LABORATORY SIMULATION AND FINITE ELEMENT ANALYSIS OF IRRIGATION-INDUCED SETTLEMENTS OF BUILT ENVIRONMENT OVERLYING COLLAPSIBLE SOIL STRATA IN UNITED ARAB EMIRATES

A thesis *by*

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ABSTRACT

Collapsible soils exist in some arid environments and have a unique deformation response upon saturation so they cannot be reliably assessed with conventional elastic and consolidation theories. Such soils may be stable in the dry state but suddenly collapse upon wetting due to loss of suction and breakage of inter-particle friction and cementing bonds. This necessitates the use of unsaturated soil mechanics theories or field wetting-loading tests, both of which are too sophisticated and uneconomic for routine geotechnical design.

This thesis emanated from two case studies in the United Arab Emirates (UAE) where a forensic geotechnical investigation had been undertaken, as a consequence of severe settlement and structural damage associated with infiltration of irrigation water deep into collapsible strata. Records from instrumentation and test boreholes drilled by a specialist consultant are analysed along with specific irrigation regimes actually applied in the case study areas. The primary aim of this thesis is to develop alternative cost-effective approaches to simulating the deformation characteristics of a collapsible soil and formulating mathematical solutions for collapsible soil settlement under specified irrigation patterns. A laboratory simulation test is developed to study the irrigation-induced response of a collapsible soil specimen under overburden and seepage conditions that represent the case study situations. The test incorporates a large steel tank in which a layer of collapsible soil, of variable thickness, sandwiched between two other layers and subjected to varying water levels and constant irrigation intensity and surcharge. The test procedure and variables are designed to model the case study site conditions as realistically as possible. Test results showed that the number of wetting cycles required for the soil to reach collapse state increases with increase in depth of water table. Also, upon the start of collapse, the rate at which it collapses remains identical regardless of its thickness. Using back-analysis and a 3D finite element approach, predictive methods are developed which can be used to estimate actual settlement in the field.

The suggested numerical method is implemented in *Midas*^{TM} software and tested against data from the case studies to demonstrate a remarkable agreement with the measured surface settlements and cracking patterns in the affected structures. The suggested method takes into account important factors including the depths and thicknesses of the collapsible strata, the *in-situ* stresses, transient water flow, irrigation cycles, water table depth and the soil-structure mechanical properties. The proposed method of assessing irrigation-induced settlement will assist future geotechnical designs as well as in selection of suitable methods of protecting existing structures built on collapsible strata.

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GLOSSARY OF TERMS

Term	Definition
Arid region	Regions in which the climate is mostly dry throughout the year with very less rainfall (less than 20cm of annual rainfall).
Borehole log	Record obtained by drilling into ground up to desired depth that shows the type of soil/rock with depth.
Collapsible soil	Unsaturated soils that can withstand relatively high pressure in dry conditions without significant volume change, but subjected to sudden reduction in volume upon wetting.
Consolidation	Reduction in volume of soil due to expulsion of water.
Drip irrigation	Type of irrigation system that involves trickling of water at slower rates near to plants.
Finite element method	Numerical method that utilizes mathematical physics to replicate behaviour of a material in solving engineering problems.
Infiltration	Permeation of liquid into soil by filtration.
Meta-stable soil	Soils that possess stable soil structure in their dry state.
Silt sized particles	Particle with size range between 0.06 mm and 0.002 mm.
Simulation	Process of producing (in lab or in software) an abstract representation to represent the actual situation in the real world.
Structural collapse	Reduction in total volume of soil due to loss of inter-particle friction via entry of water.
Transient flow	Unsteady flow in which rate of application of water is variable.
Wind-blown soil	Soils transported from one place to another place by wind. They are known as Aeolian soils.

CHAPTER 1 - INTRODUCTION

1.1 Introduction

Collapsible soils are found in many parts of the world such as USA, Central and South America, China, Africa, Russia, India and the Middle East (Derbyshire *et al.* 1995; Mitchell and Soga, 2005; Murthy, 2010) and cover approximately 10% of the earth's surface (Evans *et al.* 2004; Northmore *et al.* 2008). It should be understood that the term "collapsible soils" does not mean a particular soil type but rather a whole variety of soils that are susceptible to structural collapse and examples include wind-blown sand, loess or alluvial soil types (Kalantari, 2013).

Collapsible soils primarily comprise silt sized particles and are generally found in an unsaturated state in their natural condition in arid and semi-arid regions. This observation was made by, among others, Zhu and Chen (2009). In the opinion of many researchers, notably Noutash *et al.* (2010), collapsible soils are generally characterized by their natural dryness, openness in structure and high porosity. As for many other soils, the mechanical properties of a collapsible soil are strongly influenced by the particle structure and this is supported by the works of Schmertmann (1955), Graham and Li (1985), Holtz *et al.* (1986), Leroueil and Vaughan (1990) and Wesley (1990).

Many researchers (Brandon *et al.* 1990; Ayadat and Hanna, 2007 & 2008; Jotisankasa, 2005; McCarthy, 2006; Rezaei *et al.* 2012) have recognized the unique characteristics of collapsible soils as regards susceptibility to sudden strength loss due to water infiltration resulting from precipitation or irrigation. In the dry state, collapsible soils may be stable and competent (Alain *et al.* 2012) but in a saturated state the loss of suction and tensile strength causes rapid structural breakdown of the cementing bonds. Casagrande (1932), Barden *et.al.* (1973), Mitchell (1976), Lawton *et al.* (1989), Pereira and Fredlund (2000), Haeri *et al.* (2014) and Langroudi *et al.* (2018) concurred that water ingress into a meta-stable soil, such as a collapsible layer, can significantly destroy the frictional resistance between the soil grains hence cause volume reduction and settlement. Other researchers e.g. Dudley (1970), Petrukhin (1989) and Reginatto and Ferrero (1973) suggested that the wetting-induced strength loss of a collapsible soil is linked to the dissolution of compounds that bond loosely arranged soil particles. A direct consequence of soil collapse is settlement, which is detrimental to any structures directly bearing on a collapsible layer that can be infiltrated by water.

It should be noted that collapse mechanism is unrelated to consolidation since, as reported by Pye and Tsoar (1990), the high rate of soil moisture loss in dry regions implies insufficient time for an underlying collapsible soil stratum to consolidate under the in-situ stresses. The onset of collapse requires a relatively short period of time once saturation levels are sufficiently high. Clemence and Finbarr (1981) observed that strength loss in a collapsible soil is markedly significant when the degree of saturation is above 50%. Houston *et al.* (1993) and Abbeche *et al.* (2010) mentioned that full saturation is not necessarily required for the soil to exhibit collapse. This opinion contrasts that of Houston *et al.* (2002) who stated that total collapse of certain soils at a given stress level requires a state of full saturation. In urbanized arid/semi-arid sites, water from pipeline leakages, irrigation operations and industrial activities can also percolate deeply into beds of collapsible soils underlying the site (Adnan and Erdil, 1992).

1.2 Problem statement / motivation for this study

Until recently, the potential risks associated with collapsible soils in the United Arab Emirates (UAE) had not been widely understood by Civil Engineers as well as infrastructure owners. In particular there has been very little recognition of the effects of cyclic wetting and drying of collapsible strata on which structures are supported. For this reason, many property owners in the UAE have applied irrigation to green their environment while completely oblivious of the likely ground settlement that would occur if collapsible strata existed superficially beneath the site. This thesis reports two case studies in UAE regions where extensive structural damage occurred owing to large subsidence caused by irrigation water seeping into collapsible strata beneath.

In pursuit of solutions to this problem, vast data in the form of borehole logs, laboratory soil tests and on-site monitoring of various infrastructures like boundary walls, gazebos, green areas (drip irrigated areas), paved areas (e.g. footpaths) was collected from the case studies. Utilizing the special opportunity presented by the case studies, the current research was initiated to develop a fuller understanding and predictive solutions for collapsible soils. The main research methodology includes laboratory simulation and finite element modelling of the mechanisms of surface water percolation into collapsible soil strata and the understanding the resulting effect on geotechnical structures. Moreover, the target was to provide recommendation on how best to estimate the possible settlement magnitudes due to water ingress and to avoid or ameliorate the problem of structural distress occasioned by collapsible soils.

1.3 Research gap and contribution to the existing knowledge

Many researchers have previously attempted laboratory and field methods aimed at characterizing collapsible soils. However most of the methods involve time consuming and resource intensive tests such as tri-axial and oedometer tests. In addition, the methods do not account for the effects of water ingress into soil yet this is an important consequence of drip irrigation, pipeline leakage and precipitation. Field tests may yield results that are more representative of the real soil than laboratory tests, but they are considerably more expensive and so the importance of empirical correlations in geotechnical analysis cannot be overstated. Therefore, a deep understanding of how infiltration affects collapsible soils is paramount, if the problems experienced at the case study sites are to be avoided in future.

1.4 Objectives of the research work

The primary aim of the present work is to develop laboratory and numerical methods for use in predicting the magnitude of settlement of collapsible strata under the influence of surface irrigation. The deliverable objectives of the research are:

1) To catalogue and evaluate the applicability of existing methods of assessing collapsible soil settlement, in the light of the lessons learnt from the UAE case studies.
2) To build a comprehensive database of ground investigation and structural deformation monitoring for the case UAE sites where severe movement of collapsible strata caused extensive damage to structures.
3) To develop a laboratory method of simulating the response of a collapsible soil sample loaded incrementally while subjected to constant infiltration.
4) To use the results from objective (3) above to formulate predictive equations for estimating collapsible soil settlement in real field situations.
5) To develop a 3D finite element procedure for analysing irrigated landscapes underlain by collapsible soils and to extend the method to predict the pattern of structural deformations that would occur in the real field situation.

1.5 Structure of thesis

The thesis structures is arranged as 8 consecutive chapters, each articulating a distinct stage of the work although a sensible overlap of information is maintained to give a natural flow of ideas and steps undertaken.

Chapter - 1 Introduction

This chapter begins by defining collapsible soils, their unique challenges and need and timeliness of this research, following case records of structural damage in two case study sites overlying collapsible strata. The problem statement is articulated and the motivation for the work outlined. The chapter also briefly sets out the weaknesses of current approaches to analysis of collapsible soil settlement and outlines the expected contribution to knowledge that the work promises, if successful. Lastly the objectives of the research work are succinctly expressed, followed by a short statement of the proposed methodology and strategies for delivering the objectives.

Chapter - 2 Review of literature

This chapter starts by summarizing the micro-structure, the unique behavioural patterns of collapsible soils and relevance of unsaturated soil mechanics theory in analysis. The chapter also describes various approaches that have been suggested to help identify collapsible soils, factors that are influential in their response to water ingress and the critical moisture contents necessary to initiate collapse. Also examined are various published methods of assessing deformation magnitudes under the effects of water ingress as well as possible field methods that could be used to prevent or ameliorate the severity of collapsible soil settlement induced by infiltration of surface water. A large number or relevant papers in the wider area of collapsible soils are summarized. Particular attention is paid to the existing range of laboratory related work by a number of researchers in their quest to extend knowledge of the settlement of collapsible soils under the effects of saturation. In the light of the existing information base, critical comments are developed to bring to the fore the drawback with current methods and potential new paths to deeper understanding.

Chapter - 3 Case studies in U.A.E

This chapter summarises typical case records in UAE with particular focus on:

i. Diverse infrastructures that were affected by ground settlement associated with the unique mechanisms of collapsible soils.
ii. The ground conditions and soil properties at the affected locations.
iii. The human activities responsible that triggered soil collapse and how this was confirmed through forensic geotechnical investigation.
iv. Patterns of deformation in the various structural elements with narrative photographs.

Chapter - 4 Experimental methodology

This chapter describes:

i. A purposely developed apparatus to simulate infiltration of irrigation water and consequential effects on a collapsible stratum.
ii. A custom designed test to simulate the effects of variable water table and constant surcharge on a collapsible stratum.
iii. Measurement of settlements of a collapsible layer under constant pressure and variable irrigation regimes.

Chapter - 5 Finite element modelling

This chapter discusses:

i. Finite element modelling of a complete twin-villa complex with actual dimensions and infrastructure loads.
ii. Calculation of settlement of various infrastructures due to input infiltration rates and patterns that mirror the actual irrigation processes practised at the case study sites in U.A.E.
iii. Interpretation of the finite element results to explain the characteristic failure patterns of the boundary wall elements of the case study structures.

Chapter - 6 Results and discussions

This chapter presents:

i. The results of laboratory tests and numerical analyses carried out.
ii. Comprehensive discussions of the salient relationships typified by all graphs plotted and their implications in understanding the behavioural mechanisms of collapsible soils.
iii. Predictive equations specially formulated from the research findings for possible use in estimating collapse settlements in real ground situations.
iv. A comparison of calculated deformations with those observed in the case study sites.

Chapter - 7 Conclusions and recommendations for collapsible soil sites

This chapter consists of conclusions obtained from laboratory tests and finite element analysis. It also outlines various practical recommendations to assist geotechnical engineers in dealing with the special risks that collapsible strata pose to infrastructure.

Chapter - 8 Recommendations for future research work

This chapter briefly discusses possible avenues of using the successes of the present work, the problems experienced and the lessons learnt can be used as a foundation by researchers to refine the proposed solutions, overcome the existing barriers and harness the full potential of laboratory and numerical modelling so that engineers understand collapsible soils much better and are able to protect infrastructure from the kind of disaster witnessed in the UAE case studies.

CHAPTER 2 – REVIEW OF LITERATURE

2.1 Introduction

Geotechnical engineers often utilize *in-situ* methods such as the standard penetration test (SPT) in ground investigation to assess bearing capacity and settlement of soils. However, if collapsible soil layers are present at a site, special challenges are presented owing to the intrinsic vulnerability of such soil types. Depending on the overburden pressure and groundwater table, collapsible soils generally produce SPT blow counts ("N" values) that correspond to medium dense to dense soils. However, unlike other soil types, a collapsible soil undergoes a large reduction in strength and volume when sufficiently saturated, as the inter-particle and cementation bonds collapse due to loss of suction. The consequence is significant settlement and possible damage to any structures bearing directly on such a type of soil stratum.

Despite the complex behaviour of unsaturated collapsible soils under water ingress, some researchers have suggested analysis approaches that are based on laboratory testing of undisturbed samples in oedometer or tri-axial apparatus. However, in the context of the present work, a major limitation of such approaches is that it is extremely difficult if not impossible to extract undisturbed samples of collapsible desert soils from boreholes.

Other researchers have attempted to develop methods that utilise field testing, in order to side step the problem of obtaining truly representative soil samples.

More recently, with technological advancements, numerical analysis and computing have presented opportunities to for radical new ways of predicting collapsible soil behaviour. Advanced numerical techniques such as the 3-dimensional finite element (FE) method have enormous capabilities to model complex behavioural mechanisms while taking into account a large range of the influential factors, with which simple methods would not be able to cope. With regards to existing analysis approaches for collapsible soils, the most notable articles that align directly with the goals of the current research are listed in Table 2.1.

Table 2.1 Methodologies by various researchers in analysing collapsible soils

Approach towards analysing the collapsible soil behaviour	**References**	
Laboratory methods	Denisov (1951)	Abdrabbo and Abdelaziz (2006)
	Clevenger (1958)	Ayadat and Hanna (2007)
	Gibbs (1961)	Reznik (2007)
	Holtz and Hilf (1961)	Das (2007 and 2009)
	Benites (1968)	Yuanqing and Zhenghan (2009)
	Handy (1973)	Soliman and Hanna (2010)
	Jennings and Knight (1975)	Wang et al. (2010)
	Jasmer and Ore (1987)	Brink (2011)
	Lawton et al. (1992)	Kakoli and Hanna (2011)
	Tadepalli et al. (1992)	Thorel et al. (2011)
	Houston et al. (1993)	Gaaver (2012)
	Rollins and Rogers (1994)	Rezaei et al. (2012)
	Anderson and Reimer (1995)	Fagundes et al. (2015)
	Bell (2000)	Fattah et al. (2015)
	Celestino et al. (2000)	Garakani et al. (2015)
	Houston et al. (2001)	Mashhour and Hanna (2016)
	Rao and Revanasiddappa (2002)	Arabani and Lasaki (2017)
	Khalili et al. (2004)	Ayeldeen et al. (2017)
Field tests	Reznik (1993)	Souza et al. (1995)
	Houston et al. (1995)	Lollo et al. (2011)
	Mahmoud et al. (1995)	Freitas et al. (2017)
Numerical analysis and computing	Alonso et al. (1990)	Wheeler et al. (2003)
	Balmaceda et al. (1992)	Basma and Kallas (2004)
	Gens and Alonso (1992)	Sun et al. (2007)
	Wheeler and Sivakumar (1995)	Kakoli et al. (2009)
	Cui and Delage (1996)	Sheng (2011)
	Wheeler (1996)	Arairo et al. (2013)
	Habibagahi and Mokhberi (1998)	Rotisciani et al. (2015)
	Kato and Kawai (2000)	Noor (2017)

2.2 Collapse characterization through laboratory tests

Holtz and Hilf (1961) suggested that if the voids ratio of a loess-like soil is large enough that the moisture content exceeds the liquid limit, then that soil would be susceptible to collapse. The above researchers developed a simple criterion for identifying whether or not a soil is collapsible, provided the dry density and liquid limit are known. The criterion is illustrated in Figure 2.1. If the data point describing the above two parameters falls on or below the solid line then that soil will collapse under the ingress of water. In the more recent articles, Houston *et al.* (1993) and Das (2007 and 2009) also suggested that collapsibility can be evaluated though determination of dry density and liquid limit. However, soils in UAE are largely dry silty sands and hence to determine liquid limit, indirect methods such as the cone penetrometer test may be more appropriate than routine laboratory methods.

Tadepalli *et al.* (1992) suggested a simple experimental procedure using an oedometer to measure changes in matric suction and soil volume during progressive addition of water. Test results from the experiments indicated that collapsible behaviour of soil can be described by means of unsaturated soil mechanics theories.

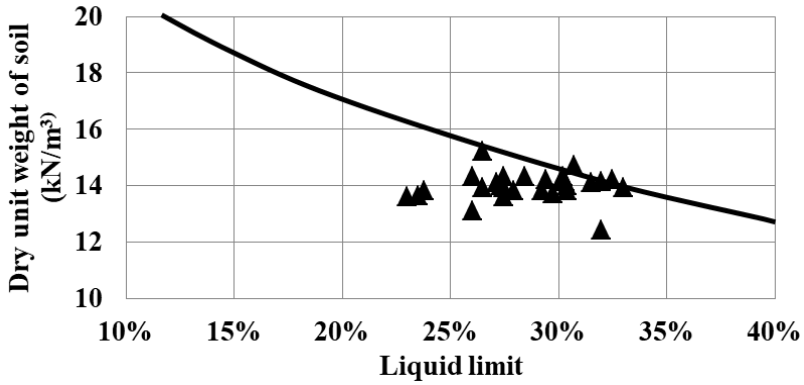


Figure 2.1 Dry unit weight of soil versus liquid limit (Holtz and Hilf, 1961)

Anderson and Riemer (1995) used constant-shear-drained (CSD) tri-axial tests on samples of uniformly graded sand and undisturbed alluvial soil and concluded that the potential of a soil to collapse can only be determined based on knowledge of the stress paths.

Bell (2000) suggested a qualitative description (Table 2.2) of collapse severity by analysing results from collapse potential tests carried out using the conventional oedometer apparatus. The collapse was calculated as the percentage reduction in height of the soil specimen compared with initial height at the start of the test.

Table 2.2 Collapse percentage as an indication of severity of collapse (Bell, 2000)

Collapse (%)	Severity of problem
0 - 1	No problem
1 - 5	Moderate trouble
5 - 10	Trouble
10 - 20	Severe trouble
Over 20	Very severe trouble

Celestino *et al.* (2000) carried out experiments and mechanical response modelling of samples of the silty sand which formed the core of the Metramo dam in Italy. The soil samples were compacted at optimum moisture content using the modified Proctor method after which oedometer and stress path triaxial cell tests were conducted with controlled suctions in the range 0–400 kPa. The results showed a strong influence of suction on stiffness, shear strength and compressibility. Moreover, even at low stress levels the soil exhibited collapsible behaviour upon wetting.

Houston *et al.* (2001) conducted consolidation tests and measured collapse strains on 3 collapsible soil specimens under partial to full wetting conditions. It was found that vertical deformations of all three samples directly depended on the degree of saturation of the soil, so that the higher the moisture content the greater was the settlement. It was also found that the collapse strains were a maximum when the samples were close to their full saturation (100%).

Rao and Revanasiddappa (2002) made an attempt to characterise the collapse behaviour of a sample of residual red soil in Bangalore district of Karnataka State, India. The soils were porous and tended to collapse on wetting. Filter paper method was used to determine the matric suction of the bonded and unbonded (remoulded) specimens. Additionally the soil micro-structural character was analysed using Mercury Intrusion Porosimetry (MIP) technique. The studies collectively showed that bonding had a significant role in the collapse behaviour of the residual soil in unsaturated state.

Khalili *et al.* (2004) carried out an extensive effective stress analysis and laboratory tests on undisturbed clays from the site of Hume Dam, south-eastern Australia and showed that the settlement of the soil was mainly due to reduction in the yield stress.

Some researchers (Tadepalli *et al.* 1992; Anderson and Riemer, 1995; Bell, 2000; Celestino *et al.* 2000, Houston *et al.* 2001; Rao and Revanasiddappa, 2002; Khalili *et al.* 2004) have attempted to use oedometer and triaxial tests on undisturbed samples to evaluate soil collapsibility. However, such tests are time consuming and additionally it is

very difficult to obtain truly representative samples from the dry granular soils at the UAE sites.

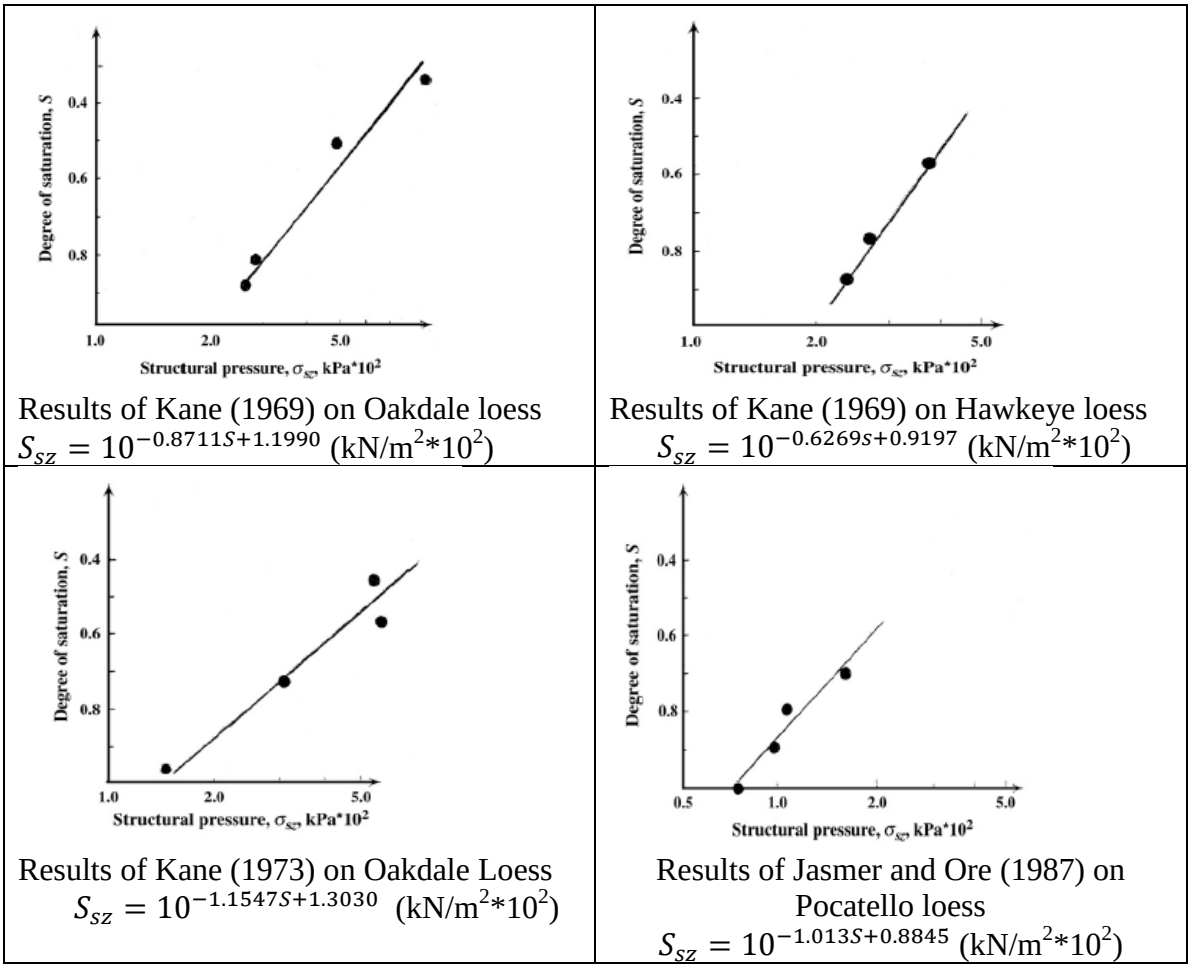
Abdrabbo and Abdelaziz (2006) conducted laboratory infiltration tests, through simulation of rainfall, to predict the extent of wetting within a collapsible soil zone loaded with a model footing. The tests allowed real time monitoring of moisture distribution below and around the footing and revealed that, in the initial infiltration stages the wetted zone was not horizontal however beyond a certain limiting time the zone established a horizontal profile. The limiting time for the early wetting stage was found to be dependent on various soil properties as well as the footing size. Further trials and observations showed that the progression rate of the wetted zone was directly proportional to the intensity of infiltration and the initial moisture content but indirectly proportional to the relative density of soil. The researchers also measured the optimum inundated depth range in the soil for several cases tested and went further to develop a mathematical model for estimating it. Tests conducted by Abdrabbo and Abdelaziz (2006) are more realistic in replicating the actual field conditions. In dry areas such as the UAE case study locations where rainfall is very low, the only cause of significant infiltration to trigger soil collapse was the intense irrigation activities carried out on the landscapes.

Ayadat and Hanna (2007) used oedometer tests to study the stress-deformation response of a collapsible soil and formulated an empirical model for predicting soil collapse as a function of three variables namely the initial water content, the initial dry unit weight and the particle size distribution. The researchers tested the empirical model and found it to produce predictions that were in reasonable agreement with the experimental results as well as data from the literature.

Reznik (2007) utilized oedometer test data published by various researchers to develop equations for estimating the structural pressure (S_{sz}) as a function of the degree of saturation (S). The work led to conclusion that soil collapse starts when the applied stress exceeds the soil structural pressure values and that collapsibility occurs as a non-elastic deformation. A new parameter called “structural pressure value” was introduced and defined as separation ‘points’ between elastic and plastic states of any soil (including collapsible soils) under loading. Figure 2.2 illustrate graphs constructed by Reznik (2007) to show degree of saturation versus structural pressure (at various stress levels) at which the soil changes from elastic to plastic state in oedometer tests. The work led to development of a general relationship as shown in equation (2.1), based on logarithmic regression analysis.

$$S_{sz}(S) = C_0 + 10^{-CS+D} \quad (2.1)$$

where, C_0, C and D are coefficients



Results of Kane (1969) on Oakdale loess
$S_{sz} = 10^{-0.8711S+1.1990}$ (kN/m^2*10^2)

Results of Kane (1969) on Hawkeye loess
$S_{sz} = 10^{-0.6269s+0.9197}$ (kN/m^2*10^2)

Results of Kane (1973) on Oakdale Loess
$S_{sz} = 10^{-1.1547S+1.3030}$ (kN/m^2*10^2)

Results of Jasmer and Ore (1987) on Pocatello loess
$S_{sz} = 10^{-1.013S+0.8845}$ (kN/m^2*10^2)

Figure 2.2 Degree of saturation versus structural pressure obtained from oedometer tests on various soils (Reznik, 2007)

Reznik (2007) also suggested that geophysical methods could be used to determine the in-situ voids ratio and natural moisture content, both of which when combined with oedometer test results would enable development of correlations similar to equation (2.1) for predicting the structural pressure. Due to the obvious impracticality of extracting truly undisturbed samples, geophysical methods such as those suggested by Reznik (2007) can offer alternative means of determining various properties of the soil.

Soliman and Hanna (2010) conducted experimental investigation of a strip rigid footing resting on homogeneous and reinforced collapsible soils progressively saturated by the effect of groundwater rise. The primary aim of the experiments was to study the influence of reinforcements on the collapse settlement of a footing. Laboratory tests were conducted on homogeneous collapsible soil by partially replacing it with compacted sand with geotextile reinforcement layer at the interface. Also, additional tests were conducted after inserting geogrid reinforcement layer(s) within the compacted sand layer. Finally, an empirical formula was formulated in order to predict the collapse settlement of the strip

footing on homogeneous collapsible soil. This investigation conducted by Soliman and Hanna (2010) was closer to the field situation and so the proposed empirical formulae could be used on sites having similar conditions.

Wang *et al.* (2010) conducted consolidation tests and studied the effect of initial water content on collapsibility of loess soil obtained from Yuncheng in Shanxi province of China. It was concluded that with increase in initial water content, collapsibility of loess increases and collapsibility coefficient (δ_s) decreases. The collapsibility coefficient was defined as:

$$\delta_s = (h_p - h_p')/h_o \tag{2.2}$$

where,

h_p = height of the specimen at the initial water content

h_p' = height of the specimen after immersion in water and

h_0 = initial height of the specimen.

Finally, when the value of the initial water content was increased, scanning microscopy analysis showed that the voids area ratio decreased.

Research by Brink (2011) on the collapse phenomena of transported soil and residual soils in South Africa led to suggestion that the collapse process in a partially saturated soil can be evaluated in terms of effective stress and applied stress associated with suction. The collapse behaviour of the two soil types were examined in the laboratory in dry and partially saturated states. The change in suction pressure with changes in moisture content was first monitored as the collapse process developed. It was recognized that suction pressures of soils at low moisture content and degree of saturation would be considerably greater than the stress induced on the soil due to structural loading. With gradual increase in moisture content, suction pressures were found to decrease steadily up to the stage of attainment of a critical matric suction value. The tests revealed that the onset of collapse equated to a sudden decrease in voids ratio. Beyond the collapse stage, the applied stress was the predominant factor influencing the effective stress state of the material. It was also found that transported soils, which generally have high dry densities and low initial voids ratios in contrast to residual soils, undergo similar or greater voids ratio decrease compared to residual soils. This behaviour was attributed to remnant structure of residual materials contributing to strength. Finally, it was concluded that at same voids ratio, residual soils and transported soil do not exhibit similar behaviour; rather transported soils have greater probability of suffering reduction in voids ratio.

Based on laboratory studies, Kakoli and Hanna (2011) highlighted various causes of foundation failures attributable to sudden reduction in volume during saturation of a collapsible soil. The researchers analysed the influence of capillary force (i.e. matric suction) on the development of collapse due to saturation and recommended use of the pressure-settlement curve at fully saturated state of the soil rather than the natural unsaturated moisture condition. This approach was considered to be useful for practical assessment of safe load that a foundation bearing on a collapsible soil can safely support without failure. It was also recommended to apply this principle only for light weight structures since the downward thrust would be smaller during the collapse process of the soil. Owing to the invariably high permeability of collapsible soils, they require a relatively short period of time to become saturated when water is introduced into them from an initially dry state. Collapsible soils also require relatively lower amounts of water to reach 100% saturation, in comparison to other soils. Kakoli and Hanna (2011) also pointed out that the high saturation pressures in a deeply bedded stratum of a collapsible soil result in greater surface settlement when compared to a shallow stratum of the same type. The foregoing discussion of collapsible soil mechanisms and field conditions relate closely to the UAE sites comprising mostly dry soils and where light structures e.g. boundary walls and paved footpaths experienced distresses.

Rezaei *et al.* (2012) evaluated the reliability of the relationships depicted in Figure 2.1 which they successfully used to identify the collapse behaviour of a soil at the site of a project named South Rudasht Irrigation Network Channel, Iran.

Gaaver (2012) conducted a range of laboratory experiments on desert soils sampled from Borg-El-Arab, western Egypt, where ground settlement had caused extensive damage to various structures. Tests on disturbed and undisturbed samples facilitated identification of the nature of the soils and possible methods of ground improvement to deal with the settlement problem. Results were presented as graphs similar to Figure 2.1, where the region below the curve defines collapsible soils. Hence as gleaned from the plots, the data points (from results reported by Gaaver, 2012) strongly indicate collapse behaviour of the desert soils. Gaaver (2012) also carried out direct shear tests on soils with a view to predict the collapsibility of soils. These tests were done under an overburden pressure equivalent to a soil height of 1.50m, which corresponded to the foundation depths for most structures in the Iranian region. All tests were conducted in soaked and un-soaked conditions for undisturbed and compacted soil collected from different sites. From the analysis of the results, a new term called ‘reduction factor in shearing resistance (RFSR)’ was introduced and defined as the ratio of shearing resistance of soil in the soaked condition to that in the

un-soaked condition. The quantity RFSR was found to be a useful parameter representing the decrease in the bearing capacity of the soil at foundation level due to soaking process. For undisturbed samples, the initial moisture contents were adopted as the *in-situ* values. In contrast, for the compacted samples the initial moisture contents were taken as the values prior to the shear tests. The RFSR was found to increase with increase in initial moisture content, for both undisturbed and compacted soils. For collapsible soils in the natural state, the RFSR was found to fall in the range 0.43-0.58 (see Figure 2.3), with an average of 0.50. This means that the bearing capacity of the natural collapsible soil may be decreased to about 50%, and so the imperative recommendation is to double the factor of safety when designing foundations on collapsible soils.

One can use the above approach to assign a safety factor while analysing bearing capacity of such soil types. Accordingly, simple relationships between RFSR and initial moisture content (w_c) were developed as shown in equations (2.3) & (2.4) which can be used to estimate the reduction in bearing capacity. Tests for collapse potential (C_p) were carried out using the procedure proposed by Jennings and Knight (1975) and typical results obtained were as shown in Figure 2.4. From the results, equations (2.3) and (2.4) were formulated to relate RFSR to initial moisture content, for undisturbed and compacted soils.

For undisturbed samples, $RFSR = 1.5(w_c) + 0.34$ (2.3)

For 95% compacted samples, $RFSR = 2.15(w_c) + 0.40$ (2.4)

From the observed variation trends in Figure 2.4 the following relationships shown in equations (2.5) and (2.6) were extracted to express collapse potential in terms of the initial moisture content, for undisturbed and compacted soils:

For undisturbed samples, $C_p = 0.177 - 0.59(w_c)$ (2.5)

For 95% compacted samples, $C_p = 0.033 - 0.11(w_c)$ (2.6)

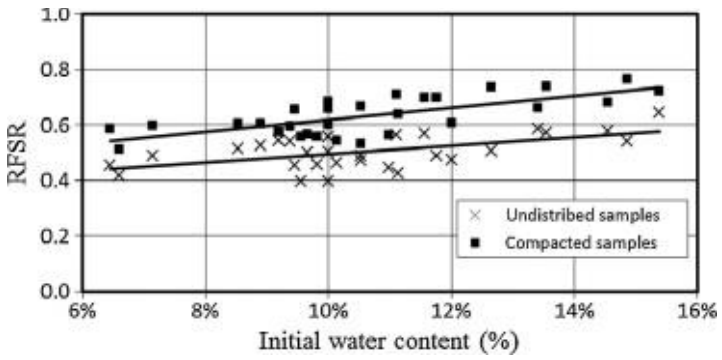


Figure 2.3 Reduction factor in shear strength at a depth of 1.50m below ground level versus initial water content (Gaaver, 2012)

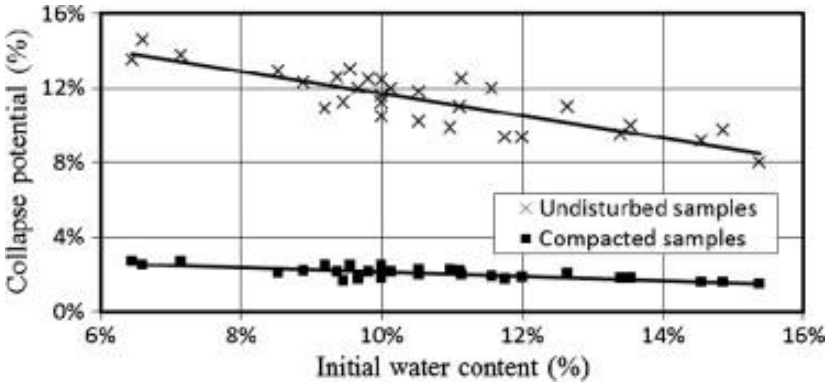


Figure 2.4 Collapse potential versus initial water content (Gaaver, 2012)

Mashhour and Hanna (2016) conducted extensive laboratory studies and suggested an analytical procedure to quantify the drag load on end bearing piles in collapsible soil due to progressive saturation. The researchers developed a chart for determining a factor R_c (reduction factor) as a function of saturation pressure. The value of R_c would then be multiplied by the conventional load carrying capacity of a pile in order to arrive at the drag load on the pile associated with saturation of the collapsible soil beneath the pile base.

Despite being promising, most of the works carried out by the aforementioned researchers are largely incompatible with the actual field situation where collapsible strata are in a state of stress and experiencing gradual saturation and groundwater table change due to water ingress.

2.3 Collapse characterization through field tests

Reznik (1993) conducted static field plate load tests on collapsible soils using rigid bearing plates (of area 0.50 m^2) at the center of the bottom of rectangular pits of size 1.8 m x 1.5 m. The test loads were applied using hydraulic jacks and settlements were recorded at maintained loads until the settlement rate decreased to 0.05 mm per hour subsequent to which the next load increment was applied. The pore pressure conditions developed during the plate load tests were deemed to be similar to the site studied by the author in south-western Ukraine where rapid increase of settlements occurred due to uncontrolled wetting of soils beneath existing structures. A parameter called the proportionality limit (P_{pr}) was defined to represent the maximum pressure corresponding to the linear part of the curve obtained from plate load test. Values of the P_{pr} obtained for collapsible soils were found to decrease with increase in water content. However, it was recognized that theoretically a minimum value of P_{pr} would occur at 100% saturation degree. It is useful to note that such situations will happen only if saturation due to undesirable sources of water is not eliminated immediately.

The author's viewpoint is that, underneath a normal structure, the degree of saturation of the bearing soil due to accidental wetting rarely exceeds 70-80%. Therefore the above described technique of load testing collapsible soils is considered acceptable for design purposes. This is reasonable since the degree of saturation calculated after conducting the plate load test including wetting was found to be always below 80%. Table 2.3 lists a number of parameters collated from the work of Reznik (1993).

Table 2.3 Properties of tested soils (Reznik, 1993)

Test	Moisture content (w) after test (%)	Degree of saturation (S) after test (%)	Proportionality Limit, P_{pr} (kN/m^2)
1	14.5	45	265
2	28.1	70	110
3	16.1	44	170
4	34.0	70	100

Houston *et al.* (1995) developed an in-situ test named 'downhole collapse test' (Figure 2.5) and conducted a series of tests at a site known to exhibit wetting induced collapse.

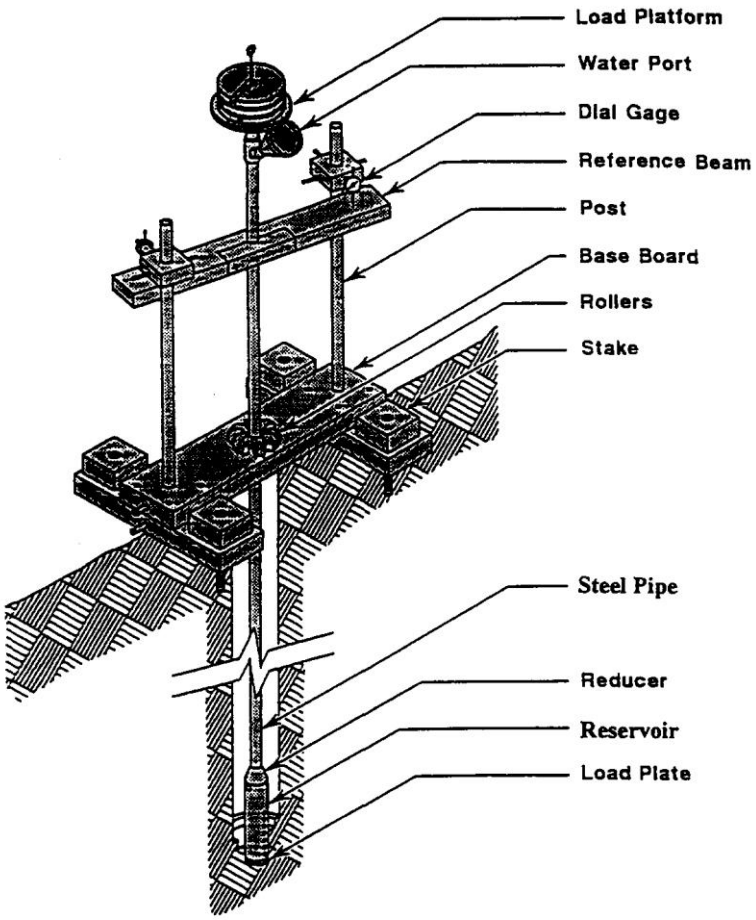


Figure 2.5 Down-hole collapse test system (Houston *et al.* 1995)

The test was conducted in a borehole with load being applied to the plate at the bottom of the borehole. Water was introduced in the test and load-settlement response of the soil monitored. The data from of all tests and equations developed thereof were used to determine the wetting induced collapse.

In addition, a full-scale load test was conducted on a footing of size 0.81 x 0.81 m embedded 0.46 m below the ground surface. Over a 10 hour period, 400 gallons of water was introduced by surface ponding. At the end of wetting process, the final settlement of the footing was measured to be 39.5 mm, which compared favourably with the value of 36.6 mm obtained from equations developed from down-hole collapse test results. It was also emphasized that while performing tests on collapsible soils, lab specimens could be subjected to higher degree of saturation than field soils, samples could be disturbed and soils such as gravels could be difficult to sample.

Souza *et al.* (1995) conducted field plate load tests on a site in Sao Paulo State, Brazil that consists of collapsible soil of more than 10 m in depth. The main aim of the study was to understand the effect of soil compaction in terms of reducing collapse settlement. Two brick walls of each 1.6 m height founded on strip footings 3.0 m long and 0.6 m wide were constructed on natural and compacted soil. To simulate the field conditions, both walls were loaded with additional surcharge and then wetted. Field test results showed that 87% reduction of settlement and about 110% increase in the allowable bearing capacity can be achieved due to compaction. However, when tested after wetting, soil compaction showed a reduction of settlement by 50%, whereas after the application of surcharge on the walls, the figure was 80% after wetting of the soil.

Lollo *et al.* (2011) have attempted to identify the collapsible soils through field electrical resistivity studies in Ilha Solteira, Brazil. Data obtained from electrical resistivity tests were compared with tests conducted on soil samples extracted from boreholes and test pits. It was also stated that though electrical sounding method was reasonable for locating the top surface of a collapsible soil stratum, the bottom surface was less clearly defined and hence more research was needed to extend the applicability of the method.

Freitas *et al.* (2017) observed that, despite suffering reduction in volume when saturated, collapsible soils in parts of Brazil possess some useful mechanical properties that make them suitable as construction materials for certain dams, embankments and road bases. The above researchers reported cases of utilization of compacted layers of collapsible soils under shallow foundations where soil columns of 250mm dia. were constructed down to

3.5m depth. Later the soil was excavated to form a 1.5 m deep pit, the bottom of which was subjected to a plate bearing test. Three different plate tests were performed:

(a) without soil column

(b) with compacted soil column and in-situ water content and

(c) with compacted soil combined with effects of inundation of soil with water

The plate test (c) above showed the greatest soil strength and there was no matric suction. Hence for this test there was a low reduction in saturation-induced ultimate stress in comparison test (b) where a matric suction of 13 kPa was inferred.

Although some of the aforementioned suggestions for identification and characterization of collapsible soils are plausible, the required tests tend to be too time consuming and uneconomic to be used for routine design purposes. Therefore there is merit in devising laboratory testing strategies that can be modified to create consistency with field conditions.

2.4 Finite element modelling of collapsible soils

Alonso *et al.* (1990) reviewed the main characteristics that influence the behaviour of partially saturated soils and concluded that existing models cover only limited aspects of the stress-strain response. The researchers developed an elasto-plastic hardening model for soil by defining two independent stress variables, namely: (i) the difference between total stress and air pressure and (ii) suction in the soil. The model considered the changes in the stiffness of soil induced by changes in suction and provided a way for estimating the quantity of collapse. Thus the model produced irreversible response against stress and suction reversals.

Balmaceda *et al.* (1992) also advanced a model using an elasto-plasticity approach but with capability of reproducing the principal characteristics of the behaviour of non-expansive partially saturated soils. The main feature of the model was its capability to estimate the maximum collapse usually exhibited by partially saturated soils. Through the model, it is possible to predict hyperbolic relationships between voids ratio and suction. Results from the model were compared with experimental results and showed good agreement.

Habibagahi and Mokhberi (1998) investigated the relationship between bulk modulus and water content by measuring volume changes in isotropic compression tests conducted on unsaturated compacted soil specimens. Using a finite element analysis, the researchers proposed a hyperbolic representation for volume change of the compacted soil. Tests were

also performed on soil specimens with moisture contents between 12% and 16% and the results used to validate the finite element model derived for estimating collapse.

Kakoli *et al.* (2009) formulated a finite element model to simulate collapsible soil in three different states namely before, during and after inundation. The model was envisaged as a tool for analysis of foundations in collapsible soil, with consideration of the decrease in suction due to inundation. The suction decrease was recognized as influential in the soil stress state and the irreversible volume changes that occur in the saturated soil. Fundamental theories of unsaturated soil mechanics were used in the model which also took into consideration the consequences of wetting on the properties of the collapsible soil. This work seems more practical and consistent with the circumstances that triggered the failure of shallow founded structures as observed from UAE case studies.

Rotisciani *et al.* (2015) examined the behaviour of a partially saturated soil during surface water infiltration though elasto-plastic constitutive model based on effective stress and extended to unsaturated conditions. The results of the model were compared with large scale experimental results obtained from oedometer and triaxial tests under suction controlled conditions. A close agreement was found between the results of the constitutive model and the laboratory results.

Noor (2017) suggested that ingress of water into collapsible soils can cause negative skin friction in piles due to the collapse process around the pile shaft. It was also pointed out that several factors influenced the collapse mechanism, rendering the use of consolidation or liquefaction concepts inappropriate or unreliable. Instead, a numerical axi-symmetric finite element model was formulated considering most of the crucial factors that influence the kinematic down-drag on the pile.

As seen above, most numerical approaches on collapsible soils were based on elasto-plastic approach and results obtained were verified using triaxial tests. In minimizing dependence on undisturbed samples, in the current research finite element analysis using *Midas*^{TM} *GTS NX* professional software (Midas, 2014) was used to analyse the full scale model of a twin-villa complex (from a case study in UAE, section 3.3) and results compared with actual distresses observed on site.

2.5 Improvement of collapsible soils

Holtz and Hilf (1961) reported some case studies in the United States of America where collapsible soils were improved using a pre-wetting technique prior to construction of structures. Two of the techniques are described below:

i. Collapsible loess on one side of the Medicine Creek Dam in Nebraska was loose and risked settlement and so it was decided to pre-wet the whole dam foundation area before placement of a fill. In order to check the effectiveness of the process, four base plates were installed to measure settlements, which were found to vary from 240 mm to 600 mm.

ii. In advance of the construction of a canal, saturation ponds were built on one side of San Joaquin Valley of California, in order to assess collapsibility behaviour of the soil subjected to moisture movement. Seepage from the pond caused collapse of the soil structure leading to a settlement of about 3 m in 2 years after which construction of the canal was commenced.

To protect buildings and bridges Houston *et al.* (2002) suggested a way of dealing with collapsible soils by either removing or replacing the top 1-2 m depths of the soils, for relatively small areas involved in the improvement. However, for highway projects, the researchers recognized that the process would be uneconomic due to large areas covered. Instead alternative methods such as chemical stabilization, grouting and dynamic compaction (Ménard and Broise, 1975; Rollins and Rogers, 1994) could be used along with pre-wetting (Gibbs and Bara, 1967; Al Rawas, 2000). Although soil compaction was suggested by a number of researchers (Chin, 1988; Choudry, 1988; Vargas, 1988; Souza *et al.* 1995 and Otálvaro *et al.* 2015) as a successful technique for reducing the settlement of collapsible soils, it is clear that the extent of the improved zone would be further enhanced using dynamic compaction. However, like in the UAE case studies where collapsible soils were identified long after infrastructure construction, dynamic compaction would not be feasible because of its adverse effects on the existing nearby structures. In such case either pre-wetting or chemical stabilization offers a safer alternative as a means of ground improvement.

Despite the several improvement alternatives proposed by Houston *et al.* (2002), as a general rule of thumb, the response of a collapsible soil to wetting should be assessed based on laboratory tests or field tests. It important to recognize that soil collapse does not always require full saturation to be attained, as happens in an oedometer test. Lessons learnt from the UAE case studies showed that the triggering factor in soil collapse was the

cyclic irrigation of landscapes which gradually wetted the underlying collapsible strata. Therefore extensive efforts are made in this research to simulate the irrigation effects in a small scale laboratory model, validated numerically from a finite element approach.

Ayadat and Hanna (2005) suggested that wherever collapsible soils are encountered, as an alternative to conventional deep foundations, stone columns encapsulated in geofabric reinforcement could be used for transmitting foundation loads to suitable bearing strata below the collapsible soil layer. They conducted extensive experimental investigation to understand the performance of stone columns encapsulated in geofabric installed in a collapsible soil layer and subjected to inundation. The load carrying capacity of the stone columns along with their settlement behaviour was investigated. Variables in the research included length of stone columns, level of soil saturation and strength of geo-fabric inserted. It was noticed that unreinforced sand columns in collapsible soil did not contribute considerably to the performance of the soil and there was premature failure of the stone. The load carrying capacity of the columns increased with increasing strength of the geo-fabric material. In addition, due to increase in column rigidity, there was a marked reduction in the effect of the external loading and saturation on the settlement of the column head.

Abbeche *et al.* (2010) conducted laboratory experiments to verify the possibility of increasing the mechanical resistance of the soil against collapse. With the aid of oedometer tests, a formula (equation 2.7) was developed for the collapse potential, C_p, of the soil at different compaction energies and at different concentrations of salt (ammonium sulphate and potassium chlorides) used to treat the soil. The salt concentrations applied were 0.5, 1.0, 1.5 and 2.0 moles per litre.

$$C_p(\%) = (\Delta e/e)x10 \tag{2.7}$$

where,

C_p = Collapse potential

Δe = Variation of the voids ratio between the dry sample and the saturated sample

e_o = Initial voids ratio of the sample

The work led to a conclusion that the mineral salts were effective in reducing the soil collapse potential. In particular, at 1.5 moles per litre concentration, KCl was found to be more effective compared to $(NH_4)_2SO_4$ and reduced the rate of collapse by about 60%. It was also suggested that, for a collapsible surface layer thickness less than 4 m, it was economical to excavate, treat the soil with salt solution and re-compact it. However, for greater layer thickness, the best treatment method involved injection of saline solutions.

Despite the impressive success, it was the author's opinion that the effectiveness of the above treatment will reduce if significant leaching of the salts take place. Another observation here is that the above study was silent on durability of the proposed salt treatment.

Fattah *et al.* (2012) examined the suitability of a dynamic compaction process for a site in Iraq where a soil deposit experienced collapse due to water entry and dissolution of gypsum present in the soil. Laboratory tests were undertaken for three soils containing different gypsum contents of 60.5, 41.1 and 27%. Compaction, collapsibility and compressibility characteristics were studied before and after treatment by small scale dynamic compaction process under different number of blows, falling weights and heights of fall. Out of all trials made, the best improvement in compressibility was achieved when the sample was compacted with 20 blows. Above this number there was only a negligible decrease in the compression index of the soil. Also, as the gypsum content increased, the dynamic compaction had a greater effect on improving the compressibility of the soil. As for the compaction effort, when the hammer drop height increased the compression index also decreased. Though dynamic compaction could be considered as a general alternative for improving collapsible soil properties, it could not be used in the UAE case study areas due to close proximity to existing structures.

Mohamed and Gamal (2013) attempted to improve a collapsible soil by treating it with sulphur cement. Normal and modified sulphur, fly ash, and soil aggregates were used to prepare the collapsible soil specimens which were then treated in air, water, and saline solutions at different temperatures (room temperature up to 60°C). All tests were done with a curing period ranging from 28 days to one year. Upon treatment, compressive strengths of the specimens were measured. The results showed a three-fold increase in the soil strengths compared to samples treated with ordinary Portland cement. The hydraulic conductivity of the treated soils was found to range between 1.46×10^{-13} and 7.66×10^{-11} m/s hence the material could be potentially used as a stabiliser for arid soils.

Fagundes *et al.* (2015) carried out an experimental investigation to assess the possibility of improving the collapse behaviour of lateritic soil using rice husk ash (RHA). Addition of RHA was found to reduce the collapse potential of the soil by about 74.4%.

Fattah *et al.* (2015) conducted laboratory studies on four types of gypsiferous soils with different properties and gypsum contents. Compressibility characteristics were tested on undisturbed soils under different conditions. All samples were grouted with acrylate liquid. Results showed a reduction in soil compressibility by more than 60-70% due to

formation of an acrylate liquid film coating on the gypsum particles, which were therefore isolated from the effect of water.

Moayed and Kamalzare (2015) conducted extensive laboratory investigation on collapsible soils near Tehran-Semnan railroad tracks in Iran at a section where wide cracks had formed (Figure 2.6). In trying to improve the ground, however it was impossible to block the railroad due to heavy traffic. Thus, it was decided to use injection method as a ground improvement option. Lime, cement and micro silica slurries were injected into the ground by drilling boreholes, from where samples were collected after 28 days after injection. Consolidation tests were carried out on collected samples and collapsibility potential calculated in accordance with ASTM D5333 (2003). When compared with values obtained with natural soil (before improvement), it was found that collapse reduced by about 70%, 63% and 40% for samples injected with lime, cement and micro silica respectively. In addition, shear strength of soil samples were tested in triaxial apparatus for a consolidated-undrained condition. Considerable enhancement in the coefficient of internal friction of the soil was observed in all the chemical-injected samples.

Figure 2.6 Vertical crack near the railroad in Semnan plain (Moayed & Kamalzare, 2015)

Ali (2016) conducted tests on collapsible soils at a site situated in Borg El Arab near Alexandria, Egypt. The soils were known to have a high susceptibility to collapse when saturated. In order to suppress the effects of the collapsibility of the soils, tests were performed to investigate saturation effect on the collapse potential and permeability behavior of the soils. The collapse phenomena of the soils were found to result in low bearing capacity and rapid settlement, which rendered the soils unsuitable for foundations in their natural condition. To reduce the collapsing nature of soil, a treatment method involving removal and replacement of the problematic soils was recommended. A series of oedometer tests were carried out to search for the most suitable types of partial replacement and the location of source of surface wetting to evaluate their effects on the reduction of settlement of a footing on collapsible soil due to inundation. Results showed

that inundation stresses had a considerable effect on collapse potential and permeability coefficient. It was found that removal and replacement of collapsible soil with cohesionless soil over a certain depth zone enhanced the stability of the collapsible strata. Such modification resulted in settlement reduction by 50% and improvement in bearing capacity by about 80-100%. However, it should be noted that removal and replacement technique would be suitable for small scale projects and up to limited depths only. In the context of the UAE case studies, removal and replacement could be cumbersome because of the dense concentration of occupied villas and buildings.

Iranpour and Haddad (2016) studied the influence of various nano-materials on the collapse potential of soils. Soil samples collected from test pits in a sub-tropical area of Iran were tested for collapsibility in accordance with ASTM D5333 (2003). The influence of various additives e.g. nano-clay, nano-copper, nano-alumina, and nano-silica was studied at different percentages of total dry unit weight. Tests were conducted at natural water content and density. It was found that the most influential property of the nano-material was specific surface area. Nano-clay with a high specific surface area than nano-silica was found more effective in reducing the collapsibility of the soil. Similar improvement was noticed with nano-copper and nano-alumina. The successes notwithstanding, it was noted that use of the nano-materials in wrong percentages could cause agglomeration of particles and lead to a negative effect on the mechanical properties of the soil. In order to overcome such negative consequences, it was therefore suggested to combine them in soil in the form of colloidal solutions.

Arabani and Lasaki (2017) conducted oedometer tests on simulated collapsible soil specimens classified as low plasticity silts as per Unified Soil Classification System. XPS (extruded polystyrene)-cement mixture was used as an additive to improve the soil by reducing its collapsibility. XPS-cement was added in different percentages ranging from 1% to 8% and collapse potential calculated. The collapse potential of the soil started to decrease with addition of 2-3% of XPS-cement. Although there was improvement in collapse resistance beyond 3% XPS-cement content this was not with regard to reduction in collapse potential but rather decrease in voids ratio. Evidence of this was manifest in the results of scanning electron microscope (SEM) tests performed. However, an XPS-cement content of 6% was considered as a maximum percentage to produced meaningful reduction in collapse potential. The above experiments were performed at different vertical stress levels and it was noticed that collapse potential decreased with increase in vertical stress, so that at stresses exceeding 750 kN/m^2, the state of soil changed to non-collapsible.

Chemical stabilization, as a plausible improvement method for collapsible soils, has been mooted by several researchers including Sokolovitch (1971) and Mitchell (1981). Ayeldeen *et al.* (2017) carried out an extensive experimental investigation to assess the possibility of adding biopolymers to enhance the mechanical properties of collapsible soils. Due to being readily available and cost-effective, two types of biopolymers, namely xanthan gum and guar gum were suggested. The study by Ayeldeen *et al.* (2017) concentrated on assessing the compaction characteristics, shear parameters and collapse potential of soil. After addition of the biopolymers, the above tests were carried out on samples cured for two different periods i.e. zero and seven days. Shear parameters were measured both in un-soaked and soaked conditions, whereas collapse potential was measured under different mixing conditions (wet and dry mix). The dry unit weight was found to fall from 19 kN/m^3 to 17 kN/m^3 and optimum moisture content increased from 12% to 14.6%. Results showed that collapse potential was reduced to 1% from 9% for 2% content of both biopolymer types. It was noted that wet mix had a more positive effect on collapse resistance than the dry mix. The improvement in shear strength was 30% greater for the mix comprising guar gum compared to that having xanthan gum.

Many researchers in the past have proposed improvement of collapsible soils by grouting (using different materials) techniques. A grouting trial was made in one of the case study (low-rise housing project, section 3.3) in UAE and limited improvement was noticed. Thus, instead of grouting, pre-wetting method was recommended as ground improvement method for improving collapsible soil.

2.6 Prediction of soil collapsibility

Many researchers in the past had suggested different formulae / conditions / approaches to predict the collapsibility of soil using simple laboratory tests and most appropriate ones are narrated below.

Abelev (1948) proposed the following equation for measuring collapsibility (Δ_s).

$$\Delta_s = [\frac{\Delta e}{(1+e_1)}]x100 \quad (2.8)$$

where,

Δ_e = voids ratio reduction during soil saturation

e_1 = voids ratio before soil saturation

The work led to a suggestion that a soil is collapsible if the value of Δ_s exceeds 2 %.

Denisov (1951) proposed the following equation to estimate the coefficient of subsidence, based on which collapsibility of soil can be assessed:

$$\text{Coefficient of subsidence, } K = e_L/e_0 \tag{2.9}$$

where,

e_L = voids ratio at liquid limit
e_0 = voids ratio before the saturation of soil or natural water content

The criteria to identify the collapsibility is based on coefficient of subsidence is given in Table 2.4.

Table 2.4 Collapse criteria by Denisov (1951)

Value of K	**Collapsibility**
0.50-0.75	Highly collapsible soil
1.00	Non-collapsible loam
1.50-2.00	Non-collapsible soil

Clevenger (1958) proposed that once a soil is confirmed as collapsible, the severity of its collapse depends on the dry unit weight γ_d such that γ_d < 12.6 kN/m^3 implies large settlements whilst γ_d > 12.6 kN/m^3 means small settlements.

Gibbs and Bara (1962) predicted that a soil is prone to collapse if the water volume at saturation, W_{max}, exceeds the water volume at its liquid limit, LL. Furthermore a criterion was proposed that LL / $W_{max} \leq 1$ means a soil is collapsible. Later Densiov (1963) suggested a parallel condition that e_0/e_L>1 implies a soil is collapsible.

Feda (1988) provided a parameter, K_L for predicting the soil collapsibility:

$$K_L = \left[\left(\frac{w_n}{S}\right) - PL\right]/PI \tag{2.10}$$

where,

w_n=natural moisture content or moisture content before saturation
S = Degree of saturation
PL=Plastic limit
PI=Plasticity Index

For S<100 %, if K_L>0.85, the soil is considered collapsible.

Handy (1973) recommended that collapsibility could be calculated using the percentage of clay fraction in soil as mentioned in Table 2.5 below.

Table 2.5 Collapse criteria by Handy (1973)

Clay content	**Collapsibility**
< 16%	Highly collapsible soil
16 to 24%	Possibly collapsible
24 to 32%	Possibly less than 50% collapse
> 32%	Non-collapsible soil

Zur and Wiseman (1973) proposed the following criterion for predicting the collapsibility of soil.

If $\rho_d / \rho_{dL} < 1.1$, the soil is collapsible in nature.

where,

ρ_d = dry density of the soil at natural moisture content

ρ_{dL}=dry density of the soil at liquid limit

Lin and Wang (1988) proposed the following equation and criteria for predicting the soil collapsibility (I_{cz}).

$$I_{CZ} = (h_z - h_{cz})/h_1 \tag{2.11}$$

where,

h_1 = initial soil sample thickness

h_z = soil sample thicknesses in odometer test at overburden pressure in natural condition

h_{zs} = soil sample thicknesses in odometer test at overburden pressure in saturated condition.

Once I_{cz} is determined, collapsibility is assessed as per the criteria given in Table 2.6.

Table 2.6 Collapse criteria according to Lin and Wang (1988)

Value	**Collapsibility**
$0 < I_{cz} < 1$	no collapsibility
$1 < I_{cz} < 5$,	medium collapsibility
$5 < I_{cz} < 10$	high collapsibility
$10 < I_{cz} < 20$	very high collapsibility
$I_{cz} > 20$	extremely collapsible

Bell (2004) stated that collapsibility of a soil can be determined based on the ratio of liquid limit to saturation moisture content. Values of the ratio less than unity imply a collapsible soil whilst greater ratios mean a non- collapsible soil.

Xie *et al.* (2018) stated that collapsible soil undergoes the following 3 different phases in response to matric suction due to wetting: (a) pre-collapse phase, (b) collapse phase and (c) post-collapse phase. Based on soil water characteristic curve (SWCC) a method was proposed for predicting the soil collapse due to wetting. The method requires use of two parameters, namely critical suction and collapse rate.

Li and Vanapalli (2018) observed that the variation of voids ratio derived from wetting tests on collapsible soils is similar to that of water content in response to decrease associated with matric suction. Li and Vanapalli (2018) also extended van Genuchten's equation for SWCC to fit a trend line for the laboratory measured data on collapse to explain the change in voids ratio with matric suction. In this process, an equation was

developed with new curve fitting parameters and tested for consistency with data obtained from conventional collapse tests. Good agreement was observed between predicted and measured collapse magnitudes. A notable advantage of this method is that it requires less experimental data and is relatively easy to use analysis.

Though many alternative and simple methodologies were suggested by various researchers above, it is always advisable to verify their applicability by comparing with results of standard collapse tests before applying to specific sites.

2.7 Case studies on collapsible soil

A large number of published case studies have been identified and explored but for brevity, only the most relevant sets of them are reviewed here.

i. Holtz and Hilf (1961) stated that best examples of structures experienced distresses due to wetting of loessial soil are found in mid-western and in parts of the north-western United States of America. In these areas wind deposited loess soils in very loose state are found. Though plenty of case studies were reported the authors, a few are briefly narrated below.
    a) At Levant, Kansas, a grain elevator was found tilted due to wetting and collapse of loess underneath the structure. Rather uniformly settled, the grain elevator was tilted due to the fact that the surface runoff ponded on one side and collapsed the soil.
    b) In Columbia Basin Project, Washington, a waste-water chute structure founded on a silty soil has failed. During investigation, it was found that the root cause is the collapse of soil after water was introduced into the system.

ii. A commercial building in semi-arid New Mexico won an award from the city as the year's most beautiful lawn and landscaping. However, it experienced foundation damage owing to differential settlement due to wetting of collapsible foundation soils underneath (Houston *et al.* 2001). To prevent possible instability due to the underling collapsible soils, a number of solutions were suggested by Houston *et al.* (2001). These includes: (i) removal of volume moisture-sensitive soil, (ii) removal and replacement or compaction of collapsible soil, (iii) avoidance of wetting, (iv) chemical stabilization or grouting, (v) pre-wetting, (vi) controlled wetting, (vii) dynamic compaction, (viii) pile or pier foundations and (ix) differential settlement resistant foundations. Among the above, the most suitable option would then be selected

based on further considerations such as availability of resources, effects on nearby structures, construction economics and safety issues.

iii. Houston *et al.* (2002) presented a number of case studies in the USA where pavement and bridge foundations had experienced distresses due to collapsible soils underneath. The distresses included excessive pavement waviness noticed on Interstate 10 near Benson, Arizona, sections of Interstate 25 in the vicinity of Algadonas, New Mexico (Lovelace *et al.* 1982) and settlement of bridge foundations in Steins Pass, Arizona (Russman, 1987).

iv. Noutash *et al.* (2010) reported that mitigation of collapse risks through impoundment of the Khoda Afarin canal in northern Iran led to large cracks on the berm following completion of the pre-treatment technique.

v. Farawan and Majidzadeh (1988), Al-Abdul Wahhab and Ramadhan (1990), Al-amoudi *et al.* (1991), Aiban (1994), Al-Amoudi (1994), Aiban *et al.* (1995), and Moosavi and Kalantari (2011) summarised various problems caused by collapsible soils to highway pavements. Examples cited were:

    a) Formation of depressions and settlement due to decrease in volume of soil due to entry of water.
    b) Higher collapse potential in soils with high salt content, a phenomenon that was attributed to dissolution of salts as water flowed through open voids.
    c) Differential settlements variations in the depth location and thickness of the collapsible strata below the pavement.

vi. Rollins and Kim (2010) suggested dynamic compaction (DC) as a cost-effective method for mitigating the hazard associated with wetting of collapsible soils, especially lying at depths exceeding 3-4 m below the surface. The above authors presented the following case records where dynamic compaction was used as treatment for collapsible soil:

    a) A test section and full-scale project on Interstate-90 between Whitehall and Cardwell, Montana.
    b) A test section and full-scale project on Interstate-25 near Algodones, New Mexico.
    c) A test section and full-scale project at a state prison near Avenal, California.
    d) Two test sections and three full-scale projects on Interstate-25 between Kaycee and Buffalo, Wyoming.

Some difficulties were experienced during the dynamic compaction process, namely;

a) Excessive crater depths owing to high moisture contents.
b) Insufficient densification in very dry soils.
c) Poorer compaction in the near surface layers.
d) Reduced effectiveness where clay layers existed within the soil profile.

vii. Kalantari (2013) reported a forensic investigation in San Diego, California, where increased level of precipitation resulted in substantial settlements of the underlying compacted fill.

In all the above mentioned cases, the reason for collapse of soil was due to water ingress through whatever means. This is similar to the UAE cases, where the ingress was due to irrigation processes that eventually led to settlement and damage to shallowly founded structures, as illustrated in chapter-3. The problem did not apply to villas or buildings supported by deep pile foundations extending beyond collapsible strata and into the bedrock.

CHAPTER 3 - CASE STUDIES IN UNITED ARAB EMIRATES

3.1 Introduction

In the recent past, some developed parts of the United Arab Emirates (UAE) have been severely affected by ground settlement and consequent infrastructure damage. The relevant authorities have acted on the problem by engaging specialist geotechnical companies to carry out a full investigation of the circumstances and to recommend practical solutions. Within the framework of data obtained from the geotechnical investigation companies, two case studies from the affected locations are described and analysed in this chapter. For client confidentiality reasons and to abide by the conditions under which the investigation data was released, the project locations and names and addresses of the various parties involved are not disclosed in this thesis. In both case studies, it emerged that the settlement and structural distresses were strongly linked to the effects of lawn irrigation on underlying collapsible strata, which had not been properly considered during the design and construction of the infrastructures. The discovery was convincing because distresses were only observed in light structures such as boundary walls, footpaths and pavements exerting load on the superficial deposits of collapsible soil. No distress was suffered by piled structures where load was transferred past the collapsible strata and into rock further below. The site investigations also monitored and reported the magnitudes of damages to the structures and mapped out the actual irrigated areas. Detailed explanations on the two case studies are presented in the following sections.

3.2 Large guest house project in Al Ain city, UAE

This project, which is situated in Al Ain city in UAE, was developed with a large guest house structure. More than 85% of the site was covered with landscaped gardens and terraces exceeding 15000 m^2 of lawn, and the total garden area lies on a 12 m thick fill of topsoil. The total area of fill is surrounded by a two-step precast gravity-retaining wall structures (Figure 3.1), which deformed due to irregular settlement of the ground below. It was revealed that the settlements that occurred here were due to the effect of percolation of irrigation water into collapsible strata existing at depth. Fortunately, the actual guest house structure had been built on piles extending down to rock head and did not experience any distresses. Initially when settlements were noticed, repair works were carried out in order to keep the structures serviceable. However, structural damage continued even after completion of remedial works. Before any irrigation took place, no settlements were noticed either during the placement of fill or landscaping works.

However, soon after the start of irrigation activities, within 8 months, surface settlements and associated structural distresses were observed.

Figure 3.1 Two-step precast gravity retaining wall structure

Although structural distresses were observed in several locations at this site, only a selection of them is included below for brevity.

(a) Kerbstones adjacent to landscaped areas were separated from the walkways by approximately 40 mm. (Figure 3.2)

Figure 3.2 Separation of edge kerbstones due to settlement of adjacent green area

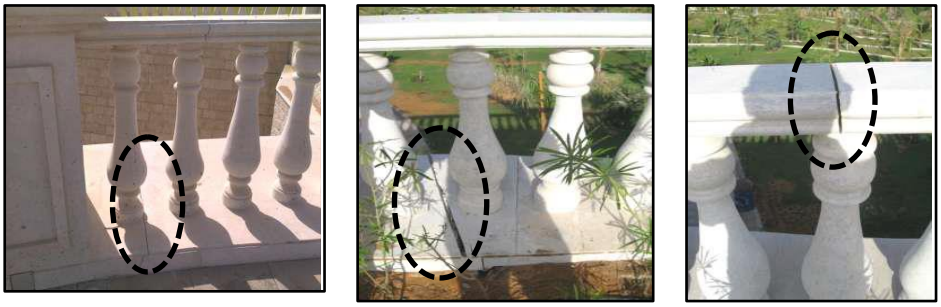
(b) Staircases and steps near the landscaped areas of the guest house had subsided, whereas the actual structure founded on piles did not (Figure 3.3).

Figure 3.3 Separation of stairs from adjacent wall due to differential settlement

(c) Large settlements (approximately 80-100 mm) were measured in concrete paved areas situated close to the landscaped zones. (Figure 3.4)

Figure 3.4 Ground subsidence beneath pavement slab

(d) Localized cracks occurred due to differential settlements of parapets at the edge of retaining wall (Figure 3.5).



a) Cracking in coping b) Opening of joints c) Crack in rails

Figure 3.5 Cracks noticed in parapets

Settlements at the site were measured to be in the range of 20–30 mm on the low side and 80–100 mm on the high side. The contracted geotechnical investigation company drilled a total of 12 boreholes (10 boreholes of 15 m deep and 2 boreholes of 20 m deep) along with 4 test pits down to 2 m below the ground. In addition, the following field tests were carried out at specified locations.

(a) Standard Penetration Tests (SPT)

While borehole drilling and SPTs were in progress, soils were sampled and their consistency (or relative density) measured at every 1.0 m depth intervals. The SPT provided number of blows (N) of a 140 lbs. (63.5 kg) standard hammer free-falling from a standard drop height of 30 inches (760 mm) drive a standard split spoon sampler into the soil a distance of 12 inches (305 mm). The blow count number (N-value) recorded at various borehole locations and depths were used to determinate the stratigraphy of the site. The main use for the SPT was to enable correlation of *in-situ* relative densities at depths of interest so that any potentially uniquely weak strata could be identified.

(b) Mackintosh probe tests

Since it was impractical to deploy SPT and borehole drilling at frequent points and especially in close proximity to existing structures, Mackintosh Probe Tests (MPT) were conducted to complement data from SPTs. The MPT also enabled determination of the *in-situ* density profiles from the surface down to a few meters below ground level. The MPT was seen as a rapid yet simple manually operated test appropriate for the site conditions. The equipment consisted of a system of rods connected to a standard cone designed to reach appropriate depth with a sliding driving hammer attached to one end. The test procedure involved lifting the hammer to the full height of travel and allowing it to fall freely. The cumulative number of blows required to drive the tool through every foot (30 cm) of soil was interpreted as a measure of the consistency of the soil. If the number blows required to drive the tool for 30 cm exceeded 50 then this was regarded as the refusal stage hence the test was terminated.

(c) Permeability tests

Taking into account the non-cohesive nature of the site soils, constant head permeability tests were conducted in selected boreholes and additionally soakaway tests were conducted in test pits.

i. Constant head permeability tests

   In this test, water was pumped into the borehole at different pressures and corresponding flow measurements recorded. For every pressure stage, flow rates were recorded at selected intervals of time to allow permeability to be calculated

accordingly. Constant water head was maintained by continuously adding water in the casing inside the borehole to replace the water leaving.

ii. Soakaway tests

The purpose of this test was to evaluate the permeability characteristics of the top soil in particular. A pit of 30x30x30 cm in size was excavated at the bottom of a test pit of measuring 60x60x60 cm. The inner pit was completely filled with water and allowed to seep for 24 hours. If water completely seeped through then the inner pit was re-filled with water to a depth of 25 cm and allowed to continue seeping for another 24 hours. The test was repeated for at least 4 days and at any stage on the next day after filling, if water remained in the pit, the test was considered completed.

Based on the soil layers obtained in different boreholes and the observed SPT blow counts, an indicative stratification profile representing the general site is interpreted as shown in Table 3.1. The average permeability of the soil attained from field permeability tests was found to be of the order of $6{\cdot}83x10^{-7}$ m/s, which typifies soils with high silt content.

The criteria provided by Bell (2000) for assessing the severity of collapse are summarized in Table 2.2. Collapse potential tests carried out on soil samples from the test pits are shown in Table 3.2, and the magnitudes obtained indicate that the soils are susceptible to collapse and the severity is characterized as 'very severe trouble' (Bell, 2000).

Table 3.1 General stratigraphy of the guest house site

Depth (m)	**Description of soil**	**Range of SPT N-Value**	**Relative density (based on SPT)**
0.0-1.0	Silty SAND (agricultural soil as fill material)	3-19	Very loose to medium dense
1.0-13.0	Silty GRAVEL / Gravelly SILT (fill material)	3-30	Very loose to medium dense
13.0-15.0	Silty SAND (dune sand)	32-50	Dense to very dense
15.0-19.0	SILT (alluvial soil)	37-50	Dense to very dense
19.0-20.0	Silty GRAVEL (residual soil)	>50	Very dense

Table 3.2 Collapse potential test results

Test pit no.	**Depth (m)**	**Collapse potential (%)**
2	0.50	64.5
3	1.20	86.4
4	1.85	86.7

Grouting was initially mooted as a possible ground improvement method to reduce the voids in the soil mass. However, given the vast area of the affected ground to be improved, the above idea judged to be uneconomic. Therefore, after examination of other alternatives, hydro-compaction was a chosen as the solution that gave the best balance between cost-effectiveness and technical suitability. Even then, it was still borne in mind that the process would expose existing structures to vibration risks, so it was proposed to deploy hydraulic jacks to temporarily underpin some of the structures (e.g. gazebos and swimming pool structures) during the process of hydro-compaction. Once the process was completed and ground settlements ceased, it was recommended to inject cement grout through any residual gaps. This was to ensure that the bases of the lifted structures were once again in proper contact with the ground.

3.3 Low-rise housing project in Abu Dhabi, UAE

A low-rise housing development comprising villas, community buildings, amenity buildings, and open green areas was undertaken in Abu Dhabi (UAE). The project included a network of sector roads that traverse the whole development and connects to the regional highway system. After the completion of construction and during the first year of occupation, signs of distresses due to excessive settlements started appearing in parts of the development. Incidentally, there were no signs of distress in large structures such as villas and community buildings that were supported on pile foundations. The affected areas were predominantly shallowly founded structures such as footpaths, roads, boundary walls etc. Specific types of distresses observed in various selected structures are described below.

(a) Paved areas at several locations adjacent to landscaped areas (Figure 3.6) experienced settlements of approximately 75 mm due to continuous infiltration of irrigating water into underlying collapsible strata.

(b) A number of boundary walls experienced damage but the walls located either side of the landscaped areas (Figure 3.7) suffered the greatest settlement, up to 165 mm.

(c) Flexible pavements adjacent to open landscaped areas (Figure 3.8), experienced settlements of approximately 100 mm. Since load transfer beneath pavements are generally limited to depths of 2·0–2·5 m, there was immediate suspicion that loose soils susceptible to water-induced collapse were present at shallow depths. This was later confirmed from boreholes drilled as part of the geotechnical investigations, where very low SPT blow counts were observed at shallow depths of 1·0–1·5 m.

Figure 3.6 Subsidence in paved areas (footpaths)

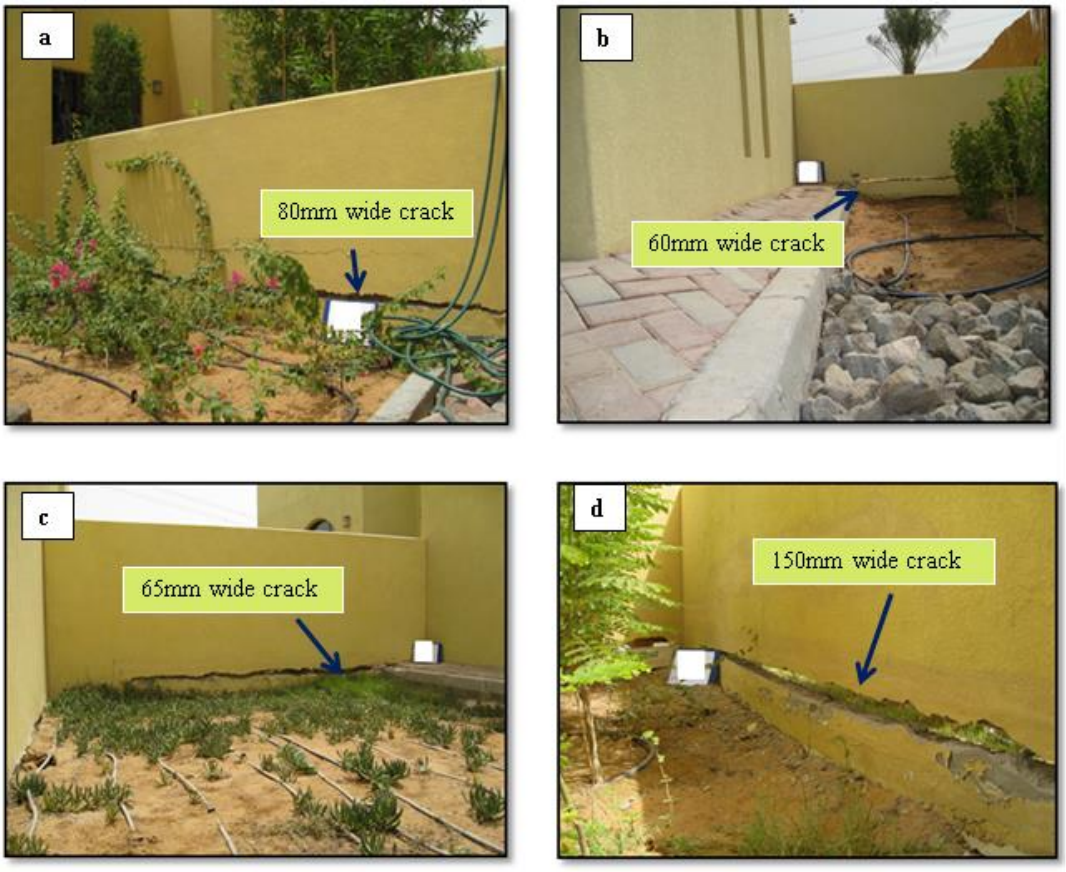


Figure 3.7 Cracking and settlement of boundary walls

Figure 3.8 Sagging of road

(d) Interestingly, open green landscaped areas with no structures (Figure 3.9) were also found to have subsided (approximately 150 mm). This caused some doubt as to whether all settlement observed was attributable to irrigation water seeping down to collapsible soils at depth.

