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SCHOOL OF ENGINEERING

Effect of non-homogeneity on consolidation behaviour of clay

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Table of Contents

Chapter 1: Introduction

Consolidation describes the process of volume reduction that occurs in a saturated soil stratum under an application of stress. The magnitude of consolidation can be predicted using many different methods (Carillo 1942, Barron 1948, Biot 1955 and Gibson et al. 1967), which utilize Terzaghi's classical one-dimensional consolidation theory. Here, soils are tested using an oedometer to determine their compression index through which the degree of consolidation can be estimated. Accurate methods for estimating the consolidation settlement of dredged clays and slurries after placement as fills in reclamation areas are imperative for the success of future infrastructure development.

Terzaghi's one-dimensional consolidation theory is dependent upon a number of fundamental assumptions, the foremost of which is soil homogeneity. Whilst this may be a valid assumption in some physical cases, the composition of a consolidating dredged mud or slurry is far from homogeneous. Due to the settling process of these materials upon placement as fills, coarser particles settle toward the base of the soil layer, whilst finer particles remain near the surface. Thus, the permeability of the soil system can expect to increase with depth. The coefficient of consolidation is a function of permeability and volume compressibility, but is constant according to Terzaghi's consolidation theory. As a result, consolidation estimations of dredged mud using this method are not realistic and can vary greatly from actual values.

The effect of non-homogeneity has been previously studied by Davis and Raymond (1965), Gibson et al. (1967), Mesri and Rokhsar (1974), Mesri and Tavenas (1983), Mesri and Choi (1985), Li et al. (1999a, 1999b), Xie et al. (2002) and Zhuang et al. (2005). However, the effect of varying permeability and volume compressibility with void ratio and effective stress was primarily investigated. This project proposes to extend Terzaghi's classic one-dimensional consolidation theory to include functions of permeability and volume compressibility that linearly and exponentially vary with depth. Although similar to the investigation conducted by Gibson et al. (1967), this project will utilise a different method of analysis - series solution methods will be used in conjunction with the program MATLAB to generate results.

All previous investigations of the effects of non-homogeneity involve constant initial distributions of excess pore pressure. Thus, the possibility of varying initial distributions of pore pressure to measure the corresponding effect on consolidation behaviour is another potential avenue of study. General assumptions of instantaneous loading can also be extended to include constant rates of loading, the results of which are extremely relevant to the land reclamation industry.

Thus, this investigation will comprise of the following objectives;

- Analysis of consolidation behaviour of clays subjected to asymmetric initial distributions of pore water pressure for doubly and singly drained strata.
- Comparison between consolidation settlement calculated from degree of consolidation, and settlement calculated from volumetric flow through drainage boundaries due to hydraulic gradient.
- Analysis of consolidation behaviour of clays subjected to constant loading, in comparison with standard assumptions of instantaneous loading. The effect of various distributions of initial excess pore pressure will also be investigated.
- The effect of non-homogeneity of clays through variation of permeability and/or volume compressibility as functions of depth.

Chapter 2: Literature Review

2.1 Consolidation Theory

2.1.1. Introduction

The reduction in volume of a soil body due to an application of stress can be attributed to one or more of the following factors; a compression of the solid matter, a compression of water/air within the voids and/or an escape of water and air from the voids. Due to the relative incompressibility of pore water and solid matter, it is reasonable to assume the majority of volume decrease of the soil body is a direct result of water/air expulsion from voids. Furthermore, when conducting analyses of submerged clay strata, complete saturation is usually assumed. This assumption is both valid, as most sedimentary clay deposits are saturated, and useful, as difficulties associated with partially saturated soil analysis can be avoided (Taylor 1962).

2.1.2. Soil Constitutive Models

As soils are loaded or unloaded isotropically or anisotropically, they will contract and swell. If the pressure on a soil body is increased and then later decreased to its previous value, the subsequent volume expansion that takes place will be of a much smaller magnitude than the original compression. Thus, although soils demonstrate some elastic tendency, they are only elastic to a small degree. Settlement analyses therefore ideally require separation of strains into elastic and non-elastic components.

Constitutive model is the technical term used for describing the material behaviour of soils. Elastic and Mohr-Coulomb are the most common constitutive models due to their simplicity. However, the disadvantage of a simplistic constitutive model inherently lies within the assumptions necessary to obtain this simplicity.

Elastic Model

The simplest constitutive model is the linear elastic model, which obeys Hooke's law and does not incorporate yielding. The required material properties are therefore Young's modulus (E) and Poisson's ratio (ν). The material is assumed to be infinitely elastic, with no yielding ever taking place.

Mohr-Coulomb Plasticity Model

Mohr-Coulomb is a plastic model, where the material is assumed to behave elastically until yielding. The Mohr-Coulomb is the most common elasto-plastic constitutive model used to describe soil behaviour and is based upon Hooke's law of linear elasticity (for soil behaviour under loading conditions) and Coulomb's law of perfect plasticity (for soil behaviour under collapse state). The tangent to the Mohr failure circles from various tests at different initial effective stresses is called the Coulomb failure criterion. Thus, the Mohr-Coulomb constitutive model can be expressed as;

$$\tau_f = c' + \sigma_f' \tan\phi' \tag{2.1}$$

where τ_f and σ_f' are the shear and normal effective stresses on the failure plane, and the cohesion, c', and friction angle, ϕ', are the material parameters. Therefore, the two parameters that define the failure criteria are the friction angle and cohesion. In general stress state, the model's stress-strain behaviour is considered linear in the elastic range, where the two defining parameters are Young's modulus, E, and Poisson's ratio, ν. The Mohr-Coulomb model is assumed to be perfectly plastic and as such, does not encapsulate strain hardening/softening effects (Potts and Zdravkovic 1999).

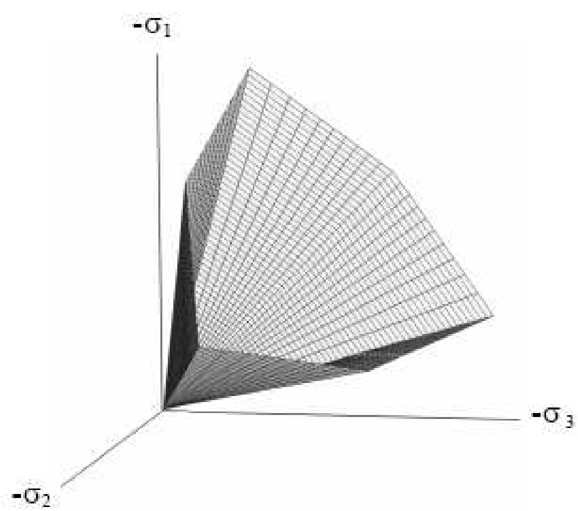

Figure 2.1 – Mohr-Coulomb yield surface in principal stress space, $c = 0$ (Ti et al. 2009)

The Drucker-Prager model is a modification of the Mohr-Coulomb model, where the hexagonal shape of the failure cone (Figure 2.1) is replaced by a simple cone, as demonstrated in Figure 2.3.

Drucker-Prager Plasticity Model

Figure 2.2 shows the yield surfaces as per von Mises and Tresca criteria. When the state of stress is defined by a point within the yield surface, the material behaves elastically. When the point is on the yield surface, the material has reached the yield point and undergoes plastic deformation. They both are elastic perfectly plastic models (Figure 2.2b).

The cross sections of von Mises and Tresca yield surfaces are a circle, and an octagon, respectively. The hydrostatic line, shown by a dashed line through the origin, defines the states where the stress is isotropic. A plane normal to the hydrostatic line is the deviatoric plane.

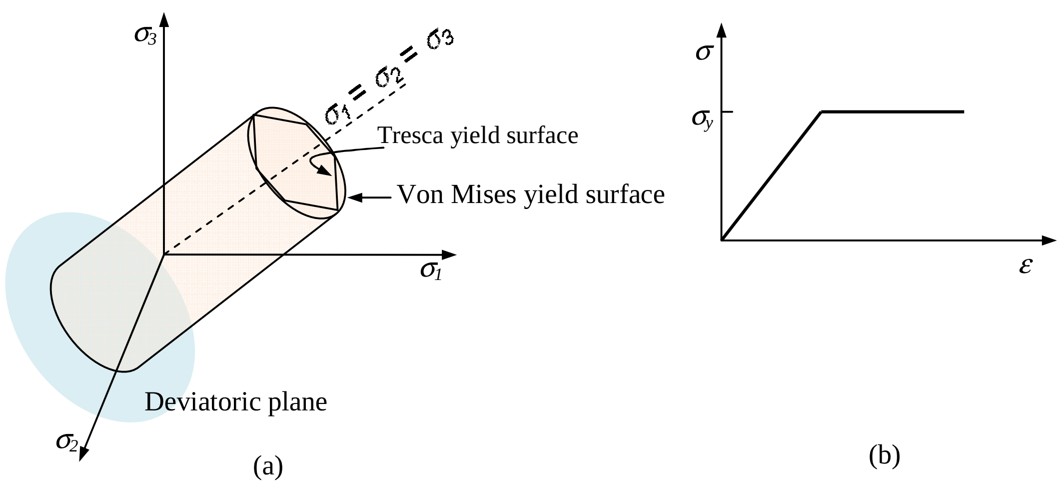

Figure 2.2 Tresca and von Mises yield criteria: (a) Yield surfaces (b) Elastic perfectly plastic model

Mohr-Coulomb and Drucker-Prager yield surfaces are shown in Figure 2.3. When there is no friction, Mohr-Coulomb is similar to Tresca and Drucker-Prager is similar to von Mises failure criteria.

