4 Variation of Rate of Settlement with Time at Various Distance from the wall and at Various Depth

In this section rate of settlement has been estimated at various depth (0m, 3 m, 7m and 14 m) below the ground surface and at various distance away from the wall (0 m, 4 m and 7 m) for cases all the cases for calculating the settlement rate different pause times have been taken into account (0 day, 7 days, 30 days, 50 days, 75 days, 100 days, 120 days, 150 days, 170 days, 190 days, 210 days, 230 days and 250 days.) Settlement Rate (δ_{Rate}) is calculated as the ratio of difference of settlement (mm) and difference of pause timings (days). For example, using the test result of pause time 0 day and 7 days, rate of settlement, δ_{Rate}, at pause time ((7+0)/(2) = 3.5 days is estimated as [{($\delta_{max(7)}$) – ($\delta_{max(0)}$)}/{7 days – 0 day)}] where $\delta_{max(7)}$ = ground deformation at pause time 7 days and $\delta_{max(0)}$ = ground deformation corresponding to 0 day or no pause.

The variation of rate of settlement with pause timing for all the cases are plotted with pause time in figures 4.16 – 4.29 respectively. From these figures it is observed that rate of settlement initially is less and then it increases with time reaching maximum at 40 – 60 days of pause, beyond which it again reduces and becomes practically zero at about 250 days. This is because initially the soil is under undrained to partially drained condition and then it switches over to fully drained condition at 40- 60 days, thereafter, when the dissipation of major portion of generated pore water pressure is over, the rate of settlement has become minimum.

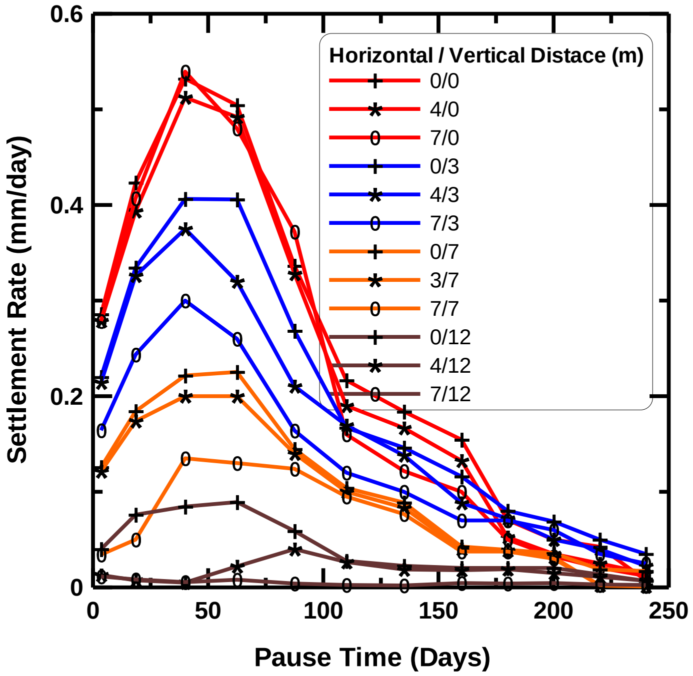


Figure 4.16 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 14 m below ground surface for excavation depth 14 m (Case 14).

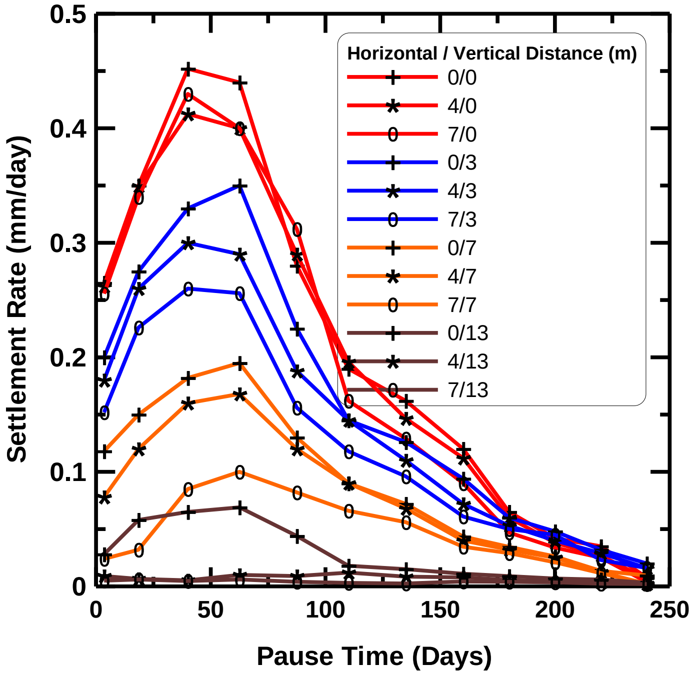


Figure 4.17 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 13 m below ground surface for excavation depth 13 m (Case 13).

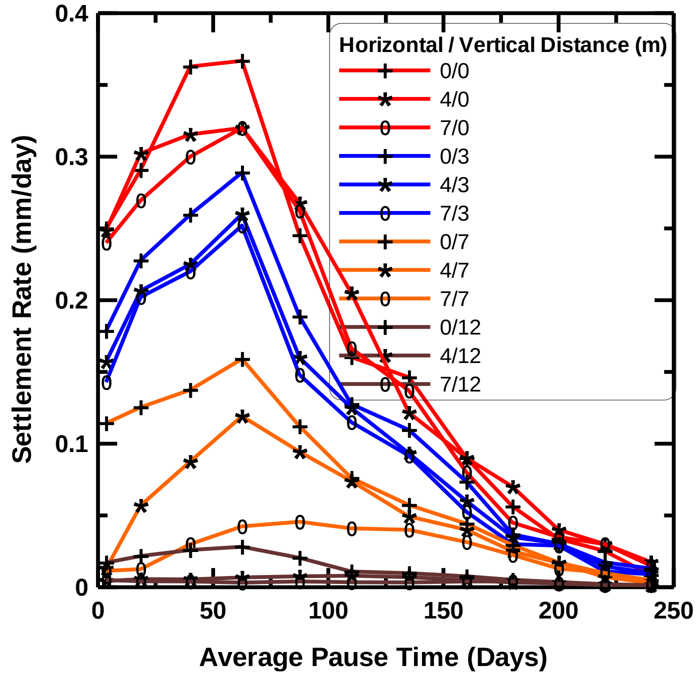


Figure 4.18 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 12 m (Case 12)

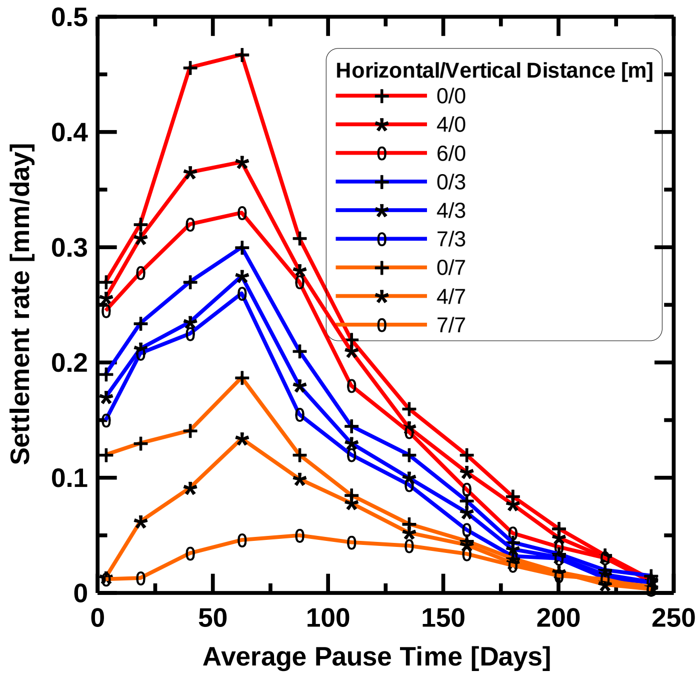


Figure 4.19 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 12 m (Case 11).

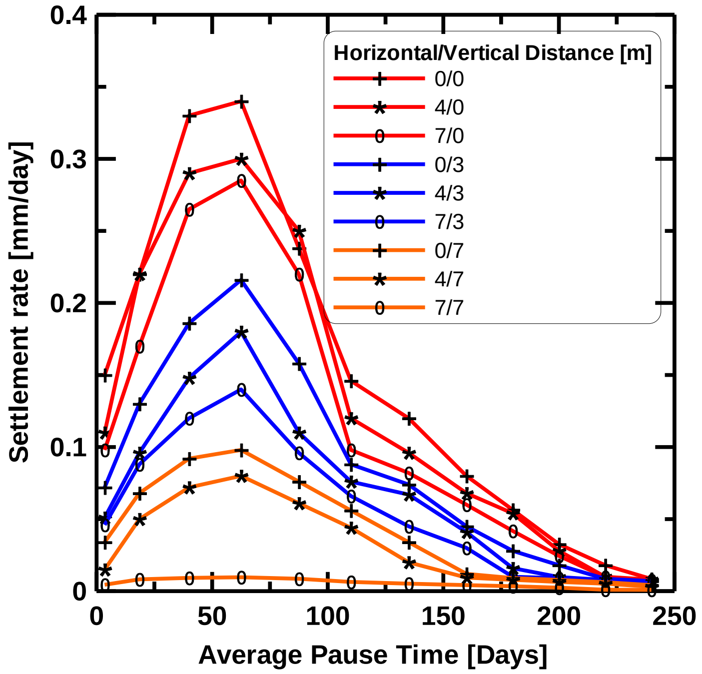


Figure 4.20 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 10 m (Case 10).

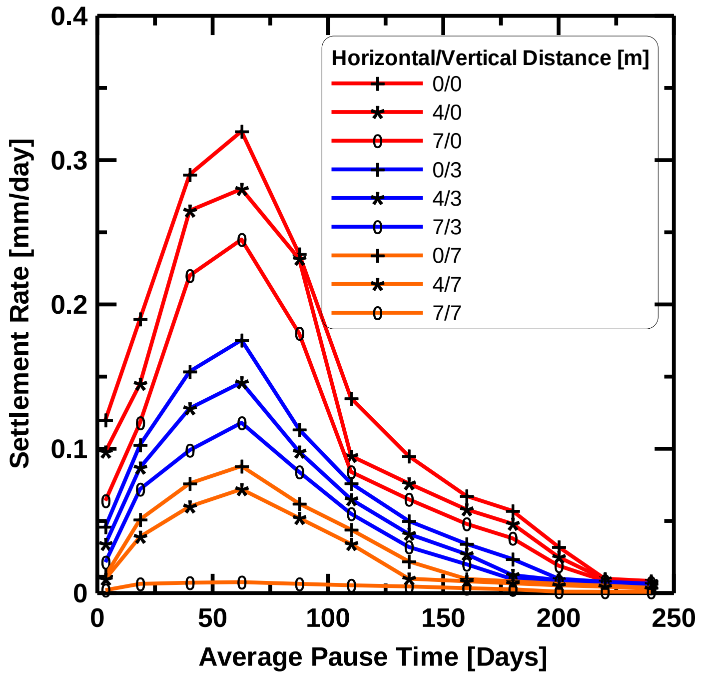


Figure 4.21 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 8 m (Case 9).

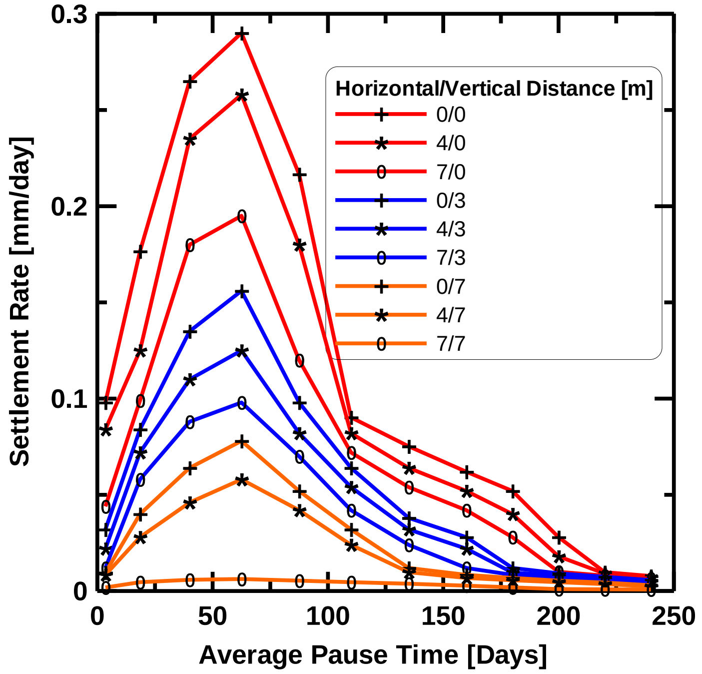


Figure 4.22 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 7 m (Case 8).

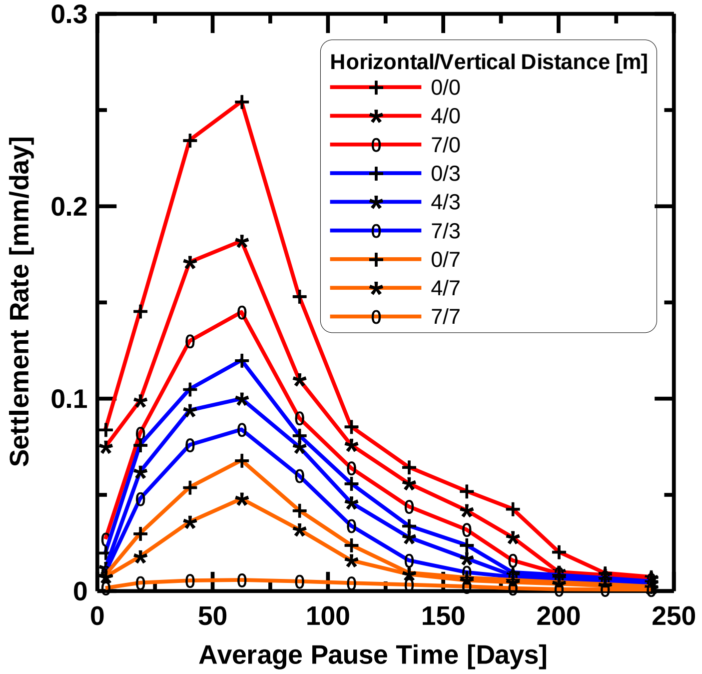


Figure 4.23 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 6 m (Case 7).