Figure 3.9 Settlement of ground within a landscaped area

To investigate this further, the appointed geotechnical company drilled two deep boreholes to a depth of 15 m in close proximity to the area of interest. The boreholes revealed a 1·5–2·0 m thick layer of topsoil and based on recorded SPT blow counts, this layer was interpreted to be very loose to loose. Groundwater table was encountered at a mean depth

of 1·5 m below ground surface. Under these circumstances, to verify how the top loose soils responded to the presence of irrigation water, some open landscaped areas were selected and flooded with water (hydro-compaction) for 15 days to seep through the soil (Figure 3.10).

Figure 3.10 Hydro-compaction in progress

Flooding of green areas with water was restricted to the height of adjacent footpaths (paved areas) to avoid indiscriminate spillage on the entire site. Therefore, it was initially decided to flood the site continuously for 12 hours followed by a rest period of 12 hours and repeat the cycle until no further seepage loss was seen. However, it was only possible to continue the above timings and processes for only 2 days, after which heavy flooding rendered it impossible to maintain the fixed cycle timings as above. Ultimately, 2 hours of flooding time was enough for the entire landscaped areas to get water logged and hence the hydro-compaction process terminated with a limited number of cycles. This rapid flooding situation could be attributed to high groundwater table and reduced zone thickness of the free draining material. Thus, the hydro-compaction process was stopped once it was clear that water had stopped percolating into the ground. In such a situation, in order to verify whether the added water had improved the density of soil, Mackintosh probe tests were conducted before and after the hydro-compaction process. It can be seen in Figure 3.11, that the soils responded to water movement because the number of blows after hydro-compaction increased for all depths down to 1·4 m. However, improvement in the ground strength was not noticed at depths of 0·4–0·6 m, possibly due to saturation of soil rather than the collapse response to hydro-compaction. A similar behaviour was noticed at a depth below 1·4 m, and this could be attributed to the nearness of the groundwater table, located at 1·5 m below ground. As stated by many researchers

(Dudley, 1970; Reginatto and Ferrero, 1973; Petrukhin, 1989; Bell, 2000; Jotisankasa, 2005; Bolzon, 2010; Rezaei *et al.* 2012), collapsible soils do respond to moisture in such a way that the reduced suction destroys the inter-particle strength leading to sudden settlement.

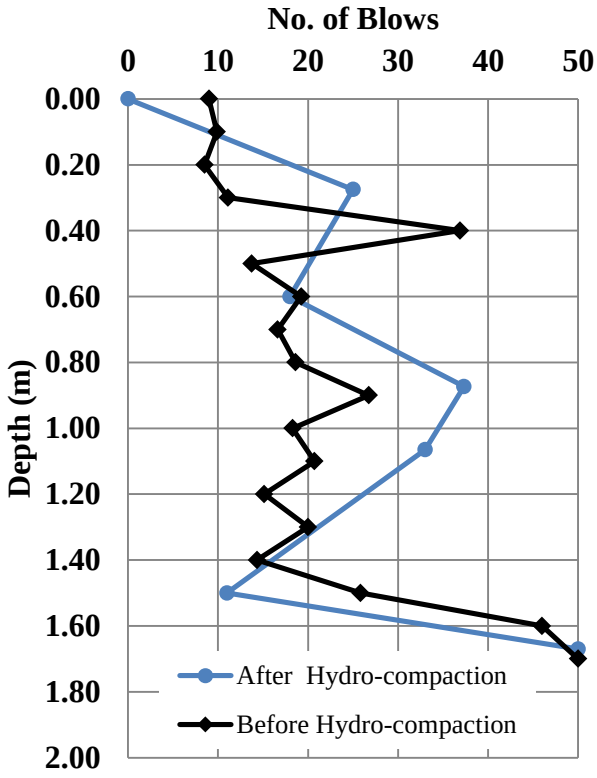


Figure 3.11 Mackintosh probe test results

Owing vibrations and noise, it was considered that hydro-compaction might cause unacceptable nuisance to the occupants of the villas, and so an alternative way of improving the loose soil at shallow depths was explored. Grout comprising 35% sodium silicate, 5% amide and 0·5% bicarbonate was injected under boundary walls and below the edges of paved areas to densify the upper 2 m of the soil stratum. This needed drilling of holes drilled down to 2·5 m depth on either side of each boundary wall and at 1·5 m centres. Holes were also staggered along the lines of private paved areas at 1·2 m centres. Under controlled pressure, grouting was done in such a way that upward heaving of the ground was prevented. Upon completion of the grouting of all drilled holes, a curing period of 4 weeks was allowed for the grout to attain strength. Mackintosh probe tests were performed before and after the grouting process to verify the effectiveness of the soil densification process. It can be seen (Figures 3.12 and 3.13) that the depth of improvement due to grouting was limited to 0·6 m below ground level compared with that for hydro-compaction process, where the improvement extended to 1·4 m depth. The limited extent of improvement from chemical grouting could be due to non-uniform permeation of grout into soil beyond 0·6–0·8 m below ground. Finally, it was suggested by the geotechnical investigation company to continue with the hydro-compaction in all areas where

settlements were noticed and allow the settlements to proceed to their maximum values before continuing with repair work to restore the distressed structures.

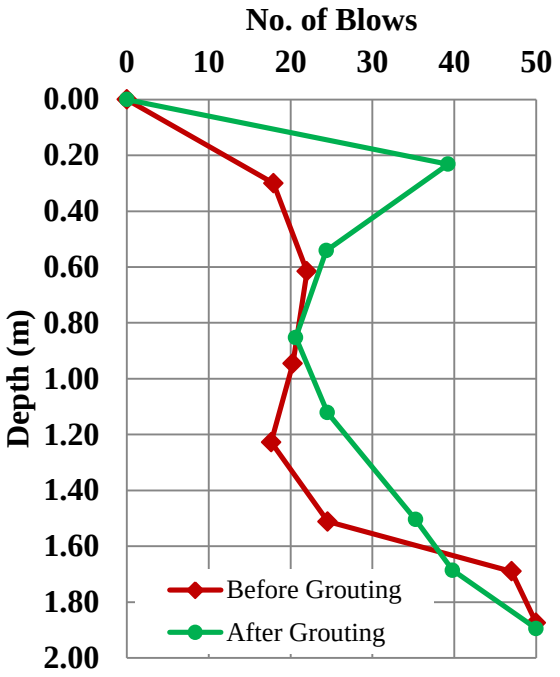


Figure 3.12 Mackintosh probe test results at boundary walls

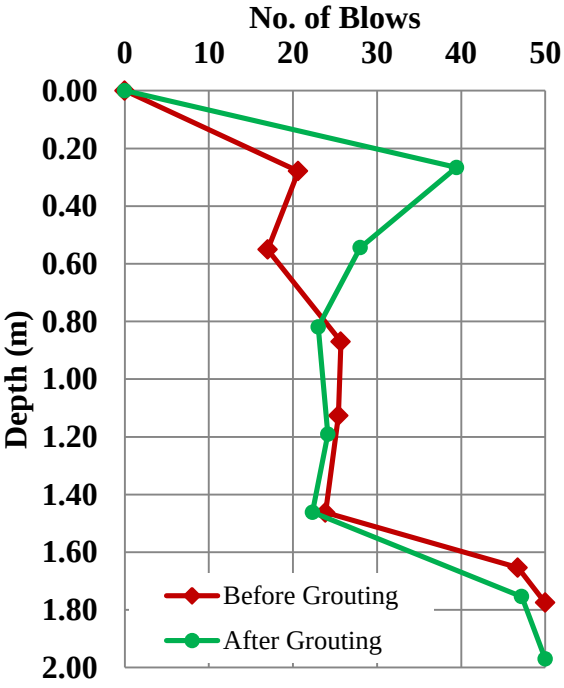


Figure 3.13 Mackintosh probe test results at paved areas

In summarising, both of the case studies discussed above indicated that structural distresses persisted long after completion and commisioning of the infrastructures. Field monitored records by the specialist geotechnical company produced a discernible clue as to how the hitherto unforeseen settlements occured. It was demonstrated with compelling evidence that the structural distress arose from continuous ground settlement caused by

unexpected collapse within some uniquely water-sensitive layers underlying the sites. Those layers were later identified to be collapsible soils but had not been properly considered at the time the structures were built many years before the problems arose. As the both case study sites lie in arid or semi-arid regions, property owners understandably resorted to irrigating their lawns in order to beautify landscapes but were completely uninformed about the existence of collapsible strata and implications of infiltration.

Collapsible soils are very different in behaviour from other soils in that their inter-particle and cementating bonds break down when suction is lost as water enters the soil. Proper analysis of collapsible soils require application of advanced theories of unsaturated soils and this explains why a routine site investigation and geotechnical design would have been inadequate for the UAE case records reported here. Several researchers have previously attempted to indentify and characterise collapsible soils, however so far no work has been done to simulate the behaviour of such soils under the influence of irrigation. Yet this is the kind of knowledge that would have prevented the problem observed in the UAE case studies. Therefore, in this thesis an innovative strategy is developed to simulate the how controlled drip irrigation influences a collapsible soil. This was achieved using purpose-designed large scale and small scale laboratory test apparatus. By analysing the results of the tests, it was possible to develop empirical equations for predicting settlement of a specified type of collapsible soil under given irrigation conditions. Detailed descriptions of the laboratory test arrangements and procedures are presented in Chapter 4.

CHAPTER 4 - EXPERIMENTAL METHODOLOGY

4.1 Introduction

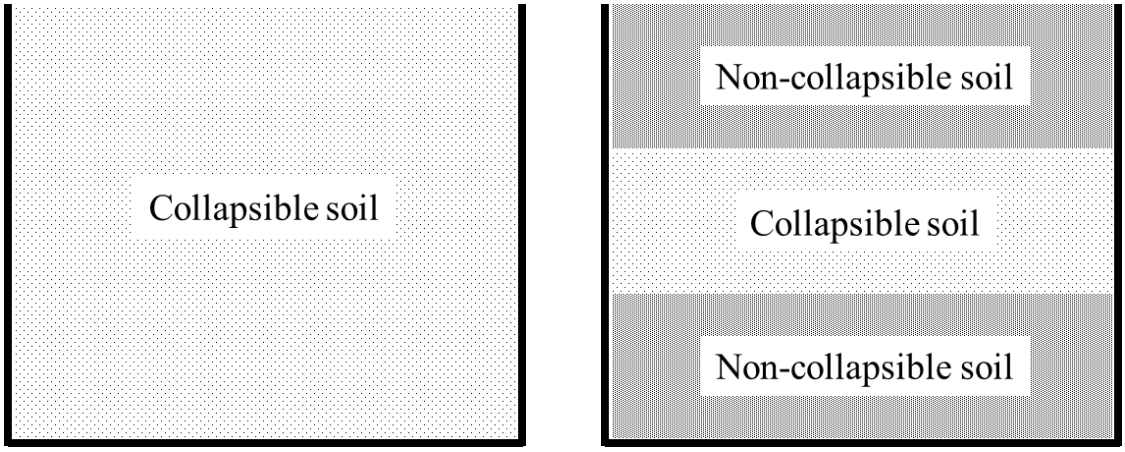
This chapter explains the systematic test procedures and data generated to deliver the research objectives and hence develop fuller understanding of the problems observed in parts of the UAE, where settlement of collapsible strata resulted in infrastructural damage. Until the damages occurred, engineers had limited awareness of the risks associated with irrigating landscapes underlain by collapsible soils bearing shallow foundations. Lessons learnt from experience in the UAE and a drive to develop solutions have motivated the current research and led to a review of the existing test procedures generally used in characterizing soils. It emerged that most of the available test methods do not properly consider the special effects of water ingress on soil collapsibility. Furthermore, in the case of the UAE, undisturbed soil test methods for shallowly founded structures are largely inapplicable because swathes of the superficial deposits are mostly non-cohesive. The present work sought to address the above problem through formulation of two custom designed tests that simulate the loaded behaviour of collapsible soils under different regimes of surface irrigation but at a particular level of overburden pressure. Sufficient care was taken to create test conditions as close as possible to reality. From analysis of the comprehensive data generated, mathematical relationships were formulated to assist engineers in estimating settlements of a specified collapsible stratum influenced by a given irrigation regime. Detailed descriptions of the arrangements and apparatus for the two test types are given in the following sections.

4.2 Experimental set-up – I

The first of the bespoke apparatus involved customizing a mini plate bearing test starting from the standard arrangement described in BS 1377-9:1990 but incorporating a custom made tank in which the collapsible soil specimen was to be tested. The tank was designed to be of sufficiently large dimensions so as to minimize boundary effects on the stressed soil zone beneath a loaded plate lying on the soil surface. Additionally, the apparatus included a special facility that enabled both the water table in the soil and the infiltration rate to be varied during the test. This was necessary in order to simulate the actual irrigation operations at locations of the UAE where settlement problems were experienced. Varying of water levels in the test also enabled simulation of the subsurface conditions observed from borehole logs at the UAE sites. Successful implementation of the control variables and test procedure was considered key to understanding the underlying mechanisms of soil collapse, so that predictive equations could be developed for assessing

rates of settlement of collapsible strata, as functions of various variables such as (1) thickness of collapsible layer, (2) its depth from ground level and (3) groundwater regime. Plate load tests were then carried out for various controlled infiltration rates and at specified magnitudes of surcharge loading.

To ensure consistence with the ground conditions at the UAE case study sites, it was imperative that the plate tests on collapsible soil lenses were performed at constant surcharges equivalent to the gross pressures exerted by the actual structures (such as perimeter walls around residential properties) which were affected by irrigation-induced settlement. Additionally, infiltration regimes applied in the plate tests, through controlled dripping rates and positions, had to mimic the actual irrigation activities at the affected UAE sites. In turn, water level changes occasioned by the various applied dripping rates had to represent true site conditions as realistically as possible. The plate load tests were carried in two different cases as typified in Figure 4.1.



Case-1 Tank filled with collapsible soil only

Case-2 A collapsible soil layer sandwiched between two other layers in the tank

Figure 4.1 Plate load test cases

In both cases mentioned above, the soil surface in the tank was loaded with a pressure equivalent to the gross load of the real structures, after which settlements were monitored in real time as controlled dripping continued. In case-1, the influence of variable depths of water in the tank on the magnitude and rate of settlement was continuously monitored and recorded. The arrangement in case-2 above was to investigate how the thickness of the collapsible stratum, sandwiched between non-collapsible layers, influenced the magnitude and progression rate of settlement. It was critical that the laboratory test conditions modelled the field situations as accurately as possible. Full details of the materials, experimental arrangement, and instrumentation specifications are narrated in the following sub-sections.

4.2.1 Plate load test set-up

A cubic tank of size 1.0 m x 1.0 m x 1.0 m was fabricated using a mild steel sheet of 4 mm thick, with carefully designed joints to form a water-tight enclosure. The fabricated tank was placed below a loading frame made of a steel beam 250 mm wide by 250 mm deep and having a mass per linear meter equal to 72.4 kg/m. The entire steel frame had an approximate mass of 500 kg and offered reaction against the hydraulic jack loading applied incrementally on the test plate placed on the soil surface. (Figures 4.2 and 4.3).

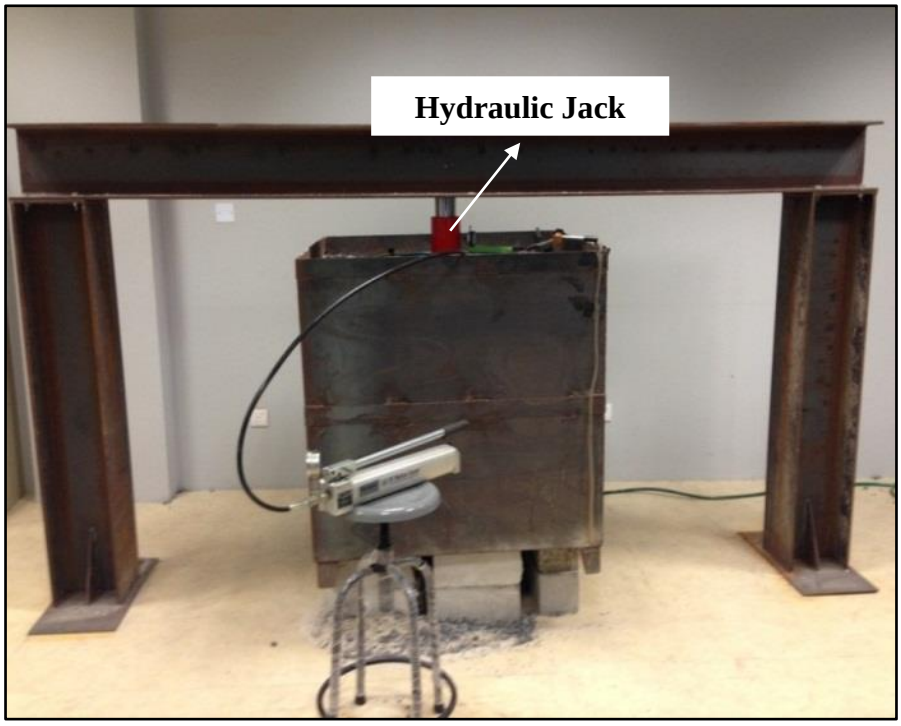


Figure 4.2 Plate loading on soil incorporating collapsible layer (elevation)

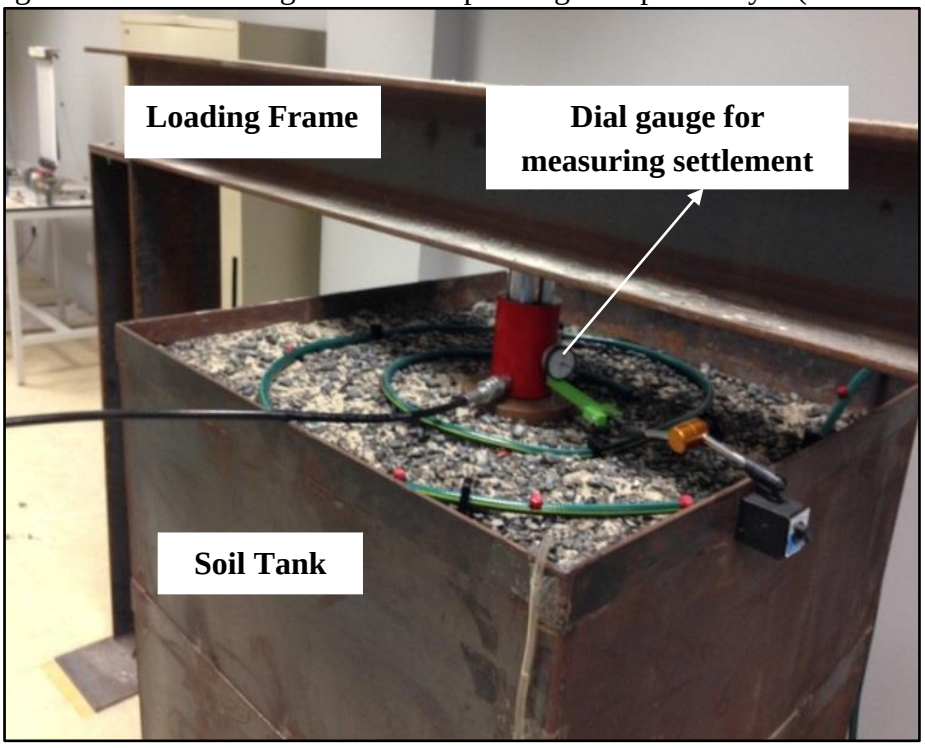


Figure 4.3 Plate loading on soil incorporating collapsible layer (top view)

4.2.2 Test Load Calculation

It was necessary to determine the mass of the loading frame required to provide adequate reaction against the jack loading up to the target maximum applied load. Table 4.1 presents a summary of the projected figures, calculations of which are set out below.

Table 4.1 Minimum required reaction load for various stresses

Stress (kN/m^2)	**Diameter of test plate (mm)**	**Minimum required reaction load (kg)**
50	200	157
100*	200	314
150	200	471

*Sample calculations

Stress = 100 kN/m^2

Area of plate = πr^2 = 3.14*(0.01)^2 = 0.0314 m^2

Reaction load required (kg) = 100 x 1000 x 0.0314/10 = 314 kg

The 500 kg loading frame (beam plus two vertical posts on either side) was capable of resisting a maximum stress equal to 150 kN/m^2. To reproduce the overburden condition of the collapsible layer in the field, the plate tests were conducted at a constant pressure of 80 kN/m^2. This figure was estimated taking into account the weights, sizes, and respective depths of foundations for the various structures affected by settlement in the UAE case studies.

4.2.3 Representation of the groundwater table regime

Previous researchers focused mainly on performing laboratory plate tests on soils in either dry or fully saturated conditions. This clearly departs from reality since natural soils in the ground rarely exist in the conditions assumed above. In the present work, every effort was made to create test conditions that reflected soil moisture contents and water table positions observed in the field. Therefore, a hole was drilled on one side of the test tank and a piezometer inserted along with graduated scale in order to measure and control the water table level (Figure 4.4). At the start when the tank was empty, water was filled up to 100 mm from bottom of the tank and dry soil was carefully added over the water. This is to ensure that all voids in soil are fully filled with water (replication of soil condition below groundwater table). Once water level in the tank matched with water level in piezometer, placement of soil and water was done slowly and simultaneously until a stable water level in the tank was established and the same is reflected in the piezometer attached to the tank.

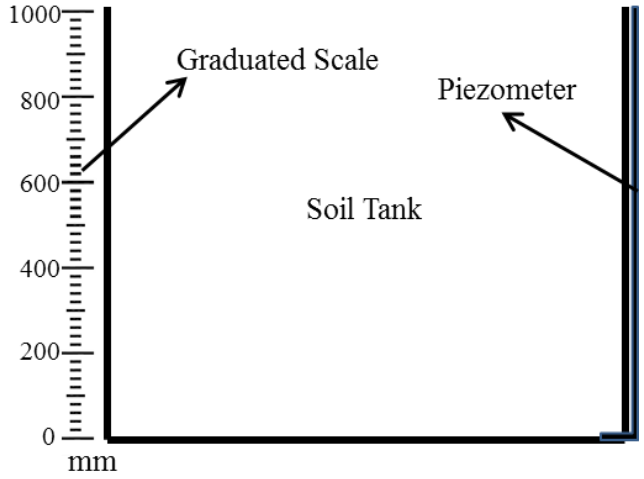


Figure 4.4 Piezometer for monitoring water levels in the soil in tank

4.2.4 Preparation of collapsible soil

Collapsible soil samples were collected from various sites around Abu Dhabi, UAE, where a specialist investigation of infrastructure damages had revealed a strong link to irrigation-induced settlement. Upon thorough analysis of the survey results, it was apparent that the problems emanated from a hitherto unforeseen behaviour of collapsible soil strata that lost strength upon ingress of water from surface irrigation activities. The investigation companies drilled a number of boreholes, which showed that collapsible soil strata existed at depths where standard penetration test (SPT) values were low (N = 4 to 10) and very low (N<4). It was from the low SPT zones where soil samples were collected for the present research work. The granular characteristics of the recovered soil samples were determined using sieve analysis (BS 1377-2:1990) to plot typical particle size distributions as shown in Figure 4.5.

The average graph which closely depicts the grain size distribution of all such soils was plotted and marked with a thick black curve (Figure 4.5), along with respective SPT values and depths. The nomenclature used in Figure 4.5 is: depth, SPT N-value. For example (3–3.45 m, 10) indicates that the soil sample was obtained in respective borehole at a depth of 3.00–3.45 m using split spoon sampler and the SPT N-value of 10 was recorded on site. Since a large quantity of soil of specific gradation was required to fill the test tank, a specialist company was contracted to grade the soil to required sizes on large scale basis using computer software. This facilitated rapid production of 3 tonnes of soil that satisfied the desired gradation. However, an independent laboratory check was still made to ascertain the accuracy of the computerized gradation. On receipt of the soil samples from the specialist gradation company, various laboratory tests were carried out on the soils to determine their fundamental properties, which are summarized in Table 4.2.

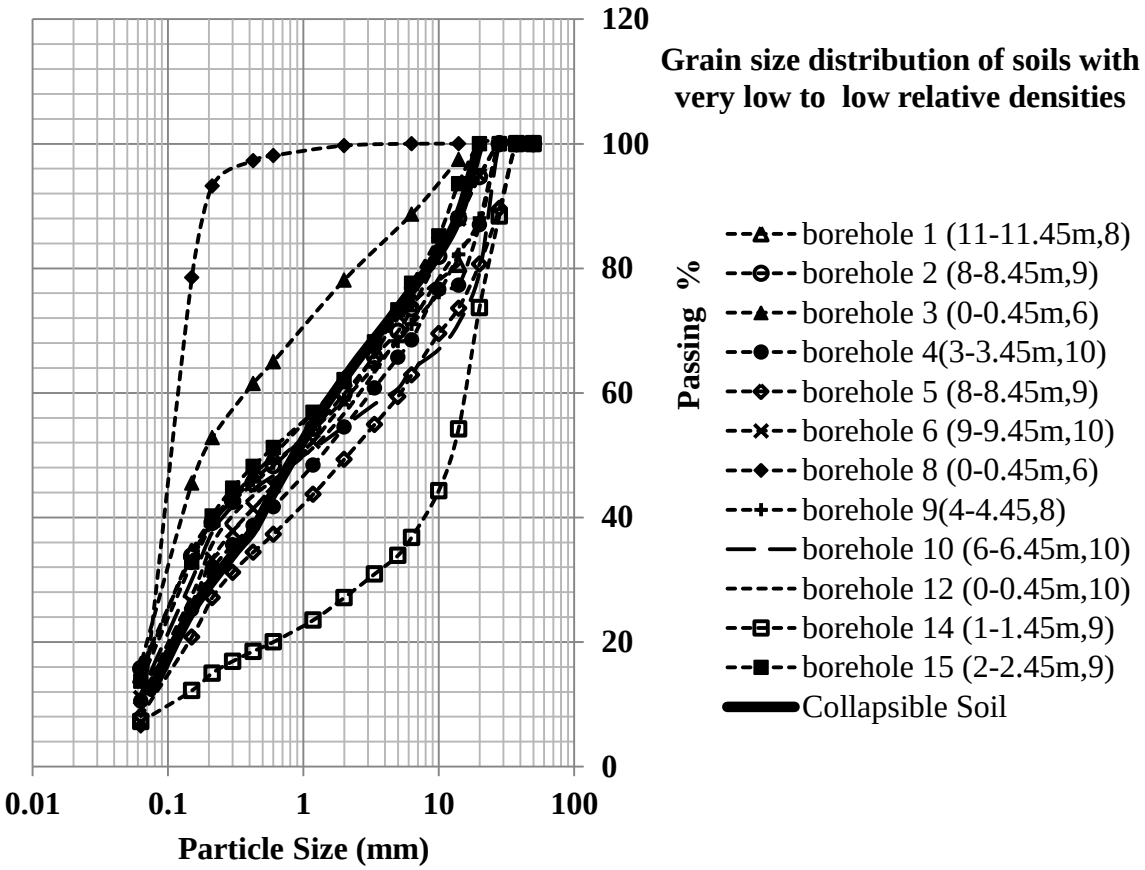


Figure 4.5 Grain size distributions

Table 4.2 Properties of representative collapsible soil

Property of collapsible soil	Value
Specific gravity (G)	2.66
Plasticity characteristics	Non-plastic
Optimum moisture content (w)	15.50%
Maximum dry density (γ_d)	18.45 kN/m^3
Permeability (k)	8.86x10^{-05} m/s

4.2.5 Plate load test details

At first, the loading plate was set-up centrally below the frame and its horizontality checked using a spirit level (Figure 4.6). After the position and level of the plate had been set-up correctly, a hydraulic jack was carefully placed on top of it and precisely below the loading frame. With the help of a magnetic stand bearing on the side of the tank, a dial gauge was set up on the surface of the loading plate to measure settlements. Loads were then applied via the hydraulic jack in increments of 1/10th to 1/12th of the targeted maximum pressure (80 kN/m^2) until the final pressure was reached. Care was taken to ensure that readings were recorded when the plate settlement reached a stable value at the end of each load increment.

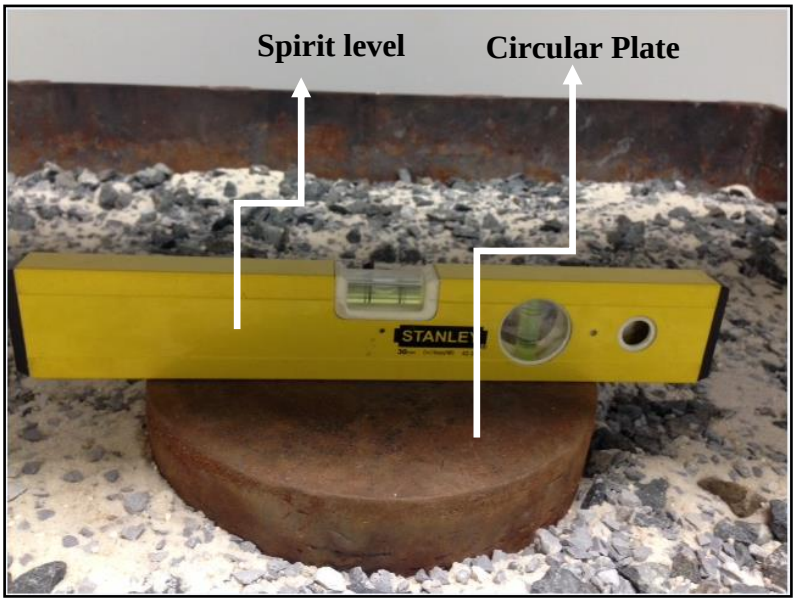


Figure 4.6 Levelling the plate top

In order to investigate how the combination of groundwater level and infiltration of water would affect the settlement of the plate (hence foundation in the field), three plate load tests were carried out at different water levels as listed in Table 4.3 (test numbers 1 to 3). After the first plate with water level at 500 mm (2.5B) below the plate was conducted, the soil above the water level was removed from the tank and dried completely. Before proceeding to the second test, the dried soil was placed in the tank and water table raised to 300 mm (1.5B) below the plate. A similar procedure was also adopted between the 2nd (1.5B) and the 3rd (1.0B) tests. It was also important to study how the thickness of the collapsible layer relative to the total thickness of the bearing strata would affect the settlement magnitude and rate. For this reason, four plate load tests (Table 4.3, test numbers 4 to 7) were performed for specified ratios of collapsible layer thickness to total strata thickness equal to 1/2, 1/3, 1/4 and 1/5 in turn. The desired ratio was achieved by inserting a collapsible layer of pre-determined thickness at the mid-height of the soil profile.

Table 4.3 Details of plate load test conducted

Test Number	**Soil details in the tank**	**Thickness of collapsible soil**	**Depth of water level below bottom of the plate**
1	Fully filled with collapsible soil	H	2.5 B (500 mm)
2		H	1.5 B (300 mm)
3		H	1.0 B (200 mm)
4	Collapsible soil at mid-height of the total soil in the tank	H/2	No water table simulated in the tank
5		H/3	
6		H/4	
7		H/5	
B = Diameter of plate=200 mm; H=Total height of soil in the tank			

4.2.6 Irrigation regimes applied in the tests

Landscaping companies operating in the monitored UAE sites were approached to supply specifications of their irrigation activities so that calculations could be done to arrive at the control parameters to be used for the plate tests under infiltration regimes representing site conditions. The plate tests used a dripper pipe commensurate in size to the actual ones used by the irrigation contractors. Perforations were created on the pipe at 150 mm intervals before dripper nozzles were fitted. The prepared pipe was placed on the surface of soil in the tank (Figure 4.7) with one of its ends closed and the other connected to a water supply. Now, in the field the collapsible strata at depth were already acted upon by the overburden pressure before commencement of irrigation.

To be consistent with this in the laboratory test, cyclic dripping was started once the target pressure of 80 kN/m^2 on the sandwiched soil sample was attained. A dripping 'cycle' was defined as the application of specified quantity of water every 12 hours for a period of 30 min. This corresponded to the irrigation specifications applied for the affected UAE sites. In the plate load test, drip irrigation was simulated over the soil in the tank at a rate of 13 l/m^2/day, which was also a specification from the landscaping companies. Water consumed by vegetation and lost in evapotranspiration was thought to be relatively small and hence not taken into account in the simulating tests. The soil was then watered twice a day (6.00 am - 6.30 am and 6.00 pm - 6.30 pm) uniformly at a rate of 6.5 l/m^2. In order to ensure the correct discharge, a water meter was fitted at the outlet and a stopwatch used to check the flow rate. The dripping cycles were continued until the observed soil settlement rate was so high that the applied pressure could no longer be maintained constant.

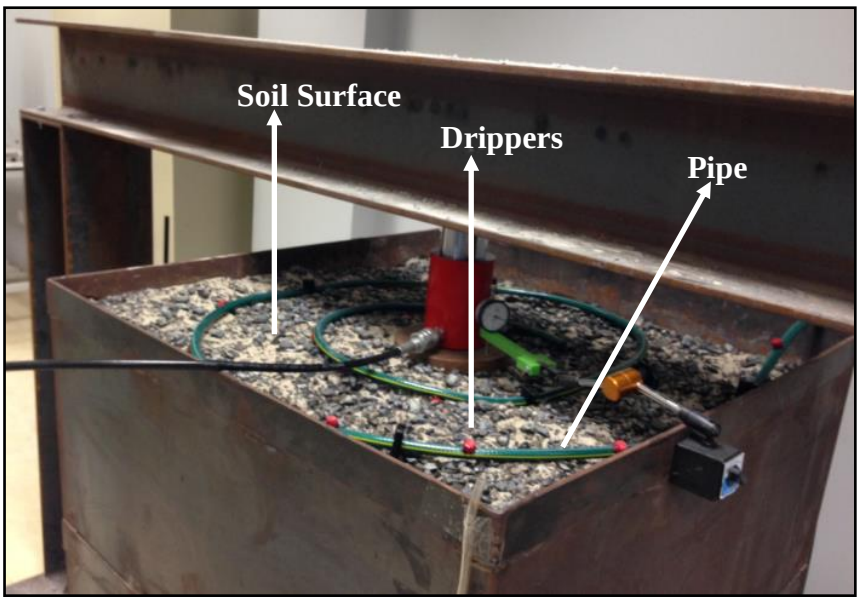


Figure 4.7 Arrangement for simulating the field watering pattern

4.2.7 Constant load application procedure

As a basis for simulating the stress condition of the foundation soils at the UAE sites, it was considered that live loads on existing structures were small in comparison to the structure self-weight. Therefore, to represent this in the laboratory, it was imperative to conduct the plate load tests at maintained pressures. With simultaneous dripping applied, the collapsible soil layer sandwiched in the tank was expected to start losing strength due to the collapse of its structure as internal suction was gradually lost. Occurrence of this event would lead to simultaneous decrease in the pressure acting on the soil. In contrast, the ground pressure beneath a structure would be constant and so to create this situation in the test tank, any reduction in pressure was immediately compensated by manually operating the hydraulic jack lever to increase the load (Figure 4.8). This was made possible due to continuous monitoring of pressure while applying the desired dripping cycles on the soil surface.

Figure 4.8 Hydraulic jack for maintaining constant pressure

4.3 Experimental set-up – II

The second of the custom designed tests was adapted as a small scale model to fit in the constrained laboratory space available. At the same time, the apparatus had to be reasonable enough to be used to study how the combined effects of the following factors influence the settlement response of a collapsible soil layer bounded by two free-draining layers. The factors are: (a) imposed water levels and (b) collapsible layer thickness.

4.3.1 Test arrangement

A large water bath was prepared along with volume-graduated bottles fitted with valves to allow variation of water drip rates. The ‘infusion bottles’ could be positioned at specific points over the soil surface to relate to field irrigation specifications for a unit landscape

area. A metal mould similar to a standard California Bearing Ratio (BS 1377-4:1990), was used to cast a three-layer soil profile with each layer compacted to pre-determined densities. This is illustrated in Figures 4.9 and 4.10. Depths of water in the large tank could be varied to simulate groundwater tables in the field, whereas infusion bottles with controllable flow rates simulated the intensity of landscape irrigation. A maintained surcharge of 4.54 kg was applied on the surface of the uppermost soil layer in the metal mould. The middle layer was formed from a specimen of collapsible soil (refer section 4.2.4) obtained from some of the boreholes that had been drilled as part of the ground investigation in the UAE case studies of settlement damage to infrastructure.

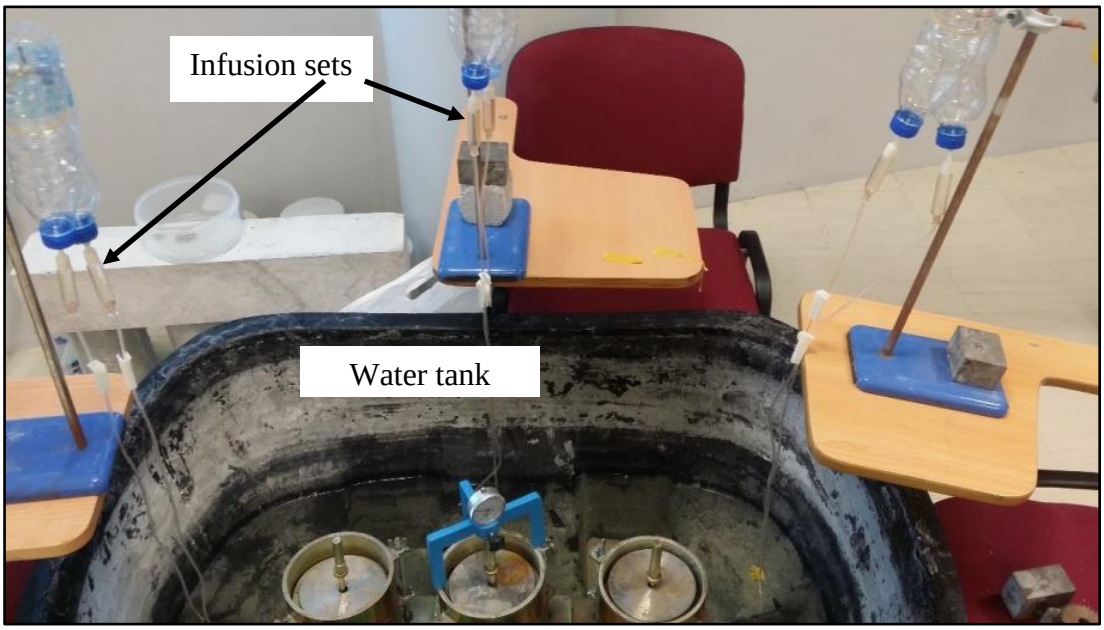


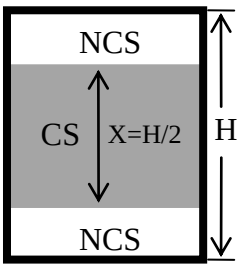
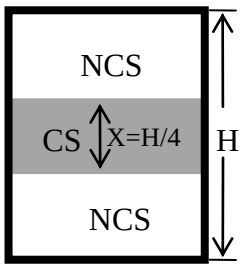
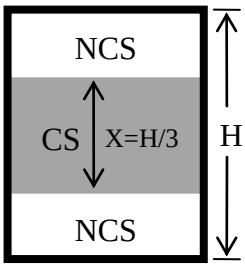
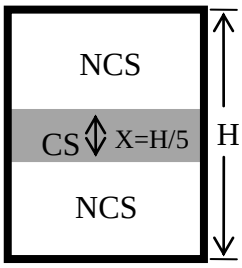
Figure 4.9 Purpose designed experimental arrangement for measuring settlement of collapsible soil under varying water levels and constant drip irrigation

Figure 4.10 Monitoring of the initial gauge readings for soil in the dry state prior to start of irrigation

4.3.2 Soil profiles and relative thicknesses in test model

In order to obtain develop data on the characteristics of settlement of collapsible soil, the layer was cast in different thicknesses in a metal mould while simultaneously imposing different water levels in the surrounding water bath. Four soil profile cases: SC-1, SC-2, SC-3 and SC-4 were formed in the moulds whereby in each case collapsible soil specimen was cast between two sand layers both of which were free-draining and non-collapsible. The collapsible soil specimen was taken from the batch that had been computer graded as explained in section 4.2.4. For each soil combination (SC), the overall thickness of the three soil layers in the mould was kept constant (H), as shown in Table 4.4. The main difference in the four cases is the thickness of the collapsible layer, which was set at H/2, H/3, H/4 and H/5 as shown in Table 4.4. For each soil combination, settlements were measured for three compacted densities: 17.5, 18.0 and 18.5 kN/m^3. Furthermore, for each density case, tests were conducted for three different water depths in the mould, i.e. H/3, H/2 and 2H/3 from bottom of mould. Thus, a total of 36 tests were performed as seen in Table 4.5. Details about experimental set-up, materials and various simulations are described in the coming sections.

Table 4.4 Soil combinations used in experimentation

Soil Combination (SC)	**Details**	**Soil Combination (SC)**	**Details**
SC-1		SC-3	
SC-2		SC-4	

Note :
H – Height of the metal mould (180 mm)
NCS – Non-collapsible soil
CS – Collapsible soil

Table 4.5 Details of all tests conducted in metal moulds

Test Number	**Soil Combination**	**Height of water table from bottom of the mould**	**Density of soil (kN/m^3)**
1	SC-1	H/3	17.5
2			18.0
3			18.5
4		H/2	17.5
5			18.0
6			18.5
7		2H/3	17.5
8			18.0
9			18.5
10	SC-2	H/3	17.5
11			18.0
12			18.5
13		H/2	17.5
14			18.0
15			18.5
16		2H/3	17.5
17			18.0
18			18.5
19	SC-3	H/3	17.5
20			18.0
21			18.5
22		H/2	17.5
23			18.0
24			18.5
25		2H/3	17.5
26			18.0
27			18.5
28	SC-4	H/3	17.5
29			18.0
30			18.5
31		H/2	17.5
32			18.0
33			18.5
34		2H/3	17.5
35			18.0
36			18.5
H = Height of metal mould = 180mm			

4.3.3 Experimental test set-up

Prior to casting soils in the metal moulds, a filter paper was placed at the bottom of the mould to prevent soil particles from clogging the perforations in the bottom plate of the mould. Required amounts of each soil type were weighed, carefully placed and compacted in the moulds to required thicknesses and densities. The moulds containing the compacted soils were then placed inside the wide-bottomed water bath (Figure 4.10). A thin spacer disc was used to keep the bottom of the mould clear from the tank base in order to ensure easy entry of water into the moulds through the perforated plate. Using the infusion bottles, water was discharged at controlled rates onto the top soil layer in the mould. This was to simulate the typical irrigation rates ($m^3/m^2/s$) actually applied for the landscapes in the UAE case studies. Given the free-draining properties of the top and bottom layers, it was possible for the water level in the soil inside the moulds to quickly stabilize, matching the level in the tank. Using a swell plate and gauge tripod assembled as shown in Figure 4.11, settlements of the top soil surface were measured at close intervals of time at different water levels while maintaining water flow from the infusion bottles at selected rates.

4.3.4 Simulation of groundwater table

As already discussed, settlement simulation for a collapsible soil must relate closely to real field conditions rather than be based on dry or fully saturated states, as most researchers have tended to portray. In the present work, the starting point was to fill the moulds with calculated weights of dry soils and statically compact them to the predetermined overall depth, H, in the mould thus achieving the targeted density. Thereafter, a swell plate was installed with surcharge weights above, followed by recording of the initial reading of the dial gauge. The moulds were then placed in the plastic tank, to which water was added gradually to the target depths H/3, H/2 and 2H/3 from bottom of the mould. Using the dial gauges, settlements of the top soil surface were measured and recorded continuously from the dry state of soil until achievement of the target water depth. Measurements were continued until cessation of settlement as water seeped through the perforated plate at the bottom of mould. The difference between the initial dial gauge reading (with the soil in the dry state) and the final reading upon cessation of settlement was attributed to the settlement induced by the water table rise.

4.3.5 Simulation of rates of landscape irrigation

On establishment of a stable value of settlement due to purely water level change, further measurements were undertaken to monitor settlement due to drip irrigation alone. For this

task, a control valve was connected to an inverted water bottle open at the top and filled with water as shown in Figure 4.10. Then the bottom end of the bottle was directed over the moulds and moved in uniform patterns to distribute water evenly on the soil surface. The water injection was done in cycles that corresponded to the irrigation specifications from the UAE case studies (section 4.2.6). In the present tests, a trial and error strategy was used and refined multiple times to determine the equivalent flow rate which would be applied to the known surface area of the soil in the mould. The trials were done by adjusting the flow control valve of the infusion bottles and using a stopwatch to record the duration of the applied drips. Settlements of the top soil surface were recorded continuously until the difference in settlement for two consecutive irrigation cycles fell below 0.01 mm. This was deemed to be a stable state for the settling soils. In order to maintain a constant discharge during an irrigation cycle, it was necessary to compensate for the gradually reducing head of water, as the drip cycle processed, by continuously feeding in more water through the open bottle top. At the end of the test, the settlement of soil due to drip irrigation alone was calculated by subtracting the dial gauge reading at the time before drip cycles commenced from the reading at completion of the drip cycles. The results obtained from the above experimental procedures are presented and discussed in detail in chapter-6.

CHAPTER 5 - FINITE ELEMENT MODELLING

5.1 Introduction

As mentioned in the preceding chapter, the laboratory simulations of the settlement effects of infiltration on collapsible soils offer certain advantages over conventional field testing. However, whatever the model scale and control parameters used, it is not possible to fully replicate the actual behaviour of the ground settling during irrigation operations. Field testing using state-of-the art equipment, if comprehensive enough, may be seen as capable of yielding more realistic results, but such tests would usually be time consuming and not cost-effective for small to medium scale projects. By the same argument, specially designed high tech laboratory methods utilising sophisticated instruments may offer good predictions but are very expensive and only available in few locations. Hence, there is merit in considering a third alternative in the form of numerical modelling and analysis. In many cases, numerical modelling works well if complemented with laboratory or field testing to determine reliable parameter values. The numerical strategy discussed in the following sections was developed and applied to the UAE case study (low-rise housing project, section 3.3) and involved:

a. Formulating a comprehensive geotechnical model for twin-villas with perimeter walls that enclosed irrigated lawns.
b. 3D finite element soil-structure interaction analysis of the villas and their perimeter walls, with simulated seepage intensity and cycle timing consistent with the actual irrigation specifications on site.
c. Non-linear finite element structural analysis of the perimeter walls, from where settlement predictions matching on-site measurements would serve to verify the validity of the analyses in (a) and (b) above.

5.2 Geotechnical modelling

Given the complexity of behaviour of collapsible soils and the incapability of routine laboratory tests to represent actual field conditions, it was considered that a fully coupled stress-seepage 3D finite element analysis would better deal with the problem and produce reasonable simulations of the ground collapse response to irrigation. To tackle this complex problem, it was necessary to design an appropriate mathematical model and deploy a powerful 3D finite element program. For this, *Midas*^{TM} *GTS NX* professional software was selected due to its advanced and customisable features. This professional program is used by leading geotechnical consultants and can cope with soil-structure problems involving transient seepage.

Since the fully coupled stress-seepage analysis does not follow the common assumption that steady pore water pressure is maintained, it is advantageous over other methods when transient seepage and stress analysis is significant in a problem. In contrast to a consolidation analysis, seepage boundary conditions are not necessarily fixed but can be defined to change as a function of time. Additionally changes in boundary flow rates can be accommodated. In other words, in a fully coupled stress-seepage analysis, it is possible to use all the transient seepage boundary conditions, structural load and boundary conditions. Thus, this analysis can be applied to the ground stability analysis for rainfall or irrigation for water level change. The seepage boundary conditions (Head/Flux) in this analysis can also be used to analyse not only the changes in excess pore water pressure, but also a primary consolidation process that is governed by pore pressure and time variations (Midas, 2014). The fundamental relationships, compatibility equations and numerical schemes underlying *Midas*^{TM} treatment of unsaturated materials and coupled stress-seepage under transient conditions are explained in the following sections.

(1) Seepage parameters and relationships

Though Darcy's law was originally derived for saturated soils, many researches (e.g. Narasimhan, 2004; Ghotbi *et al.* 2011) have shown that it can be extended to unsaturated soils as well. In the present work, seepage flow is considered along the three mutually orthogonal directions x, y, z of the model and the permeability coefficient (k) matrix is represented by equation (5.1), where only the diagonal components in each direction are considered.

$$k=\begin{bmatrix} k_x & 0 & 0 \\ 0 & k_y & 0 \\ 0 & 0 & k_z \end{bmatrix} \tag{5.1}$$

The permeability coefficients are criteria for controlling the seepage rate and depend on moisture content and voids ratio change, Δe. Since moisture content is dependent on pore pressure, it follows that permeability values also change with pore pressure, Δp. In the adopted model, Δe is used for consolidation analysis with fully coupled stress-seepage analysis. Values of Δe are calculated from the initial condition defined in the input. The unsaturated permeability coefficient is calculated from equation (5.2).

$$k=10^{\frac{\Delta e}{c_k}}k_r(p)k_{sat} \tag{5.2}$$

where,

k=unsaturated permeability coefficient

Δe = change in voids ratio

c_k= a term that defines the permeability ratio as a function of Δe

k_r (p) = permeability ratio function depending on Δp

p = Pore pressure

k_{sat} = saturated permeability coefficient

In the analysis, volumetric water content is defined in terms of the ratio between the water volume and total volume as shown in equation (5.3).

$$\theta=\frac{V_w}{V} = nS \tag{5.3}$$

where,

θ = Volumetric water content

V_w=Water volume

V = Total volume

n = Porosity

S = Degree of saturation

Calculation of element seepage and consolidation utilizes the volumetric water content for pore pressure (p), and requires differentiation of equation (5.3) and expressing the result using porosity and degree of saturation as shown in equation (5.4).

$$\frac{\partial \theta}{\partial p} = s\frac{\partial n}{\partial p} + n\frac{\partial S}{\partial p} \tag{5.4}$$

where,

θ = Volumetric water content

p = Pore pressure

n = Porosity

S = Degree of saturation

The first term of the right hand side of equation (5.4) represents the rate of change of the volumetric water content for the saturated condition. It is defined by a parameter called the specific storage (S_s), which represents the volumetric ratio of the water movement in the ground due to the pore pressure head change [equation (5.5)].

$$S\frac{\partial n}{\partial p} = \frac{\partial V_v}{\partial h}\frac{\partial h}{\partial p} = n\frac{S_s}{\gamma_w} \tag{5.5}$$

where,

n = Porosity

p = Pore pressure

V_v = Voids volume

S = Degree of saturation

h = Pore pressure head

S_s = Specific storage

γ_w = unit weight of water

The second term of the right hand side of equation (5.4) represents the slope of the volumetric water content for the unsaturated condition. This value uses the slope of the soil-water characteristic curve represents the relationship between the volumetric water content and pore pressure for unsaturated conditions. In the model, adopted in *Midas*^{TM} the non-linear characteristics of unsaturated soils are represented by various forms of ductile functions including: pressure head versus water content, water content versus permeability ratio function or pressure head versus saturation and saturation versus permeability ratio function.

(2) Modelling of seepage elements

Various relationships are used in *Midas*^{TM} to model elements for analysis of pore water seepage in both saturated and unsaturated soils. An important parameter involved here is the mass concentration of water in the ground, $\rho_w nS$. This can be defined considering the continuity equation of mass for micro-volumes. Continuity requires that the amount of water escaping from the micro-volume equals the change in mass concentration [equation (5.6)].

$$\nabla^T(\rho_w q) = \frac{\partial}{\partial t}(\rho_w nS) \tag{5.6}$$

where,

ρ_w= mass density of water

q= seepage flow velocity component

n = Porosity

S = Degree of saturation

The right term of the equation (5.6) can be expressed using the changes in water density, degree of saturation and porosity with time as shown in equation (5.7).

$$\frac{\partial}{\partial t}(\rho_w nS) = nS\frac{\partial \rho_w}{\partial t} + \rho_w n\frac{\partial s}{\partial t} + \rho_w S\frac{\partial n}{\partial t} \tag{5.7}$$

where,

ρ_w= mass density of water

n = Porosity

S = Degree of saturation

The adopted model is based on Darcy's law, considering porosity change with time only in the formulation process for element consolidation analysis. Pore pressure (p) is a variable in the seepage analysis, and the governing equation for the analysis is derived from Darcy's law as shown in equation (5.8).

$$\frac{1}{\gamma_w}\nabla^T(k\nabla p) - \nabla^T(k n_g) = (\frac{nS}{\rho_w}\frac{\partial \rho_w}{\partial p} + n\frac{\partial S}{\partial p})\frac{\partial p}{\partial t} \tag{5.8}$$

where,

γ_w = unit weight of water

k = coefficient of permeability matrix

p = Pore pressure

n_g = unit vector in gravitational direction

S = Degree of saturation

ρ_w= mass density of water

n = Porosity

To define the initial conditions for transient seepage analysis the ground water level is defined. Then steady-state analysis results are used at the initial time step load.

(3) Modelling of consolidation elements

The analyses with *Midas*^{TM} specifically use consolidation continuum elements to simulate stress-seepage coupled phenomena. During this process, consolidation analysis is essentially executed as a nonlinear analysis. Pore pressures related to both the steady state and transient states are identified and so classified. The initial water level defined in the model is considered as the steady state pore pressure, and the excess pore pressure during consolidation is considered as the transient state pore pressure. The transient state is the fundamental state of consolidation analysis. On completion of the element consolidation analysis stage, the results are expressed with reference to a user specified coordinate system.

With reference to the problem on hand, the sizes of all components of the geotechnical model were defined to match the respective on-site dimensions at the sites of the twin-villas. The components included the twin-villa complex with boundary walls, paved areas, green areas (drip irrigated areas) and respective car parks (Figure 5.1).

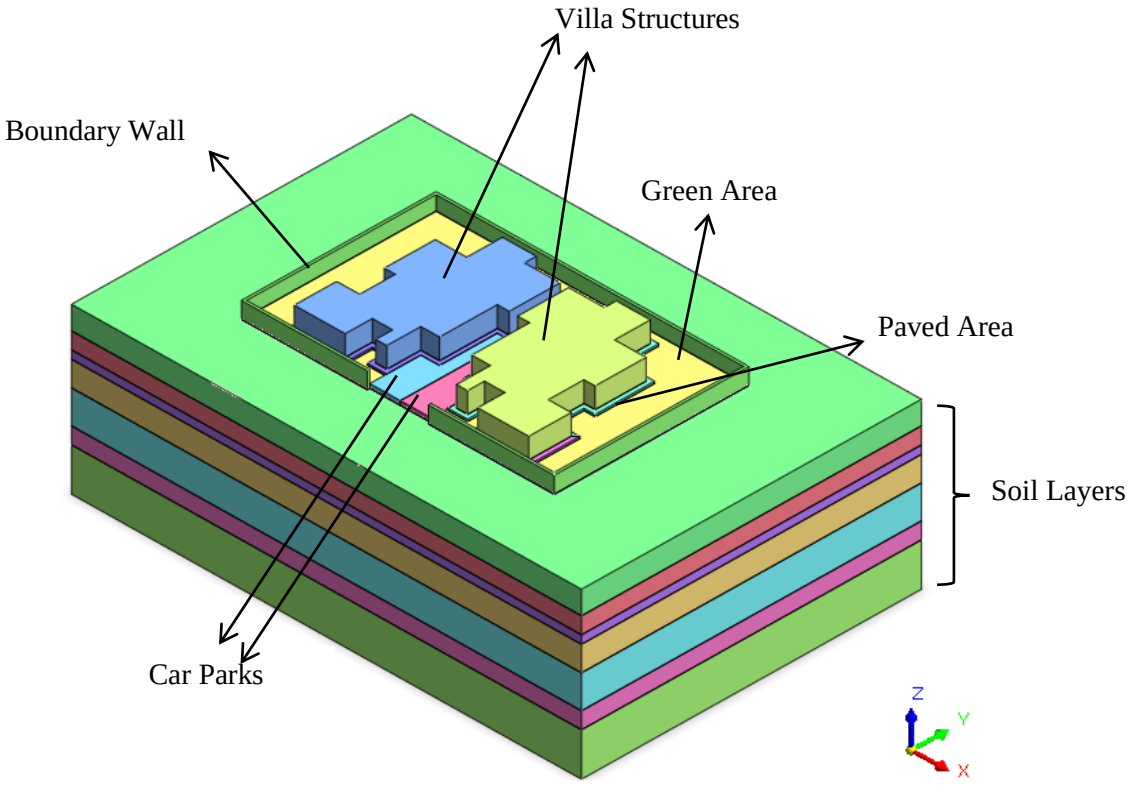


Figure 5.1 Geometric model of the twin-villa complex and underlying strata

The various control settings and parameter values used in modelling are described in the following sections.

5.2.1 Soil properties

Mohr-Coulomb model was used to describe the soil yielding and thus an elasto-brittle-plastic material model but with automatic transition to elasto-perfectly-plastic if residual and peak shear strength parameters become equal. Relevant parameters (except voids ratios and friction angles) for various soils (Table 5.1) were derived from the ground investigation report produced by the geotechnical investigation company involved in the UAE case studies. Voids ratios were calculated from known dry densities and specific gravity values; whereas friction angles corresponding to various standard penetration test "N" values were derived from correlation charts published by Bowles (1997).

Table 5.1 Input soil parameters in the analysis

Depth (m)	**Geotechnical parameters from lab tests / correlations**				
	Dry Density, γ_d (kN/m^3)	**Friction Angle, ø (degrees)**	**Voids Ratio (e)**	**Elastic Modulus, E (kN/m^2)**	**Permeability, k (m/s)**
0.0-3.0	14.00	30	0.89	5000	8.00 x10^{-5}
3.0-5.0	17.00	34	0.56	16000	3.00 x10^{-5}
5.0-6.0	14.67	31	0.81	8000	6.00 x10^{-5}
6.0-9.0	16.50	33	0.61	15000	5.00 x10^{-5}
9.0-13.0	17.60	35	0.51	18000	8.00 x10^{-6}
13.0-15.0	20.00	38	0.33	20000	4.00 x10^{-6}

5.2.2 Structural loads

Assessed loadings from villas, paved areas, boundary walls and car parks were input into the model to be as follows: 5 kN/m^2, 10 kN/m^2, 80 kN/m^2 and 60 kN/m^2 respectively. The loads were estimated based on the dimensions of the structures and respective unit weights of their constituent elements. It was recognised that a typical villa would exert negligible pressure at ground level since all the villas had long pile foundations that transferred load past the collapsible stratum down to the bedrock.

5.2.3 Meshing details

Tetrahedral elements were used to fine-mesh all soil layers and the nodal points inter-connected automatically across elements in the adjacent solids. This ensured appropriate nodal connectivity in the whole model (Figure 5.2). It shall be noted that shadings in Figure 5.2 does not have any computational significance and are seen due to colour combinations of model geometry and meshed elements in Midas visual display.

5.2.4 Drip irrigation simulation

As done in the laboratory simulations, data from actual irrigation specifications in the UAE case studies were used to assess the various infiltration parameters for defined areas of the finite element model. Details were as follows:

(a) the input flow rate was determined to be 13 l/m^2/day (i.e. litres per square metre per day). As previously stated, no allowance was made for any little water lost to vegetation or evapotranspiration.

(b) the 13 l/m^2/day flow rate was applied in two identical 30 minute cycles per a day, i.e. cycle 1 at 6.5 l/m^2 in the morning and cycle 2 at 6.5 l/m^2 in the evening. There was no irrigation in between the two cycles in any day.

In the program, the consequent transient flow from the irrigation process was modelled using the 'seepage boundary' function (Figure 5.3), which required assigning a value of flow rate per unit area of a defined flux surface (greens areas in the current model) of perpendicular water entry into the uppermost stratum considered.

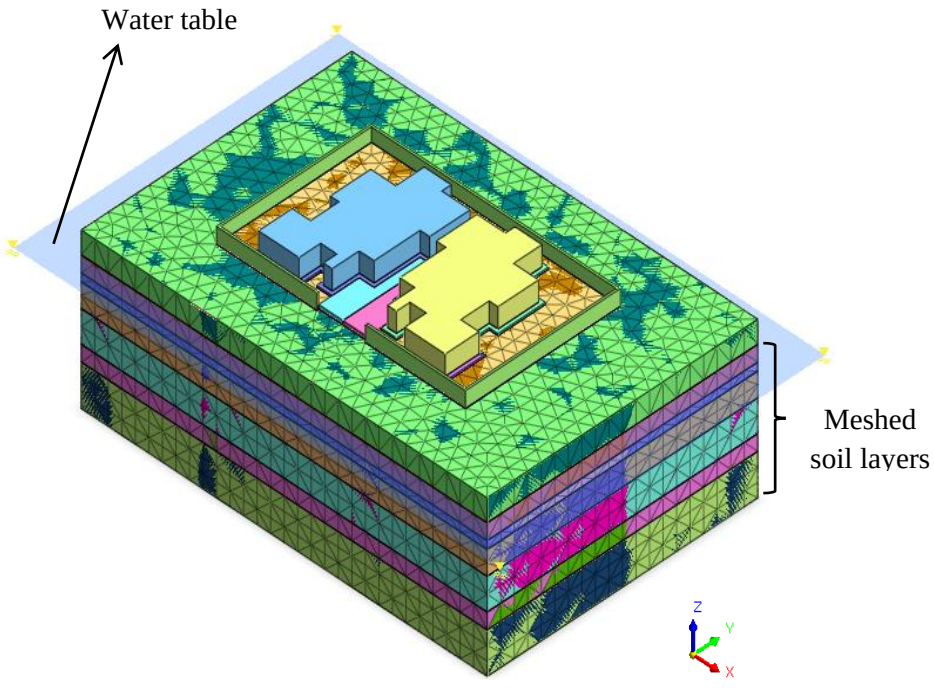


Figure 5.2 Meshed model incorporating soil profile and supported structures

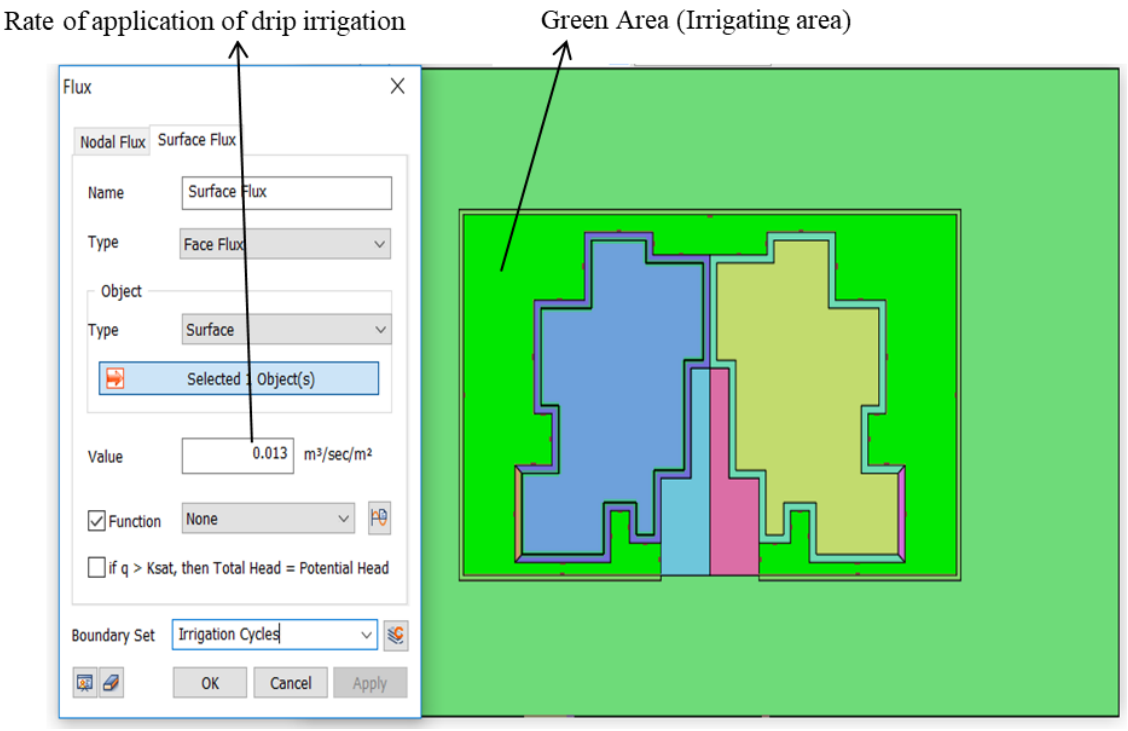


Figure 5.3 Seepage boundary conditions of the model (mesh un-selected for brevity)

5.2.5 *Boundary conditions of model*

In order to simulate the field situation as best as possible, appropriate boundary conditions of the mesh sets were defined by constraining displacements in: (i) the *x* direction for both the left and right faces of the geometry model, (ii) the *y* direction for both the front and back faces of the model, (iii) both the *x* and *y* directions for the bottom boundary of the model. Thus displacements were permitted in the z direction only, so that the calculated soil surface deformation would be interpreted as either settlement or heave.

It is recognised that in reality infiltration would be 3-dimensional, however since the ground surface at the actual UAE site was reasonably flat, the problem could be reduced to 1-dimensional, i.e. flow along the direction of gravity. Hence, to simulate this, the bottom face of the model was selected as a review boundary in order to enable customisation of seepage direction with respect to boundary surface considered (e.g. flow in a defined direction perpendicular to a specified plane).

Since the native soils at the UAE site were primarily dry silty sands and free-draining, it was reasonable to set the total head as zero for all the 29 boundaries (4 sides of the model times 7 stratum faces per side plus the bottom face) as seen in Figure 5.4. This guaranteed zero excess pore water pressure associated with loading.

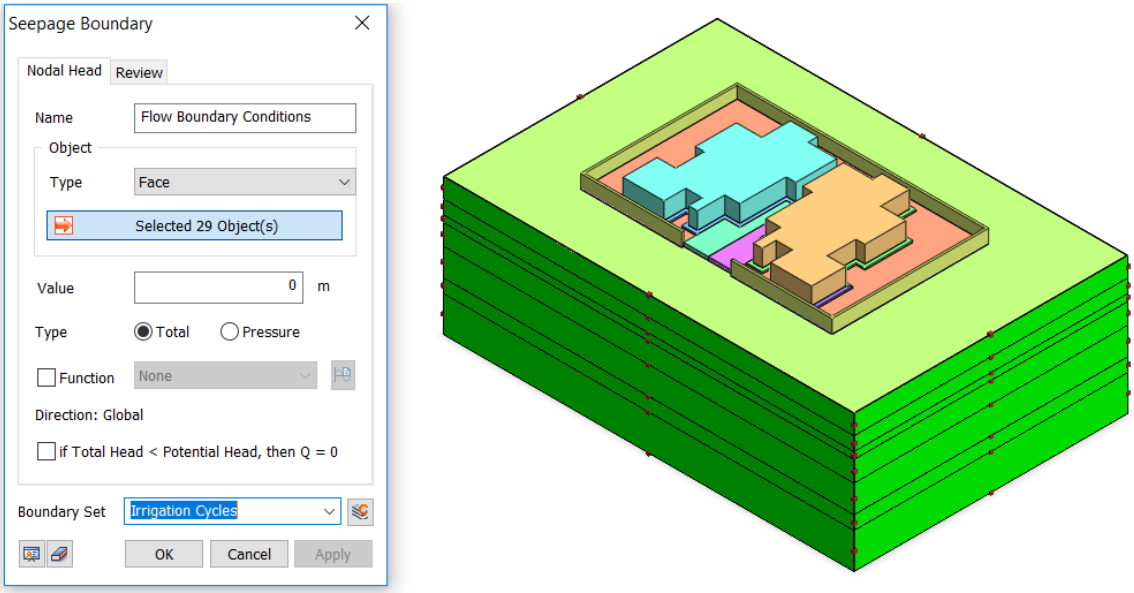


Figure 5.4 Seepage boundary conditions of the model (mesh un-selected for clear view)

5.2.6 Analysis methodology

For the model to closely represent reality, the analysis was carried out in a staged construction sequence as follows: (i) stage one equivalent to the *in-situ* conditions and accounts for the weights of the soil layers, (ii) stage two represents creation of the model villas and all other structures including boundary walls, paved areas etc. and (iii) stage three simulating the cycles of transient irrigation water flow.

In order to determine the soil deformations associated exclusively with the transient drip irrigation, ground settlements caused by soil self-weights and structures were nullified from the model. Finally, ground settlements were monitored at the end of every irrigation cycle or until there was either (a) no further settlement change or (b) the solution started to diverge, for the set convergence criteria, for the subsequent irrigation cycle.

5.3 Structural modelling of boundary walls

The geotechnical investigations at the site in UAE showed that the boundary walls around the villas suffered the greatest deformation as a result of irrigation-induced settlement of the collapsible strata. As seen in Figure 3.7 (a)-(d), as the soil beneath the boundary walls settled, the top surface of the wall remained unaffected and horizontal. Furthermore there was no evidence of the entire wall sagging as a unit. Instead, extreme movements occurred along the masonry bedding joints at 300-400 mm above the ground. It would have been expected that the wall would deform in a different pattern since both of its ends were supported on the settling soil. Hence, to examine how the observed failure mechanism was

possible, further analysis was undertaken using a separate non-linear structural analysis module within the *Midas*^{TM} program.

5.3.1 Modelling parameters

Midas^{TM} program was used to analyse a boundary wall around a typical villa at the UAE case study site. Actual dimensions of the wall were input in the analysis together other relevant parameters shown in Table 5.2. The properties of the bricks and mortar of the wall were supplied by a local contractor, whereas interface properties for the FE model were gleaned from *Midas*^{TM} user manual.

Table 5.2 Input parameters for soil-structure interaction analysis of the boundary wall

Material	Parameter	Unit	Value/Description
Brick	Material	-	Cement concrete
	Size (length x height x width)	mm	400x200x200
	Elastic modulus (E)	N/mm^2	16700
	Weight density (γ)	kN/m^3	21.6
Mortar	Material	-	Cement mortar (1:6)
	Compressive strength (σ_c)	N/mm^2	7.5
	Thickness (t)	mm	10
	Tensile strength, (σ_t)	N/mm^2	0.15
Interface properties	Normal stiffness modulus (K_n)	N/mm^3	14
	Shear stiffness modulus (K_t)	N/mm^3	62

5.3.2 Understanding and analysis methodology

It was known that the boundary walls were directly supported on strip foundations bearing on the same ground that started settling when the collapsible stratum lost its structural strength under seepage influence. However, the observed deformation pattern of the boundary wall, where the ends remained intact as the lowermost masonry courses sheared off, indicated that the wall ends were effectively tied and that self-supporting or interlocking mechanisms prevailed across most of the masonry courses. Also, in reality the entire soil underneath the boundary wall would neither commence settlement at the same time nor have a uniform settlement rate. Hence, in the first part of the analysis a hypothetical situation was assumed where the complete wall lost support due to settlement of the supporting soil below.

Therefore, to improve the calculation results, a further analysis was carried out properly considering soil-structure interaction influences. The interaction meant that, as the soil support was gradually lost below the wall base, stresses within the wall were redistributed such that more load was transferred to the end ties, with the wall increasingly mobilising its own self-supporting capability until the mortar joints failed. These mechanisms were modelled using a non-linear structure analysis module of *Midas*^{TM} by specifying input values of incremental wall self-weights and performing calculations to monitor the consequent load transfer and deformation response of the wall. In the analysis, the wall end constraint conditions were defined as “pinned” before imposing self-weights in 20 equal steps, each equivalent to 5% of the actual weight of the wall. The results obtained from both the geotechnical and structural models presented and discussed in chapter-6.

CHAPTER 6 - RESULTS AND DISCUSSIONS

6.1 Introduction

This chapter presents the results of the laboratory tests and the numerical analyses undertaken to deliver the objectives of the research. Important observations and discussions are also presented and principal research findings interpreted and explained in three different headings, namely:

a. The results associated with experimental setup-I i.e. laboratory plate load tests conducted in a custom designed tank.
b. The outcomes related to experimental setup-II i.e. laboratory based collapse tests carried out in metal moulds along with corresponding finite element modelling.
c. The results obtained from detailed geotechnical 3D finite element analysis of a twin-villa complex and structural modelling of distresses in boundary walls.

6.2 Results of experimental set-up – I

In the following sections, the full range of data collected from the laboratory plate loading tests is explained in detail.

6.2.1 Plate load tests – Full collapsible soil

As stated previously, it was of paramount importance for the plate tests to be conducted with water depths selected to be consistent in scale with the width of foundations in UAE case study sites where ground settlement and consequent structural damages were experienced. The ratio of width of foundation and depth of groundwater table in the lab was as in the field. Accordingly the plate load tests were carried out at different water levels i.e. simulated water table in the tank at 2.5B (500 mm), 1.5B (300 mm) and 1.0B (200 mm) below the bottom of a 200 mm dia. (B) test plate. Data from the plate load tests were transferred into Microsoft Excel workbooks for further processing in a bid to study the underlying patterns. Graphs of pressure against settlement and of settlement versus time were plotted and are discussed in the following sections.

6.2.1.1 Effect of dripping water on settlement of soil

Pressure–settlement graphs for all three tests carried out are shown in Figures 6.1, 6.2, and 6.3. Wetting cycles were commenced soon after the pressure on the plate reached from 0 to 80 kN/m^2. Details of wetting cycles can be seen in data sheets of tests A1, A2 and A3 in appendix-A, in which from each ‘water started’ to water stopped’ shall be counted as one cycle. Also, the number of wetting cycles before collapse is summarized in Table 6.1. It

was observed that the number of wetting cycles required for the soil to reach collapse state increased with increase in the depth of the water table below the plate (Table 6.1). This could be attributed to the presence of a deeper zone of soil (2.5B) involved in the collapse mechanism when the water level was at 500 mm (2.5B) below the plate, where the number of wetting cycles needed to cause soil collapse was greatest in comparison to the other cases. It was also apparent that the further the location of the collapsible soil zone below the plate foundation the greater was the number of cycles of wetting necessary to initiate soil collapse. It shall be noted that magnitude of collapse settlement at constant pressure (80 kN/m^2) is the numerical difference between the plate settlement at start of wetting cycles and at the end of test. Example: In Figure 6.1, collapse settlement is 13.75mm obtained by deducting 2.02mm from 15.77mm.

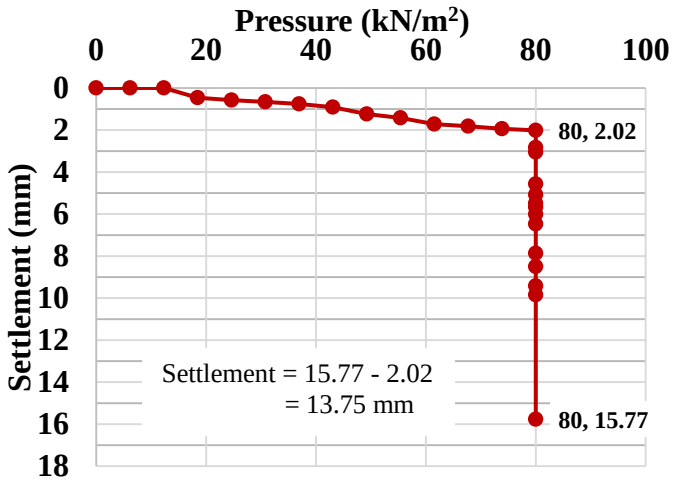


Figure 6.1 Pressure-Settlement curve with groundwater table at depth of 2.5B

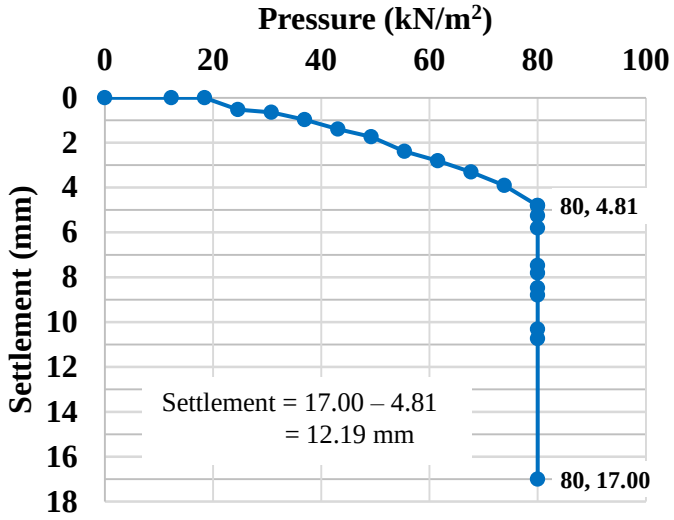


Figure 6.2 Pressure-Settlement curve with groundwater table at depth of 1.5B

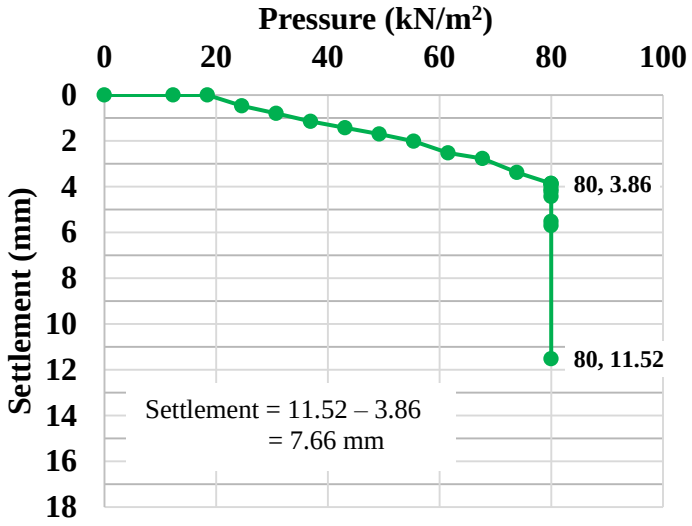


Figure 6.3 Pressure-Settlement curve with groundwater table at depth of 1.0B

Table 6.1 Wetting cycles before collapse

Depth of groundwater level below foundation	Number of wetting cycles
2.5B	7
1.5B	5
1.0 B	4
'B' refers to width of foundation (diameter of plate in the plate load test)	

6.2.1.2 Effect of time on settlement of soil

Time–settlement graphs for all three tests carried out are shown in Figure. 6.4. The graphs illustrate that the time required for the soil to exhibit collapse behaviour increased with increasing thickness of the collapsible soil below the plate foundation.