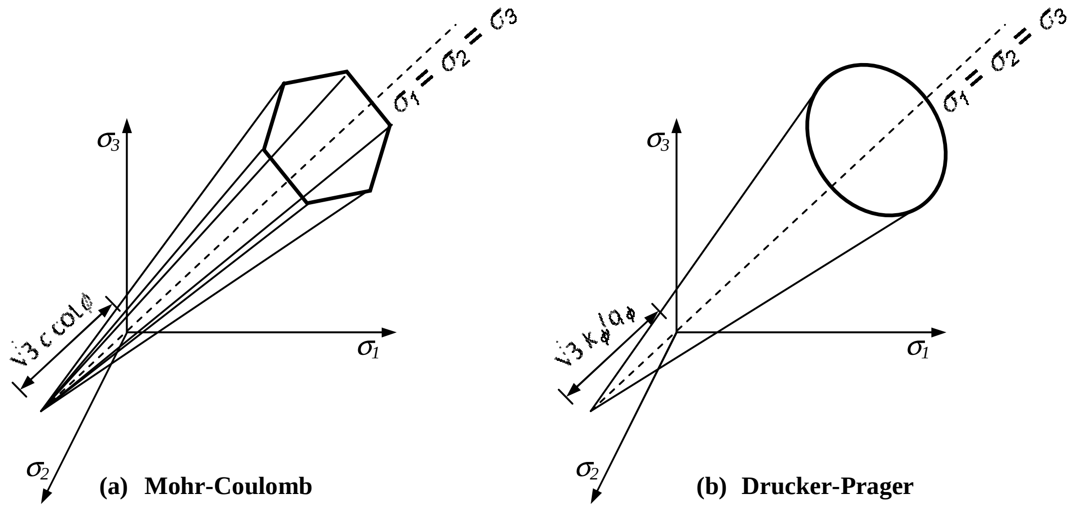

Figure 2.3 Mohr-Coulomb and Drucker Prager Yield Surfaces

Cam clay Model

The more complex Cam clay model endeavours to include the strain hardening/softening phenomenon in the analysis of soil behaviour. This constitutive model utilises the strain hardening theory of plasticity to develop a complete stress-strain model for normally consolidated or slightly over-consolidated clay in triaxial tests. Soil is considered frictional with logarithmic compression, where hardening is related to the plastic volumetric strains (Atkinson 2007). This model involves the following parameters; the isotropic logarithmic compression index, λ, the swelling index, κ, Poisson's ratio for unloading and reloading, ν_{ur}, friction constant, M, pre-consolidation stress, p_c, and the initial void ratio, e.The original Cam clay model was described by Schofield and Wroth (1968), where logarithmic spirals are used to describe the yield curves. The modified version of the Cam-clay model, developed by Roscoe and Burland (1968) uses ellipses to describe the yield curves instead.

2.1.3. One-dimensional Consolidation

Soil compression beneath loads such as buildings is three-dimensional at shallow depths, but can be considered one-dimensional in deeply buried strata. The simultaneous process of water expulsion and gradual compression is known as consolidation. Consequently, the rate of consolidation is a function of permeability and stiffness (see Equation 2.3). According to Taylor (1962), consolidation is a "gradual process involving drainage, compression, and stress transfer."

If a pressure, $\Delta\sigma$, is applied to a clay, the pressure increase in the water filling the voids of the soil will be $\Delta\sigma$ at the instant of application. That is, the pore water pressure (u) is initially equal to the total applied pressure. If consolidation of the soil operates under doubly drained conditions, drainage will proceed from top and bottom layers of the soil body. As time increases, the void ratios decrease, the excess pore water pressure decreases and the effective stress (intergranular pressure) simultaneously increases. This process is always more advanced at the surface and base of the soil, where drainage boundaries are present. Once the excess pore water pressure has reached zero and the pressure in the soil skeleton has increased by $\Delta\sigma$, the sample is said to be consolidated under this magnitude of pressure increase.

A theoretical analysis of one-dimensional consolidation developed by Terzaghi (1925) allows the change in overall thickness of a stratum of consolidating soil to be calculated. However, a number of assumptions are necessary for the applicability of Terzaghi's theory, some of which were introduced above.

Table 2.1 – Terzaghi theory assumptions (Taylor 1962)

	Assumption	Validity
1	Homogeneous soil	Represent assumed conditions that do not considerably vary from actual conditions.
2	Complete saturation	
3	Negligible compressibility of soil grains and water	
4	Action of infinitesimal masses no different from that of larger, representative masses	Academically acceptable and has little bearing from a practical standpoint.
5	One-dimensional compression	Accurately reflected by laboratory tests.
6	One-dimensional flow	
7	Validity of Darcy's Law	Indisputable.
8	Constant values for certain soil properties which actually vary somewhat with pressure	In most instances, these variations are of minor importance.
9	The greatly idealized pressure vs. void ratio relationship	Can considerably limit the validity of Terzaghi's theory.

The validity of assumption nine is limited due to the effects of secondary compression of the soil. Secondary compression occurs at a speed dependent only on the plastic characteristics of the clay – the expulsion of water is no longer a controlling factor. Whilst secondary compression occurs after the conclusion of primary compression, it also occurs during primary consolidation, a phenomenon which is not considered in the Terzaghi theory.

Assumption one, whilst valid for most analyses, does not always accurately reflect soil conditions. In cases where the consolidation behaviour of dredged mud is being studied, it is important to consider the method through which the slurry is deposited. This process (further explained in Chapter 4) often comprises of an initial settling process which generates a layered effect within the consolidating soil – coarse grains settle toward the bottom whilst fine grains remain near the surface of the deposit. The resulting soil stratum is therefore far from homogenous and renders assumption one invalid.

Further advancements have since been made regarding settlement analysis – the two-dimensional consolidation theory with sand drains by Carillo (1942) and Barron (1948), the three-dimensional consolidation model by Biot (1955), and the large-strain model by Gibson et al. (1967) are challenging extensions of Terzaghi's theory. However, despite these advances, Terzaghi's theory still remains widely accepted among practising engineers due to its simplicity and ability to provide accurate estimations of the time rate of consolidation (Sivakugan and Vigneswaran 1991).

In Terzaghi's one-dimensional small strain theory, it is assumed that the additional load is initially carried by the pore water, as the permeability of the soil is very low. As time progresses, the soil skeleton will slowly take over the load from the pore water, resulting in a decrease in pore water pressure and subsequent gain in effective stress. Due to the high water content of slurry-like soils, there may not be a soil skeleton structure capable of taking the transfer of load from the water. As a result, Terzaghi's one-dimensional consolidation theory is not applicable to slurry-like soil in the initial stages of deformation (Bo et al. 1999).

The Terzaghi Consolidation Theory

Analysis of water flow due to head differential through an element during consolidation, and the application of Darcy's law yields Terzaghi's equation for one-dimensional consolidation theory (see Appendix A for detailed proof). The resulting differential equation is analogous of the heat diffusion equation.

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{2.2}$$

where c_v is the coefficient of consolidation, u is the excess pore pressure, z represents the distance measured downward from the surface of the clay sample and t represents time. The thickness of the sample is designated as $2H$, with H being the longest drainage path length if doubly drained.

The coefficient of consolidation (c_v) is a function of properties unique to the soil deposit being analysed and can be expressed as follows;

$$c_v = \frac{k}{m_v \gamma_w} \tag{2.3}$$

where k is the permeability, m_v is the coefficient of volume compressibility and γ_w is the unit weight of water and thus remains constant. In accordance with assumption one (Table 2.1), these properties are assumed to remain constant over the entire depth of the soil deposit, an assumption that is often rendered invalid when considering dredged clays, which are non-homogeneous in nature.

Using the following boundary conditions, a series solution to Equation (2.2) can be specified (Table 2.2).

Table 2.2 – Boundary conditions for the case of one-dimensional consolidation

Boundary Condition	Mathematical Expression
There is complete drainage at the top of the soil layer.	When $z = 0,\ u = 0$
There is complete drainage at the base of the soil layer.	When $z = H, u = 0$
The initial hydrostatic excess pressure (u) is equal to u_i.	When $t = 0, u = u_i$

For a soil deposit that has a drainage path length H, solution of Equation (2.2) gives the excess pore pressure u, at distance z (from the surface of the soil layer) and at time t, from the start of consolidation;

$$u = \sum_{n=1}^{n=\infty} \frac{1}{H} \left(\int_0^{2H} u_i \sin \frac{n\pi z}{2H} dz \right) \left(\sin \frac{n\pi z}{2H} \right) e^{-\left(\frac{n\pi}{2}\right)^2 T} \tag{2.4}$$

$$T = \frac{c_v t}{H^2} \tag{2.5}$$

where T is the time factor, and is dimensionless. Equation (2.4) is applicable for any homogeneous stratum. A detailed outline of the proof required to solve Equation (2.2) is available in Appendix A.

In most cases, the initial hydrostatic excess pressure is assumed to be uniform over the entire depth of the soil deposit. The corresponding excess pore pressure relationship can be derived from Equation (2.4) where u_i equals some constant u_0 (Equation 2.6).

$$u = \sum_{m=0}^{m=\infty} \frac{2u_0}{M} \left(\sin \frac{Mz}{H} \right) e^{-M^2 T} \tag{2.6}$$

$$M = \frac{\pi}{2}(2m+1) \tag{2.7}$$

The proportion of the hydrostatic excess pressure that has dissipated at time t is known as the degree of consolidation at that time and is denoted by U_z.

$$U_z = \frac{u_i - u_t}{u_i} \tag{2.8}$$

$$\therefore U_z = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M} \left(\sin \frac{Mz}{H} \right) e^{-M^2 T} \tag{2.9}$$

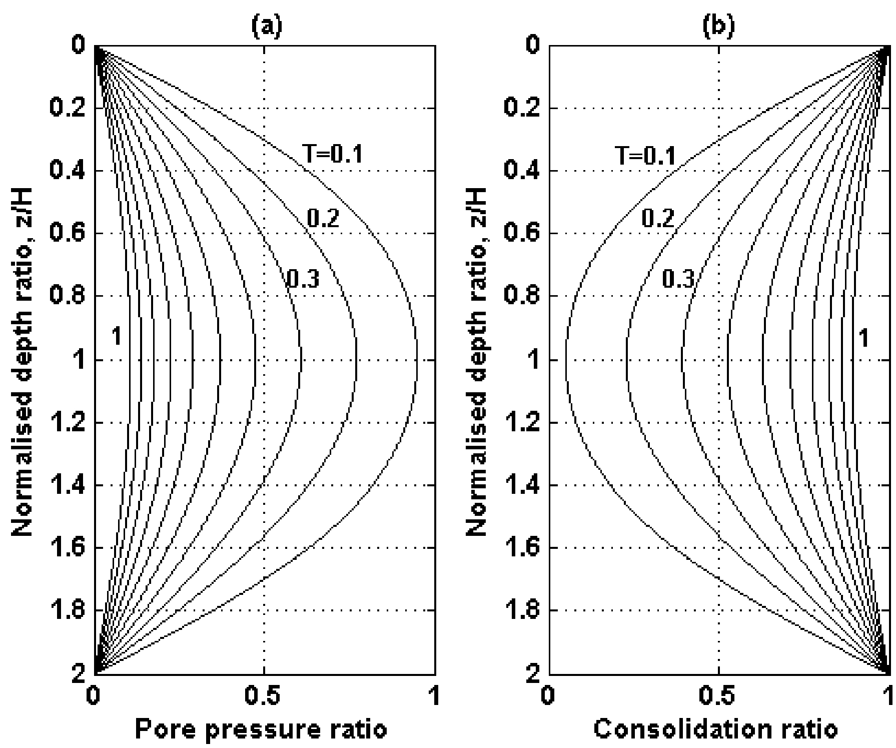

Figure 2.4 – Consolidation as a function of time and depth
(a) pore pressure ratio isochrones and (b) consolidation ratio isochrones

Figure 2.4(b) represents a graph of U_z versus z/H for different time factor values. The resultant curved lines correspond to constant values of time and are referred to as isochrones. The collection of isochrones in Figure 2.4 represents the theoretical process of consolidation – consolidation proceeds most rapidly at the drained boundaries and slowest at the centre of the soil layer. The isochrones in Figure 2.4(a) represent the pore pressure, normalised by initial excess pore pressure value. Due to the constant nature of initial excess pore pressure, the consolidation ratio isochrones are a mirror image of the normalised pore pressure isochrones.