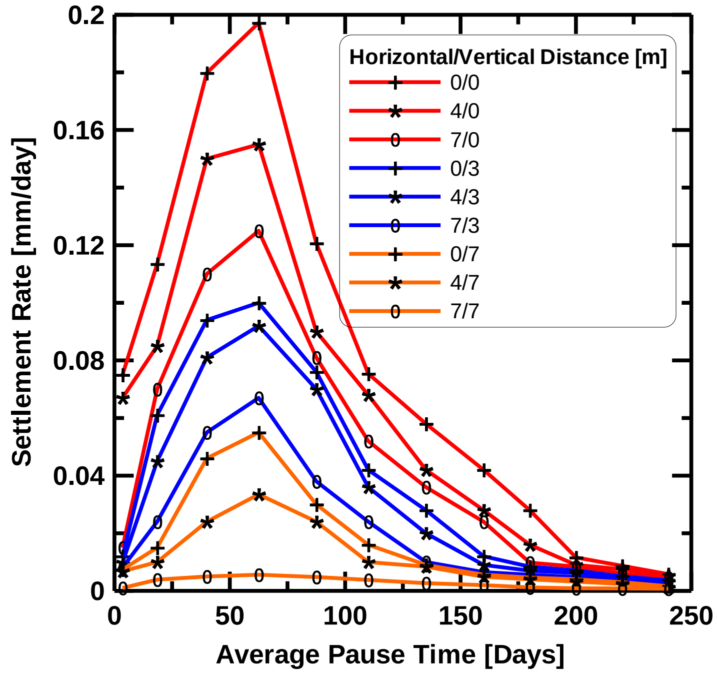


Figure 4.24 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 5 m (Case 6).

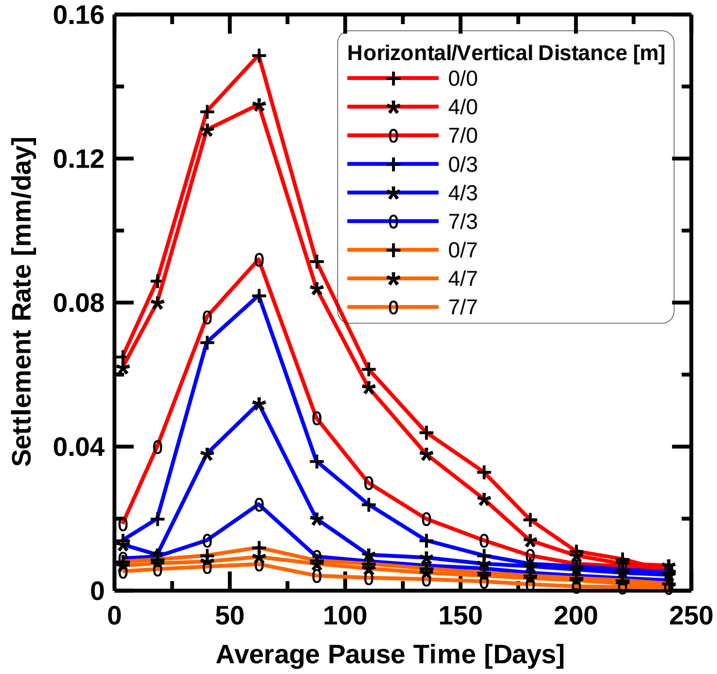


Figure 4.25 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 4 m (Case 5).

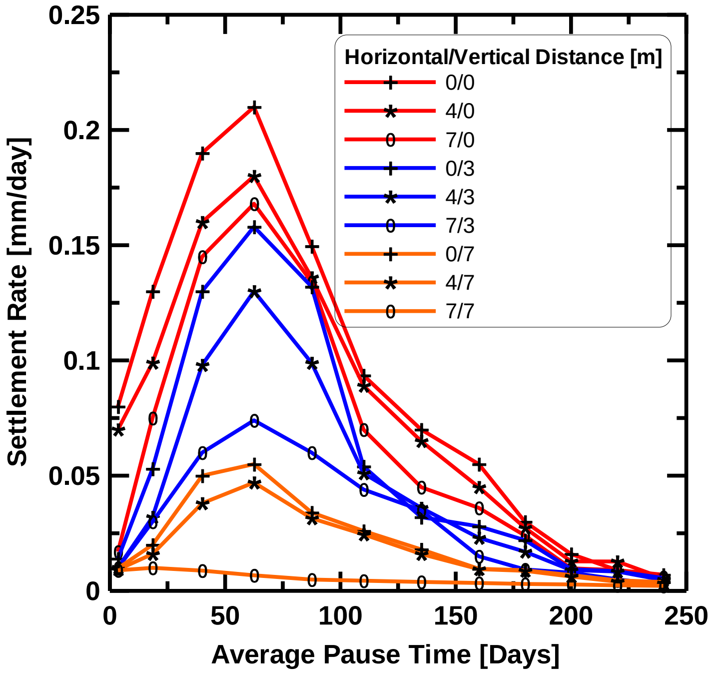


Figure 4.26 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 5 m (Case 4).

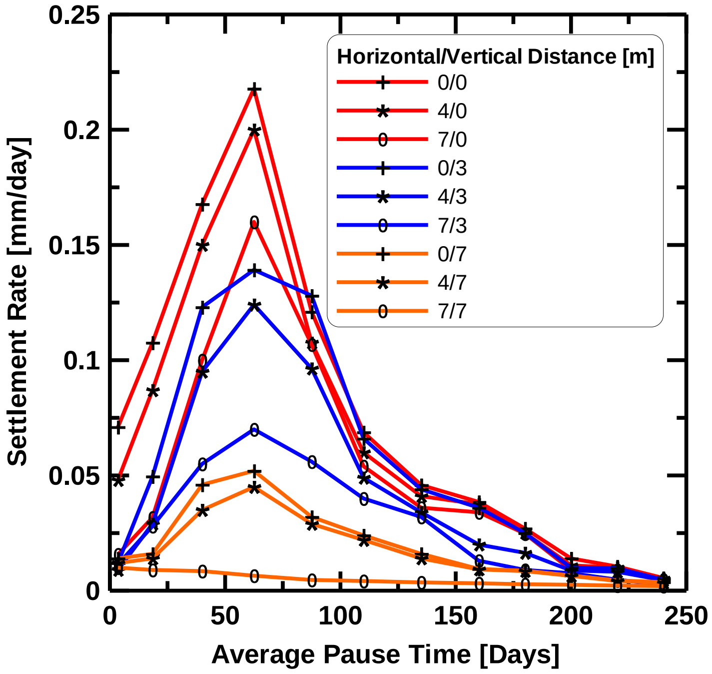


Figure 4.27 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 4 m (Case 3).

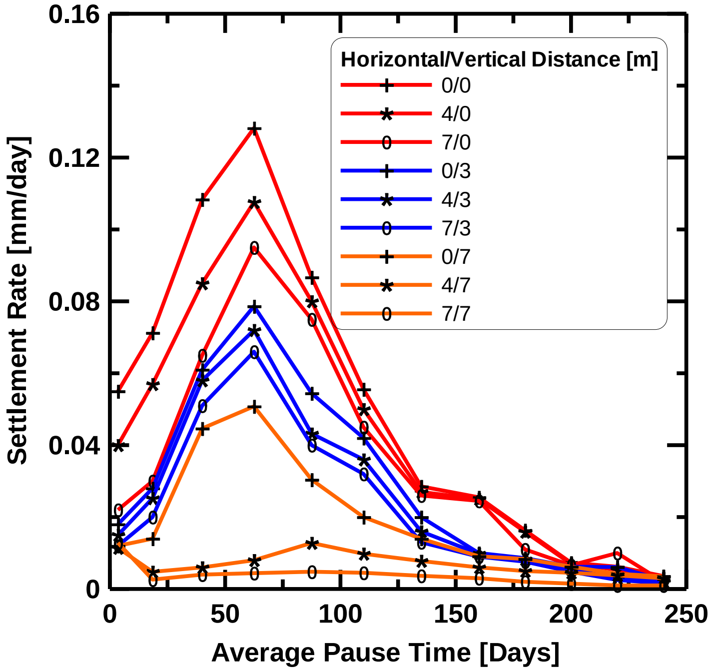


Figure 4.28 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 3 m (Case 2).

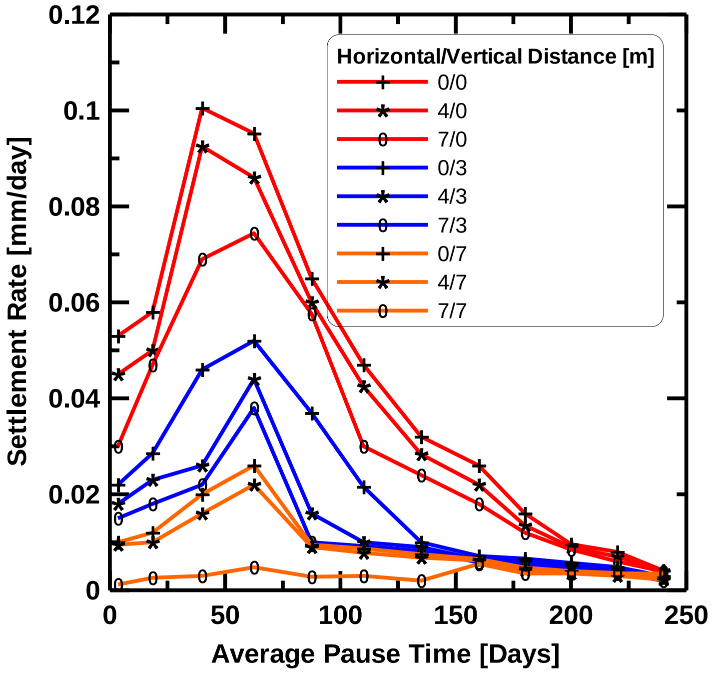
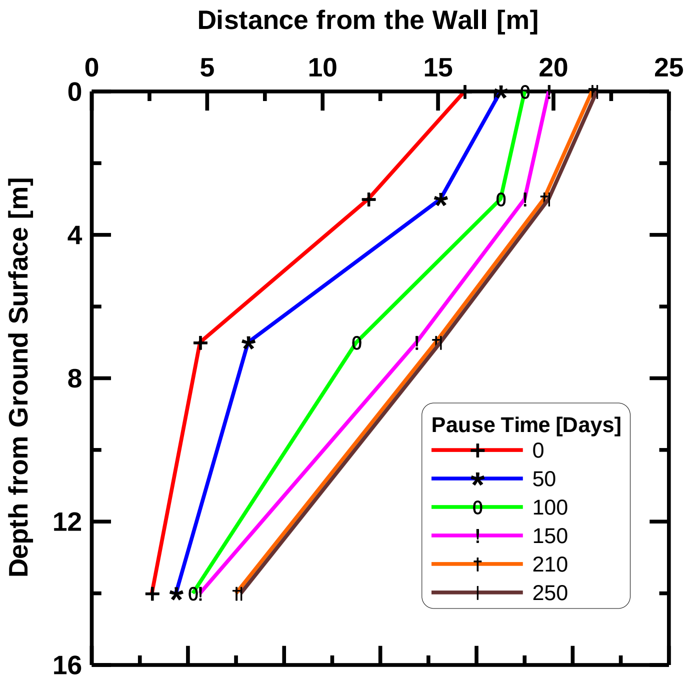


Figure 4.29 - Variation of settlement rate with pause time for 0 m, 4 m and 7m away from the wall at Ground Surface and 3 m, 7 m and 12 m below ground surface for excavation depth 2 m (Case 1).

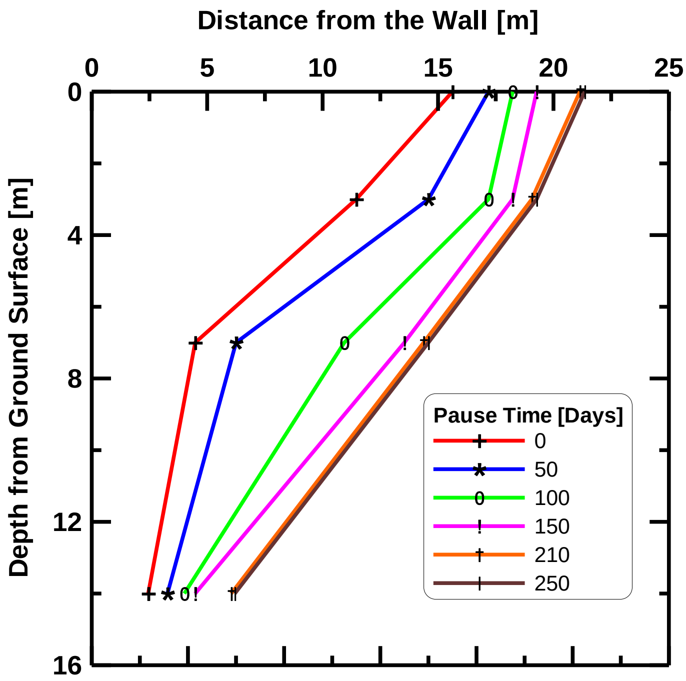
4.3.5 Change of Influence Zone (By taking 10% of maximum Settlement) with Time

In this section analysis has been carried out to get an idea of influence zone around excavation where significant soil deformation occurred. The increase in zone of influence with pause time has been plotted for all the cases (figures 4.30 a - n) behind the excavation by locating the points in the soil bed where 10 % of maximum ground deformation takes place. It has been plotted for different pause times (0 day, 50 days, 100 days, 150 days, 210 days and 250 days) and for different excavation depth (12 m, 13 m and 14 m). [for example, for depth 0 m and for pause time 0 day the maximum ground deformation happens at the edge of the wall of 78.25 mm so at horizontal distance 15.5 m from wall the ground deformation reduces to 10% of 78.25 mm i.e., 7.825 mm]

From the figures 4.30 (a)-(n) it may be seen that the zone of influence is initially up to a distance of about 10 m-15.5 m behind the wall at the ground surface to about 3.5 m-5 m at a depth of 6m below ground surface for comparatively deeper depth of excavation (cases 9 -14) and for comparatively shallow excavation depth (cases 1-8) the zone of influence is initially up to a distance of about 7 m-5.5 m behind the wall at the ground surface to about 1.5 m-3 m at a depth of 6 m below ground surface. Further, for all the cases it has been observed that after about 50 days the width of the influence zone started increasing at a faster rate and finally at about 200 days it becomes more or less reached a stable state. This is because initially the soil is under undrained condition, thereby the magnitude of earth pressure mobilized on the wall was comparatively less. At a later stage with the increase in pause time, the soil bed changes from undrained to partially / fully drained condition which causes an increase in earth pressure on the wall leading to higher deformation. Thus, the size of the zone of influence increases and becomes stable at about 200 days. The enhancement of influence zone due to pause/delay is more significant near the ground surface, while near excavation this increase of influence zone is small.