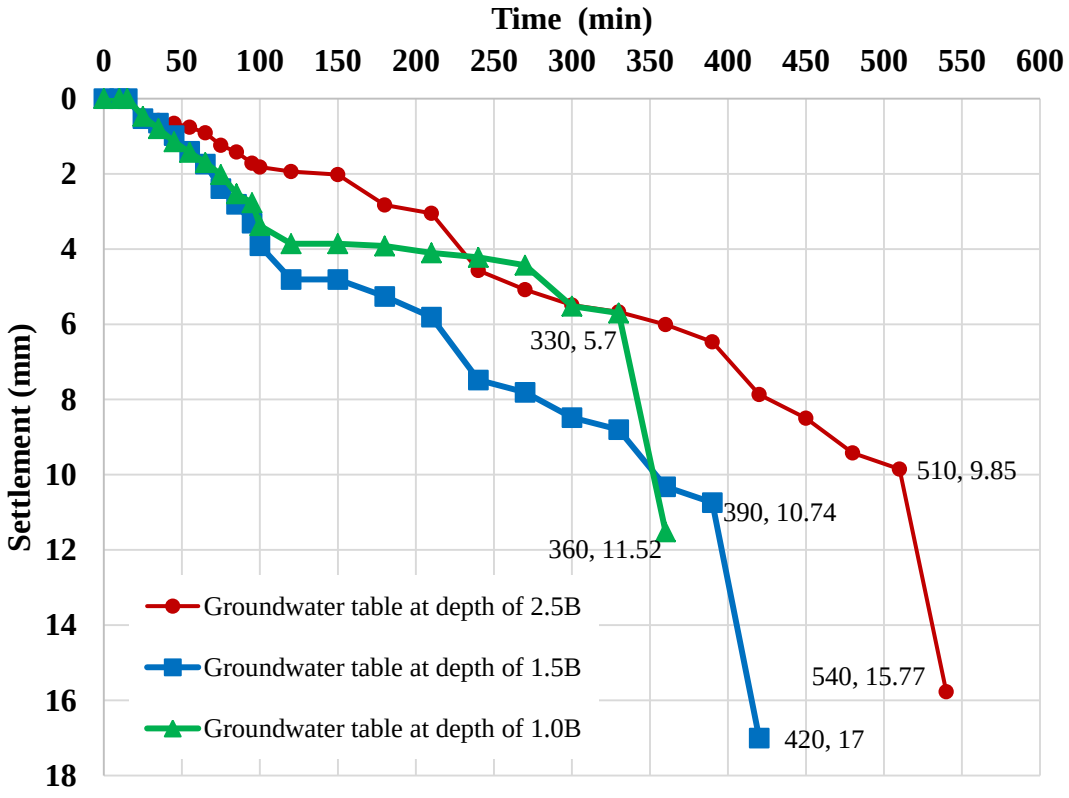


Figure 6.4 Time-Settlement curves at various groundwater levels

The time durations from commencement of test to the onset of soil collapse are listed in Table 6.2 for brevity and ease of understanding. A linear behaviour is evident from the data in Table 6.2, so that a 0.5B increase in depth of water is equivalent to a time gap of 60 min between the start of test and the onset of soil collapse.

Table 6.2 Time taken to achieve soil collapse

Depth of groundwater table	Time (minutes)
2.5B	510
1.5B	390
1.0 B	330
'B' refers to width of foundation (diameter of plate in the plate load test)	

6.2.1.3 Rate of collapse

Each plate load test was terminated once the rate of settlement became so rapid that the prime objective of maintaining constant pressure could not be achieved. To interpret and quantify the rate of collapse, the time–settlement data from the last wetting cycle in each test was used to calculate the rate of collapse. The calculations and corresponding results are shown in Table 6.3. For example, in the case when the depth of groundwater table was 2.5B, the collapse settlement of 5.92 mm is obtained as numerical difference between 15.77mm settlement at the end of the test and 9.85 mm settlement at the start of last wetting cycle (Figure 6.4). It is evident that irrespective of the thickness of the collapsible soil below the base of the plate, the rate of collapse exhibited by the soil in all three tests was fairly uniform at 6 mm in 30 min (0.2 mm/min).

Table 6.3 Settlement rate calculations

Depth of groundwater table	Settlement of soil before the start of collapse (mm)	Settlement at the end of test (mm)	Time between start of collapse and end of test (minutes)	Collapse Settlement (mm)
2.5B	9.85	15.77	30	5.92
1.5B	10.74	17.00	30	6.26
1.0 B	5.70	11.52	30	5.82

6.2.1.4 Effect of loading–reloading on modelled groundwater table

Each time, once the soil was removed and replaced after drying, there was a drop in water level in the piezometer when weight was placed on the soil. To examine this behaviour, moisture content and specific gravity of soil were determined after removing the soil before replacing it with dry soil and was found to be 12% and 2.6 respectively. Compaction test (BS 1377-4:1990) was conducted on collapsible soil and the resulting

curve plotted as shown in Figure 6.5. It was apparent that the soil was not compacted to maximum dry density (MDD) although it was on the path towards attaining it, with placement of more soil over. Calculations to support this observation are shown below.

From compaction test (Figure 6.5),

Optimum moisture content (OMC) =15.5% and

Maximum Dry Density (MDD) =18.45 kN/m^2

Using the formula, $\gamma_d = G\gamma_w/(1+e)$ (6.1)

Where,

γ_d = Dry density of soil

G = specific gravity of soil

γ_w = Density of water

e = voids ratio of soil

Substituting values in equation 6.1,

18.45 = (2.6 x 10)/ (1+e)

e=0.41

Therefore, voids ratio at maximum dry density is 0.41

Using the formula, $s = \frac{wG}{e}$ (6.2)

Where,

S = Degree of saturation of soil

w = Moisture content of soil

G = Specific gravity of soil

e = voids ratio of soil

Substituting values in equation 6.2,

S = (15.5/100) x 2.6/ 0.41

= 0.983 (98.3%)

Therefore, at optimum moisture content, degree is saturation = 98.3%

From Fig.6.5, at 12% moisture content, dry density of soil, γ_d=17.8 kN/m^3

Using equation 6.1, $\gamma_d = G\gamma_w/(1+e)$

Upon substitution of values,

17.80 = (2.6x10)/(1+e)

e=0.46

Therefore, at dry density of 17.8 kN/m^3, voids ratio is 0.46

Now, using equation 6.2, $s = \frac{wG}{e}$

Upon substitution of values,

S = (12/100)x2.6/0.46

= 0.68 (68%)

Therefore, at 12% moisture content, degree is saturation = 68%

From the calculations it was inferred that the downward movement in water level due to placement of weight (soil) was attributable to the relief of pore pressure in the voids within the partially saturated soil (S = 68%) as it transited to a fully saturated condition.

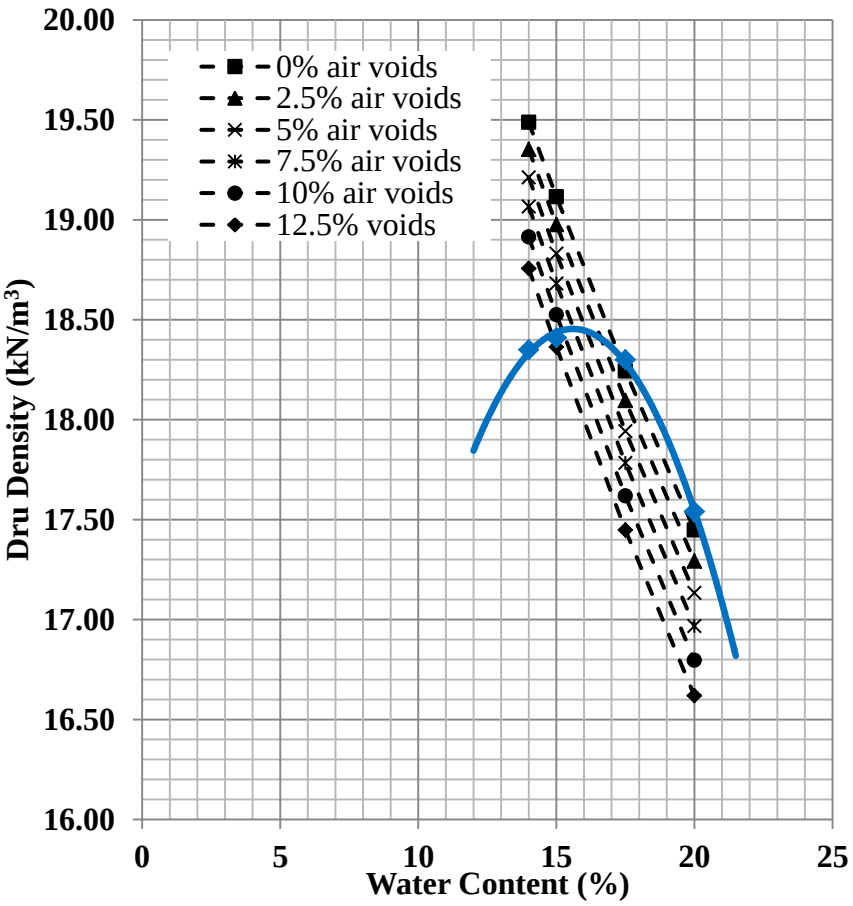


Figure 6.5 Compaction curve

6.2.2 Plate load tests–Collapsible soil as a layer

In this part of the work, permeability tests (BS 1377-5:1990) and laboratory plate load tests were conducted for a layered soil profile containing a collapsible soil lens, of variable thickness, inserted at mid-depth of the soil stratum below the plate. The layered soil profile acted as a bearing medium to simulate ground support for a superstructure. The results from all permeability tests are plotted against the thickness of collapsible soil layer in order to understand its behaviour. Results from all plate load tests were also presented graphically. For this purpose various pressure–settlement and time–settlement graphs were constructed and are discussed in the sections below.

6.2.2.1 Effect of permeability on thickness of collapsible soil layer

It is observed from Fig. 6.6 that there is marginal decrease in permeability (from 9.04×10^{-5} m/s to 7.22×10^{-5} m/s) of the soil with increase in thickness of the collapsible layer. This could be attributed to the very low (mostly negligible) increase in density of collapsible soil owing to inward movement of water.

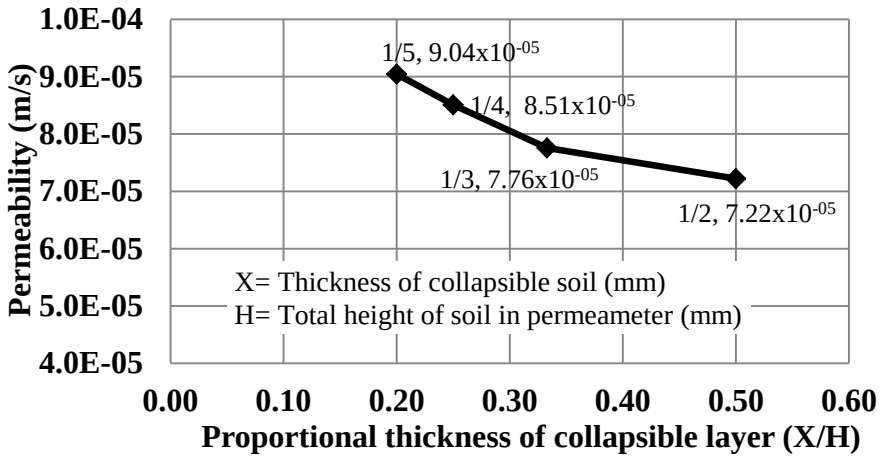


Figure 6.6 Thickness of collapsible layer versus permeability

6.2.2.2 Effect of dripping water on settlement of soil

It can be seen from the graphs in Figures 6.7, 6.8, 6.9, and 6.10 that settlement decreases with decreasing thickness of the collapsible soil layer. This is consistent with expectation since the collapsible layer (rather than other layers) which is sensitive to saturation and deforms more the thicker it is, for a given overburden pressure. The relationship between final settlement (obtained from Figures 6.7 - 6.10) and thickness of collapsible soil is illustrated by the graph in Figure 6.11. The variation trend line is represented by equation (6.3), which can be applied to a real problem in predicting settlement due to collapsible soil behaviour, provided the proportionate thickness of the collapsible soil is known.

$$y = 2x10^{-5}X^2 + 0.0176X + 1.1267 \qquad (6.3)$$

where, X = thickness of collapsible soil (mm); y = settlement (mm)

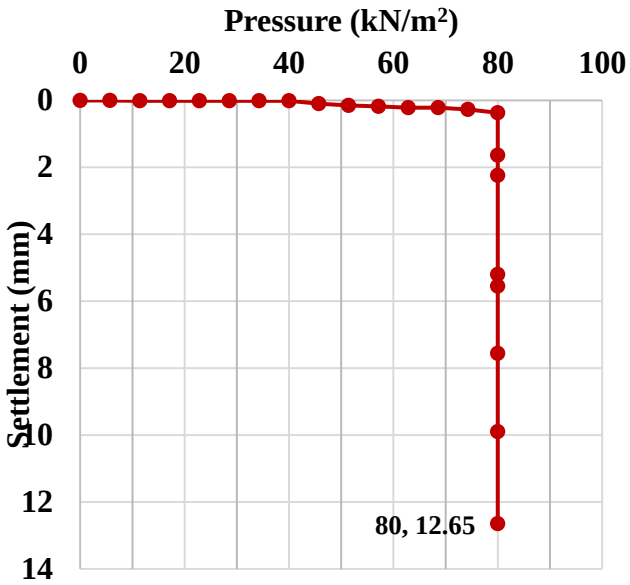


Figure 6.7 Pressure versus settlement with collapsible soil at central half of the tank

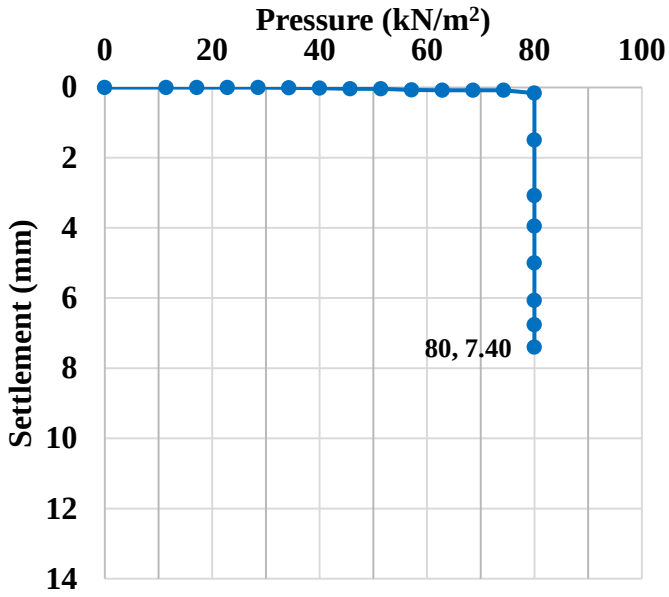


Figure 6.8 Pressure versus settlement with collapsible soil at central 1/3rd of the tank

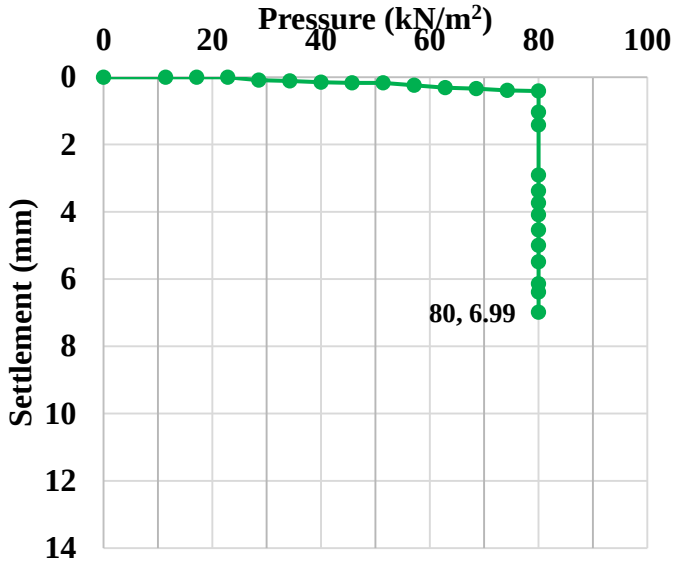


Figure 6.9 Pressure versus settlement with collapsible soil at central 1/4th of the tank

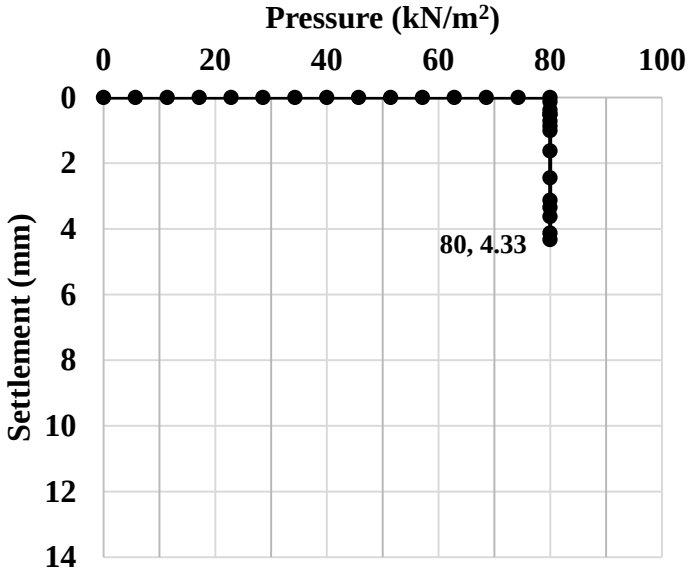


Figure 6.10 Pressure versus settlement with collapsible soil at central 1/5th of the tank

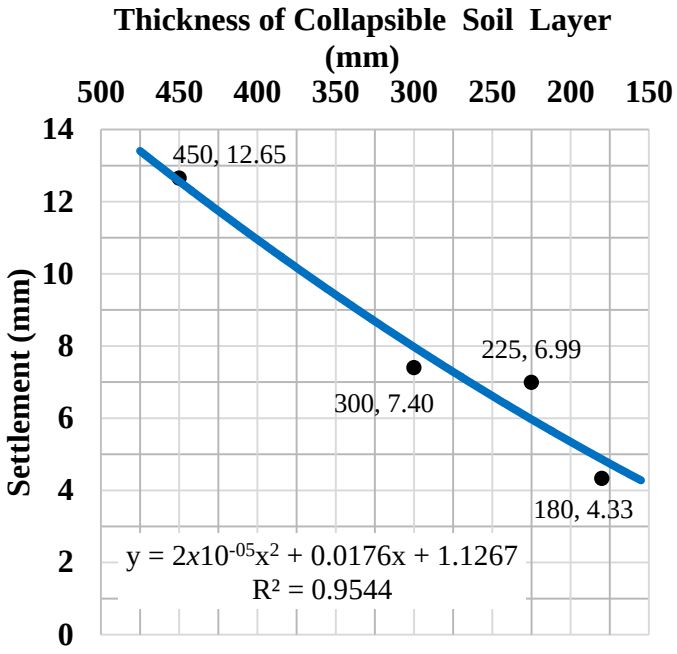


Figure 6.11 Variation of settlement with decrease in thickness of collapsible soil layer

6.2.2.3 Effect of time on settlement of soil

Time–settlement graphs for all plate load tests conducted are shown in Figure 6.12. It is seen that in all cases, settlement increases with time but at different rates depending on the position and thickness of the collapsible soil relative to the tank depth. This is again attributed to the proportionate influence of collapsible soil responsible for settlement.

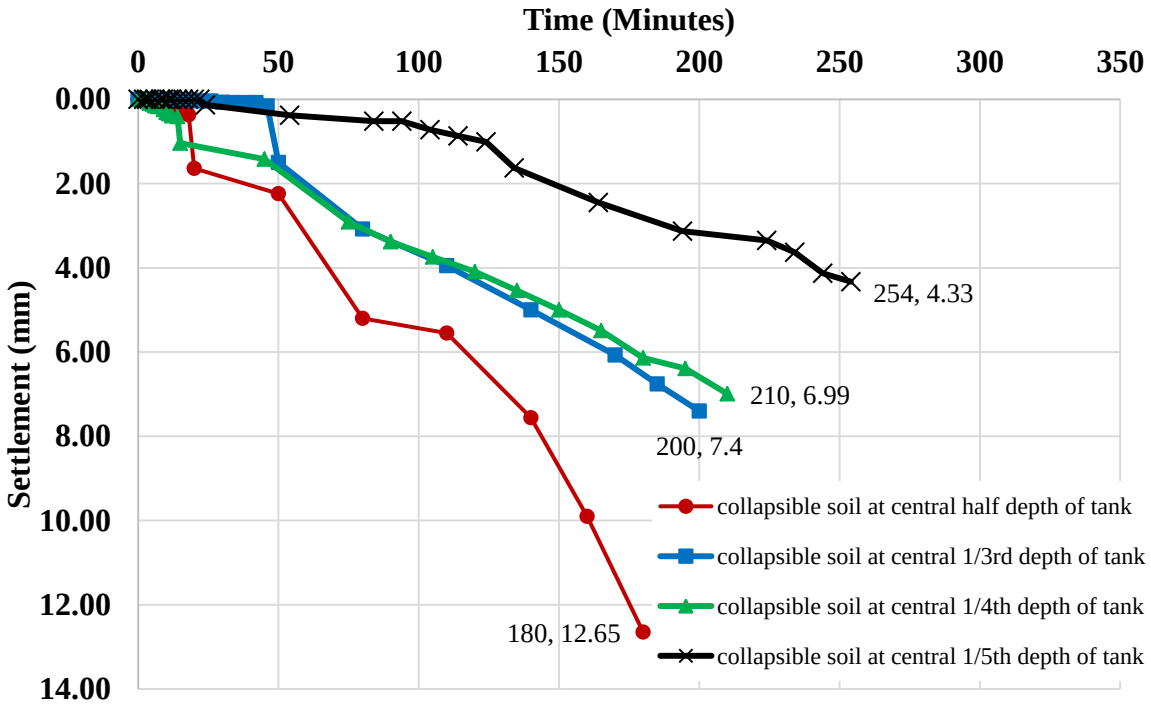


Figure 6.12 Time versus settlement with various collapsible soil thicknesses at central depth of tank

The relationship between times at which collapse tests were terminated (e.g. 180 min shown by red curve of Figure 6.12) and thickness of collapsible soil is shown in Figure 6.13, where the trend of variation represented by equation (6.4).

$$y = 1429.9X^{-0.343} \quad (6.4)$$

where, y = time (min) at which test ends; X = thickness of collapsible soil (mm)

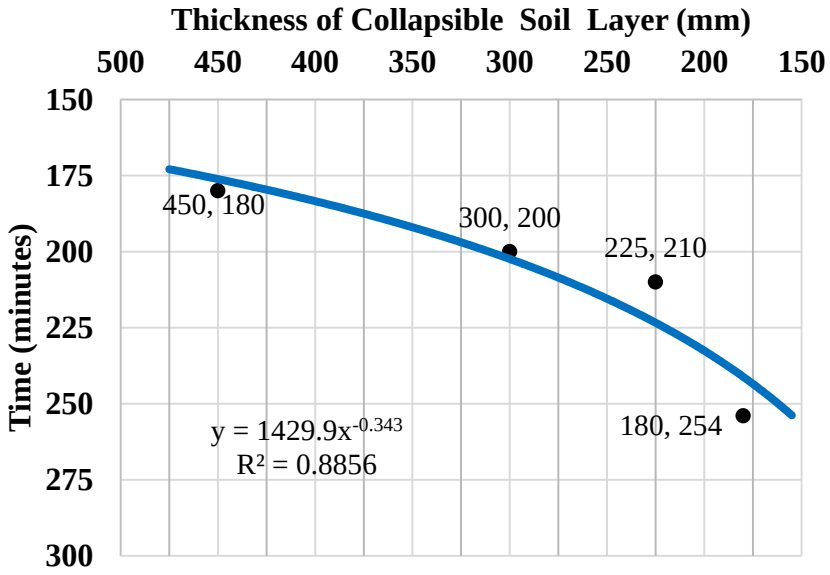


Figure 6.13 Thickness of collapsible soil layer versus time

Equation (6.4) may be used to predict the time taken by the soil to exhibit final settlement at the end of the test (when the settlement is so rapid that the objective of maintaining constant pressure could not be achieved) if proportionate thickness and overburden pressure on the collapsible soil is knownp.

Test details along with observed laboratory readings of all plate load tests carried out as part of this research work are included in Appendix-A.

6.3 Results of experimental set-up – II

Data obtained from the 36 test runs were presented in graphical form typifying trends of variation between:

(i) Surface settlement due to rise in water level only and normalized water depth (water table factor), for each of the three compacted densities and for each of the four soil strata combinations (Figure 6.14).

(ii) Surface settlement due to rise in water level only and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 6.15).

(iii) Surface settlement due to drip irrigation only and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 6.16).

(iv) Surface settlement due to combined rise in water level and drip irrigation and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 6.17).
(v) Average surface settlement due to rise in water level only and thickness of collapsible layer (Figure 6.18).
(vi) Average surface settlement due to combined rise in water level and drip irrigation and thickness of collapsible layer (Figure 6.19).

For purposes of normalization, the 'water table factor' was defined as the ratio of height of water table surface above the base of soil column to the overall thickness of the soils in the mould. Thus, the water table factor is plotted as a dimensionless quantity.

6.3.1 Variation of settlement with normalized water table depth for various soil densities

As can be seen in Figure 6.14 for all compacted densities, the soil settlement increased with increasing depth of the water table. This was attributed to an increasing proportion of soil mass gaining higher saturation degrees due to gradual ingress of water. Also, at any density level, settlement increased with increasing thickness of the collapsible soil within the profile. This was attributable to a correspondingly greater zone of collapsible soil being influenced by the infiltration water. In addition, it can be seen that in overall terms, increase in the compacted density resulted in decrease in settlements. This was anticipated because the low air voids in the dense soil obviously meant decreased potential for the particles to re-adjust or deform further upon ingress of water.

Furthermore, of all the soil profile combinations, the maximum settlement of 7.72 mm was observed in SC-1, at water table factor of 2/3, highest thickness of collapsible soil layer and maximum water table height. Thus this may be regarded as the most critical combination of factors for the collapsible to settle the most. For this case, it was observed that with a density increase from 17.5 to 18.5 kN/m^3 the settlement decreased by a factor of 1.8 (7.72 to 4.29 mm). The observation here suggests that the *in-situ* density of a collapsible stratum is crucially important in influencing the stability of the soil structure and hence settlement potential. For this reason it is imperative that application of deep compaction methods to enhance soil density is likely to be the most effective ground improvement technique to reduce settlement problems related to collapsible soil strata under the influence of water.

6.3.2 Variation of settlement with normalized water table depth for average compacted soil density

The graph in Figure 6.15 represents the variation trend for settlement versus water depth for averaged soil density. It can be seen that in general, settlement still increased with increasing water table depth as was observed for different densities in Figure 6.14. However, there was no significant difference in settlement in profile cases SC-3 and SC-4 at a normalized water depth of 1/3. This happened because, despite the differences in the thickness of collapsible soil layers in cases SC-3 and SC-4, the water level was still below the collapsible stratum hence unaffected by it. However, the slight increase in average settlement from 1.35 to 1.41 could be attributed to the capillary rise of water due to the close proximity of the collapsible soil to the water level.

6.3.3 Variation of settlement due to drip irrigation with water level rise

Figure 6.16 serves to show collapse settlement due to drip irrigation continues beyond the level associated with water level depth. Further settlements as drip irrigation continued was expected because once the soils below the water table had reached collapse stage, the soil particles above the water table were still increasingly being moistened by the irrigation water, hence resulting in additional collapse. It can be seen in Figure 6.16 that due to drip irrigation alone, the settlement decreased with increasing water table factor. This contrasts sharply with the previous observation that settlement due to rise in water table alone increased with increasing water table factor. The reason was that when large portions of the collapsible layer were already under water, there was no significant increase in settlement under continuing drip irrigation because only the upper layer could give additional compression yet this layer was thin and less saturated.

6.3.4 Variation of settlement due to combined effects of water level rise and drip irrigation

The combined effect of rise in water table and drip irrigation on settlement on soil is shown in Figure 6.17. Here, the settlement behaviour is essentially similar to that due to rise in water table only. Thus it is apparent that settlement of collapsible soils is influenced much more by the water table depth than by irrigation process, provided that much of the layer is already submerged.

Test details and observed laboratory readings of all collapse tests carried out in metal moulds are included in Appendix-A.

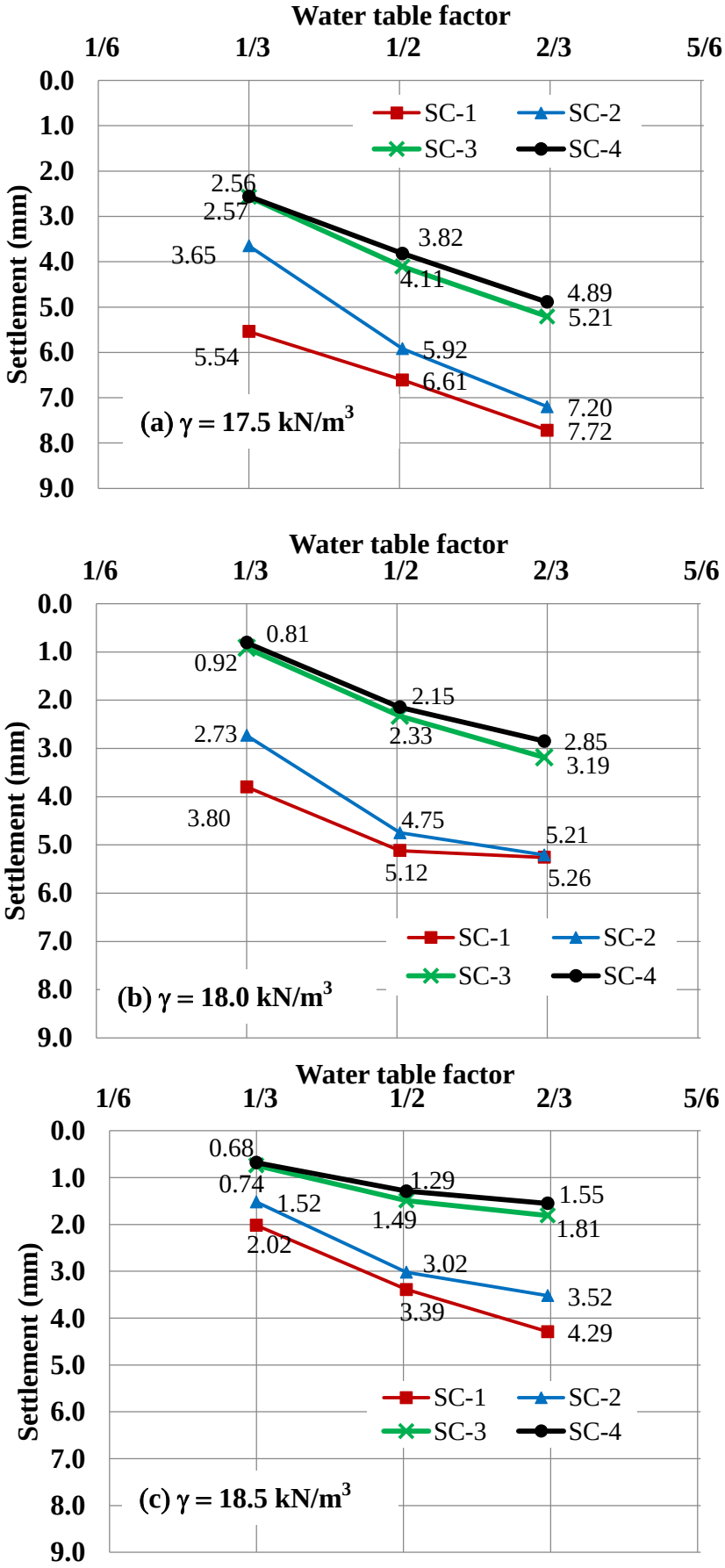


Figure 6.14 Variation between soil settlement due to water table rise and water table factor (results for different soil densities: 17.5 -18.5 kN/m^3)

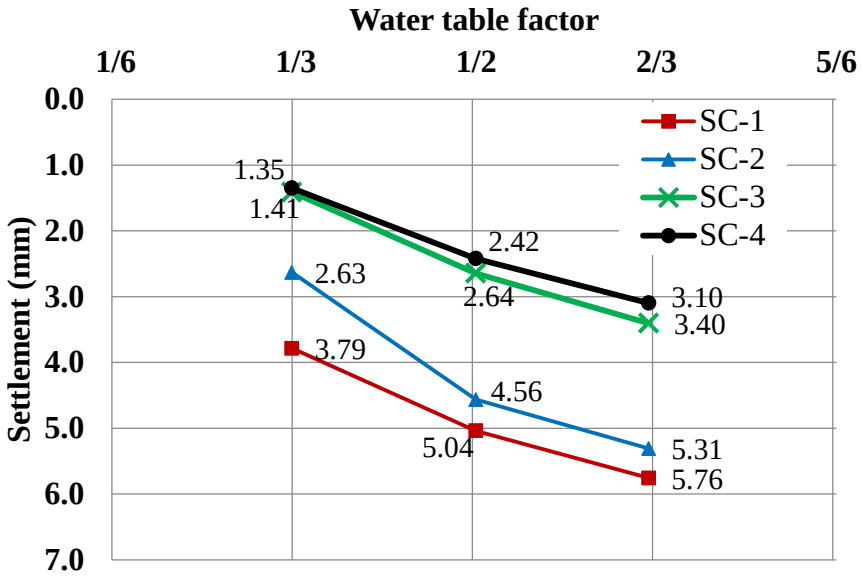


Figure 6.15 Variation between average soil settlement due to water table rise and water table factor

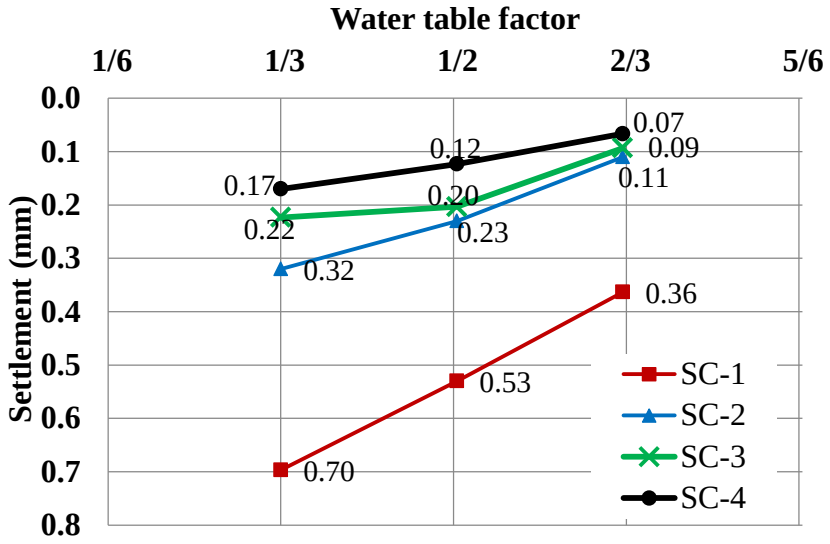


Figure 6.16 Variation between average soil settlement due to drip irrigation and water table factor

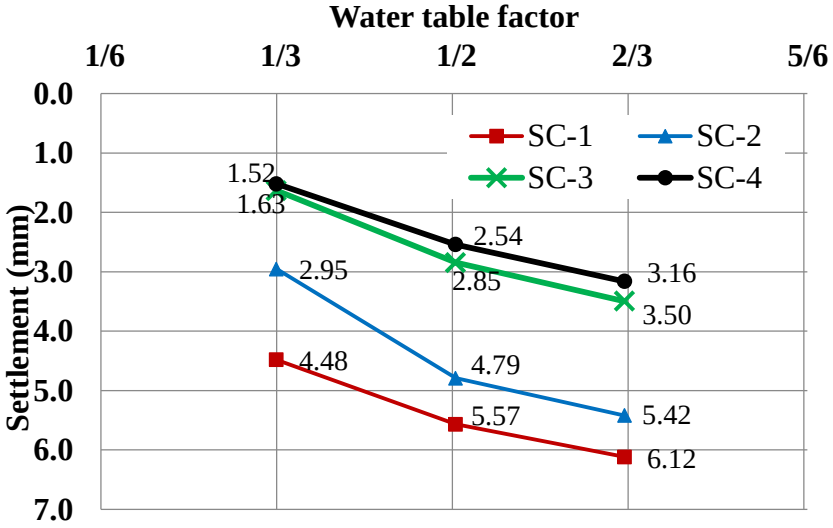


Figure 6.17 Variation of average soil settlement with water table factor due to the combined effects of water table rise and drip irrigation

6.3.5 Settlement predictions

It can be seen from Figures 6.18 and 6.19 that there is an increase in settlement with increase in the thickness of the collapsible layer. This happens due to water table rise alone (Figure 6.18) as well as due to combined rise in water table and drip irrigation (Figure 6.19). Under the combined influence of water table rise and drip irrigation, the surface settlement increases with decreasing density of soil, irrespective of the thickness of collapsible soil. A similar pattern of behaviour is exhibited for higher thickness of collapsible stratum (120 mm), due to rise in water table alone. It is seen that, at lower thicknesses (60 and 90 mm), the settlement behaviour is markedly different. This is attributable to the fact that the water table rise now affects only a partial zone of the collapsible layer, rather than the full height of the layer. With more extensive data points, curve fitting techniques can be used to model distinct trends of variation between thickness of collapsible soil and average surface settlement, for effects of: (a) rise in water table alone and (b) combined rise in water table and drip irrigation. The models can then be applied to real problems in predicting settlement, for known thickness and properties of the collapsible layer. Settlement due to drip irrigation alone can be predicted as the difference between the corresponding values modelled from Figures 6.18 and 6.19.

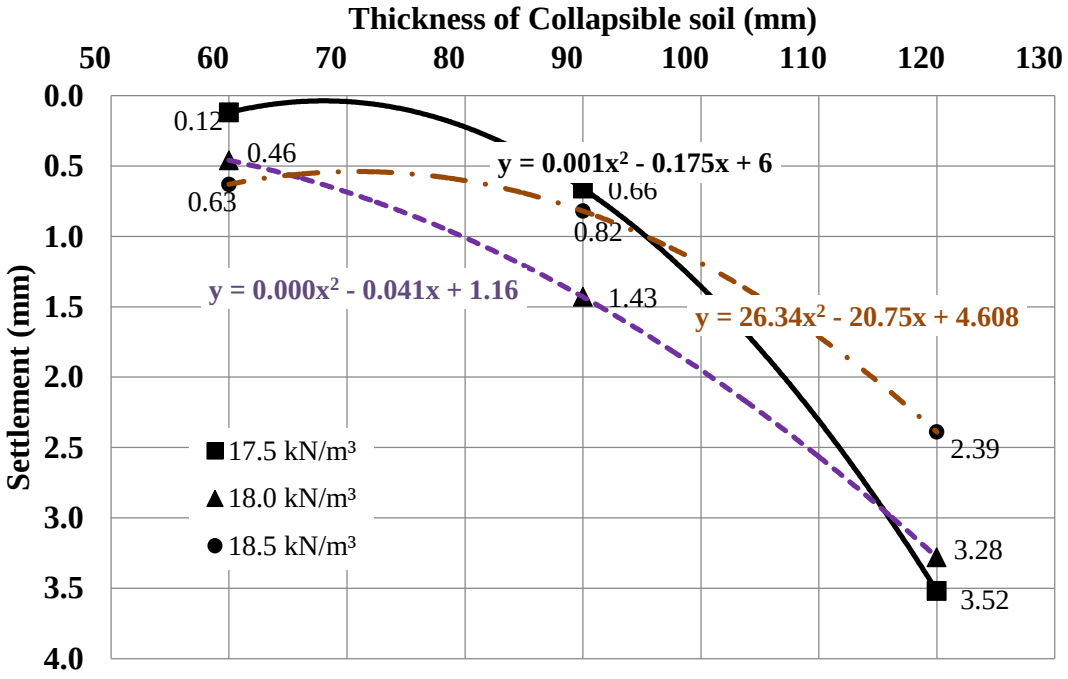


Figure 6.18 Variation between average settlements due to rise in water table and with thickness of collapsible soil

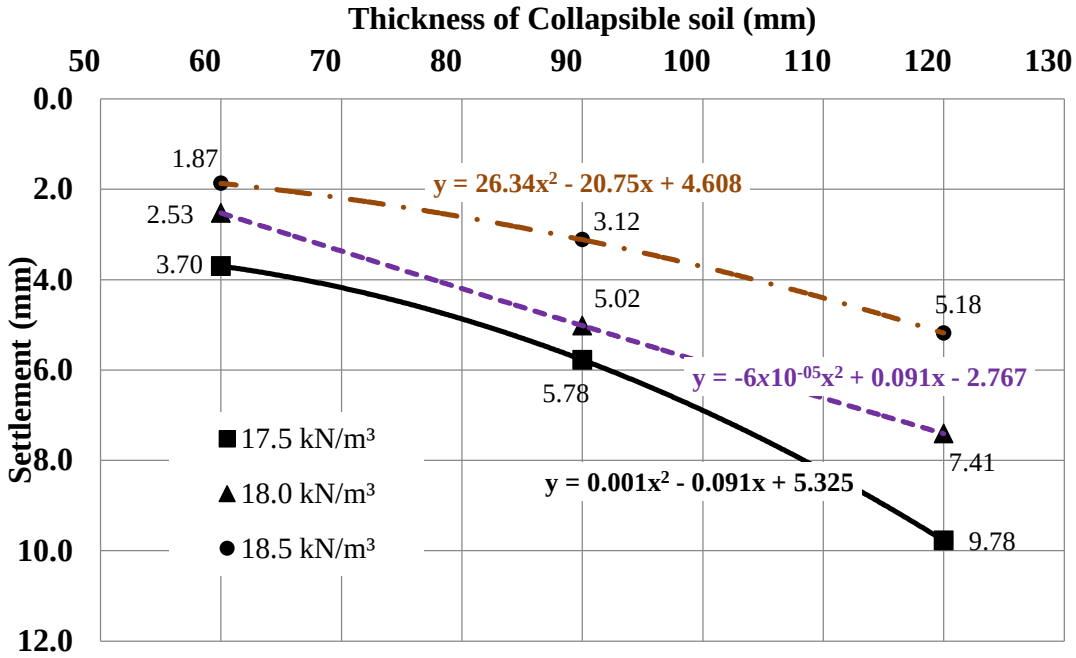


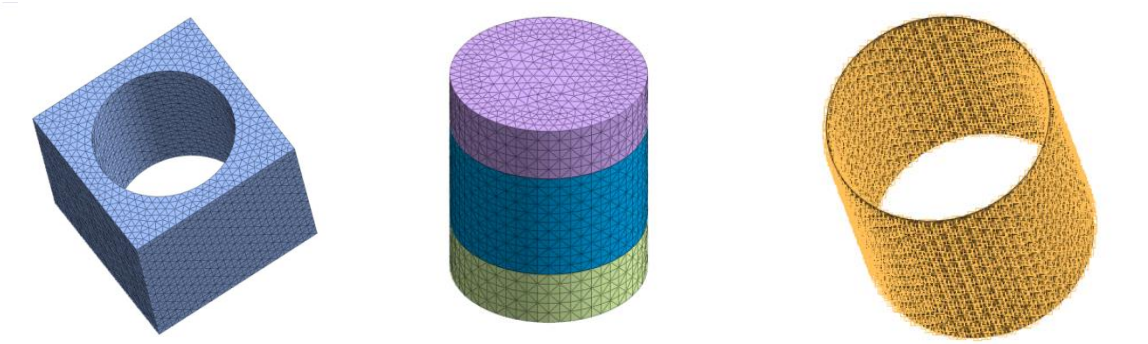
Figure 6.19 Influence of thickness of collapsible soil on average settlement due to combined effects of water table rise and drip irrigation

6.3.6 Finite element modelling to verify lab results

A fully coupled stress and seepage finite element analysis of the laboratory model of the 3-layer profile was carried out using *Midas*^{TM} *GTS NX* software. This was the most reasonable way to consider the important factors, such as the interface friction between the metal mould and soil, influencing the settlement response of the collapsible layer of soil under maintained load and infiltration of seepage water. Although a total of 36 test runs were conducted in the laboratory, finite element analysis was carried out only for the critical case (SC-1) that was associated with maximum surface settlement (7.72 mm). Case SC-1 had the largest thickness (90 mm) of collapsible soil, lowest soil density (17.5 kN/m^3) and highest water table (i.e. water table factor of 2/3). To simulate friction between the soil and mould, the strength reduction factor was set as R=0.65, with automatic calculation of the normal stiffness modulus, K_n and shear stiffness modulus, K_t hence influencing the output parameters (Figure 6.20). Analysis was conducted in multiple trials by continuously refining the mesh until the computed result was insensitive to mesh fineness. As shown in Figure 6.14(a), the measured surface settlement due to rise in water table alone was 7.72 mm, whereas the finite element result was 8.57 mm (Figure 6.21) implying an error only +11%. This could be attributed to the intrinsic limitations of the material model, uncertainty of soil parameters owing to natural variability and the numerical approximations involved in finite element analysis. Therefore it was to be expected that calculated results would perfectly match the measured. A parallel analysis run without incorporating the friction parameter proved that any influence of soil-mould interface friction on the computed results was negligible. The apparent insensitivity of the

settlement at the centre to mould-soil friction gave confidence that the mould diameter was large enough to remove boundary effects. Hence there was settlement nullification at the mould edges when water table was varied due to the discharge from the surface drippers.

It can be seen that the average settlement (for all three densities tested) due to drip irrigation alone, for SC-1 with water table factor of 2/3 is 0.36 mm (Figure 6.16), while the settlement for 17.5 kN/m^3 density case is 0.54 mm. Although the layers above and below the collapsible lens were free-draining materials, in reality the volume change (however small) of these layers have an influence on the measured surface settlement. To take this factor into account, the settlement value of 0.48 mm (Figure 6.22) at the top surface of the collapsible layer in the model is 0.06 mm less than the 0.54 mm mentioned above. This translates to a consistent error of -11%, which has already been discussed in earlier sections of this thesis. Once confidence was established that calculated and measured results were reasonably close, it was considered useful to formulate empirical relationships for use in predicting ground settlement as a function of the collapsible soil thickness and properties, water table depth and irrigation intensity for given overburden conditions. All modelling parameters, replication details, boundary conditions and results obtained at various stages of this simulation process are included in Appendix-B.



a) Mould containing soil layers

b) Soil layers

c) Friction component between soil and metal mould

Figure 6.20 Meshed models of the components of the test set-up

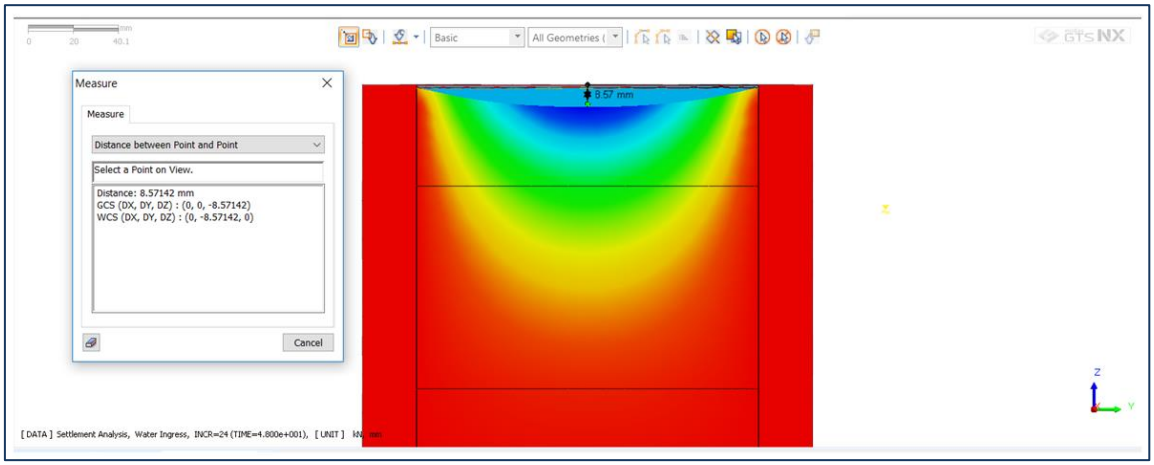
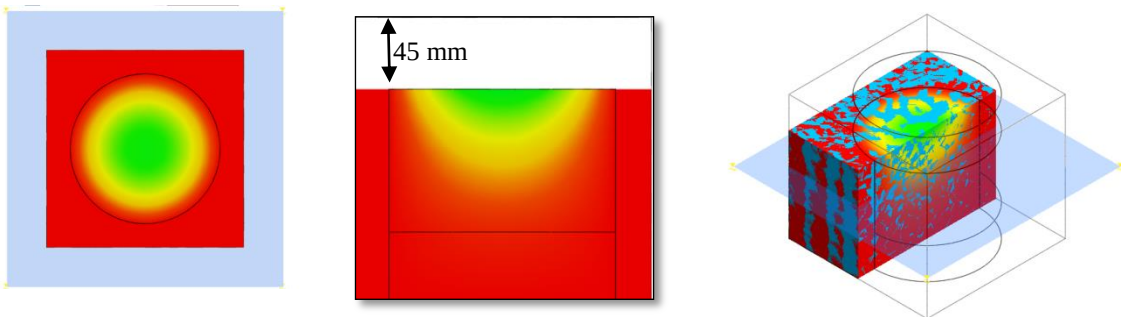


Figure 6.21 Surface settlement contours (SC-1, water table factor = 2/3)



Settlement = 0.48 mm

a) Top view b) Side view c) Isometric view

Figure 6.22 Settlement contours at top of collapsible soil layer (45 mm below surface) for SC-1 with water table factor of 2/3

6.4 Results of finite element analyses

In the subsequent sections, results obtained from geotechnical and structural finite element modelling and analyses are presented and explained in detail.

6.4.1 Results and discussions-Geotechnical modelling

From the finite element results, the ground settlement beneath the boundary walls at three different water depths, viz. 1.5m, 2.0m and 3.0 m were summarised. Figure 6.23 maps out a specimen result of magnitudes of ground settlement beneath a boundary wall at the end of the 17th irrigation cycle, which corresponds to a water table depth of 1.5m.

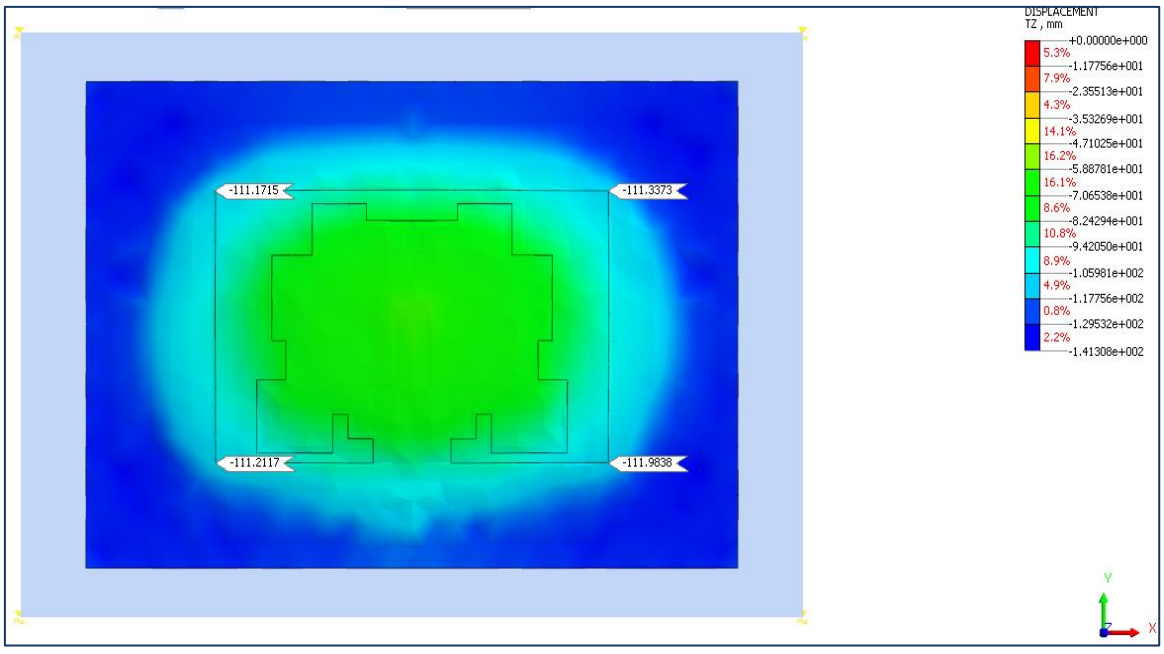


Figure 6.23 Settlement of soil under boundary wall at the end of 17th irrigation cycle with ground water at 1.5m depth

Figure 6.24 shows the calculated trends of variation of settlement beneath boundary wall versus number of irrigation cycles, for three particular water table levels. It is evident that the number of irrigation cycles required for the supporting ground to exhibit total collapse increases with increasing water table depth. The observed abruptness of bearing capacity loss, coupled with strong sensitivity to water table position, is an indication of the presence of collapsible layer(s) in the soil profile. Similar trend was observed from laboratory tests on a collapsible soil sandwiched between two other layers and load tested under different water table levels while irrigation continued.

Figure 6.24 also reveals that, after sufficient wetting in 4-5 irrigation cycles, the ground surface settlement at the end of a given irrigation cycle increased with increasing water table depth. This evidences that once the collapsible stratum had been saturated sufficiently to fail with the ground water table (GWT) at a certain depth, there was very little additional settlement with increasing water table depth due to the relatively less sensitivity of the non-collapsible layers to water table rise. It is interesting to note that the calculated maximum settlement beneath the boundary wall was 157 mm, which compares favourably with the measured value of 165 mm on site. This gave confidence that the 3D finite element model and the assessed parameters are reliable and consistent with the real ground behaviour.

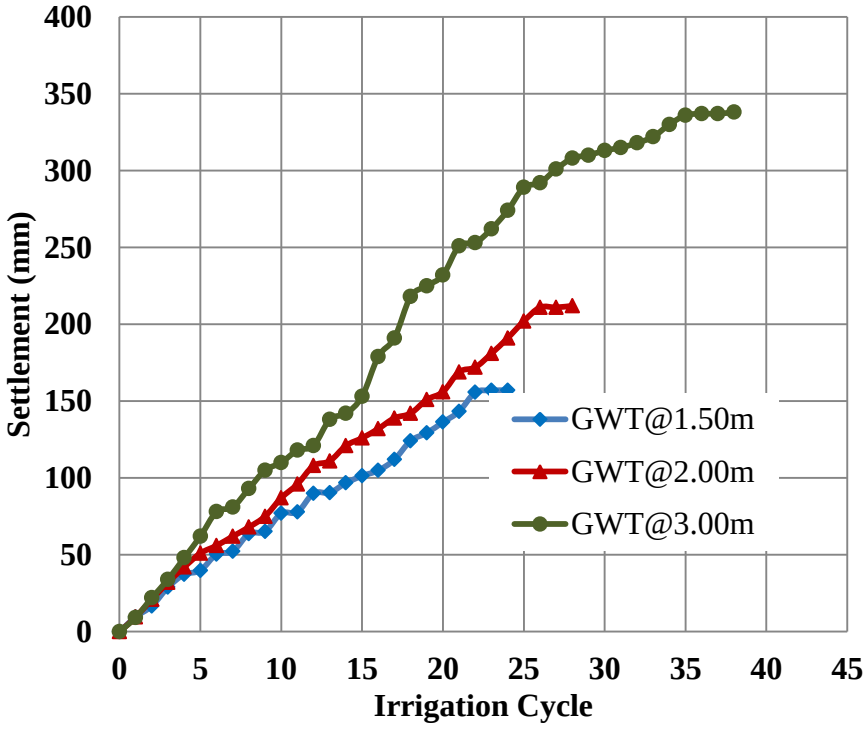


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Figure 6.24 Settlement versus irrigation cycles at various depths of groundwater table

6.4.2 Results and discussions-Structural modelling of boundary walls

Figure 6.25 shows the calculated maximum wall settlements corresponding to various increments of percentage self-weight. The plot shows a discernible bi-linear trend, with the wall settlement initially increasing at a marginal rate but once the percentage self-weight reached 35%, there the wall settlement increased suddenly from 0.7 mm to 15.65 mm. This is equivalent to a 22 times increase in settlement for a 5% increase in applied weight from 35% to 40%. Figure 6.26 shows the output deformation pattern of the wall at 40% weight increment corresponding to the drastic settlement increase. Essentially the wall had failed at this stage because of continuous divergence of subsequent calculation solutions and unrealistic settlement outputs producing incompatible failure patterns.

It can be seen that the predicted failure patterns of the wall (Figure 6.26) are similar to the site observations (Figure 3.7), where failure of mortar bedding joints caused complete dislocation of the lower masonry courses while other parts of the wall remained largely intact. The close agreement between the measured and predicted mechanisms gave confidence that the adopted finite element approach and parameter values reflect the real conditions at the UAE case study sites. Unsurprisingly, the structural distress was not due to rigid settlement of the wall as a unit but rather failure of the mortar joints in response to extreme settlements and redistribution of stresses in the wall and its ties

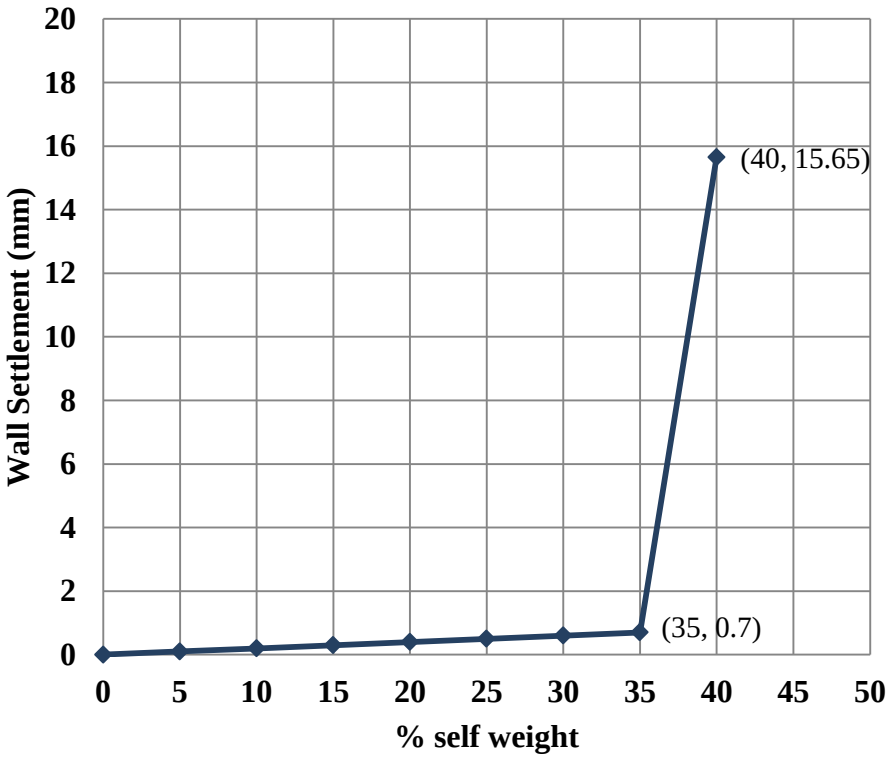

Figure 6.25 Wall settlements at various percentage of self-weights

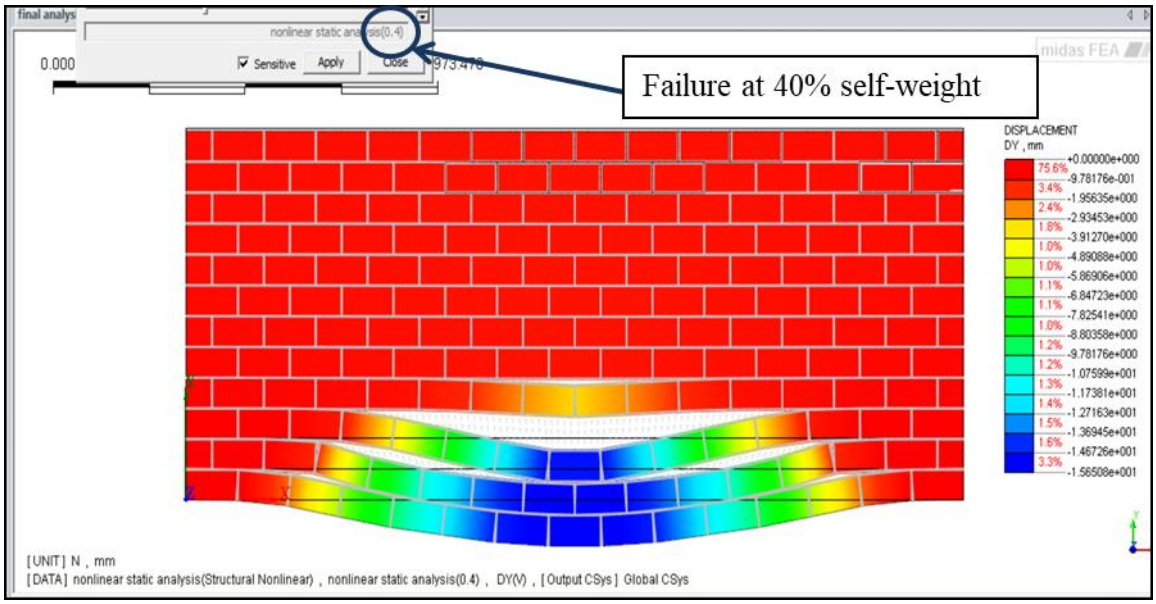

Figure 6.26 Failure pattern of wall at 40% self-weight

In summary, the foregoing sections of this chapter present measured results and corresponding observations made from the custom designed apparatus for laboratory testing of the collapsible soil when acting in isolation and also when part of a layered profile. The test systems devised as part of this research mainly concentrated on understanding the influence of water table as well as drip irrigation on the settlement response of a collapsible soil. Also, an effort is made to numerically simulate site through 3D finite element modelling to assess how irrigation-induced settlement of a collapsible stratum affected a typical villa boundary wall in the UAE case study (low-rise housing project, section 3.3). Additionally the tests provide a useful insight into the technical factors pertinent to the peculiar failure patterns observed at the UAE sites. The next chapter discusses the lessons learnt and recommendations made from the UAE case studies, supported with the extensive results from both the experimental and numerical analysis stages carried out in the present research.

Step-by-step procedures of geotechnical modelling of twin-villa complex and structural modelling of distressed boundary walls including formation of true model, parameters used, replication details, application of real field boundary conditions and results obtained at various stages of this simulation process can be seen in Appendix-B.

6.5 Discussion in wider context and practical applications

As pointed out already, the main drawback with most of the existing methods of analysis in estimating collapse settlement is reliance on lengthy and expensive testing methods. Another weakness of existing methods is the assumption of fully saturated conditions and gradual loading in laboratory soil tests, both conditions being inconsistent with the real

ground situations. For example in the UAE case study, where collapsible strata lying above water table were mostly dry hence unsaturated, collapsibility was a direct influence of external factors such as landscape irrigation, rainfall etc. In contrast, the present work sought to overcome some of the limitations of the existing methods through using laboratory simulation of irrigation infiltration and maintenance of an overburden stress on the stratum under test.

Now, it is readily appreciated that once structures are built, the loads they exert on ground will not normally change with time and this lends credence to the decision to conduct simulation tests with constant applied pressure maintained up to the collapse state.

In addition, the loose nature of desert soils such as those in the UAE case study means that samples will invariably be disturbed, yet the existing methods of analysis of collapsible soils rely on undisturbed soil testing. In contrast, the methodology proposed here eliminates the above requirement and so offers a distinct advantage. An additional advantage of the suggested method is its capability to accommodate a full-sized structure in a numerical model of collapsible strata.

Finally, the formulae derived here can be used by geotechnical engineers to assess the rate and magnitude of settlements, as functions of collapsible stratum thickness, water table, and overburden stress for a particular site. Though every effort has been made in the current study to prepare sufficiently large sized models to simulate field conditions relevant to the UAE case studies, inevitably there will be variations to be taken into account from one site to another. These variations include: the rate and frequency of irrigation, thickness of collapsible soil stratum and its depth below ground level as well as depth of groundwater table. Thus, geotechnical engineers need to exercise utmost care when assessing the important parameters such as time, rate and magnitude of collapse settlements in the particular locality of concern.

A reliable assessment of the relationship between the intensity of landscape irrigation, water table level, thickness and location of collapsible strata can enable geotechnical engineers to develop guidance for property owners/ members of the public to help them control rates of irrigation hence avoid extreme ground settlement that would cause structural distress of the kind reported in the UAE case studies.

The successful implementation of the full scale boundary wall of the villas, where measured and predicted wall deformations matched closely, paves the way for future adoption of the method in evaluating the influence of collapsing soils on similar structures. A similar model can then be used as a design tool when planning a development project on sites underlain by collapsible layers.

CHAPTER 7 – CONCLUSIONS AND RECOMMENDATIONS FOR COLLAPSIBLE SOIL SITES

7.1 Introduction

This chapter presents a concise summary of the thesis achievements, outlining its originality, timeliness and contribution to understanding of the subject area. Focus is on the success in addressing each objective, the lessons learnt and thoughts for further development of the work to accrue more benefits to the geotechnical engineering profession. In light of the new findings, the original research question is recapitulated, with careful consideration of the appropriateness of the research data and reasonableness of the conclusions inferred.

This research emanated from the need to find solutions to major ground instability problems that occurred at various sites within Abu Dhabi and Al Ain cities in the UAE. Swathes of land had been developed with commercial and residential properties, along with ancillary infrastructure such as access roads, pavements, security walling and pipelines. At the time of construction, little attention had been paid to the existence of collapsible strata beneath the sites and the potential problems that would be caused by continuous irrigation of lawns and verges to beatify and green areas. Therefore property owners were ignorant of how their irrigation activities could affect shallowly installed structures such as perimeter walls, footpaths and pipelines. So significant was the problem that authorities had to commission a large scale investigation by consultants to identify the reasons for the distresses and explore possible solutions.

It was revealed that damage to structures had been on-going for many years as the ground settled progressively and unevenly due to the effects of water percolating from surface irrigation into uniquely problematic strata now identified to be collapsible soils. The only structures which were unaffected were the major ones (villas) founded on pile foundations that transferred load past the collapsible layer into competent strata beneath. Furthermore due to the special characteristics of collapsible soils, settlement calculation formulae from consolidation theory are inapplicable. Thus, with the identified gaps in knowledge, the answer to the problem necessitated original research and hence created the opportunity for this doctoral work. The potential opportunities from the research and its timeliness are immense because the affected areas in the UAE are expansive, prime and still attracting heavy investment so the lack of innovative and improved design solutions would adversely affect the regional economy and local communities. Lastly objective comments are given on the applicability of the numerical predictions developed.

7.2 Research achievements and contribution to knowledge

This research demonstrates that if an arid site is underlain by collapsible strata, special considerations additional to conventional bearing capacity and settlement analysis are required when planning for shallow foundations. It is necessary to conduct both ground investigation and numerical analysis to not only detect and characterise water sensitive strata but also assess saturation related strength loss and consequent irreversible settlements. In this regard, the present research has succeeded in extending knowledge by developing methods of simulating the field response of a collapsible soil, formulating equations for predicting the onset of collapse and devising a numerical approach for analysing ground settlement when for an irrigated site underlain by collapsible strata. The solutions contributed here go some way in overcoming the limitations of existing methods, which rely solely on elastic settlement formulae without accounting for the all-important effect of saturation on a collapsible stratum.

7.3 Conclusions from plate load tests on collapsible soil

1. The number of wetting cycles required for the soil to exhibit the collapse increases with increase in depth of groundwater table below the foundation level.
2. Once the soil starts exhibiting its collapsible behaviour, the rate at which it collapses was found to be uniform irrespective of its thickness.
3. The decrease in water level in the tank due to loading the soil was attributed to relief in pore pressure within the voids of the partially saturated soil as it transited to a fully saturated condition.
4. There is a marginal decrease in permeability of soil stratum with increase in thickness of collapsible soil portion in it.
5. The magnitude of settlement increases with increased proportion of collapsible soil in a soil strata.
6. The time required for the soil to start exhibiting collapse increases with increasing depth of the groundwater table below the foundation. In addition, despite high magnitudes of ground settlement, the time required to attain the maximum settlement decreases with increase in the thickness of the collapsible stratum.
7. Predictive relationships were developed for linking the time period for maximum settlement to thickness of collapsible soil as well as magnitude of settlement to thickness of collapsible layer.

7.4 Conclusions from collapse tests in metal moulds

1. The surface settlement of the soil profile was found to increase with increasing water table factor irrespective of the density of the layers.

2. For all soil density values examined, the settlement at the surface was found to increase with increase in thickness of the collapsible layer in the profile.
3. The settlement decreased with increase in density of soil in such a manner that a 1 kN/m^3 increase in density of soil caused the surface settlement to decrease by a factor of 1.8.
4. In the absence of drip irrigation, the surface settlement increased with increasing water levels. However, under the effect of drip irrigation alone, the settlement decreased with increasing water table factor.
5. Modelled relationships between the magnitude of settlement and thickness of collapsible soil can be used to predict the magnitude of ground settlements in real field situations, provided the thickness of the collapsible soil layer and properties of other layers in the profile are available from borehole investigations.

7.5 Conclusions from finite element modelling and analyses

1. A numerically based analysis model was developed and applied with the aid of *Midas*^{TM} finite element software to enable prediction of irrigation-settlement of a soil profile containing collapsible strata. Proof of the applicability of the approach was demonstrated by validating the model against the observed data from the UAE case study (low-rise housing project, section 3.3). The computed settlements were found to be in close agreement with the measured ones, hence giving confidence that the proposed approach could be used as an advance assessment tool for sites underlain by collapsible strata. Computation results showed that the sudden loss of strength of the collapsible layer required the water table to reach a certain depth, which corresponded to a certain number of irrigation cycles. Further increase of water table depth would have increasingly less impact on settlement since the collapsible layer would have already lost its full inter-particle strength.
2. Using a non-linear finite element approach, a procedure for predicting the development and extent of structural cracks in masonry was also advanced. The validity of the procedure was verified by using it to simulate the pattern of failure of the walls surveyed in the UAE case studies and demonstrating the failure modes to be consistent with the site observations. This again gave confidence that the method could be applied to another site, as part of a structural design process.
3. With the discernibly accurate results obtained, the developed finite element solutions are shown to complement laboratory or field tests in assessing the settlement of collapsible soils under irrigation and the consequent effects on shallowly founded structures.

CHAPTER 8-RECOMMENDATIONS FOR FUTURE RESEARCH WORK

Notwithstanding the demonstrable extent to which the objectives of the present work have been addressed, in reality, the behavioural mechanisms of anisotropic collapsible soils under unsteady and differential seepage are very complex and many influencing factors are still not accounted for by existing methods. Therefore there is still a need for further refinements of current methods and to extend their capability including the ones proposed here. At present, it is still unclear as to the influence of certain factors such as scale and confinement effects in laboratory models, initial stress conditions and cyclic effects and non-homogeneity effects.

Geotechnical engineers have the challenge of having to assess a plethora of soil parameters some of which have special complexity due to dependence on stress state, pore pressure, cyclic response, hysteresis, temperature among others. All these uncertainties impact on the predictability of initiation of collapse as well as collapse rate of a given soil. A reliable assessment of the relationship between the intensity of landscape irrigation, water table level, thickness and location of collapsible strata can enable geotechnical engineers to develop guidance for property owners / members of the public to help them control rates of irrigation hence avoid extreme ground settlement that would cause structural distress of the kind reported in the case studies in this research work.

Regarding the laboratory test arrangement for simulation of collapsible soil settlement under irrigation, the following specific recommendations are suggested for improvement of results and for enhancement of the applicability of the predictive equations proposed:

1. Soil variability from one sampling depth (or site locality) to another, even within a small area investigated, could significantly have affected the results of the simulation hence the accuracy of the empirical equations formulated. Therefore any further research carried out should take into account the additional control factors listed below:
    a. Irrigation frequencies and rates as well as coverage area and any variations of these factors between one irrigated area and adjoining areas
    b. Thicknesses and depth locations of the collapsible layer tested
    c. Thicknesses and properties of other layers beneath and above a collapsible layer
    d. Initial ground water table position
    e. Level of the overburden stress acting on the collapsible soil stratum

f. Variations in ground elevation of irrigated areas overlying collapsible strata
g. Distribution of infrastructure loads on grade as well as dissipation of that load with depth
h. Variations in environmental conditions such as temperature, pressure, radiation and evaporation

2. Additional to measurements of surface settlements, which were successfully undertaken here, it would be useful to include more comprehensive instrumentation such as pore pressure sensors, displacement transducers (linear variable and boundary orthogonal transducers), stress sensors and temperature monitoring at several points on different cross-sections of the model soil profile tested in the laboratory under drip irrigation. The extra data would then be used to further corroborate the patterns reported and to aid in increasing the reliability of any derived correlations for predicting collapse rate and time of initiation.
3. Since in an actual irrigation, especially given the hot UAE climate, evaporation and radiation effects bring some uncertainty in the time and space dependent variations of discharge actually reaching the collapsible strata. Therefore improved laboratory simulation work should include some means of regulating the range of ambient conditions likely to affect results.

Turning to the finite element analysis procedure proposed, it is also recognised that results would depend on the ability to take into account a range of uncertainties, most of which are not just confined to collapsible soils but rather apply to other geo-materials in general. The main factors which similar research should concentrate on in future relate to non-linear and cyclic response under load, intrinsic rheological models for the soil and structures, transient water flow model parameters, temperature effects on viscosity and compressibility of water, soil-structure failure criteria used, anisotropic characteristics and intrinsic limitations in the built-in numerical approximations in the software used to implement the procedure. In dealing with some of the above drawbacks, the following improvements may be suggested:

1. Parameter values from laboratory tests should be subject to repeated verification with multiple specimens before being used in the finite element model. In addition, analysis for thermal effects to account for ambient temperatures (typically 45^0C during summer in UAE) and short-term fluctuations on seepage rates.
2. Dynamic effects such as traffic movement, machine vibrations, industrial/ construction activity, wind etc. should be included in the finite element model especially for superficial layers of collapsible layers undergoing sudden inter-particle bond breakage.

3. It is appreciated that the twin-villa complex model analysed in this research, was just part of a large housing development yet other pavements and boundary walls of nearby villas also experienced distress from settlement of the collapsible strata. Therefore, there is a potential advantage in extending the capability of the numerical analysis to cope with larger interacting zones where the foundation movement in one structure affects the next structure. Such an approach may yield more realistic results in cases where there is clear inter-dependence between soil-structure and structure-structure interaction at foundation level.

In summarising, it noted that although data for the work relates to the UAE region, the test methodologies and numerical analyses proposed may also be applied to collapsible soils from other regions, particularly semi-arid and arid climates. It is expected that similar outcomes will be obtained (such as time to exhibit collapse) provided that care is taken to ensure that the test and field conditions are as consistent as possible. Examples of the most important conditions to control are: proximity of water table, in-situ densities of soils, irrigation regime and overburden pressure (80 kN/m^2 used for lightweight structures for UAE cases). Inconsistencies in predictions from different sites would likely arise from the use of incorrect variables especially the most influential triggers for collapse mechanism. Therefore the importance of further research on collapsible soils from specific regions cannot be overstated.