Whilst Figure 2.4 allows the determination of degree of consolidation with respect to time at any depth over the soil stratum, it is often more useful to express the relationship between time and average state of consolidation (U) over the height of the stratum. The average state of consolidation represents the consolidation of the stratum as a whole. When applying the consolidation theory to predict settlements, only the average consolidation is required (Taylor 1962). Since the average consolidation ratio U is the average value of U_z over the depth of the stratum, the following expression can be developed;

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2\int_0^{2H} u_i \sin\frac{Mz}{H} dz}{M\int_0^{2H} u_i dz} e^{-M^2 T} \tag{2.10}$$

A number of standard initial excess pore pressure distributions have been considered in common soil literature (Taylor 1962, Powrie 1997 and Lancellotta 2009) and are shown in Figure 2.5.

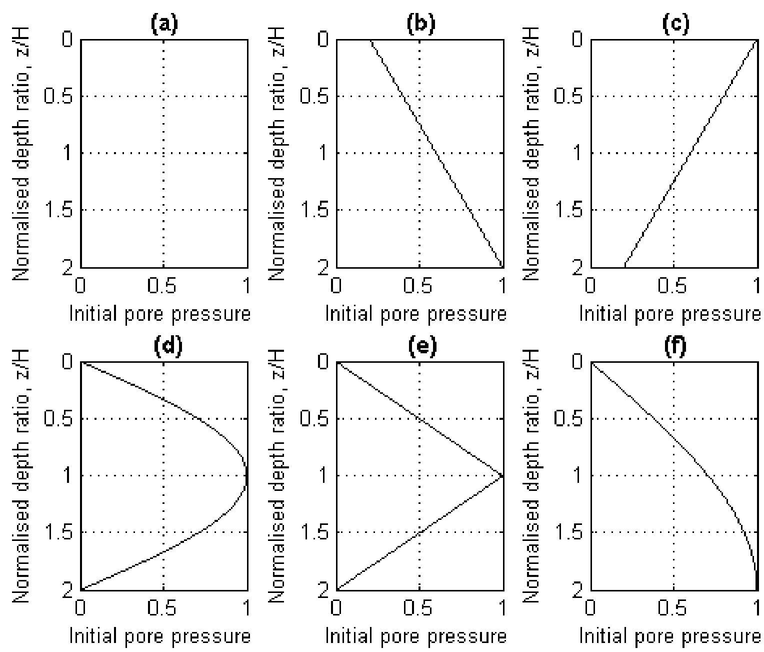

Figure 2.5 – Standard initial excess pore pressure distributions

The application of a constant or linearly varying initial excess pore pressure into Equation (2.10) yields the same expression;

$$U = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M^2} \mathrm{e}^{-M^2 T} \tag{2.11}$$

Therefore, the average consolidation curves generated by initial excess pore pressure distributions in Figure 2.5(a), (b) and (c) can all be represented by the dashed line in Figure 2.6.

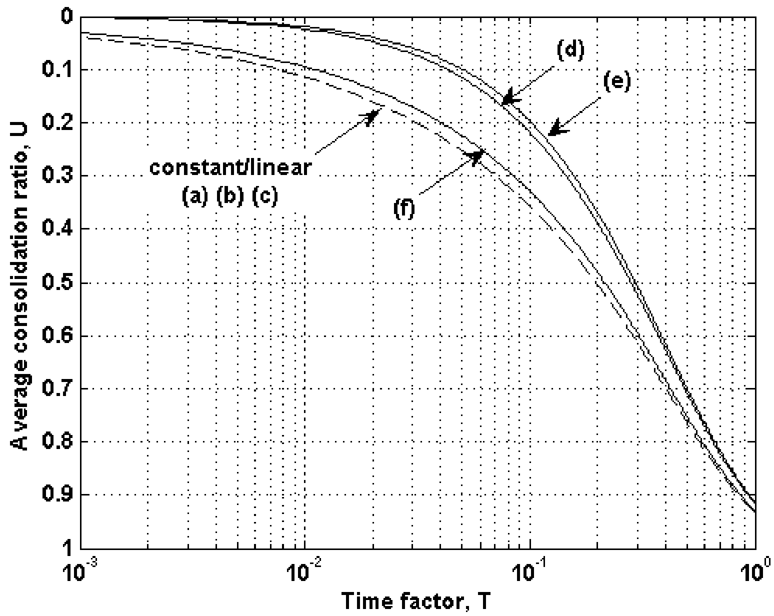

Figure 2.6 – Average consolidation curves according to the Terzaghi theory

When a half-sine variation of initial hydrostatic excess is considered as in Figure 2.5(f), the average consolidation curve (f) is produced – a result which is similar to the linear/constant variation of initial pore pressures. The average consolidation curve (d) shows a sinusoidal variation of initial hydrostatic excess pressures. A similar curve is also obtained when a linear initial distribution with maximum excess pressure at *z/H* = 1.0 is considered as in Figure 2.5(e). A paper on the effects of asymmetric initial distributions has been submitted to the International Journal of Geomechanics ASCE and is available in Appendix B.

2.1.4. Constant rate of loading

The time rates of settlement predicted using Terzaghi's theory are based on the assumption that the entire pressure increase ($\Delta\sigma$) is applied instantaneously. However, in the construction of a typical building, the application of pressure requires a significant amount of time. Land reclamation, in particular, is usually conducted in stages. A surcharge preload is often applied in erratic increments that are dependent upon fill availability, weather conditions, the presence of vertical drains etc. Thus, the application of pressure to facilitate consolidation is not instantaneous, in direct contrast with Terzaghi theory conditions.

To account for the construction period, Terzaghi suggested that the loading be assumed instantaneous but only act for half the duration (Taylor 1962). In this case, the load is assumed to increase linearly with time over the construction period (t_0). This method was later extended by Gilboy (1936); the settlement at t (where t represents some time that has lapsed since construction began) is approximately equal to t/t_0 times the settlement that corresponds to $t/2$ on the instantaneous loading curve. An analytical model was later proposed to predict the degree of consolidation for constant rate of loading scenarios where the permeability varies with the dissipation of excess pore pressure (Schiffman 1958).

A simpler analysis based on Terzaghi's consolidation theory was later developed to account for loading at a constant rate, where the degree of consolidation can be predicted at any time, during or after construction/surcharge application. It should be noted that the proposed method assumes a linear or constant initial excess pore pressure variation with depth (Sivakugan and Vigneswaran 1991).

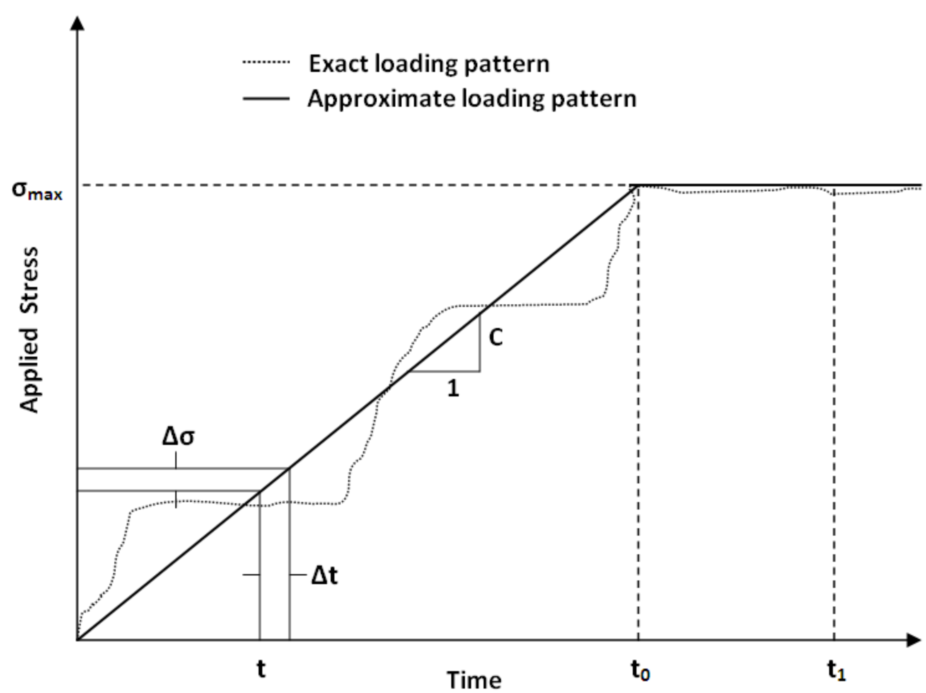

Figure 2.7 – Exact and approximate loading diagrams for constant rate loading

The 'stepped' loading pattern usually associated with surcharge application in reclamation areas, and the corresponding constant rate approximation are shown in Figure 2.6, where the load is being applied at a constant rate C given by;

$$C = \frac{\sigma_{\max}}{t_0} \tag{2.12}$$

Compliant with Terzaghi's theory, the rate of pore pressure dissipation due to an infinitesimal stress increment is not influenced by the previous or subsequent increment. This principle of superposition is also applied with respect to loading. Sivakugan and Vigneswaran (1991) considered the process of constant rate loading as an application of a series of infinitesimal stress increments one after the other.