Distance from the Wall [m]
0
5
10
15
20
25
Depth from Ground Surface [m]
0
4
8
12
16
Pause Time [Days]
0
50
100
150
210
250

(a)


Distance from the Wall [m]
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15
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Depth from Ground Surface [m]
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12
16
Pause Time [Days]
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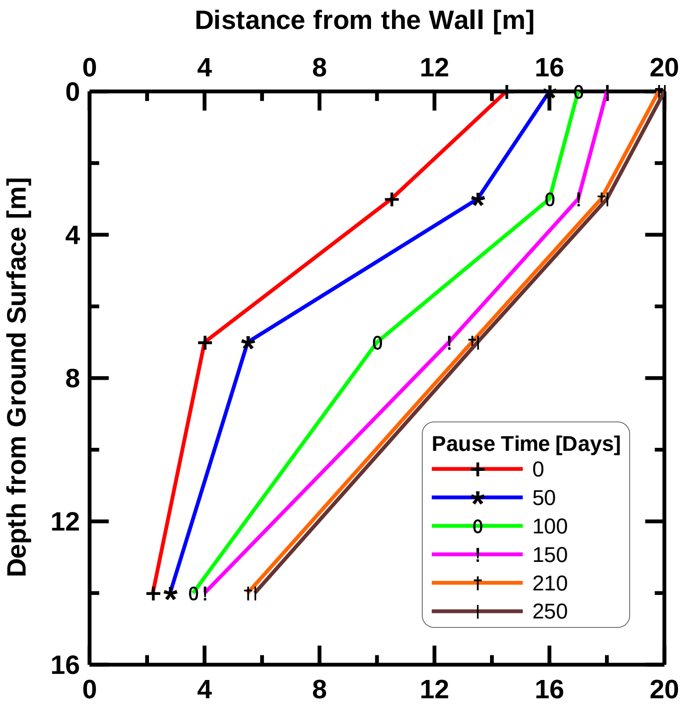
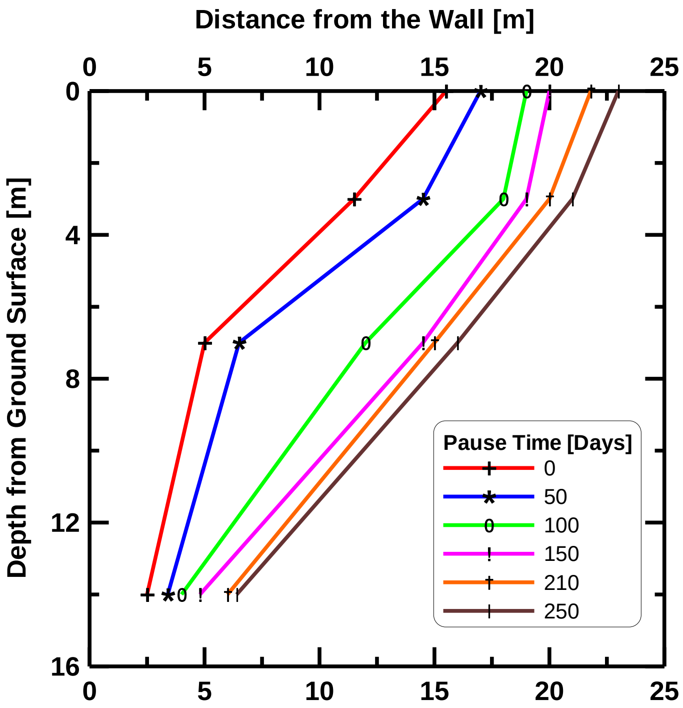
50
100
150
210
250

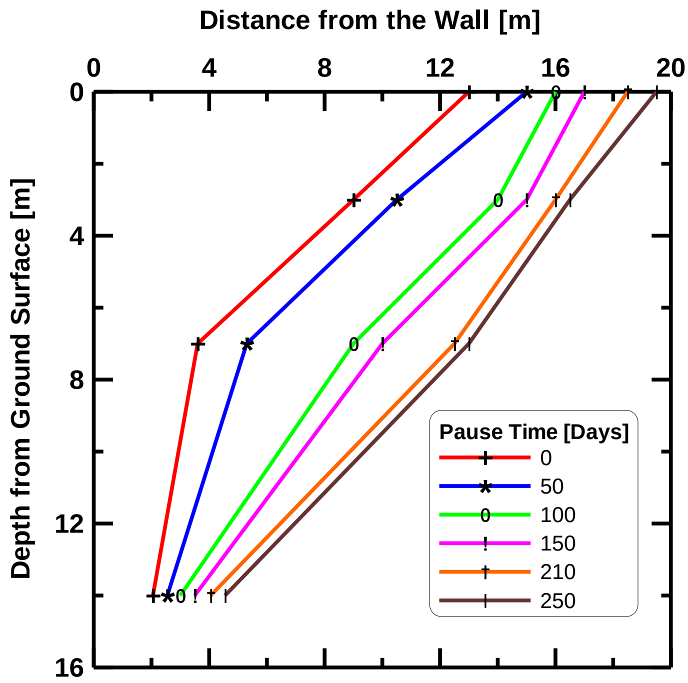
(b)


Distance from the Wall [m]
Depth from Ground Surface [m]
Pause Time [Days]
0
50
100
150
210
250

(c)


Distance from the Wall [m]
Depth from Ground Surface [m]
Pause Time [Days]
0
50
100
150
210
250

(d)



Distance from the Wall [m]
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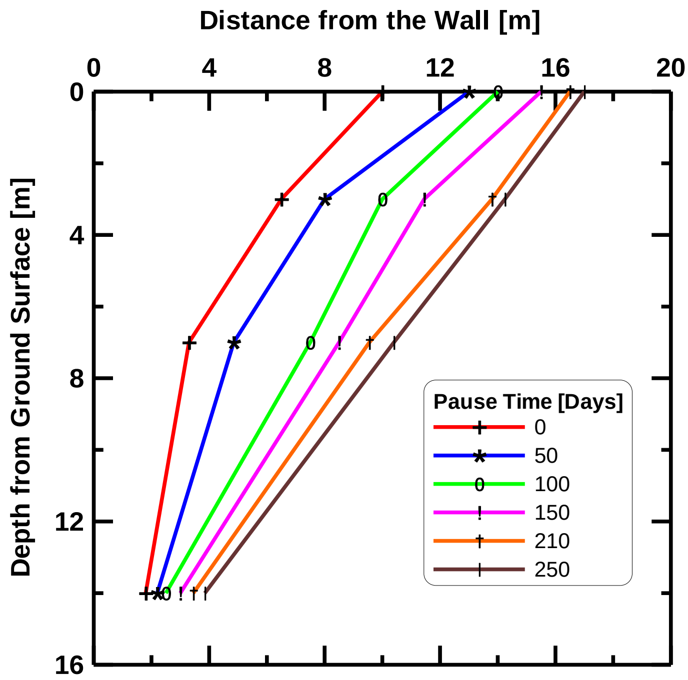
8
12
16
20
Depth from Ground Surface [m]
0
4
8
12
16
Pause Time [Days]
0
50
100
150
210
250

(e)



Distance from the Wall [m]
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4
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12
16
20
Depth from Ground Surface [m]
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4
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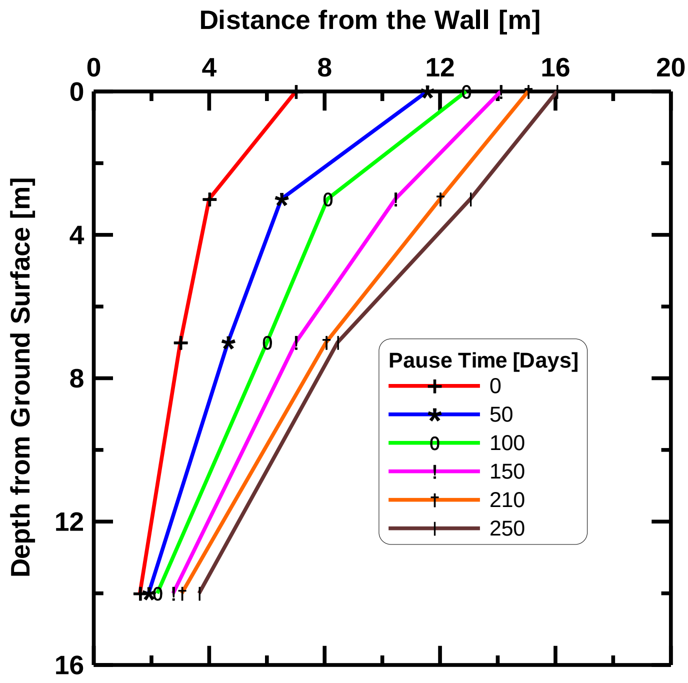
12
16
Pause Time [Days]
0
50
100
150
210
250

(f)


Distance from the Wall [m]
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Depth from Ground Surface [m]
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Pause Time [Days]
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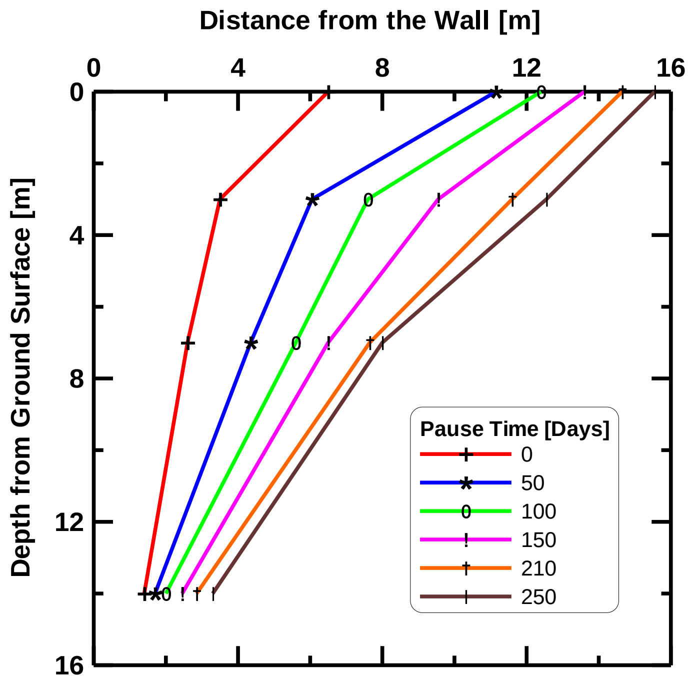
(g)


Distance from the Wall [m]
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12
16
Depth from Ground Surface [m]
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12
16
Pause Time [Days]
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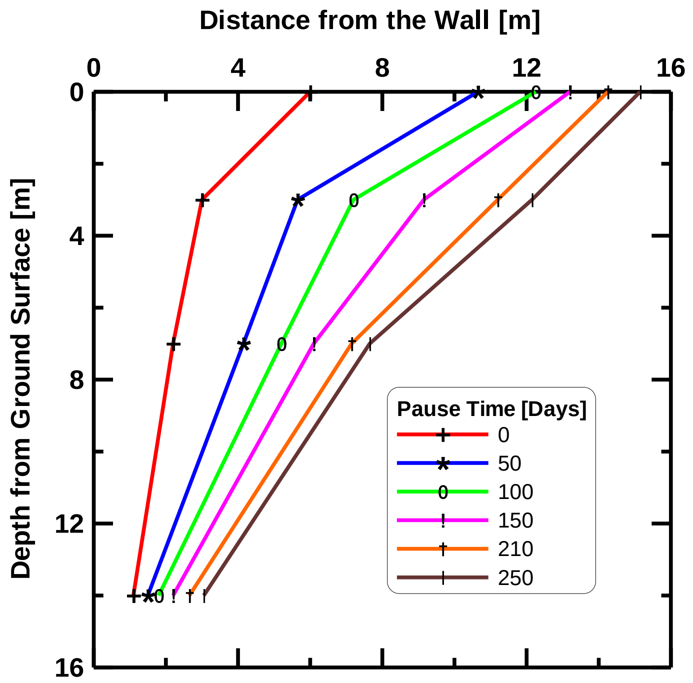
100
150
210
250

(h)


Distance from the Wall [m]
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4
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12
16
Depth from Ground Surface [m]
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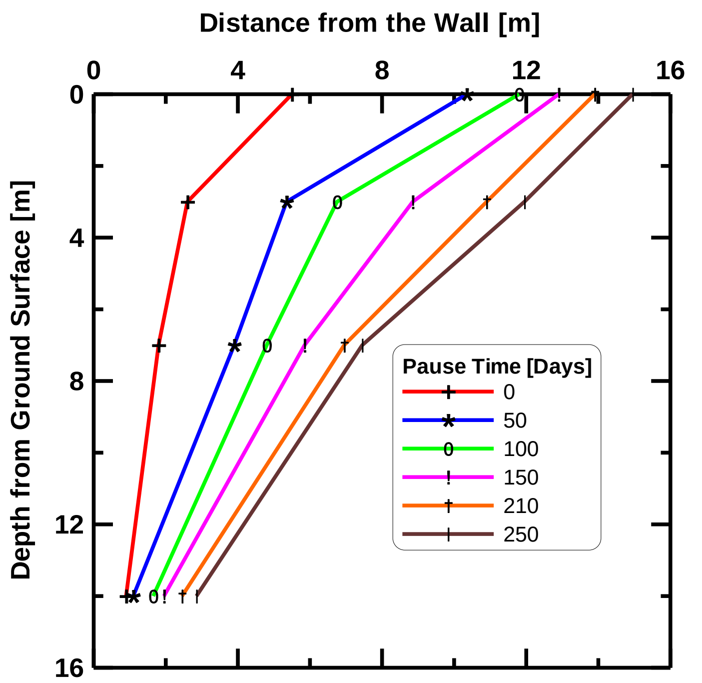
4
8
12
16
Pause Time [Days]
0
50
100
150
210
250

(i)