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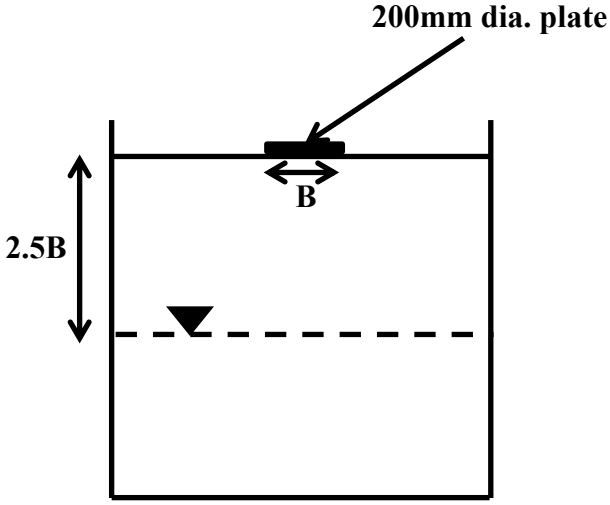
APPENDIX-A

LABORATOY TESTS - DATA SHEETS

LABORATOY TEST SET-UP - I

PLATE LOAD TESTS

A1. Plate load test with fully collapsible soil in the tank (water table at depth of 2.5B)

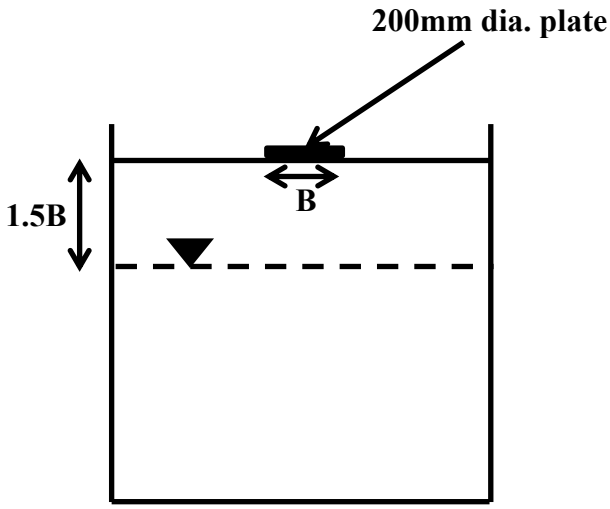


Scale: 1:20

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	80.00	2.83	water stopped
6.15	0.00	-	80.00	3.05	water started
12.31	0.00	-	80.00	4.57	water stopped
18.46	0.46	-	80.00	5.08	water started
24.62	0.58	-	80.00	5.49	water stopped
30.77	0.66	-	80.00	5.67	water started
36.92	0.76	-	80.00	6.01	water stopped
43.08	0.91	-	80.00	6.47	water started
49.23	1.24	-	80.00	7.87	water stopped
55.38	1.42	-	80.00	8.5	water started
61.54	1.72	-	80.00	9.42	water stopped
67.69	1.82	-	80.00	9.85	water started
73.85	1.94	-	80.00	15.77	water stopped
80.00	2.02	water started	End of the test		

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
10	0.00
15	0.00
25	0.46
35	0.58
45	0.66
55	0.76
65	0.91
75	1.24
85	1.42
95	1.72
100	1.82
120	1.94
150	2.02
180	2.83
210	3.05
240	4.57
270	5.08
300	5.49
330	5.67
360	6.01
390	6.47
420	7.87
450	8.5
480	9.42
510	9.85
540	15.77
End of the test	

A2. Plate load test with fully collapsible soil in the tank (water table at depth of 1.5B)

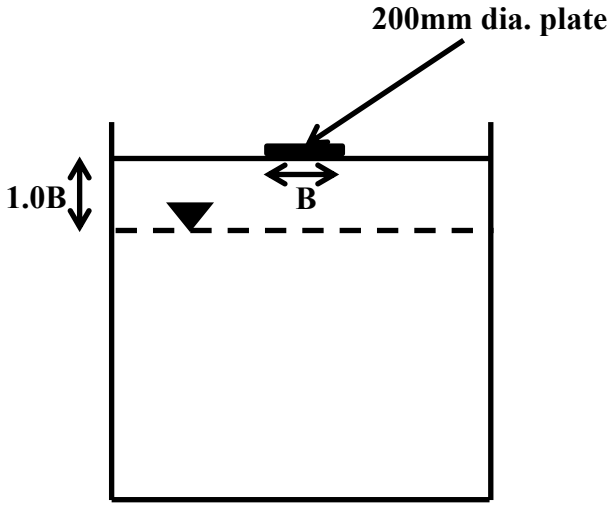


Scale: 1:20

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	80.00	4.81	water stopped
12.31	0.00	-	80.00	5.26	water started
18.46	0.00	-	80.00	5.81	water stopped
24.62	0.53	-	80.00	7.48	water started
30.77	0.65	-	80.00	7.81	water stopped
36.92	0.98	-	80.00	8.48	water started
43.08	1.40	-	80.00	8.80	water stopped
49.23	1.74	-	80.00	10.32	water started
55.38	2.39	-	80.00	10.74	water stopped
61.54	2.81	-	80.00	17.00	water started
67.69	3.31	-	End of the test		
73.85	3.91	-			
80.00	4.81	water started			

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
10	0.00
15	0.00
25	0.53
35	0.65
45	0.98
55	1.40
65	1.74
75	2.39
85	2.81
95	3.31
100	3.91
120	4.81
150	4.81
180	5.26
210	5.81
240	7.48
270	7.81
300	8.48
330	8.80
360	10.32
390	10.74
420	17.00
End of the test	

A3. Plate load test with fully collapsible soil in the tank (water table at depth of 1.0B)

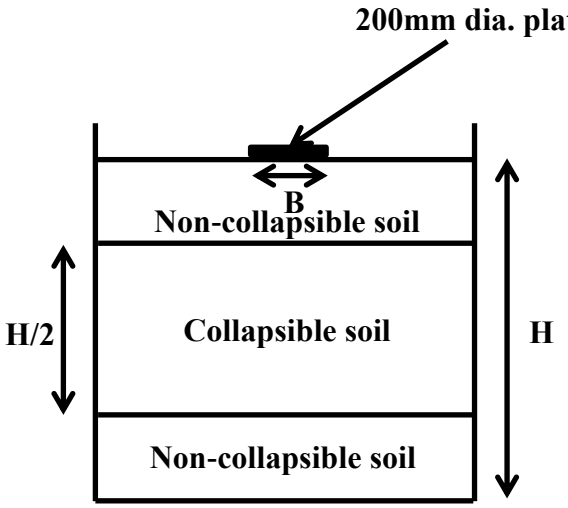


Scale: 1:20

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	73.85	3.38	-
12.31	0.00	-	80.00	3.86	water started
18.46	0.00	-	80.00	3.86	water stopped
24.62	0.47	-	80.00	3.92	water started
30.77	0.80	-	80.00	4.10	water stopped
36.92	1.15	-	80.00	4.22	water started
43.08	1.43	-	80.00	4.43	water stopped
49.23	1.70	-	80.00	5.52	water started
55.38	2.02	-	80.00	5.70	water stopped
61.54	2.53	-	80.00	11.52	water started
67.69	2.77	-	End of the test		

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
10	0.00
15	0.00
25	0.47
35	0.80
45	1.15
55	1.43
65	1.70
75	2.02
85	2.53
95	2.77
100	3.38
120	3.86
150	3.86
180	3.92
210	4.10
240	4.22
270	4.43
300	5.52
330	5.70
360	11.52
End of the test	

A4. Plate load test with collapsible soil at central half of the tank

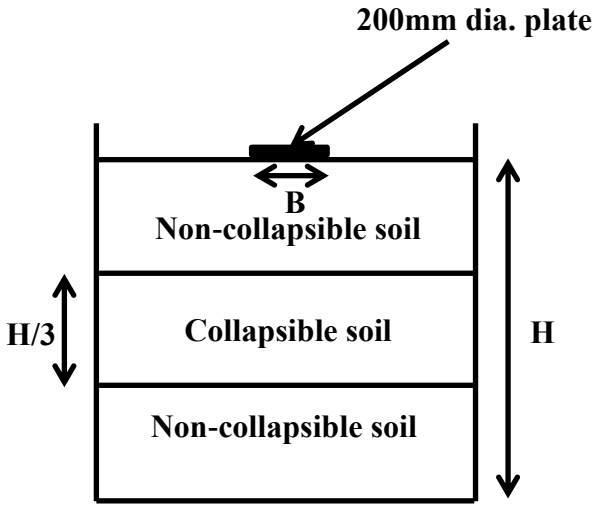


Scale:

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	68.57	0.22	-
5.71	0.00	-	74.29	0.27	-
11.43	0.01	-	80.00	0.37	water started
17.14	0.01	-	80.00	1.64	water stopped
22.86	0.01	-	80.00	2.24	water started
28.57	0.01	-	80.00	5.20	water stopped
34.29	0.01	-	80.00	5.55	water started
40.00	0.01	-	80.00	7.56	water stopped
45.71	0.10	-	80.00	9.90	water started
51.43	0.15	-	80.00	12.65	water stopped
57.14	0.18	-	End of the test		
62.86	0.22	-			

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
2	0.01
4	0.01
6	0.01
8	0.01
10	0.01
11	0.01
12	0.10
13	0.15
14	0.18
15	0.22
16	0.22
17	0.27
18	0.37
20	1.64
50	2.24
80	5.20
110	5.55
140	7.56
160	9.90
180	12.65
End of the test	

A5. Plate load test with collapsible soil at central one-third of the tank

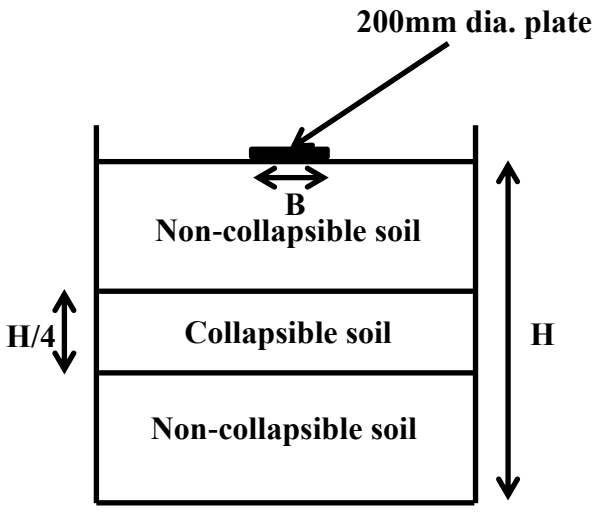


Scale:

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	68.57	0.08	-
11.43	0.00	-	74.29	0.08	-
17.14	0.00	-	80.00	0.16	water started
22.86	0.00	-	80.00	1.50	water stopped
28.57	0.00	-	80.00	3.08	water started
34.29	0.01	-	80.00	3.95	water stopped
40.00	0.02	-	80.00	5.00	water started
45.71	0.04	-	80.00	6.07	water stopped
51.43	0.04	-	80.00	6.76	water started
57.14	0.07	-	80.00	7.40	water stopped
62.86	0.08	-	End of the test		

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
2	0.00
6	0.00
10	0.00
12	0.00
16	0.01
19	0.02
22	0.04
26	0.04
30	0.07
34	0.08
38	0.08
42	0.08
46	0.16
50	1.50
80	3.08
110	3.95
140	5.00
170	6.07
185	6.76
200	7.40
End of the test	

A6. Plate load test with collapsible soil at central one-fourth of the tank

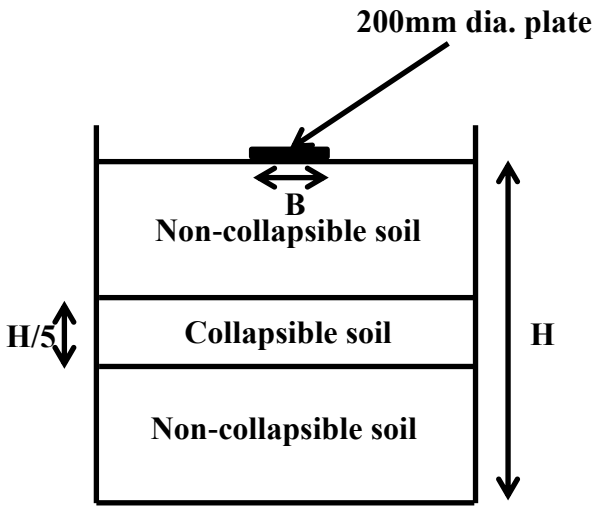


Scale:

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00	-	80.00	1.04	water stopped
11.43	0.00	-	80.00	1.42	water started
17.14	0.00	-	80.00	2.91	water stopped
22.86	0.00	-	80.00	3.38	water started
28.57	0.09	-	80.00	3.74	water stopped
34.29	0.11	-	80.00	4.09	water started
40.00	0.15	-	80.00	4.54	water stopped
45.71	0.17	-	80.00	5.00	water started
51.43	0.17	-	80.00	5.49	water stopped
57.14	0.24	-	80.00	6.14	water started
62.86	0.31	-	80.00	6.39	water stopped
68.57	0.34	-	80.00	6.99	water started
74.29	0.39	-	End of the test		
80.00	0.41	water started			

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
1	0.00
2	0.00
3	0.00
4	0.09
5	0.11
6	0.15
7	0.17
8	0.17
9	0.24
10	0.31
11	0.34
12	0.39
14	0.41
15	1.04
45	1.42
75	2.91
90	3.38
105	3.74
120	4.09
135	4.54
150	5.00
165	5.49
180	6.14
195	6.39
210	6.99
End of the test	

A7. Plate load test with collapsible soil at central one-fifth of the tank



Scale:

Pressure-Settlement Data					
Applied pressure (kN/m^2)	**Settlement (mm)**	**Remarks**	**Applied pressure (kN/m^2)**	**Settlement (mm)**	**Remarks**
0.00	0.00		80.00	0.13	water stopped
5.71	0.00	-	80.00	0.38	water started
11.43	0.00	-	80.00	0.52	water stopped
17.14	0.00	-	80.00	0.52	water started
22.86	0.00	-	80.00	0.72	water stopped
28.57	0.00	-	80.00	0.87	water started
34.29	0.00	-	80.00	1.01	water stopped
40.00	0.00	-	80.00	1.63	water started
45.71	0.00	-	80.00	2.45	water stopped
51.43	0.00	-	80.00	3.13	water started
57.14	0.00	-	80.00	3.35	water stopped
62.86	0.00	-	80.00	3.63	water started
68.57	0.00	-	80.00	4.13	water stopped
74.28	0.00	-	80.00	4.33	water started
80.00	0.00	water started	End of the test		

Time-Settlement Data	
Time (minutes)	**Settlement (mm)**
0	0.00
2	0.00
3	0.00
4	0.00
6	0.00
8	0.00
9	0.00
11	0.00
12	0.00
14	0.00
16	0.00
18	0.00
20	0.00
22	0.00
24	0.13
54	0.38
84	0.52
94	0.52
104	0.72
114	0.87
124	1.01
134	1.63
164	2.45
194	3.13
224	3.35
234	3.63
244	4.13
254	4.33
End of the test	

A8. Compaction test results on collapsible soil

Specific gravity of soil = 2.68

Unit weigh t of water = 9.81 kN/m^3

Trial No.	Weight of the mould with wet soil (g)	Weight of empty mould (g)	Weight of wet soil (g)	Bulk Density (g/cc)	Moisture content (%)	Dry Density (g/cc)
1	6274	4182	2092	2.09	14.00	1.835
2	6299	4182	2117	2.12	15.00	1.841
3	6332	4182	2150	2.15	17.50	1.830
4	6287	4182	2105	2.11	20.00	1.754

Calculations for plotting air-void lines on compaction curve

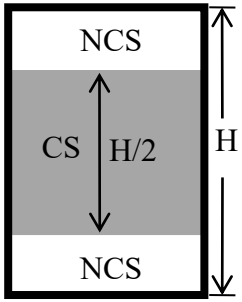
Moisture content (%)	Dry Density (g/cc)	Dry density (g/cc) at 0% air voids (100% saturated)	Dry density (g/cc) at 2.5% air voids (97.5% saturated)	Dry density (g/cc) at 5% air voids (95% saturated)	Dry density (g/cc) at 7.5% air voids (92.5% saturated)	Dry density (g/cc) at 10% air voids (90% saturated)	Dry density (g/cc) at 12.5% air voids (87.5% saturated)
14.00	1.835	1.949	1.935	1.921	1.907	1.891	1.876
15.00	1.841	1.912	1.898	1.883	1.868	1.853	1.836
17.50	1.830	1.824	1.810	1.794	1.778	1.762	1.745
20.00	1.754	1.745	1.729	1.713	1.697	1.680	1.662

LABORATOY TEST SET-UP - II

COLLAPSE TESTS IN METAL MOULDS

A9. Collapse test results in metal moulds for soil combination-1 (SC-1)

Details of soil combination-1



Where,
H – Height of the CBR mould (180mm)
NCS – Non-collapsible soil
CS – Collapsible soil

Case-1: Depth of water table is at H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)									Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation									
			1	2	3	4	5	6	7			
17.5	9.29	3.75	3.14	2.93	2.92	2.91	2.83	2.81	2.82	5.54	0.93	6.47
18.0	7.19	3.39	2.88	2.77	2.74	2.74	2.7	2.69	2.68	3.80	0.71	4.51
18.5	11.41	9.39	9.08	9.03	9.00	8.99	8.94	8.95	8.94	2.02	0.45	2.47
									Average (mm)	3.79	0.70	4.48

Case-2: Depth of water table is at H/2 from the base of the soil column

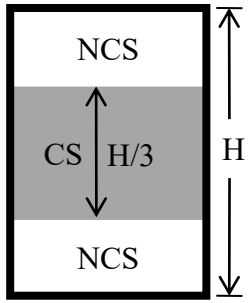
Density of soil (kN/m^3)	Dial gauge readings (mm)								Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation								
			1	2	3	4	5	6			
17.5	9.76	3.15	3.02	2.83	2.6	2.48	2.43	2.42	6.61	0.73	7.34
18.0	9.21	4.09	4.02	3.86	3.74	3.63	3.58	3.58	5.12	0.51	5.63
18.5	10.83	7.44	7.36	7.21	7.17	7.13	7.10	7.09	3.39	0.35	3.74
Average (mm)									5.04	0.53	5.57

Case-3: Depth of water table is at 2H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
17.5	8.46	0.74	0.77	0.32	0.21	0.20	7.72	0.54	8.26
18.0	11.47	6.21	6.26	6.08	5.89	5.88	5.26	0.33	5.59
18.5	9.51	5.22	5.17	5.07	5.01	5.00	4.29	0.22	4.51
Average (mm)							5.76	0.36	6.12

A10. Collapse test results in metal moulds for soil combination-2 (SC-2)

Details of soil combination-2



Where,
H – Height of the CBR mould (180mm)
NCS – Non-collapsible soil
CS – Collapsible soil

Case-1: Depth of water table is at H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)							Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation							
			1	2	3	4	5			
1.75	8.92	5.27	5.12	4.86	4.84	4.83	4.84	3.65	0.43	4.08
1.80	10.58	7.85	7.72	7.63	7.59	7.60	7.60	2.73	0.25	2.98
1.85	10.69	9.17	9.11	9.01	8.97	8.90	8.89	1.52	0.28	1.80
Average (mm)								2.63	0.32	2.95

Case-2: Depth of water table is at H/2 from the base of the soil column

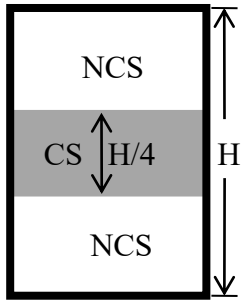
Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
1.75	9.38	3.46	3.35	3.30	3.19	3.18	5.92	0.28	6.20
1.80	10.16	5.41	5.31	5.28	5.17	5.17	4.75	0.24	4.99
1.85	10.88	7.86	7.82	7.81	7.69	7.69	3.02	0.17	3.19
Average (mm)							4.56	0.23	4.79

Case-3: Depth of water table is at 2H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)					Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation					
			1	2	3			
1.75	8.55	1.35	1.27	1.19	1.18	7.20	0.17	7.37
1.80	9.92	4.71	4.68	4.61	4.61	5.21	0.10	5.31
1.85	10.32	6.8	6.79	6.74	6.73	3.52	0.07	3.59
Average (mm)						5.31	0.11	5.42

A11. Collapse test results in metal moulds for soil combination-3 (SC-3)

Details of soil combination-3



Where,
H – Height of the CBR mould (180mm)
NCS – Non-collapsible soil
CS – Collapsible soil

Case-1: Depth of water table is at H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)							Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation							
			1	2	3	4	5			
1.75	10.27	7.70	7.61	7.51	7.51	7.52	7.51	2.57	0.19	2.76
1.80	8.79	7.87	7.82	7.74	7.72	7.73	7.74	0.92	0.13	1.05
1.85	12.00	11.26	11.08	10.89	10.92	10.90	10.91	0.74	0.35	1.09
Average (mm)								1.41	0.22	1.63

Case-2: Depth of water table is at H/2 from the base of the soil column

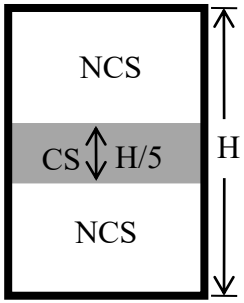
Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
1.75	9.58	5.47	5.31	5.20	5.17	5.18	4.11	0.29	4.40
1.80	10.16	7.83	7.71	7.67	7.67	7.66	2.33	0.17	2.50
1.85	9.71	8.22	8.15	8.10	8.07	8.07	1.49	0.15	1.64
Average (mm)							2.64	0.20	2.85

Case-3: Depth of water table is at 2H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)					Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation					
			1	2	3			
1.75	8.99	3.78	3.71	3.65	3.66	5.21	0.12	5.33
1.80	9.81	6.62	6.56	6.52	6.51	3.19	0.11	3.30
1.85	9.46	7.65	7.63	7.61	7.60	1.81	0.05	1.86
Average (mm)						3.40	0.09	3.50

A12. Collapse test results in metal moulds for soil combination-4 (SC-4)

Details of soil combination-4



Where,
H – Height of the CBR mould (180mm)
NCS – Non-collapsible soil
CS – Collapsible soil

Case-1: Depth of water table is at H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
1.75	11.31	8.75	8.68	8.62	8.51	8.51	2.56	0.24	2.80
1.80	12.50	11.69	11.61	11.57	11.55	11.54	0.81	0.15	0.96
1.85	11.77	11.09	11.02	10.99	10.96	10.96	0.68	0.13	0.81
						Average (mm)	1.35	0.17	1.52

Case-2: Depth of water table is at H/2 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
1.75	10.51	6.69	6.56	6.52	6.52	6.53	3.82	0.16	3.98
1.80	10.93	8.78	8.69	8.66	8.65	8.65	2.15	0.13	2.28
1.85	11.45	10.16	10.11	10.09	10.08	10.08	1.29	0.08	1.37
						Average (mm)	2.42	0.12	2.54

Case-3: Depth of water table is at 2H/3 from the base of the soil column

Density of soil (kN/m^3)	Dial gauge readings (mm)						Settlement due to rise in water table alone (mm)	Settlement due to drip irrigation alone (mm)	Total settlement due rise in water table and drip irrigation (mm)
	When the soil is fully dry	Stabilized after simulating the water table	At the end of each cycle of drip irrigation						
			1	2	3	4			
1.75	10.91	6.02	5.95	5.92	5.93	5.92	4.89	0.10	4.99
1.80	11.66	8.81	8.78	8.75	8.75	8.74	2.85	0.07	2.92
1.85	10.98	9.43	9.41	9.40	9.40	9.40	1.55	0.03	1.58
						Average (mm)	3.10	0.07	3.16

APPENDIX-B

FINITE ELEMENT MODELLING RESULTS

- Geotechnical modelling of twin-villa complex
- Finite Element simulation of collapse tests in metal moulds
- Structural modelling of boundary walls

GEOTECHNICAL MODELLING OF TWIN-VILLA COMPLEX

Geotechnical modelling of twin-villa complex

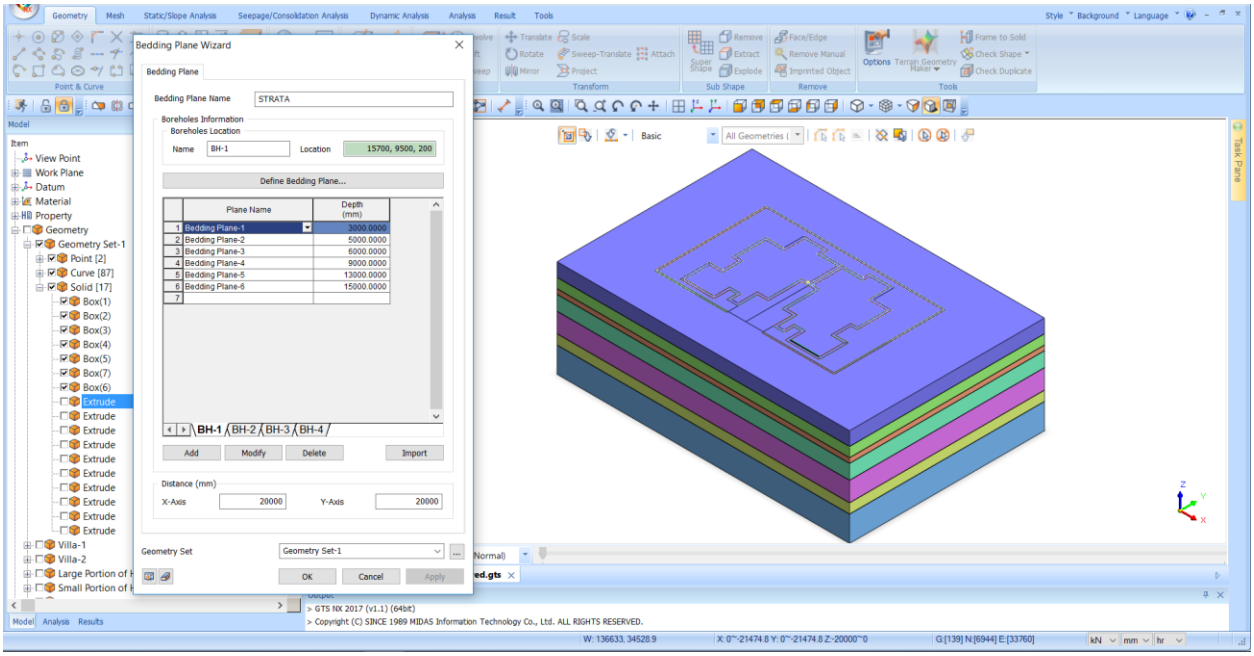
1) Representative soil layers on site

   a. Total depth of ground strata used in the model = 20m

   b. Data related to field borehole data obtained from geotechnical companies = 15m

   c. Additional bottom most layer of 5m thick was modelled to represent the continuity of ground below known data of 15m deep.

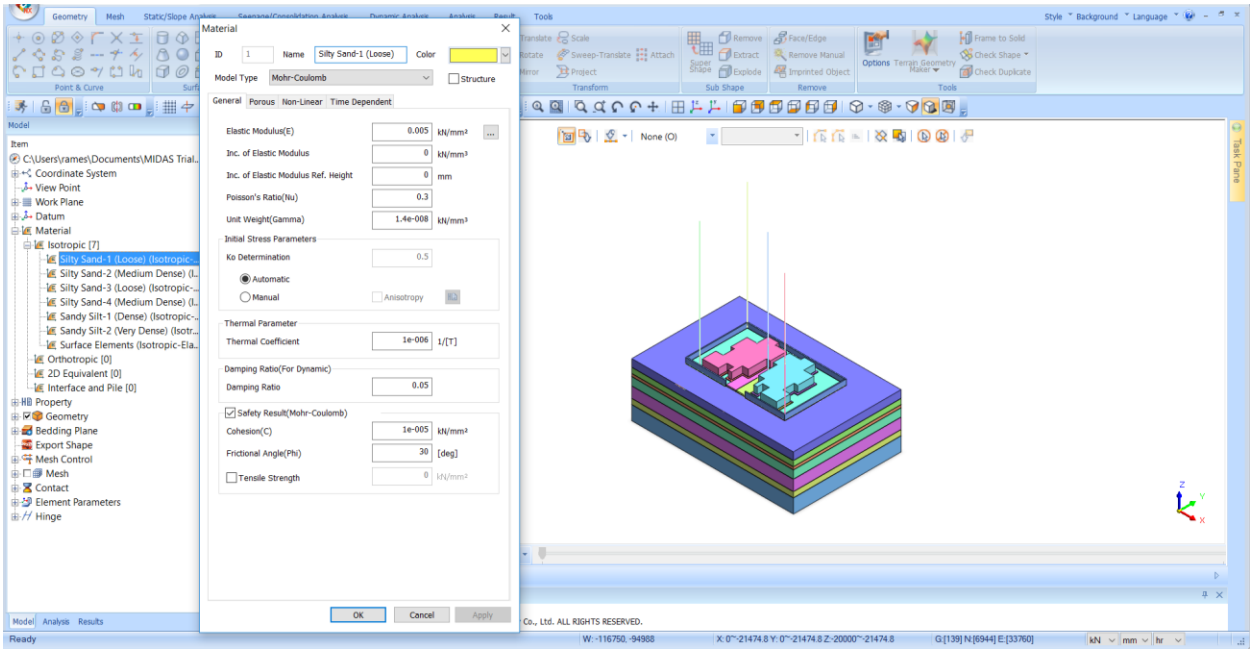
Layer	Depth of layer (m)	Layer thickness (m)	Soil type	Representative SPT
1	0-3	3	Silty SAND	4
2	3-5	2	Silty SAND	20
3	5-6	1	Silty SAND	6
4	6-9	3	Silty SAND	15
5	9-13	4	Sandy SILT	26
6	13-15	2	Sandy SILT	50
7	15-20	5	Sandy SILT	50



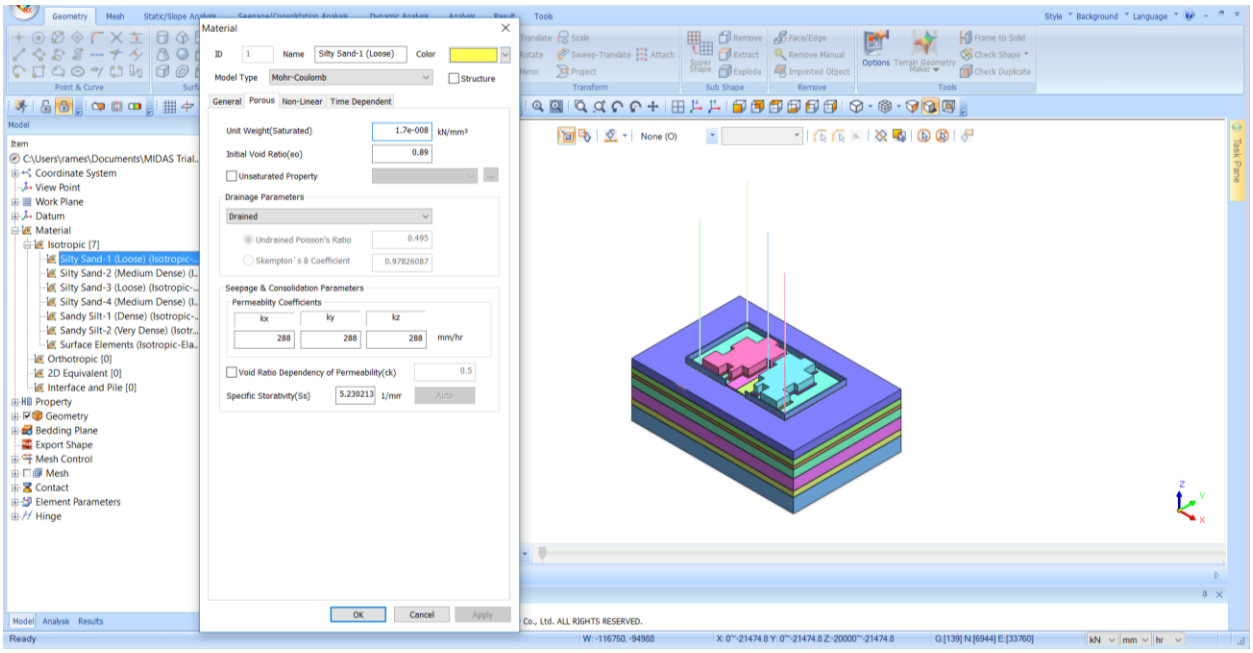
Geometric model of soil layers

2) Properties of different soil layers (sample is shown for layer-1)

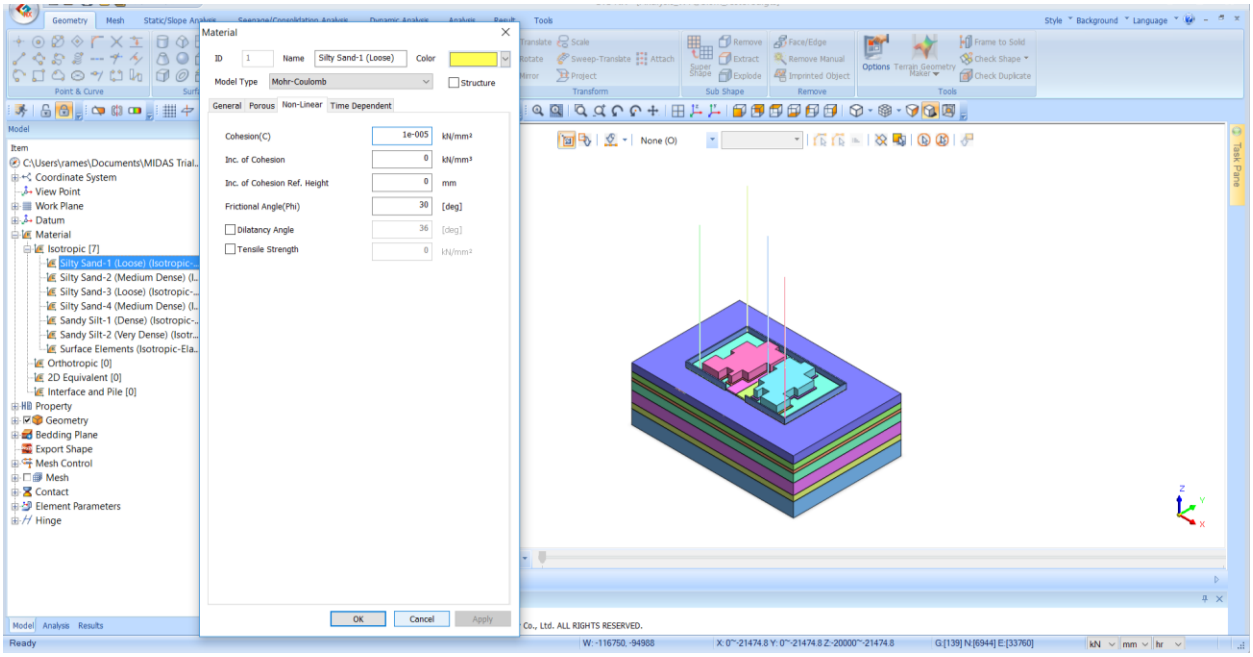
i. General properties



ii. Porous properties



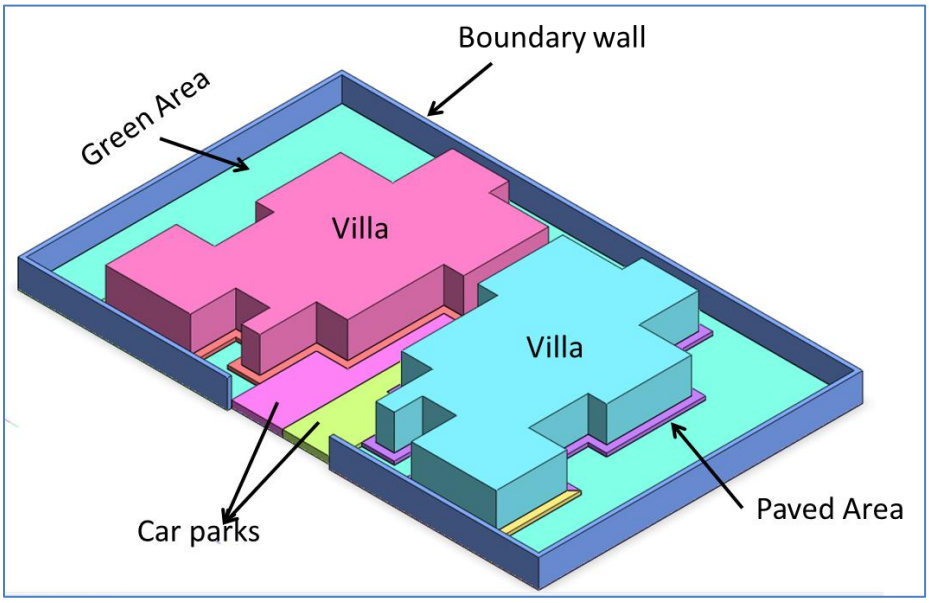
iii. Non-linear properties



Properties of all soil layers in the model

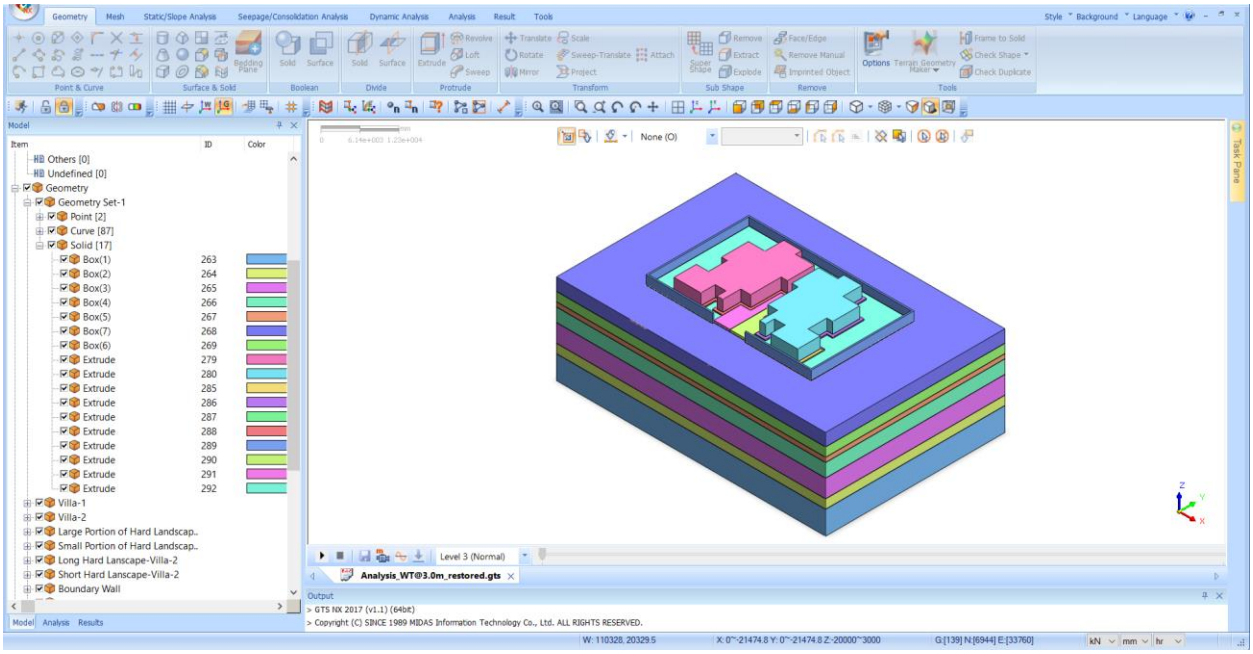
Layer	Relative density	Dry Density (kN/m^3)	Friction Angle (Degrees)	Void Ratio(e)	Elastic Modulus (kN/m^2)	Permeability (m/sec)
1	Low	14.00	30	0.89	5000	8.00E-05
2	Medium dense	17.00	34	0.56	16000	3.00E-05
3	Low	14.67	31	0.81	8000	6.00E-05
4	Medium dense	16.50	33	0.61	15000	5.00E-05
5	Dense	17.60	35	0.51	18000	8.00E-06
6	Very dense	20.00	38	0.33	20000	4.00E-06
7	Very dense	20.00	38	0.33	20000	4.00E-06

3) Modelling of twin villa complex

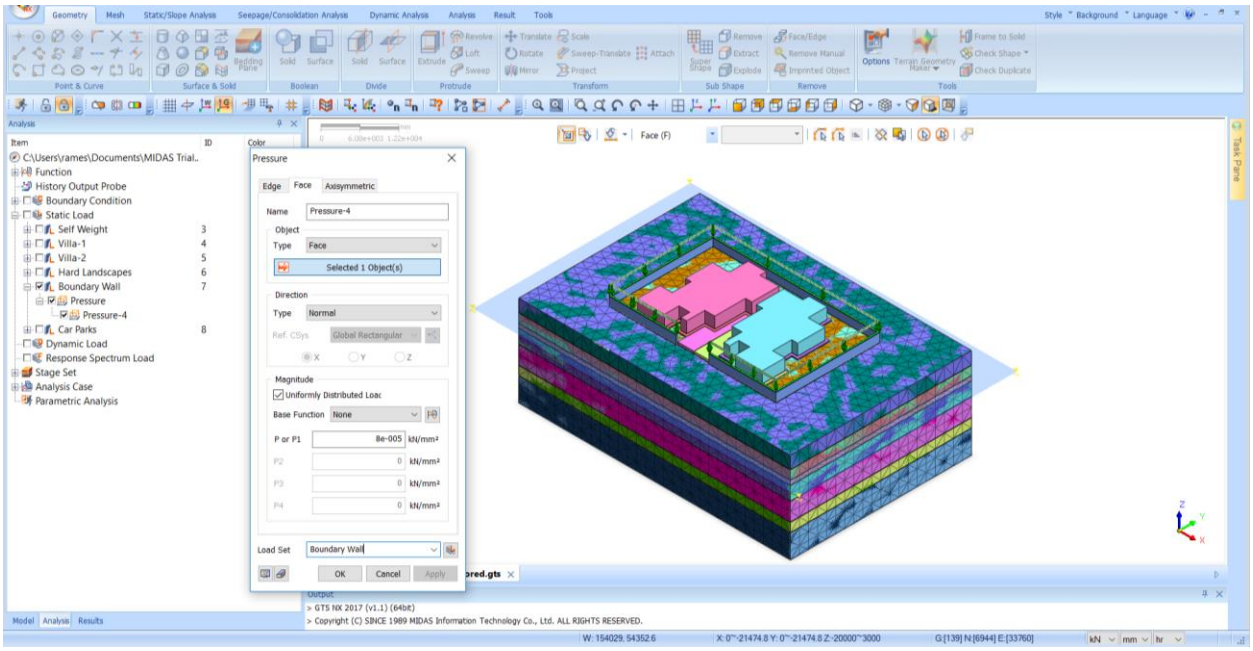


Note-1: Green areas are where drip irrigation was carried out that caused surround shallow founded structures to experience distresses.
Note-2: Complete modelling was carried out using on-site dimensions.

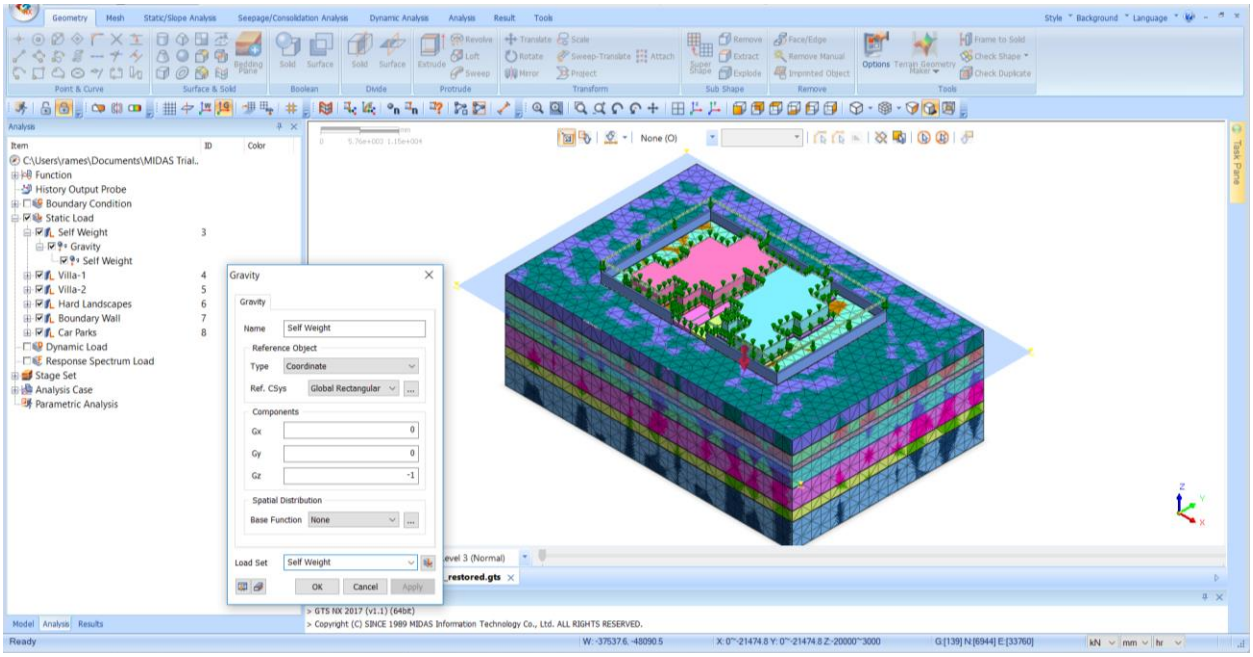
4) Complete model (All infrastructures along with soil layers underneath)



5) Pressure applied on ground by various infrastructures (villas, boundary walls, hard landscapes, car parks etc.

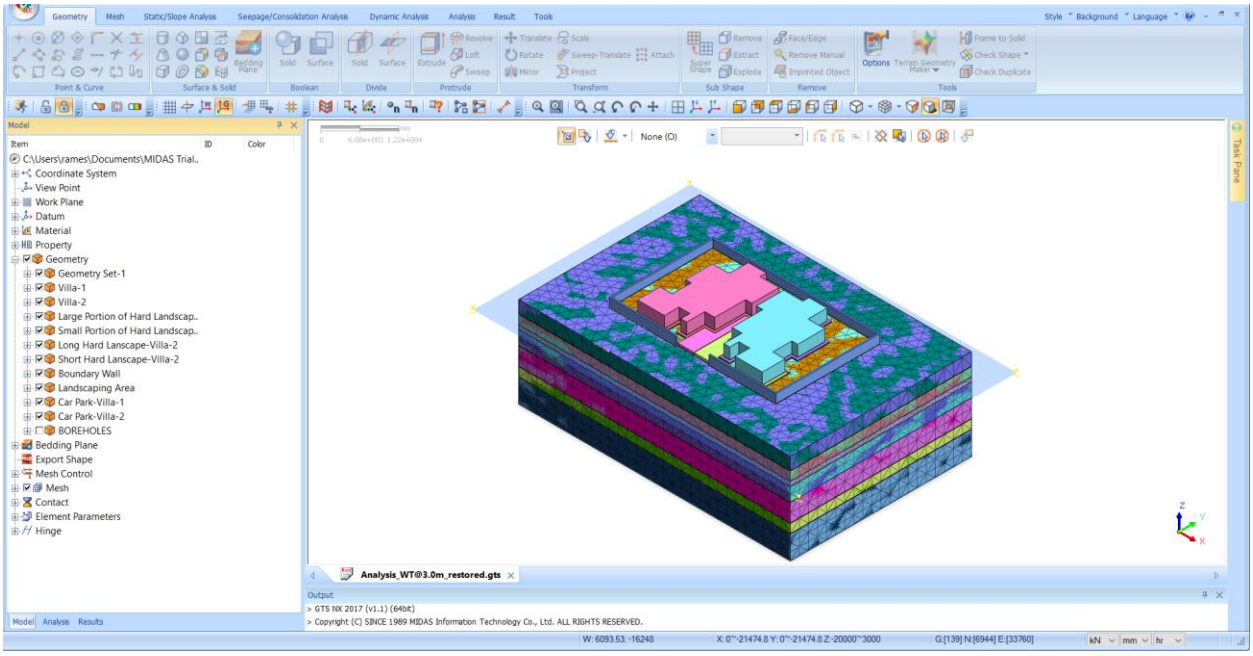


Pressure exerted by boundary walls on the ground
(80 kN/m^2 calculated in accordance with the onsite dimensions)

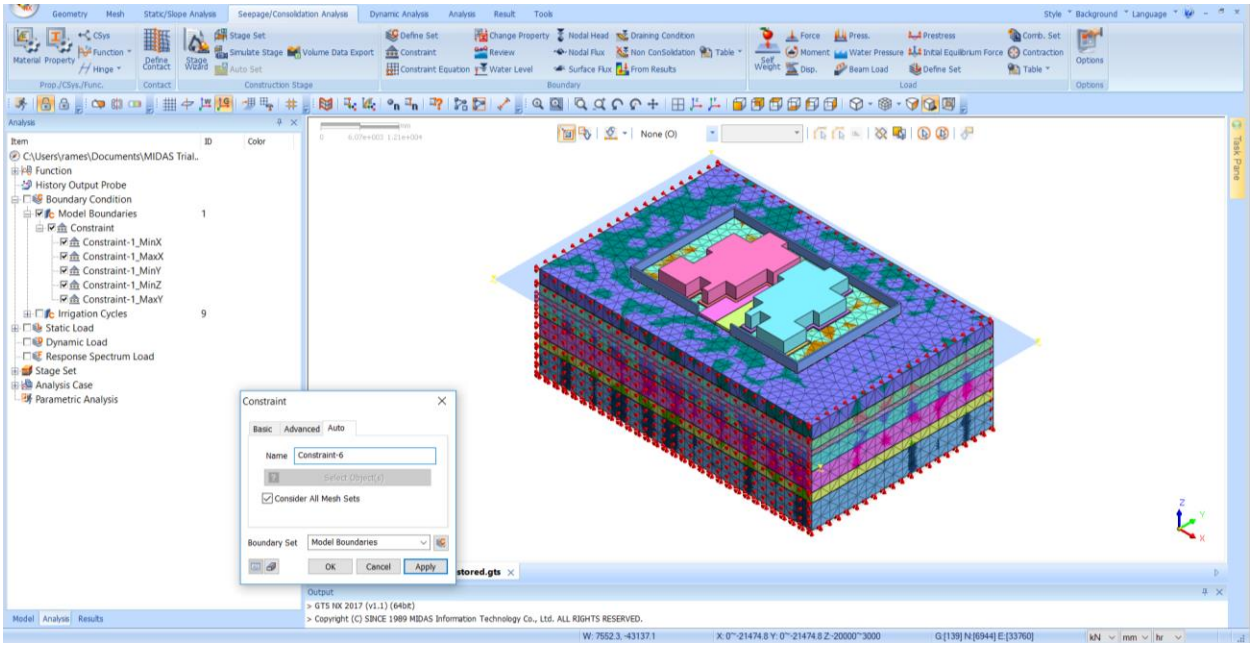


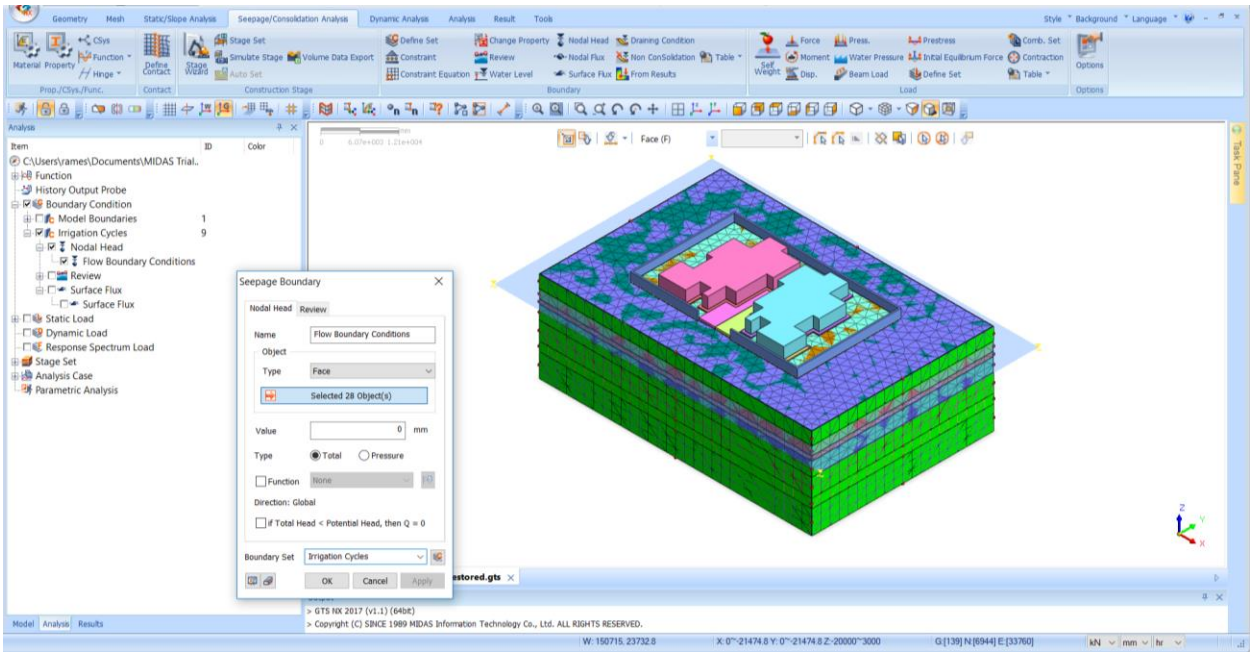
Pressure exerted by various infrastructures including self-weight of the ground

6) Meshed model

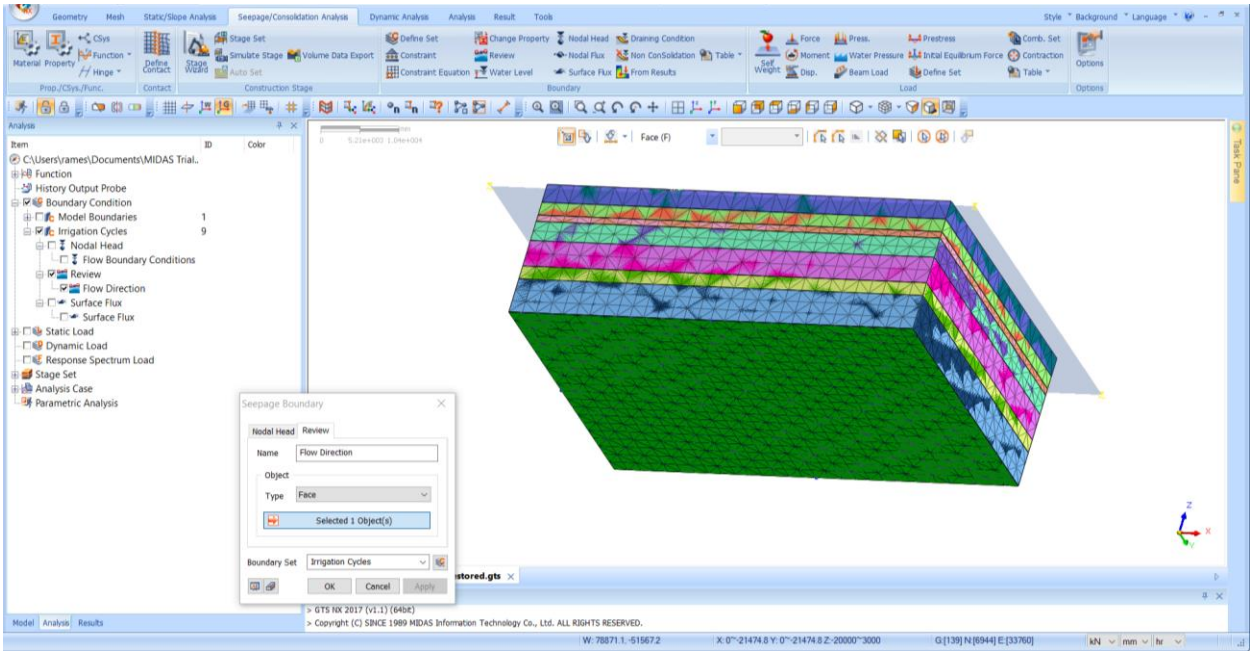


7) Boundary conditions of the model simulating the natural ground situation



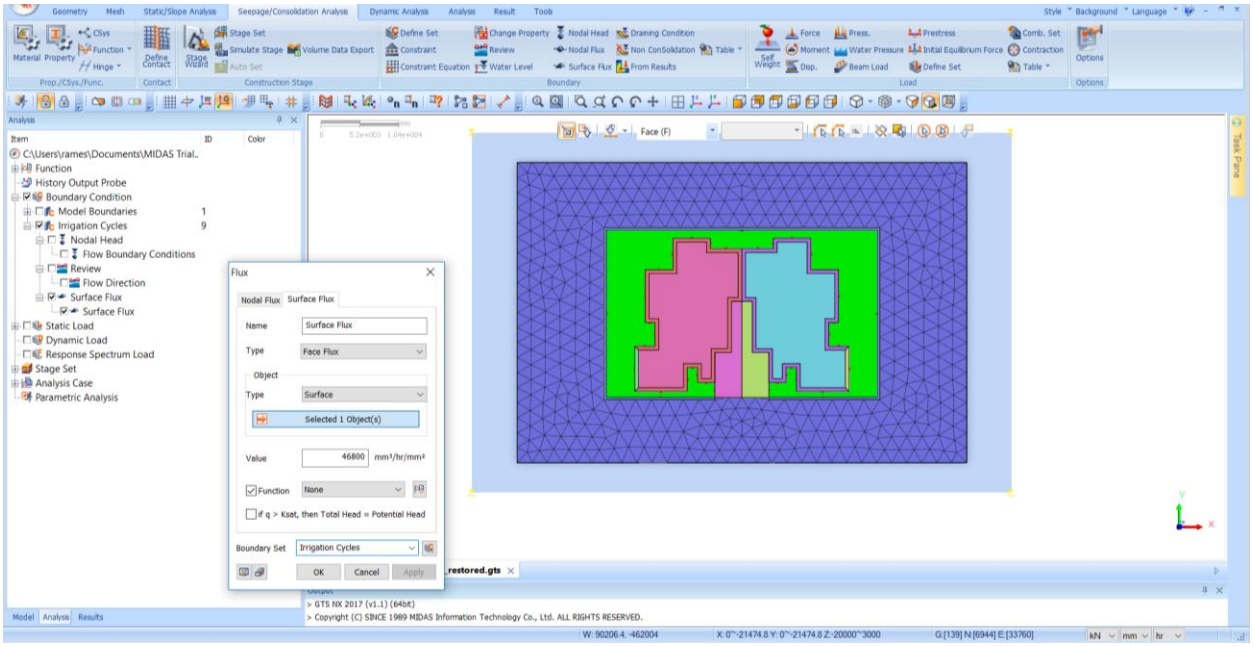


Pore pressure fully dissipating condition

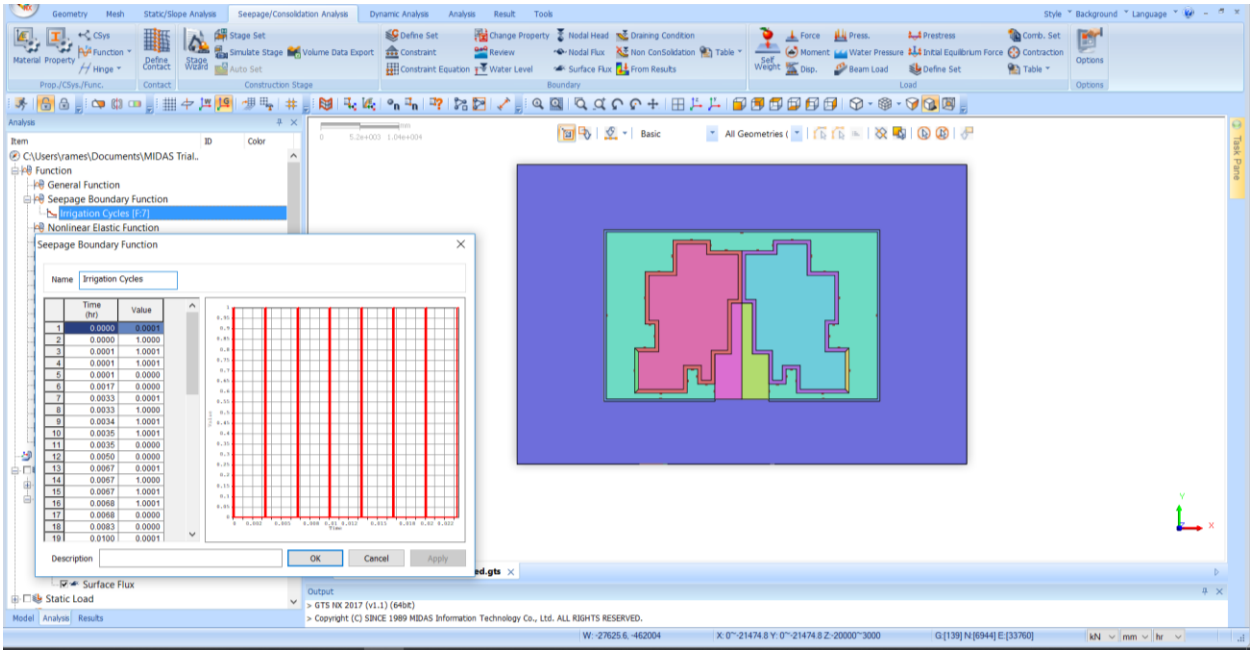


Assigning the flow direction (review boundary)

8) Simulation of on-site drip irrigation cycles in the model

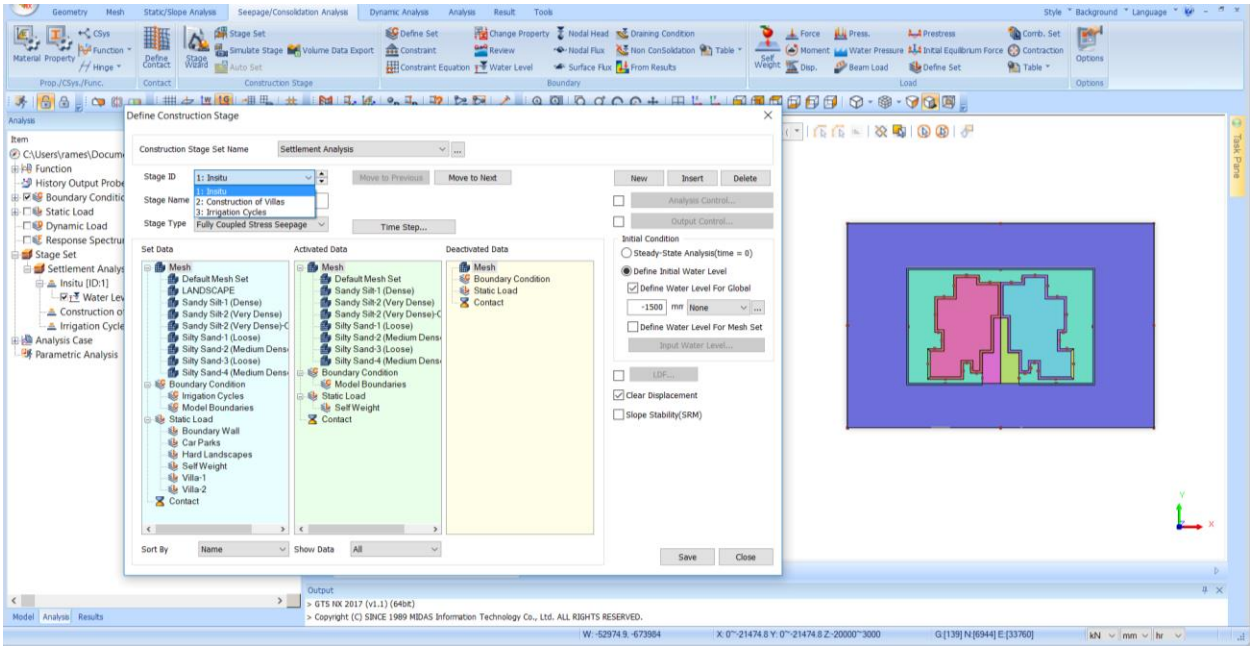


Drip irrigation quantity simulation

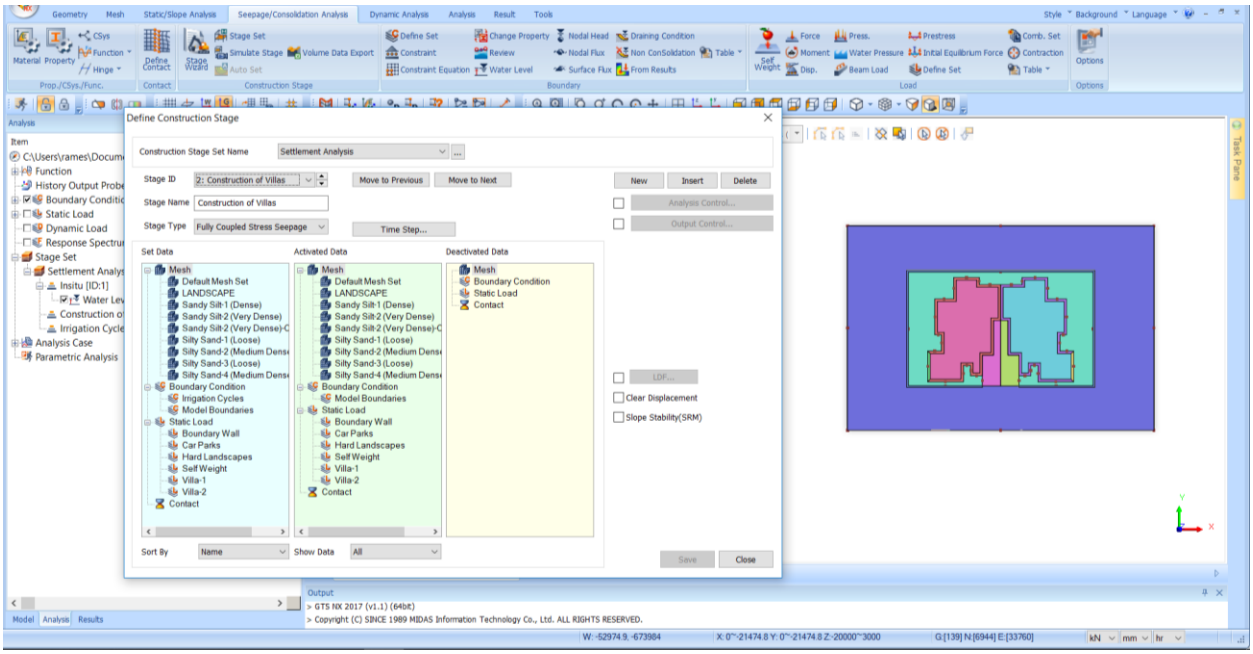


Simulation of drip irrigation cycles

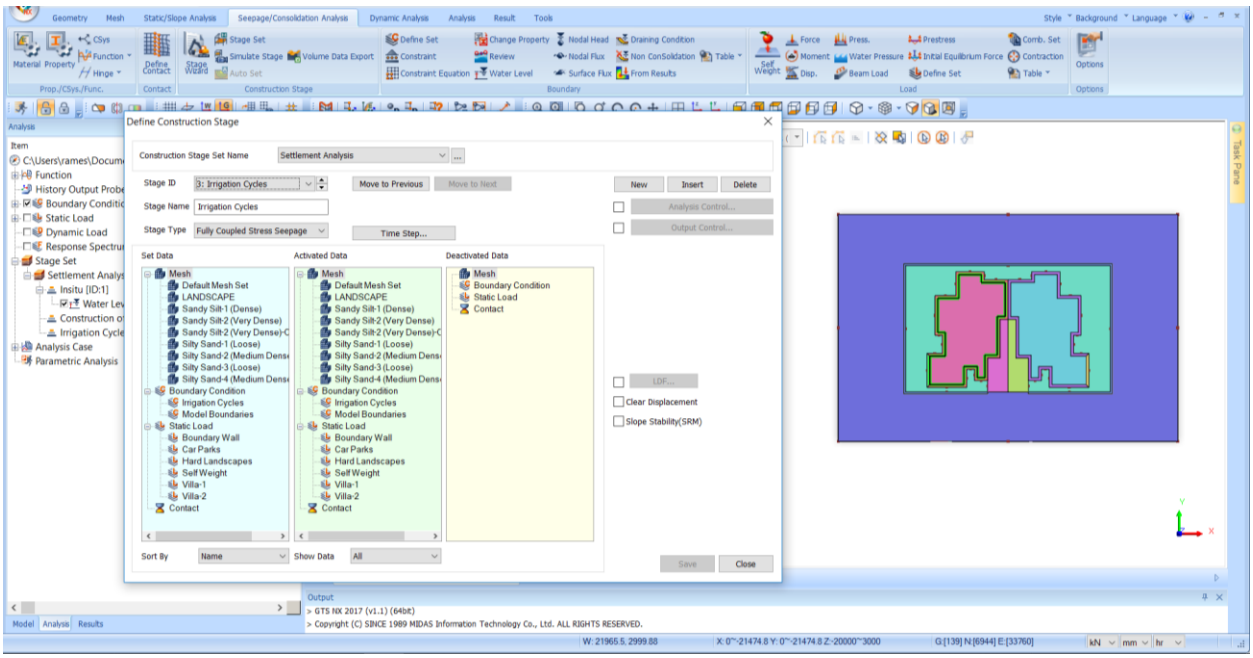
9) Stage construction simulation as on site for fully coupled stress-seepage analysis



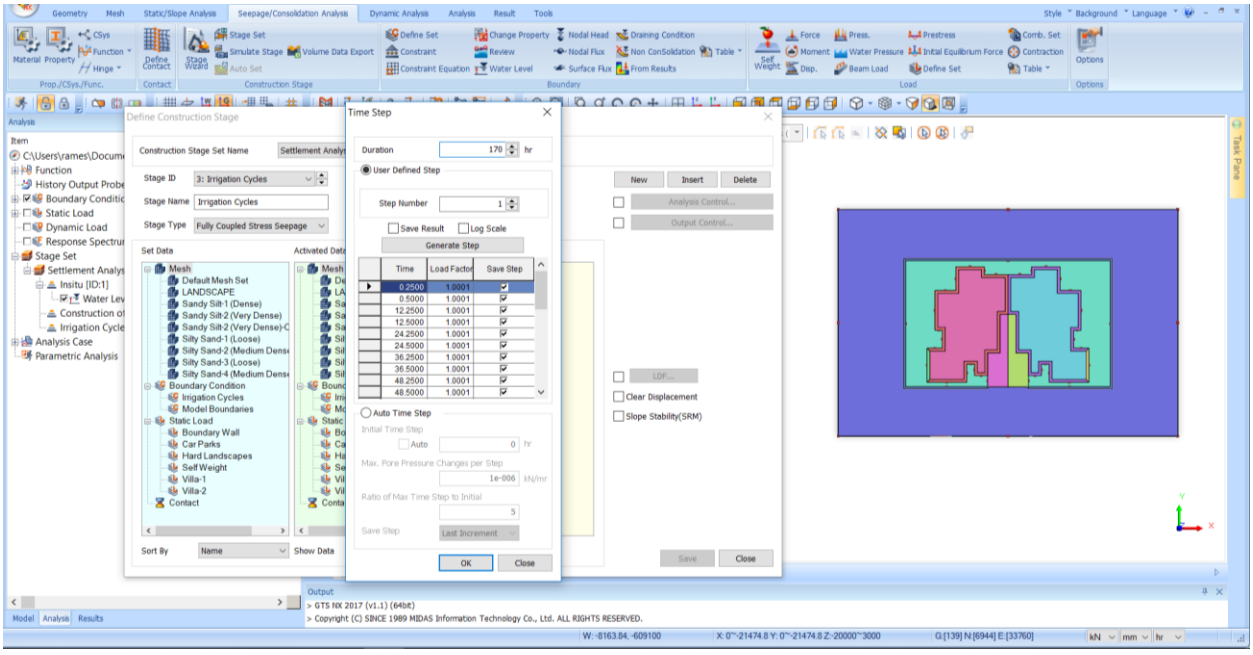
In-situ condition with GWT@1.5m depth and nullifying the settlement due to self-weight



Transferring all loads to the ground

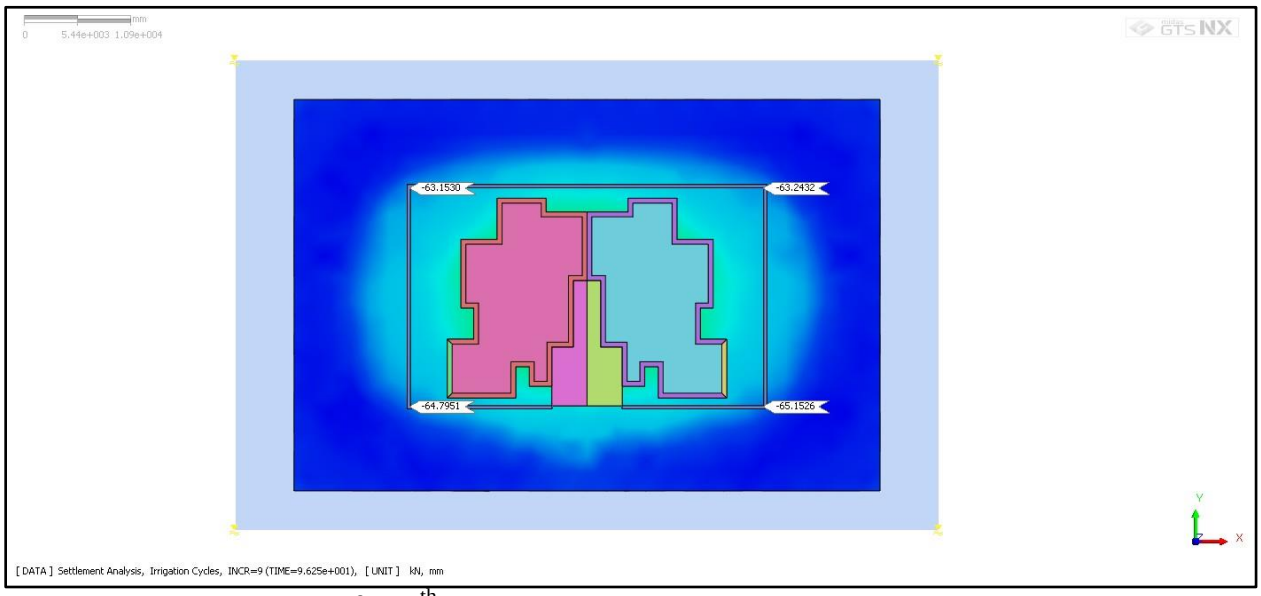


Activating irrigation cycles in the model

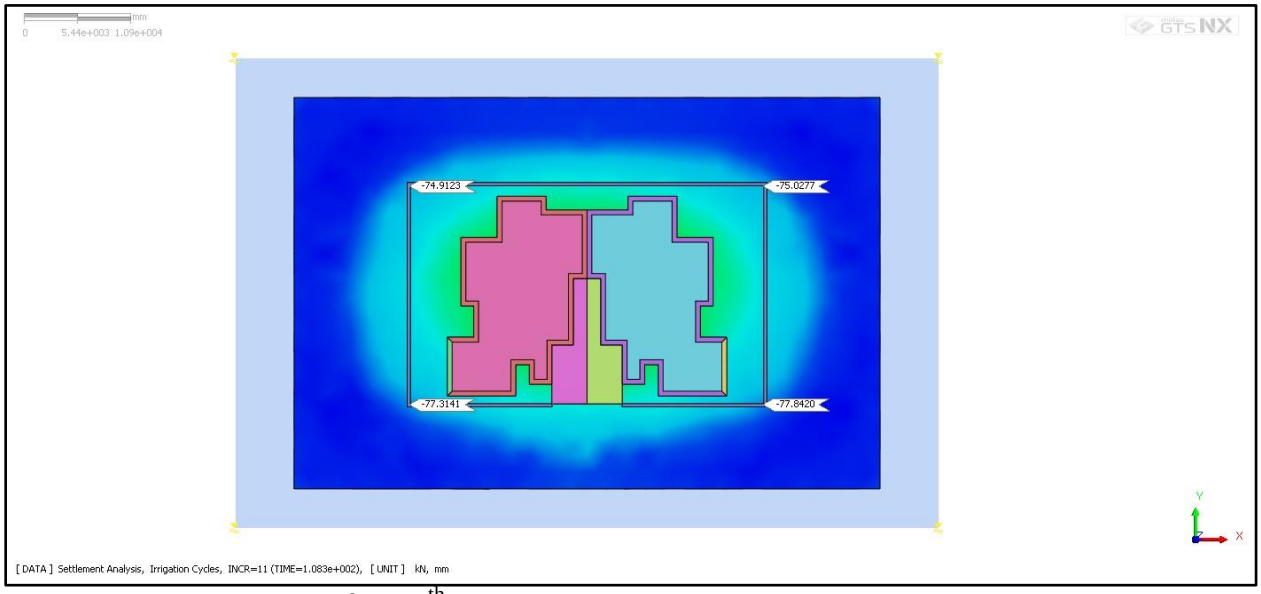


Time steps of irrigation cycles

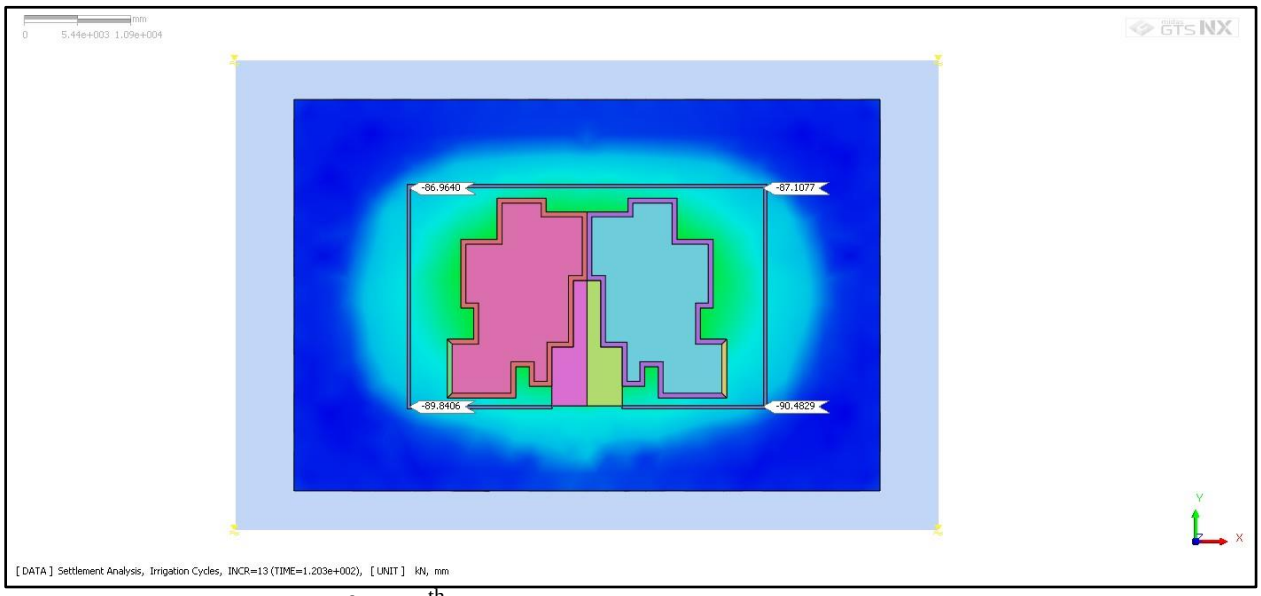
10) Settlement results of boundary walls – samples at various irrigation cycles



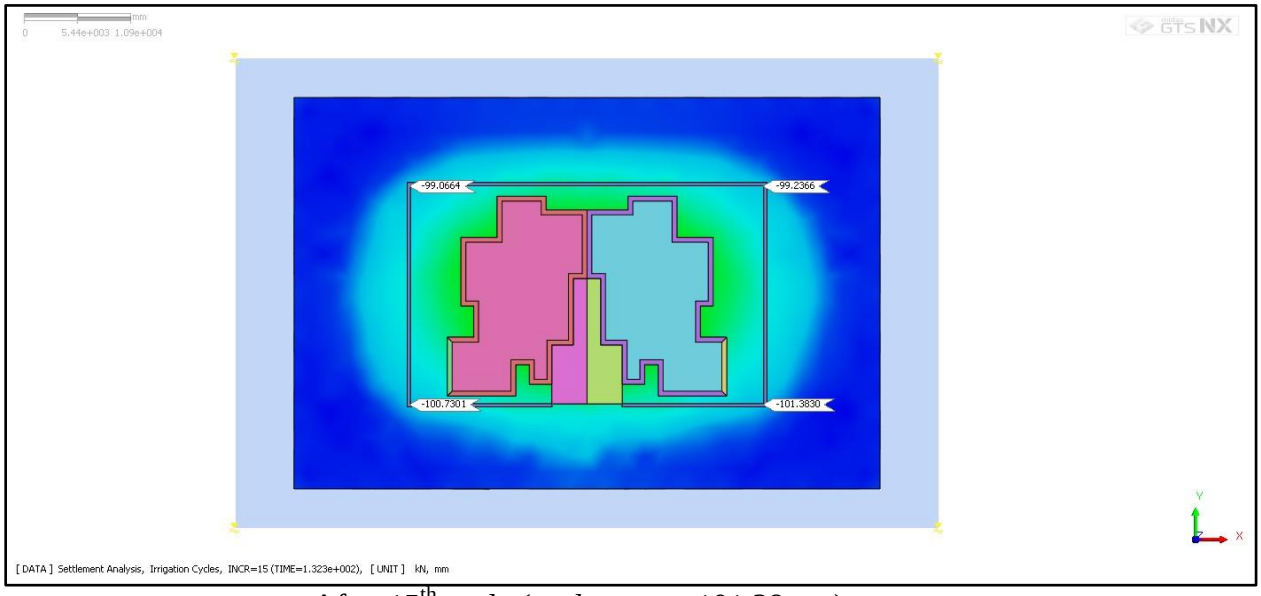
After 9th cycle (settlement = 65.15mm)



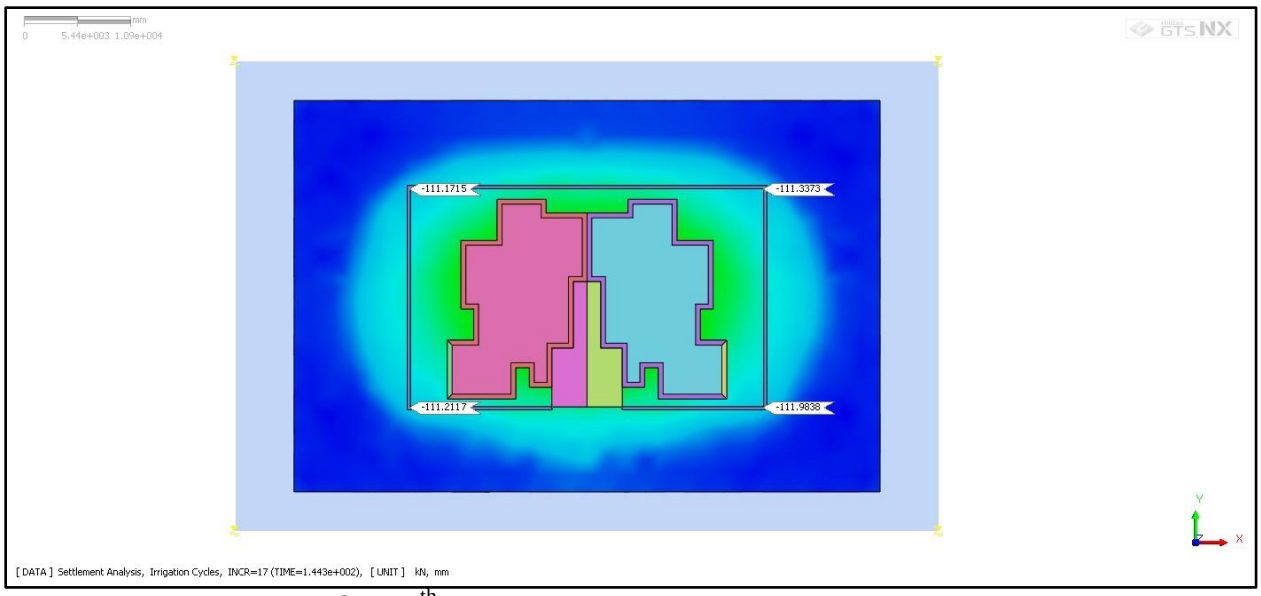
After 11th cycle (settlement = 77.84mm)



After 13th cycle (settlement = 90.48mm)



After 15th cycle (settlement = 101.38mm)



After 17th cycle (settlement = 111.98mm)