2.2 Properties of Dredge Materials

2.2.1. Introduction

Whilst land reclamation is ideally executed using granular material, the cost and demand of such material results in the need for an alternative source to be considered. The abundance of unwanted material generated from dredging processes is consequently a potential source for land reclamation. Due to the poor performance and handling of dredge mud in its natural state, stabilisation methods and/or consolidation acceleration techniques are usually employed. It is therefore necessary to classify the properties of dredge waste. There exist two types of dredge material which are classified based on the material's consistency; ultra-soft non-structure soil and firm lump clay (Bo et al. 2001).

2.2.2. Ultra-soft non-structure soil

Ultra-soft non-structure soil (or 'slurry' when combined with excess water) usually consists of fine clay material and is more difficult to handle than firm lumpy clay. There are a number of disadvantages of ultra-soft non-structure soil in comparison with firm lumpy clay. The volume of fill generated after consolidation of ultra-soft soil is generally very small due to the large settlements experienced during sedimentation and consolidation. Also, to enable placement, the water content of ultra-soft soil must first be increased to facilitate pumping, and then subsequently decreased as part of the reclamation process. This double handling of slurry can complicate material characterisation and further protract the placement process time (Robinson et al. 2005).

Ultra-soft soil is pumped in slurry form into paddocks enclosed using containment bunds to prevent dispersion. As a result, sufficient time is required to allow the slurry to complete the initial sedimentation process. The length of sedimentation time is dependent upon the size and mineralogy of fine material and volume of slurry. This process can often continue for several years. It is important to note that the sedimentation stage and the consolidation stage are not mutually exclusive – these processes occur simultaneously as the base layer becomes a soil with a developed fabric.

Due to the ultra-soft nature of the slurry-like soil at the conclusion of the sedimentation stage, only two in-situ measurements can be conducted to determine the soil properties; in-situ field vane tests or Radio-isotope Gamma-Gamma probe tests (Bo et al. 2001).

2.2.3. Firm lumpy clay

Firm to stiff lumpy clay is sourced from firm clay layers using Grab dredging – the size of lump is dependent upon the Grab size which ranges from 3 to 5 m. After comparison between the use of slurry and large clay lumps as reclamation fills, Bo et al. (2001) concluded that reclamation using clay lumps is easier than using slurry. However, the ultimate state of land reclaimed using clay lumps is highly variable in comparison with ultra-soft clay slurry, where homogeneous properties are assumed (Karthikeyan et al. 2004).

Due to the nature of clay lumps, placement does not require containment bunds. However, Bo et al. (2001) recommend an underlying sand blanket of 1-2 m be pumped onto the seabed prior to placement of clay lumps to accelerate the final consolidation process.

Yang et al. (2002) noted that there are two systems of voids in a lumpy clay fill; inter-lump voids (voids between lumps) and intra-lump voids (voids within lumps). If, at ultimate state, the size of inter-lump voids has been reduced to the size of intra-lump voids, the lumpy fill can be classified as homogeneous. The two main engineering issues related to the use of clay lumps for land reclamation are the size of inter-lump voids at the end of consolidation and the strength and deformation characteristics of the reclaimed ground (Karthikeyan et al. 2004).

Manivannan et al. (1998) determined that substantial closure of inter-lump voids can be achieved under a surcharge pressure of 120 kPa, based on centrifuge tests using stiff silt clay lumps of 1 cm^3 in volume. One-dimensional compression tests conducted by Leung et al. (2001) showed that the majority of inter-lump voids were closed during the first stage of loading of 25 kPa. Robinson et al. (2005) carried out tests using clay lumps cut from the lower Marine clay (LMC) in Singapore. Results show that the consolidation pressure required to reduce the permeability of the fill to that of uniform clay was approximately 100 kPa. It is important to note that cone penetration tests conducted to determine shear strength indicate that the fill is still highly heterogeneous under this pressure.

The permeability of the inter-lump system is much greater than that of the intra-lump system, and thus dominates the overall consolidation process. Through rapid dissipation of pore pressure in the inter-lump voids, the corresponding difference in pore pressure aids in accelerating fluid transfer from intra-lump voids to inter-lump voids (Yang et al. 2002). Therefore, it is reasonable to assume that the inter-lump voids shorten the drainage path and thereby accelerate the consolidation process.

Lumpy clay fills should be characterised both after placement and proceeding reclamation, as the engineering properties of the material can significantly change over time. Initially, the clay lumps are placed such that significant voids exist between them. Thus, despite the strength of the clay in lump form, the presence of large voids can prevent the material from possessing confining strength. This was confirmed by tests conducted by Bo et al. (2001) in the eastern region of Singapore – the lumpy clay material showed significant cone resistance after placement, but did not demonstrate any evidence of friction values due to the absence of confining stress.

2.2.4. Ground Improvement Techniques

Due to the prevalence of thick deposits of marine clay in areas to be reclaimed, methods are often employed to accelerate the consolidation process – a process that, if left to advance naturally, could take decades to complete.

In order to prevent future settlement of the reclaimed area under projected dead and live loads, many ground improvement strategies have been developed. The simplest method of ground improvement involves preloading the foundation soil with a pressure equal to or greater than the future pressure so that the soil may be pre-consolidated to gain the required effective stress (Arulrajah et al. 2004). Often, the length of time and cost associated with the application of surcharge preload alone is unacceptable, and further stabilisation methods are required in conjunction with this technique. Today, the most widely used ground improvement method utilizes a combination of prefabricated vertical drains and surcharge preload. However, other methods such as vacuum consolidation and prefabricated horizontal drains have also been developed for ground improvement techniques.

Surcharge Pre-stress

Preloading involves the application of a surcharge, often in the form of sand fill. By applying a load greater than the future design load, the maximum settlement generated under the design load will be reached much faster. It is at this point that the surcharge is removed and construction can commence.

Figure 2.8 provides a graphical representation of the preload process, assuming the marine clay is at its pre-consolidation pressure (σ_{v1}'). If the design stress was immediately applied, the marine clay would undergo large deformation (Δe_1), which would be detrimental to future structural stability. Instead, a surcharge preload of $\sigma_{v2}' - \sigma_{v1}'$ is applied (i.e. sand fill) so that a new pre-consolidation pressure σ_{v2}' is achieved. When the surcharge is then entirely removed, the future application of the design load will only result in minor deformation (Δe_2).

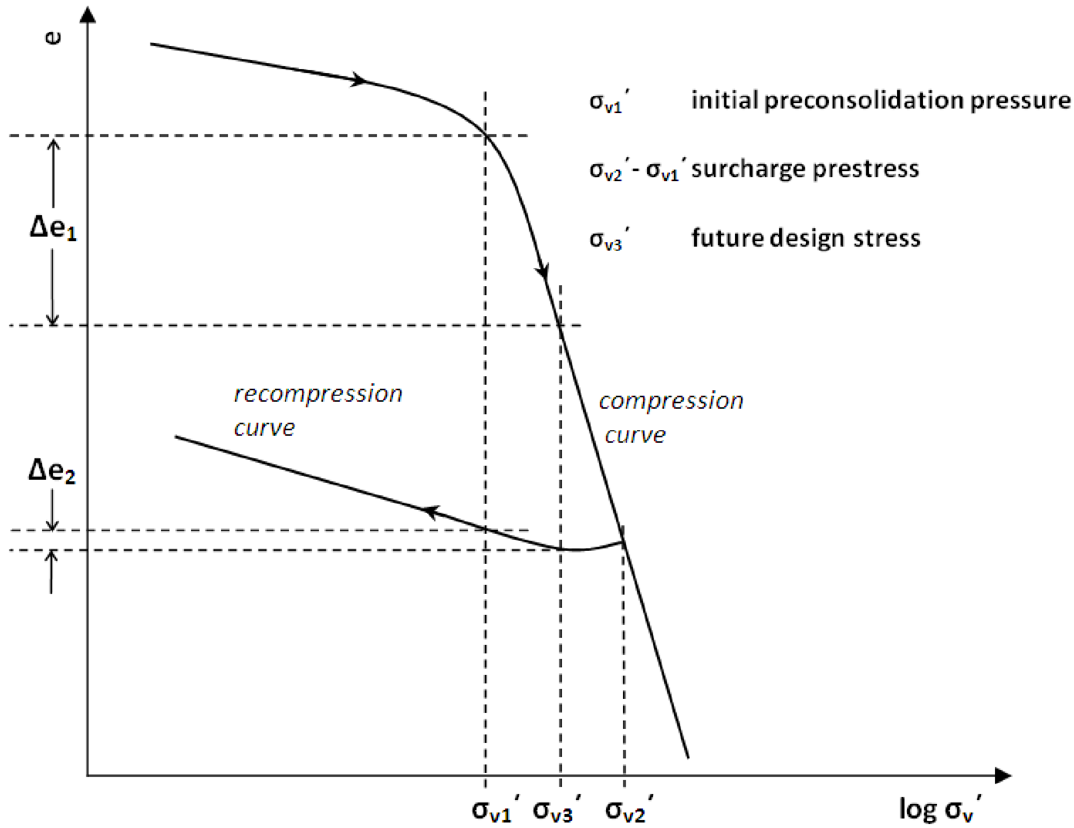

Figure 2.8 – Idealised pressure-versus-void-ratio curve

Once the clay has surpassed its pre-consolidation pressure, the void ratio decrease is governed by the compression index, which describes the slope of the compression curve. If the clay is unloaded, the resulting expansion is governed by the recompression index which describes the slope of the recompression curve. Any subsequent reloading will follow the recompression curve until the new pre-consolidation pressure is reached, upon which the compression curve will dominate, and the clay is said to be normally consolidated. If the clay is at a pressure state less than the pre-consolidation pressure, it is said to be over-consolidated.

More importantly, preloading reduces overall project costs by decreasing the time required to reach consolidation under the design load. The coefficient of consolidation (c_v) of a clay when over-consolidated is an order of magnitude larger than when it is normally consolidated. Thus, when the design pressure ($\sigma_{v3}' - \sigma_{v1}'$) is applied to an over-consolidated clay, the final settlement under that load will be reached at time t_1, in comparison with its normally consolidated counterpart, which reaches the final settlement at time t_2, as demonstrated in Figure 2.9.