Distance from the Wall [m]
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4
8
12
16
Depth from Ground Surface [m]
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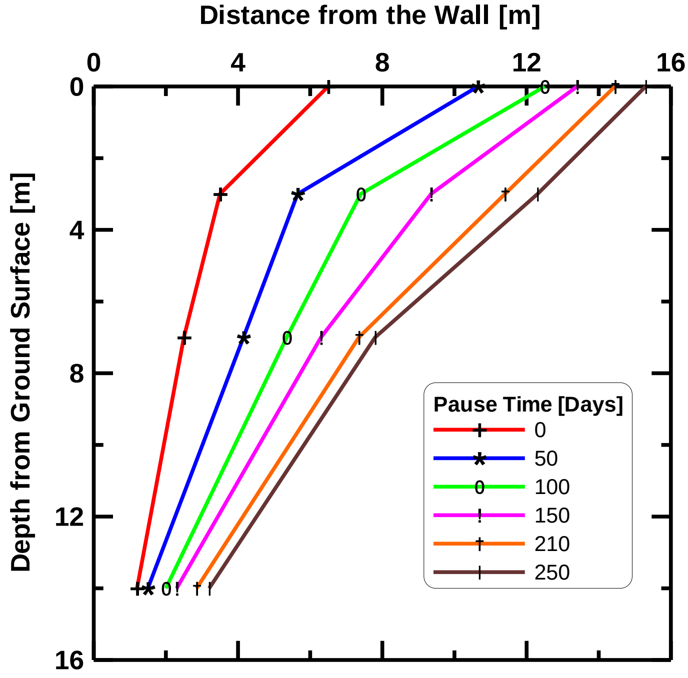
4
8
12
16
Pause Time [Days]
0
50
100
150
210
250

(j)


Distance from the Wall [m]
0
4
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12
16
Depth from Ground Surface [m]
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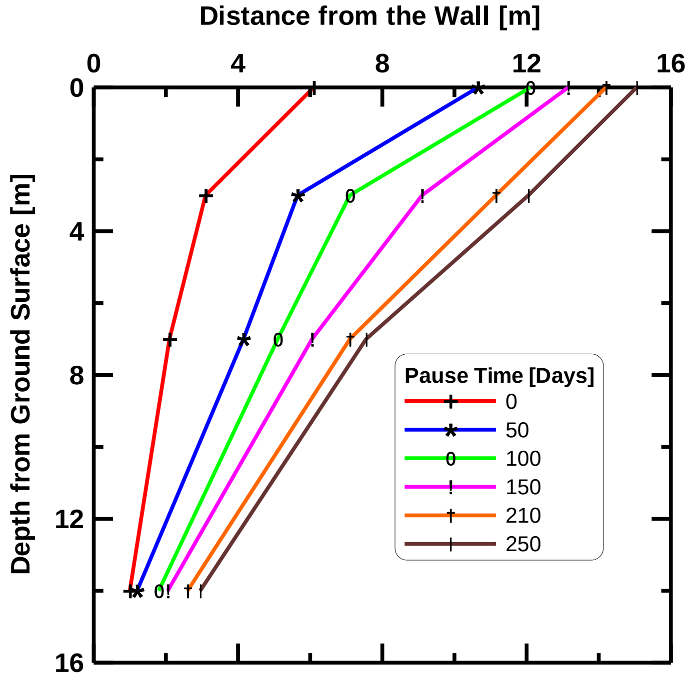
4
8
12
16
Pause Time [Days]
0
50
100
150
210
250

(k)


Distance from the Wall [m]
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8
12
16
Depth from Ground Surface [m]
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4
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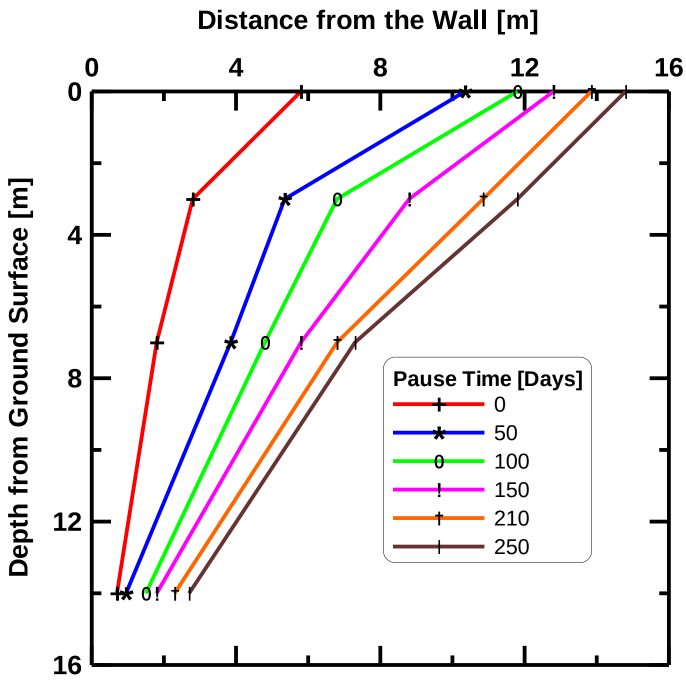
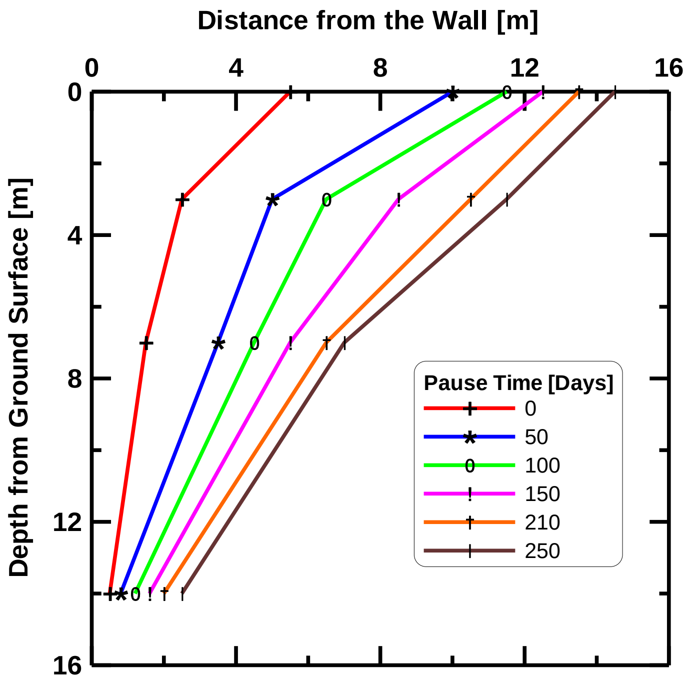
12
16
Pause Time [Days]
0
50
100
150
210
250

(l)



(m)



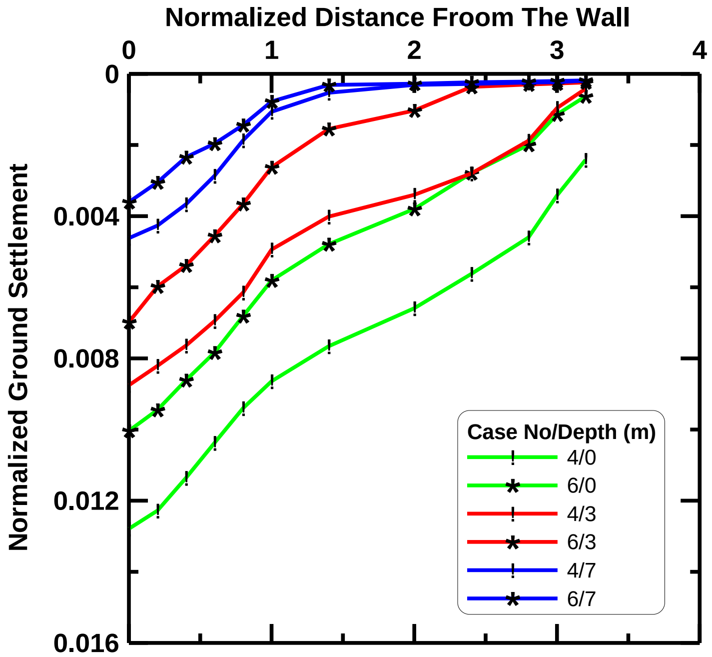
(n)

Figure 4.30 - Variation of influence zone from the wall with depth below ground surface for different pause times (0 day, 50 days, 100 days, 150 days, 210 days and 250 days) for (a) excavation depth 14 m-3 struts (case 14), (b) excavation depth 13 m-3 struts (case 13), (c) excavation depth 12 m-3 struts (case 12), (d) excavation depth 12 m-2 struts (case 11), (e) excavation depth 10 m-2 struts (case 10), (f) excavation depth 8 m-2 struts (case 9), (g) excavation depth 7 m-1 strut (case 8), (h) excavation depth 6 m-1 strut (case 7), (i) excavation

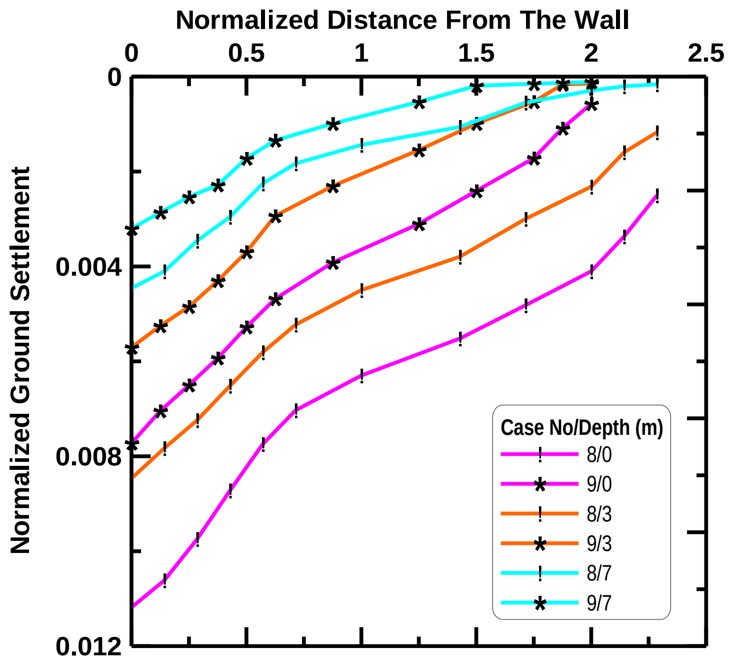
depth 5 m-1 strut (case 6), (j) excavation depth 4 m-1 strut (case 5), (k) excavation depth 5 m-0 strut (case 4), (l) excavation depth 4 m-0 strut (case 3), (m) excavation depth 3 m-0 strut (case 2) and (n) excavation depth 2 m-0 strut (case 1) respectively.

4.3.6 Effect of number of struts on settlement profile

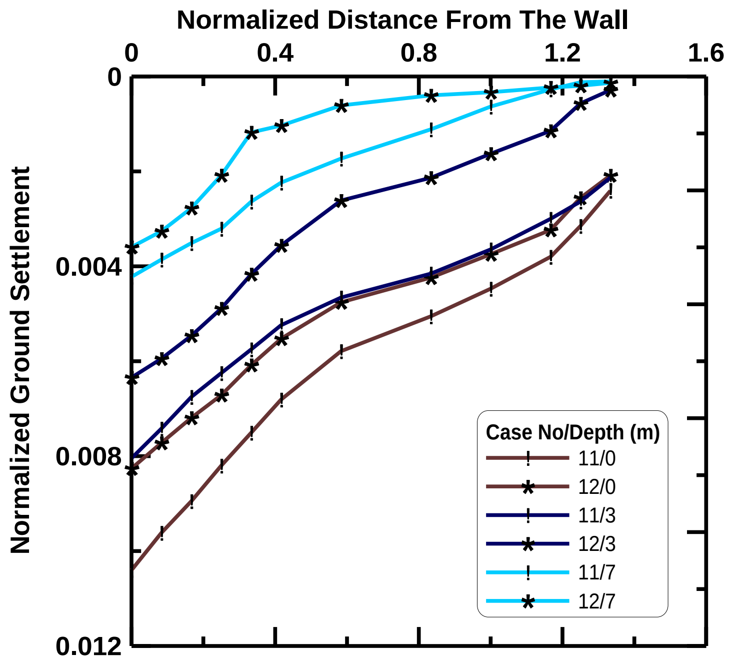
To assess the effect of number of struts, normalized elastic settlement has been plotted in figures 4.31 (a-c) against normalized distance from the wall for different strut conditions and for different depths without considering pause time. Ground deformation for different depths and different distance from the wall are normalized with excavation depths respectively. To show the comparison in normalized ground settlement three different combinations have been considered with varying strut condition keeping other excavation parameters same. In each combination, two cases have been plotted. The combinations were case 4(5m - no strut) and case 6(5m - 1 strut); case 8(7m -1 strut) and case 9(8m – 2 strut) and case 11(12m – 2 strut) and case 12(12m – 3 strut) respectively. It has been observed from figures 4.31 (a-c) that as the number of struts increases the normalized ground settlement decreases and this decrease can be as high as 28% when one strut is included for same excavation depth of 5 m i.e., for cases 4 and 6, 22% when one additional strut is included for almost same excavation depth i.e., cases 8 and 9 and 17.5% when one additional strut is included for same excavation depth of 12 m i.e., cases 11 and 12.



(a)



(b)



(c)

Figure 4.31 – Normalized settlement profile at ground surface and 3m and 7m depth below ground surface for different strut combinations (a)case 4(E_D=5m-no strut) - case 6(E_D=5m-1 strut), (b) case 8(E_D=7m-1 strut) – case 9(E_D=8m-2 strut) (c) case 11(E_D=12m-2 strut) – case 12(E_D=12m-3 strut). [E_D=Excavation Depth]

4.3.7 Validation of present study with available case studies

The appropriateness of the present experimental study is validated by comparing the experimental study results of obtained additional ground deformation due to pause in construction with other case study results reported by Som (1994), Ou (1998) and Liu (2005) and are documented in figure 4.32. From the figure it may be seen that the model test results agree reasonably well with the field test results up to about 100 days, beyond which the field data are lesser than the model data which may be due to lesser cohesion of the clay bed in the centrifuge.