FINITE ELEMENT SIMULATION OF COLLAPSE TESTS IN METAL MOULDS

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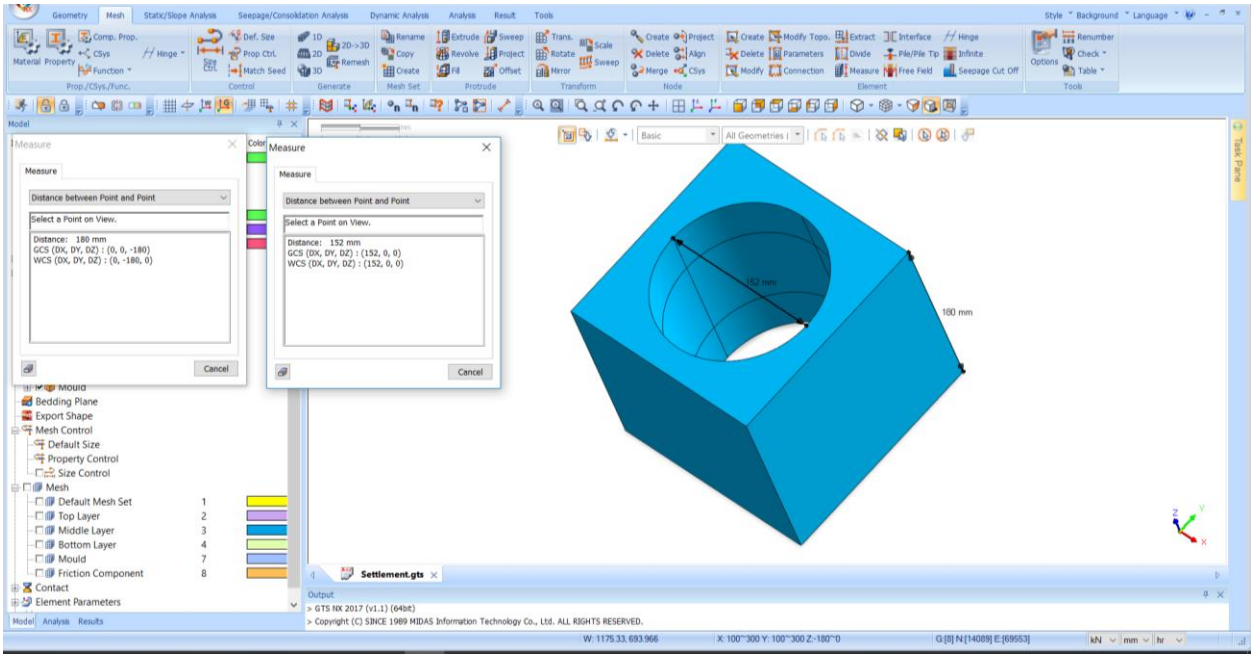
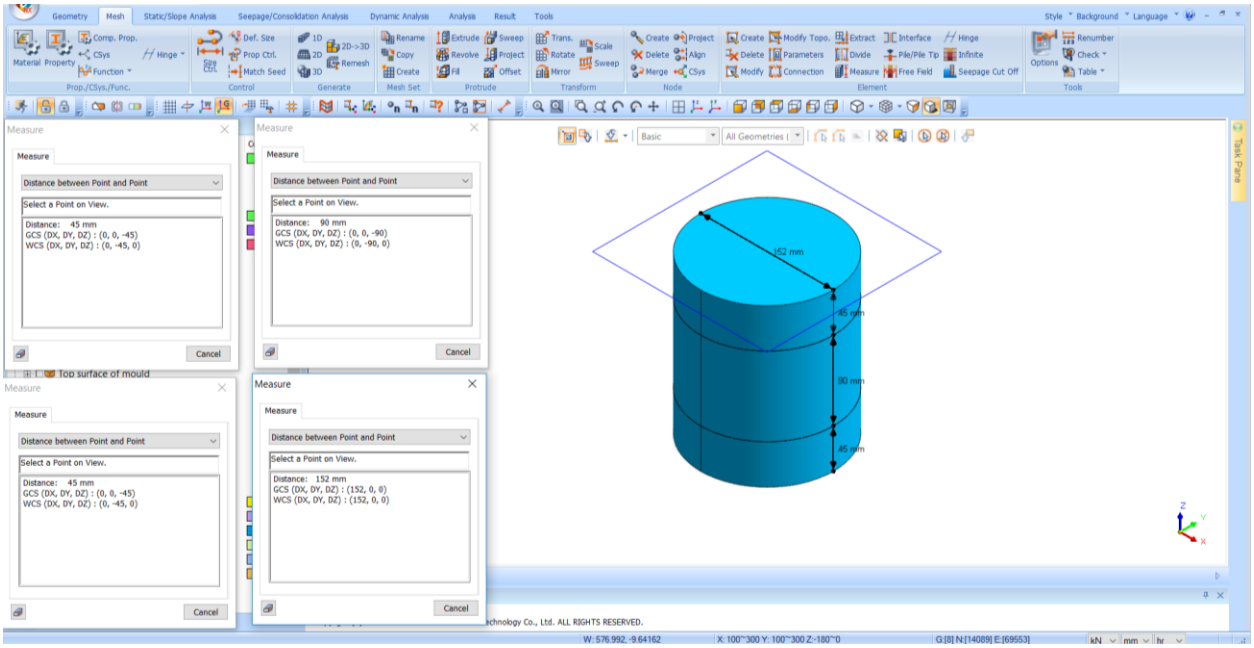
Finite element analysis of SC-1 with density of soil as 17.5 kN/m^3 and water table factor of 2/3

1) Dimensions of the steel mould
   i. Internal diameter = 152m
   ii. Height = 180mm

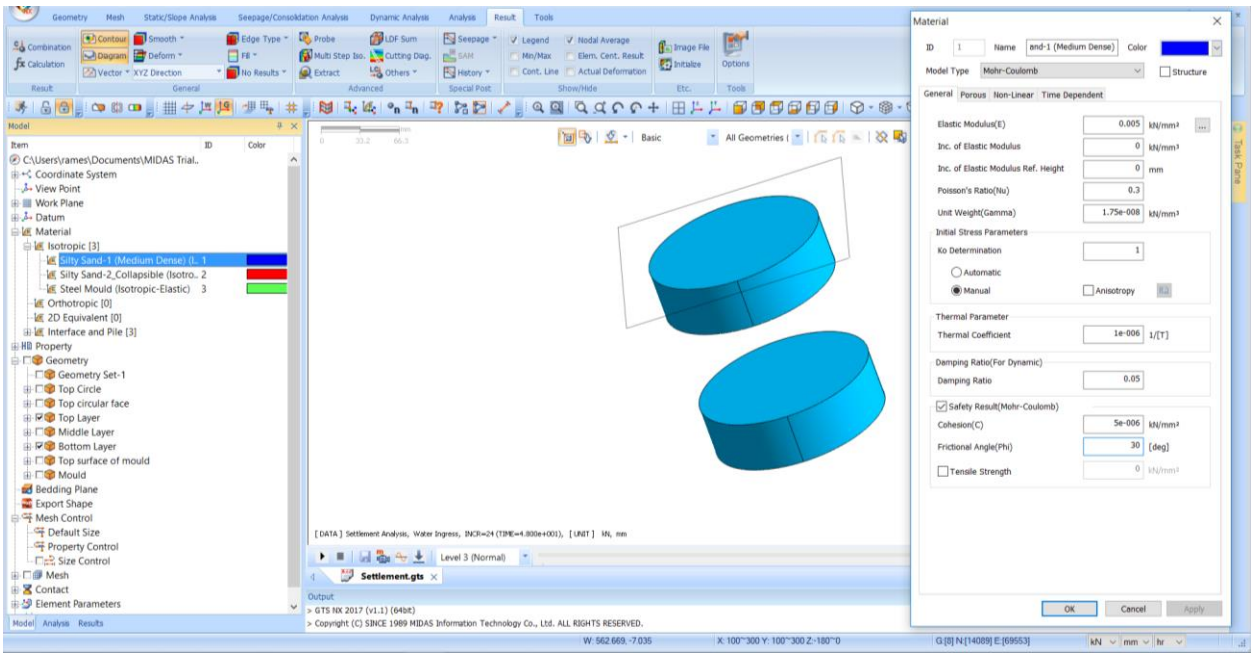


2) Details of soil layers
   i. Top layer (Non-collapsible soil) = 45mm
   ii. Middle layer (Collapsible soil) = 90mm
   iii. Bottom layer (Non-collapsible soil) = 45mm

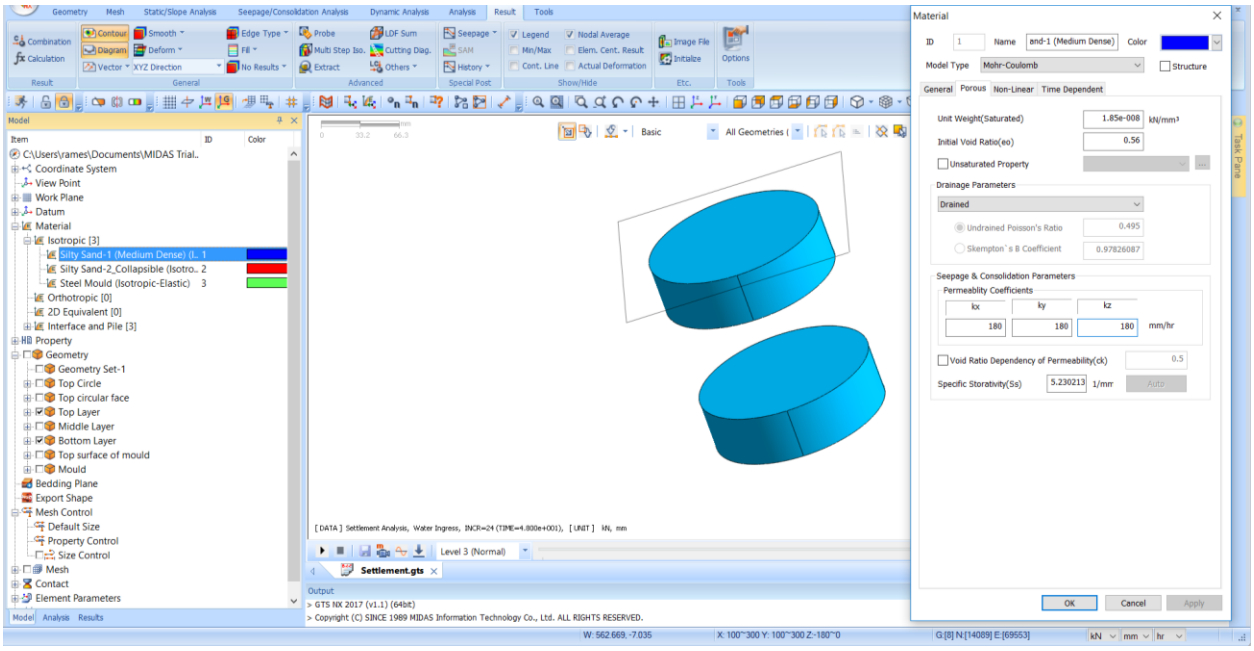


3) Properties of non-collapsible soil

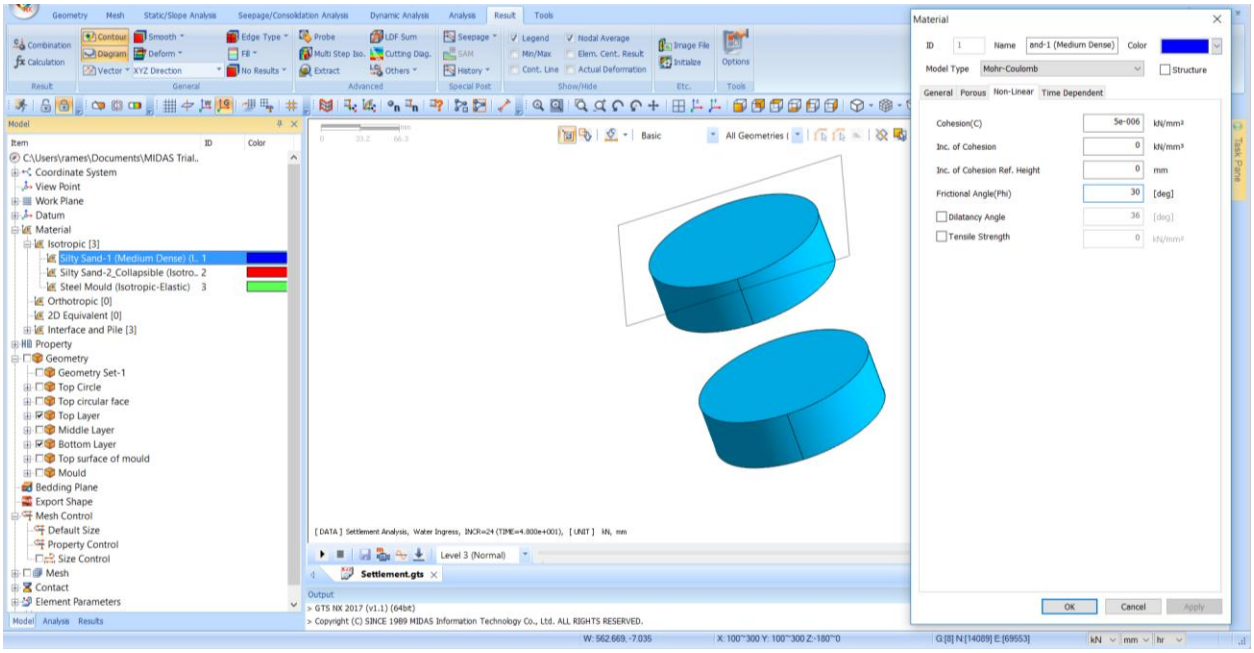
i. General properties



ii. Porous properties

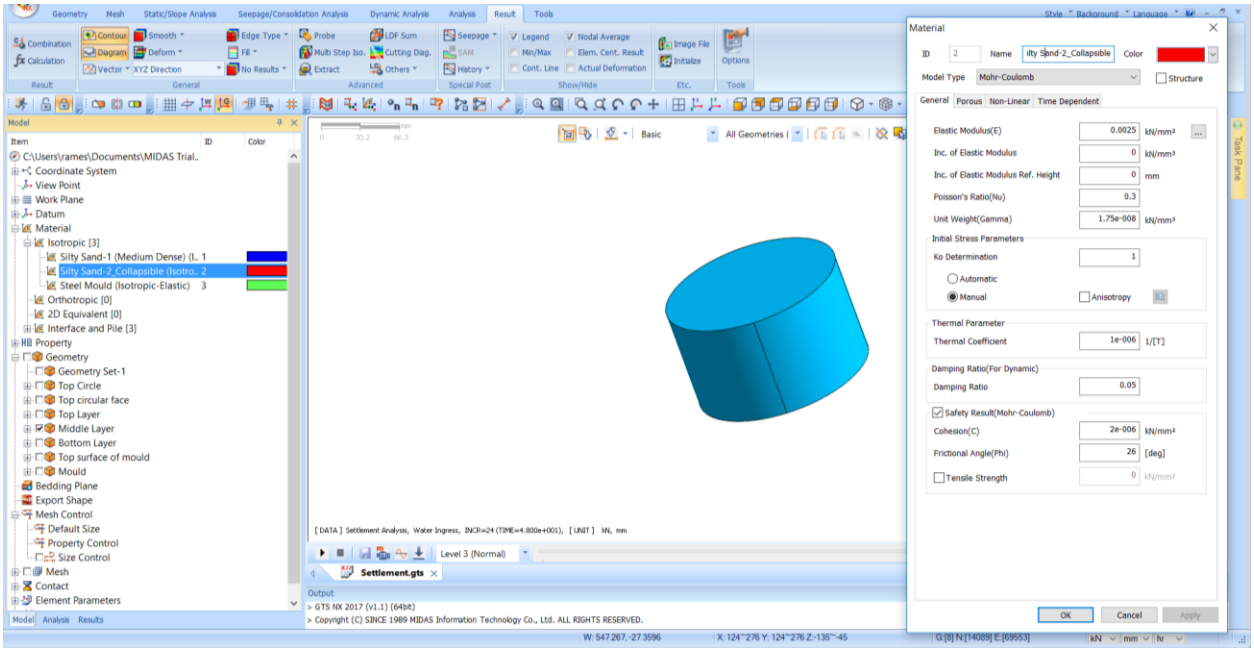


iii. Non-linear properties

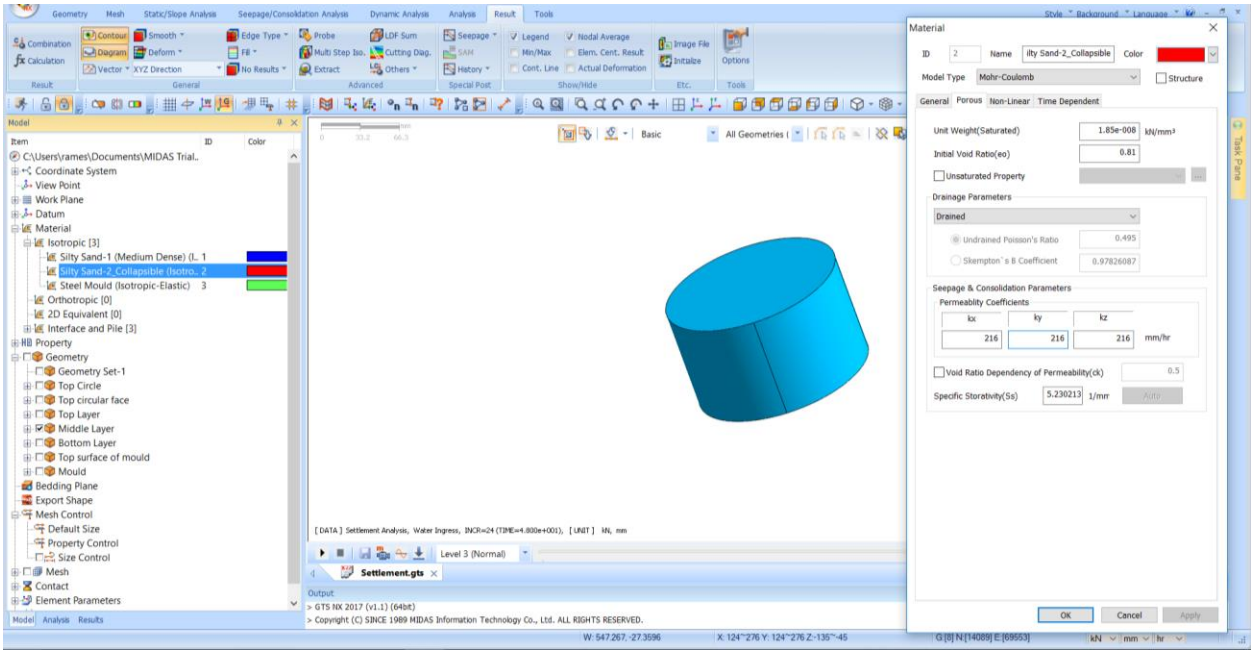


4) Properties of collapsible soil

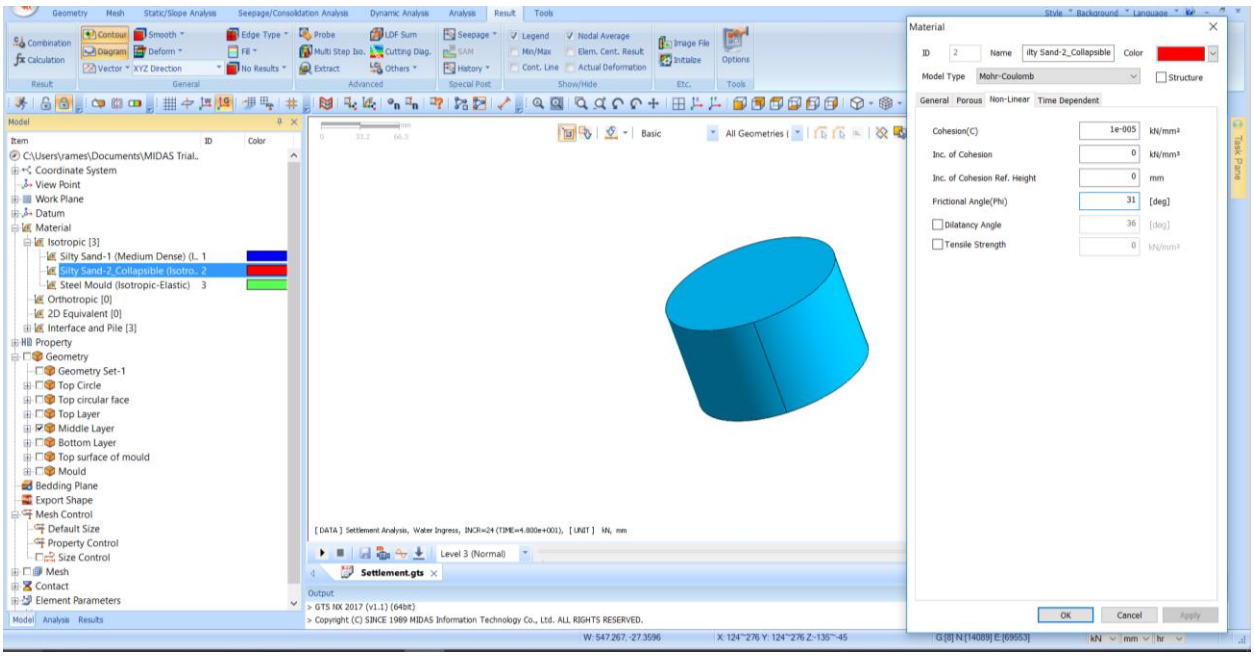
i. General properties



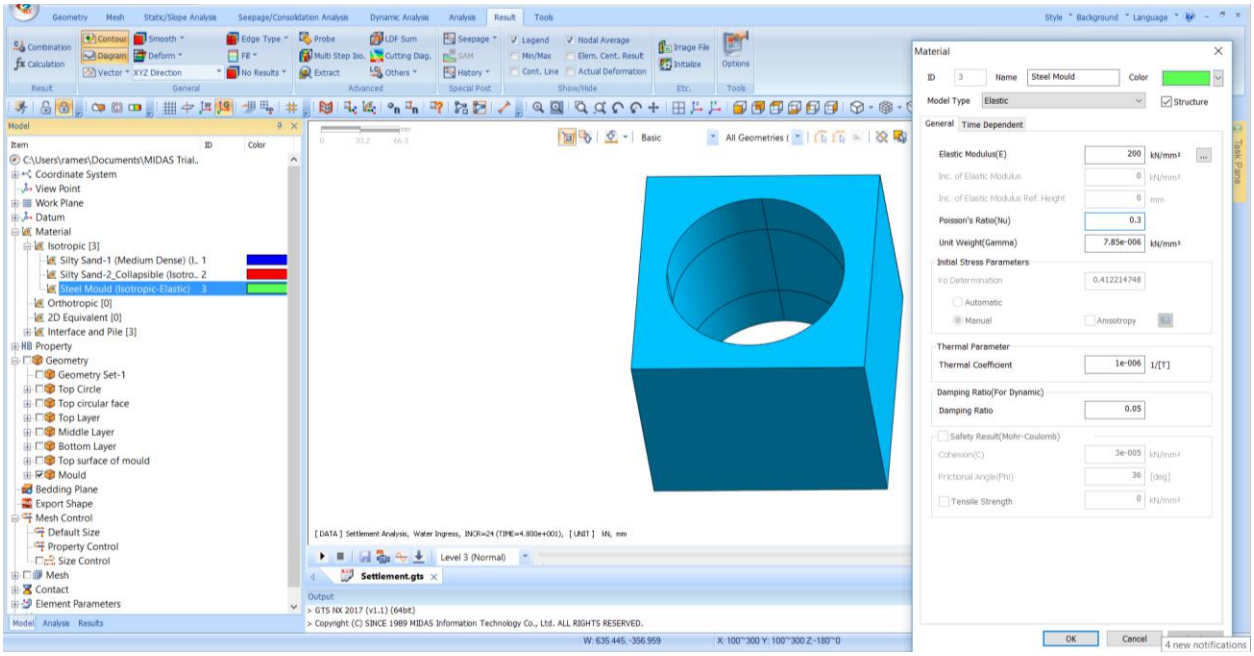
ii. Porous properties



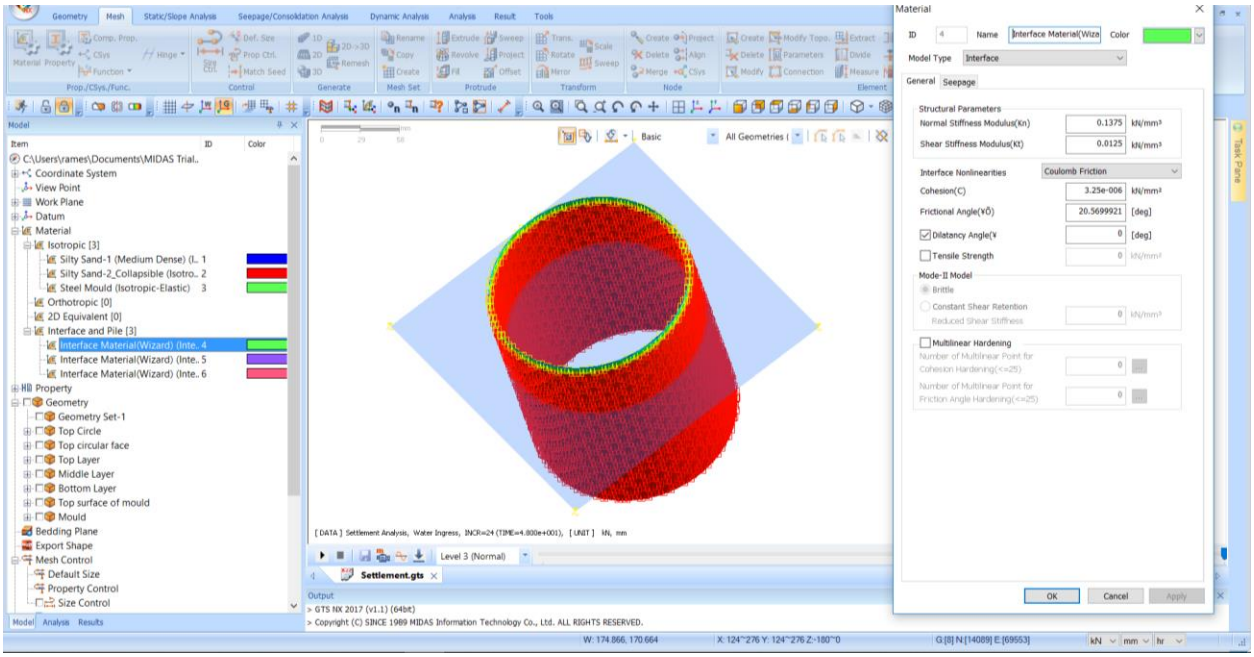
iii. Non-linear properties



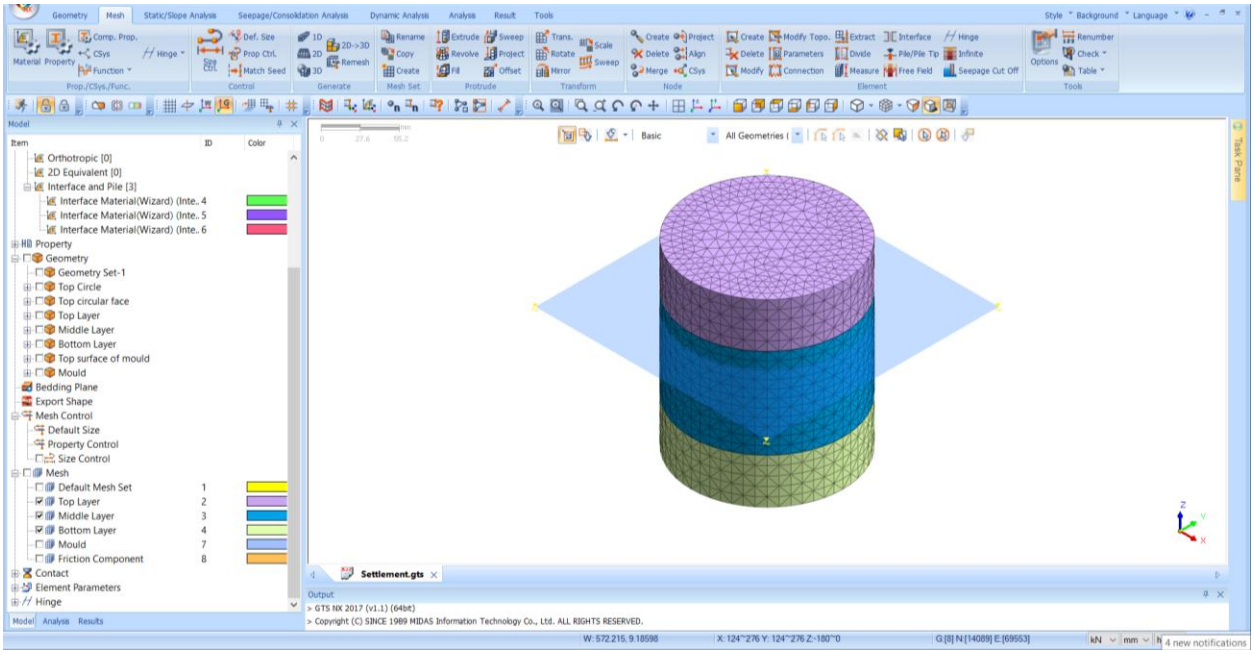
5) Properties of steel mould



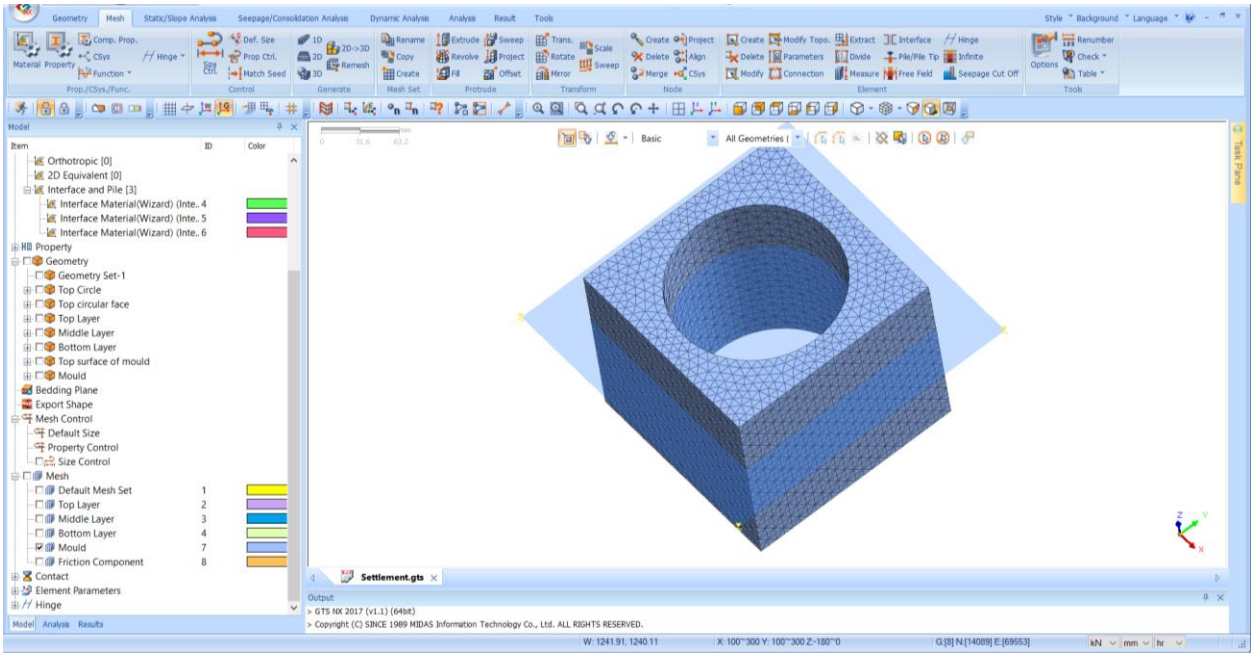
6) Interface properties i.e. simulation of friction between soil and steel mould with given strength reduction factor ® as 0.65



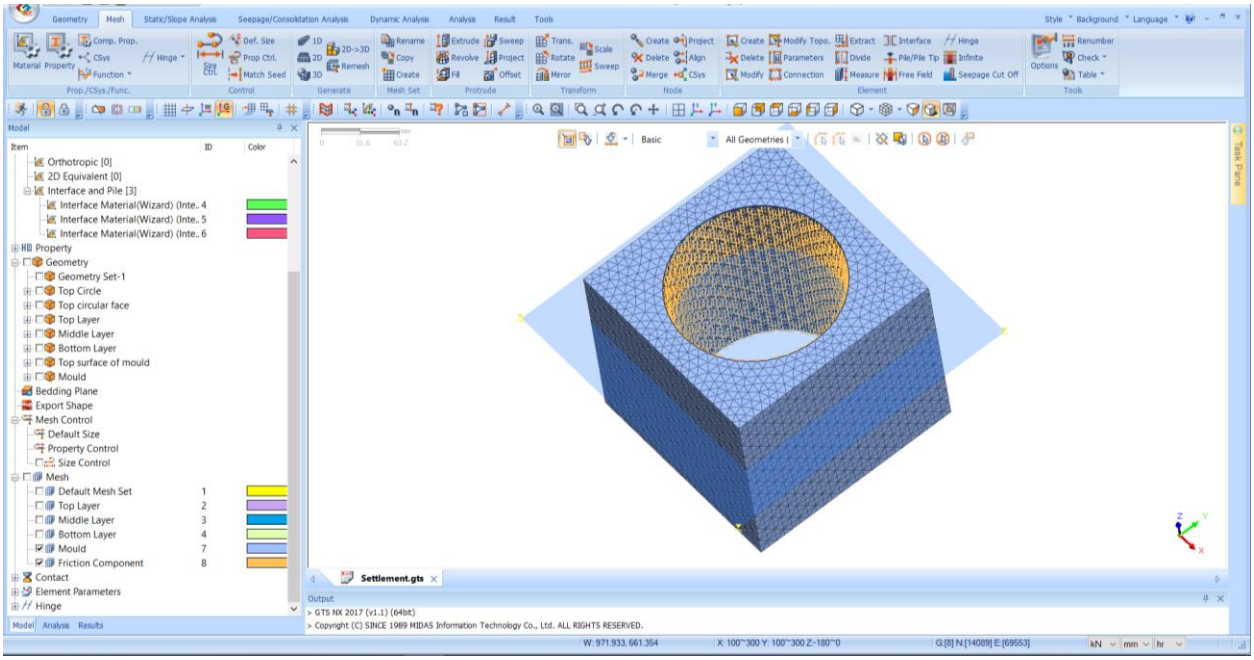
7) Meshed soil layers

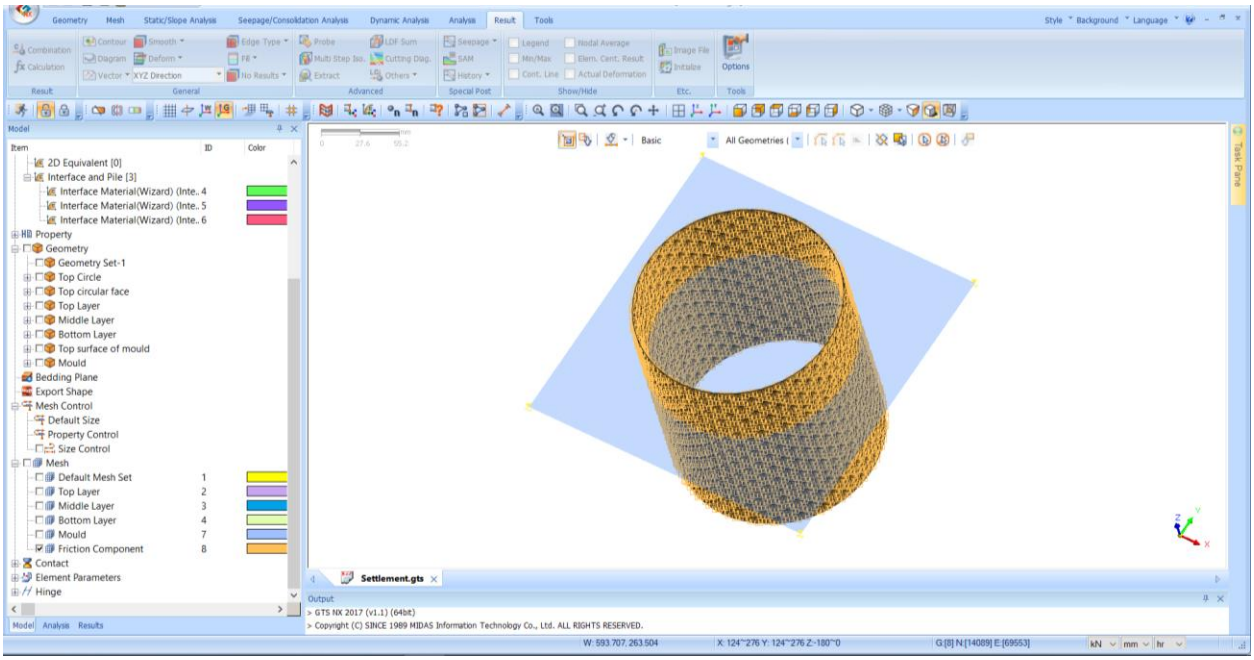


8) Meshed mould

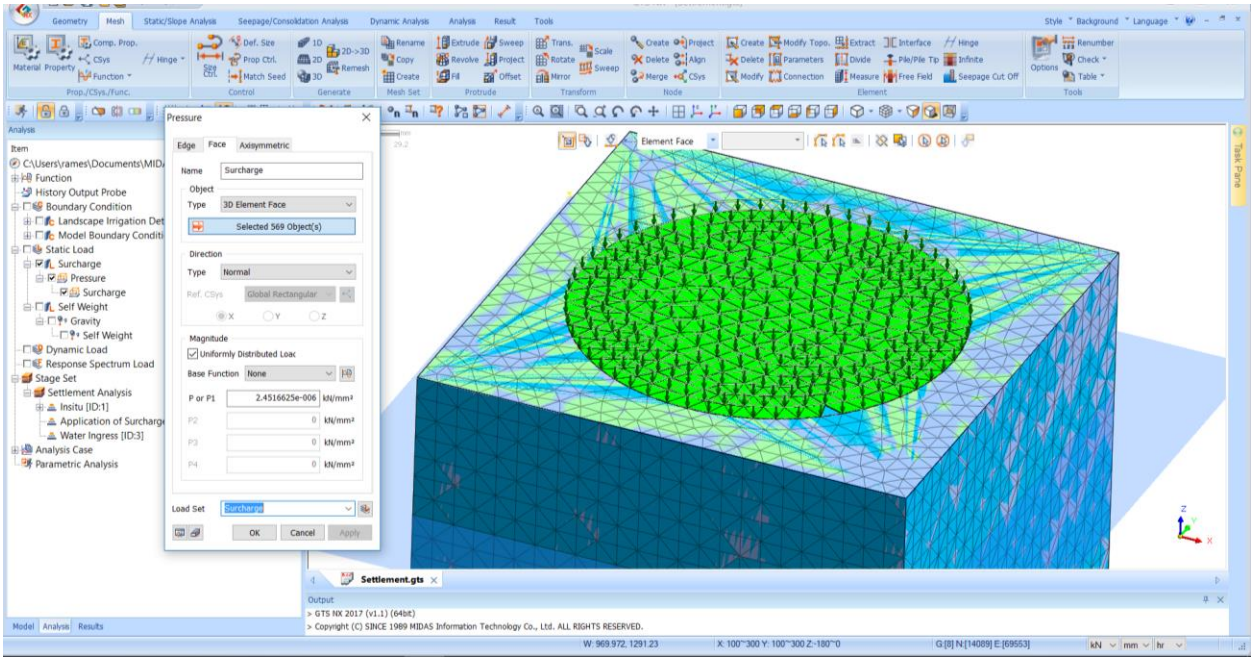


9) Meshed model with friction simulation between mould and soil

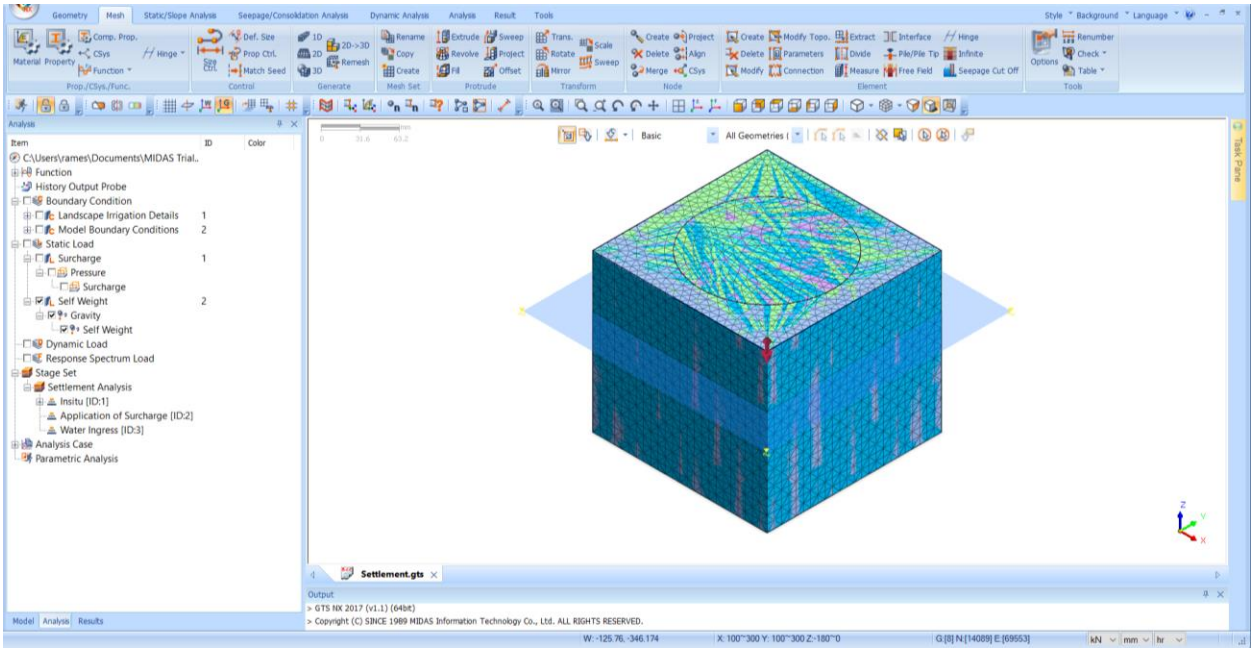




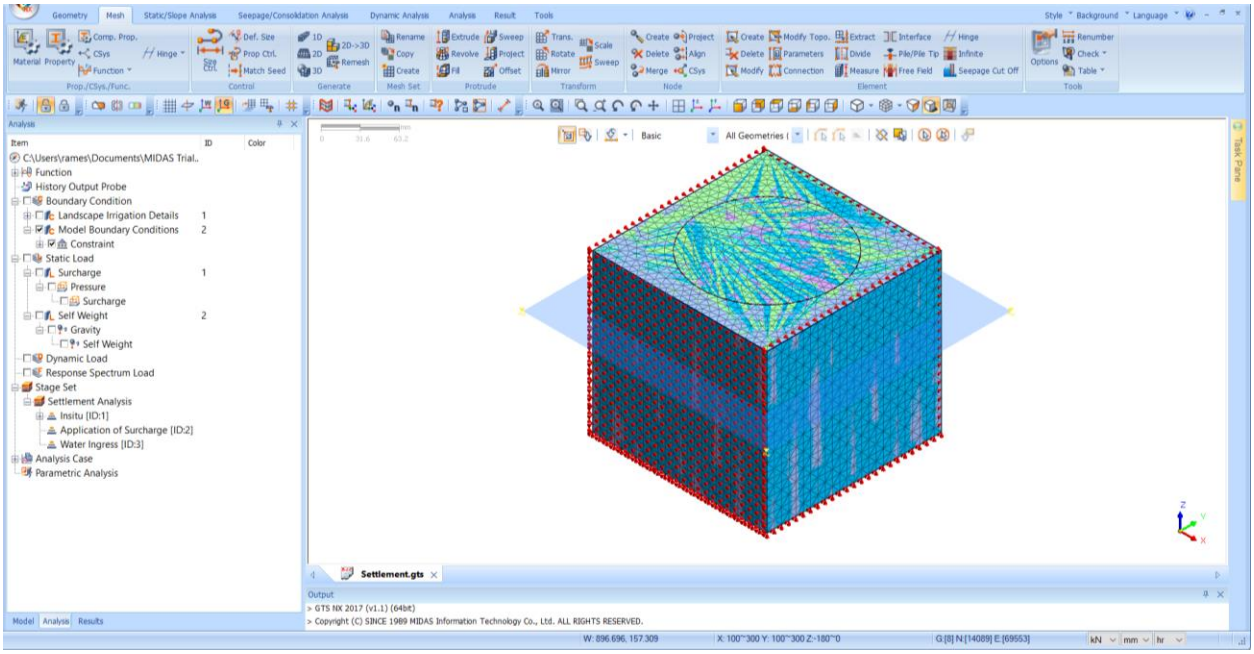
10) Surcharge load



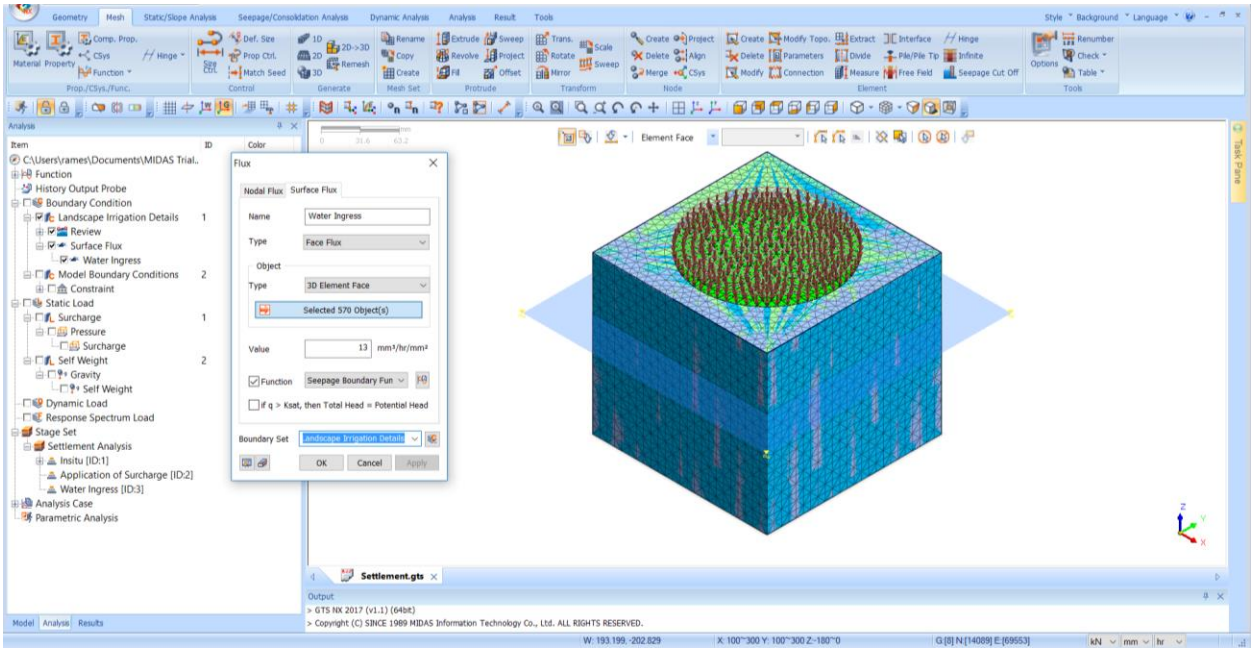
11) Self-weight of the model

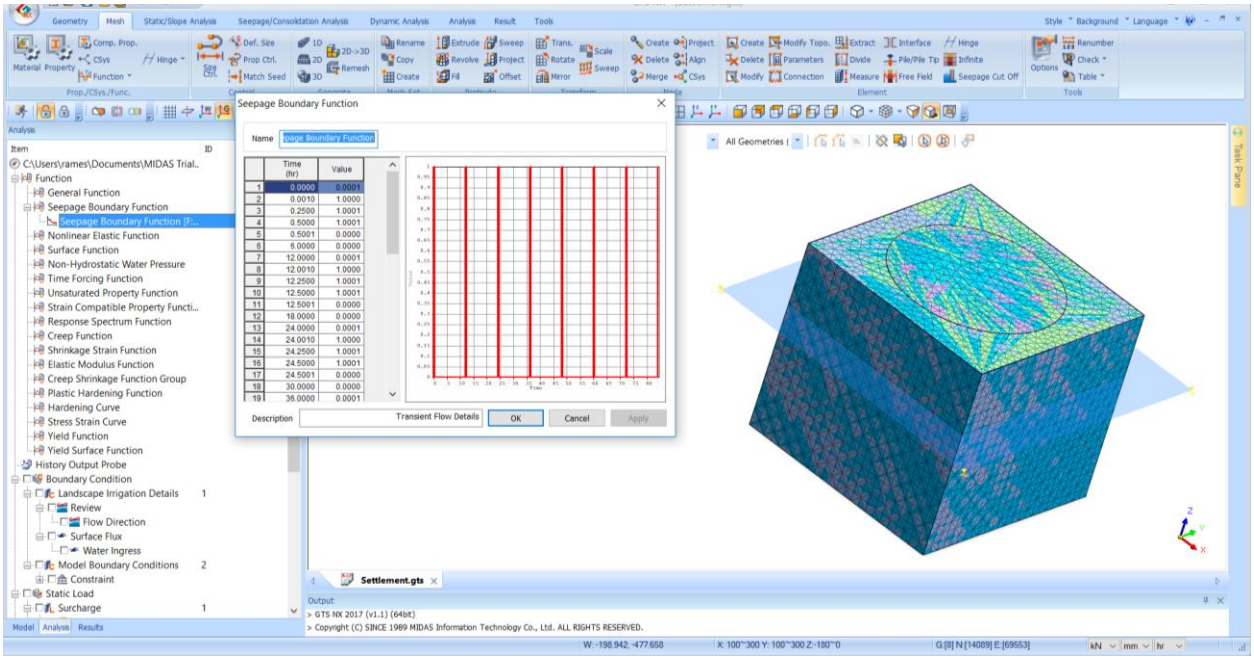


12) Boundary conditions of the model simulating the natural ground situation



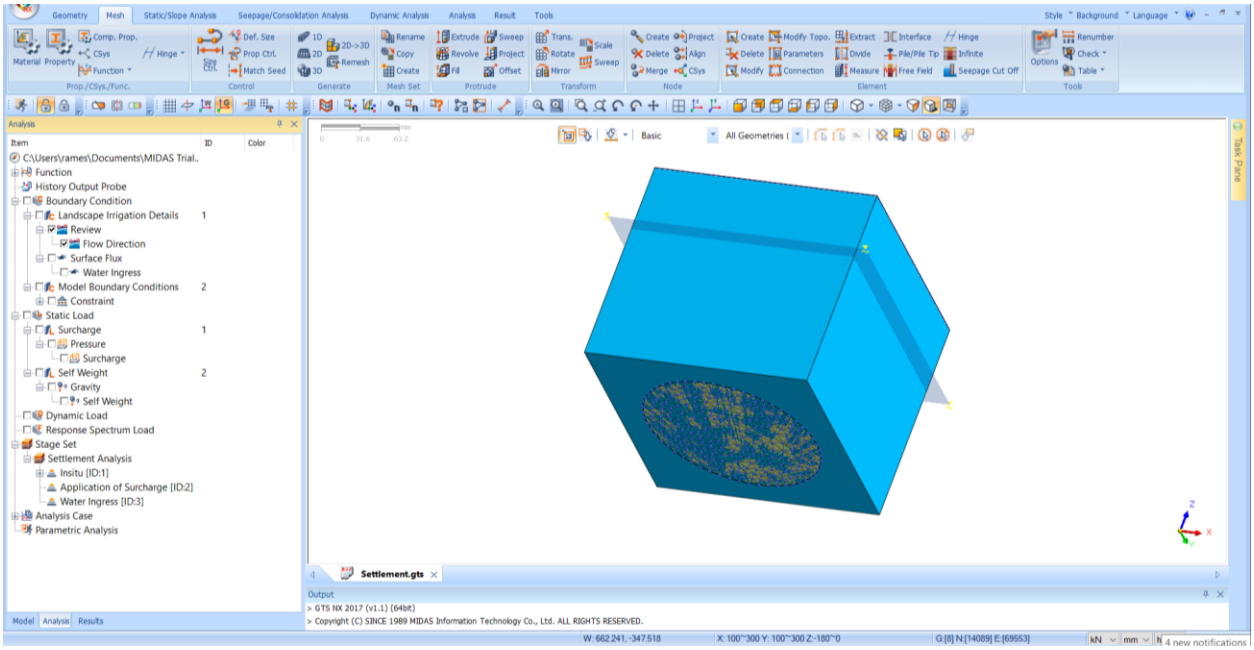
13) Simulation of drip irrigation





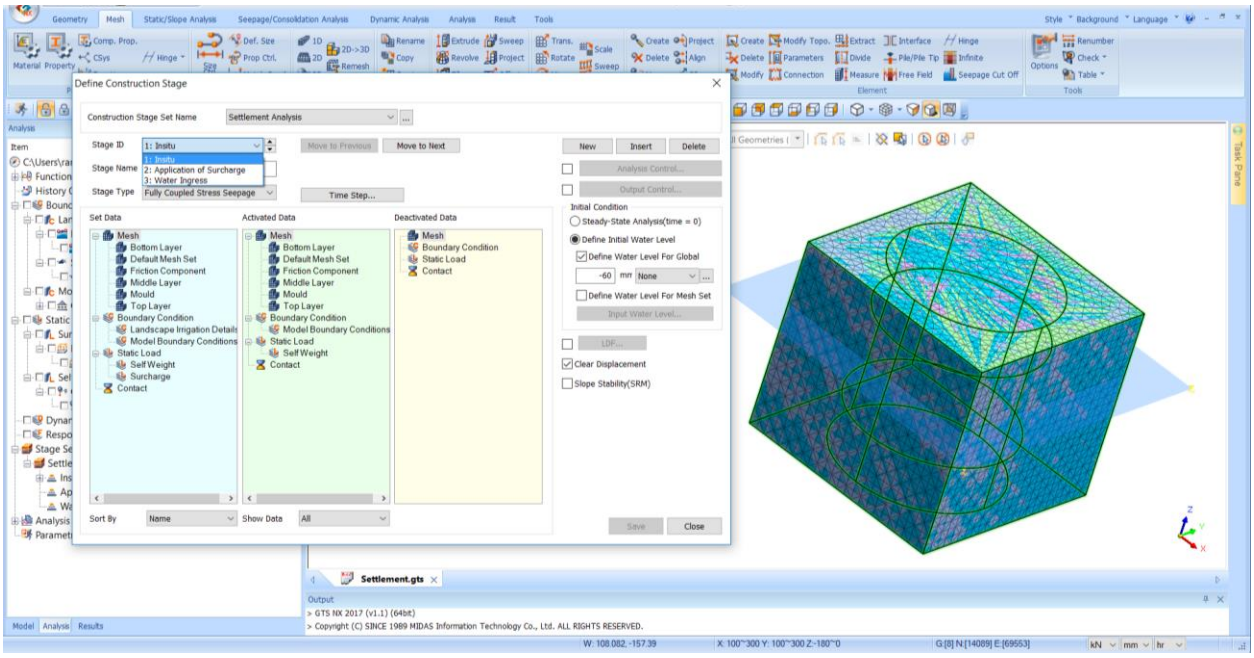
Simulation of drip irrigation cycles

14) Assigning the flow direction (review boundary)

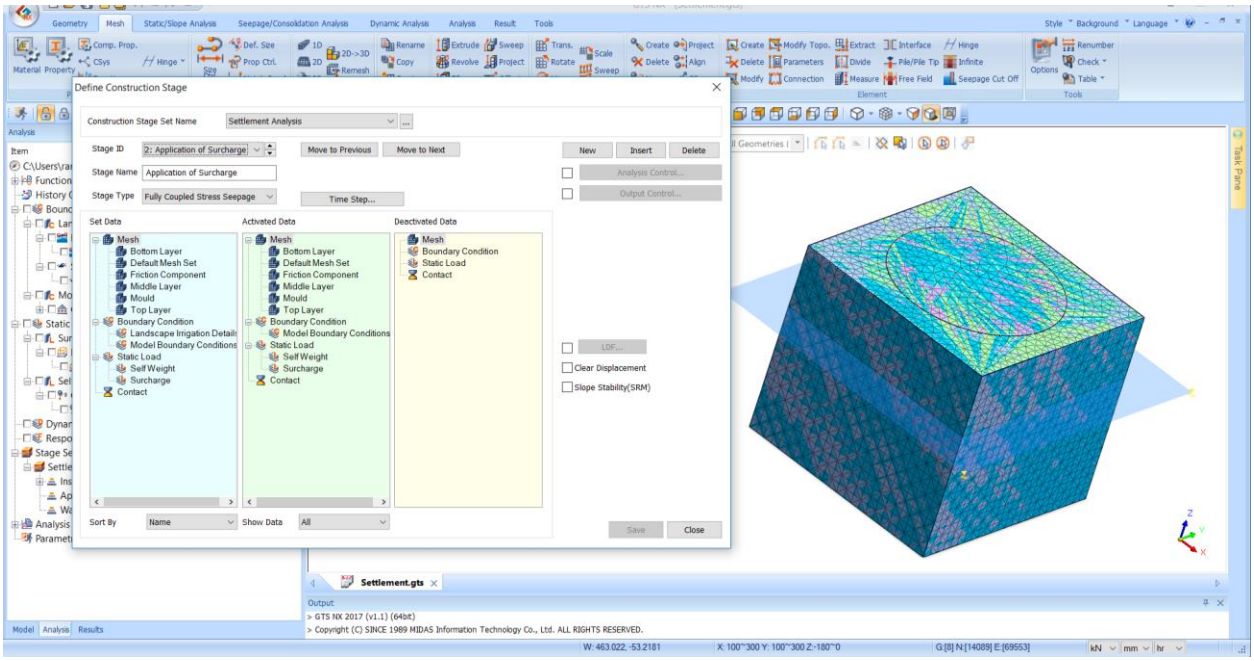


15) Different stages of analysis

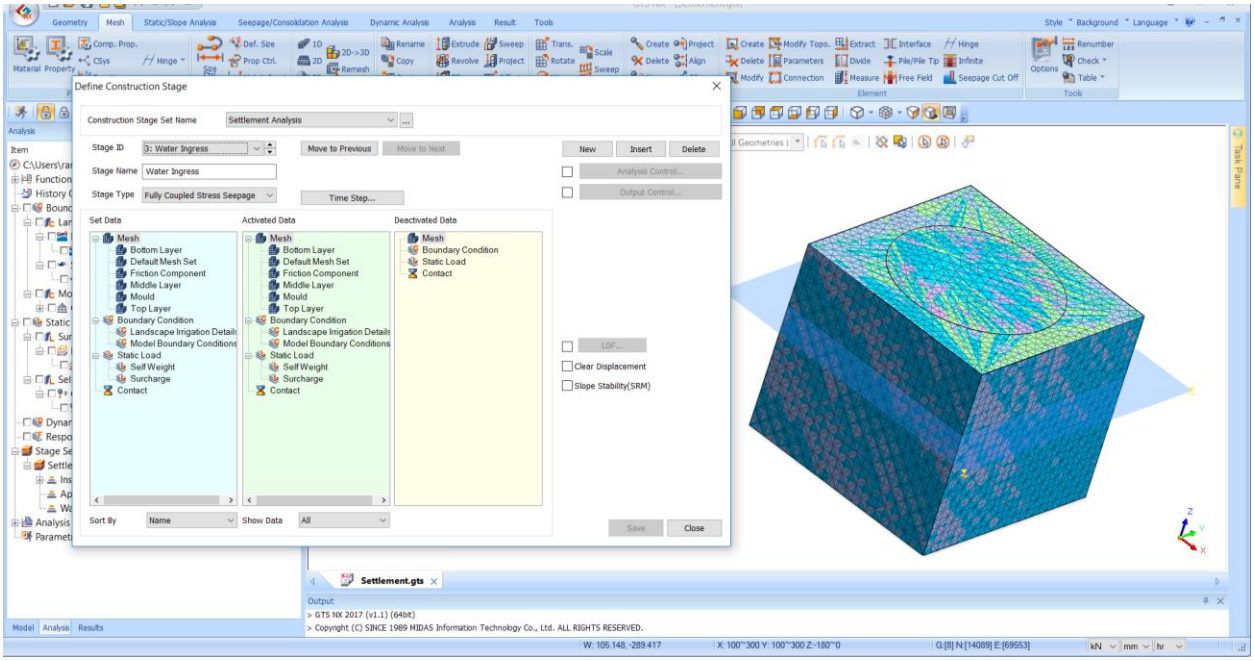
i. In-situ state with water table factor of 2/3 and nullifying the settlement due to self-weight



ii. Application of surcharge

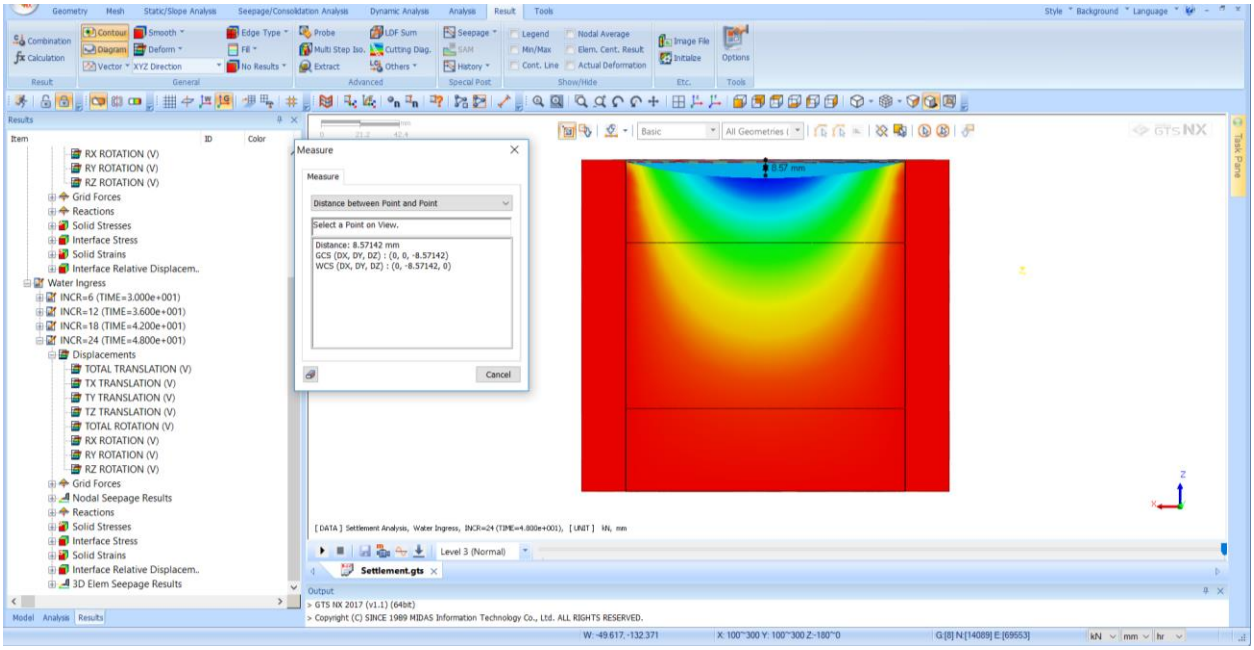


iii. Water ingress via drip irrigations

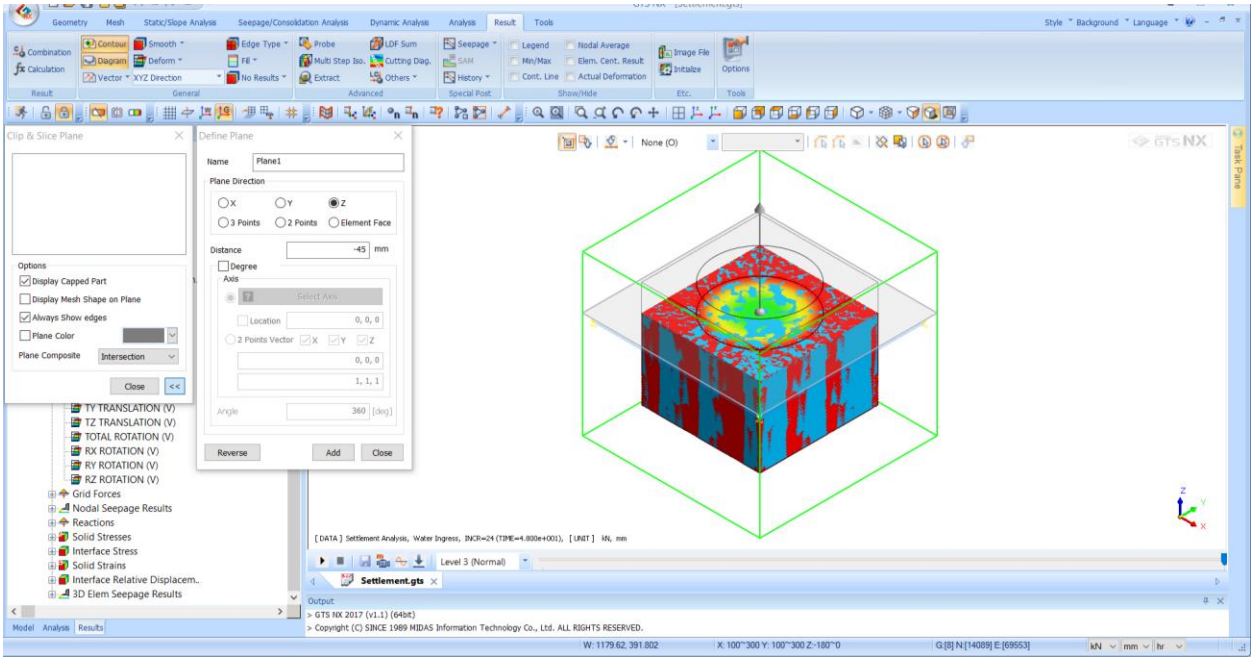


16) Results

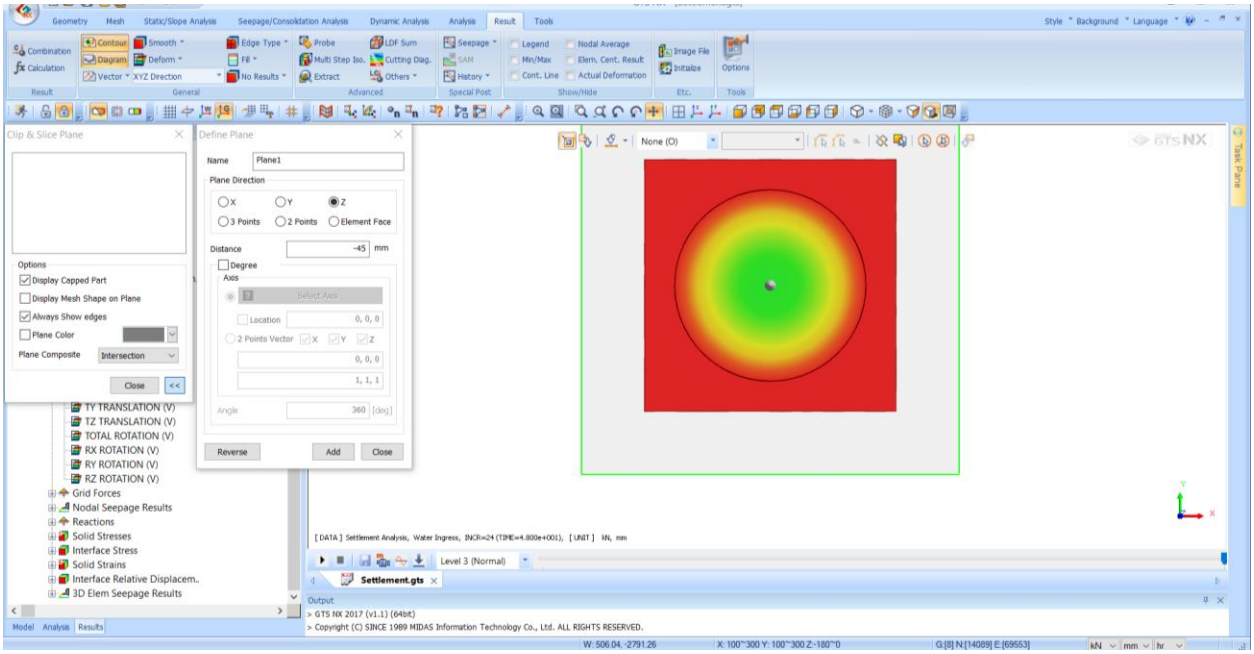
i. Maximum surface settlement = 8.57mm



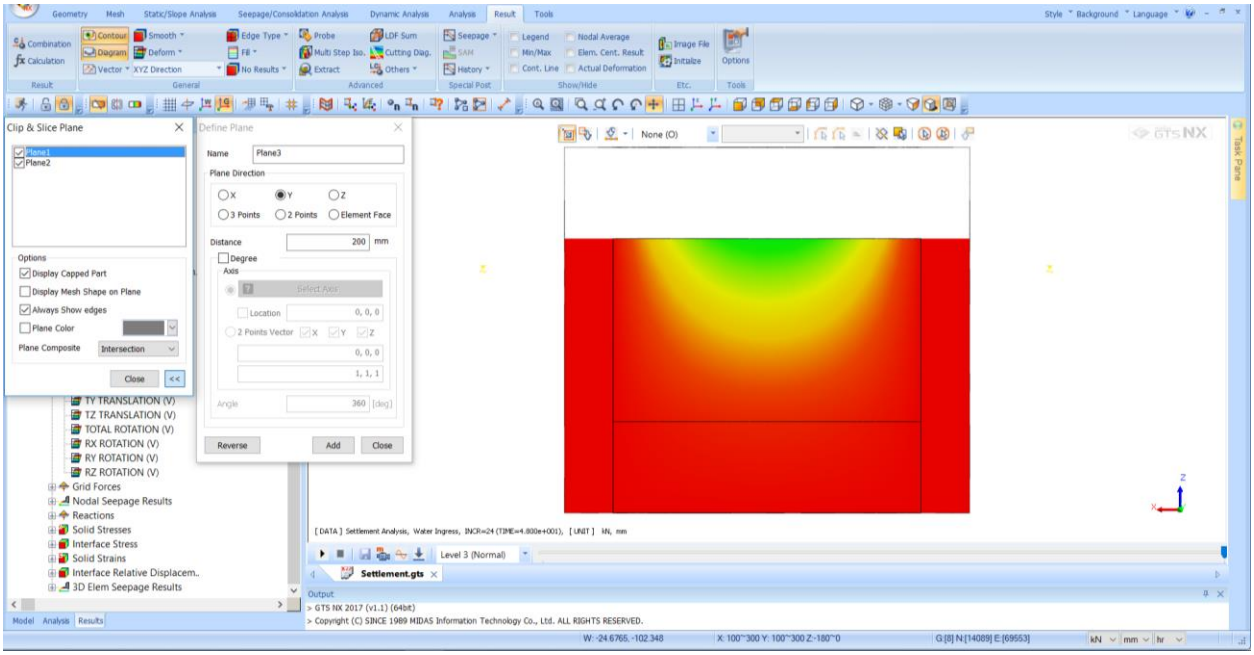
ii. Plane observed at the top of collapsible soil (45mm below the top surface)



3D view



Plan



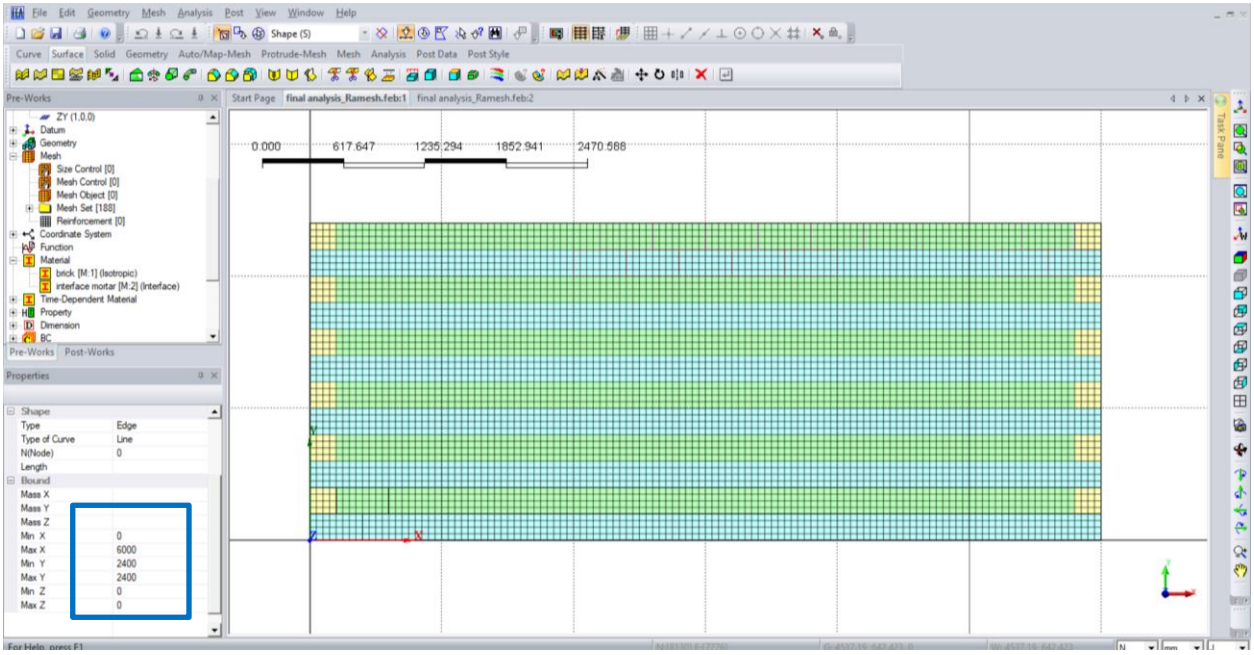
Section (settlement noticed = 0.48mm, which is not seen clearly in the image due to very less in magnitude)

STRUCTURAL MODELLING OF BOUNDARY WALLS

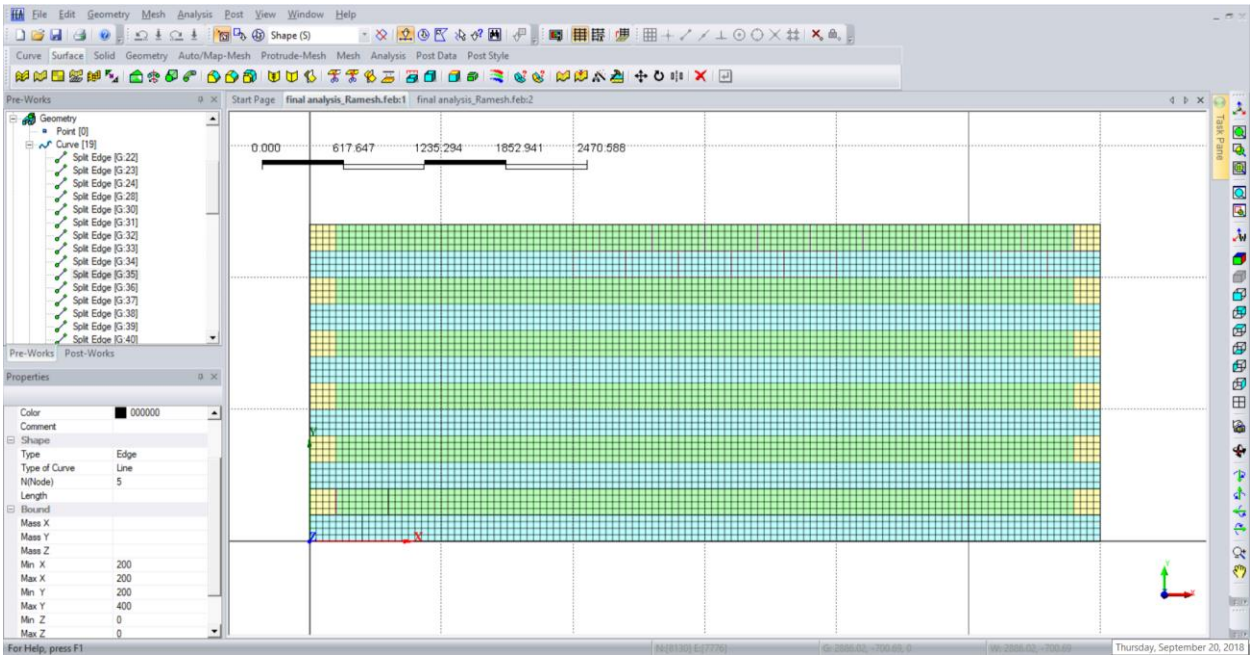
Structural modelling of boundary walls

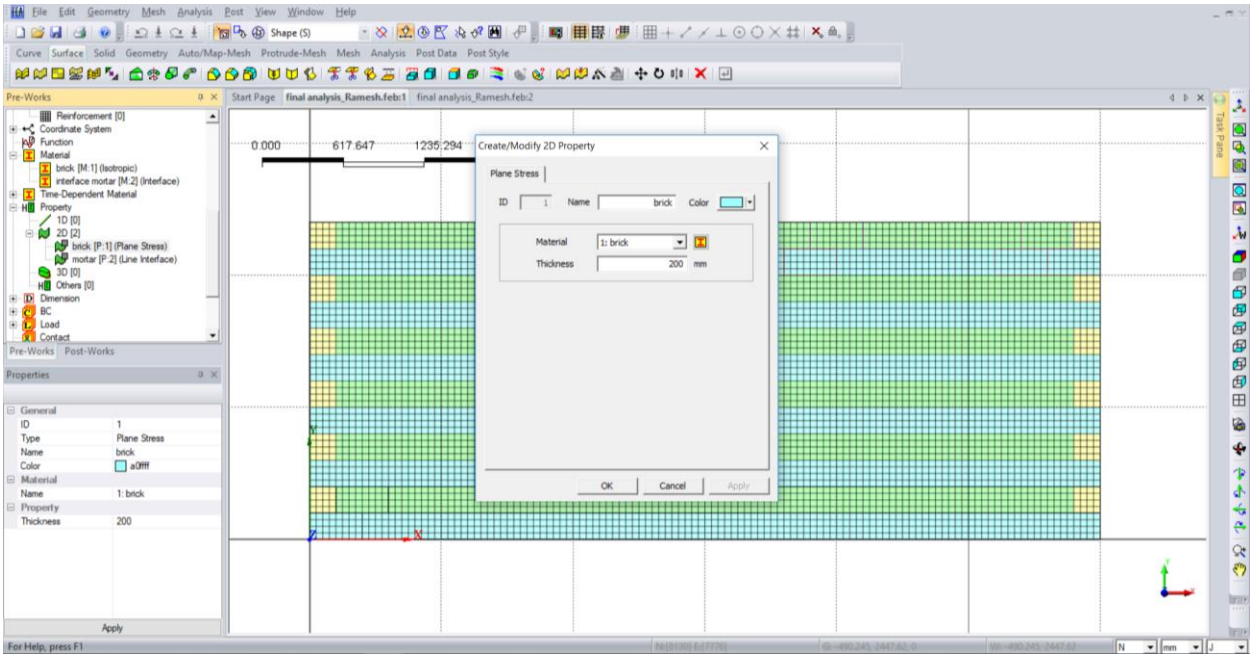
1) Dimensions of the modelled boundary wall

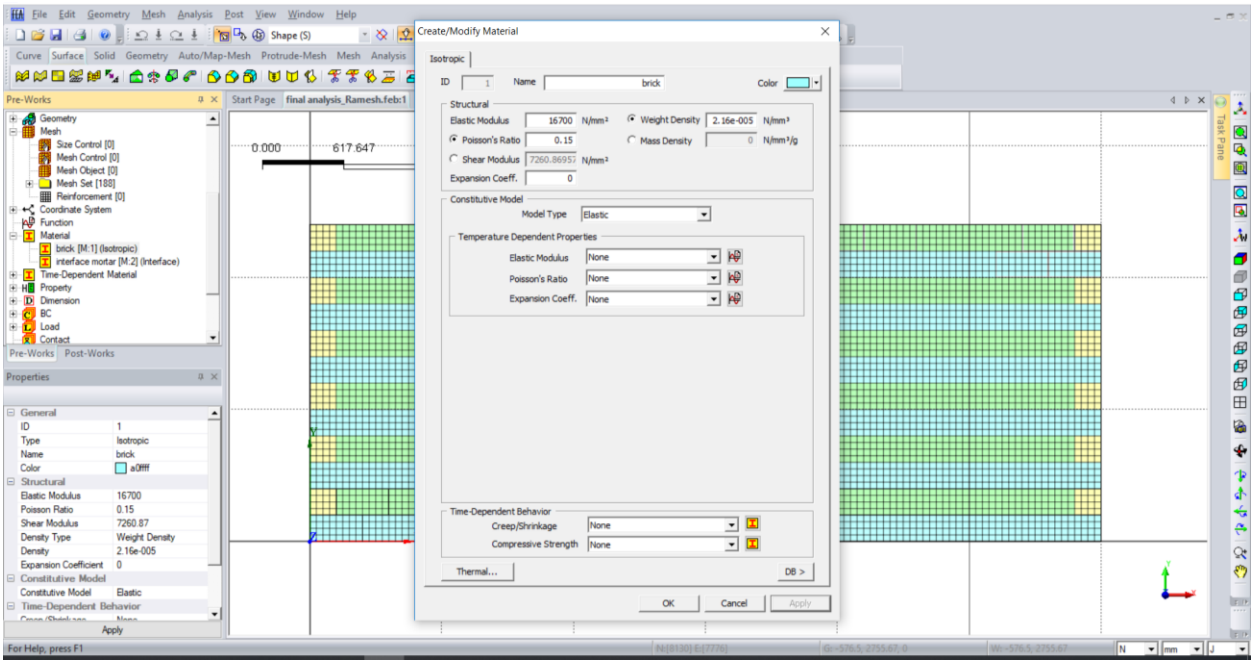
- Length = 6.0m
- Height = 2.4m
- Thickness=200mm