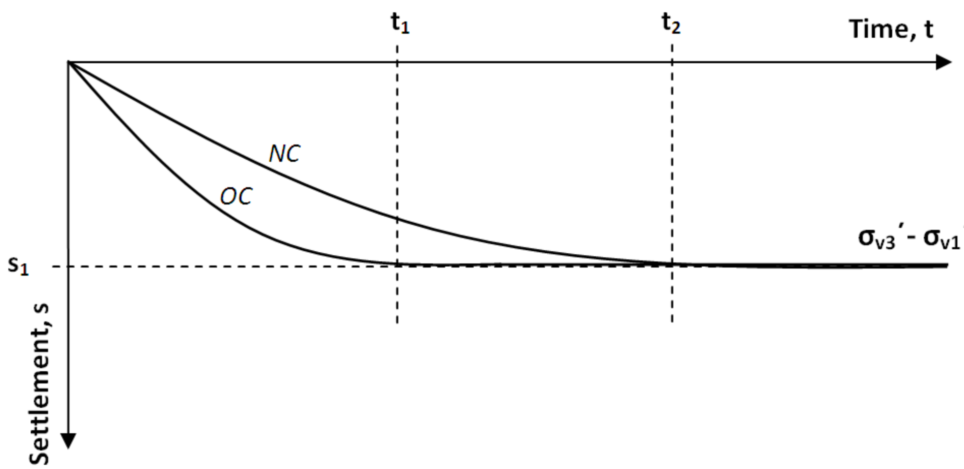

Figure 2.9 – Settlement-time consolidation curves

Whilst preloading generally does improve the stability of soft ground, it generally results in an average degree of consolidation rather than a homogeneous improvement throughout the entire soil thickness (Bo et al. 2007). A more effective soil improvement method involves the implementation of prefabricated vertical drains supplemented by preloading.

Prefabricated Vertical Drains

The use of prefabricated vertical drains (PVD) in conjunction with preloading is the most widely used ground improvement method (Arulrajah et al. 2004). Vertical drains are inevitably required in clay soils with little or no fabric that facilitates natural drainage (Rowe 1972). However, PVDs are also useful in laminated or layered soils where the permeability of the soil mass is significantly anisotropic, assuming the loaded area is sufficiently large (Hird et al. 1992). The combination of vertical drains and preloading ensures a more homogeneous improvement, and simultaneously enables pre-stressing and accelerated consolidation of soft soil.

The primary design variables are; the surcharge magnitude, the spacing of vertical drains, the duration of preloading, the degree of consolidation and the coefficient of consolidation for horizontal flow (c_h). The consolidation process with PVDs involves consolidation due to both vertical flow (c_v) and horizontal flow (c_h). However, since the soil thickness is generally much greater than the drain spacing (i.e. horizontal drainage path), the consolidation due to horizontal flow becomes the governing factor. It is generally common practice for vertical drain installation in marine clays to assume a c_h value of twice c_v (i.e. $c_h = 2c_v$), to account for lamination and foliation in the marine clay formation (Bo et al. 2007).

Carrillo (1942) derived an expression for the average degree of consolidation for combined vertical and horizontal (radial) water flow, U_{vh};

$$1 - U_{vh} = (1 - U_v)(1 - U_h) \tag{2.13}$$

where U_v and U_h are the average degrees of consolidation with for vertical and horizontal flow, respectively.

Thus, the time rate of settlement (s_t) for vertical drains can be calculated at any particular time for various surcharge heights by;

$$s_t = s_{ult}(U_{vh}) \tag{2.14}$$

where s_t is the settlement at time t, and s_{ult} is the ultimate settlement.

Prefabricated Horizontal Drains

Another alternative to PVDs is the implementation of prefabricated horizontal drains (PHD). This ground improvement method often involves the installation of plastic drains in the reclaimed ground in the horizontal direction, upon which either vacuum pressure or gravity drainage is applied at the end of the drains. Furthermore, the principal of horizontal drains can be applied to modify the hydraulic gradient of vertical flow of pore water in clay to accelerate the consolidation, provided there is sufficient horizontal drainage rate in the drains (Nogami and Li 2003). Thin sand layers have been used for horizontal drains in reclaimed lands with clay fill (Watarii 1984, Lee et al. 1987, Karunarate et al. 1990). Submerged horizontal drains generally require a process through which collected water can be drained out. Nogami and Li (2003) therefore suggest using a combination of vertical and horizontal drains to accelerate consolidation in soft clay.

In contrast to prefabricated vertical drains, submerged horizontal drains are vulnerable to confining pressure, which results in deformation of cross-sectional area and subsequent reduction in discharge capacity. Discharge capacity tests were carried out by Kim et al. (2003) to investigate the factors influencing the discharge capacity of prefabricated horizontal drains for improving soft dredged clays. Factors investigated include; elapsed time, confining pressure, hydraulic gradient and strength of filter and core on discharge capacities. As expected, the discharge capacity of a drain with a deformable core decreased further with elapsed time. Interestingly, as confining pressure increased, the discharge capacities in all drains linearly decreased, a result attributed to the reduction in available flow area.

2.3 Settling Process of Clay Slurry

2.3.1. Introduction

Slurry, a type of dredge disposal, is defined as a dilute soil-water mixture with a water content much greater than its liquid limit (Bo et al. 1997). According to Yong and Townsend (1984), slurry-like soils commonly have a total solids concentration of 2-5% by weight, although this is not a definitive criterion of slurry. Under the field of gravitation, a clay slurry can never be stable (Imai 1980). After placement, the suspended soil particles gradually settle, thereby forming a soil layer at the bottom of the water column which subsequently undergoes consolidation under its own weight. Thus, the total settling performance of the suspended solids is a combination of sedimentation and self-weight consolidation.

According to Imai (1980), there are four main types of settling for clay materials; dispersed free settling, flocculated free settling, zone settling and consolidation settling. The difference in settling type is a result of two factors, the degree of flocculation and the degree of mutual interactions among soil particles. The degree of flocculation is a factor of the water's salt concentration, whereas the degree of mutual interactions among soil particles is dependent upon the mixture's solid concentration.

2.3.2. Development of Soil Deposits

As shown in Figure 2.10, the birth of new sediment from a clay slurry is a result of three stages; flocculation, settling and consolidation.

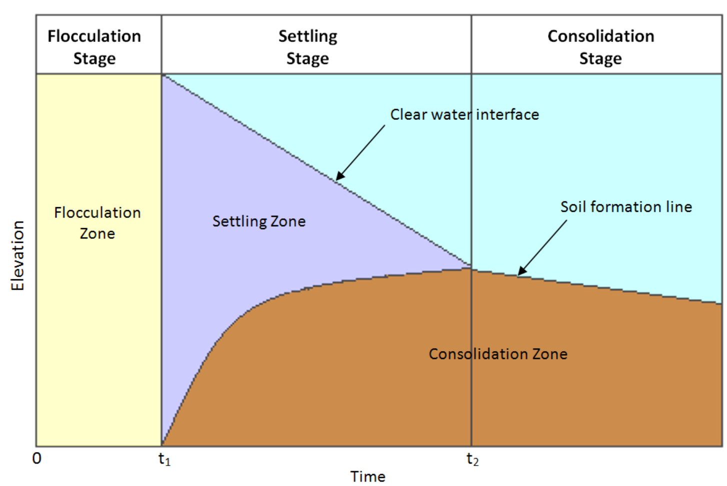

Figure 2.10 – General characteristics of sedimentation and self-weight consolidation (Imai 1981)

In the flocculation stage, flocs are initially generated but no settling takes place. These flocs gradually settle within the settling stage to form a layer of sediment. The thickness of the settling zone decreases over time until the settlement of the resulting continuous soil structure is solely due to self-weight consolidation.

As documented by Imai (1981), the original properties of the slurry dictate whether a flocculation stage is required – in some cases, the flocculation stage is non-existent, with only the settling and consolidation stages visible (i.e. $t_1 = 0$).

Settling stage

Free settling

Free settling can be further separated into two categories; dispersed free settling and flocculated free settling. Dispersed free settling, also known as unhindered "free-fall" discrete solids-settling (Yong and Townsend 1984), takes place when the mixture's solid concentration is very low and as such, soil particles freely settle without mutual interactions. Therefore, the initial settling rate is governed by gravitational mechanisms and Stokes' Law is applicable. However, as solids continue to settle, the concentration increases until particle interactions become significant and the solids cease to settle in accordance with gravitational mechanisms. Since coarser particles settle faster than finer particles, a resulting soil segregation is formed. Consequently, common assumptions of soil homogeneity cannot be applied to settlement calculations involving slurries.

Flocculated free settling is where soil particles form flocs of different sizes and settle freely with rates corresponding to their sizes. When flocculation of soil particles occurs, the initial uniformity of water content is preserved (Imai 1981).

Zone Settling

As the porosity of the solid-suspension decreases, the mutual interaction between particles dominates so that particles of different size and shape may agglomerate and settle uniformly – the settling rate is constant during the settling stage. As a result, particle segregation during settling is precluded and a distinct interface is formed between suspension and clear water above (Pane and Schiffman 1997).

Consolidation Settling

As demonstrated in Figure 2.10, during the settling stage both a settling zone and consolidation zone are present. The boundary between these two zones is known as the sediment formation line. It is during this transition that the soil fabric develops with inter-particle contact and the sediments cease to behave as isolated particles (Blewett et al. 2001).

As demonstrated by the sediment formation line, the thickness of the sediment gradually increases with the build-up of more sediment. Later, the sediment thickness decreases; the corresponding settlement is a result of consolidation and begins once the soil grains are in contact.

Sediment Formation

The density at which the soil structure is formed is known as the "structural density." Consolidation can only commence when a continuous soil structure is formed within the sediment (Sills 1995).

An accepted definition for the point at which the soil particles change into a mass of sediment is "when the soil particles settled onto the bed sediment begin to interact with each other and form an aggregate which transmits an effective stress by virtue of particle-to-particle contacts (Imai 1981)". Monte and Krizek (1976) developed a quantitative method of identifying the formation of a soil mass known as the "fluid limit" which is constant along the sediment formation line. When the water content of a slurry decreases to the fluid limit, the soil-water system becomes a soil mass.

Consolidation stage

Consolidation is the "time-dependent compaction of the soil skeleton as a result of a load," which, in case of slurry-like soils, is generally its own weight (Toorman 1996). The analysis of consolidation behaviour of a soil structure is extremely complex, and consequently, several mathematical models have been developed (Gibson et al. 1981, Toorman 1996, Yang et al. 2002, Hawlader et al. 2008). The non-linear finite-strain consolidation theory developed by Gibson et al. (1981) is the most commonly used theory for predicting self-weight consolidation (Hawlader et al. 2008). However, the accuracy of self-weight consolidation behaviour predictions, using any mathematical model, is dependent on the selection of an appropriate constitutive model. This is evidenced by the proceedings at Sidere, a seminar held in Oxford (2000), where participants overestimated the compressibility behaviour of soil at low effective stresses. As a result, settlement predictions were as high as 300% of the observed settlement values, despite the application of correct numerical procedures (Bartholomeeusen et al. 2002).