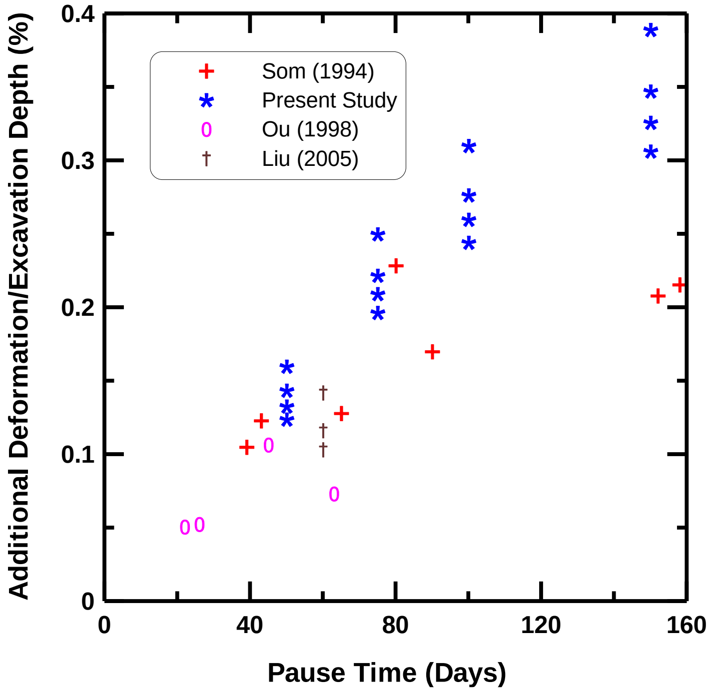


Figure 4.32 - Comparison of centrifuge data with those obtained by Som (1994), Ou (1998) and Liu (2005)

4.4 CONCLUSION FROM CHAPTER 4.0

In the present study, a systematic investigation of the ground deformation behind a braced excavation in soft clayey soil similar to that available in Kolkata is performed using physical model study in geotechnical centrifuge considering different depth of excavation, number of struts construction delay/construction stoppage. The mechanism of this deformation was also assessed by evaluating the contribution of undrained, consolidation and creep deformation to the total ground deformation. The tests results were also used to predict the variation in elastic as well as time dependent settlement with horizontal and vertical distance away from the wall for different excavation depths, number of struts and test results are also used to access the effect of construction delay on various important factors like rate of settlement, change of zone of influence behind a braced wall and additional ground deformation etc. Further, the experimental results are also validated with the observed values obtained from reported case studies (Som, 2000, Ou, 1998 and Liu, 2005). From the study the following points can be concluded.

(i) Magnitude of vertical soil deformation decreases considerably with depth below the ground surface. In other words, ground deformation values become less with increase in depth below ground level.

(ii) As pause time increases ground deformation increases and the rate of increment near ground surface is more prominent. The deformation values reduce with distance from the wall especially near excavation depth.

(iii) As pause time increases normalized ground deformation increases and the rate of increment near ground surface is more prominent. However, the magnitude is found to reduce with depth below the ground. Further it has been also clearly observed that as pause time increases the zone of ground profile deformation also shifts significantly and the zone of deformation increases rapidly.

(iv) Major part of the ground settlement is due to undrained deformation and consolidation deformation, while magnitude of creep deformation is not significant. Contribution of undrained deformation to the total deformation increases at near the wall.

(v) Rate of settlement initially is less and then it increases with time.

(vi) As pause time increases the zone of deformed soil also increases behind the wall for ground surface as well as at different depths and becomes more or less stable after about 200 days.

(vii) Normalized ground settlement decreases 20% - 30% with the installation of one additional strut for same depth of excavation.

CHAPTER 5 - SUMMARY, CONCLUSION, CONTRIBUTION AND FUTURE SCOPE OF THE PRESENT STUDY.

5.1 SUMMARY OF THE PRESENT STUDY

Deep excavations are widely used in urban areas for the development of underground space like subway stations, basements for high-rise buildings, underground car parks and shopping centers. However, the excavation process inevitably alters the stress states in the ground and may cause significant wall deformation and ground movements. Performance of these excavations, in general, depends upon nature of the subsoil deposit, depth of excavation and type of support system. Sometimes the presence of adjacent existing infrastructure like buildings, buried pipe lines, foundations etc., may also control the ground/wall movements and as a whole, the performance of the system. Excavation induced settlement becomes higher if proper sequence as well as technique/material is not adopted during construction.

Though deformation behaviours of braced excavations were studied depending upon variations of excavation parameters in the literatures, limited works have been done on systematic parametric analysis of braced excavation presenting design guidelines to achieve optimized deformation values. It is necessary to conduct parametric analysis to assess the response of soil and excavation parameters in order to control soil movements especially in congested urban areas. Additionally, it's worth noting that the existing literature lacks a thorough examination of how various design parameters, including excavation depth, excavation width, diaphragm wall thickness, wall embedment depth, soil shear strength, and strut spacing, contribute to the estimation of ground and wall deformation values. Consequently, the accurate prediction of both the scale and pattern of ground movement is essential. Developing a simplified design approach could facilitate swift and accurate forecasts of soil displacement.

As an alternative approach to replicate excavation behaviour, researchers have employed small-scale centrifuge modelling. In this method, a centrifuge is utilized to create an artificial acceleration field, effectively mimicking gravitational stress and enabling accurate scaling of the model. This permits the emulation of excavation behaviour on a reduced scale, offering valuable insights into the mechanisms of soil deformation during the excavation process. The chief benefit of this technique lies in its capacity to conduct repetitive tests and to continue experimentation until failure occurs. Such extensive testing is often impractical in real-world field conditions or with finite element programs. Given these advantages, the utilization of physical modelling within a centrifuge has garnered global recognition and has been adopted as the principal methodology for this particular study.

The current research encompasses a comprehensive investigation involving both numerical and experimental methods. In the numerical analysis section, a parametric study utilizing finite element analysis was undertaken to evaluate the impact of diverse factors, including time-dependent variables like excavation rate and pause duration, on the deformation behavior of braced excavations situated in soft clay deposits. The accurate estimation of soil parameters was also emphasized as crucial for effective braced excavation design. For the analysis of typical braced excavations in soft clay, the study employed the PLAXIS 2D software, incorporating a soft soil creep constitutive model. Based on the insights from the numerical investigation, a practical design guideline was formulated. Additionally, the research entailed the development of multi-variate regression models that encompassed essential excavation parameters, including time-dependent variables, to facilitate precise predictions of maximum wall and ground displacement. These models also offered insight into the deformation profile of wall and ground surfaces. To formulate these regression equations, an extensive dataset was employed, consisting of both real-world case histories and artificially generated data from finite element analyses. The proposed model's validity was established by comparing its outcomes with results from sources not utilized during the model's development.

Within the experimental study segment, a methodical examination of layer-by-layer ground deformation occurring behind a braced excavation in soft clayey soil, akin to the conditions found in Kolkata, was undertaken through physical model experimentation. Various excavation depths and numbers of struts were considered, as well as scenarios involving construction delays or stoppages subsequent to reaching the final cut level. The fundamental mechanism driving this deformation was examined by analyzing the contributions of undrained, consolidation, and creep deformations to the overall ground deformation. The results from these tests were employed to forecast the implications of construction delays on critical factors such as settlement rate and alterations in the zone of influence behind a braced wall. Further, the experimental results are also validated with the observed values obtained from reported case studies (Som, 2000; Ou, 1998 and Liu, 2005).

5.2 CONCLUSION FROM THE PRESENT STUDY

In the current investigation, a two-fold approach was followed. Initially, a parametric study was undertaken to explore the impact of diverse design parameters on the deformation characteristics. This study aimed to derive insights into the behavior of various parameters such as wall embedment depth, wall thickness, and strut arrangements. Subsequently, utilizing the findings from this parametric analysis, design guidelines were formulated, delineating the optimal values for these parameters. Concurrently, an experimental study was executed to assess the ground deformation patterns exhibited by braced excavations in soft clay. This comprehensive analysis involved the examination of real-world behavior and contributed valuable empirical data. The conclusions section of the study was divided into two distinct subsections corresponding to the numerical and experimental investigations, specifically detailed in sections 5.2.1 and 5.2.2 respectively.

5.2.1 Conclusions from numerical study

(i) Wall embedment depth should be at least 0.7 to 1.0 times of depth of excavation to avoid excessive ground movement when excavation is done in soft clay.

(ii) Strut arrangement should be such that unsupported wall length is not large. For depth of excavation more than 10 m three level strut system is recommended with struts are located at 0.14-0.17 H_{exc}, 0.42-0.5 H_{exc} and 0.75-0.80 H_{exc} respectively to achieve minimum displacement values.

(iii) With greater wall thickness small deformation values are obtained. Wall thickness may be kept as 0.08 times of depth of excavation to get optimal result.

(iv) Values of soil strength and consolidation parameters should be chosen or estimated carefully as a little variation produces markedly different deformation.

(v) If slower excavation rate is combined with construction stoppage it causes excessive ground movement. This situation should be avoided to restrict damage on adjacent structures

(vi) When excavation depth is greater (20 m or more), displacement values change very rapidly with reduced excavation rate. Designer should be careful so that this situation will not arise.

(vii) Increases of soil movements are less significant near the wall and far behind the wall due to construction stoppage and/ or slower rate of excavation. Though variations of deformations are considerably high at some distance behind wall (generally 0.5 to 1.8 times depth of excavation).

(viii) When excavation rate is reduced up to 50 days/ m soil deformation increases steadily but if further delayed excavation rate is followed small additional displacement is observed.

(ix) Time effect has lesser impact on wall movement than ground surface movement. Ground settlement can increase by about 215% or even more while wall movement can increase close to 100% at certain locations. Though increases of maximum ground and maximum wall deformations for delayed construction are about 184% and 40 % respectively. So, the need of construction control in braced excavation is critically important especially in soft clayey soil.

(x) Due to only pause in construction maximum ground settlement increases around 35 to 40% while values of maximum wall displacement go up in the region of 12 to 20%.

Secondly simplified models are proposed to predict ground and wall movements effectively. Using data obtained from case histories and generated artificially from FE analysis a simplified model is presented performing multi variable regression method. The applicability of proposed model is verified comparing results with reported literatures. Normalized root mean square error (NRMSE) of predicted values are calculated which is within satisfying range (below 20%). Student's t-test is performed which establishes the fact that predicted and measured data series are of similar type. Thus, the aforementioned analysis leads to following conclusions:

(i) Proposed method can be used as alternative to tedious finite element analysis, where quick and correct prediction of soil deformation around braced excavation is possible.

(ii) Prediction of maximum soil surface settlement and maximum wall displacement during braced excavation in soft clay may be done using the proposed model with various design parameters like H_{exc}, B_{exc}, T_{wall}, D/B_{exc}, s_u/σ_v', h_{avg}, E_R, S_T as input variables. Thus, estimation of maximum deformation values can be possible if little variation of excavation and soil parameters occur.

(iii) Once the magnitude of maximum soil surface settlement and maximum wall displacement are obtained, deformation profiles may be predicted to estimate δ_{vl} and δ_{hd} using L/H_{exc} and d/H_{exc} as input variables.

5.2.2 Conclusions from experimental study

From the study the following points can be concluded.

(i) Magnitude of vertical soil deformation decreases considerably with depth below the ground surface. In other words, the effect due to ground deformation become less with increase in depth below ground level.

(ii) As pause time increases ground deformation increases and the rate of increment near ground surface is more prominent. The deformation values reduce with distance from the wall especially near excavation depth.

(iii) As pause time increases normalized ground deformation increases and the rate of increment near ground surface is more prominent. However, the magnitude is found to reduce with depth below the ground. Further it has been also clearly observed that as pause time increases the zone of ground profile deformation also shifts significantly and the zone of deformation increases rapidly. So, it can be an alarming point to note that as pause time increases during construction the extent or area of the deformation zone also increases rapidly. Therefore, during any excavation project it is suggested to minimize the pause time so that the neighboring structures would not affect due to these deformations.

(iv) Major part of the ground settlement is due to undrained deformation and consolidation deformation, while magnitude of creep deformation is not significant. Contribution of undrained deformation to the total deformation increases at near the wall.

(v) Rate of settlement initially is less and then it increases with time reaching maximum at 40 – 60 days of pause, beyond which it again reduces and becomes practically zero at about 250 days. The behavior is similar for all fourteen cases.

(vi) As pause time increases the zone of deformed soil also increases behind the wall for ground surface as well as at different depths and becomes more or less stable after about 200 days.

(vii) Normalized ground settlement decreases 20% - 30% with the installation of one additional strut for same depth of excavation.

5.3 CONTRIBUTION OF PRESENT INVESTIGATION TO THE EXISTING KNOWHOW IN LITERATURE

The main contributions of this extensive research on braced excavations are as follows:

(i) A design guideline for braced excavation problems, particularly in soft clay, has been proposed based on numerical studies that do not take time effects into account.

(ii) A simplified model has been proposed for estimating ground and braced wall displacements without accounting for time effects. This method offers an alternative to the more complex finite element analysis, allowing for quick and accurate predictions of soil deformation around braced excavations.

(iii) A design guideline for braced excavation problems, particularly in soft clay, has been proposed based on numerical studies that incorporate time effects. This approach offers practical insights for more accurate predictions in excavation design.

(iv) A simplified model is proposed for estimating ground and braced wall displacements while considering time effects. This model provides a more comprehensive approach to predicting soil deformation over time, enhancing accuracy in the analysis of braced excavations.

(v) A systematic experimental study of layer-wise ground deformation behind a braced excavation in soft clayey soil, similar to the conditions in Kolkata, was conducted using physical model tests. The study investigated various excavation depths, the number of struts, and the effects of construction delays or stoppages after reaching the final cut level on key factors such as the rate of settlement and the change in the zone of influence behind a braced wall. This research offers valuable insights into the deformation behavior under these different conditions.

(vi) A new theory has been proposed by accessing the mechanism of this deformation by highlighting the contribution of undrained, consolidation and creep deformation to the total ground deformation.

5.4 SCOPE FOR FUTURE WORK

(i) Plaxis 3D can be used for conducting the numerical study.

(ii) For the parametric study of braced excavations some other parameters such as wall stiffness, strut stiffness, earth pressure induced, strut load etc. can be used.

(iii) For conducting the numerical study or experimental model study the soil bed can be changed to stiff to medium clay or sand.

(iv) For conducting the numerical study some other numerical tool can be used such as ABACUS, Surfer, Geoprobe, GEOVIA Surpac etc.

(v) Inflight Excavator can be used as a tool for excavation in place of hand excavation during geotechnical centrifuge model study.

(vi) More deeper excavation depth i.e., beyond 14 m can be considered for the future study.