2) Details of bricks
    - Material: Cement concrete
    - Size (length x height x width) = 400x200x200mm
    - Elastic modulus = 16700 N/mm^2
    - Weight density = 21.6 kN/m^3

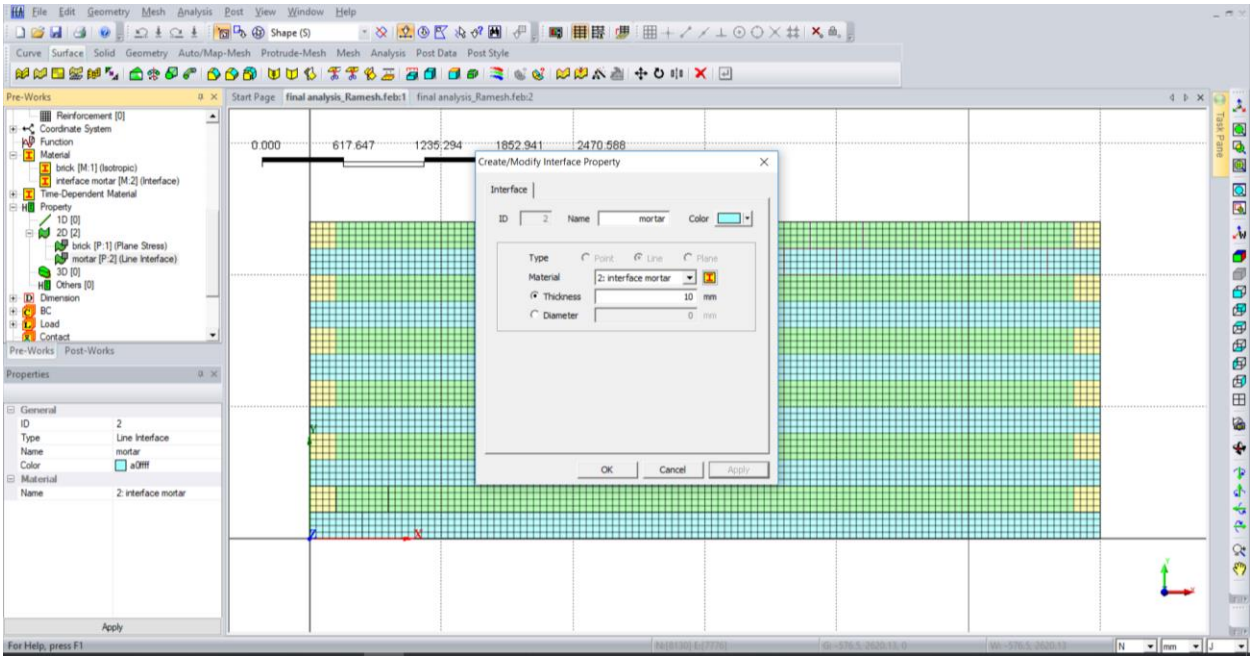






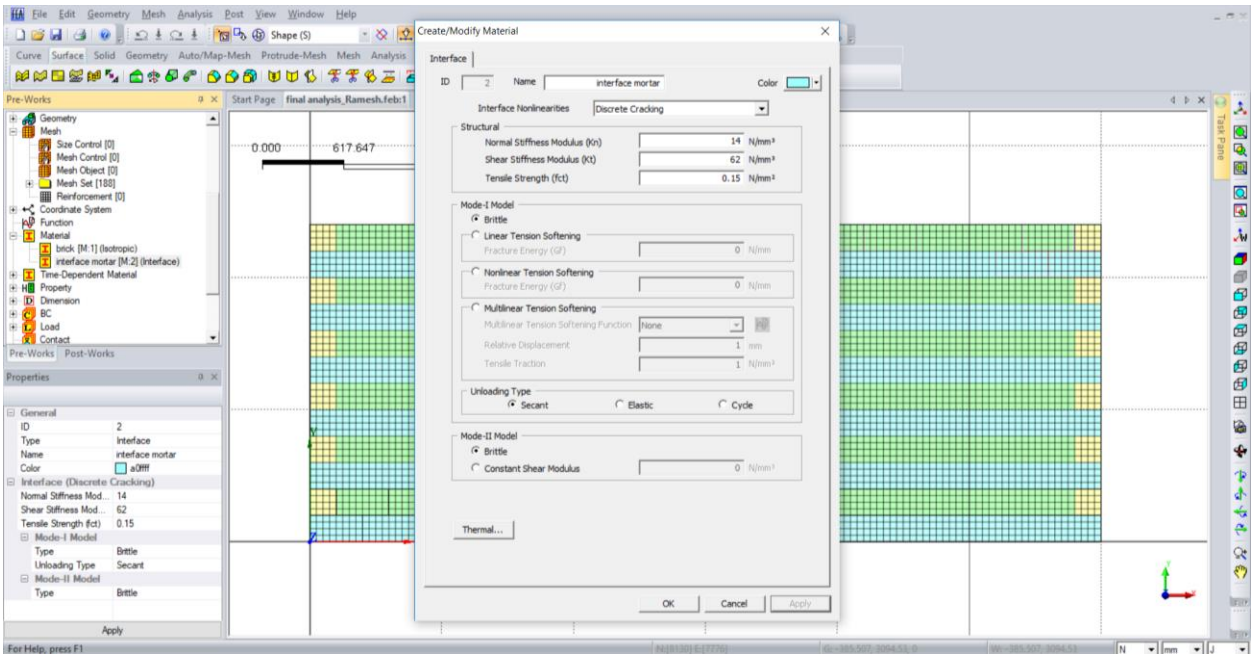
3) Details of mortar

- Material Cement mortar (1:6)
- Compressive strength = 7.5 N/mm^2
- Thickness = 10mm
- Tensile strength = 0.15 N/mm^2

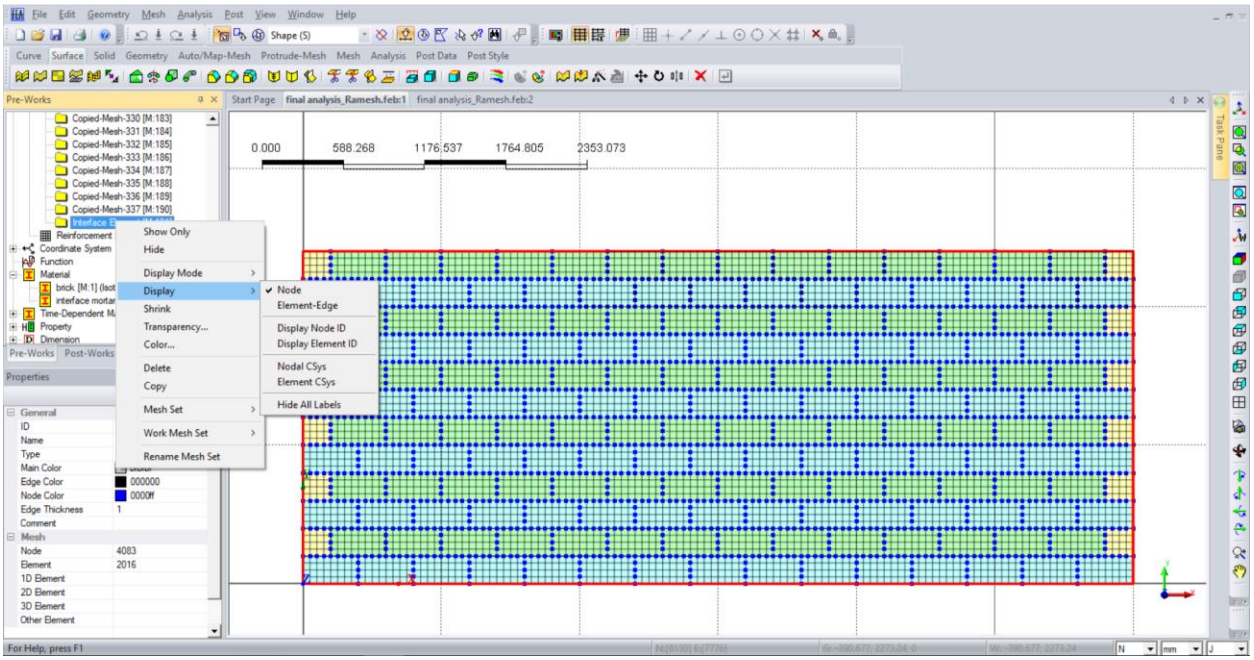


4) Interface properties

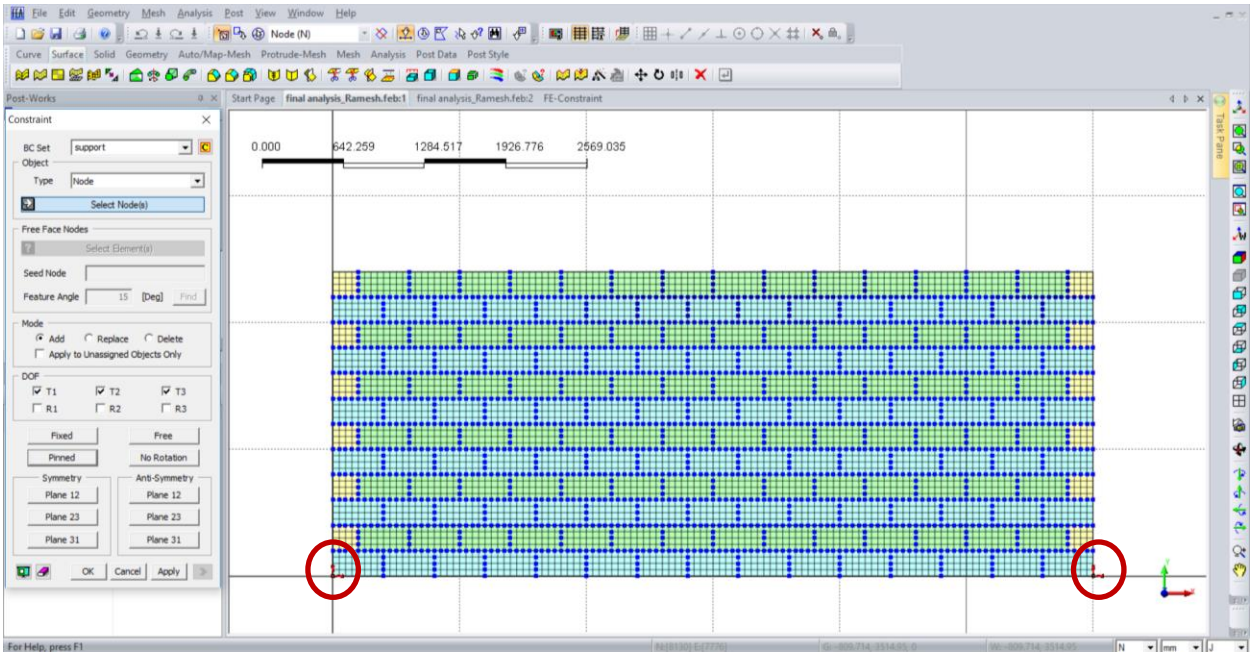
- Normal stiffness modulus = 14 N/mm^3
- Shear stiffness modulus = 62 N/mm^3



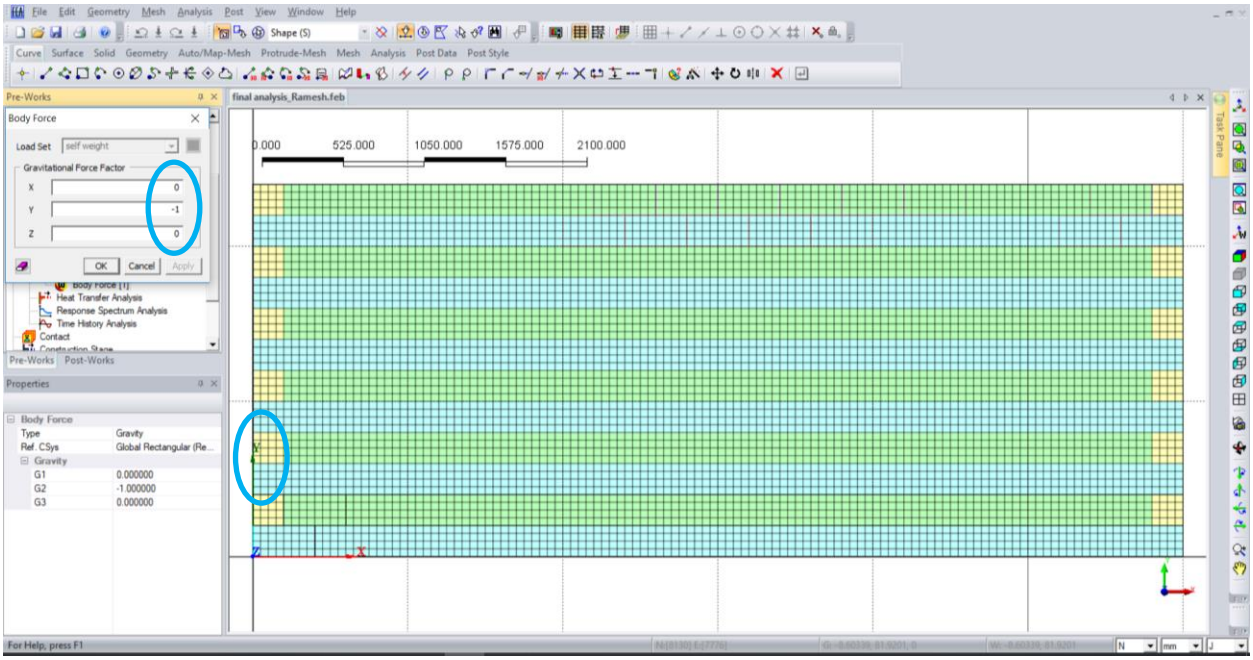
5) Meshed model with created nodes at all interfaces



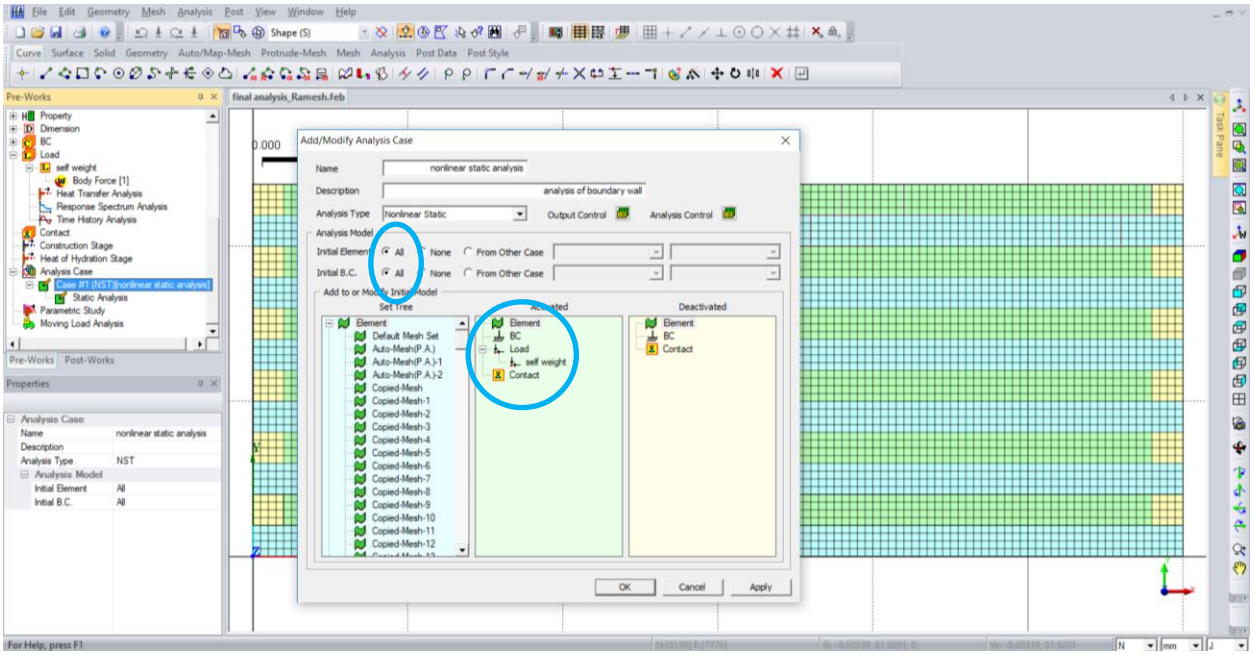
6) End supports : Pinned at both ends



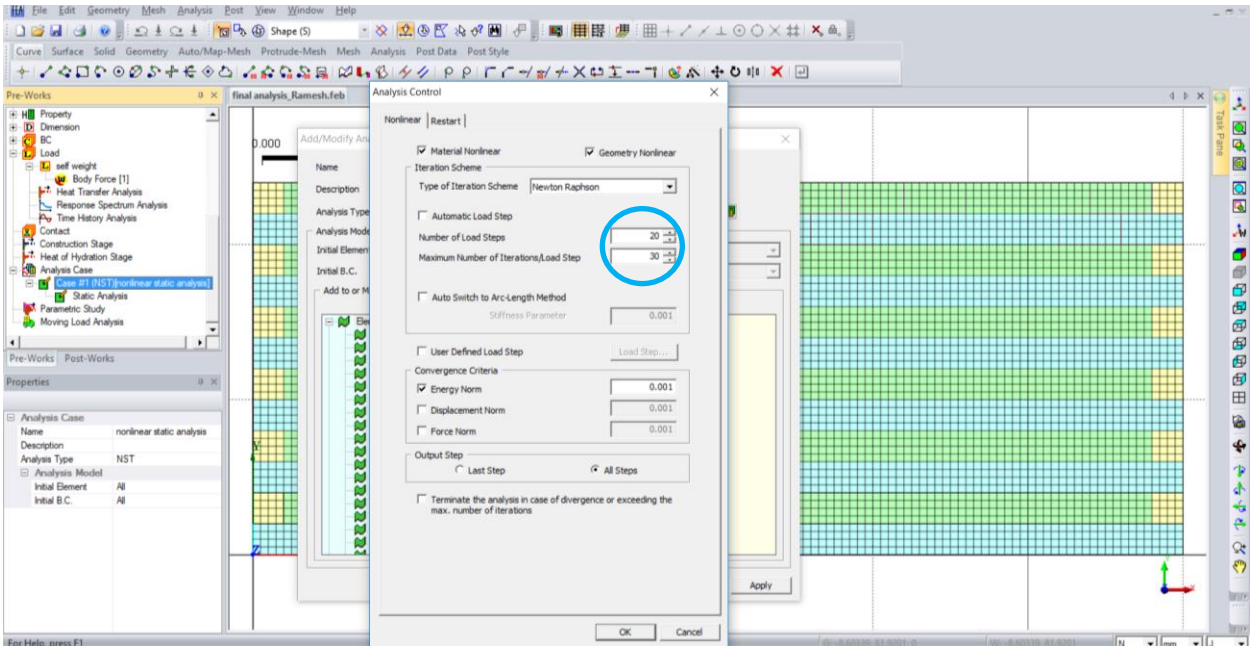
7) Loads of the boundary : Only self weight



8) Analysis Case – self weight of the wall is activated with no other external load

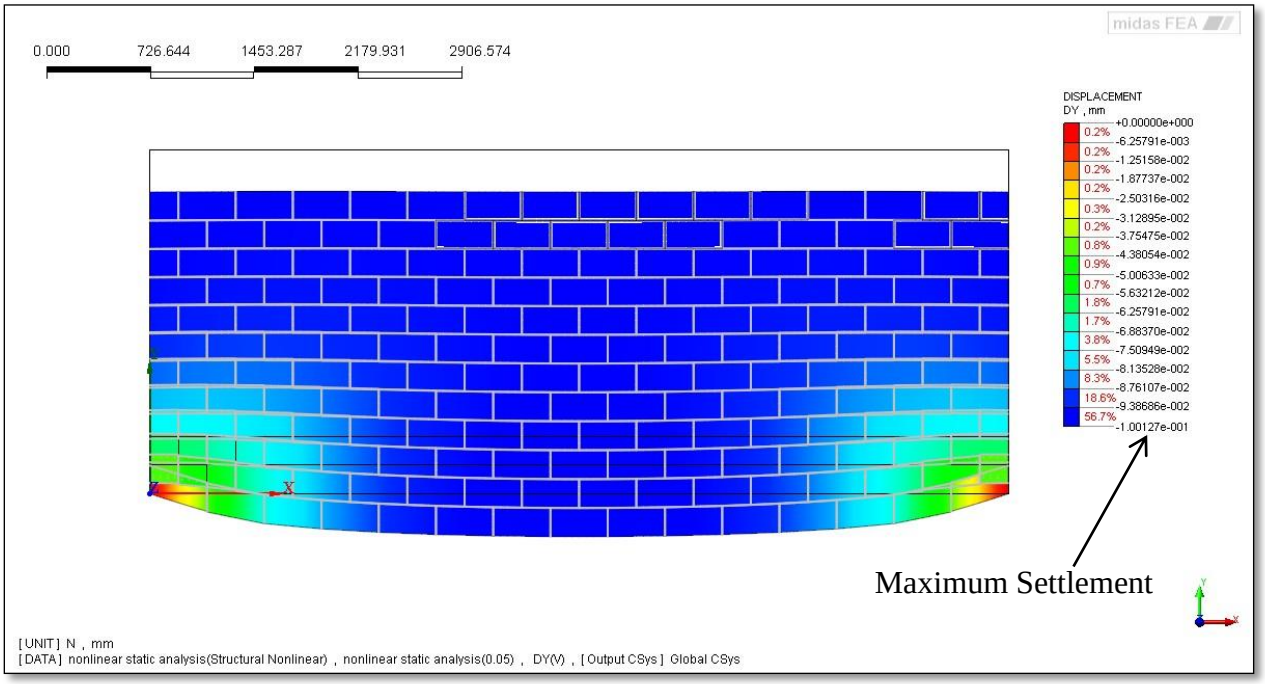


9) Analysis control – Selfweight of the wall is transferred to the analysis system in 20 incretemental stages (i.e. 5%, 10%, 15%, 20%,……..90%, 95% and 100%).

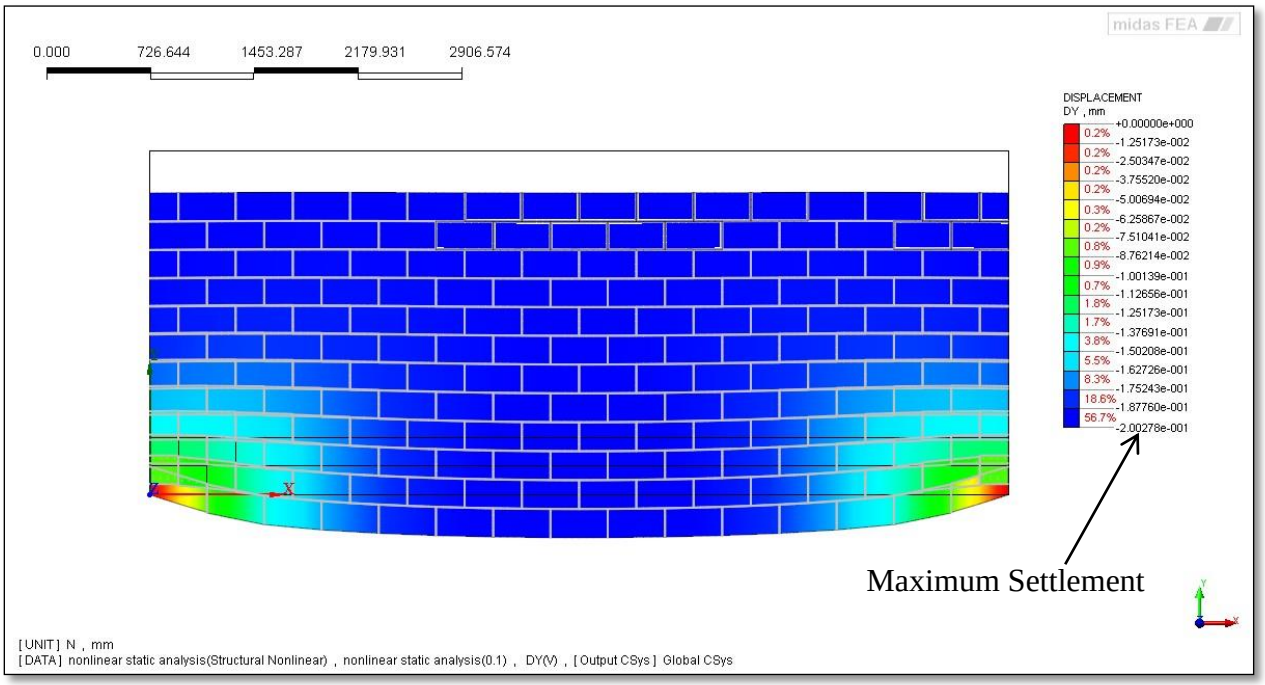


10) Results

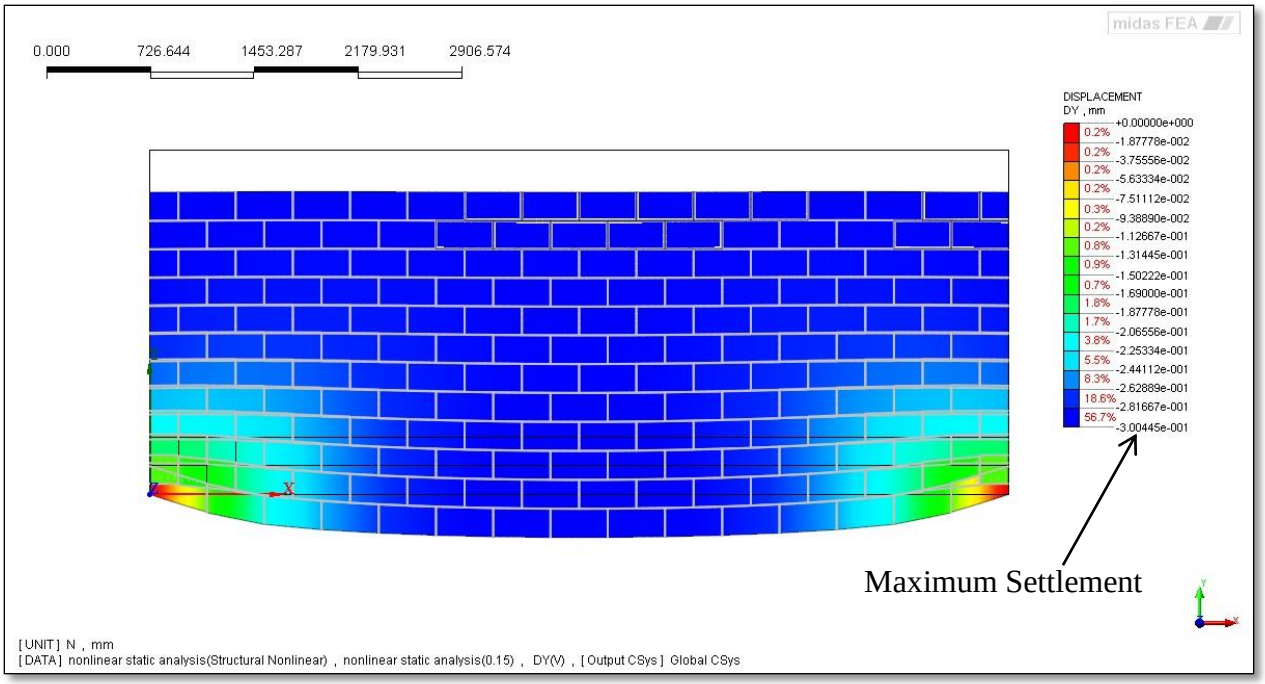
a) Deformation of boundary wall @5% self weight



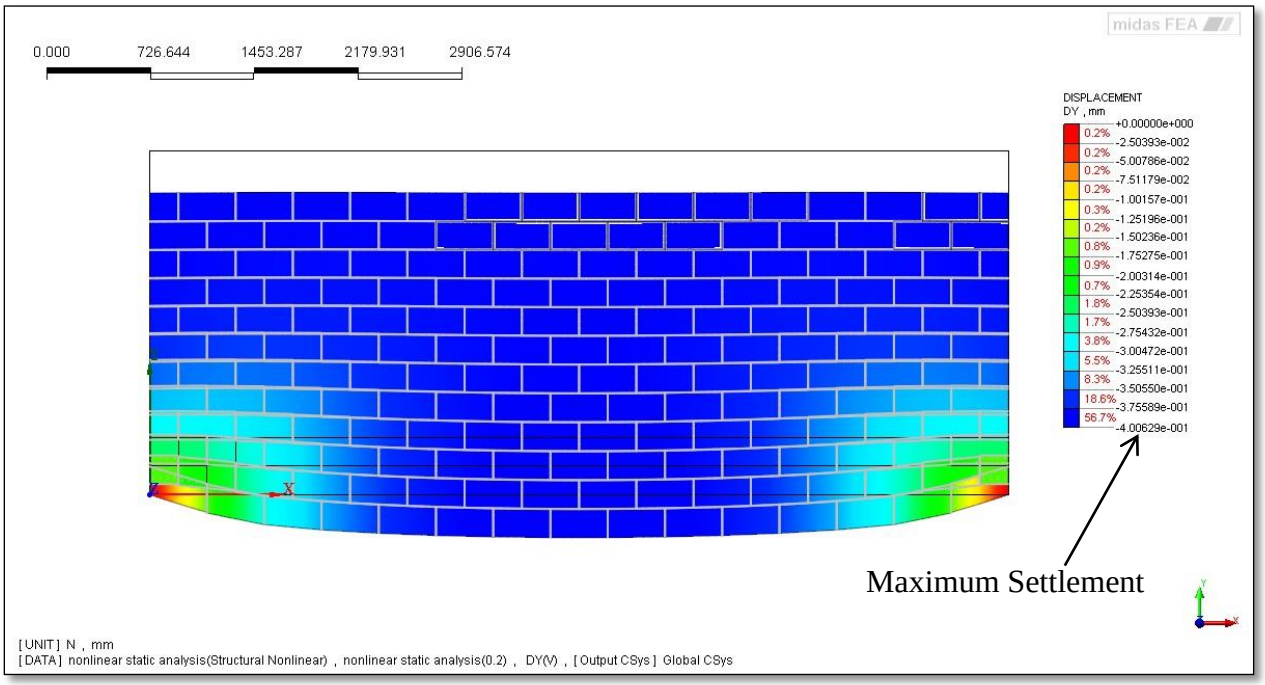
b) Deformation of boundary wall @10% self weight



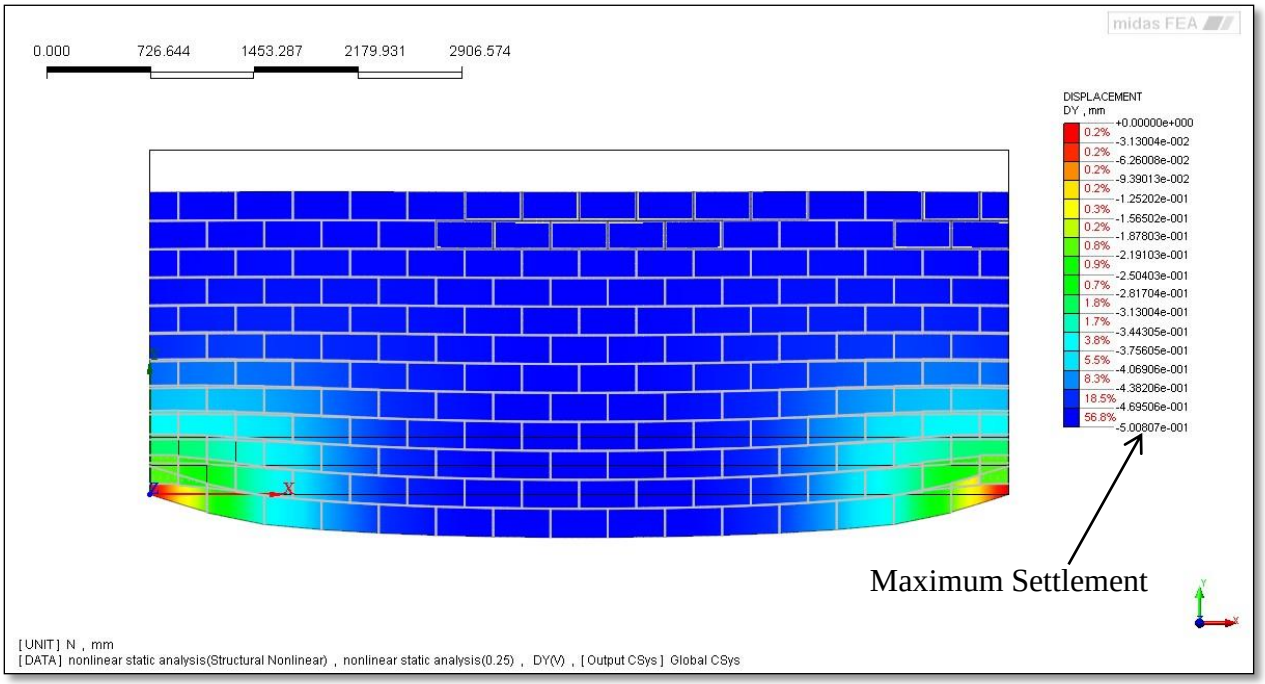
c) Deformation of boundary wall @15% self weight



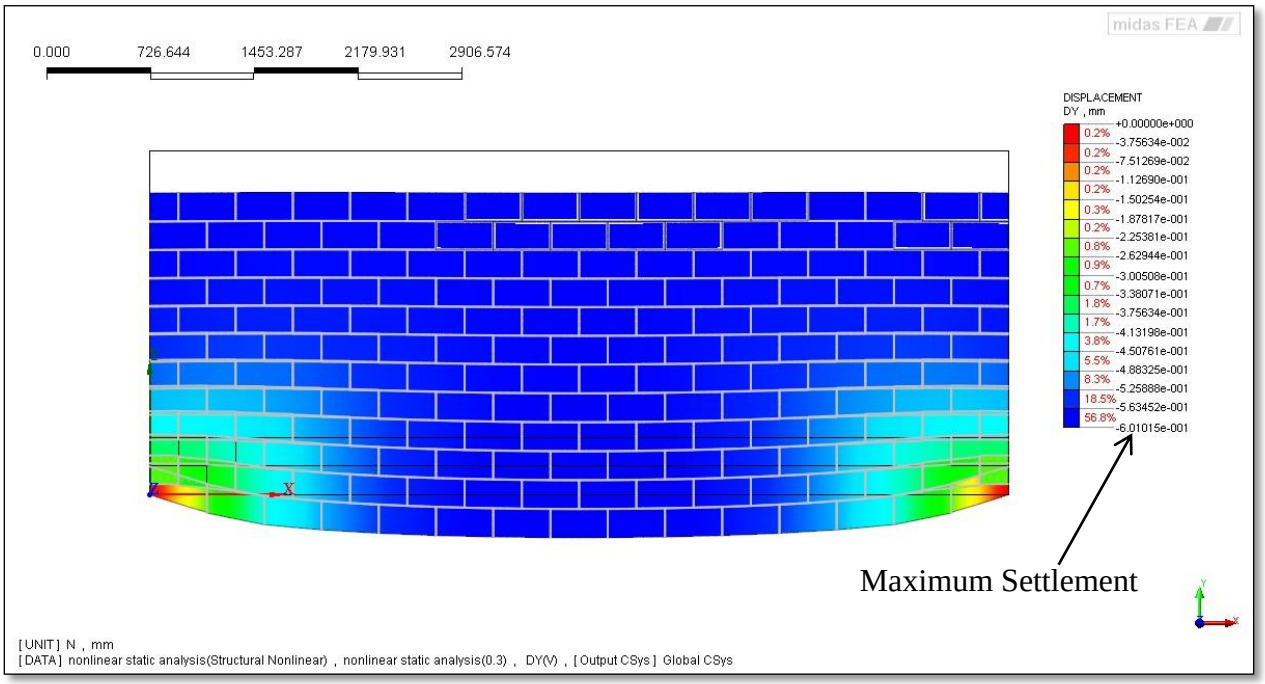
d) Deformation of boundary wall @20% self weight



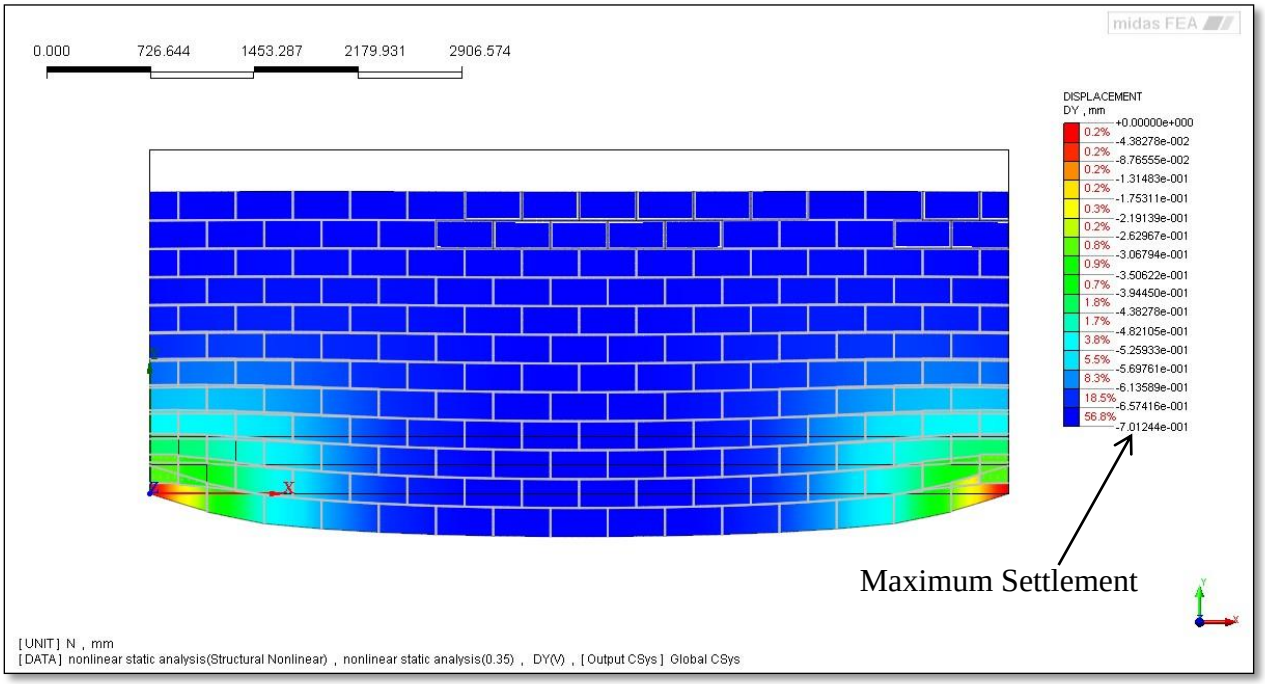
e) Deformation of boundary wall @25% self weight



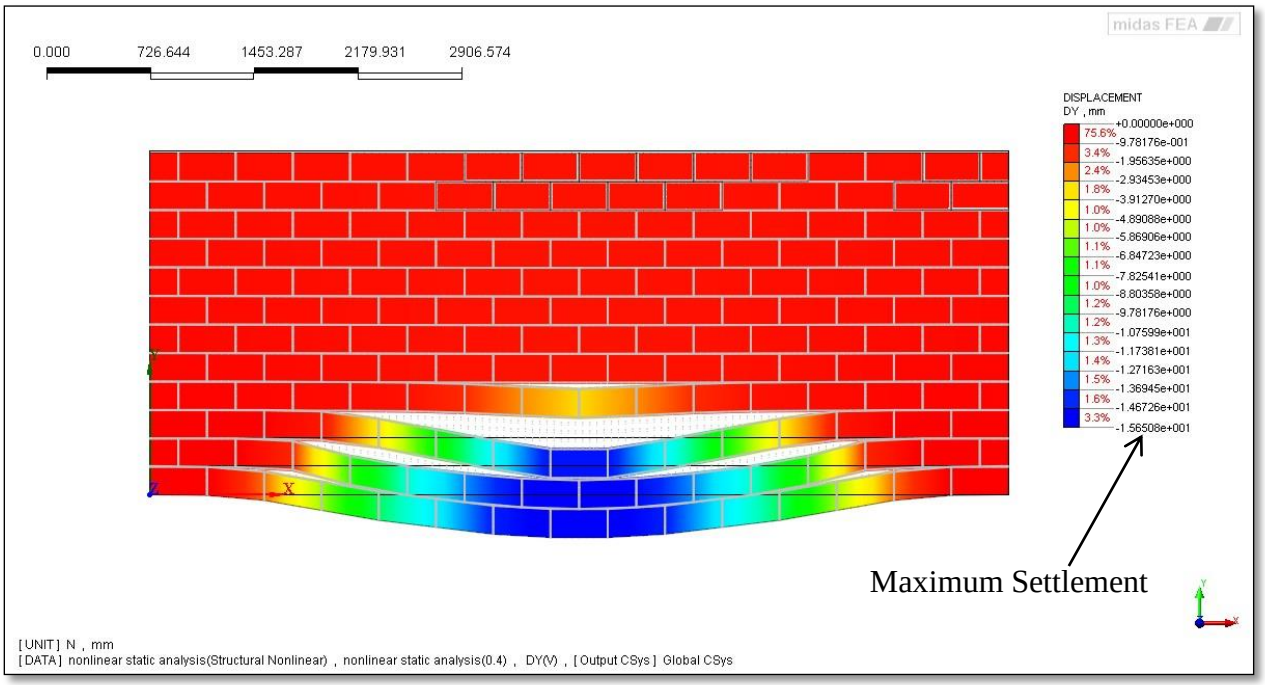
f) Deformation of boundary wall @30% self weight



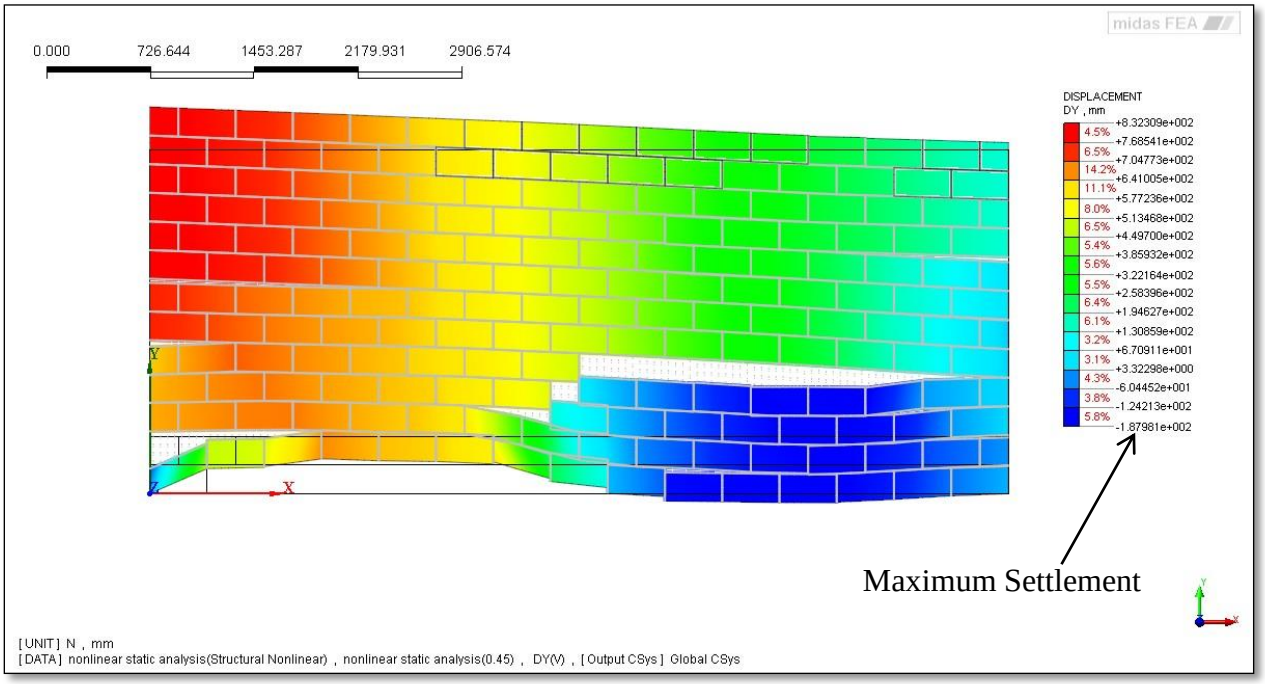
g) Deformation of boundary wall @35% self weight



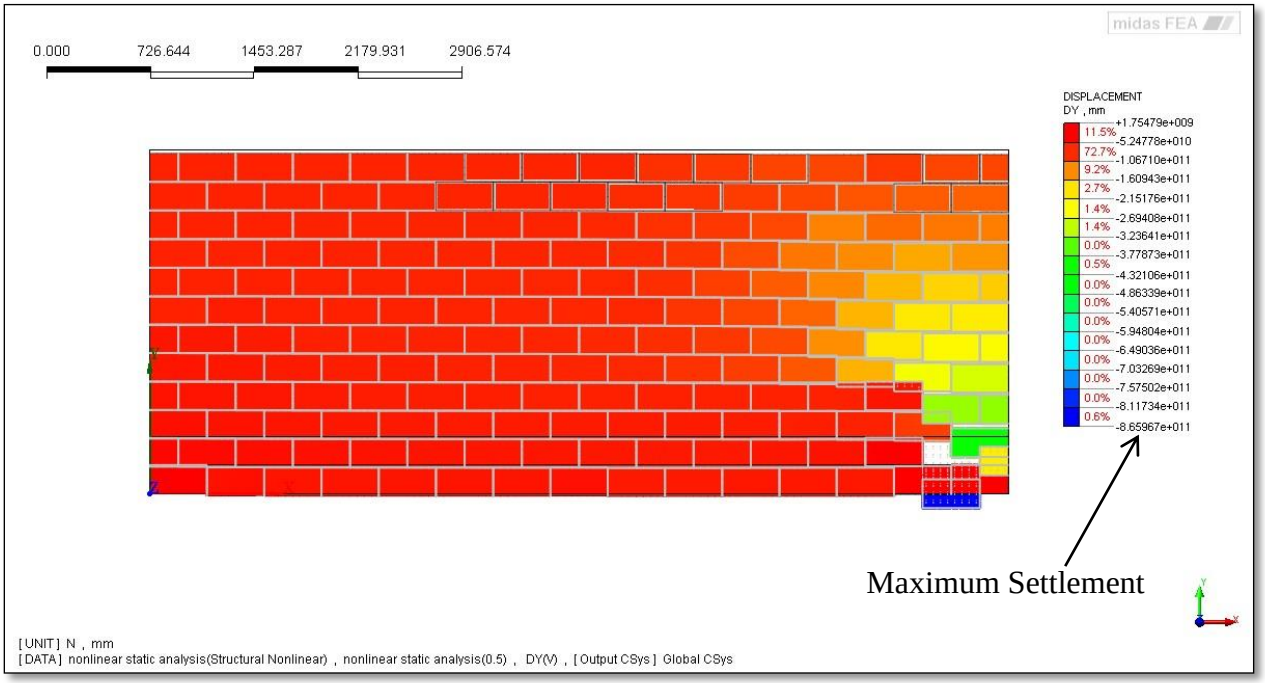
h) Deformation of boundary wall @40% self weight (sumulated failure is similar to deformation pattern noticed in the field)



i) Deformation of boundary wall @45% self weight (unrealistic deformations noticed)



j) Deformation of boundary wall @50% self weight (unrealistic deformations noticed)



APPENDIX-C

- Conference publications
- Journal publications
- Magazine publication
- Award
- Fundings

CONFERENCE PUBLICATIONS

1. **R Vandanapu**, J. R. Omer, and M. F. Attom (2016) "*A Review on Testing of Collapsible Soils*". In Proceedings of Second International Conference on Infrastructure Management, Assessment and Rehabilitation Techniques (ICIMART '16), Sharjah, United Arab Emirates, pp. 82-87.

2. **R Vandanapu**, J. R. Omer, and M. F. Attom (2017) "*Laboratory study of the effects of surface irrigation on the settlement of a collapsible stratum beneath a lightly loaded structure* ". In Proceedings of 6th International Conference on Civil, Architectural and Environmental Sciences (CAES-17), Dubai (UAE), March 13-14, 2017. (Published as poster)

A Review on Testing of Collapsible Soils

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ABSTRACT

Soils are highly heterogeneous in nature irrespective of the origin or location hence the usual uncertainty in characterising soils to assess parameters for geotechnical design.For this reason it is important to understand as far as possible how each soil type behaves and responds to stresses, deformation and pore water effects. In addition, there are other various phenomena e.g. weather, earthquakes, human activities that can affect geotechnical structures after construction. This implies that in foundation engineering, for example, satisfaction of bearing capacity and settlement requirements alone may not be sufficient criteria to maximise the probability of survival of a structure under complex and changing in-service conditions of the structure. In particular, collapsible soils, which occur mainly in arid and semi-arid regions, may be capable of resisting fairly large loads in the dry condition but such soils often exhibit instability and strength loss when in contact with water. A number of researches havebeen carried out in order to understand and quantify the behaviour of collapsible soils, based on laboratory experiments and field testing. In this paper, an opportunity is taken to catalogue and publish findings by different investigators as to the basic characteristics of collapsible soils and how their behaviour may affect geotechnical structures.

KEYWORDS: Collapsible soils,laboratory tests, field tests, deformation, settlement.

1 INTRODUCTION

Collapsible soils are found in many parts of the world such as USA, Central and South America, China, Africa, Russia, India and the Middle East (Murthy, 2010). These soils cause problems to geotechnical engineers who have to deal with analysis and design issues for substructures in arid and semi-arid regions. In dry conditions, collapsible soils may be competent in bearing load but are prone to instability and structural breakdown when in contact with water. This can be severely detrimental to the structures built on such soils. This undesirable behaviour is primarily due to loss of the friction component of shear strength upon ingress of water. Generally, collapsible soils undergo abrupt changes in volume when their moisture content increases, with or without loading, and this is markedly significant when the degree of saturation is above 50%. However, full saturation is not necessarily required for the soil to exhibit collapse behaviour (Abbeche et al., 2010). Water ingress, by whatever means, into collapsible soil strata causes the groundwater table to rise. In developed arid/semi-arid sites, water from unnoticed leakages in underground pipelines, irrigation operations and industrial activities can reach collapsible strata at depth.

It should be understand that the term "collapsible soils" does not mean a particular soil type but rather a whole variety of soils that are susceptible to structural collapse and examples include wind-blown sand, loess or alluvial soil types (Kalantari, 2012).

Other than effects of water, another cause of soil structure collapse is reduction in the strength of the bonding between soil particles, e.g. in loosely cemented sands where the cementing material is liable

to softening and weakening by water. The collapse occurs at any stress level greater than that at which the soil has been previously wetted. Therefore, collapse under low stress level is majorly due to overburden pressure alone. Although Houston et al. (2002) suggested that full saturation is required for the full collapse of a soil at any given stress level, it should be noted that partial wetting will only result in partial collapse of the soil (Houston et al., 1993).
Even though it is difficult to predict the behaviour of soils that exhibit collapse under unexpected or undesirable water ingress, many researchers have undertaken laboratory and field tests in an attempt to identify certain characteristics of such soils. Examples of related studies done by previous researchers are summarised in the following sections.

2 LABORATORY TESTS

Holtz and Hilf (1961) suggested that loess-like soils that have a void ratio large enough to exceed its moisture content beyond its liquid limit upon saturation are vulnerable to collapse. A graph (Figure.1) has been developed to help in identifying whether a soil exhibits collapse behaviour or not. The graph requires knowledge of just two basic properties: dry density and liquid limit. Once determined, if the soil falls on/below the line, it shows that that soil is collapsibleif there is ingress of water.

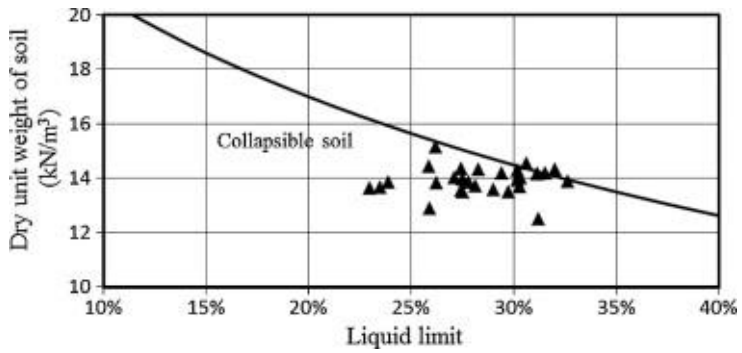


Figure 1: Dry unit weight of soil versus liquid limit

Gaaver(2012) conducted various laboratory experiments on the behaviour of soils in the western Egyptian desert region of Borg-El-Arab, ground settlement was observed to have caused damage to various structures, leading to expensive repairs. Gaaver(2012) conducted tests on various disturbed and undisturbed soils to identify the nature of the soils and possible methods of ground improvement. The test results were plotted in as a curve similar to the one shown in Fig. 1, where the region below the curve defines collapsible soils, hence as seen the plotted points (from data reported by Gaaver, 2012) strongly indicate collapse behaviour. The predictive accuracy of Fig. 1 plot was verified by Rezaei et al. (2012) who successfully used it to identifythe collapse behaviour of a soilat the site of a project named South Rudasht Irrigation Network Channel, Iran.Rezaei et al. (2012)also carried out direct shear tests on soilswith a view to predict the collapsibility of soils.The tests were done under an overburden pressureequivalent to 1.50m of the soil in question. This height was selected because most structures in the Iranian region are founded at such depth. All tests were conducted in soaked and un-soaked conditions for undisturbed and compacted soil collected from different sites. From the analysis of the results, a new term called '*reduction factor in shearing resistance* (RFSR)' was introduced and defined as the ratio of shearing resistance of soil in the soaked condition to that in the un-soaked condition. The quantity RFSR was found to be a useful parameter representing the decrease in the bearing capacity of the soil at foundation level due to soaking process. For undisturbed samples, the initial moisture contentswere adopted as the in-situ values. In contrast, for the compacted samplesthe initial moisture contents were taken as the values prior to the shear tests. The RFSR was found toincrease with increase in initial moisture content, for both undisturbed and compacted soils. For collapsible soils in the natural state, the RFSR was found to fall in the range 0.43-0.58 (see Fig. 2), with an average of 0.50. This means that the bearing capacity of the natural collapsible soil may be decreased to about 50%, and so the imperative recommendation is to double the factor of safety when designing in foundations on collapsible soils. One can use this approach in terms of assigning the

safety factor while analysing bearing capacity of such soil types. Accordingly, simple relationships between RFSR and initial moisture content (w_c) were developed as shown in Equations. (1) & (2) which enable engineers to estimate the reduction in bearing capacity.Tests for collapse potential (C_p)were carried out using the procedure proposed by Jennings and Knight (1975) and typical results obtained were as shown in Figure. 3. From the results, equations (1) and (2) were formulated to relate RFSR to initial moisture content, for undisturbed and compacted soils.

For undisturbed samples; $RFSR = 1.53(w_c) + 0.34$ (1)

For 95% compacted samples; $RFSR = 2.16\ (w_c) + 0.40$ (2)

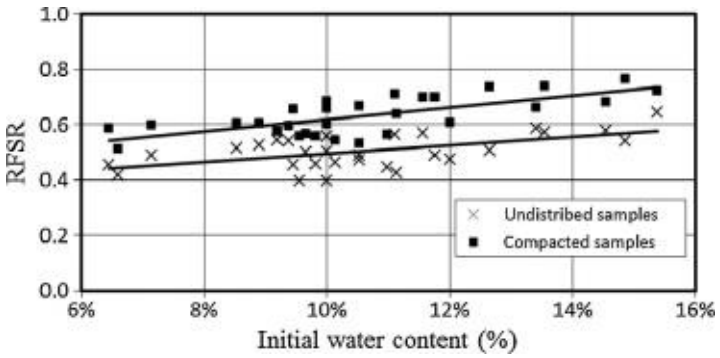


Figure 2: Reduction factor in shear strength at a depth of 1.50 mbelow ground level versus initial water content

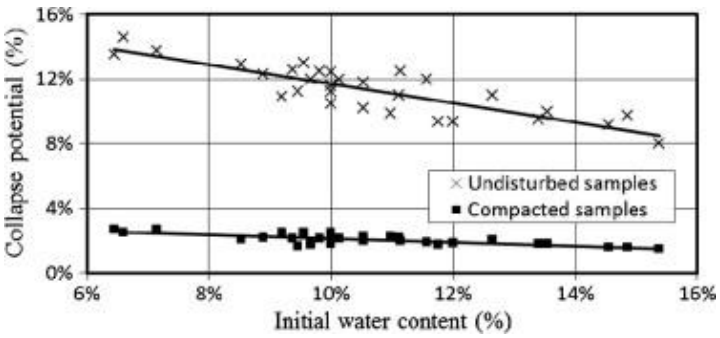


Figure 3: Collapse potential versus initial water content

From the observed variation trends in Fig. 3 the following relationships shown in equations (3) and (4) can be extracted to express collapse potential in terms of the initial moisture content, for undisturbed and compacted soils:

For undisturbed samples; $C_p=0.177\text{-}0.59\ (w_c)$ (3)

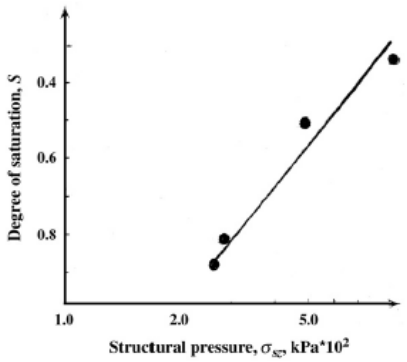
For 95% compacted samples; $C_p=0.033\text{-}0.11(w_c)$ (4)

Anderson et al. (1995) conducted constant-shear-drained (CSD) tests using tri-axial apparatus for two different soils: (i) a uniformly graded sand and (ii) an undisturbed clayey alluvial soil. It was shown from the tests that the collapse potential is related to the stress path andit also emerged that that knowledge of stress path is necessary to accurately predict the collapse potential of such a soil.
Reznik (2007) developed equations to estimate the structural pressure (σ_{sz}) as a function of the degree of saturation (S) using oedometer test results reported from various researchers. Reznik (2007) suggested that collapse of soil starts when the applied stress exceeds soil structural pressure values and postulated that the collapsibility of soil is a non-elastic deformation.In addition, Reznik (2007) introduced a new parameter "structural pressure value", defined as separation 'points' between elastic and plastic states of any soil (including collapsible soils) under loading. Figure. 4 illustrate graphs constructed by Reznik (2007) plotting degree of saturation versus structural pressure (at various stress levels) at which the soil changes from elastic to plastic state in oedometer tests. The work led to

development of a general relationship as shown in Equation. (5), based on logarithmic regression analysis.

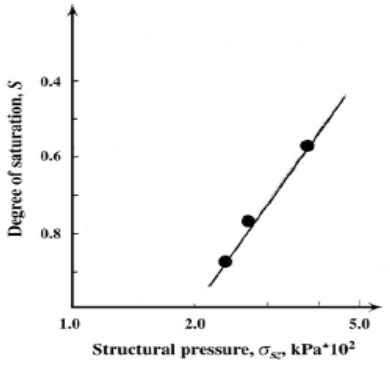
$$\sigma_{sz}(S) = C_0 + 10^{-CS+D} \quad (5)$$

where, C_0, C and D are coefficients



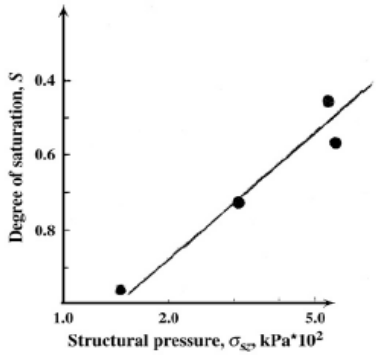
Results of Kane (1969) on Oakdale loess

$\sigma_{sz}=10^{-0.8711S+1.1990}$ (kN/m^2*10^2)



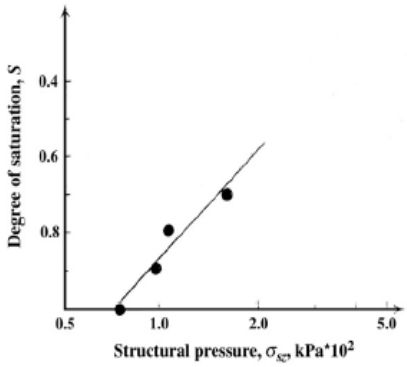
Results of Kane (1969) on Hawkeye loess

$\sigma_{sz}=10^{-0.6269S+0.9197}$ (kN/m^2*10^2)



Results of Kane (1973) on Oakdale Loess

$\sigma_{sz}=10^{-1.1547S+1.3030}$ (kN/m^2*10^2)



Results of Jasmer and Ore (1987) on Pocatello loess

$\sigma_{sz}=10^{-1.013S+0.8845}$ (kN/m^2*10^2)

Figure 4: Degree of saturation versus structural pressure obtained from oedometer tests on various soils

Reznik (2007) suggested that the in-situ void ratio and natural moisture could be determined using geophysical methods. By combining such data with oedometer test results, it is possible to develop correlations similar to Equation. (5) for predicting the structural pressure.

3 FIELD TESTS

Houston et al.(1995) developed an in-situ test named 'downhole collapse test' (Figure.5)and conducted a series of tests at a site known to exhibit wetting induced collapse. The test was conducted in a borehole with load being applied to the plate at the bottom of the borehole. Water was introduced in the test and load-settlement response of the soil monitored. The data from of all tests and equations developed thereof were used to determine the wetting induced collapse. In addition, a full-sale load test was conducted on a footing of size 0.81 x 0.81m embedded 0.46m below the ground surface. Over a 10 hour period, 400 gallons of water was introduced by surface ponding. At the end of wetting process, the final settlement of footing was measured to be 39.5 mm, which compares well with the settlement of 36.6 mm obtained from equations developed from down-hole collapse test results. It was also emphasized thatwhile performing tests on collapsible soils, lab specimens could be subjected to higher degree of saturation than field soils, samples could be disturbed and soils such as gravels could be difficult to sample.

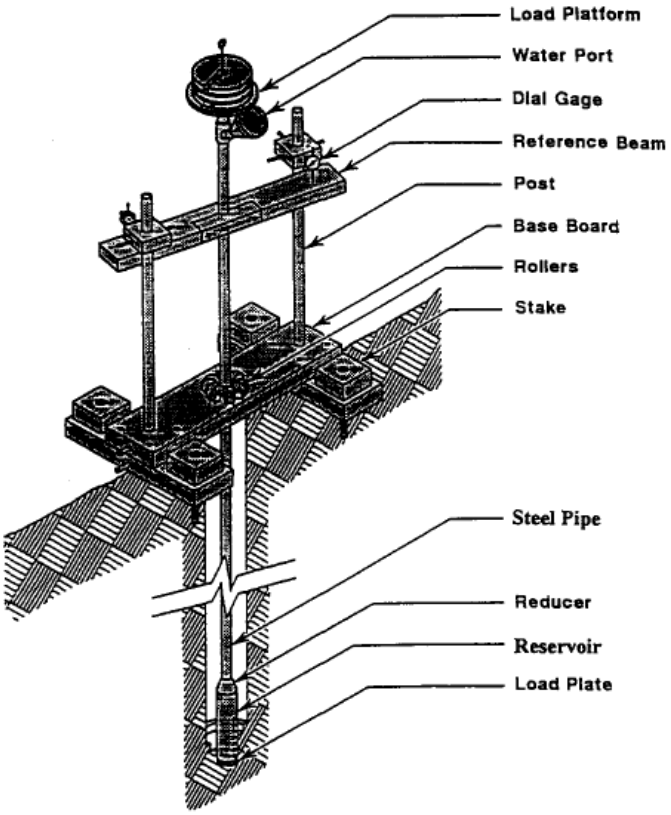


Figure 5: Downhole collapse test system

Reznik (1993) conducted static field plate load tests on collapsible soils using rigid bearing plates (of area 0.50 m^2) at the centre of the bottom of rectangular pits of size 1.8 m x 1.5 m. The test loads were applied using hydraulic jacks and settlements were recorded at maintained loads until the settlement rate decreased to 0.05 mm per hour. Subsequently the next load increment was applied. Water conditions developed during plate load tests are similar to the ones observed under constructed structures nearby (south-western part of Ukraine) where fast increase of settlements is caused by uncontrolled wetting of soils. A parameter called the proportionality limit (P_{pr}) was defined to represent the maximum pressure corresponding to the linear part of the curve obtained from plate load test. Values of the P_{pr}obtained for collapsible soils were found to decrease when the water content increased. However, theoretically the minimum value of P_{pr}should occur when the soil saturation degree reaches 100%. It is useful to note that such situations will happen only if saturation due to undesirable sources of water is not eliminated immediately. According to the author's experience, the degree of saturation of soils under structures due to accidental wetting rarely exceed 70-80% and therefore the aforementioned technique of load testing collapsible soils is considered acceptable for design purposes. This is because the degree of saturation calculated after conducting the plate load test including wetting was found to be always below 80%.Table. 1 lists a number of parameters collated from the work of Reznik (1993).

Table1: Properties of tested soils

Test	Moisture content after the test (%)	Degree of saturation after the test (%)	Proportionality Limit (kN/m^2)
1	14.5	45	265
2	28.1	70	110
3	16.1	44	170
4	34.0	70	100

4 CONCLUSION

An effort has been made in this paper to give a thorough insight on key laboratory and field experiments so far conducted to aid understanding of the behaviour of collapsible soils. As geotechnical engineering primarily deals with naturally occurring soils which are inherently heterogeneous in nature, it is difficult to replicate the real soils in the laboratory. Therefore the emphasis is on how to test the soils in as near natural conditions as possible. Field tests may yield results that are more representative of the real soil that laboratory tests but they are considerably more expensive and so the importance of empirical correlation in geotechnical analysis cannot be overstated. This paper has presented a summary of some simple yet valuable correlations that can be used to assess collapsible soils and explain the governing mechanisms, for typical soils prone to structural instability in the wet condition. The present review of existing publications related to collapsible soils will form part of the background research in an on-going doctoral project aimed at assessing the structural distress caused by collapsible soils in the United Arab Emirates (UAE). The long term aim is to develop a methodology for profiling collapsible soils and predicting their effects on structures and how those effects can be ameliorated using ground improvement methods.

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Laboratory Study of the Effects of Surface Irrigation on the Settlement of a Collapsible Stratum beneath a Lightly Loaded Structure

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INTRODUCTION

Collapsible soils are generally found in arid and semi-arid regions like UAE. These wind-deposited desert soils are more susceptible to ground water fluctuations, thereby making it especially uncertain to attempt to predict the bearing capacity and settlement using conventional methods. Collapsible soils may be capable of sustaining large bearing pressures when in the dry state, but suffer significant strength loss when in contact with water. In this research, case studies involving structural deformation of boundary walls, road pavements and footpaths caused by settlement of collapsible soils due to water infiltration from irrigation and landscaping activities at various sites in the United Arab Emirates (UAE) were examined. A laboratory simulation test was devised where samples of the collapsible soil were tested for deformation characteristics at specific total stresses and water infiltration rates. Finally, mathematical relationships were formulated for estimating the length of irrigation period necessary to initiate settlement and the magnitude of that settlement, for given thickness of the collapsible stratum and surcharge loading.

KEY OBJECTIVES

1. To develop a deeper understanding of the behavioral characteristics of collapsible soil by conducting laboratory plate load tests in soil tank through simulating the effects of water infiltration due to irrigation of landscapes.
2. To understand underlying mechanisms and formulate predictive equations for rates of settlement of collapsible soil, as functions of several variables mentioned below
   (i) thickness of collapsible layer
   (ii) its depth from ground level
   (iii) groundwater regime

EXPERIMENTAL STUDY

METHODOLOGY

The whole work is divided into two phases

Phase-1

Understand the influence of variable depths (simulating the actual groundwater table) of water in the tank on time and magnitude of settlement of collapsible soil.

Phase-2

Identify the influence of variable thickness of collapsible soil layer sandwiched between non-collapsible soil layers on time and magnitude of settlement

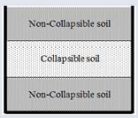
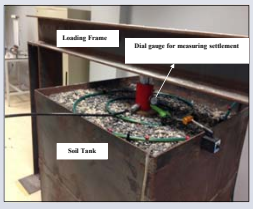
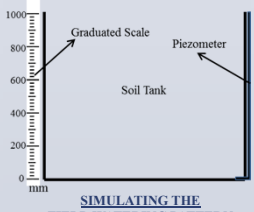


PLATE LOAD TEST SET-UP

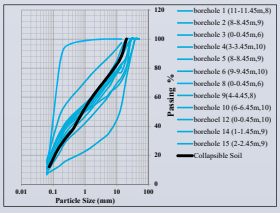


Cubic tank - 1m x 1m x 1m, Thickness of mild steel sheet - 4 mm
Loading frame : Steel beam of 250mm width and 250 mm depth @ 72.4 kg/m

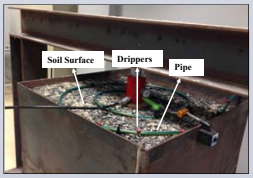
REPLICATION OF GROUNDWATER TABLE



PREPARATION OF COLLAPSIBLE SOIL



SIMULATING THE FIELD WATERING PATTERN



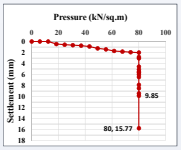
Twice a day for 30min @ 6.5l/min

MAINTAINING CONSTANT PRESSURE

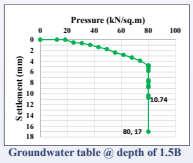
Pressure = 80 kN/m²

EXPERIMENTAL RESULTS – Phase-1

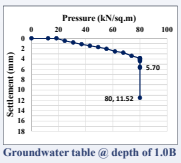
Effect of dripping water on settlement of soil



Groundwater table @ depth of 2.5B

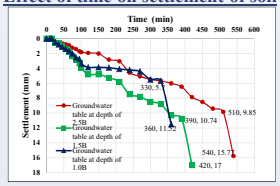


Groundwater table @ depth of 1.5B



Groundwater table @ depth of 1.0B

Effect of time on settlement of soil



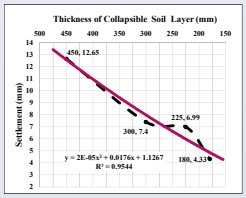
Test	Depth of water level below bottom of plate	Number of wetting cycles	Settlement of soil before the start of collapse (mm)	Settlement at the end of test (mm)	Collapse Settlement (mm)
1	2.5 B (500 mm)	7	9.85	15.77	5.92
2	1.5 B (300 mm)	5	10.74	17.00	6.26
3	1.0 B (200 mm)	4	5.70	11.52	5.82

B = Diameter of plate = 200mm

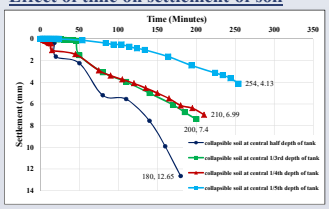
EXPERIMENTAL RESULTS – Phase-2

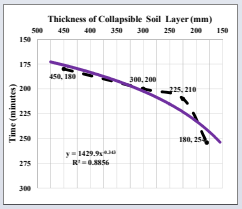
Effect of dripping water on settlement of soil

- Collapsible soil inserted at mid depth of the soil stratum below the plate.
- Four plate load tests were conducted with thickness of collapsible soil as ½, 1/3rd , ¼th, 1/5th of the total soil depth in the tank.



Effect of time on settlement of soil





CONCLUSIONS

1. The number of wetting cycles required for the soil to exhibit the collapse increases with increase in depth of groundwater table below the foundation level.
2. Time required for the soil to exhibit the collapse increases with increases in depth of groundwater table below the foundation.
3. Once the soil starts exhibiting its collapsible behavior, the rate at which it collapses was found to be uniform irrespective of its thickness.
4. The magnitude of settlement increases with increased proportion of collapsible soil in a soil strata.
5. The time required for soil to exhibit the ultimate settlement decreases with increase in thickness of collapsible soil in a soil strata.
6. The higher the thickness of collapsible soil, the lower will be the time required to collapse and settle though the magnitude of settlements are high.
7. Relationships developed between the time of settlement and thickness of collapsible soil as well as magnitude of settlement and thickness of collapsible soil can be used by practicing engineers to adopt in terms of predicting the time or magnitude of settlements depending on the thickness of the collapsible soil encountered during the geotechnical investigations.

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Laboratory Study of the Effects of Surface Irrigation on the Settlement of a Collapsible Stratum beneath a Lightly Loaded Structure

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Geotechnical case studies: emphasis on collapsible soil cases

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Direct exposure of soil to certain atmospheric agents, such as water, can influence adversely or favourably the engineering behaviour of the soil. For instance, saturated and unsaturated/partially saturated soils behave differently, so do soils under seepage and hydrostatic pressures. Many theories in soil mechanics idealise soils as either cohesive or non-cohesive, and this has allowed much research to be done on saturated cohesive soils. However, non-cohesive soils have not received as much attention, apart from recent strength and dilatancy theories, yet in some parts of the world, certain non-cohesive soils pose significant risk to structures built on them. The most problematic examples of such soils are collapsible soils that may not be detected and properly considered in routine ground investigation activities. In this paper some case studies of collapsible soils in the United Arab Emirates are examined to analyse the effect of their collapse on infrastructure and the possible techniques to ameliorate the situation. The case studies include various sites that were found to suffer structural damage traceable to collapsible soils. It is found that in most cases the soil collapse was due to infiltration of rainwater or water from sustained irrigation activities at the surface.

1. Introduction

Civil engineers build different types of infrastructure on various soil types in different parts of the world. The range of infrastructure includes light and heavy overground structures, subsurface installations, slender but tall buildings structures and many more. The structures are supported on variable soils that include broadly both residual and transported soils. Residual soils are those that were formed due to weathering of rocks and have remained at their original locations, whereas transported soils are deposited away from their place of origin (Rezaei *et al*., 2012). Transportation of soils is caused by movement due to gravity, wind, water, glacier or human activities. Usually the properties of transported soils are influenced by the mechanisms of transportation and deposition (McCarthy, 2006). Although many soil types are competent as load-bearing media, some soils exhibit swelling, dispersing and collapsible characteristics due to change in water content, which often presents a variety of challenges to engineers (Rezaei *et al*., 2012). Such soils may require special attention and treatment when being considered for use as foundation materials for important structures. This paper primarily emphasises collapsible soil cases, and such soils, usually sand, consist primarily of silt-sized particles (Kalantari, 2012) and possess characteristics such as being naturally quite dry, having an open structure and a high porosity (Noutash *et al*., 2010). The main drawback of these soils seen in the current case studies is that when standard penetration tests (SPTs) are carried out in boreholes, the soils exhibit *N*-values in the medium dense range (N = 4–10) as observed from geotechnical reports, where collapsible soils are attributed finally as the cause of the distresses experienced. The penetration resistances observed are majorly due to the intergranular friction between the particles, when they are dry. However, when these soils become wet for any reason, and coupled with loading, they exhibit collapse in their structure, leading to a reduction in volume (Jotisankasa, 2005), causing settlements to structures built or being built on them. Identification of the collapsibility of soil has been emphasised by many researchers in the past through laboratory tests (Anderson and Riemer, 1995; Gaaver, 2012; Holtz and Hilf, 1961; Jasmer and Ore, 1987; Jennings and Knight, 1975; Kalantari, 2012;

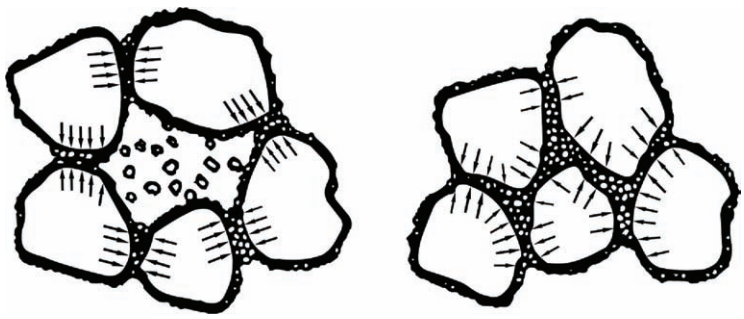
Rezaei *et al*., 2012; Reznik, 2007) and field test tests (Houston *et al*., 1995; Reznik, 1993). Field tests are undoubtedly expensive in ground investigations, and most of these laboratory procedures involve performing tests on undisturbed soil samples through direct shear tests and oedometers, which are very difficult to sample, particularly those of the cohesionless soils in the case studies depicted in this paper. The procedure proposed by Holt and Hilf (1961), which was later verified by Gaaver (2012) and Rezaei *et al*. (2012), is the simplest of all the procedures, and it involves determining the dry density and the liquid limit. As soils in the United Arab Emirates (UAE) are mostly of a dry and cohesionless type, a cone penetrometer can be used as an alternative to the Casagrande apparatus for determining the liquid limit. However, accurately determining the dry density remains questionable, as it is very difficult to retrieve an undisturbed sample in such soils, and the simplest way is to use standard correlations between SPT *N*-values and dry densities. But SPT tests are generally carried out before the actual construction of a project starts, and the characteristics of soils will be changed with the ingress of water into the ground due to continuous irrigation of landscapes, unnoticed leakage of water lines or sewage lines and so on. Also, ingress of water mostly due to irrigation of landscapes was found to be the cause of the distresses observed in the case studies described. Thus, it was understood that further research is required to be carried out in this context, and the long-term aim is to develop a methodology for profiling collapsible soils and predicting their effects on structures and how those effects can be ameliorated using ground improvement methods. This paper examines the behaviour of certain collapsible soils in the UAE, how they cause distresses to structures and the possible solutions that engineers can implement to ameliorate the structural distress problem.

2. Collapsible soils

Collapsible soils are found in many regions of the world including parts of the USA, China, Africa, Russia, Central and South America, India and the Middle East (Murthy, 2010). These are loess-type soils (Kalantari, 2012) and are generally unsaturated in state as found naturally (Zhu and Chen, 2009). Examples of such soils are windblown sand, loess or alluvial deposits found generally in arid or semi-arid environments where the evaporation of soil moisture is so high that the deposits do not have sufficient time to consolidate under their own weight (Pye and Tsoar, 1990). They are moisture-sensitive soils in that moisture increase causes them to undergo sudden volume reduction and settlement (Figure 1), particularly under the load of a structure (Bell, 2000). These soils generally possess porous textures with high void ratios and low relative densities (Rezaei *et al*., 2012).

As recognised by many researchers (Graham and Li, 1985; Holtz *et al*., 1986; Leroueil and Vaughan, 1990; Schmertmann, 1955; Wesley, 1990), the structure of a soil significantly affects its mechanical properties. Collapsible soils and fills are susceptible to abrupt increase in density due to increase in moisture content or temperature, or as a result of the dissolution of compounds that bond loosely arranged soil particles (Dudley, 1970; Petrukhin, 1989; Reginatto and Ferrero, 1973). In the natural state of collapsible soils, their void ratios are so large as to hold moisture equivalent to the liquid limit value. In the dry state, such soils may offer sufficient resistance to structural loads, but suffer large reductions in void ratio due to wetting and rearrangement of particles (Jotisankasa, 2005). Additionally, these soil types can show rapid collapse response to saturation (Bolzon, 2010).