2.3.3. Physical Representation

An ideal representation of suspended solids settling in a containment pond and the corresponding concentration profile is shown in Figure 2.11. Zone A, the supernatant layer, is a result of water released in the immediate settling of the slurry, coupled with the water released from sedimentation/consolidation processes taking place below. The solids concentration in this zone is not sufficient to account for proximal hindrances (Yong and Townsend 1984). Zone B represents the transition layer, where the solids concentration dramatically increases. The relatively constant solids profile that emerges below the transition layer is known as the stagnant zone (Zone C). Although no physical contact between suspended solids in Zone C occurs, a small degree of hydrostatic pressure can be measured and compression settling of the solids can therefore take place. When the settlement of these particles reaches a point where physical contact between adjacent particles is achieved, consolidation becomes the governing settlement process. Zone D corresponds to the consolidation zone in Figure 2.10.

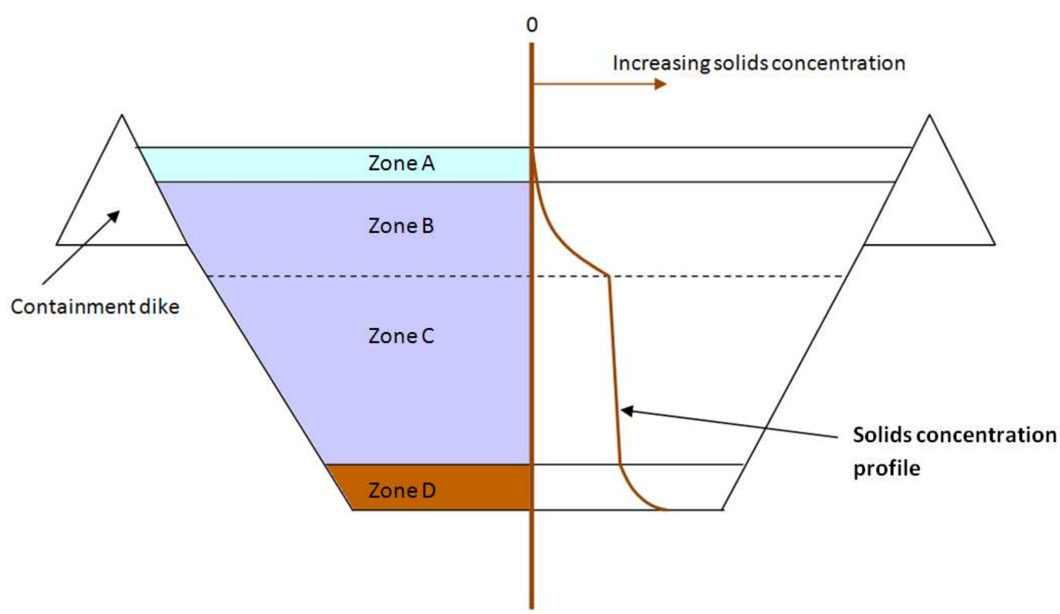

Figure 2.11 – Ideal representation of settling of suspended solids in a containment pond (Yong and Townsend 1984)

The real field simulation of dredged slurry being pumped into a containment pond is shown in Figure 2.13. Coarser particles will settle close to the pipe exit, whilst finer particles will travel to further areas of the containment pond, resulting in an uneven distribution of particles. Thus, in practice, the discharge pipe is often moved periodically to other locations around the containment pond to generate a more even distribution of particles and consequently build a more stable beach. A photograph of the slurry exiting the dredge pipe at the Port of Brisbane is available in Figure 2.12.

Figure 2.12 - Dredge mud exit from pipe

The concentration of coarse particles in Section 2 is greater than that in Section 3, since the sedimentation process of coarse particles is influenced by the location and input conditions of the discharge pipe. The settling performance of the suspended solids in Section 3 can be assumed to emulate the settling process of the ideal containment pond described in Figure 2.13 (Yong and Townsend 1984).

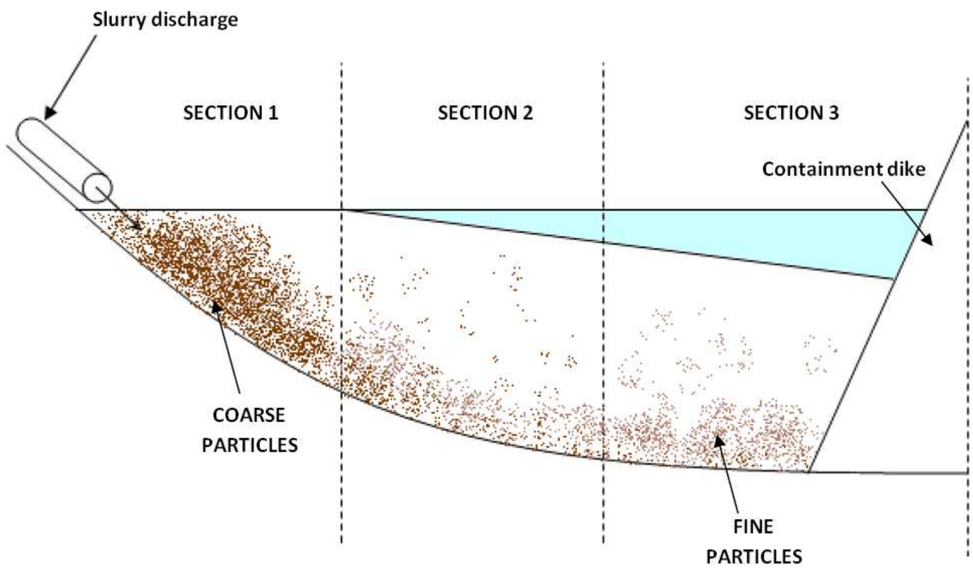

Figure 2.13 – Actual schematic representation of a typical containment pond showing slurry discharge from the input pipe at left (Yong and Townsend 1984)

2.3.4. Experimental Simulation

Using the settling column apparatus described later in Chapter 3, the final sample is expected to resemble the idealised model represented in Figure 2.11. As a result, coarser grained particles will settle toward the bottom of the column, whilst finer particles such as silt and clay will be located at the top of the sample. Therefore, Terzaghi's assumption of homogeneity is invalid when applying one-dimensional consolidation theory to this sample. The coefficient of consolidation (c_v), which is a function of vertical permeability (k) and compressibility (m_v), will consequently be a function of depth. Whilst the variation in compressibility of the sample will be minimal, the variation in permeability is expected to be quite significant, as demonstrated in Figure 2.14 and Figure 2.15. Therefore, an alternative method to account for the non-homogeneity of the sample must be considered.

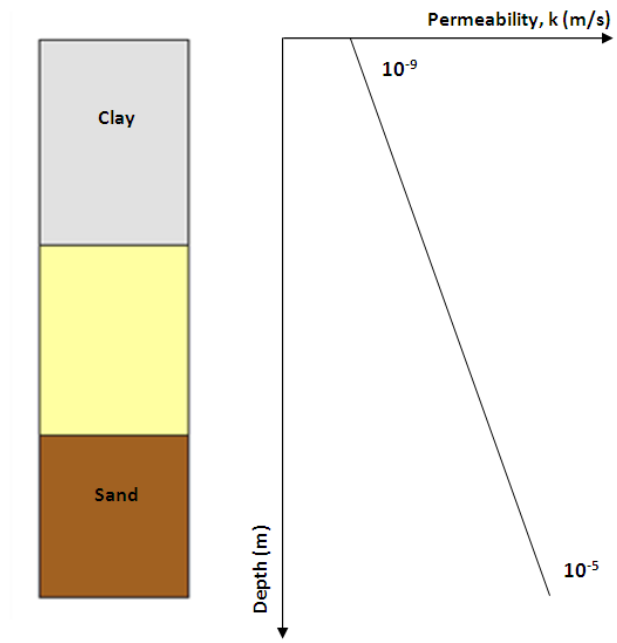

Figure 2.14 - Variation of permeability along depth of sample

Coefficient of Permeability k (m/s)	10^{0}	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}	10^{-11}
Drainage	Good						Poor		Practically Impervious			
Soil types	Clean gravel		Clean sands, clean sand and gravel mixtures			Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.				"Impervious" soils, e.g., homogeneous clays below zone of weathering		
					"Impervious" soils modified by effects of vegetation and weathering							

Figure 2.15 - Permeability and drainage characteristics of soils (Terzaghi et al. 1996)

It is also important to consider the effect of consolidation pressure. As demonstrated in Figure 2.16, the permeability of fine soils significantly decrease with an increase in consolidation pressure, a phenomenon that becomes more pronounced as particle size decreases.

Both permeability and compressibility decrease as consolidation pressure increases. Since both k and m_v decrease as the pressure increases, it is often assumed that the value of c_v remains roughly constant as consolidation pressure increases. However, as evidenced by Duncan (1991) as part of the 27th Terzaghi lecture, the value of c_v decreases greatly as the effective stress reaches the pre-consolidation pressure, and its value generally increases with increasing pressure for normally consolidated clays. Therefore, the coefficient of consolidation actually varies with depth in the layer, and with time during consolidation.