REFERENCES

1. Addenberooke, T. I., Potts, D. M. and Dabee, B. 2000. Displacement flexibility number for multi-propped retaining wall design. Journal of Geotechnical and Geo-environmental engineering, Vol. 126, No. 8, 718-726.
2. Allersma, H. G. B. 1991. Using Image Processing in Centrifuge Research. Proc. of the international conference, Centrifuge 91. A.A Balkema, Colorado, 551-558.
3. Athanasiu, C. M., Simonsen, A. S. and Ronning, S. 1991. Back-calculation of case records to calibrate soil-structure interaction analysis by finite element method of deep excavation in soft soil, *Proc. 10th European Conference on Soil Mechanics and Foundation Engineering, Florence, Italy*, Vol. 1, pp. 297-300.
4. Babu, G. L. S., Srivastava, A., Rao, K. S. N., and Venkatesha, S. 2011. Analysis and design of vibration isolation system using open trenches. *Int. J. Geomech*., 11(5), 364–369.
5. Balasubramaniam, A., Bergado, D., and Sutabutr, T. 1994. Deformation analysis of deep excavation in Bangkok subsoils. *Proceedings of the Thirteen International Conference on Soil Mechanics and Foundation Engineering*, *13(2)*, pp. 909-914.
6. Bjerrum, L. and Eide, O., 1956. Stability of strutted excavations in clay. *Geotechnique*, 6 (1), 32–47. doi:10.1680/geot.1956.6.1.32.
7. Bolton, M. D. and Stewart, D. I. 1994. The effect on propped diaphragm walls of rising groundwater in stiff clay. Geotechnique 44, No.1, 111-127.
8. Borja, R. I. and Lee, S. R. 1990. Cam-clay plasticity, part 1: implicit integration of elasto-plastic constitutive relations, Comput. Methods Appl. Mech. Eng., 78, 48-72.
9. Bowles, J. E., 1988. *Foundation analysis and design*. 4th Ed., New York: McGraw Hill.
10. Brinkgreve, R. B. J. and Vermeer, P. A., 2002. *Plaxis version 8 user׳s manual*. Rotterdam, The Netherlands: Balkema.

11. Broms, B. B., Wong, I. H. and Wong, K. S. 1986. Experience with finite element analysis of braced excavations in Singapore. *Proc. 2nd Int. Symp. On Numer. Methods in Geomechanics, Balkema*, Rotterdam, The Netherlands, 309-324.

12. Chowdhury, S. S., Deb, K., and Sengupta, A., 2013. Estimation of design parameters for braced excavation: Numerical study. *International Journal of Geomechanics c ASCE*,13 (3), 234–247. doi:10.1061/(ASCE)GM.1943-5622. 0000207.

13. Chowdhury, S. S., Deb, K., and Sengupta, A., 2016. Effect of fines on behavior of braced excavation in sand: Experimental and numerical study. *International Journal of Geomechanics ASCE*, 16 (1). doi:10.1061/(ASCE)GM.1943-5622.0000487.

14. Clough, G. W. and Hansen, L. A. 1981. Clay anisotropy and braced wall behaviour. *ASCE Journal of Geotechnical Engineering*, 107(7), pp 893-913.

15. Clough, G. W. and O′Rourkee, T. D. 1990. Construction induced movements of in situ walls. In *Proc. Conf. on Design and Performance of Earth Retaining Structures, ASCE, Geotechnical Special Publication ASCE*, New York, 25, 439–470.

16. Clough, G. W., Smith, E. W., Sweeney and B. P. 1989. Movement control of excavation support system by iterative design. *Foundation Engineering Current Principles and Practices*, Vol. 2, ASCE, New York, NY, pp. 869–882

17. Clough, G., and Tsui, Y. 1974. Performance of Tied-Back Walls in Clay. (ASCE, Ed.) *Journal of Geotechnical Engineering Division , 10* (12), 1259-1273.

18. Dan, K. and Sahu, R. B., 2012. A theoretical study on ground movement prediction for braced excavation in soft clay. *International Journal of Geotechnical Engineering*, 6 (1), 53–64. doi:10.3328/IJGE.2012.06.01.53-64.

19. Dan, K. and Sahu, R. B., 2018. Estimation of ground movement and wall deflection in braced excavation by minimum potential energy approach. *International Journal of Geomechanics*, 18 (7), 1–20. c ASCE, ISSN 1532-3641. doi:https://doi.org/10.1061/(ASCE)GM.1943-5622. 0001105.

20. Dan, K. and Sahu, R. B., 2022. Effect of arching on active earth pressure distribution against braced wall. *International Journal of Geotechnical Engineering*, 16 (7), 826–837. doi:10.1080/19386362.2021.1912921.

21. Davies, M. C. R. and Jones, A. M. 1998. Image Acquisition Using an On-board film Camera. *Proc. of the Int. Conference Centrifuge 98*. Tokyo, Japan

22. De Lyra Nogueira, C., de Azevedo, R., and Zornberg, J., 2009. Coupled analyses of excavations in saturated soil. *International Journal of Geomech*, 9 (2), 73–81. doi:10. 1061/(ASCE)1532-3641(2009)9:2(73).

23. Dong, M. and Jia, P., 2020. Stability analysis and parameter optimization of deep excavation supporting system in granular soils. *Advances in Civil Engineering*, 2020, 1–10. doi:10.1155/2020/8873655

24. Finno, R. J. and Harahap, I. S., 1991. Finite element analysis of HDR-4 excavation. *Journal of Geotechnical Engineering*, 117 (10), 1590–1609. doi:10.1061/(ASCE)0733-9410(1991) 117:10(1590).

25. Finno, R. J. and Chung, C. K. 1992. Stress strain strength responses of compressible Chicago Glacial clays. J. Geotech. Div. ASCE, Vol.118, No.10.

26. Finno, R. J., Atmatzidis, D. K., and Roboski, J. F., 2007. Three dimensional effects for supported excavations in clay. *Journal of Geotechnical and Geoenvironmental Engineering*, 133 (1), 30–36. 10.1061/ASCE1090-02412007133:130

27. Fourie, A. B. 1994. Finite element analysis of braced excavations in soft clay. Proceedings of the Thirteenth Interational Conference on Soil Mechanics and Foundation Engineering, 13(4), 1375-1380.
28. Garnier, J., Chambon, P., Ranaivoson, and Mathurin, C. 1991. Computer image processing for displacement measurements. *Proceeding of the international conference Centrifuge 91.* A. A Balkerma, Colorado
29. Gill, S. A., and Lukas, R. G. 1990. Ground movement adjacent to braced cuts. *ASCE Proc., Design and Performance of Earth Retaining Structures*, Cornell University, Ithaca, N.Y., No. 25, 471–488.
30. Goh, A. 1990. Assessment of basal stability for braced excavation system using finite element method. *Computer and Geotechnics* , 10 (4), 325-338.
31. Goh, A. T. C. 1994. Estimating basal heave stability for braced excavation in soft clay. *Technical note, Journal of Geotechnical Engineering*, Vol. 120, No.8, August, 1994, 1430-1436.
32. Goh, A. T. C., Fan, Z., Zhang, W., Zhang, Y. and Hanlong, L. 2017. A simple estimation model for 3D braced excavation wall deflection. *Computers and Geotechnics,* 83, 106–113. doi:10.1016/j.compgeo.2016.10. 022
33. Goh, A. T. C., Wong, K. S., and Broms, B. B. 1995. Estimation of lateral wall movements in braced excavations using neural networks. Canadian Geotechnical Journal, 32(6), 1059-1064.
34. Goldberg, D. T., Jaworski, W. E., and Gordon, M. D. 1976. Lateral support systems and underpinning. Vol. 1: Design and construction, Vol. 2: Design fundamentals, Vol. 3: Construction methods. *Federal Highway Administration, Reports*, 465. https://rosap.ntl.bts.gov/view/dot/14528
35. Guo, P., Gang, L., Lina L., Xiaonan G., Yixian W., Baojian L., Xunjian H., and Haibo H. 2022. Soil creep effect on time-dependent deformation of deep braced excavation. *Hindawi Advances in Material Science and Engineering*, 2022, 1–14. doi:10.1155/2022/5655592
36. Hashash, Y. M. A. and Whittle, A. J. 1996. Ground movement prediction for deep excavations in soft clay. *J Geotech Eng, ASCE*, 122(6):474–486.

37. Hashash, Y. M. A. and Whittle, A. J. 2002. Mechanisms of load transfer and arching for braced excavations in clay. *Journal of. Geotechnical and Geoenvironmental Engineering ASCE*. 128(3): 187–197

38. Ho, D. K. H. and Smith, I. M. 1991. Analysis of construction processes in braced excavations. *Proceedings of the Tenth European Conference on Soil Mechanics and Foundation Engineering*, 10(1), 213-218.

39. Hong, L., Chen, L., and Wang, X., 2022. Reliability analysis of serviceability limit state for braced excavation considering multiple failure modes in spatially variable soil. *Buildings*, 12 (6), 722. doi:https://doi.org/10.3390/buildings12060722.

40. Houghton, R. C. and Dietz, D. L. 1990. Design and performance of a deep excavation support system in Boston, Massachusetts. *Design and Performance of Earth Retaining Structures, ASCE Geotechnical Special Publication 25*, 795-816.

41. Hsiung, B. C. B., 2009. A case study of behavior of deep excavation in sand. *Computers and Geotechnics,* 36 (4), 665–675. doi:10.1016/j.compgeo.2008.10.003.

42. Hsieh, P. G. and Ou, C. Y., 1998. Shape of ground surface settlement profiles caused by excavation. *Canadian Geotechnical Journal*, 35 (6), 1004–1017. doi:10.1139/t98-056.

43. Imtiaz, H., Naseer, S., Khan, Z. and Mahmood, H. 2019. 2D numerical analysis of deformations in diaphragm wall supported by horizontal struts, ninth Intl. *Conference on Advances in Civil, Structural and Mechanical Engineering – ACSM,* ISBN No. 978-1-63248- 173-3, doi: 10.15224/978-1-63248-173-3-01.

44. Jen, L. C. 1998. The design and performance of deep excavation in clay. *Ph.D thesis, MIT*.

45. Kaiser, P. K. and Hewitt, K. J. 1982. The effect of groundwater flow on the stability and design of retained excavations. Canadian Geotechnical Journal, 19(2), 139-153.

46. Kimura, T., Takemura, J., Hiro-oka, A., Okamura, M. and Park, J. 1994. Excavation in soft clay using in-flight excavator. *Proc. Int. Conf. on Centrifuge, Singapore*. 94. 649-654

47. Kongsomboon, T., Tan, T. S., Yew, Y. K. and Mairaing, W. 2004. Centrifuge modeling of an excavation stabilized with embedded improved soil berm. *12th Asian Regional Conference on soil mechanics and geotechnical engineering.* Singapore.

48. Konkol, J. 2014. Derivation of the scaling laws used in geotechnical centrifuge modelling-application of dimensional analysis and buckingham P theorem. *Technical Sciences*. 17(1): 31–44

49. Kung, G. T. C., Juang, C. H., Hsiao, E. C. L, and Hashash, Y. M. A. 2007. Simplified model for wall deflection and ground surface settlement caused by braced excavation in clays, *Journal of Geotechnical and Geoenvironmental Engineering*, 133(6), 731-747.

50. Lambe, T. W. 1970. Braced excavations. *Proceedings Speciality Conference Lateral stresses in the ground and Design of Earth Retaining Structures*, 70. Ithaca, N.Y. 149–218.

51. Leung, R. K. Y., Li, E. S. F. and Leung, C. L. 2009. A Case Study: Design and Construction of Foundation and Braced Excavation at a Reclaimed Site at Waterfront, *The State-of-the-art Technology and Experience on Geotechnical Engineering in Malaysia and Hongkong*, 25th February.

52. Lien, B. H., Abedi, H., Ramos, J. A. and Porter, T. G. 1993. Performance of a Semi-Rigid Braced Excavation in Soft Clay. *3rd Conference of the International Conference on Case Histories in Geotechnical Engineering.* University of Missouri – Rolla.

53. Lim, G. T. 2003. Stabilization of an excavation by an embedded improved soil layer. Ph.D thesis, *National University of Singapore*.

54. Liu, G. B., Ng, C. W. W. and Wang, Z. W. 2005. Observed Performance of a Deep Multi-strutted Excavation in Shanghai Soft Clays. *Journal of Geotechnical and Geoenvironmental Engineering ASCE*. 131(8):1004-1013

55. Long, M. 2001. Database for retaining wall and ground movements due to deep excavation. *J Geotech Geoenviron Eng, ASCE,* 127(3):203–224

56. Mana, A. I., Clough, G. W. 1981. Prediction of movements for braced cuts in clay. *J Geotech Eng, ASCE*, 107(6):759–777

57. Madabhushi, G. 2015. Centrifuge modelling for civil engineers. *CRC press. Taylor and Francis Group*.

58. McNamara, A. M. and Taylor, R. N. 2002. Use of heave reducing piles to control ground movements around excavations. *Physical modelling in geotechnics: ICPMG'02, Newfoundland (eds R Phillips, PJ Guo and R Popescu) AA Balkema Publishers, Lisse*, 847-852.

59. Moormann, C. 2004. Analysis of wall and ground movements due to deep excavations in soft soil based in a new worldwide database. *Soils Found* 44(1):87–98.

60. Milligan, G. W. E. 1983. Soil Deformations near anchored sheet - pile Walls, *Geotechnique Journal*, 33, No. 1, pp. 41 – 55.

61. Ng, C. W.W., 1998. Observed Performance of multi-propped excavation in stiff clay. *J. Geotech & Geoenviron. Engrg.* Vol.124 (9). pp. 889-905.

62. Ong, D. E. L., Leung, C. E. and Chow, Y. K. 2006 Pile behaviour due to excavation-induced soil movement in clay I: stable wall. Journal of Geotechnical and Geoenvironmental Engineering. 132(1): January 36-44, @ASCE, ISSN1090-0241/2006/1

63. O′Rourke, T. D., 1981. Ground movements caused by braced excavations. *Journal of the Geotechnical Engineering Division, ASCE*, 107 (9), 1159–1178. doi:10.1061/AJGEB6. 0001183.

64. O'Rourke, T. D. 1993. Base stability and ground movement prediction for excavations in soft clay. Retaining structures, Thomas Telford, London, 131-139.

65. Ou, C. Y., Chiou, D. C. and Wu, T. S. 1996. Three-dimensional finite element analysis of deep excavations. *Journal of geotechnical engineering*, Vol 122, No.5, May,1996, pp337-345.

66. Ou, C. Y., Hsieh, P. G., and Chiou, D. C., 1993. Characteristics of ground surface settlement during excavation. *Canadian Geotechnical Journal*, 30 (5), 758–767. doi:10.1139/t93-068.