Efforts have been made by various workers (Anderson and Riemer, 1995; Gaaver, 2012; Holtz and Hilf, 1961; Jasmer and Ore, 1987; Jennings and Knight, 1975; Kalantari, 2012; Rezaei *et al*., 2012; Reznik, 2007) to characterise collapsible soils based on laboratory testing. As stated earlier, Holtz and Hilf (1961) suggested that loess-like soils that have a void ratio large enough to exceed their moisture content beyond their liquid limit upon saturation are vulnerable to collapse. A graph (Figure 2) has been developed to help in identifying whether a soil exhibits collapse behaviour or not. The graph requires knowledge of just two basic properties: dry



Dry soil with honeycombed structure before inundation

(a)

Soil structure after inundation

(b)

Figure 1. Loaded collapsible soil (a) before and (b) after inundation with water

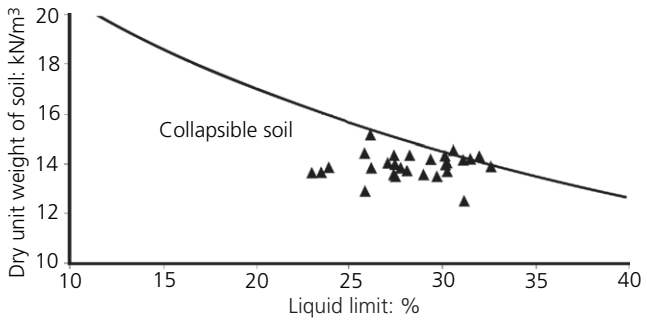


Figure 2. Dry unit weight of soil against liquid limit

density and liquid limit. Once determined, if the values for the soil fall on/below the line, this shows that the soil is collapsible if there is ingress of water. More recently, Houston *et al*. (1993) and Das (2009) also suggested that collapsibility can be evaluated by determining the dry density and liquid limit. Jasmer and Ore (1987) proposed an approach for identifying the collapsibility of soils through the use of direct shear tests on undisturbed and compacted soils. Anderson and Riemer (1995) conducted constant-shear-drained tests by using triaxial methods and concluded that knowledge of the stress path is essential to predict accurately the collapse potential of such soils. Reznik (2007) conducted a series of oedometer tests and reported that soil collapse starts when the applied stress exceeds the structural pressure level of the soil, 'structural pressure' being defined as pressure corresponding to the separation 'point' between the elastic and plastic states of any soil (including collapsible soils) under loading. Reznik (2007) suggested that on-site void ratio and natural moisture content could be determined using geophysical methods and such data combined with oedometer test results could be used for predicting the magnitudes of structural pressures in collapsible soils.

As stated earlier, some researchers (Houston *et al*., 1995; Reznik, 1993) have conducted field tests to help characterise collapsible soils. Reznik (1993) conducted field plate loading tests on collapsible soils and reported the tests to be useful for identifying the collapsibility of soils. Houston *et al*. (1995) developed an on-site test known as the 'downhole collapse test', which they utilised on sites of soils known to collapse due to wetting. The results of Houston *et al*.'s (1995) work were compared with actual settlements and found to be reasonably consistent.

Although several case studies have been reported earlier by many researchers, only a few of them are mentioned in the following.

(*a*) In semi-arid New Mexico, USA, a commercial building won an award for the year's most beautiful lawn and landscaping. However, it suffered foundation damage owing to differential settlement due to the wetting of collapsible foundation soils underneath (Houston *et al*., 2001).
(*b*) Noutash *et al*. (2010) reported that the impounding of the Khoda Afarin canal, located in northern Iran, to mitigate existing collapse potential in the area had caused large cracks on both sides of the canal's berms after the pretreatment technique had been completed.
(*c*) Kalantari (2012) reported a forensic investigation in San Diego, California, USA, where the annual precipitation had been about 30 cm before a residential subdivision was built and, including landscape irrigation, had increased to about 170 cm after it was built. Such an increased level of precipitation had resulted in substantial settlements of the underlying compacted fill. In addition, the lawns were spongy to walk on and the street side kerbs had moss growing on them as a result of heavy landscape watering.

In all three cases, the cause of the collapse of soil is the ingress of water, either purposely or unintentionally. Similar kinds of cases were noticed in the UAE, where continual irrigation of landscapes had led to distresses in neighbouring infrastructure such as boundary walls and pavements; these cases are elucidated in the next section.

3. Case studies

In this section, two case studies at locations in the UAE are presented, whereby collapsible soils were suspected to have caused structural distress to lightly loaded structures such as boundary walls, pavements, footpaths and landscapes. In the case studies, professional geotechnical companies were commissioned to investigate how the problem occurred, quantify the level of distress and propose methods of reducing the undesirable impacts. In both case studies, it was revealed that the collapse of underlying soils was the cause of distresses experienced by the structures. For data confidentiality reasons, the precise project locations and names of the investigation companies or their clients are not disclosed in this paper to comply with the conditions under which the data were made available for this research.

3.1 The guest house project

The project was located in Al Ain City in the UAE. The site had been developed with a guest house with landscaped gardens and terraces covering 85% of the site. This equates to more than 15 000 m^2 of lawn, and the garden area is formed on a 12 m thick fill of topsoil. The fill area is bounded by a two-step precast gravity-retaining wall structure, which deformed due to uneven settlement of the ground beneath. As deduced later, the settlements were linked to the effect of irrigation water on collapsible soils existing at some depth in the area. Fortunately, the actual guest house structure did not experience any distresses, as it was supported on pile foundations. When settlements were initially observed, it was decided to carry out remedial works in an effort to keep the structures serviceable. However, settlements continued even after the repair works were completed. No settlements were observed during placement of the fill and the associated landscaping work features prior to irrigation. However, as soon as irrigation activities started, within 8–10 months, very clear signs of surface settlements and associated distresses were

seen. Although distresses were observed on site at several locations, only a few of them are highlighted in the following.

(*a*) Kerbstones adjacent to landscaped areas were separated from the walkways by approximately 40 mm.
(*b*) The steps which are in close proximity to landscaped areas of the guest house structure experienced subsidence, whereas the actual structure (founded on piles) did not (Figure 3).
(*c*) Large settlements (approximately 80 mm) were observed in areas paved with concrete slabs, which are in close proximity with the landscaping areas.

The magnitude of settlements observed on site was measured to be in the range of 2–3 cm on the low side and 9–10 cm on the high side. Consequently, site investigations were commissioned to evaluate and explain the causes of the distresses (settlements) observed in the soft and hard landscaped features around the guest house structure. Ten boreholes 15 m deep and two others 20 m deep were drilled along with four excavation test pits, each 2 m deep. Additionally, the following field tests were carried out: (*a*) SPTs, (*b*) permeability tests, (*c*) Mackintosh probe tests and (*d*) soakaway tests. The general stratigraphy of the site and the observed SPT blow counts are given in Table 1. The mean permeability of the soil obtained from field permeability tests was found to be of the order of $6{\cdot}83 \times 10^{-7}$ m/s and is typical of soils with high silt content. Bell (2000) provided an indication of the potential severity of the collapse (Table 2). Collapse potential tests carried out on soil samples from the test pits are shown in Table 3, and the values indicate that the soils are susceptible to collapse and the severity of the problem is categorised as 'very severe trouble' (Bell, 2000).

Figure 3. Separation of stairs from adjacent wall due to differential settlement

Considering the various structural distresses observed at the site and given the vast area of ground to be improved, it was thought that grouting would not be an economic option. Thus, hydrocompaction was recommended as a preferable and inexpensive option. To avoid further distresses due to settlement of soil while hydrocompaction was in progress, it was also recommended to use hydraulic jacks to lift the existing gazebos and swimming pool structures at the site. Upon completion of hydrocompaction and cessation of ground settlements, cement grout would be injected along any resulting gaps, to ensure that the bases of the gazebos and swimming pool structures make a complete contact with the ground.

3.2 An infrastructure project

This low-rise housing development in Abu Dhabi (UAE) consists of villas, amenity buildings, community buildings and open green spaces. A network of sector roads traverses the area and connects to the surrounding highway system. Upon completion of construction and during the first year of occupation and service, evidence of distress (due to excessive settlements) began to appear in certain areas of the development. Buildings including villas and other communal or amenity buildings show absolutely

Collapse: %	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
Over 20	Very severe trouble

Table 2. Collapse percentage as an indication of potential severity

Depth: m	Description of soil	Range of SPT *N*-values	Relative density (based on SPT)
0·0–1·0	Silty sand (agricultural soil as fill material)	3–19	Very loose to medium dense
1·0–13·0	Silty gravel/gravelly silt (fill material)	3–30	Very loose to medium dense
13·0–15·0	Silty sand (dune sand)	32–50	Dense to very dense
15·0–19·0	Silt (alluvial soil)	37–50	Dense to very dense
19·0–20·0	Silty gravel (residual soil)	>50	Very dense

Table 1. General stratigraphy of the guest house site

Test pit number	Depth: m	Collapse potential: %
2	0·50	64·5
3	1·20	86·4
4	1·85	86·7

Table 3. Collapse potential test results

no signs of distress since they rest on rigid pile foundation systems. The affected areas were mainly in shallow founded structures/features such as boundary walls, hard landscapes, soft landscapes and internal roads. Although many distresses were noticed on site, only a few are mentioned in the following.

(*a*) Footpaths at locations adjacent to landscaped areas experienced settlements (approximately 75 mm) under the effect of continuous water ingress.
(*b*) Although several boundary walls had distressed on site, those walls which are located with landscaping on either side had suffered the highest level of distress, with settlements approximately 260 mm (Figure 4).
(*c*) Flexible pavements, particularly those adjacent to open landscaped areas, had experienced distress (settled approximately 100 mm) as well. As stress transfer under flexible pavements is limited largely to 2·0–2·5 m below ground, it was thought initially that very loose to loose soils that are susceptible to collapse due to movement of water were present at shallow depths. This was later confirmed from the low SPT blow counts observed at very shallow depths (1·0–1·5 m) in the drilled boreholes.
(*d*) Interestingly, it was found that the ground in some green landscaped areas with no structures also subsided (approximately 150 mm). Hence, it was suspected that the ground movements could be due to the percolation of the irrigation water down to collapsible soils at depth.

Figure 4. Cracking and settlement of boundary wall

To confirm this, a geotechnical company was enlisted to carry out a thorough investigation of the structural damages and to propose suitable methods of remediation. Two 15 m deep boreholes were drilled close to the areas of observed distress. The boreholes revealed a 1·5–2·0 m thick layer of topsoil, which was interpreted to be very loose to loose, based on the recorded SPT blow counts. Also, the groundwater table was encountered at an average depth of 1·5 m below the surface. Under these circumstances, to verify how the top loose soils responded to the presence of irrigation water, some open landscaped areas were selected and flooded with water (hydrocompaction) for 15 d to seep through the soil. Such flooding of water on soft landscapes was limited to the height of adjacent hard landscapes (footpaths) to avoid overflowing of water indiscriminately everywhere on the site. It was decided initially to adopt flooding (continuously 12 h) and desiccation (continuously 12 h) in equal intervals of time in a day until no further seepage of water into the ground is observed. However, this was continued for only 2 d, and such fixed cycle timings could not be continued due to heavy flooding in a short period of time. Finally, the site reached such a condition that 2 h of flooding time was sufficient for the entire landscaping areas to get flooded; hence, the hydrocompaction process terminated with a limited number of cycles. This speedy flooding situation could be attributed to less free draining material and a high groundwater table on site. The hydrocompaction process was terminated once it was noticed that no more water was seeping into the ground. To check whether the seepage of excess water into the soil had improved the density of soil, Mackintosh probe tests were undertaken before and after the hydrocompaction process. As shown in Figure 5, it was found that the soils responded to water movement because the number of blows after hydrocompaction increased for all depths down to 1·4 m. However, the

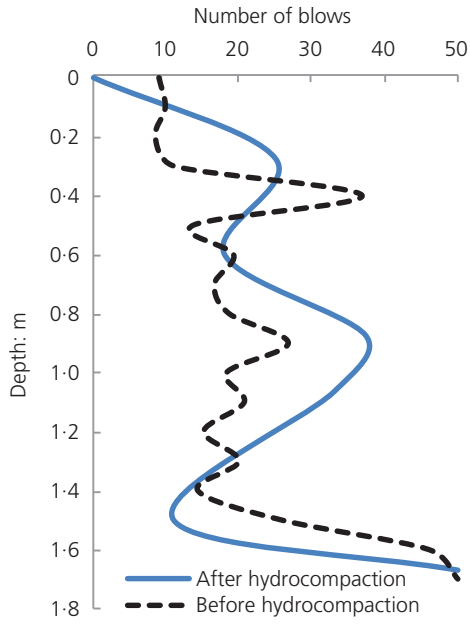


Figure 5. Mackintosh probe test results

improvement in the ground was not noticed locally at depths of 0·4–0·6 m, and this could be due to saturation of soil instead of response to collapse of soil structure due to hydrocompaction, which otherwise might have responded to water movement. A similar behaviour was noticed at a depth below 1·4 m, and this could be attributed to the nearness of the groundwater table, located at 1·5 m below ground. As stated by many researchers (Bell, 2000; Bolzon, 2010; Dudley, 1970; Jotisankasa, 2005; Petrukhin, 1989; Reginatto and Ferrero, 1973; Rezaei *et al.*, 2012), collapsible soils do respond to moisture and their density increases with movement of water due to the rearrangement of soil structure into denser packing; the presence of collapsible soils in the area of concern was confirmed.

It was considered that hydrocompaction might cause nuisance to the occupants of the villas, and so an alternative way of improving the loose soil at shallow depths was explored. Chemical grout (using 35% sodium silicate, 5% amide and 0·5% bicarbonate) was injected under boundary walls and the edges of hard landscaped areas to densify the upper 2 m of the soil stratum. For this purpose, holes were drilled down to 2·5 m below ground on either side of boundary walls at 1·5 m centres in a staggered manner and along the lines of private hard landscapes at 1·2 m centres in a linear manner. Under controlled pressure, grouting was done in such a way that upward heaving of the ground was prevented. Upon accomplishment of the grouting of all drilled holes, a period of 4 weeks was allowed for the grout to cure. Mackintosh probe tests were carried out before and after the grouting process to verify the effectiveness of the soil densification process. It can be seen (Figures 6 and 7) that the depth of improvement due to grouting was limited down to 0·6 m compared with that for hydrocompaction, where the improvement was noticed up to

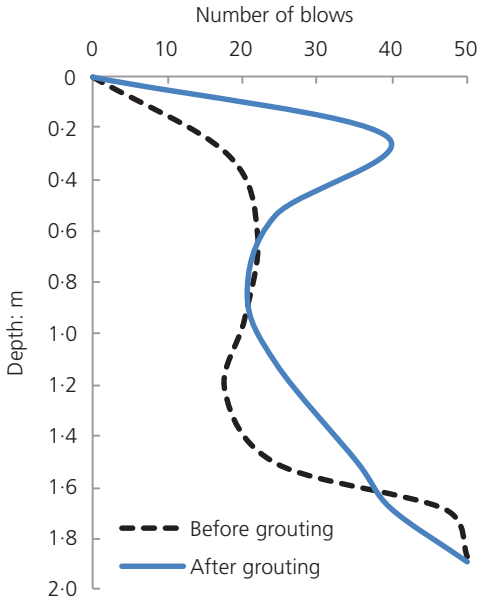


Figure 6. Mackintosh probe test results at boundary walls

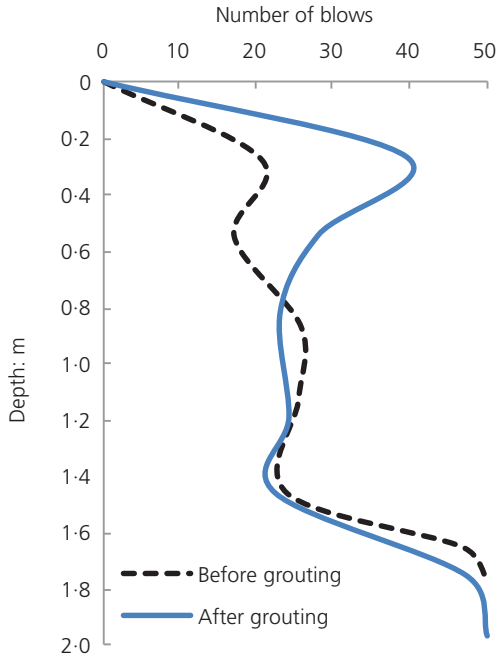


Figure 7. Mackintosh probe test results at hard landscaped areas

1·4 m below ground. Such limited depth of improvement in ground due to chemical grouting could be due to non-uniform permeation of grout into soil beyond 0·6–0·8 m below ground. Hence, it was suggested to continue with the hydrocompaction in all areas where settlements were noticed, allowing the settlements to proceed to their maximum values before continuing with repair work to reinstate the distressed structures.

4. Possible solutions

Taking into account the collapsibility of soil, solutions/techniques recommended by various researchers were summarised by Houston *et al.* (2001) and are given as follows

- removal of volume moisture-sensitive soil
- removal and replacement or compaction
- avoidance of wetting
- chemical stabilisation or grouting
- prewetting
- controlled wetting
- dynamic compaction
- pile or pier foundations
- differential-settlement-resistant foundations.

However, these possible solutions are recommended to be considered based on the site location, type of soil, practicability and so on. In view of understanding the suitability of the aforementioned solutions suggested by various researchers to the specific case studies discussed, removal followed by replacement and compaction or complete removal of moisture-sensitive soil options cannot be considered viable, as it is a tedious task and creates chaotic conditions for the existing tenants. Avoidance of

unwanted wetting can be considered as a solution in terms of controlling any undesirable leakages from underground conduits provided that efficient monitoring system is in place. Chemical stabilisation and prewetting (hydrocompaction) are feasible solutions on both sites provided that the efficacy of such techniques are verified beforehand. These techniques were tried in the infrastructure project, and, finally, it was suggested to opt for hydrocompaction compared with chemical grouting, as non-uniform permeation of grout was noticed. Controlled wetting could be considered as a solution in both cases provided that a specific quantity of water supply to the existing landscapes that does not lead to collapse of soil can be calculated and implemented strictly. Dynamic compaction is not an option in both case studies, as they are already developed sites and residents are in place. In both case studies, actual structures are already founded on piles and problems are associated with light loaded structures. Pile and pier foundations could be considered as a proper solution, particularly for boundary walls, provided that sufficient finances are available. Strap foundations can be considered for founding the boundary walls, which helps in controlling the differential settlements.

Keeping in view the problems associated with collapsible soils in the case studies described in this paper, the following solutions could be considered where such soils lie at limited depths not exceeding 2·5–3·0 m below the surface.

- Permanent sheet piling should be installed all along the periphery of villas/buildings founded on shallow footings, the development budget permitting.
- For low-rise buildings/villas, all isolated foundations should be either connected with continuous stiff strap beams or formed of raft foundations.
- Boundary walls should be bearing on long stiff beams all around the perimeter of the building. Optionally, the walls could be made with lightweight but sufficient materials or founded on minipiles.
- Where greenery (soft landscape areas) is planned around structures with no deep-rooted plants, existing soil could be excavated down to the top of collapsible soils and a layer of impermeable membrane inserted, followed by backfilling.

However, the deeper layers could be densified by pre-wetting through boreholes, using overburden pressure to drive the collapse (Houston *et al*., 2001).

5. Conclusions

Case studies of structural damage at locations in the UAE were examined to study the problem of collapsible soils in the areas and how human activities such as lawn irrigation exacerbate the problem. Lessons are learnt that the design of foundations in such environments calls for further considerations beyond the usual bearing capacity and settlement of just the founding soils. The problem lies at greater depths where collapsible soils exist and where infiltration of surface water can cause irreversible collapse of the soils to lead to structural damage over time. Therefore, the need to understand and properly consider the site geology in such sites cannot be overemphasised. Prior to development at such sites, a thorough geotechnical exploration is needed to detect and characterise any problematic soils possibly existing at depths far below the levels where boreholes would be terminated in straightforward cases. The case studies discussed in this paper will form part of an ongoing doctoral research project aimed at assessing the mechanisms of structural distress caused by collapsible soils in the UAE.

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ORIGINAL PAPER

Laboratory Simulation of Irrigation-Induced Settlement of Collapsible Desert Soils Under Constant Surcharge

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Abstract The heterogeneous nature of soil as a load bearing material, coupled with varying environmental conditions, pose challenges to geotechnical engineers in their quest to characterize and understand ground behavior for safe design of structures. Standard procedures for checking bearing capacity and settlement alone may sometimes be insufficient to achieve an acceptable degree of durability and in-service performance of a structure, particularly under varying environmental conditions, whether natural or man-made. There exists a wide variety of problematic soils that exhibit swelling, shrinkage, dispersion and collapse characteristics occasioned by changes in moisture content. Specific examples are collapsible soils, which occur mainly in arid and semi-arid regions, are generally capable of resisting fairly large loads in the dry condition but suffer instability and significant strength loss when in contact with water. A number of case studies in the United Arab Emirates were examined, where lightly loaded structures such as boundary walls, pavements and footpaths had been built on ground overlying collapsible soil strata. Sustained irrigation of the dry landscapes was found to have caused uneven settlement of the collapsible soils leading to continuous distress to the structures as evident from cracking and deformation. To help address the problem, an opportunity has been taken to develop a laboratory method of simulating the loaded behavior of collapsible soils in varying situations and to measure its deformation at constant surcharge and ground water infiltration rates. Finally, relationships were developed to estimate the time and magnitude of settlement, if thickness of collapsible soil is known.

Keywords Collapsible soil · Laboratory simulation · Deformation · Plate loading test

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1 Introduction

Collapsible soils are found in many parts of the world such as USA, Central and South America, China, Africa, Russia, India and the Middle East (Murthy 2010). Collapsible soils behave interestingly in that they may be competent as load bearing media in certain situations yet in other situations they present special challenges to engineers (Rezaei et al. 2012). Such soils are usually arid and semi-arid sands

consisting primarily of silt sized particles (Kalantari 2012) and may be susceptible to failure when subjected to water ingress from intermittent precipitation or deliberate water disposal. This is because collapsible soils suffer instability and structural breakdown when in contact with water. Furthermore, due to the high evaporation rate of soil moisture in dry regions, any underlying collapsible soil strata do not have sufficient time to consolidate under the in situ stresses (Pye and Tsoar 1990). Collapsible soils are generally characterized by their natural dryness, openness in structure and high porosity (Noutash et al. 2010). The structure of a collapsible soil strongly influences its mechanical properties as is true for other soil types (Leroueil and Vaughan 1990; Wesley 1990).

Jotisankasa (2005) stated that wetting of collapsible soil, through whatever mechanisms, coupled with loading causes a significant reduction in volume followed by structural collapse. A direct consequence of this is settlement and differential settlement of any structures founded on such soils, which undesirably lose much of their friction component of shear strength. In fact, with or without loading, increase in moisture content cause collapsible soils to exhibit abrupt changes in both volume and strength and this is markedly significant when the degree of saturation is above 50%. Nonetheless, partial collapse behavior of such soils can take place even without full saturation, as reported by Houston et al. (1993) and Abbeche et al. (2010), although other workers for example (Houstan et al. 2002) noted the total collapse of certain soils at given stress level requires a state of full saturation. Water ingress, by whatever means, into collapsible soil strata causes the groundwater table to rise. In urbanized arid/semi-arid sites, water from pipeline leakages, surface irrigation activities and industrial effluents can also percolate deeply into beds of collapsible soils underlying the site. It should be understood that the term "collapsible soils" does not mean a particular soil type but rather a whole variety of soils that are susceptible to structural collapse and examples include wind-blown sand, loess or alluvial soil types (Kalantari 2012). These soils are generally found in an unsaturated state in their natural condition (Zhu and Chen 2009). Other than effects of water, another cause of soil structure collapse is reduction in the strength of the bonding between soil particles, e.g. in loosely cemented sands where the cementing material is liable to softening and weakening by water. Even though it is difficult to predict the behavior of soils that exhibit collapse under unexpected or undesirable water ingress, many researchers have undertaken laboratory tests (Holtz and Hilf 1961; Jennings and Knight 1975; Jasmer and Ore 1987; Anderson and Riemer 1995; Reznik 2007; Gaaver 2012; Kalantari 2012; Rezaei et al. 2012) and field tests (Reznik 1993; Houston et al. 1995) in an attempt to identify certain characteristics of such soils. Among the most significant articles reviewed so far, only a small number are directly related to the current research and are summarized below.

1.1 Laboratory Tests

Holtz and Hilf (1961) suggested that loess-like soils are vulnerable to collapse when they have high void ratios and are saturated to the extent that their moisture content exceeds the liquid limit. A graph was developed for use in identifying whether or not a soil is likely to exhibit collapse behavior. Use of the graph requires knowledge of just two basic properties: dry density and liquid limit. In regions like UAE, where the current research is underway, most soils are silty sands that are either non-plastic or possess little or negligible plasticity. Thus, the procedure suggested by Holtz and Hilf (1961) may not be useful in the region of concern.

Anderson and Riemer (1995) used tri-axial equipment to perform constant-shear-drained (CSD) tests on uniformly graded sand and an undisturbed clayey alluvial soil. The test results showed that the collapse potential was related to the stress path, knowledge of which is necessary to accurately predict the collapse potential of such a soil.

Reznik (2007) developed equations to estimate the structural pressure (σ_{sz}) as a function of the degree of saturation (S) using oedometer test results reported by various researchers. 'Structural pressure' value is defined as stress at separation 'points' between elastic and plastic states of any soil (including collapsible soils) under loading. Soil collapse was observed to start when the applied stress exceeded the soil structural pressure values. This led to conclusion that the collapsibility of soil is a non-elastic deformation.

Gaaver (2012) conducted tests on various disturbed and undisturbed soils in an effort to identify the nature of the soils and possible methods of ground improvement. Equations were developed for predicting

collapse potential of soil, based on determination of the initial moisture content of the soil from lab tests. A new parameter RFSR (reduction factor in shearing resistance) was introduced and could be calculated if the initial moisture content of the soil is known, thereby enabling estimation of the reduction in bearing capacity. In UAE, due to hot climate mostly throughout the year, soils are very dry above groundwater table and thus moisture contents are too low. In view of collecting samples via boreholes for determination of moisture content, the drilling fluids being used will significantly alter the moisture content of soil thereby making it difficult to obtain samples in their true natural state.

One disadvantage of the strategies in the above mentioned researchers is the reliance on collecting undisturbed samples and carrying out time-consuming measurements such as oedometer and tri-axial tests. Another disadvantage is that it would be extremely difficult to obtain undisturbed and truly representative soil samples of cohesion-less silty sands particularly for UAE ground conditions. Therefore it can be commented that whilst the previously reported research is promising for the purpose of assessing whether a soil is collapsible and its degree of collapsibility, it does not closely represent actual field situations. This is because of not taking into account the effects of water ingress, groundwater influence and most significantly the influence of surcharge stresses due to structures resting over the soil in question.

1.2 Field Tests

Reznik (1993) conducted plate load tests on collapsible soils at a location in south-western Ukraine by simulating the water flow conditions that were consistent with those observed beneath nearby real structures, where rapid increase of settlements occurred due to uncontrolled wetting of soils. A parameter called the proportionality limit (P_{pr}) was introduced and is defined to represent the maximum pressure corresponding in the linear part of the load-settlement curve obtained from plate load test. Values of the P_{pr} obtained for collapsible soils were found to decrease with increase in water content. It was observed that the degree of saturation of soils under structures due to accidental wetting rarely exceeded 70–80%. Additionally the degree of saturation calculated after conducting the plate load test including wetting was found to be always below 80%. The findings indicated that the load testing technique applied for the collapsible soils was reasonable for design purposes.

Houston et al. (1995) developed an in situ test named 'downhole collapse test' and conducted a series of tests at a site known to exhibit wetting induced collapse. The test was performed in a borehole with load being applied to the plate at the bottom of the borehole. Water was introduced in the test and load-settlement response of the soil monitored. Using data from all tests, equations were developed thereof and used to estimate the collapse induced by wetting.

Field tests conducted by the two researchers mentioned above attempt to replicate the field conditions and the results would seem realistic for use in geotechnical design. Despite this advantage, such field replicating tests suffer one drawback in that they are laborious and often not cost-effective for some infrastructural projects. Thus the alternative of laboratory tests is still attractive to geotechnical designers provided that there is sufficient modification to create test conditions which simulate reality as closely as possible.

In this research work, the primary aim is to develop a deeper understanding of the behavioral characteristics of collapsible soil and to develop predictive methods together with appropriate parameter values to increase safety and economy in geotechnical design. This work concentrates mainly on laboratory testing of collapsible soils to simulate the effects of water infiltration due to irrigation of landscapes underlain by collapsible soil layers in arid/semi arid environments.

2 Timeliness and Significance of the Current Research Work

As already stated, many researchers have attempted to use laboratory and field methods in characterizing collapsible soils, however most of the methods have disadvantages in that they are time consuming and resource intensive. In addition, the methods do not adequately account for the effects of water ingress into soil, yet this is an important consequence of drip irrigation, pipeline leakage and precipitation. Also, field tests are considerably more expensive than laboratory tests as direct sources of design parameters for substructures built on problematic soils such as

collapsible soils. Therefore, as a better alternative, a carefully designed laboratory simulative test seems plausible as a method of developing empirical parameters for use in geotechnical design for structures built on over collapsible soil strata.

3 Experimental Study

In the current research work, it was planned to conduct plate load tests (BS 1377-9:1990) on collapsible soil in a custom designed tank of sufficiently large dimensions, to minimize boundary effects on the stressed zone of soil underneath a loaded plate lying on the soil surface. The tests include introduction of variable water table in the sand tank as well as controlled water infiltration rate to enable simulation of drip irrigation from which water would percolate deeply into underlying strata of collapsible soils supporting structures. The primary purpose of the tests is to understand underlying mechanisms and develop comprehensive data that would be used to formulate predictive equations for rates of settlement of collapsible soil, as functions of several variables such as (1) thickness of collapsible layer, (2) its depth from ground level, (3) groundwater regime. All tests were conducted at controlled infiltration rates and at specified magnitudes of surcharge loading.

3.1 Methodology

A number of case studies of structural damage examined in the UAE by Vandanapu et al. (2016) clearly showed structural distresses in lightly loaded structures such as boundary walls, hard landscapes, footpaths and pavements adjacent to areas under drip irrigation. No signs of distresses were noticed in larger structures such as residential houses and office buildings as most of them were founded on deep piles unaffected by superficial strata of collapsible soils. Consistent with the subsurface conditions under the distressed structures, it was planned to conduct constant-pressure (equivalent to the ground pressure exerted by boundary walls) laboratory plate load tests on collapsible soil to study the response of such light structures to changing water table levels occasioned by drip irrigation. The plate load tests were carried in two different cases as seen in Fig. 1.

In both cases, surface of the soil in the tank was loaded with a pressure equivalent to that exerted on the ground by the light structures and then settlements were observed with water infiltrating (simulating drip irrigation) from the surface. The tests were devised to help understand the settlement behavior of collapsible behavior of soil while the drip irrigation is underway. The influence of variable depths (simulating the actual groundwater table) of water the tank on time and magnitude of settlement was observed in case-1. In case-2, the influence of variable thickness of collapsible soil layer sandwiched between non-collapsible soil layers on time and magnitude of settlement was studied.

It is imperative that the laboratory test conditions represent the field situation as far as possible. Details of the experimental arrangement, materials and instrumentation specifications are described in the following sections.

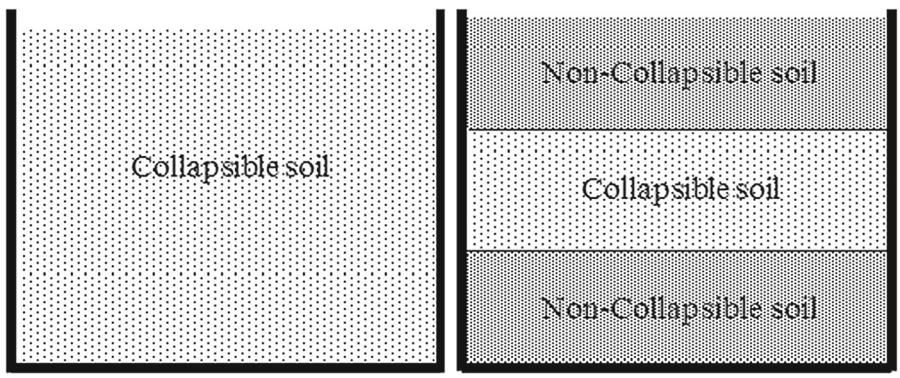


Fig. 1 Plate load test cases

Case-1 :Tank filled with collapsible soil only

Case-2: A collapsible soil layer sandwiched between two other layers in the tank.

3.2 Plate Load Test Set-Up

A cubic tank measuring 1 m × 1 m × 1 m was fabricated using a mild steel sheet of 4 mm thick, with carefully designed joints to create a water-tight enclosure. The prepared tank was placed under a loading frame made from a steel beam of 250 mm width and 250 mm depth with mass per linear meter equal to 72.4 kg/m and total mass of about 500 kg including the supports. This was done in order to act as a reaction while applying load on the soil using hydraulic jacks (Figs. 2, 3).

3.3 Test Load Calculations

As listed in Table 1, necessary calculations were made to derive the required weight of the loading frame

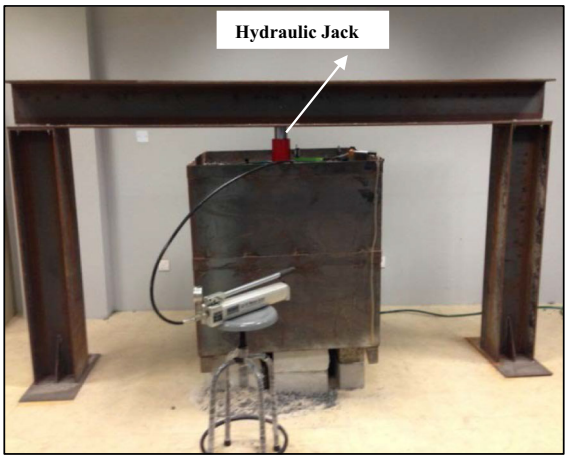


Fig. 2 Elevation of the experimental setup

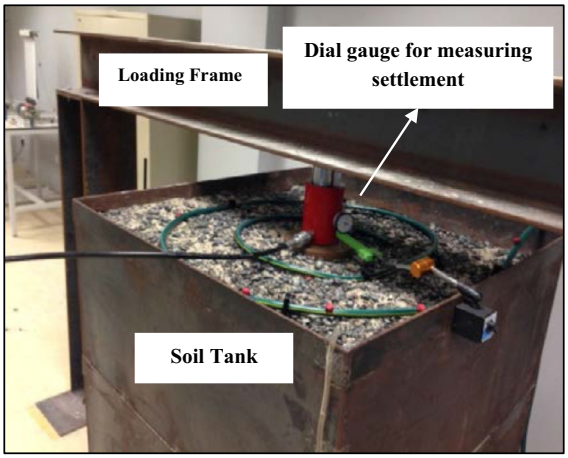


Fig. 3 The top view of the setup

Table 1 Minimum required reaction load for various stresses

Stress (kN/m^2)	Diameter of test plate (mm)	Minimum required reaction load (kg)
50*	200	157
100	200	315
150	200	472

sufficient to provide adequate reaction while testing the collapsible soil.

**Sample calculations*
Stress = 50 kN/m^2
Area of plate = πr^2 = 3.14*(0.01)^2 = 0.0314 m^2
Reaction load required (kg) = 50 × 1000 × 0.0314/10 = 157 kg

The dead weight of the loading frame used in the current work is 500 kg which evidently can resist a stress of up to 150 kN/m^2. The emphasis was to simulate the behavior of collapsible soil strata in the field as realistically as possible. All plate load tests were conducted at a constant pressure of 80 kN/m^2(calculated in accordance with the sizes, weights and foundations depths of various boundary walls and gazebos commonly used in the area of concern), which corresponds to the actual maximum pressure exerted on the ground by the kinds of installations cited above.

3.4 Replication of Groundwater Table

Many researchers have performed laboratory plate load tests but for either dry or fully saturated soils yet natural soils in the ground rarely fit this condition. So, it was considered more realistic to carry out model scale plate load tests with soil moisture contents and water table positions that relate to real field conditions. For this purpose, a hole was made on one side of the test tank and a piezometer inserted along with graduated scale to measure and control the water table level (Fig. 4). Initially water was poured into the empty tank to a depth of 10 cm from bottom and then dry soil was slowly added over the water. Furthermore, the placement of water and soil was done simultaneously until a stable water level in the tank was established.

3.5 Preparation of Collapsible Soil

Samples of collapsible soils were collected from various sites around Abu Dhabi, UAE, where

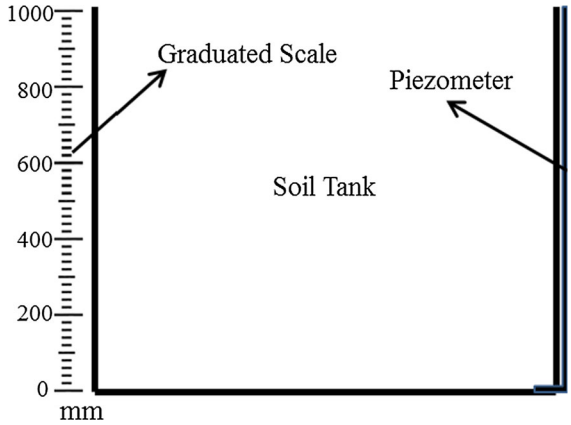


Fig. 4 Piezometer for monitoring water levels in the soil in tank

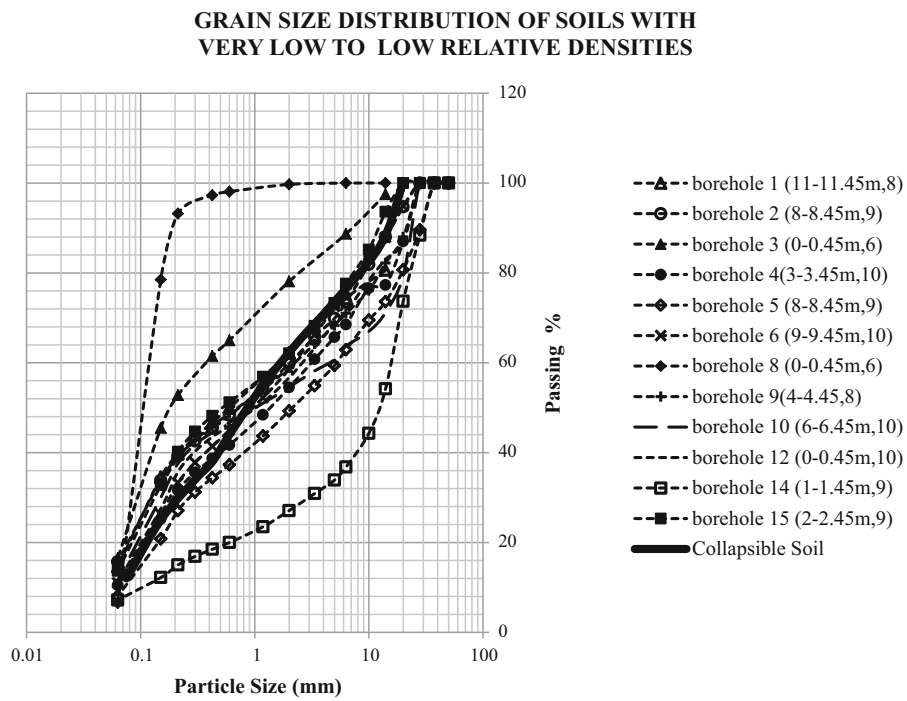
geotechnical companies had previously been contracted to undertake extensive investigations of structural distresses arising from ground movements. The main reason for the distress was subsoil collapse caused by ingress of irrigation water from the surrounding soft landscapes. The investigation companies drilled a number of boreholes, which showed that collapsible strata existed at depths where standard penetration test (SPT) values were low (N = 4 to 10) and very low (N < 4). It is from such depths where soil samples were collected for this research. The samples were analyzed using sieve tests (BS 1377-2:1990) to plot typical particle size distributions as shown in Fig. 5. The mean graph which closely represents the grain size distribution of all such soils was plotted and marked with a thick black curve in Fig. 5, along with SPT values and depths. The nomenclature followed in Fig. 5 is: depth, SPT N-value. For example (8–8.45 m, 9) indicates that the soil sample was obtained in respective borehole at a depth of 8.00–8.45 m using split spoon sampler and the SPT N-value recorded was 9.

Since filling the tank would require a large quantity of soil of specific gradation, a specialist company was contracted to grade the soil to required sizes on large scale basis using computer software. This facilitated production of 3 tons of soil fulfilling the desired gradation. In order to verify the software graded soil, random samples of the soil were subjected to laboratory sieving and found to be of acceptable particle size composition.

3.6 Plate Load Test Details

Initially the plate was set-up centrally beneath the loading frame and then a spirit level used to check and

Fig. 5 Grain size distribution



level the loading plate in horizontal position (Fig. 6). Once the plate had been set up correctly in position, a hydraulic jack was carefully placed over it and precisely below the loading frame. A dial gauge was then set up on the plate surface and mounted on a magnetic stand bearing on the side of the tank. Jack loads were then applied in increments equivalent to 1/10th to 1/12th of the targeted maximum pressure (80 kN/m^2) until the final pressure was reached. Care was exercised to ensure that, at each load increment, the plate settlement reached a stable value before readings were recorded. Thereafter, wetting of soil was done in a controlled manner.

In view of understanding the effect of groundwater regime coupled with controlled irrigation on settlement of foundation, three plate load tests were carried out at different water levels as listed in Table 2. Once the first plate load test with groundwater level at 2.5B below the plate was conducted, the soil lying above the water level was removed from the tank and dried completely. The dried soil was then placed in the tank and water table raised to 1.5B in order to proceed with the second test. The same procedure was also followed between the 2nd (1.5B) and the 3rd (1.0B) tests.

In addition, four plate load tests at various collapsible soil to total soil stratum ratios (1/2, 1/3, 1/4 and 1/5) were carried out to understand the effect of collapsible soil as a layer in the soil strata. For this purpose, collapsible soil was inserted as a layer at the mid-depth and plate load test tests at constant pressure were conducted accordingly.

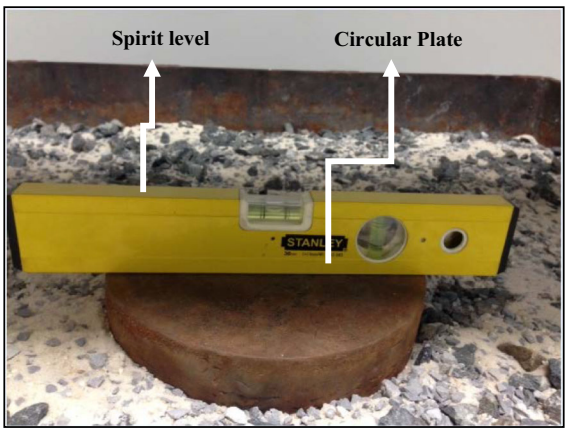


Fig. 6 Leveling the plate top

Table 2 Water table positions studied in plate tests

Test number	Depth of water level below bottom of plate
1	2.5B (500 mm)
2	1.5B (300 mm)
3	1.0B (200 mm)

B = diameter of plate = 200 mm

3.7 Watering Pattern

In order to create a laboratory simulation of the actual watering pattern due to irrigation of real landscapes, a pipe similar in diameter to the actual ones used in the UAE was prepared with perforations created at 15 cm intervals followed by fitting of drippers. The prepared pipe was placed on the surface of soil in the tank (Fig. 7) with one of its ends closed and the other connected to water supply. Once the targeted pressure of 80 kN/m^2 was reached, water was allowed to flow in definite cycles. A 'cycle' is defined as application of specified quantity of water every 12 h for a duration of 30 min. Drip irrigation was simulated over the soil in tank at the rate of 13 l/m^2/day (as per data obtained from local landscaping companies), watered twice a day (6.00 A.M.–6.30 A.M. and 6.00 P.M.–6.30 P.M.) equally at the rate of each 6.5 l/m^2. Rate of discharge of water was ensured with the help of water-meter fitted at the water outlet and a stopwatch. Such cycles were continued until rate of increase in settlement was so fast that our primary aim of maintaining constant pressure was not possible.

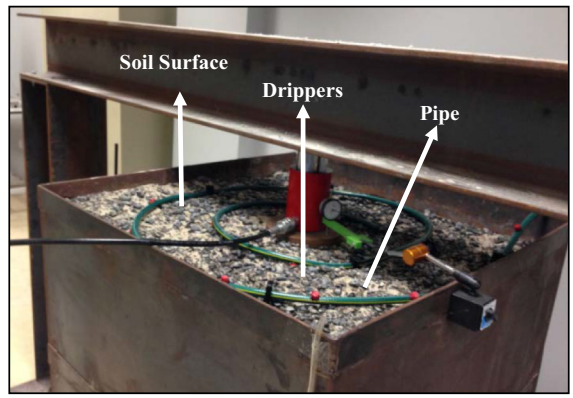


Fig. 7 Arrangement for simulating the field watering pattern

3.8 Constant Load Application Procedure

In actual practice, once any lightly loaded structure is constructed, its dead weight is largely constant throughout the lifetime, any live loads on such a structure being relatively small. On this basis, it was considered that the plate load tests ought to be conducted with maintained ground pressure to simulate the condition of a bearing soil medium. Once the dripping of water was initiated the soil started losing its strength due to the collapse of its structure which resulted in a decrease in the pressure exerted on the soil. Such a reduction in pressure was immediately compensated by manually applying pressure via the lever of the hydraulic jack (Fig. 8). This was possible due to continuous monitoring of pressure while applying the desired watering cycles on the soil surface.

4 Test Results and Discussions

All outcomes obtained from constant load plate load tests were elucidated in forthcoming sections.

4.1 Plate Load Tests–Full Collapsible Soil

Three plate load tests were carried out at different water levels as listed in Table 2. The water depths beneath the bottom of test plate were chosen to be consistent in scale to the actual foundation situations at the locations where structural distresses were investigated in the UAE case studies. Data from the plate load tests were transferred into Microsoft Excel workbooks for further processing in a bid to study the underlying patterns. Graphs of pressure against settlement and of settlement versus time were plotted and are discussed in the following sections.

Fig. 8 Maintaining constant pressure

4.1.1 Effect of Dripping Water on Settlement of Soil

Pressure–settlement graphs for all three tests conducted are shown in Figs. 9, 10, and 11. It was also observed that the number of wetting cycles required for the soil to reach collapsing state increased with increase in the depth of the water table below the plate

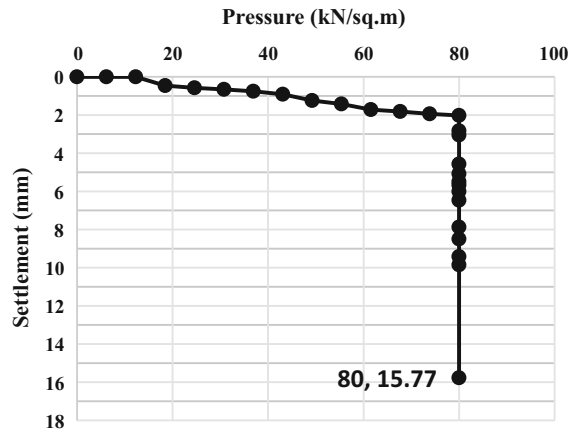


Fig. 9 Pressure–settlement curve with groundwater table at depth of 2.5B

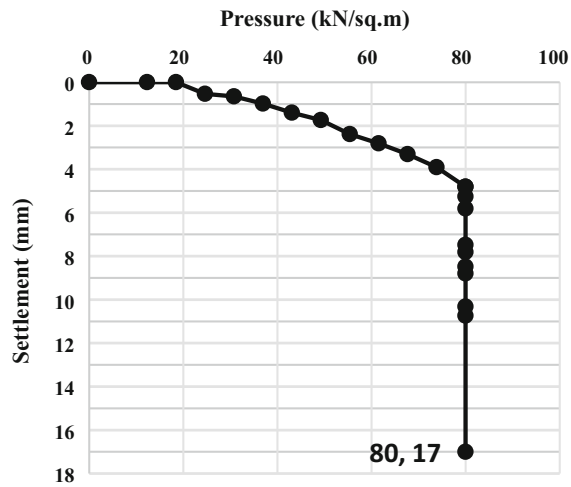


Fig. 10 Pressure–settlement curve with groundwater table at depth of 1.5B

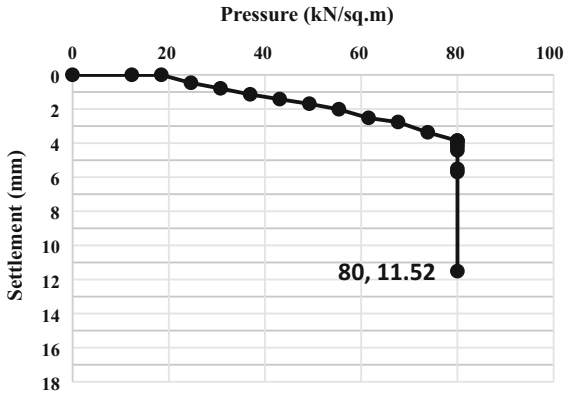


Fig. 11 Pressure–settlement curve with groundwater table at depth of 1.0B

Table 3 Wetting cycles before collapse

Depth of groundwater level below foundation	Number of wetting cycles
2.5B	7
1.5B	5
1.0B	4

'B' refers to width of foundation (width of plate in the plate load test)

(Table 3). This could be attributed to the presence of a deeper zone of soil (2.5B) involved in the collapse mechanism when the water level is at 2.5B, where the number of wetting cycles needed to cause soil collapse was greatest in comparison to the other cases. It is also apparent that the further the collapsible soil zone is below the plate foundation the higher is the number of cycles of wetting necessary to initiate soil collapse.

4.1.2 *Effect of Time on Settlement of Soil*

Time–settlement graphs for all three tests carried out are shown in Fig. 12. The graphs illustrate that the time required for the soil to exhibit its collapse behavior increases with increase in thickness of collapsible soil below the plate foundation. The time durations from commencement of test to start of soil collapse are listed in Table 4 for brevity and ease of understanding. A linear behavior is evident from the data in Table 4, so that a 0.5B increase in depth of water is equivalent to a time gap of 60 min between the start of test and the onset of soil collapse.

4.1.3 *Rate of Collapse*

As previously stated, each plate load test was terminated once settlement rate was so rapid that the prime objective of maintaining constant pressure cannot be continued. To understand the rate of collapse, the time–settlement data of the last wetting cycle from each test was used to calculate the rate of collapse. The calculations and corresponding results are shown in Table 5. It is evident that irrespective of the thickness of collapsible soil below the base of the plate, the rate of collapse exhibited by the soil in all three tests was fairly uniform at 6 mm in 30 min (0.2 mm/min).

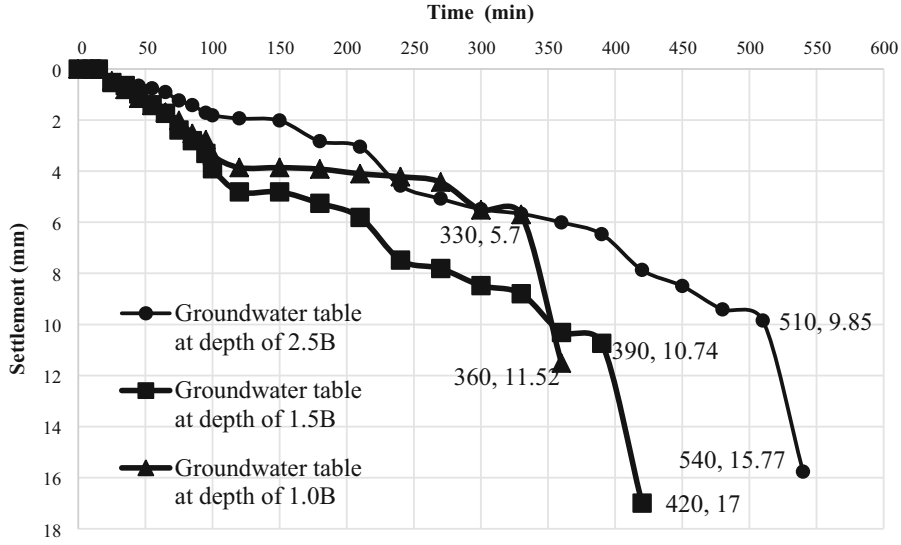


Fig. 12 Time–settlement curves at various groundwater levels

Table 4 Time taken to achieve soil collapse

Depth of groundwater table	Time (min)
2.5B	510
1.5B	390
1.0B	330

'B' refers to width of foundation (width of plate in the plate load test)

4.1.4 *Effect of Loading–Reloading on Modeled Groundwater Table*

Each time, once the soil was removed and replaced after drying, there was a drop in water level in the piezometer when weight was placed on the soil. To understand this behavior moisture content and specific gravity of soil after removing the soil was determined before replacing it with dry soil and was found to be 12% and 2.6 respectively. Compaction test (BS 1377-4:1990) was conducted on collapsible soil and the resulting curve plotted as shown in Fig. 13. It was interesting to note that the soil was not compacted to maximum dry deity (MDD) although it was on the path towards attaining it, with placement of more soil over. Calculations to support this observation are shown below.

From compaction test (Fig. 13)

OMC = 15.5%, MDD = 18.45 kN/m^2

At MDD, $\gamma_{d=G\gamma_{w/(1+e)}}$

$18.45 = (2.6 \times 10)/(1 + e)$

$e = 0.4$

$$\text{Now } s = \frac{wG}{e} = (15.5/100) \times 2.6/0.4 = 0.983 = 98.3\%$$

At 12% moisture content, $\gamma_d = 17.8$ kN/m^3

$\gamma_{d=G\gamma_{w/(1+e)}}$

$17.80 = (2.6 \times 10)/(1 + e)$

$e = 0.46$

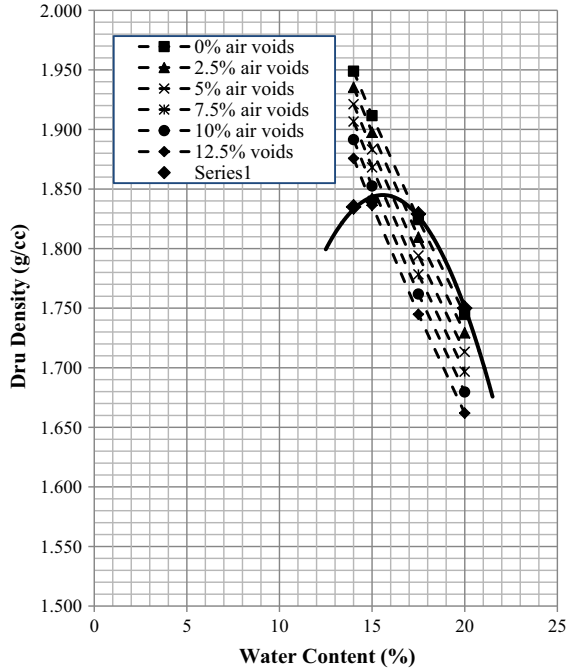


Fig. 13 Compaction curve

$$s = \frac{wG}{e} = (12/100) \times 2.6/0.46 = 0.68\,(68\%)$$

From the calculations it was inferred that the downward movement in water level due to placement of weight (soil) was attributable to the relief of pore pressure in the voids within the partially saturated soil (S = 68%) as it transited to a fully saturated condition.

4.2 Plate Load Tests–Collapsible Soil as a Layer

In this part of the work, permeability tests (BS 1377-5:1990) and laboratory plate load tests were conducted for a layered soil profile containing a collapsible soil lens, of variable thickness, inserted at mid-depth of the soil stratum below the plate. The layered soil profile acted as a bearing medium to simulate ground support for a superstructure. The results from all permeability tests are plotted against

Table 5 Settlement rate calculations

Depth of groundwater table	Settlement of soil before the start of collapse (mm)	Settlement at the end of test (mm)	Time between start of collapse and end of test (min)	Collapse settlement (mm)
2.5B	9.85	15.77	30	5.92
1.5B	10.74	17.00	30	6.26
1.0B	5.70	11.52	30	5.82

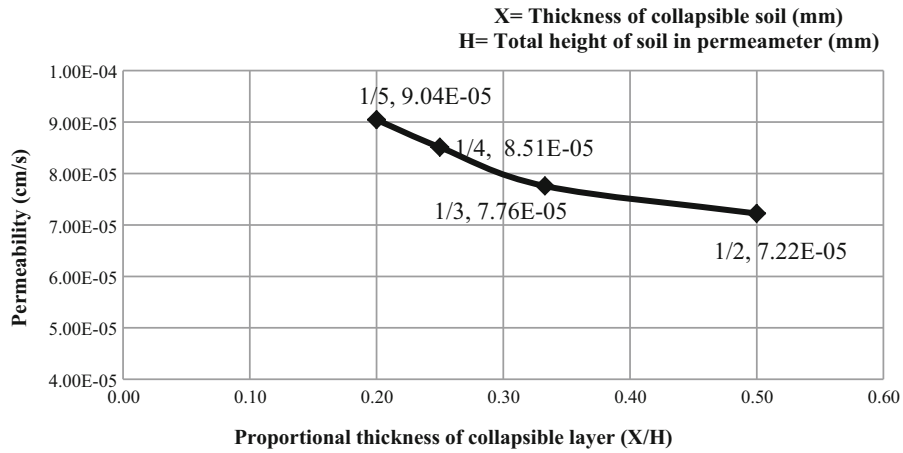


Fig. 14 Thickness of collapsible layer versus permeability

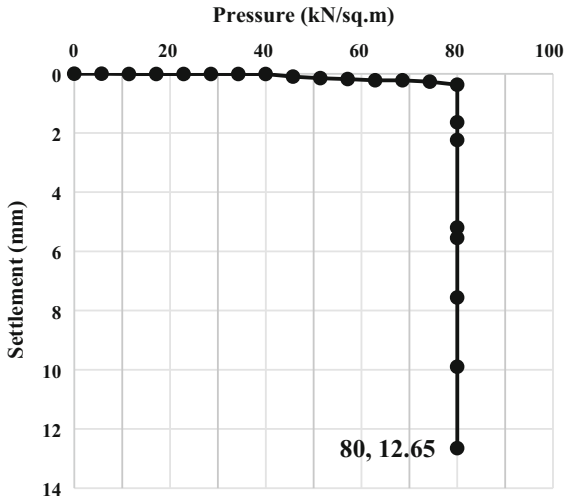


Fig. 15 Pressure versus settlement with collapsible soil at central half of the tank

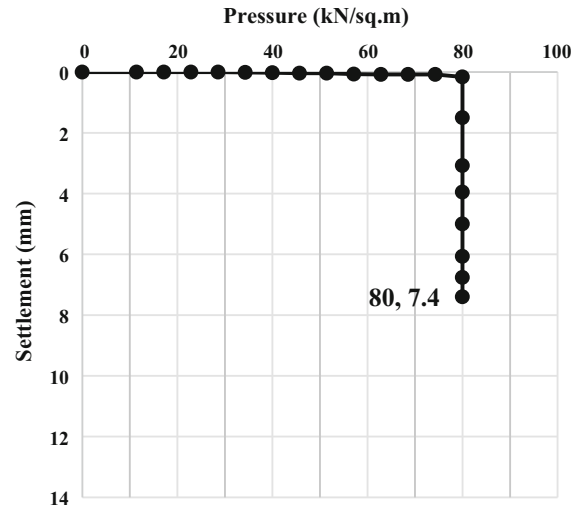


Fig. 16 Pressure versus settlement with collapsible soil at central 1/3rd of the tank

the thickness of collapsible soil layer in order to understand its behavior. Results from all plate load tests were also presented graphically. For this purpose various pressure–settlement and time–settlement graphs were constructed and are discussed in the next sections.

4.2.1 Effect of Permeability on Thickness of Collapsible Soil Layer

It is observed (Fig. 14) that there is a decrease in permeability of the soil with increase in thickness of the collapsible layer. This could be attributed to the increase in density of collapsible soil due to inward movement of water consequently leading to a decrease in permeability as the collapsible layer thickness increases.

4.2.2 Effect of Dripping Water on Settlement of Soil

It can be seen from the graphs in Figs. 15, 16, 17, and 18 that settlement decreases with decreasing thickness of the collapsible soil layer. This is expected because it is the collapsible layer and its thickness (rather than other layers) that are responsible for settlement under these situations. The relationship between settlement and thickness of collapsible soil is illustrated by the graph in Fig. 19. The variation trend line is represented by Eq. (1), which can be applied to a real problem in predicting settlement due to collapsible

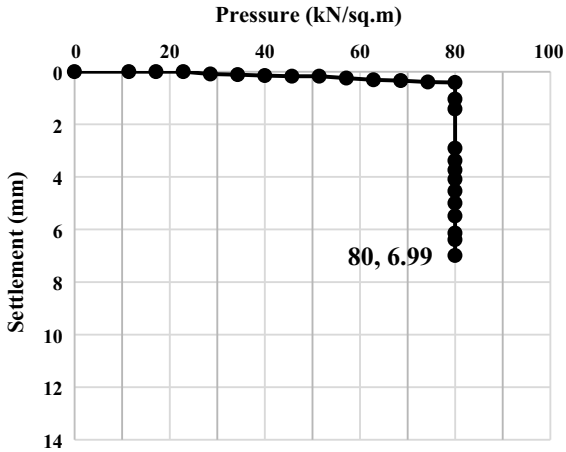


Fig. 17 Pressure versus settlement with collapsible soil at central 1/4th of the tank

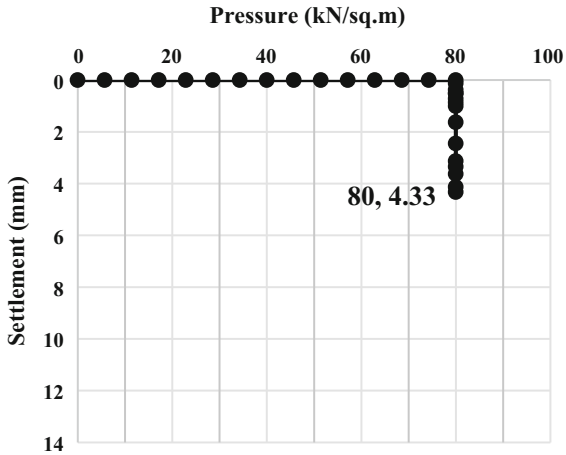


Fig. 18 Pressure versus settlement with collapsible soil at central 1/5th of the tank

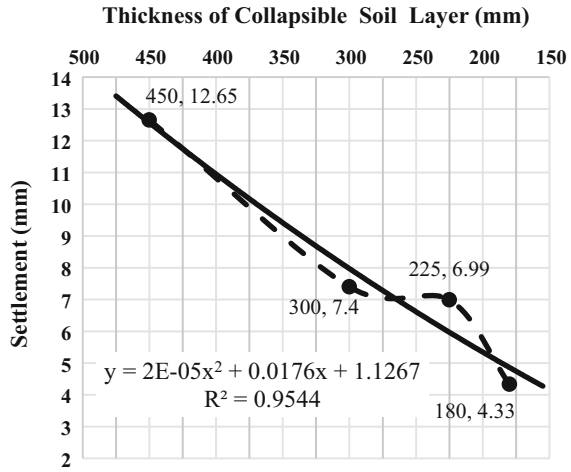


Fig. 19 Thickness of collapsible soil layer versus settlement

soil behavior, provided the proportionate thickness of the collapsible soil is known.

$$y = 2E - 05x^2 + 0.0176x + 1.1267 \quad (1)$$

where x = thickness of collapsible soil (mm); y = settlement (mm)

4.2.3 Effect of Time on Settlement of Soil

Time–settlement graphs for all plate load tests conducted are shown in Fig. 20. It is seen that in all cases, settlement increases with time but at different rates depending on the position and thickness of the collapsible soil relative to the tank depth. This is again attributed to the proportionate influence of collapsible

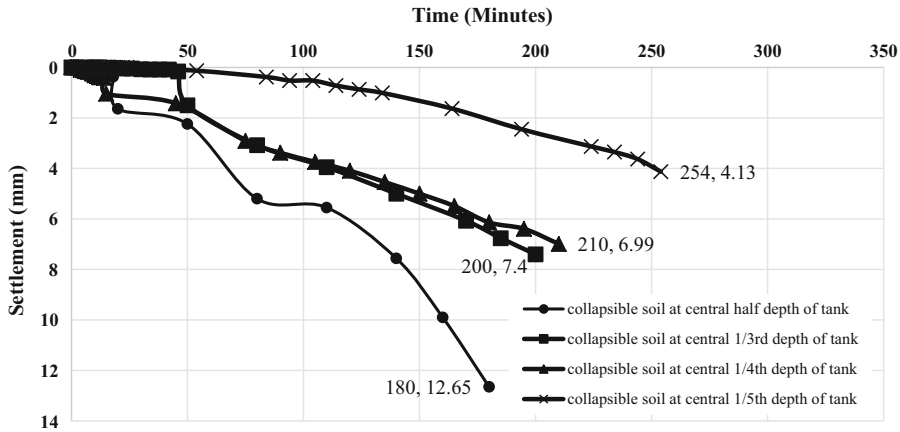


Fig. 20 Time versus settlement with various collapsible soil thicknesses at central depth of tank

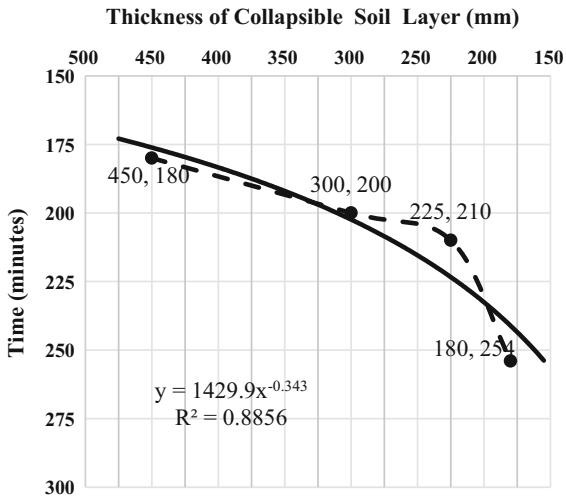


Fig. 21 Thickness of collapsible soil layer versus time

soil responsible for settlement. The relationship between time and thickness of collapsible soil is shown in Fig. 21, where the trend of variation represented by Eq. (2).

$$y = 1429.9x^{-0.343} \quad (2)$$

where y = time (min); x = thickness of collapsible soil (mm)

Equation (2) may be used to predict the time taken by the soil to exhibit settlement if proportionate thickness and depth location of the collapsible soil is known.

5 Conclusions

1. The number of wetting cycles required for the soil to exhibit the collapse increases with increase in depth of groundwater table below the foundation level.
2. Once the soil starts exhibiting its collapsible behavior, the rate at which it collapses was found to be uniform irrespective of its thickness.
3. The decrease in water level in the soil due to placing the soil (after removing) could be attributed to the removing pressure which being exerted by the air in air voids on the voids that are partially saturated to reach into a fully saturated conditions.
4. The permeability of the soil stratum decreases with increase in thickness of collapsible soil portion in it.
5. The magnitude of settlement increases with increased proportion of collapsible soil in a soil strata.
6. The time required for the soil to start exhibiting collapse increases with increasing depth of the groundwater table below the foundation. In addition, despite high magnitudes of ground settlement, the time required to attain the maximum settlement decreases with increase in the thickness of the collapsible stratum.
7. Predictive relationships were developed for linking the time period for maximum settlement to thickness of collapsible soil as well as magnitude of settlement to thickness of collapsible layer. These relationships can be used by geotechnical engineers to assess the rate and magnitude of settlements, depending on the thickness of the collapsible soil at a particular site. Though every effort has been made in the current study to prepare sufficiently large sized models to simulate field conditions relevant to the UAE case studies, inevitably there will be variations to be taken into account from one site to another. These variations include: the rate and frequency of irrigation, thickness of collapsible soil stratum and its depth below ground level as well as depth of groundwater table. Thus, geotechnical engineers need to exercise utmost care when assessing the important parameters such as time, rate and magnitude of collapse settlements in the particular locality of concern. A reliable assessment of the relationship between the intensity of landscape irrigation, water table level, thickness and location of collapsible strata can enable UAE geotechnical engineers to develop guidance for property owners/members of the public to help them control rates of irrigation hence avoid extreme ground settlement that would cause structural distress of the kind reported in the case studies in this paper.

The current laboratory test results will form part of an ongoing doctoral research project aimed at assess-

ing the behavior of collapsible soil under various structural loads. In addition, numerical analysis using finite element methods is planned to carryout in view of understand the behavior on large scale, whereby eventual aim is to develop equations and course of actions that geotechnical engineers may find useful while performing geotechnical analysis of structures resting on collapsible soil layers.

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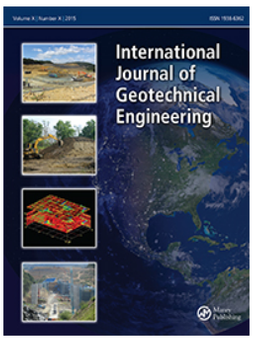

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Laboratory simulation of the influence of groundwater rise and drip irrigation on the settlement of a sample of collapsible desert soil

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Laboratory simulation of the influence of groundwater rise and drip irrigation on the settlement of a sample of collapsible desert soil

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ABSTRACT

Most types of sand have low susceptibility to settlement when in dry, dense and well-graded states but certain sands, when saturated, exhibit a decrease in suction and tensile strength hence leading to a sudden decrease in volume. Such sands are known as collapsible soils and require special consideration because their behaviour cannot be assessed using conventional settlement analysis methods. Irrigated landscapes overlying collapsible strata have caused settlement problems and infrastructure damage in Abu Dhabi area. Using borehole samples from the area, the present work simulates the settlement response of the collapsible sand when sandwiched between two other layers inside a metal mould and loaded with a constant surcharge pressure while varying water infiltration rates and static water levels. The primary goal of the research is to develop relationships for estimating settlement due to collapsible strata by taking into account layer thicknesses, groundwater table and irrigation intensity and duration.

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KEYWORDS
Collapsible soil; laboratory simulation; deformation; drip irrigation

Introduction

Collapsible soils are of a special kind in that they exhibit a decrease in suction and tensile strength when they are sufficiently saturated. Hence, this leads to a sudden decrease in volume and consequently settlement. Collapsible soils are usually characterized by high void ratio, low density, openness in structure, high porosity and low degree of saturation (Noutash, Hajialilue, and Cheshmdoost 2010). Soils of this type are found in many parts of the world such as U.S.A, Central and South America, China, Africa, Russia, India and the Middle East (Murthy 2010). Collapsible soils usually exist in shallow deposits and water ingress is the most salient reason for their abrupt reduction in volume occasioning structural collapse (Jotisankasa 2005). Water ingress can be through rainfall, continuous pipeline leakages, intensive landscape irrigation or large spillages at the surface. Despite having reasonable bearing capacity in the dry state, the tendency of collapsible soils to deform significantly and loose strength upon saturation poses special challenges to geotechnical engineers (Rezaei, Ajalloeian, and Ghafoori 2012). Most types of collapsible sands consist primarily of silt sized particles (Kalantari 2012) and occur in arid and semi-arid regions. In arid regions, high temperatures mean that the ground dries off rapidly and evaporation rates are high, thus there is very little time for underlying collapsible soil layers at superficial levels to consolidate under the prevailing overburden (Pye and Tsoar 1990).

The mechanisms of collapsible soils can be appreciated by considering how wetness destroys the metastable structure of the soil, with resulting breakage of bonds between the soil grains, leading to re-arrangement of soil particles into a denser mass hence volume reduction (Barden, McGown, and Collins 1973; Mitchell 1976; Jotisankasa 2005; Bolzon 2010). It should be noted that collapsible soils are not a particular type of soil, but are soils that are prone to structural collapse through loss of inter-particle friction (Kalantari 2012). Naturally collapsible soils usually exist in the unsaturated state (Zhu and Chen 2009) hence their prevalence in arid and semi-arid regions. It should also be understood that such soils require only a relatively short period of time to reach the collapse state when saturation levels are sufficiently high. In practice, the existence of collapsible soil deposits in close proximity to a water source has been found to create problems for ground bearing infrastructures such as pipelines, roads and buildings which can suffer damage due to excessive ground settlement.

Houston, Mahmoud, and Houston (1993) suggested that, even when not 100% saturated, certain soils may exhibit partial collapse behaviour, but Houstan, Houston, and Lawrence (2002) contended that full saturation is necessary for complete collapse to take place. Khalili, Geiser, and Blight (2004) conducted extensive tests and effective stress analysis on undisturbed clays from the site of Hume Dam, south-eastern Australia, and concluded that the settlement of the soil was largely due to a reduction in the yield stress. Houston, Hisham, and Houston (1995) developed a 'downhole collapse test' by placing a plate in a drilled borehole, adding water to the hole and applying incremental loading to the plate to measure load-settlement response. This led to equations for estimating the soil collapse due to wetting. Whilst such a practical test is consistent with reality, the cost involved may be

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undoubtedly too high and unjustified for some small projects. Notwithstanding the complexity of mechanisms involved in soil structural collapse, attempts have been made by various researchers (Holtz and Hilf 1961; Jennings and Knight 1975; Jasmer and Ore 1987; Tadepalli, Rahardjo, and Fredlund 1992; Anderson and Riemer 1995; Reznik 2007; Gaaver 2012; Kalantari 2012; Rezaei, Ajalloeian, and Ghafoori 2012) to experimentally assess and characterize the deformation behaviour of certain collapsible soil types in laboratory conditions.

Much of the laboratory work carried out by the above mentioned authors concentrated on: (a) undisturbed soil samples, which contrasts the situation with ground conditions in the UAE (United Arab Emirates) region, where most superficial deposits are non-cohesive silty sands that are extremely difficult to extract as undisturbed and (b) soils that are either perfectly dry or fully saturated yet this is obviously inconsistent with real situations where alternate cycles of drought, rainfall and other infiltration causing events are to be expected. Therefore, in this paper, an attempt is made to devise test conditions which are as representative as possible of actual ground situations in the UAE. The laboratory tests carried out in this research seek to examine and quantify how variations in groundwater levels and relative depths and thicknesses of a collapsible stratum influence settlement, for given rates of water infiltration and magnitudes of surface surcharge.

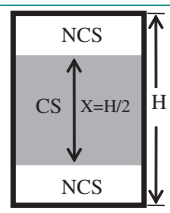
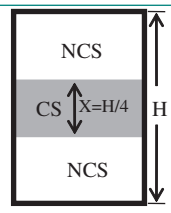
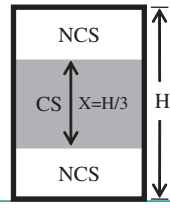
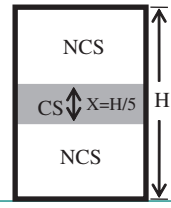
Experimental arrangements

From the outset, the challenge was to improvise a simple, cost-effective yet reasonable test arrangement to fit in the limited laboratory space available. Regardless of the equipment constraints, the experiment had to yield good enough data to enable understanding of the influence of controlled water levels, surcharges and stratum thickness on the settlement behaviour of a collapsible soil layer bounded by two free-draining layers. It was proposed to use a water supply tank fitted with 'infusion bottles' with controllable rates of discharge. This was to simulate intensity of landscape irrigation and consequent water level rises within a subsurface profile comprising a collapsible stratum. A metal mould, of the same type specified for a standard CBR (California Bearing Ratio) test in BS 1377-4:1990, was used to cast a three-layer soil profile with each layer compacted to pre-determined densities. A maintained surcharge of 4.54 kg was applied on the top of the uppermost soil layer in the CBR mould. The middle layer was formed from a specimen of collapsible soil obtained from some of the boreholes that had been drilled by a Geotechnical consultant in a part of Abu Dhabi City, where structural damage had been observed (Vandanapu, Omer, and Attom 2016) to be linked to irrigation-induced settlement of collapsible soil strata at depth. As reported by Vandanapu, Omer, and Attom (2016) signs of structural distress were detected in footpaths, road pavements and perimeter walls that were located close to irrigated lawns. No signs of distresses were noticed in residential villas and buildings since these were supported on piles penetrating collapsible strata and extending down to the rock head below.

Soil profiles and relative thicknesses in test model

In order to generate adequate data to tackle the objectives of the research, the settlement response of a collapsible soil specimen was measured by casting the soil to different thicknesses in a CBR mould under different water levels. Four soil profile cases: SC-1, SC-2, SC-3 and SC-4 were formed in moulds by casting the collapsible layer in between two layers of free-draining, non-collapsible types of sand. For each soil combination (SC), the overall thickness of the three soil layers in the mould was kept constant (H), as shown in Table 1. The difference in the four cases was is the thickness of the collapsible layer, which was set at H/2, H/3, H/4 and H/5 as shown in Table 1. For each soil combination, load-settlement data were measured for three compacted densities: 17.5, 18.0 and 18.5 kN/m^3. Furthermore, for each density case tests were run with water filled to three different heights of water in the mould, i.e. H/3, H/2 and 2H/3 from bottom of mould. Thus, a total of 36 tests were conducted. The intention was to recreate as far as possible the ground situation in the locations from where the collapsible soils were sampled, as part of the investigation of structural distress witnessed in a certain UAE region. Details about experimental set-up, materials and instrumentation specifications are described in the forthcoming sections.

Table 1. Soil combinations used in experimentation.

Soil combination (SC)	Details	Soil combination	Details
SC-1		SC-3	
SC-2		SC-4	

Notes: H – Height of the CBR mould (180 mm).
NCS – Non-collapsible soil.
CS – Collapsible soil.

Experimental test set-up

Before casting soils in the CBR moulds, a filter paper was inserted at the bottom of the mould to prevent soil particles from clogging the perforations in the bottom plate of the mould. Weighed amounts of each soil type were carefully placed and compacted in the moulds to desired thicknesses and densities. The moulds containing the compacted soils were then placed inside a wide-bottomed plastic tank in which water could be added to desired levels, as shown in Figure 1. This was done in an effort to simulate field conditions where the settlement of a collapsible stratum is influenced differently by different ground water table depths. To ensure easy entry of water into the moulds through the perforations, adequate care was taken to keep the underside of the mould sufficiently clear from the base of the tank using a thin spacer disc or seat.

Infusion sets were used to trickle water at controlled and measurable rates onto the top layer in the mould. This was to

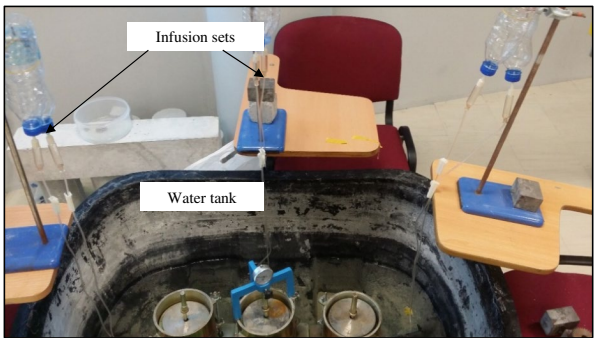


Figure 1. Purpose designed experimental arrangement for measuring settlement of collapsible soil under varying irrigation rates and water levels.

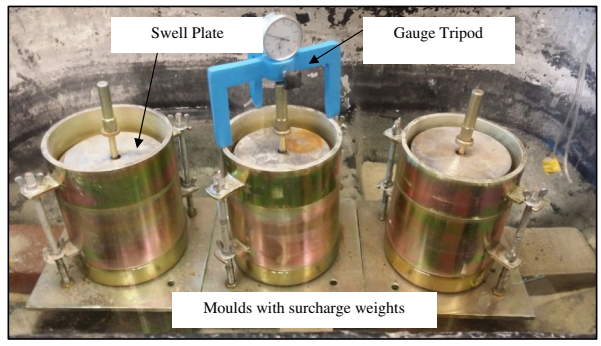


Figure 2. Monitoring of the initial gauge readings for soil in the dry state prior to start of irrigation.

simulate the typical irrigation rates ($m^3/m^2/s$) applied for lawns and landscapes in the areas of the UAE where settlement of subsurface collapsible strata had caused structural damage due to sustained water infiltration. With the free-draining nature of the top and bottom layers, the water level in the soil inside the moulds could quickly stabilize and match that in the tank. Using a swell plate and gauge tripod assembled as shown in Figure 2, settlements of the top soil surface were measured at close intervals of time as the water table was varied while continuing drip irrigation with the infusion sets at specific discharge rates.