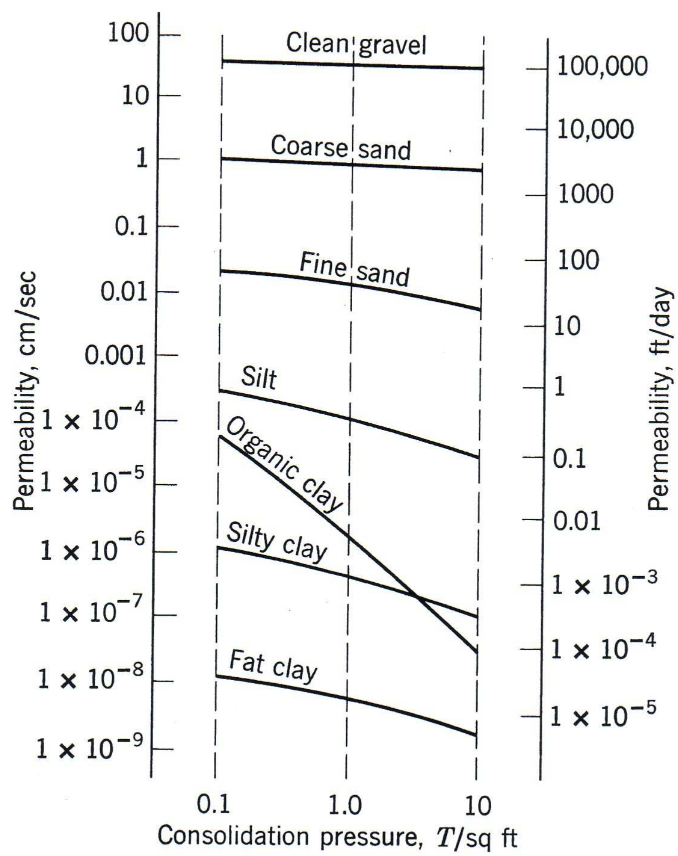

Figure 2.16 - Effect of consolidation pressure on permeability (Cedegren 1967)
1 tsf = 95.76 kPa

Effect of non-homogeneity in clay

The effect of non-homogeneity on the consolidation behaviour of fine soils was first studied by Davis and Raymond (1965), where k_v / m_v and c_v were assumed to be constant with increase of pressure, and a corresponding nonlinear consolidation theory was developed. Gibson et al. (1967) analysed the consolidation problem assuming large strain and nonlinear compressibility and permeability. By assuming k_v and m_v to be polynomial or exponential functions of depth and adopting a differential method, a solution was obtained for one-dimensional consolidation of a single layer of soft clay under instantaneous loading. Equations 2.15 and 2.16 show the variations of k_v and m_v adopted by Gibson et al. (1967);

$$k(z) = k_0 \left\{ 1 + (1+\alpha)^p - \left[1 + \alpha(1 - \frac{z}{H}) \right]^p \right\} \tag{2.15}$$

$$m_v(z) = m_0 \left\{ 1 + (1+\beta)^q - \left[1 + \beta(1 - \frac{z}{H}) \right]^q \right\} \tag{2.16}$$

where k_0 and m_0 denote the permeability and compressibility at the top of the clay layer, H indicates the thickness of the clay layer, and α, β, p and q represent parameters that characterise the manner of variation of k_v and m_v with depth. In all calculations, the clay layer was doubly drained, and the initial excess pore water pressure was constant.

Mesri and Rokhsar (1974), Mesri and Tavenas (1983), and Mesri and Choi (1985) solved the governing differential equations numerically whilst considering the effects of variable compressibility and permeability. Solutions to the one-dimensional consolidation problem, in which k_v and m_v directly relate to c_v and vary with the effective stress and void ratio, were also developed by Li et al. (1999a, 1999b) and Xie et al. (2002). Zhuang et al. (2005) extended these solutions to form a semi-analytical solution for the case of void ratio e-log effective stress p and e-log permeability k_v. Semi-analytical methods were also used to solve the problem of one-dimensional consolidation of non-homogeneous soft clay under time-dependent loading by Xie et al. (2005). However, rather than allow k_v and m_v to be continuous functions alone (as Gibson et al. (1967) did), Zhuang et al. (2005) and Xie et al. (2005) divided the soil stratum into discrete layers until k_v and m_v could be considered constant for each small clay layer.

Interestingly, Xie et al. (2005) concluded that, when the variation of k_v and m_v is identical, the influence of k_v on the distribution of pore water pressure is more significant. In conclusion, it was stated that the one-dimensional consolidation problem of non-homogeneous soil is not dependent on m_v, although k_v should be taken into consideration.

As expected, all results indicate that when m_v and k_v vary greatly, there is a significant difference between the excess pore water pressure in non-homogeneous soil and the pore water pressure of Terzaghi's solution of homogeneous soil calculated using the average consolidation coefficient.

2.4 Port of Brisbane Reclamation Site

2.4.1. Introduction

A period of three weeks was spent under the employment of Coffey Geotechnics at their site office at the Port of Brisbane. During this time, a general understanding of dredging operations and surcharge placement was developed. This was accomplished through routine data analysis and periodic site visits. Furthermore, a direct examination of the Port of Brisbane Corporation's resident cutter suction dredger, the Amity, was allowed. Tours such as these provided a valuable firsthand experience of reclamation operations to complement the analysis of port data that was conducted during the three-week period. Figures 2.17 (a) and (b) show aerial photographs of the Port of Brisbane, taken June 2008.

(a)

(b)

Figure 2.17 - Aerial view of Fisherman Islands as of June 2008
(a) oblique view (b) elevation view

2.4.2. Port of Brisbane Reclamation History

The Port of Brisbane is located at the mouth of Brisbane River and adjacent to the Moreton Bay Marine Park, an area of ecological and conservation value. The Port of Brisbane Corporation (PBC) has embarked on a 270 hectare expansion of the Fisherman Islands Container Terminal in Brisbane (Ameratunga et al. 2003). A seawall extending 1.8 km into Moreton Bay from the existing reclamation encloses the area for future placement of dredged material. Approximately one million cubic metres of material must be annually dredged so that commercial shipping can maintain access to the port. Ideally, sands and gravels would be used for land reclamation fill due to their desirable placement and dredging properties. However, the dredge material available comprises clays and silts with extremely high water contents that require stabilization before they can be utilized as land reclamation fills. The dredge mud exhibits extremely large settlements when subjected to loading and has a low bearing capacity. Thus, the stabilization of these dredge mud fills is necessary prior to construction in order to prevent differential settlements and the subsequent damage these settlements may cause to future structures.

The stabilization of dredge mud fills is a tedious and expensive process and several years are required to facilitate consolidation. There are various ground improvement techniques available today, the most common of which involves preloading. However, due to the very low permeability of the dredge mud, the consolidation period is very long, even if a very high surcharge load is applied. Therefore, further methods of accelerating consolidation are required in conjunction with the preloading process. A system of vertical drains (wick drains) can be implemented to accelerate consolidation by reducing the length of the drainage paths. The vacuum consolidation method is also a valid technique for reducing the required consolidation time.

The process of land reclamation is continual, and many outermost paddocks are currently undergoing land stabilisation to prepare for the construction and loading of future infrastructure. A diagram of reclamation paddocks is available in Figure 2.18, where paddocks R1, R2, J2, S2, S3A and S3B are being preloaded under sand fill surcharge. Paddocks C1, C2, C3, B1, B2 and B3 contain 3 to 5 year old dredge mud, whilst R3 contains recently pumped dredge mud with large quantities of seawater that has drained from other paddocks. The outer FPE (Future Port Expansion) area is yet to be reclaimed and serves as a filtering obstacle – the water that emerges through the seawall is as ‘clean’ as seawater.

Figure 2.18 – Reclamation paddocks

Ground Stabilisation Assessment

The 1.8 km extension of Fisherman Island will be achieved through the placement of dredge mud within the seawall, essentially reclaiming the land. Although the stabilisation of dredge mud is of importance, it is imperative that the soil beneath the dredge mud (Holocene clay) be stabilised, as this is where the majority of future settlements will occur. As evidenced by the coefficient of consolidation, the rate of consolidation is proportional to the thickness of clay squared.

To date, ground improvement techniques such as preloading accompanied by vertical drains have been applied to accelerate the consolidation of Holocene clay of thickness 10 to 15 m. Unfortunately, the extension of Fisherman Island has entered regions of Holocene thickness greater than 30 m. The PBC is currently investigating several methods for accelerating the consolidation process, so that the most efficient and cost-effective method can be selected for these problematic areas. Figure 2.19 provides a contour plot of Holocene clay thicknesses.

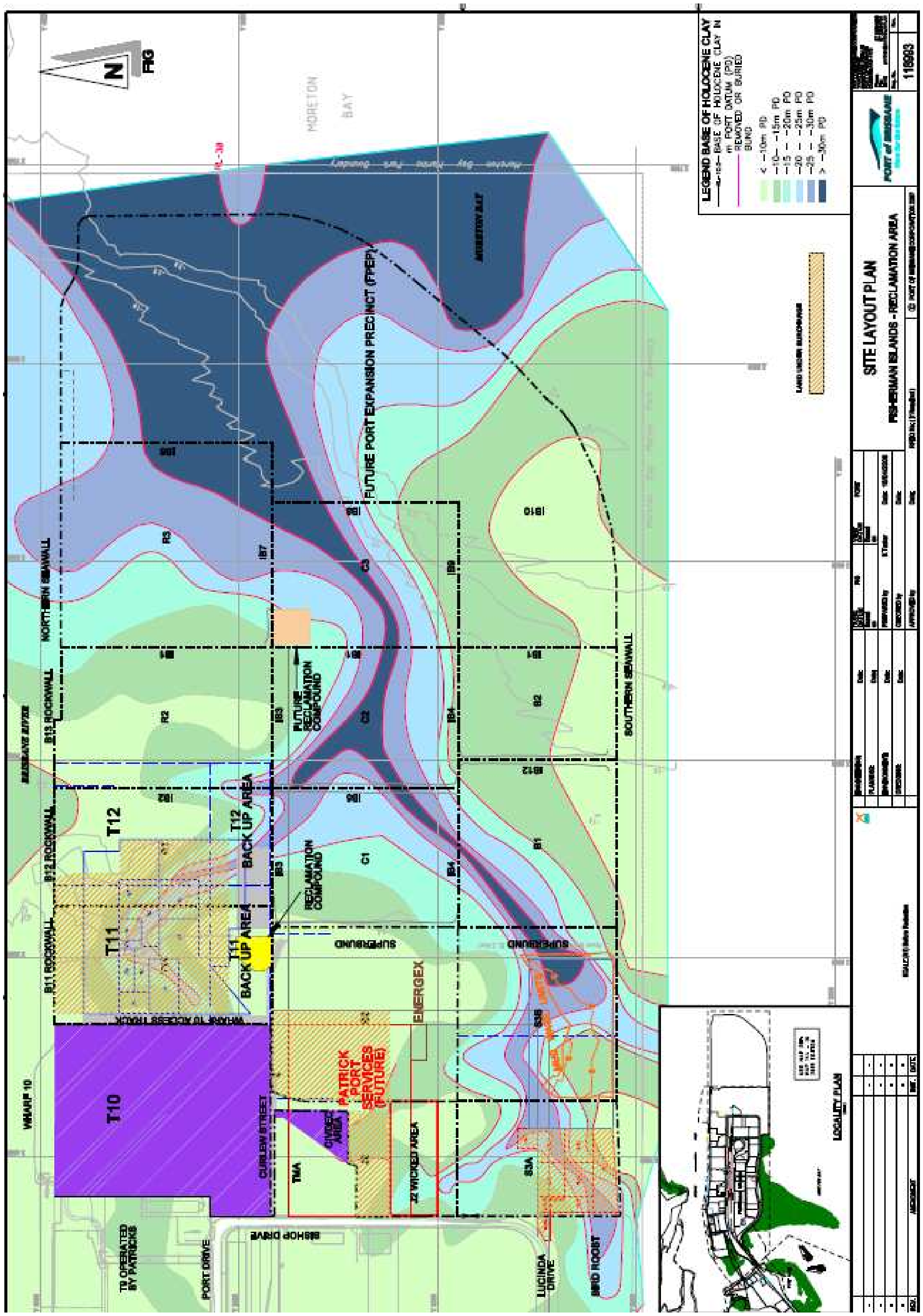

Figure 2.19 – Holocene clay thickness contours

Three dredging and marine contracting companies, Van Oord, Boscalis and DGI-Menard Inc., were commissioned to implement several methods for accelerating the consolidation process. Vacuum consolidation and the effect of various wick drain spacing and configuration were primarily studied. Coffey Geotechnics were required to analyse the resulting data to determine the most efficient and effective acceleration method. The Asaoka method was used to back-calculate coefficient of consolidation values and predict the ultimate primary settlement, based on the accumulated data already obtained (Asaoka 1978).