67. Ou, C. Y. and Lai, C. H. 1994. Finite element analysis of deep excavation in layered sandy and clayed soil deposit. *Canadian Geotechnical Journal*, Vol. 31,204-214.

68. Ou, C. Y., Liao, J. T. and Lin, H. D. 1998. Performance of diaphragm wall constructed using top-down method. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 124 (9), 808-987.

69. Peck, R. B. 1969. Deep excavations and tunneling in soft-ground. *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Mexico, 3, 225–290.

70. Potts, D. M., Fourie, A. B. 1984. The behaviour of a propped retaining wall-the results of a numerical experiment. *Géotechnique* 34(3):383–404.

71. Powrie, W. and Li, E. S. F. 1991. Finite element analyses of an in situ wall propped at formation level, *Geotechnique*, Vol. 41, No. 4, pp. 499-514.

72. Reinfurt, J. E., Anderson, T. C., Reitz, P., and Licari, T. 1994. Temporary shoring support systems in an urban environment. *Serviceability of Earth Retaining Structures*, *ASCE Geotechnical Special Publication* 42, 24-42.

73. Richard, D. J. and Powrie, W. 1998. Centrifuge tests on doubly propped embedded retaining walls in over-consolidated kaolin clay. Geotechnique 48, No. 6, 833-846.

74. Richards, D. J., Powrie, W., Roscoe, H. and Clark, J. 2007. Pore water pressure and horizontal stress changes measured during construction of a contiguous bored pile multi-propped retaining wall in Lower Cretaceous clays. *Géotechnique* 57(2):197–205.

75. Rodriguez-Ortiz, J. M. 1994. Influence of strut and anchor stiffness on the behaviour of diaphragm walls. *Proceedings of the Thirteenth International Conference on Soil Mechanics and Foundation Engineering*, 13(4), 1709-1712.

76. Sen, K. K., Alostaz, Y., Pellegrino, G. and Hagh, A. 2004. Support of Deep Excavation in Soft Clay: A Case History Study, *Proc., 5th International Conference on Case Histories in Geotechnical Engineering, New York*, April 13-17.

77. Som, N. N. and Raju, I. R. K. 1989. "Measurement of In-Situ Lateral Stresses in Normal Calcutta Deposit." *Proc. 12th ICSMFE* '89. Rio-de-Janeiro, Brazil, 3, 1519-1522.

78. Som, N. N. 1991. Performance study of Braced Cuts for Calcutta Metro Construction. Proc. 9thAsianRegional Conf. on SMFE, Bangkok, Vol.2, 387-394.

79. Som, N. N. 2000. *Report on Building settlement in Cut and Cover Construction for Northern Stretches of Calcutta Metro*. (Unpublished Report). Jadavpur University.

80. Sonnenberg, R., Bransby, M. F., Hallett, P. D., Bengough, A. G., Mickovski, S. B. and Davies, M. C. R. 2010. Centrifuge modelling of soil slopes reinforced with vegetation. *Canadian Geotechnical Journal*. 47: 1415–1430

81. Stolle, D. F. E., Vermeer, P. A., Bonnier, P. G. 1999. Time Integration of a Constitutive Law for Soft Clays. *Communications in Numerical Methods in Engineering Commun. Numer. Meth. Engng*, 15, 603-609.

82. Swanson, P. G. and Larson, T. W. 1990. Shoring failure in soft clay. *Design and Performance of Earth Retaining Structures*, ASCE GSP 25, 551- 561.

83. Takemura, J., Kondoh, M., Esaki, T., Kouda, M. and Kusakabe, O. 1999. Centrifuge model tests on double propped wall excavation in soft clay. *Soils and Foundations*. 39(3): 75-87

84. Tan, T. S., Yong, K. Y., Goh, T. L. and Kongsomboon, T. 2003. Behaviour of an embedded improved soil layer in an excavation. *Proceedings of Underground Singapore*. 27-28 November 2003, NTU, Singapore

85. Taylor, R. N., Robson, S., Grant, R. J. and Kuwano, J. 1998. An Image Analysis System for Determining Plane and 3-D Displacements in Soil Models. *Proc. of the Int. Conference Centrifuge 98.* Tokyo, Japan

86. Tefera, T. H., Nordal, S., Grande, L., Sandven, R. and Emdal, A. 2006. Ground settlement and wall deformation from a large-scale model teston a single strutted sheet pile wall in sand. *International Journal of Physical Modelling in Geotechnics*, 6 (2), 01–13. doi:10.1680/ijpmg.2006.060201.

87. Teparaksa, W. and Teparaksa, J., 2019. Comparison of diaphragm wall movement prediction and field performance for different construction techniques. *Underground Space*, 4 (3), 225–234. doi:10.1016/j.undsp.2019.01.001.

88. Terzaghi, K., 1943. *Theoretical soil Mechanics*. New York: John Wiley & Sons, Inc. doi:10.1002/9780470172766.

89. Terzaghi, K. and Peck, R. B., 1967. Soil Mechanics in engineering practice, *John Wiley and Sons, Inc., New York, Toronto, London*. pp.729.

90. Thasnanipan, N., Maung, A. W., Tanseng, P. and Wei, S. H. 1998. Performance of a braced excavation in Bangkok clay. *Diaphragm Wall Subject to Unbalanced Loading Conditions. Thirteenth Southeast Asian Geotechnical Conference*, Taipei, Taiwan: ROC. 655–660.

91. Whittle, A. J., Hashash, Y. M. A., and Whitman, R. V., 1993. Analysis of deep excavation in Boston. *Journal of Geotechnical Engineering*, 119 (1), 69–90. doi:10.1061/ (ASCE)0733-9410(1993)119:1(69).

92. Wong, K. S. and Broms, B. B. 1989. Lateral wall deflections or braced excavation in clay. *J. Geotech. Engrg*., Vol. 115, No.6, 853-870.

93. Wong I. H., Poh T. Y. and Chuah.H. L., 1997. Performance of excavations for depressed expressway in Singapore, J. *Geo. & Geoenv. Engrg.*, Vol. 123 (7). 617-625.

94. Vermeer, P. A., and Neher, H. P. 1999. A soft soil model that accounts for creep. *Beyond 2000 in computational geotechnics 10 years of PLAXIS International*, *Amsterdam*, *The Netherlands*, 18-20 March, 249.

95. Xiang, Y., Goh, A., Zhang, W. and Zhang, R. 2018. A multivariate adaptive regression splines model for estimation of maximum wall deflections induced by braced excavation. *Geomechanics and Engineering*, 14 (4), 315–324.

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A parametric study on deformation behaviour for design of braced excavation in soft clay

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ABSTRACT

Adequate prediction of surrounding ground movement during braced excavation is critically important as excessive soil movement damages adjacent structures. The magnitudes and patterns of ground movement and wall deflection largely depend on excavation parameters like thickness of diaphragm wall, wall embedment depth, strut locations and soil parameters such as soil strength, compressibility and creep parameter. In the present paper a thorough, parametric study has been conducted using finite element (FE) analysis to address the influence of various parameters on deformation characteristics of braced excavation in soft clayey deposits. The importance of correct estimation of soil parameters for braced excavation design is also documented. The analysis of typical braced excavations in soft clay is carried out using PLAXIS 2D software where soft soil creep constitutive model is used. On the basis of numerical study a handy design guideline is recommended. Further multivariate regression models are developed incorporating various important excavation parameters for the adequate prediction of maximum wall and ground displacement along with wall and ground surface deformation profile. Here large numbers of data reported in case histories and generated artificially from FE analysis are used for formation of regression equations. The proposed model is validated comparing results from literatures not used for the development of the model.

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Finite element analysis; ground movement; multivariate regression models; soft soil creep model; excavation; wall deflection

1. Introduction

In the recent years, rapid infrastructure development and scarcity of space for new constructions in urban India have resulted in constructions such as underground commuter (metro) railway, tall buildings with multiple basement floors, tunnels and similar other structures that require excavations to significant depths. These excavations require vertical sides with bracings (instead of sloped sides) for lack of space, and continuously braced wall structures are often used to ensure the stability of the excavations and to reduce the detrimental construction effects on the neighbouring structures and underground utilities. Vertical cuts with bracings, if not properly designed, may lead to excessive ground movement and wall deflections, which may cause distress to neighbouring structures. Therefore, ground movement and wall displacements should be estimated carefully and accurately while designing the braced excavation systems.

Early studies on braced excavations were based on field observations, and those studies focused on excavation-base instability caused by bottom heave, lateral movement of support systems, ground settlement adjacent to excavations, effects of soil type and excavation geometry on the performance of the excavation system, and earth pressure on braced walls (Terzaghi 1943, Bjerrum and Eide 1956, Peck 1969, Lambe 1970, Goldberg *et al.* 1976). Lambe (1970) concluded that the state of the art for design and analysis of braced excavations was far from satisfactory and suggested the use of finite element (FE) method (FEM) in conjunction with field studies as the way forwards for gaining proper understanding of deep excavation performance. Palmer and Kenney (1972) evaluated influences of different variables on braced excavations where interaction and behaviour of soil and supporting materials were taken into consideration. From their observation it was found that soil deformation modulus, wall stiffness, and strut stiffness have influence mostly on the behaviour of excavation. O'Rourke (1981) and Clough and O'Rourkee (1990) categorised movements in a braced cut into two types: movement related to excavation and support process and movement related to auxiliary construction activities. Finno and Harahap (1991) simulated the construction of a 40-feet-deep braced excavation in saturated clays in Chicago using

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An experimental study on time dependent ground settlement behind a braced excavation in soft clay using geotechnical centrifuge

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Abstract. Model studies using geotechnical centrifuge is now-a-days very popular for understanding the behaviour of foundations and braced excavations in soil. In the present investigation, a systematic study of the ground deformation behind a braced excavation in soft clayey soil similar to that available in Kolkata is performed using physical model study in geotechnical centrifuge considering construction delay/construction stoppage at different depths and distances away from the wall. The mechanism of this deformation was also assessed by evaluating the contribution of undrained, consolidation and creep deformation to the total ground deformation. The tests results were also used to predict the effect of construction delay on various important factors like rate of settlement, change of zone of influence behind a braced wall, etc. Further, the experimental results are also validated with the observed values obtained from the reported case studies.

Keywords. Geotechnical centrifuge; braced excavation; undrained deformation; consolidation deformation; creep deformation; construction stoppage.

1. Introduction

During the recent times, rapid infrastructure development and shortage of space for new constructions in urban India have resulted in constructions, such as underground (metro) railway, high rise buildings with multiple basement floors, tunnels, and similar other structures, that require excavations to significant depths. For most Indian cities, the excavations are executed along restricted traffic corridors with adjoining structures that are often old (including heritage buildings) and tall. Naturally, these excavations require vertical cuts with bracings, in place of sloped sides due to scarcity of space, and continuously braced wall structures to ensure the stability of excavations and to reduce the calamitous construction effects on the neighboring structures and underground services. Braced cuts with bracings, if not properly constructed and designed, may lead to excessive ground movement and wall deflections, consequently causing distresses to the neighboring structures. Therefore, ground movement and wall displacements should be estimated carefully and accurately while designing the braced excavation systems.

Early studies on braced excavations were based on field observations, and those studies focused on excavation-base instability caused by bottom heave, lateral movement of support systems, ground settlement adjacent to excavations, effects of soil type and excavation geometry on the performance of the excavation system, and earth pressure on braced walls [1–5] concluded that the state-of-the-art for design and analysis of braced excavations was far from satisfactory, and suggested the use of finite element method in conjunction with field studies as the way forward for gaining proper understanding of deep excavation performance. O'Rourke [6] pointed out the importance of site preparation in ground excavation work and related the lateral movement during of excavations to ground settlements, based on field observations. Clough and O'Rourke [7] categorized movements in a braced cut into two types: movement related to excavation and support process, and movement related to auxiliary construction activities. Hsiung [8] investigated the deformation characteristics of several excavations in Taiwan and highlighted that the maximum lateral wall displacement is approximately 0.03-0.3% of maximum excavation depth. It was also observed that the subsidence of the ground surface behind the diaphragm wall extended to a distance of up to three times the maximum excavation depth. Finno and Harahap [9] simulated the construction of a 40ftdeep braced excavation in saturated clays in Chicago by using a coupled finite element (FE) analysis. Whittle *et al* [10] performed coupled FE

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Analysis of Design Parameters Affecting Deformation Behaviour of a Braced Excavation in Soft Clay: Numerical Study

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Abstract - *Adequate prediction of ground movement during braced excavation is critically important as excessive soil displacement damage adjacent properties. Various factors like diaphragm wall thickness, wall embedment depth, strut locations influence the magnitudes and patterns of ground movement and wall deflection. In present paper an analysis of design parameters affecting deformation characteristics of braced excavation has been performed using finite element analysis. The importance of correct estimation of soil parameters for braced excavation design is also documented. The finite element analysis of typical braced excavations is implemented in soft clayey deposits using the software package Plaxis 2D, employing the soft soil creep constitutive model. On the basis of parametric investigation a design guideline is recommended which may be handy for design engineers..*

Key Words: Finite element analysis, ground movement, soft soil creep model, excavation, wall deflection.

1.INTRODUCTION

In the recent years, rapid infrastructure development and scarcity of space for new constructions in urban India have resulted in constructions such as underground commuter (metro) railway, tall buildings with multiple basement floors, tunnels, and similar other structures that require excavations to significant depths. These excavations require vertical sides with bracings (instead of sloped sides) for lack of space, and continuously braced wall structures are often used to ensure the stability of the excavations and to reduce the detrimental construction effects on the neighboring structures and underground utilities. Vertical cuts with bracings, if not properly designed, may lead to excessive ground movement and wall deflections, which may cause distress to neighboring structures. Therefore, ground movement and wall displacements should be estimated carefully and accurately while designing the braced excavation systems.