Selection and preparation of the collapsible soil specimen

Following extensive ground investigations carried out by geotechnical contractors, collapsible soils in various areas around Abu Dhabi, UAE, were revealed as the reason for the distresses and damages caused to various shallowly founded structures. The settlement of the collapsible soil layers in the field was mainly due to deep percolation of water from human activities related to irrigation of lawns and landscapes around properties. From the ground investigations, borehole logs were produced which identified water levels as well as depth locations of collapsible strata where low SPT (Standard Penetration Test) values (from $N < 4$ to $4 < N < 10$) were encountered. Samples of the collapsible soils were collected from the field and made available for the present research. Representative samples of the collapsible soil from 12 exploratory boreholes were subjected to sieve analysis, from which the particle size distribution was plotted as shown in Figure 3. The thick continuous curve shows the mean particle size curve. The depth locations of the extracted samples as well as the corresponding SPT values are clearly shown in the legend of Figure 3, in the format: (depth, SPT N-value). For example (4–4.45 m, 8) indicates that the soil sample was obtained at a depth of 4.00–4.45 m using split spoon sampler and the SPT value measured was $N = 8$. Due to the large quantity of soil required for this research, the enormous task of sieving the collapsible soils from numerous boreholes was outsourced to a specialist company. Upon receipt of the soil samples from

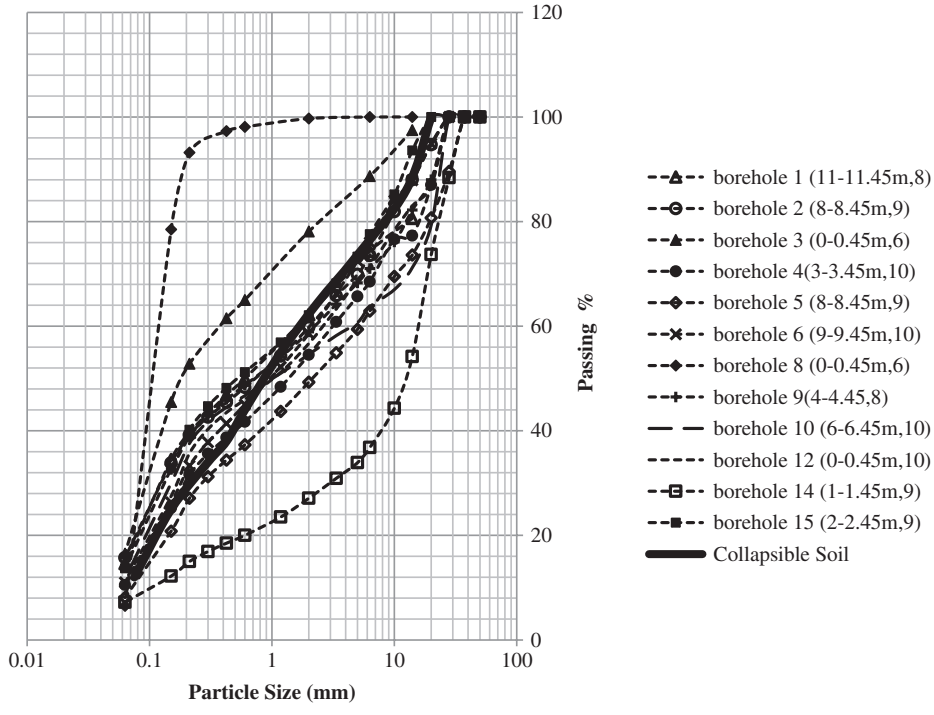


Figure 3. Grain size distributions of representative collapsible soil samples from 12 boreholes (sampling depths and SPT values shown in the legend).

Table 2. Depth location of representative samples from boreholes and properties of collapsible soil.

Borehole number	Depth of sampling (m)	SPT N-Value
1	11.00–11.45	8
2	8.00–8.45	9
3	0.00–0.45	6
4	3.00–3.45	10
5	8.00–8.45	9
6	9.00–9.45	10
8	0.00–0.45	6
9	4.00–4.45	8
10	6.00–6.45	10
12	0.00–0.45	10
14	1.00–1.45	9
15	2.00–2.45	9
Property of collapsible soil	Value	
Specific gravity	2.66	
Plasticity characteristics	Non-plastic	
Optimum moisture content	15.50%	
Maximum dry density	18.45 kN/m^3	
Permeability	8.86E-05 m/s	

Note: SPT – Standard Penetration Test.

the company, a range of laboratory tests were carried out on them to determine the basic properties, which are reported in Table 2 along with the sampling depth locations and borehole references.

Simulation of groundwater table

As previously stated, most researchers have concentrated on measuring settlement of collapsible soil in either dry or fully saturated conditions, despite such conditions being scarcely applicable to the natural environment in the ground. In the present work, the starting point was to fill the moulds with calculated weights of dry soils and statically compact them to the predetermined overall depth, H, in the mould hence achieving the targeted density. Thereafter, swell plate along with surcharge weights are placed and initial reading was taken using gauge tripod. The moulds were then placed in the plastic tank, to which water was added gradually to the target depths H/3, H/2, 2H/3 from the bottom of mould. Using the dial gauges, settlements of the top soil surface were measured and recorded continuously from the dry soil state until achievement of the target water depth. The measurements were continued until cessation of settlement as water seeped from the perforated plate at the bottom of mould. The difference between the initial dial gauge reading (with the soil still in the dry state) and the final reading upon cessation of settlement was attributed to the settlement induced by the water table rise.

Simulation of rates of landscape irrigation

Once the settlement due to rise in water table alone was established, further testing was undertaken to measure the soil settlement caused by the drip irrigation alone. To do this, a valve controlled infusion set was connected to an inverted water bottle opened at the top and filled with water as shown in Figure 1. Then the bottom end of the bottle, through which water exited via the infusion tube, was directed over the moulds and moved in uniform patterns to distribute water evenly on the soil surface in cycles of irrigation. A cycle was defined as discharge of water at a constant rate of 13 litres/m^2/day maintained for 30 min and repeated every 12 h. These figures were selected to be consistent with the data on irrigation rates and patterns obtained from local landscaping contractors operating in the areas of UAE where settlement-related damage was caused to infrastructure. Most of the irrigation contractors watered the ground twice a day (6.00am to 6.30am and 6.00 pm to 6.30 pm) uniformly at a spreading rate of 6.5litres/m^2. For the laboratory tests here, a trial and error approach was used and refined several times to find the equivalent rate of discharge which would be applicable to the surface area of the soil in the mould. The trials were done by altering the setting the flow control valve of the infusion sets and using a stopwatch to note the time durations of the drips applied.

Settlements of the top soil surface were recorded continuously until there was virtually no difference (≤0.01 mm) in settlement magnitude for two consecutive irrigation cycles. This was deemed to be a stable state for the settling soils. In order to maintain a constant discharge during an irrigation cycle, it was necessary to compensate for the gradually reducing head of water, as the drip cycle processed, by continuously feeding in more water through the open bottle top. At the end of the test, the settlement of soil due to drip irrigation alone was calculated by subtracting the dial gauge reading at the time before drip cycles commenced from the reading at completion of the drip cycles.

Test results and discussions

Data from the 36 test runs were presented in graphical format typifying trends of variation between:

(i) Surface settlement due to rise in water level only and normalized water depth (water table factor), for each of the three compacted densities and for each of the four soil strata combinations (Figure 4)
(ii) Surface settlement due to rise in water level only and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 5)
(iii) Surface settlement due to drip irrigation only and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 6)
(iv) Surface settlement due to combined rise in water level and drip irrigation and water table factor, for an average value of compacted densities and for each of the four soil strata combinations (Figure 7)
(v) Average surface settlement due to rise in water level only and thickness of collapsible layer (Figure 8)
(vi) Average surface settlement due to combined rise in water level and drip irrigation and thickness of collapsible layer (Figure 9)

For purposes of normalization, the 'water table factor' was defined as the ratio of water table depth to the overall thickness of the soils in the mould. Thus, the water table factor is plotted as a dimensionless quantity.

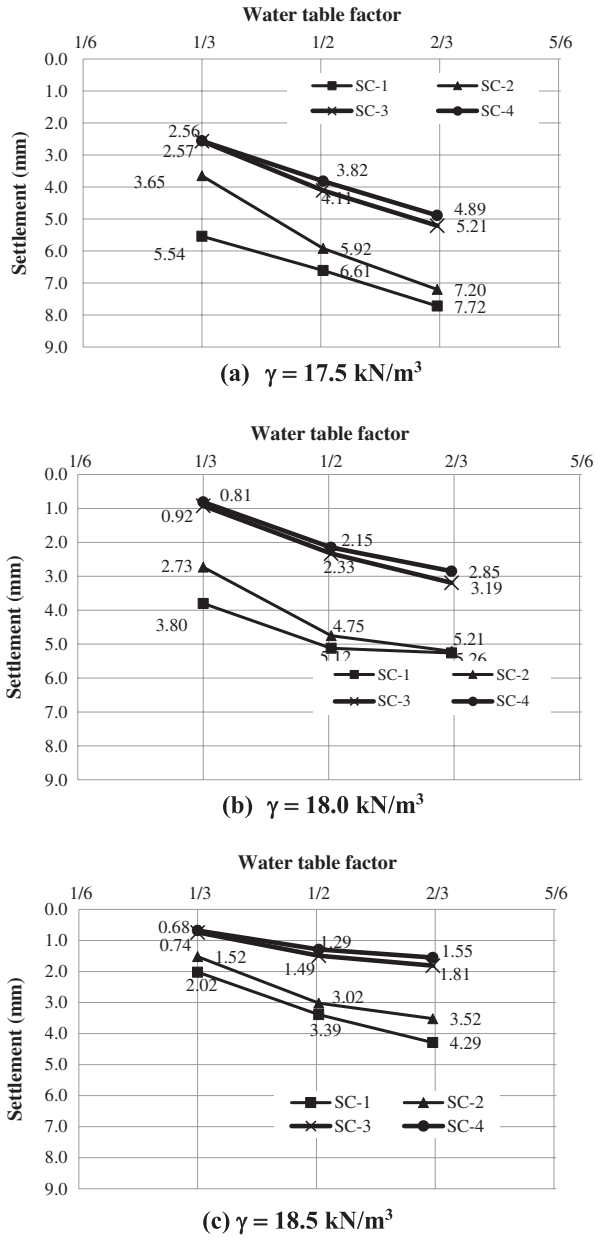


Figure 4. Variation between soil settlement due to water table rise and water table factor (results for different soil densities: 17.5–18.5 kN/m³).

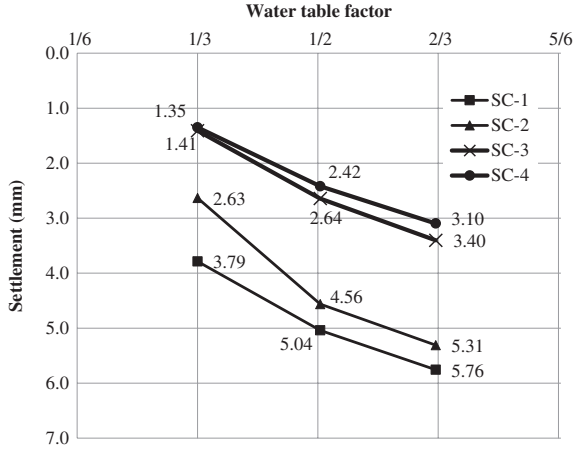


Figure 5. Variation between average soil settlement due to water table rise and water table factor.

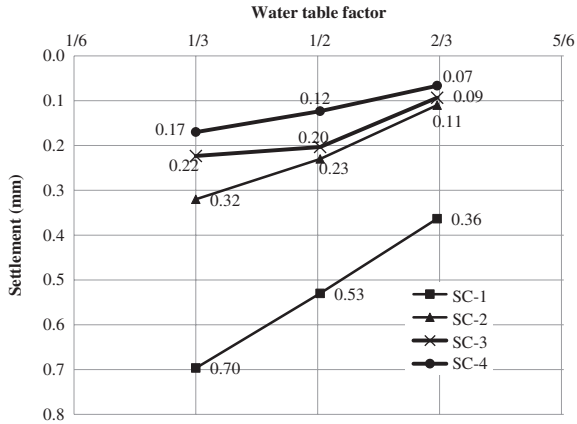


Figure 6. Variation between average soil settlement due to drip irrigation and water table factor.

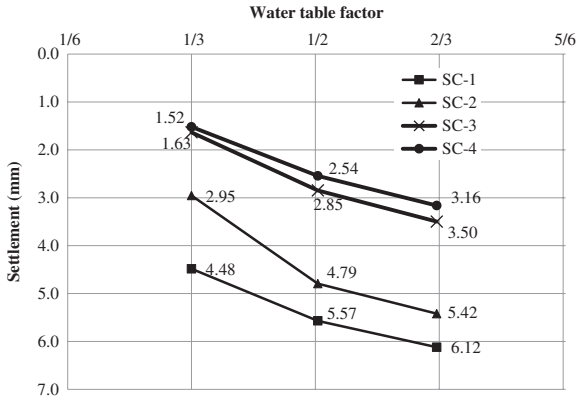


Figure 7. Variation of average soil settlement with water table factor due to the combined effects of water table rise and drip irrigation.

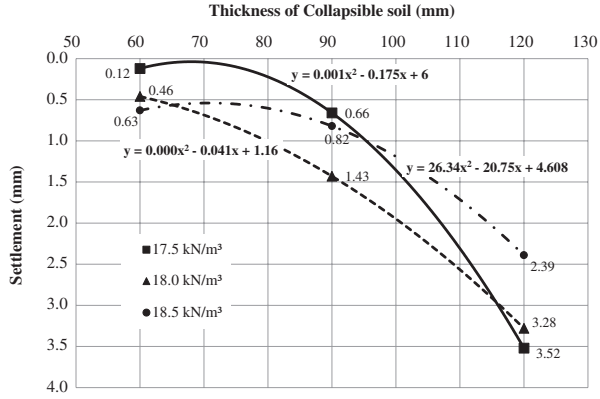


Figure 8. Variation between average settlement due to rise in water table and with thickness of collapsible soil.

Variation of settlement with normalized water table depth, for various soil densities

As can be seen in Figure 4 for all compacted densities, the soil settlement increased with increasing depth of the water table. This was attributed to an increasing proportion of soil mass gaining higher saturation degrees due to gradual ingress of water. Also, at any density level, settlement increased with increasing thickness of the collapsible soil within the profile. This was attributable to a correspondingly greater thickness of collapsible soil being

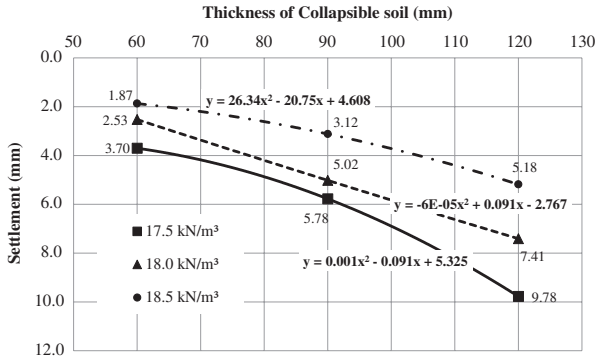


Figure 9. Influence of thickness of collapsible soil on average settlement due to combined effects of water table rise and drip irrigation.

influenced by the infiltration water. In addition, it can be seen that in overall terms, increase in the compacted density resulted in decrease in settlements. This was anticipated because the low air voids in the dense soil obviously meant decreased potential for the particles to re-adjust or deform further upon ingress of water.

Furthermore, of all the soil profile combinations, the maximum settlement of 7.72 mm was observed in SC-1, at water table factor of 2/3, highest thickness of collapsible soil layer and maximum water table height. Thus this may be regarded as the most critical combination of factors for the collapsible to settle the most. For this case, it was observed that with a density increase from 17.5 to 18.5 kN/m^3 the settlement decreased by a factor of 1.8 (7.72–4.29 mm). The observation here suggests that the *in situ* density of a collapsible stratum is crucially important in influencing the stability of the soil structure and hence settlement potential. For this reason it is imperative that densification by deep compaction is likely to be the most effective ground improvement technique to reduce settlement problems related to collapsible soil strata under the influence of water.

Variation of settlement with normalized water table depth for average compacted soil density

The graph in Figure 5 represents the variation trend for settlement versus water depth for averaged soil density. It can be seen that in general settlement still increased with increasing water table depth as was observed for different densities in Figure 4. However, there was no significant difference in settlement in profile cases SC-3 and SC-4 at a normalized water depth of 1/3. This happened because, despite the differences in the thickness of collapsible soil layers in cases SC-3 and SC-4, the water level was still below the collapsible stratum hence unaffected by it. However, the slight increase in average settlement from 1.35 to 1.41 could be attributed to the capillary rise of water due to the close proximity of the collapsible soil to the water level.

Variation of settlement due to drip irrigation with water level

In Figure 6, the aim was to study collapse settlements due to drip irrigation after the attainment of the full settlement caused by rises in the water table level. Further settlements as drip irrigation continued was expected because once the soils below the water table had reached collapse stage, the parts above the water table were still being wetted by irrigation water hence progressively causing additional collapse. It can be seen in Figure 6 that due to drip irrigation alone, the settlement decreased with increasing water table factor. This contrasts sharply with the previous observation that settlement due to rise in water table alone increased with increasing water table factor. The reason was that when large portions of the collapsible layer were already under water, the less saturated upper parts were rather too thin to give further settlement even under drip irrigation.

Variation of settlement due to combined effects of water level rise and drip irrigation

The combined effect of rise in water table and drip irrigation on settlement on soil is shown in Figure 7. Here, the settlement behaviour is essentially similar to that due to rise in water table only. Thus it is apparent that settlement of collapsible soils is influenced much more by the water table depth than by irrigation process, provided that much of the layer is already submerged.

Settlement predictions

It can be seen from Figures 8 and 9 that there is an increase in settlement with increase in the thickness of the collapsible layer. This happens due to water table rise alone (Figure 8) as well as due to combined rise in water table and drip irrigation (Figure 9). Under the combined influence of water table rise and drip irrigation, the surface settlement increases with decreasing density of soil, irrespective of the thickness of collapsible soil. A similar pattern of behaviour is exhibited at higher thickness of collapsible stratum (120 mm), due to rise in water table alone. It is seen that, at lower thicknesses (60 and 90 mm), the settlement behaviour is markedly different. This is attributable to the fact that the water table rise now affects only a partial zone of the collapsible layer, rather than the full height of the layer. With more extensive data points, curve fitting techniques can be used to model distinct trends of variation between thickness of collapsible soil and average surface settlement, for effects of: (a) rise in water table alone and (b) combined rise in water table and drip irrigation. The models can then be applied to real problems in predicting settlement, for known thickness and properties of the collapsible layer. Settlement due to drip irrigation alone can be predicted as the difference between the corresponding values modelled from Figures 8 and 9.

Conclusions

(1) The surface settlement of the soil profile was found to increase with increasing water table factor irrespective of the density of the layers.
(2) For all soil density values examined, the settlement at the surface was found to increase with increase in thickness of the collapsible layer in the profile.
(3) The settlement decreased with increase in density of soil in such a manner that a 1 kN/m^3 increase in density

of soil caused the surface settlement to decrease by a factor of 1.8.

(4) In the absence of drip irrigation, the surface settlement increased with increasing water levels. However, under the effect of drip irrigation alone, the settlement decreased with increasing water table factor.

(5) From the graphs of results, modelled relationships between the magnitude of settlement and thickness of collapsible soil can be used to predict the magnitude of ground settlements in real field situations, provided the thickness of the collapsible soil layer and properties of other layers in the profile are available from borehole investigations.

The present work is part of an on-going research project aimed at deepening knowledge of the settlement behaviour of a collapsible sand stratum when under the influence of irrigation-induced infiltration and overburden pressure. It is hoped that a further article will be produced focussing on numerical solutions and construction guidelines to engineers and property owners/irrigation contractors in regions where collapsible soils pose risks to infrastructure.

Disclosure statement

No potential conflict of interest was reported by the authors.

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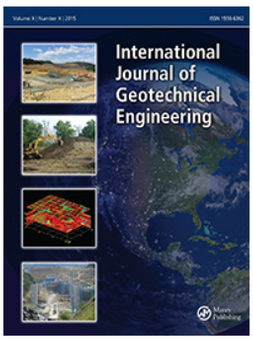

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Three-dimensional finite element analyses of ground settlement and structural damage caused by irrigation of desert landscapes overlying collapsible soil strata

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ABSTRACT

Experience in the United Arab Emirates (UAE) has revealed the settlement risk to foundations built on collapsible strata when such strata become increasingly wet due to irrigation of lawns. This paper presents a numerical analysis of ground settlement at a location in the UAE where structural damage occurred, prompting a forensic investigation that involved borehole drilling and measurement of subsidence and structural failure characteristics. *Midas*^{TM} *3D* finite element programme is used with field information from boreholes and irrigation specifications to simulate and predict the settlement profile for a typical pair of residential villas surveyed. Important factors are taken into account including the depths and thicknesses of the collapsible strata, the in-situ stresses, transient water flow, irrigation cycles, water table depth and the soil-structure mechanical properties. The maximum settlement of the boundary wall is predicted to be 157 mm, which agrees closely with the measured value of 165 mm. In addition, the predicted surface displacements are consistent with the observed ground and boundary wall deformation patterns.

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KEYWORDS
Collapsible soil; settlement; drip irrigation; finite element modelling; boundary walls

Introduction

Soils that possess collapse characteristics are found in many parts of the world such as USA, China, Central and South America, Russia, Africa, India and the Middle East (Mitchell and Soga 2005; Murthy 2010). On the one hand, collapsible soils in their natural condition may have adequate strength and hence usable in bearing load (Rezaei, Ajalloeian, and Ghafoori 2012; Alain et al. 2012) but on the other hand, water can destroy the internal friction of such soils, resulting in a sudden reduction in volume and consequently settlement (Casagrande 1932; Barden, McGown, and Collins 1973; Mitchell 1976; Lawton et al. l989; Pereira and Fredlund 2000; Jotisankasa 2005). Therefore, geotechnical engineers must understand the aforementioned unique behaviour of collapsible soils in order to ensure a safe design and to put in place appropriate measures that may be necessary to manage the risks caused to a structure. Collapsibility due to water is generally shown by certain types of sands and silts whereas for clays the tendency is to expand rather than collapse when wetted. Water can enter a collapsible stratum through precipitation, irrigation activities, wastewater disposal, pipeline leakages, seepage from water bodies and groundwater table fluctuation (Adnan and Erdil 1992).

A number of researchers (Denisov 1951; Clevenger 1958; Gibbs 1961; Benites 1968; Handy 1973; Houston, Mahmoud, and Houston 1993; Das 2007) have attempted to use simple laboratory index tests, with varying degrees of success, to elucidate the settlement behaviour of collapsible soil. Some researchers (Reznik 1993; Houston, Hisham, and Houston 1995; Mahmoud, Houston, and Houston 1995) have attempted to characterise collapsible soils based on field tests, which are generally more expensive than laboratory tests but better representative of *in-situ* conditions. Other researchers (Holtz and Hilf 1961; Jennings and Knight 1975; Jasmer and Ore 1987; Lawton, Fragaszy, and Hetherington 1992; Anderson and Riemer 1995; Celestino, Claudio, and Filippo 2000; Reznik 2007; Gaaver 2012; Kalantari 2013; Rezaei, Ajalloeian, and Ghafoori 2012; Vandanapu, Omer, and Attom 2017) have gone a step further to develop laboratory tests to simulate the effects of water on a collapsible layer and to formulate settlement prediction equations. With recent advances in computing and technology, other researchers (Alonso, Gens, and Josa 1990; Gens and Alonso 1992; Josa et al. 1992; Wheeler and Sivakumar 1995; Cui and Delage 1996; Wheeler 1996; Kato and Kawai 2000; Wheeler, Sharma, and Buisson 2003; Sun et al. 2007; Kakoli, Hanna, and Adayat 2009; Sheng 2011; Arairo et al. 2013; Rotisciani et al. 2015) have applied numerical modelling to analyse the influence of collapsible soil settlement on structural foundations and superstructures. Sophisticated numerical approaches, particularly finite element (FE) analysis offer numerous advantages not only because they can cope with complex soil-structure interaction mechanisms but also they take into account more factors than would be possible with simpler methods. These advantages are exploited in the present work, by focusing on 3D FE treatment of structures and foundations built on a soil profile incorporating collapsible strata.

The problem of moisture-induced strength loss of a collapsible soil and consequent structural distress has been studied by several researchers, including (Houston et al.

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2001; Noutash, Hajialilue, and Cheshmdoost 2010; Kalantari 2013; Vandanapu, Omer, and Attom 2016). In the current work, a case study is considered where various infrastructures (e.g. boundary walls and footpaths) at diverse locations in the UAE had suffered foundation failure or damage due to extreme settlement of collapsible strata occasioned by irrigation of adjacent landscapes. Therefore, an opportunity is taken here to implement a FE approach, with the aid of *Midas^{TM} GTS NX (v1.1) 3D program* (Midas Information Technology Co., Ltd 2014) to model the ground behaviour under simulated cycles of drip irrigation. For a realistic simulation, the irrigation input data (e.g. infiltration distribution and flow rates, sequence and timing of irrigation cycles) applied as exactly the same as those actually used by the landscape irrigation contractors at the sites where settlement problems occurred. Ultimately, the computed ground settlements are benchmarked against the actual values measured in the field. Additionally, the soil-structure module of *Midas^{TM}* is used to model the progressive collapse of masonry boundary that had been observed to have lost ground support from underneath. The aim of this was to understand the failure triggering mechanisms and hence suggest possible mitigation solutions.

Case study of settlement of collapsible soils in UAE

The project is a large scale infrastructure development located in Abu Dhabi (UAE), which comprises villas, shopping centres, indoor game complexes, open playgrounds, tennis courts, open green areas etc. Within a period of one year after completion of the construction and commissioning of the developments, many shallowly founded structures such as roads, hard landscapes (Figure 1) and soft landscapes underwent subsidence, whilst boundary walls (Figures 2 and 3) showed severe distress and cracking. By contrast, the villas, shopping centre and game complex were intact understandably because they were founded on piles embedded in rock. The maximum settlements in the hard landscapes, roads and boundary walls were measured to be 75, 100 and 165 mm, respectively.

Figure 1. Subsidence of hard landscape adjacent to a villa.

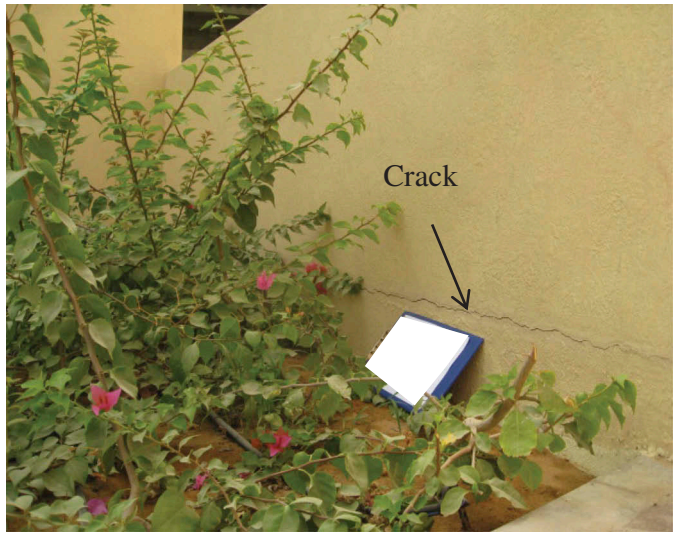

Figure 2. Initiation of cracking in a boundary wall.

As a consequence of the aforementioned structural failures, the property owners engaged a geotechnical specialist company to investigate the causes of the problem and recommend methods of alleviating them. The company therefore drilled two exploratory boreholes to 15 m depth, establishing the groundwater table to be at an average depth of 1.5 m below the surface. The boreholes revealed the general stratification profile as shown in Table 1.

Initially there was some doubt by the geotechnical engineers as to whether the observed settlement problem could be blamed on infiltration of water from the irrigation of the adjacent landscapes. But at the same time it was noted that all the affected areas adjacent were in fact close to or within the irrigated landscape areas. Therefore, to eliminate any doubts, a trial part of the landscaped area was flooded with excess irrigation water (Figure 4) and allowed time for the water to seep through, before performing a hydro-compaction process. This set of activities was carried out for 2 days, subsequent to which it was noticed that no more water seeped through the soil. In order to check the efficiency of this technique and to identify whether the underlying soils were responding to water ingress, a series of Mackintosh probe tests (Figure 5) were carried out before and after the hydro-compaction. It was noticed

Table 1. General stratification profile of the case study site.

Depth (m)	Description of soil	Range of SPT values	Relative density
3.0–5.0	Silty SAND	14–27	Medium dense
5.0–6.0	Silty SAND	6–30	Loose to medium dense
6.0–9.0	Silty SAND	13–24	Medium dense
9.0–13.0	Sandy SILT	16–50	Medium dense to dense
13.0–15.0	Sandy SILT	>50	Very dense

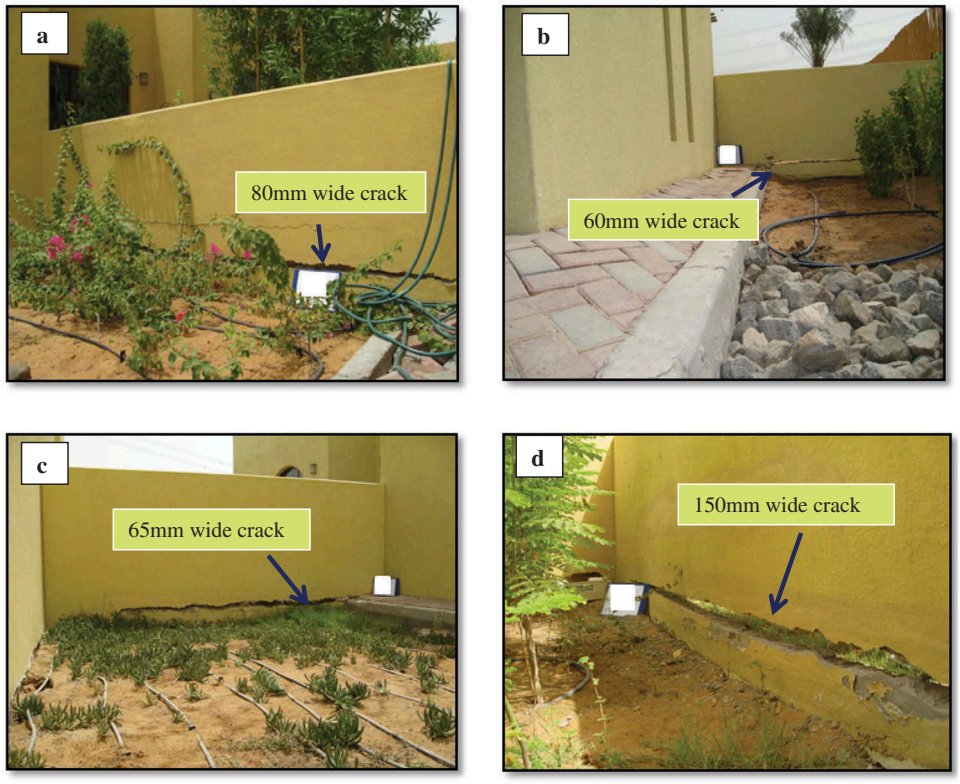

Figure 3. Severely distressed boundary walls due to cracking and settlement.

that soils at depths above the water table had responded to water infiltration, except for few local pockets located at 0.4 –0.6 m depths below ground level. This observation was clearly due to saturation effects on a uniquely responsive soil, rather than compaction effects (Vandanapu, Omer, and Attom 2016). Thus the presence of a collapsible layer, loosing inter-particle strength when sufficiently wetted, above the water table was confirmed.

FE modelling

In an attempt to overcome some of the limitations of laboratory and field tests used in studying collapsible soils, the current work advances a radically different approach in the quest for a more realistic, powerful and reliable numerical solution for the above problem. The new strategy involves:

(a) A comprehensive geotechnical model of twin villas with surrounding lawns with numerically simulated seepage intensity and cycle timing consistent with the actual specifications of the landscape irrigation.
(b) 3D FE soil-structure interaction analysis of the villas and their perimeter walls.
(c) Non-linear FE structural analysis of the perimeter walls, from where settlement predictions matching

Figure 4. Investigative flooding of landscaped areas.

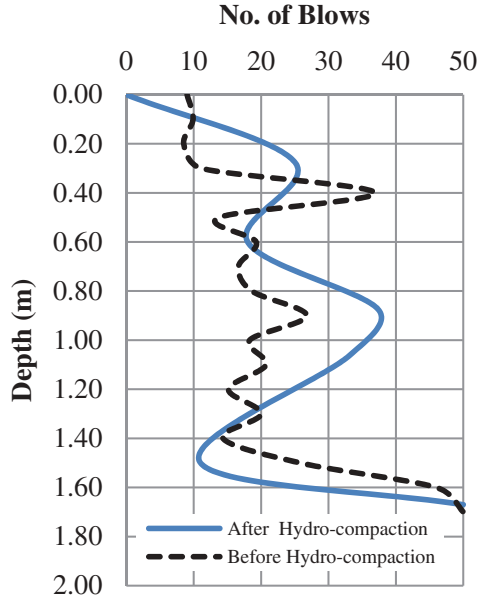

Figure 5. Mackintosh probe test results (Vandanapu, Omer, and Attom 2016).

on-site measurements would serve to verify the validity of the analyses in (a) and (b) above.

Geotechnical modelling

Given the complexity of behaviour of collapsible soils and the incapability of laboratory tests to represent actual field conditions, it was considered that a fully coupled stress-seepage 3D FE analysis would better deal with the problem and produce realistic simulations of the ground collapse response to irrigation. To tackle the complex problem, it was necessary to design an appropriate mathematical model and deploy a powerful 3D FE programme. For this purpose, *Midas*^{TM} *GTS NX* professional software (Midas 2014) was selected due to its advanced ability to cope with soil-structure problems involving 3-D transient seepage. The programme can handle seepage and ground stress as a fully coupled analysis, giving outputs of pore pressure differentials and time dependent stress and deformation variations. Since the analysis does not follow the common assumption that steady pore water pressure is maintained, it is advantageous over other methods when transient seepage and stress analysis is significant in a problem. The fundamental relationships, compatibility equations and numerical schemes underlying *Midas*^{TM} treatment of unsaturated materials and coupled stress-seepage under transient conditions are explained below.

Seepage parameters and relationships

Though Darcy's law was originally derived for soils in saturated condition, many researches (Narasimhan 2004; Ghotbi, Omidvar, and Barari 2011) have shown that it can be applied to unsaturated soils also. In the present work, seepage flow is considered along the three mutually orthogonal directions x, y, z of the model and the permeability coefficient matrix is represented as shown in Equation (1) where only the diagonal components in each direction are considered.

$$k = \begin{matrix} \mathrm{k_x} & 0 & 0 \\ 0 & \mathrm{k_y} & 0 \\ 0 & 0 & \mathrm{k_z} \end{matrix}. \tag{1}$$

The permeability coefficients are a criterion for controlling the seepage rate and depend on moisture content and void ratio change, Δe. Since moisture content is dependent on pore pressure, it follows that permeability values also change with pore pressure, Δp. In the adopted model, Δe is used for consolidation analysis with fully coupled stress-seepage analysis. Values of Δe are calculated from the initial condition defined in the input. The unsaturated permeability coefficient is calculated from Equation (2).

$$k = 10^{\frac{\Delta e}{c_k}} \mathrm{k_r(p)} \mathrm{k_{sat}}, \tag{2}$$

where,

k = unsaturated permeability coefficient

Δe = change in void ratio

c_k = the term that defines the permeability ratio as a function of Δe

k_r *(p) = permeability ratio function depending on*Δp

k_{sat} = *saturated permeability coefficient*

In the analysis, volumetric water content is defined in terms of the ratio between the water volume and total volume as shown in Equation (3).

$$\vartheta = \frac{V_w}{V} = nS, \tag{3}$$

where,

θ = Volumetric water content

V_w = Water volume

V = Total volume

n = Porosity

S = Degree of saturation

Calculation of element seepage and consolidation utilise the volumetric water content for pore pressure (p), and requires differentiation of Equation (3) and expressing the result using porosity and degree of saturation as shown in Equation (4).

$$\frac{\partial \theta}{\partial \mathrm{p}} = s\frac{\partial \mathrm{n}}{\partial \mathrm{p}} + n\frac{\partial \mathrm{S}}{\partial \mathrm{p}}. \tag{4}$$

The first term of the right-hand side of Equation (4) represents the rate of change of the volumetric water content for the saturated condition. It is defined by a parameter called the specific storage (S_s), which represents the volumetric ratio of the water movement in the ground due to the pore pressure head change (Equation (5)).

$$S\frac{\partial \mathrm{n}}{\partial \mathrm{p}} = \frac{\partial V_v}{\partial \mathrm{h}}\frac{\partial \mathrm{h}}{\partial \mathrm{p}} = n\frac{S_s}{\gamma}, \tag{5}$$

where,

V_v = Void volume

h = Pore pressure head

The second term of the right-hand side of Equation (4) represents the slope of the volumetric water content for the unsaturated condition. This value uses the slope of the soil-water characteristic curve represents the relationship between the volumetric water content and pore pressure for unsaturated conditions. In the model, adopted in *Midas*^{TM} the non-linear characteristics of unsaturated soils are represented by various forms of ductile functions including: pressure head versus water content, water content versus permeability ratio function or pressure head versus saturation and saturation versus permeability ratio function.

Modelling of seepage elements

Various relationships are used in *Midas*^{TM} to model elements for analysis of pore water seepage in both saturated and unsaturated soils. An important parameter involved here is the mass concentration of water in the ground, ρ_wnS. This can be defined considering the continuity equation of mass for micro-volumes. Continuity requires that the amount of water escaping from the micro-volume equals the change in mass concentration (Equation (6)).

$$\nabla^T(\rho_w \mathrm{q}) = \frac{\partial}{\partial \mathrm{t}}(\rho_w \mathrm{nS}), \tag{6}$$

where q = Seepage flow velocity component

The right term of the Equation (6) can be expressed using the changes in water density, degree of saturation and porosity with time as shown in Equation (7).

$$\frac{\partial}{\partial t}\left(\rho_w nS\right) = nS\frac{\partial \rho_w}{\partial t} + \rho_w n\frac{\partial s}{\partial t} + \rho_w S\frac{\partial n}{\partial t}, \quad (7)$$

The adopted model is based on Darcy's law, considering porosity change with time only in the formulation process for element consolidation analysis. Pore pressure (p) is a variable in the seepage analysis, and the governing equation for the analysis is derived from Darcy's law as shown in Equation (8).

$$\frac{1}{\gamma_w}\nabla^T(k\nabla p) - \nabla^T(kn_g) = \left(\frac{nS}{\rho_w}\frac{\partial \rho_w}{\partial p} + n\frac{\partial S}{\partial p}\right)\frac{\partial p}{\partial t}, \quad (8)$$

where,

k = coefficient of permeability matrix

n_g = unit vector in gravitational direction

To define the initial conditions for transient seepage analysis, the ground water level is defined. Then steady-state analysis results are used at the initial time step load.

Modelling of consolidation elements

The analyses with *Midas*^{TM} specifically use consolidation continuum elements to simulate stress-seepage coupled phenomena. During this process, consolidation analysis is fundamentally performed as a non-linear analysis. Pore pressures related to both the steady state and transient states are identified and so classified. The initial water level defined in the model is considered as the steady-state pore pressure, and the excess pore pressure during consolidation is considered as the transient state pore pressure. The transient state is the fundamental state of consolidation analysis. On completion of the element consolidation analysis stage, the results are expressed with reference to a user specified coordinate system.

With reference to the problem on hand, the sizes of all components of the geotechnical model were defined to match the respective on-site dimensions. The components included the twin-villa complex with boundary walls, hard landscapes, soft landscapes (drip irrigated areas) and respective car parks (Figure 6).

The various control settings and parameter values used in modelling are described in the following sections:

Soil properties. Relevant parameters for various soils (Table 2) were derived from the ground investigation report produced by the specialist geotechnical investigation company in the UAE. Where laboratory soil test data were unavailable, values were assessed using appropriate correlation charts and tables.

Loads of various infrastructures. Loads of villas, hard landscapes, boundary walls and car parks were inputted to model as 5 kN/m^2 (very less in magnitude), 10, 80 and 60 kN/m^2, respectively. All values were derived reasonably based on the dimensions of the structures and respective unit weights of their elements. It was noted that the magnitude of villa loads acting on the surface of the model was likely to be small since much of this load would have been resisted by the supporting piles and hence transferred to the bedrock.

Table 2. Input soil parameters in the analysis.

	Geotechnical parameters from lab tests/correlations				
Depth (m)	Dry density (kN/m^3)	Friction angle (degrees)	Initial void ratio	Elastic modulus (kN/m^2)	Permeability (m/s)
0.0–3.0	14.00	30	0.89	5000	8.00×10^{-5}
3.0–5.0	17.00	34	0.56	16,000	3.00×10^{-5}
5.0–6.0	14.67	31	0.81	8000	6.00×10^{-5}
6.0–9.0	16.50	33	0.61	15,000	5.00×10^{-5}
9.0–13.0	17.60	35	0.51	18,000	8.00×10^{-6}
13.0–15.0	20.00	38	0.33	20,000	4.00×10^{-6}

Meshing details. All soil layers were fine-meshed using tetrahedral elements with nodes connecting automatically across elements in the adjacent solids. This ensured appropriate nodal connectivity in the whole model (Figure 7). Refinement of mesh was carried out using several trials and no further refinement was done once no significant change was noticed in results with further decrease in mesh size.

Drip irrigation simulation. Based on information obtained from the landscape irrigation companies involved, various infiltration parameters for defined areas were assessed and for input into the programme, where specifically:

(a) the input flow rate was determined to be 13 l/m^2/day (i.e. litres per square metre per day)
(b) the 13 l/m^2/day flow rate was applied in two identical 30 min cycles per a day, i.e. cycle 1 at 6.5 l/m^2 in the morning and cycle 2 at 6.5 l/m^2 in the evening. There was no irrigation in between the two cycles in any day.

In the programme, the consequent transient flow from the irrigation process was modelled using the 'seepage boundary' function (Figure 8), which required assigning a value of flow rate per unit area of a defined flux surface (soft landscaped areas in the current model) of perpendicular water entry into the uppermost stratum considered.

Boundary conditions of model. In order to simulate the real situation in the field, appropriate boundary conditions of the mesh sets were defined by constraining displacements in: (i) the x direction for both the left and right faces of the geometry model, (ii) the y direction for both the front and back faces of the model, (iii) both the x and y directions for the bottom boundary of the model. Thus displacements were permitted in the z direction only, so that the calculated soil surface deformation would be interpreted as either settlement or heave.

Now, although in reality the infiltration through the soil would potentially be three directional, since the ground

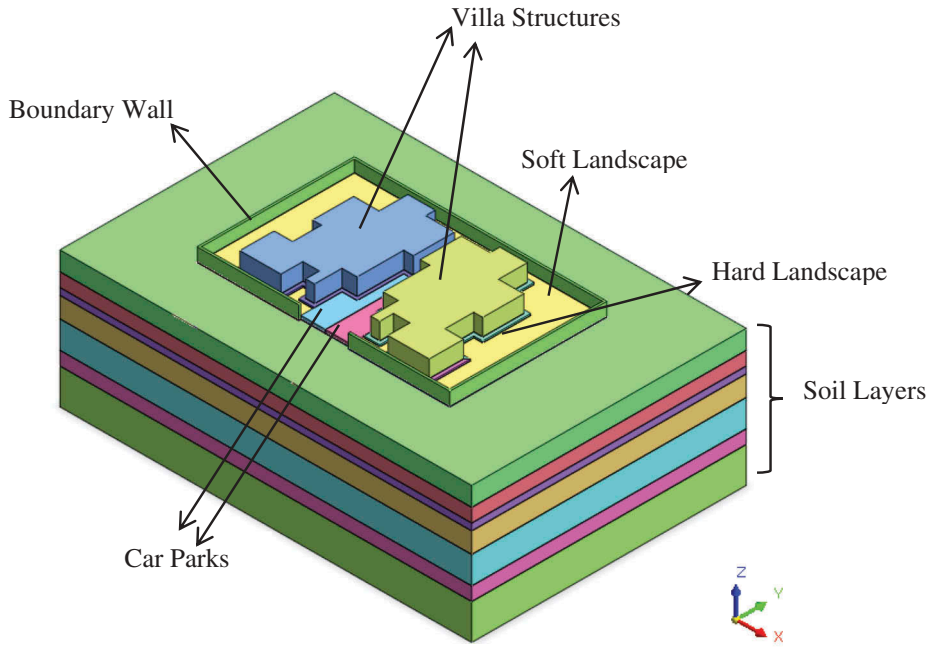

Figure 6. Geometric model of the twin-villa complex and underlying strata.

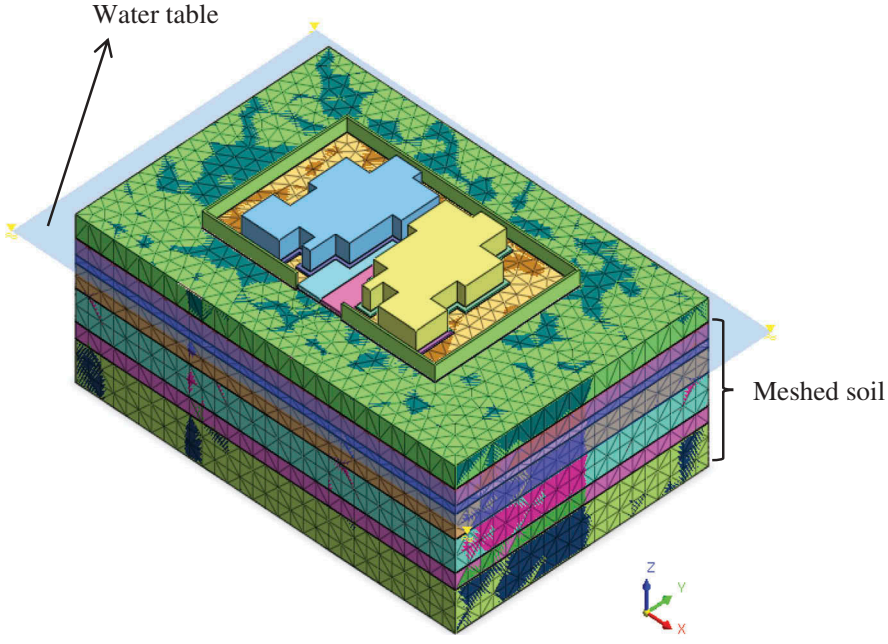

Figure 7. Meshed model incorporating soil profile and supported structures.

surface at the actual site is reasonably flat, the flow would be predominantly along the gravity direction. Hence, to simulate this, the bottom face of the model was selected as a review boundary (Figure 9), in order to enable customisation of seepage direction with respect to boundary surface considered (e.g. flow in a defined direction perpendicular to a specified plane).

Since the native soils at the UAE site analysed were principally free draining and dry silty sands, it was reasonable to set the total head as zero for all the 29 boundaries (4 sides of the model times 7 stratum faces per side plus the bottom face) as seen in Figure 10. This guaranteed zero excess pore water pressure associated with loading.

Analysis methodology. For the model to closely represent reality, the analysis was carried out in a staged construction sequence as follows: (i) stage one equivalent to the *in-situ* conditions and accounts for the weights of the soil layers, (ii) stage

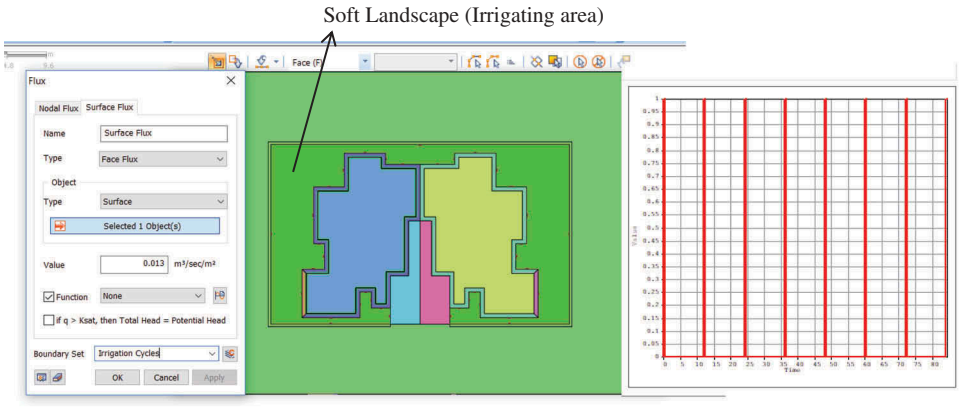

Figure 8. Seepage boundary conditions of the model (mesh un-selected for clear view).

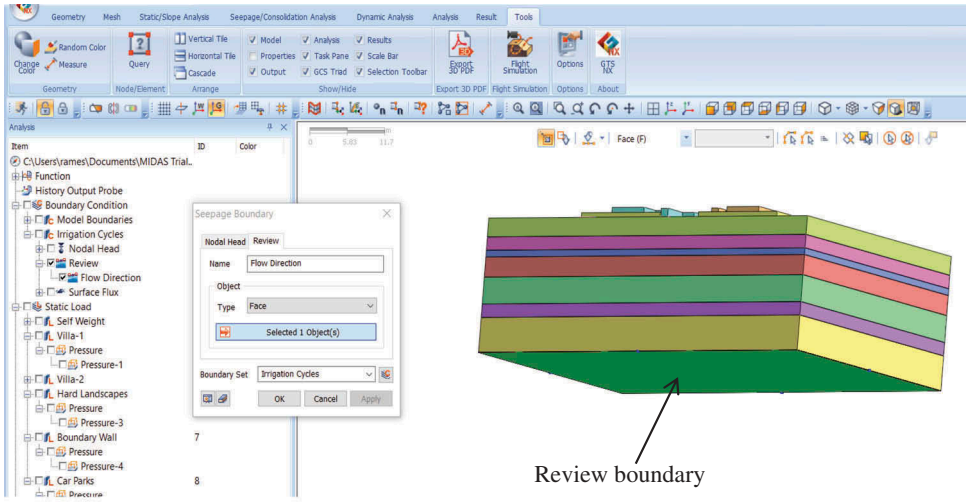

Figure 9. Direction simulation of seepage in the model (mesh un-selected for clear view).

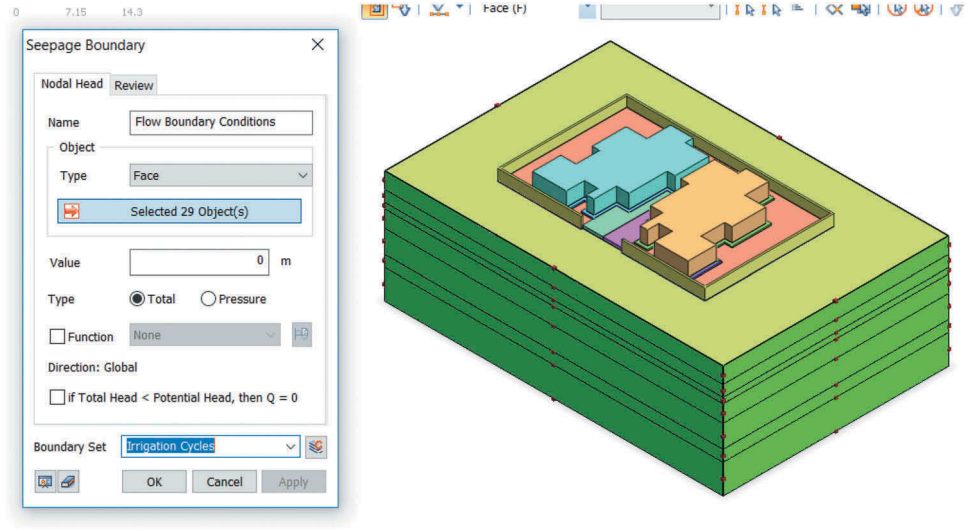

Figure 10. Seepage boundary conditions of the model (mesh un-selected for clear view).

two represents installation of the villas and all other structures including boundary walls, hard landscapes etc. and (iii) stage three simulating the cycles of transient irrigation water flow.

In order to determine the soil deformations associated exclusively with the transient drip irrigation, ground settlements caused by soil self-weights and structures were nullified from the model using the 'clear displacement' option (Figure 11). Finally, ground settlements were monitored at the end of every irrigation cycle or until there was either (a) no further settlement change or (b) the solution started to diverge, for the set convergence criteria, for the subsequent irrigation cycle.

Results and discussion. From the software calculation results, the ground settlement beneath the boundary walls at three different water depths, viz. 1.5, 2.0 and 3.0 m were summarised. Figure 12 maps out a specimen result of magnitudes of ground settlement beneath a boundary wall at the end of the 17th irrigation cycle, which corresponds to a water table depth of 1.5 m. Figure 13 shows the calculated trends of variation of settlement beneath boundary wall versus number of irrigation cycles, for three particular water table levels. It is evident that the number of irrigation cycles required for the supporting ground to exhibit total collapse increases with increasing water table depth. The observed suddenness of bearing capacity loss, coupled with strong sensitivity to water table position, is an indication of the presence of collapsible layer(s) in the soil profile. Vandanapu, Omer, and Attom (2017) observed a similar trend from laboratory tests on a collapsible soil sandwiched between two other layers and loaded under different water table levels and infiltration rates. Figure 13 also reveals that, after sufficient wetting in 4–5 irrigation cycles, the ground surface settlement at the end of a given irrigation cycle increased with increasing water table depth. This evidences that once the collapsible stratum had been saturated sufficiently to fail with the ground water table at a certain depth, there was very little additional settlement with increasing water table depth due to the relatively less sensitivity of the non-collapsible layers to water table rise. It is interesting to note that the calculated maximum settlement beneath the boundary wall was 157 mm, which compares favourably with the measured value of 165 mm on site. This gave confidence that the 3D FE model and the assessed parameters are reliable and consistent with the real ground behaviour.

Structural modelling of boundary walls

The forensic geotechnical investigations at the site in UAE showed that the boundary walls around the villas suffered the greatest deformation as a result of irrigation-induced settlement of the collapsible strata. As seen in Figure 3, as the soil beneath the boundary walls settled, the top surface of the wall remained unaffected and horizontal. Furthermore there was no evidence of the entire wall sagging as a unit. Instead, extreme movements occurred along the masonry bedding joints at 300–400 mm above the ground. It would have been expected that the wall would deform in a different pattern since both of its ends were supported on the settling soil. Hence, to examine how the observed failure mechanism was possible, further analysis was undertaken using a separate non-linear structure analysis module of *Midas*^{TM} FE programme.

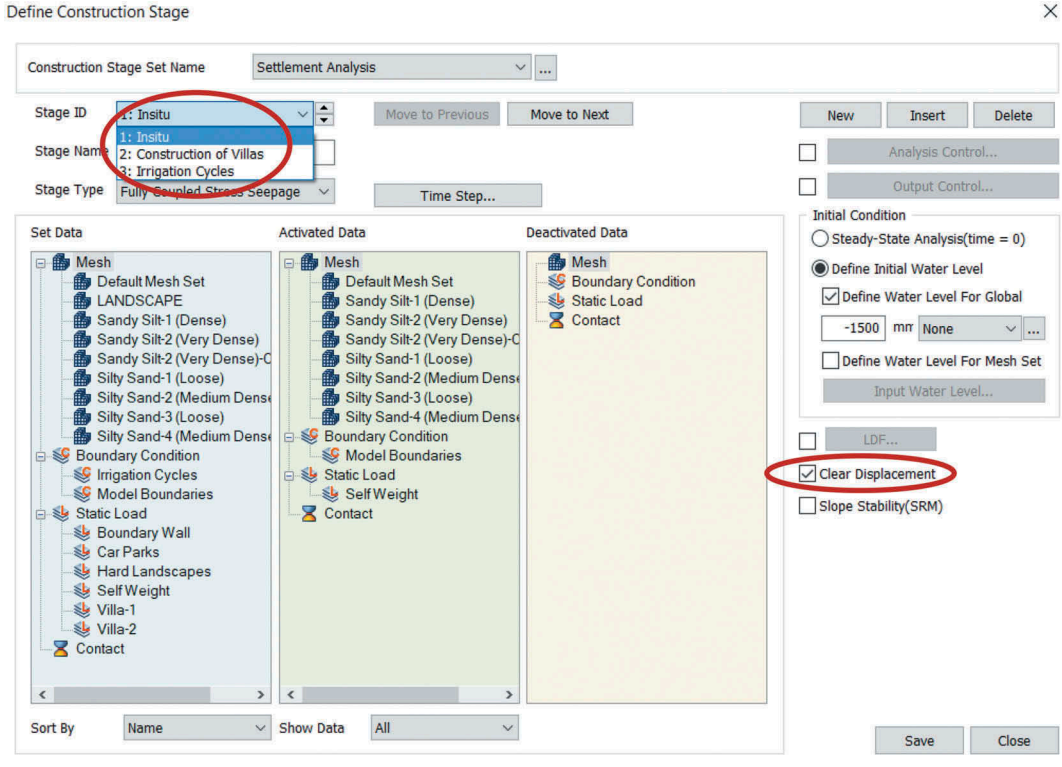

Figure 11. Staged construction sequences in the analysis.

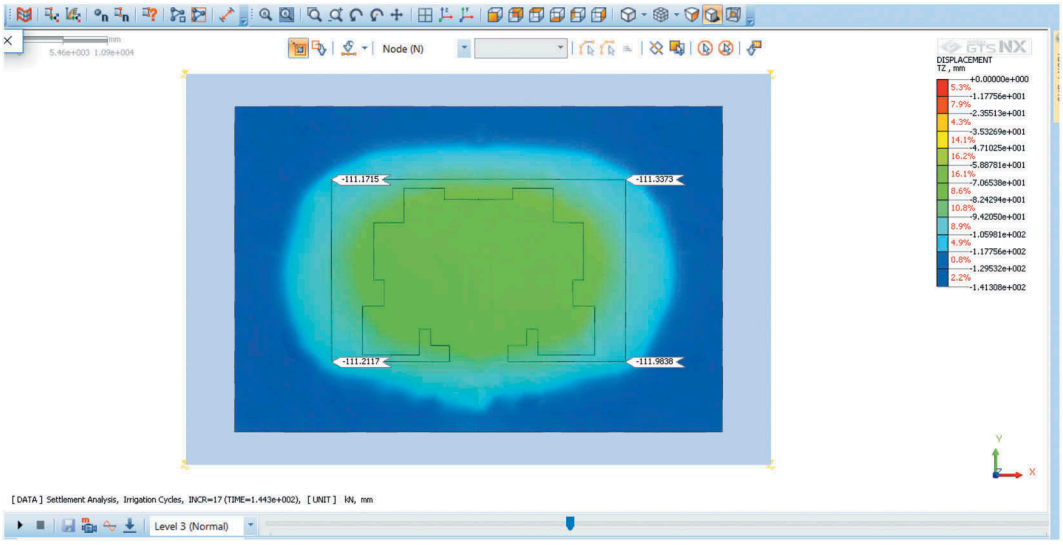

Figure 12. Settlement of soil under boundary wall at the end of 17th irrigation cycle with ground water at 1.5 m depth.

Technical details of modelling

A 2D FE analysis of boundary wall of actual size (6.0 m length and 2.4 m height) on site was carried out in the software using quadrilateral mesh elements of 50 mm in size. The size of the mesh was decided based on different trials. Initially a coarser mesh was analysed and made finer after each trial. Once no further significant change in results was noticed even after refining the mesh, mesh size was finalised and no further trials carried out. All vertical joints in the brick masonry were modelled as staggered in position such that no two vertical joints in consecutive courses will join. All mortar joints were modelled as interface elements and discrete cracking approach was used. Constraints on both end of the wall were taken as 'pinned' with three degrees of freedom in translation along all axes. Non-linear static analysis was performed with material and geometric nonlinearities. The entire self-weight of the wall was imposed as load in 20 equal steps and maximum number of iterations per load step was limited to 30. Newton Raphson iteration scheme was used and convergence criterion of the analysis was based on 'energy norm'.

Modelling parameters

Various parameters used in the analysis are shown in Table 3.

Understanding and analysis methodology

It was known that the boundary walls were directly supported on strip foundations bearing on the ground that started settling when the collapsible stratum lost its structural strength under the influence of seepage from surface irrigation. However, the observed deformation pattern indicated of the boundary wall, where the ends remained intact as the lowermost masonry courses sheared off, indicated that the wall ends were well tied and that self-supporting or interlocking mechanisms prevailed across most of the masonry courses. Also, in reality the entire soil underneath the boundary wall would neither commence settlement at the same time nor have a uniform settlement rate. Hence, in the first part of the analysis a hypothetical situation was assumed where the complete wall lost support due to settlement of the supporting soil below.

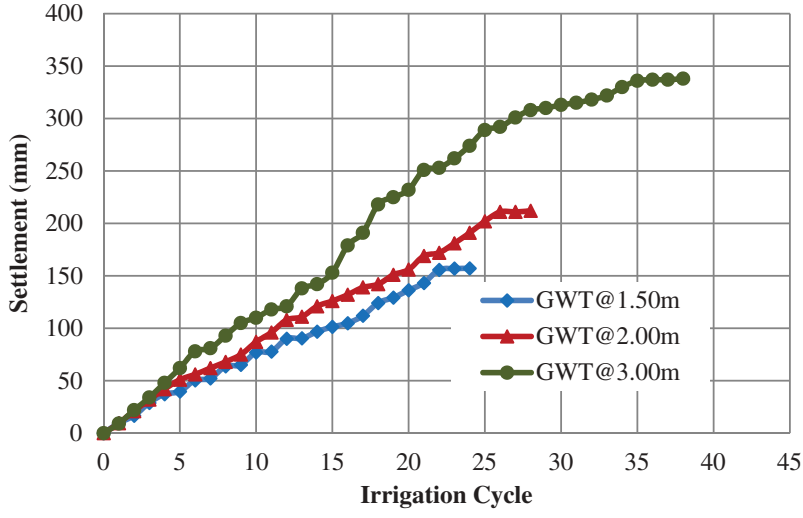

Figure 13. Settlement versus irrigation cycles at various depths of groundwater table.

Table 3. Input parameters for the soil-structure interaction analysis of the boundary wall.

Material	Parameter	Unit	Value/Description
Brick	Material	–	Cement concrete
	Size (length × height × width)	mm	400 × 200 × 200
	Elastic modulus	N/mm^2	16,700
	Weight density	kN/m^3	21.6
Mortar	Material	–	Cement mortar (1:6)
	Compressive strength	N/mm^2	7.5
	Thickness	mm	10
	Tensile strength	N/mm^2	0.15
Interface properties	Normal stiffness modulus	N/mm^3	14
	Shear stiffness modulus	N/mm^3	62

Therefore, to improve the calculation results, a further analysis was carried out properly considering soil-structure interaction influences. The interaction meant that, as the soil support was gradually lost below the wall base, stresses within the wall were redistributed such that more load was transferred to the end ties, with the wall increasingly mobilising its own self-supporting capability until the mortar joints failed. These mechanisms were modelled using a non-linear structure analysis module of *Midas*^{TM} by specifying input values of incremental wall self-weights and performing calculations to monitor the consequent load transfer and deformation response of the wall. In the analysis, the wall end constraint conditions were defined as 'pinned' before imposing self-weights in 20 equal steps, each equivalent to 5% of the actual weight of the wall.

Results and discussion

Figure 14 shows the calculated maximum wall settlements corresponding to various increments of percentage self-weight. It can be seen that the graph is bi-linear, with the wall settlement initially increasing at a marginal rate but once the percentage self-weight reached 35%, there the wall settlement increased suddenly from 0.7 to 15.65 mm. This is equivalent to a 22 times increase in settlement for a 5% increase in applied weight from 35% to 40%. Figure 15 shows the output deformation pattern of the wall at 40% weight increment corresponding to the drastic settlement increase. Essentially the wall had failed at this stage because of continuous divergence of subsequent calculation solutions and unrealistic settlement outputs producing incompatible failure patterns.

It can be seen that the predicted failure patterns of the wall (Figure 15) are similar to the site observations (Figure 3), where failure of mortar bedding joints caused complete dislocation of the lower masonry courses while other parts of the wall remained largely intact. The close agreement between the measured and predicted mechanisms gave confidence that the suggested FE analysis approach and parameter values used in *Midas*^{TM} are consistent with reality. Unsurprisingly, the structural distress was not due to rigid settlement of the wall as a unit but rather failure of the mortar joints in response to extreme settlements and redistribution of stresses in the wall and its ties.

Conclusions

Numerical analysis of ground settlement and structural distress has been successfully carried out using data from a case study in Abu Dhabi (UAE). At the site considered, various shallowly founded structures including boundary walls, roads and hard landscapes had suffered considerable deformation due to infiltration from irrigation water which saturated underlying collapsible strata sufficiently to lose inter-particle

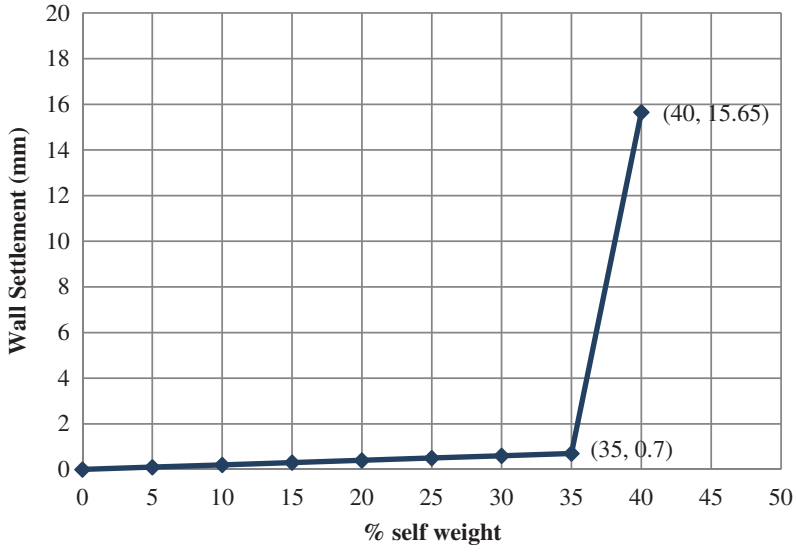

Figure 14. Wall settlements at various percentage of self-weights.

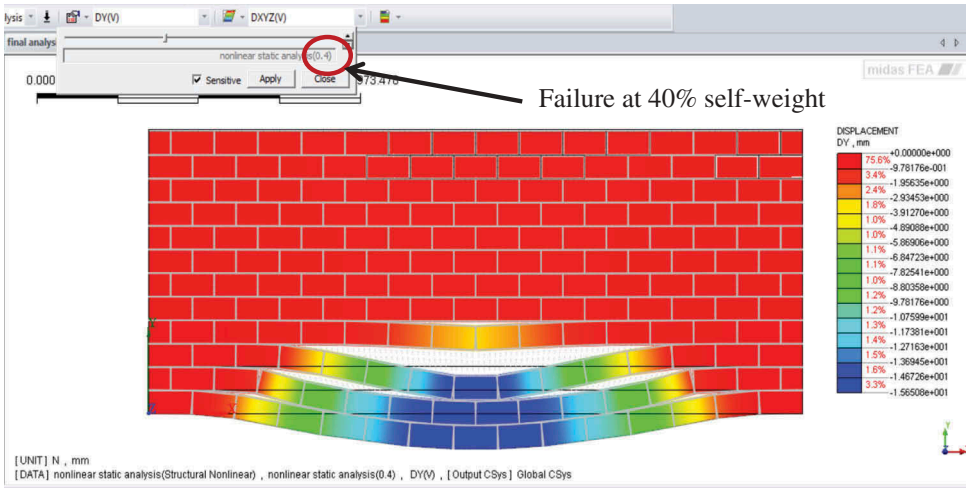

Figure 15. Failure pattern of wall at 40% self-weight.

strength hence subside significantly. The analysis involved 3D FE representation of the ground profile, supported structures and transient inflow of irrigation water to raise the water table above the collapsible strata. Complexity of the mechanisms of collapsible soils coupled with limited literature on settlement necessitated the use of the latest powerful and research oriented software which *Midas*^{TM} *GTS NX* offered. With careful interpretation of the site investigation and landscape irrigation specifications from the case study, the programme was used to analyse the ground settlements under sustained cycles of irrigation. The computed settlements were found to be in close agreement with the measured ones at specific positions on the site. Computation results showed that the sudden loss of strength of the collapsible layer required the water table to reach a certain depth, which corresponded to a certain number of irrigation cycles. Further increase of water table depth would have increasingly less impact on settlement since the collapsible layer would have already lost its full inter-particle strength.

Additionally, boundary walls were separately modelled using the non-linear structural analysis module of *Midas*^{TM} software. This was in order to examine why the walls failed in the patterns observed at the sites of the case study. It was shown that not only was the predicted failure mode consistent with the actual site observation but also the magnitudes of the calculated and measured maximum settlements were very close. Since the failure of the walls was due to loss of mortar joint strength, the distress witnessed might have been avoided or lessened had the walls been constructed with either (a) lightweight masonry unit materials, or (b) a supporting ground beam resting on deep foundations, comparable to the foundation system of the villas that were unaffected by the superficial soil collapse.

With the discernibly accurate results obtained, the proposed 3D FE approach has demonstrated capability to simulate the behaviour of the real ground and this success provides an alternative and superior solution to empiricism based on laboratory or field tests. The current study forms part of an on-going doctoral research work aimed contributing new understanding of the settlement behaviour of collapsible desert soils underlying irrigated landscapes. It is hoped that further solutions will be developed to assist engineers safeguard infrastructure and prevent the kind of distresses witnessed in the UAE case study area.

Disclosure statement

No potential conflict of interest was reported by the authors.

Notes on contributors

Ramesh Vandanapu is a doctoral researcher in Geotechnical Engineering at Kingston University, London and Assistant Professor in Civil Engineering at Amity University, Dubai, United Arab Emirates. His research area is Geotechnical Engineering mainly forensic geotechnical engineering and self-designed laboratory methods of soil testing.

Joshua Omer is a Senior Lecturer in Civil Engineering at Kingston University, London. His research area is Geotechnical Engineering particularly Soil Mechanics, Deep Foundations, Highway Pavement Design. He is a former Fellow of the Royal Society, UK, and is currently UK representative to two Technical Committees of the International Society of Soil Mechanics and Geotechnical Engineering ISSMGE.

Mousa Attom is a Professor in Civil Engineering at American University of Sharjah, Sharjah, United Arab Emirates. He has more than 50 publications in reputed journals and conferences. His research interests are soil behaviour, soil stabilization and soil erosion.

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Vandanapu, R., J. R. Omer, and M. F. Attom. 2017. "Laboratory Simulation of Irrigation-Induced Settlement of Collapsible Desert

Soils under Constant Surcharge." *Geotechnical and Geological Engineering* 35 (6): 2827–2840. doi:10.1007/s10706-017-0282-0.

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MAGAZINE PUBLICATION

Laboratory Simulation of Irrigation-Induced Settlement of Collapsible Desert Soils under Constant Surcharge

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Abstract: The heterogeneous nature of soil as a load bearing material, coupled with varying environmental conditions, pose challenges to geotechnical engineers in their quest to characterize and understand ground behavior for safe design of structures. Standard procedures for checking bearing capacity and settlement alone may sometimes be insufficient to achieve an acceptable degree of durability and in-service performance of a structure, particularly under varying environmental conditions, whether natural or man-made. There exists a wide variety of problematic soils that exhibit swelling, shrinkage dispersion and collapse characteristics occasioned by changes in moisture content. Specific examples are collapsible soils, which occur mainly in arid and semi-arid regions, are generally capable of resisting fairly large loads in the dry condition but suffer instability and significant strength loss when in contact with water. A number of case studies in the United Arab Emirates (UAE) were examined, where lightly loaded structures such as boundary walls, pavements and footpaths had been built on ground overlying collapsible soil strata. Sustained irrigation of the dry landscapes was found to have caused uneven settlement of the collapsible soils leading to continuous distress to the structures as evident from cracking and deformation. To help address the problem, an opportunity has been taken to develop a laboratory method of simulating the loaded behavior of collapsible soils in varying situations and to measure its deformation at constant surcharge and ground water infiltration rates. Finally, relationships were developed for linking the time period for maximum settlement to thickness of collapsible soil as well as magnitude of settlement to thickness of collapsible layer. These relationships can be used by geotechnical engineers to assess the rate and magnitude of settlements, depending on the thickness of the collapsible soil at a particular site. Though every effort has been made in the current study to prepare sufficiently large sized models to simulate field conditions relevant to the UAE case studies, inevitably there will be variations to be taken into account from one site to another. These variations include: the rate and frequency of irrigation, thickness of collapsible soil stratum and its depth below ground level as well as depth of groundwater table. Thus, geotechnical engineers need to exercise utmost care when assessing the important parameters such as time, rate and magnitude of collapse settlements in the particular locality of concern. A reliable assessment of the relationship between the intensity of landscape irrigation, water table level, thickness and location of collapsible strata can enable UAE Geotechnical engineers to develop guidance for property owners / members of the public to help them control rates of irrigation hence avoid extreme ground settlement that would cause structural distresses.

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AWARD RECEIVED

I successfully took part in a competition organized by the Association of British Turkish Academics (ABTA), London. ABTA awarded me an honourable Mention Award under the **Doctoral Researcher Awards -2018**. Details of the award can be seen in the following link:

http://abtanet.org.uk/Awards/Detail/4/2018-ABTA-Doctoral-Researcher-Awards

DRA Honourable Mention Award

ABTA <info@abtanet.org.uk>

Tue 01/05/2018 16:03

Inbox

To: Vandanapu, Ramesh <k1452539@kingston.ac.uk>;

Dear Ramesh,

We are pleased to inform you that your application for the 2018 ABTA Doctoral Researcher Awards competition has been evaluated to receive an Honourable Mention Award in your category. Please note that you must attend the ceremony, which will take place at University College London (UCL) on Saturday, 12 May 2018, in order to receive your award (Amazon Voucher, £25) and certificate. Unfortunately, we are unable to cover or reimburse travel expenses.

Please let us know by 5 pm Friday, 4 May, whether or not you will attend the awards ceremony. If for some reason you cannot make it let us know as soon as possible.

Please see the tentative programme for the ceremony day below. Note that there will be a poster session and all award recipients are expected to present a poster (A1 size, portrait).

Should you have any questions, please do not hesitate to contact us. We look forward to hearing from you.

Tentative Programme:
09.30 – 10.00 Registration
10.00 – 10.15 Welcome Speeches
10.15 – 12.00 Presentations
12.00 – 13.30 Reception and Poster Session
13.30 – 15.00 ~ Awards Ceremony ~

Venue: Darwin Lecture Theatre
University College London (UCL), Gower St, London WC1E 6BT
Date: Saturday, 12 May 2018

Best regards,
ABTA Team.

2018 ABTA Doctoral Researcher Awards

The grand prix is over. On May 12th ABTA held the 7th UK Doctoral Researcher Awards ceremony in the junior academics' Hall of Fame: UCL Darwin Hall. 15 finalists and

The program began by an opening from the host Dr Yuge Ma, a fellow of Oxford University, who was one of the recpients of DRA awards in the 2015 competition. In the morning session the finalists showed outstanding performance in presenting their PhD research which drew infinite number of questions from the audence (which were skilffully reduced to finite numbers by the program host).

After the morning presentations, second part of the program began where the knowledge exchange continued in the form of Q&A in the poster hall.

In the afternoon the excitement in the crowd made a peak as everyone turned into ears to hear the winners:

Natural & Life Sciences

1 - Sara Priego Moreno, University of Birmingham

2 - Gavin Rutledge, University of Cambridge

3 - Jason Pottcary, University of Bristol

4=5 - Valerio Fasono, University of Manchester; Kayn Forbes, East Anglia University

Social & Management Sciences

1 - Amin El Yousfi, University of Cambridge

2 - Kathryn Medien, University of Warwick

3 - Ozlem Eylem, Queen Mary University of London

4=5 - Abrar Chaudry, University of Oxford; Kasturi Hazarika, University of York

Engineering Sciences

1 - Ishara Dharmasena, University of Surrey

2 - Maria Pregnolato, Newcastle University AND Antonio del Rio Chanona, University of Cambridge

4=5 - Tian Tian, University of Cambridge; Yiou Wang, University College London

Honorable Mentions

Natural and Life Science

Mina Bergstad - University of Manchester
Katherine Ellis - Aston University
Leen Kalash - University of Cambridge
Diogo Mosqueira - University of Nottingham
Laura Prichard - University of Oxford
Sourav Sahoo - University of Southampton
Adam Sedgwick - University of Bath

Management and Social Sciences

Ines Ferreira - University of East Anglia
Rahul Jalil - Birmingham City University

Engineering Sciences

Adeayo Sotayo - Lancaster University
Benjamin Tam - University College London
Jaime Gaspar - University of Kent
Khalid Hashim - Liverpool John Moores University
Pinelopi Kyvelou - Imperial College London
Ramesh Vandanapu - Kingston University London
Tu Bui - University of Surrey
Yang Zheng - University of Oxford
Craig Buchanan – Imperial College London
Daryus Chandra - University of Southampton

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Multimedia

FUNDINGS RECEIVED FROM KINGSTON UNIVERSITY

- I successfully bid for
  - £160 funding to present the poster at an International Conference which was held on 13-14, March 2017 in Dubai, UAE.
  - £300 to participate in intensive training in MIDAS GTS NX software from 3rd Dec, 2017 until 7th Dec, 2017 in Dubai at MIDAS IT office.
  - £500 as completion bursary of my doctoral studies (Ph.D)

Conference funding application: CAES17

Percival, Rosalind F

Wed 01/03/2017 16:30

To: Vandanapu, Ramesh <k1452539@kingston.ac.uk>;

Cc: Omer, Joshua R <J.R.Omer@kingston.ac.uk>;

1 attachments (51 KB)

new - Student Payment Request Form - March 15.docx;

Dear Ramesh

I am pleased to let you know that your application for funding has been successful and that you have been awarded £160 as a contribution towards your expenses in attending the above conference.

As a condition of the funding, after your return you are required to give a presentation via Skype on the conference you attended, at a School seminar – this should be arranged via your first supervisor or the School's PGR Director.

To claim this contribution, please complete the attached expenses claim form – including itemising the amounts being claimed and signing where indicated in red text - and submit it to me together with your receipts and your conference report.
Please note that payment cannot be made without receipts – for on-line bookings please provide a print-out of the payment confirmation.

Please also note that reimbursement will not be given for any costs you pay (for example, registration fee, obtaining a visa for the event) if you do not do attend or your application is not approved.

Best wishes
Rosalind
Research Student Co-ordinator

Training funding application: MIDAS GTS NX, December 2017

Percival, Rosalind F

Wed 13/12/2017 12:43

To: Vandanapu, Ramesh <k1452539@kingston.ac.uk>;

Cc: Omer, Joshua R <J.R.Omer@kingston.ac.uk>;

1 attachments (51 KB)

new - Student Payment Request Form - March 15.docx;

Dear Ramesh

I am pleased to let you know that your application for funding has been successful and that you have been awarded £300 as a contribution towards your expenses in attending the above training.

As a condition of the funding, you are required to give a presentation on the training (either in person or via Skype), the content and timing of which you should discuss and agree with Dr Omer, who will subsequently need to confirm to me that it has taken place. Payment of your expenses claim can't be made without this confirmation.

To claim this contribution, please complete the sections in red on the attached Student Payment Request Form, including itemising the amounts being claimed for fee/travel/subsistence, and return it to me together with your receipts.
Please note that payment cannot be made without receipts – for on-line bookings please provide a print-out of the payment confirmation.

Best wishes
Rosalind
Research Student Co-ordinator

Reply all | Delete Junk | •••

Re: Completion bursary

Vandanapu, Ramesh
Wed 10/10, 17:22
Percival, Rosalind F

Reply all |

Sent Items

Dear Madam,
Thanks a lot for your email.
Very happy to know about success regarding the completion bursary.
Regards,
Ramesh

Get Outlook for iOS

From: Percival, Rosalind F <r.percival@kingston.ac.uk>
Sent: Wednesday, October 10, 2018 2:59 PM
To: Vandanapu, Ramesh
Cc: Omer, Joshua R
Subject: Completion bursary

Dear Ramesh

With reference to your recent completion bursary application, I am pleased to let you know that you have been awarded £500, subject to meeting the thesis submission deadline stated in your application

Your thesis (2 copies) and accompanying paperwork (RD12a and RD12b forms) must be submitted to me by the deadline stated in your application: 30 November 2018.

If you provide evidence of acceptance of paper 1 (email confirming acceptance and an electronic copy of the submitted paper), you will be awarded an additional £500.

Payment is dependent on meeting this deadline. A request for a short extension may be requested, via email tosecresearch@kingston.ac.uk , but approval is not guaranteed.

Best wishes
Rosalind
Research Student Co-ordinator