Early studies on braced excavations were based on field observations, and those studies focused on excavation-base instability caused by bottom heave, lateral movement of support systems, ground settlement adjacent to excavations, effects of soil type and excavation geometry on the performance of the excavation system, and earth pressure on braced walls (Terzaghi 1943, Bjerrum and Eide 1956, Peck 1969, Lambe 1970, Goldberg et al. 1976). Lambe (1970) concluded that the state of the art for design and analysis of braced excavations was far from satisfactory, and suggested the use of finite element method in conjunction with field studies as the way forward for gaining proper understanding of deep excavation performance. Palmer et al (1972) evaluated influences of different variables on braced excavations where interaction and behaviour of soil and supporting materials were taken into consideration. From their observation it was found that soil deformation modulus, wall stiffness and strut stiffness have influence most on the behaviour of excavation. Other parameters like soil shear strength, initial in-situ stress, soil to wall adhesion have lesser impact. O'Rourke (1981) pointed out the importance of site preparation in ground excavation work and related the lateral movement of excavations to ground settlements, based on field observations. Clough and O'Rourke (1990) categorized movements in a braced cut into two types: movement related to excavation and support process, and movement related to auxiliary construction activities. Finno and Harahap (1991) simulated the construction of a 40-ft-deep braced excavation in saturated clays in Chicago by using a coupled finite element (FE) analysis. Tefera et al. (2006) studied the ground settlement and wall deformation of a sheet pile wall during different stages of excavation using a large-scale model test in dry sand bed and compared the results with those of FE analysis. Finno et al. (2007) used the FE software PLAXIS for conducting a parametric study to show the effects of excavation geometry on the deformation behaviour of soil around braced excavations. They observed that when the ratio of the excavated length to excavated depth of a wall is greater than 6, plane strain simulations yield the same displacements in the centre of the wall as those obtained from three-dimensional FE analysis. Hsiung (2009) investigated the deformation characteristics of several excavations in Kaohsiung, Taiwan, and found that the maximum lateral wall displacement (δ_{hm}) is approximately 0.03-0.3% of

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A PHYSICAL MODEL STUDY ON GROUND DEFORMATION AROUND AN EXCAVATION USING DIAPHRAGM WALL IN SOFT CLAY

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***Abstract*—Model studies using geotechnical centrifuge is now-a-days very popular for understanding the behaviour of foundations and excavations in soil. In the proposed study a number of model tests were conducted for evaluating the deformation of soft clay around an excavation with diaphragm wall using a 1.0 m diameter geotechnical centrifuge at 100g. Size of the test box is 300 mm x 300 mm x 300 mm. Depth of the diaphragm wall was 170 mm. Tests were conducted for different depth of excavations, namely, 20 mm, 30 mm, 40 mm, 50 mm and 60 mm respectively and were run for duration of 20 minutes equivalent to about 4 months in prototype, in order to assess time effect on the deformation behaviour of a diaphragm wall during excavation. Digital image processing of the video images obtained during the tests were done using Matlab software in order to get the deformation pattern of adjacent ground at different time interval. Finally results were analyzed to highlight the effect of excavation depth, delay during excavation and installation of strut on the ground deformation behavior around an excavation.**

***Keywords* -Geotechnical Centrifuge, Excavation, Diaphragm wall, Matlab Software, Digital Image Processing, Ground Deformation.**

I. Introduction

Braced excavations are commonly used in urban areas, especially in metro cities, for the construction of underground pump houses, metro railway projects, and basements and foundations for high-rise buildings. Installation of braced walls along with lateral supports improves the stability of the excavations and, at the same time, reduces the induced settlement of the neighboring structures and service lines during construction. The overall performance of a braced cut and the effectiveness of the support system depend on their interaction with the surrounding soil. However, a particular feature of this soil-structure interaction problem is that excessive ground movement near the braced wall may occur because of delay in construction during the excavation process. The magnitude of additional deformation of the ground caused by construction delay may adversely affect the neighboring structures.

Several research studies have been performed on braced excavations since the late 1960s. The first study on the performance of excavation was made by Terzaghi and Peck in the early 1940s based on observation of structures in Chicago clay. Bjerrum and Eide (1956), Peck (1969), and Clough and O'Rourke (1990) performed several research studies related to the analysis and design of braced excavations. Finno et al. (2007) and Chouwdhury et al. (2014) computed the deformation behavior of soil around braced excavations using the finite element method (FEM). Field studies by Som and Gupta (1994), Ou et al. (1998), and Lien et al. (1993) indicate that ground deformations around braced excavations may increase because of construction delay. Lien et al. (1993) studied a 6.4 m wide and 8.5 m deep braced excavation in Detroit soft clay in the U.S.A. that was constructed in twelve stages, and it was found that, for excavation up to a depth of 5.18 m, the rate of ground deformation was quite low at about 0.164 mm/day, but, beyond that depth, there was a significant increase in ground deformation rate to about 0.677 mm/day. Som and Gupta (1994) reported the effect of time delay on deformations of braced cuts in soft clay based on extensive measurements of ground movement and building settlement at different sections of Kolkata underground metro rail excavation performed during the late 1980s. Additional ground settlement was observed when an excavation was

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Need For Construction Control in Braced Excavation

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***Abstract* – Braced excavations are commonly used in urban areas, especially in metro cities, for the construction of underground pump houses, metro railway projects, and basements and foundations for high-rise buildings. During these works, sometimes it is seen that the work is delayed or slowed down because of different reasons beyond the control of the engineer-in-charge. This is found to increase the rate as well as the magnitude of deformation of the soil within the influence zone of the excavation. Som et al. (1994) [7]., Ou et al. (1998) [8]. and Liu et al. (2005) [9]. reported case studies on ground deformations around braced excavations due to construction delay in three different soil deposits, i.e., Kolkata (India), Taipei (China) and Shanghai (China) soft clay respectively. This paper presents a detailed analysis of the time dependent ground deformation reported by above researchers. Further, it makes a comparative assessment of three typical braced excavations in soft clay deposit to assess the effect of different parameters like excavation rate and pause or delay time response on the time dependent behaviour of ground movement around the braced cut and thus focuses on the need of construction control in braced excavation especially in India.**

Keywords - Braced Excavation, Ground Deformation, Rate of Excavation, Delay or Pause Time.

INTRODUCTION

Braced excavations are commonly used in urban areas, especially in metro cities, for the construction of underground pump houses, metro railway projects, and basements and foundations for high-rise buildings. Installation of braced walls along with lateral supports improves the stability of the excavations and, at the same time, reduces the induced settlement of the neighboring structures and service lines during construction. The overall performance of a braced cut and the effectiveness of the support system depend on their interaction with the surrounding soil. However, a particular feature of this soil-structure interaction problem is that excessive ground movement near the braced wall may occur because of delay in construction during the excavation process. The magnitude of additional deformation of the ground caused by construction delay may adversely affect the neighboring structures. Several research studies have been performed on braced excavations since the late 1960s. The first study on the performance of excavation was made by Terzaghi and Peck in the early 1940s based on the observation of structures in Chicago clay. Bjerrum and Eide (1956) [1]., Skempton (1964) [2]., Peck (1969) [3]., and Clough and O'Rourke (1990) [4]. performed several research studies related to the analysis and design of braced excavations. Finno et al. (2006) [5]. and Choudhury et al. (2014) [6] computed the deformation behavior of soil around braced excavations using the finite element method (FEM). Field studies by Som and Gupta (1994) [7]., Ou et al. (1998)[8]., and Lin et al. (2005) [9]. indicate that ground deformations around braced excavations may increase because of construction delay.

In this paper a comparative study of three typical braced excavations in soft clay deposit were carried out to access the effect of different parameters like excavation rate and pause or delay time response on the time dependent behavior of ground movement around the braced cut. The three typical braced excavations which were studied were adopted from Som et al. (1994) [7]. of kolkata (India), Ou et al. (1998) [8]. of Taipei (China) and Liu et al. (2005) [9]. of Shanghai (China) soft clay deposits respectively. This study focuses on the need of construction control in braced excavation especially in India. Significant amounts of excavation work have been done in the past during the construction of various infrastructure projects, underground excavation is in progress. During these works, sometimes it is seen that the work is delayed or slowed down because of different reasons beyond the control of the engineer-in-charge. This, in general, is found to increase the rate as well as the magnitude of deformation of the soil within the influence zone of the excavation.

A. Subsoil Stratification and Settlement Data with Delay Time of Three Sites

Som and Gupta (1994) [7]. reported the effect of time delay on deformations of braced cuts in soft clay based on extensive measurements of ground movement and building settlement at different sections of Kolkata underground metro rail excavation performed during the late 1980s. In certain sections, it has been observed that excavation has been left open for longer periods of time either because concreting for the subway box construction could not be commenced early or because there were stoppages due to other reasons for that additional ground settlement was observed. The details of the soil profile along with the properties of Normal

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Time Effect on Ground Deformation around Braced Excavation

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ABSTRACT: A finite element analysis of a typical braced excavation in normal Kolkata deposit was carried out to assess the time dependent behavior of ground movement around the braced cut. A typical 30 m wide and 14 m deep excavation with 17 m deep diaphragm walls was adopted from a reported case study of Kolkata metro construction during the late eighties, and was analyzed in the present study. The analysis was performed by using the software package Plaxis 2D in which the soft soil creep constitutive model was used. The time dependent ground deformations obtained from the analysis were compared with the reported data of the case study. Analyses were performed to highlight the effect of rate of excavation on the deformation behavior of the excavation.

Keywords: *Finite element analysis, Ground movement, Time-dependent ground deformation, Excavation*

1.0 Introduction

Braced excavations are commonly used in urban areas, especially in metro cities, for the construction of underground pump houses, metro railway projects, and basements and foundations for high-rise buildings. Installation of braced walls along with lateral supports improves the stability of the excavations and, at the same time, reduces the induced settlement of the neighboring structures and service lines during construction. The overall performance of a braced cut and the effectiveness of the support system depend on their interaction with the surrounding soil. However, a particular feature of this soil-structure interaction problem is that excessive ground movement near the braced wall may occur because of delay in construction during the excavation process. The magnitude of additional deformation of the ground caused by construction delay may adversely affect the neighboring structures.

Several research studies have been performed on braced excavations since the late 1960s. The first study on the performance of excavation was made by Terzaghi and Peck in the early 1940s based on observation of structures in Chicago clay. Bjerrum and Eide (1956), Skempton (1964), Peck (1969), and Clough and O'Rourke (1990) performed several research studies related to the analysis and design of braced excavations. Finno et al. (2006) and Choudhury et al. (2014) computed the deformation behavior of soil around braced excavations using the finite element method (FEM). Field studies by Som and Gupta (1995), Ou et al. (1998), and Lin et al. (2005) indicate that ground deformations around braced excavations may increase because of construction delay. However, a systematic investigation on the effect of excavation rate and of time delay in construction on the soil displacements around a braced excavation is lacking.

In this study, finite element (FE) analysis is performed to evaluate the deformation characteristics of braced excavations using the Plaxis 2D software in which the soft soil creep constitutive model is used. A reported case study (Som 2000) on Kolkata Metro construction at the location of 164 C. R. Avenue was considered for validation of the FE analysis and further investigations. Analyses were carried out to investigate the effects of rate of excavation on ground deformation during excavation.

2.0 Excavation Site and Construction Sequence

The excavation of Kolkata Metro construction was performed using the cut and cover method. At the 164 C. R. Avenue site, the final depth of excavation was 14 m where 17 m diaphragm walls were used to retain the earth. The excavation was 30 m wide and the diaphragm wall was 0.6 m thick, and it was supported with horizontal struts at an interval of 4.25 m. The construction sequence of the excavation along with the sequence of progress of excavation work for this site (consisting of nine stages) is shown in Fig. 1 and the details are given in Table 1. The geometry and construction sequence of this site was used in the finite element analysis.

The soil profile at the site consists of four layers with a top 3-m thick desiccated brownish-grey clayey silt layer (L1) underlain by a 11-m thick soft dark-grey silty clay layer (L2) with decomposed wood. The third layer (L3) is 6 m thick and consists of a stiff bluish-grey silty clay with kankar, and the fourth layer (L4) is 10 m thick and consists of yellowish-brown sandy clayey silt – this layer is used up to the termination depth of the numerical domain. For the Kolkata Metro construction, the soil within the excavation depth is largely soft in nature (with undrained shear strength c_u = 25 KN/m^2). For this type of deposit, creep effect may cause considerable amount of ground displacement. In fact, large primary settlements

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EFFECT OF RATE OF EXCAVATION ON GROUND DEFORMATION AROUND A BRACED CUT

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ABSTRACT: A finite element analysis of a typical braced excavation in normal Kolkata deposit was carried out to assess the time dependent behavior of ground movement. A typical 30 m wide and 14 m deep excavation with 17 m deep diaphragm walls was adopted from a reported case study of Kolkata metro construction during the late eighties and was analyzed in the present study. The analysis was performed by using Plaxis 2D Foundation software with soft soil creep (SSC) model. Ground deformations obtained from the analysis were compared with the reported data of the existing case study. Parametric studies were also performed to highlight the effect of rate of excavation on the deformation behavior of the excavation.

Keywords: *Finite element analysis, Ground movement, Time dependent ground deformation, Excavation*

INTRODUCTION

Braced excavations are very commonly used now-a-days in urban areas especially in metro cities for the construction of underground pump houses, metro railway projects and basements or foundations for high-rise buildings. Installation of braced walls along with lateral supports improves the stability of the excavations and at the same time reduces the induced settlement of the neighboring structures and service lines during construction. Though the overall performance of a braced cut and effectiveness of the support system depend upon their soil-structure interaction behavior, sometimes, excessive ground movement near the braced wall may occur due to delay during excavation adversely affecting the neighboring structures. The magnitude of additional deformation of the ground caused by the delay during excavation depends on the type of soil.

Researchers and engineers have been working since late 1960s on the nature of ground deformation and design of braced excavation. Since then a lot of works has been done to design braced excavation and to investigate the effect of various parameters on deformation behavior of braced cut. The first study on the performance of excavation was made by Terzaghi in the early 1940s based on the observation of structures in Chicago clay. Skempton et al. (1948), Bjerrum et al. (1956), Peck et al. (1969) and Clough et al. (1981, 1990) have performed several research studies related to the design of braced excavation. Numerical analysis of braced cut using the Finite Element Method (FEM) is a viable option, as ground and wall deformation can be reasonably predicted even for the small strain model. Finno et al. (2007) and Choudhury et al. (2013) have computed the deformation behavior of soil during braced excavation using numerical methods. Studies have been done to predict the effect of excavation rate and time delay and these have been documented in the literature. Som et al. (1995), Ou et al. (1998) and Liu et al. (2005) reported the time dependent settlement behavior of braced excavations at different site. However, a systematic investigation regarding the effect of excavation rate or time delay on soil displacement around a braced excavation is lacking.

In this study numerical analysis is performed to evaluate the deformation characteristics of braced excavation using PLAXIS 2D FOUNDATION software with soft soil creep model. A reported case study (Som 2000) on Kolkata Metro construction at location 164, C. R. Avenue was considered for validation of the numerical analysis results. Analyses were also carried out to

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