Once the coefficient of consolidation values (c_v) were determined, a settlement prediction analysis was carried out to determine the date at which preloading could be removed, and the residual settlement under the design load. The program CAOS (Consolidation Analysis of Soils), developed by Professor Harry Poulos for Coffey Geotechnics, was used to determine these design dates. Furthermore, CAOS was used to confirm that specifications regarding the design life were achieved – the maximum residual settlement over 20 years of the design life is 150 mm.

2.4.3. Dredging Operations

The primary dredging equipment used at the Port of Brisbane consists of a suction hopper dredger, the Brisbane, and a cutter suction dredger, the Amity.

Suction Hopper Dredger

The main unit of the PBC dredging fleet is the Brisbane, a trailing suction hopper dredger shown in Figure 2.20.

Figure 2.20 – THSD Brisbane (Channel maintenance dredge)

It is capable of completing capital and maintenance dredging, and is equipped with the latest state-of-the-art automation control and navigation systems. Despite being primarily used for maintenance and development dredging, and reclamation works in Brisbane, the Brisbane dredger also assists in an annual maintenance dredging campaign in north Queensland ports.

A useful property of the Brisbane is its certification as an ocean-going dredger, as dredged material can be temporarily stored in the hopper, rather than being immediately pumped ashore as a discharge system.

The maximum dredging depth of the Brisbane is 25 m, and the hopper is capable of holding 2900 cubic meters of material before disposal via pumping is required. Whilst the material is being pumped through fixed or floating pipeline to the discharge area, the water content of the material is increased up to five times its in situ value, so that it can be pumped long distances with minimal resulting muddy water plumes.

Figure 2.21 depicts the supernatant water overflow system. The circular rings start low in the hopper and as it fills with mud, the rings are progressively raised to ensure no material is lost. The rings act like a weir, so that the supernatant water exits the vessel over the rings.

Figure 2.21 – Supernatant water overflow system

Cutter Suction Dredger

The Amity is the Port of Brisbane's resident cutter suction dredger, and is primarily used for developing berths and associated reclamation at the river mouth. The vessel is also equipped with a rotating auger tip which effectively cuts through a wide range of materials. Capable of dredging depths up to 17 m, the Amity utilises both floating and shore pipelines to transport dredged material.

Figure 2.22 - Amity

Clam Bucket Dredger

The Port of Brisbane also utilises another dredging vessel, the Ken Harvey, a clam bucket dredger with a 3.25 cubic meter bucket. This dredging vessel is used to dredge port berths and approaches, and is usually employed in small-scale dredging operations.

2.4.4. Dredge Material Properties

A number of basic properties such as particle density and sensitivity of the dredged material were provided by Coffey Geotechnics on behalf of the PBC.

Particle Density

Based on twenty test results from Paddocks S2 and S3B, the particle density of dredged material and Holocene clay varies from 2.3 to 2.74 t/m^3, respectively. An average of 2.54 t/m^3 is often assumed.

The density of slurry in the dredge pipelines is generally 1.3 t/m^3. Percent solids are assumed to be in the range of 2.2 to 2.6% by weight. Since the density of seawater is 1.025 t/m^3, the moisture content of the slurry pumped from the hopper in the Brisbane is approximately 200%. However, for the Amity, water contents of 1000% or greater can be expected.

Sensitivity of Holocene Clay

The sensitivity of a clay is defined as the ratio of undrained strength of undisturbed soil to undrained strength of the soil after it has been remoulded (Lancellotta 2009). A graphical representation of sensitivity along the depth of the Holocene clay is presented in Figure 2.23.

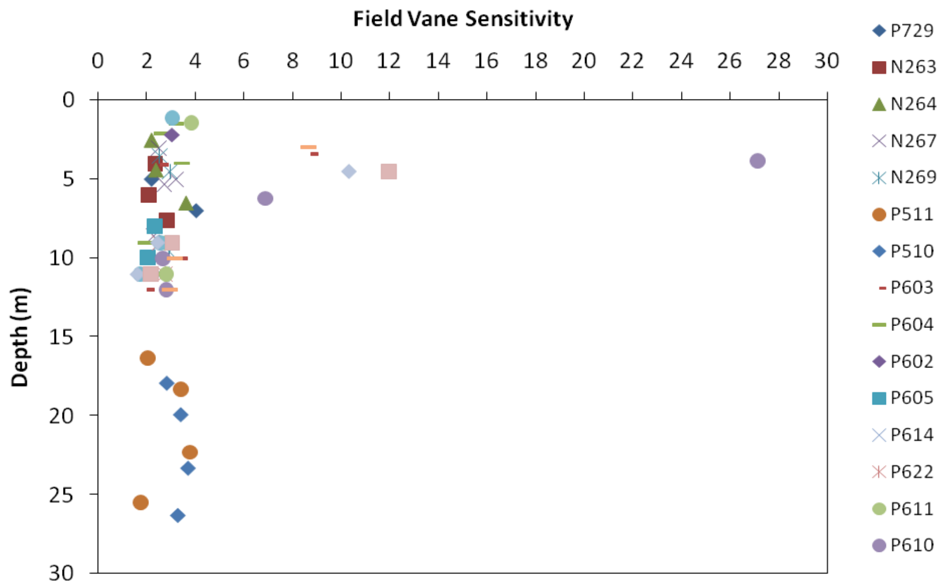

Figure 2.23 - Variation of field vane sensitivity with depth

The sensitivity of the Holocene clay at the Port of Brisbane generally ranges from 2 to 4, and therefore can be described as highly sensitive. Sensitivities of 2 to 4 are also common among normally consolidated clays, which suggest that the majority of the Holocene clay is normally consolidated (Nagaraj and Miura 2001). However, at a depth of approximately 4 m, there is a thin layer of Holocene clay that exhibits values of sensitivity as high as 27.

For comparison purposes, the sensitivity of clay at Changi East reclamation site is also provided in Figure 2.24. The sensitivity values generally range from 3 to 8, thereby indicating highly sensitive clay.

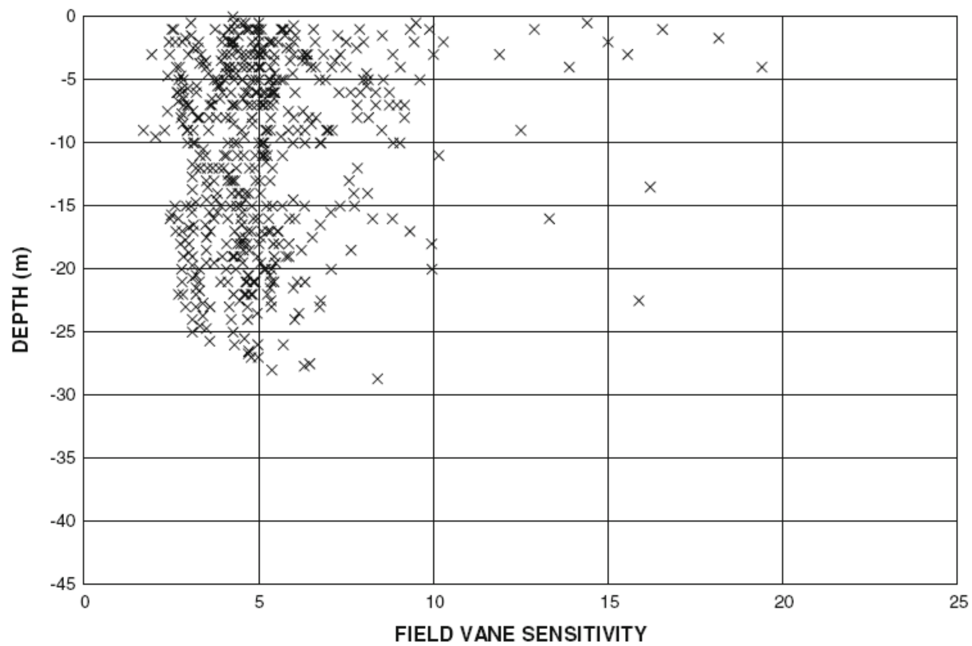

Figure 2.24 - Variation of field vane sensitivity with depth at Changi East reclamation site (Arulrajah and Bo 2008)

Chapter 3: Research and Methodology

3.1 Objectives

The primary goal of this project is to investigate the effect of non-homogeneity on the consolidation behaviour of clays and the impact these results will have on current consolidation analyses of slurry at the Port of Brisbane. A number of aspects of Terzaghi's one-dimensional consolidation theory will be analysed so that a more accurate method of consolidation analysis can be established.

3.1.1. Initial distribution of excess pore water pressure

As outlined in Section 2.1.3, the consolidation behaviour of a soil stratum is inherently dependent upon the initial distribution of excess pore water pressures. As reported in most soil literature (Taylor 1962, Holtz and Kovacs 1981, Berry and Reid 1988, Powrie 1997, Atkinson 2007, Lancellotta 2009), a soil subject to constant initial excess pore pressure is most representative of one-dimensional consolidation. However, the average consolidation behaviour of other initial excess pore water pressure distributions (such as those in Figure 3.1) has also been analysed by Taylor (1962), Powrie (1997) and Lancellotta (2009), despite the inherent two- or three-dimensional consolidation behaviour associated with such distributions.

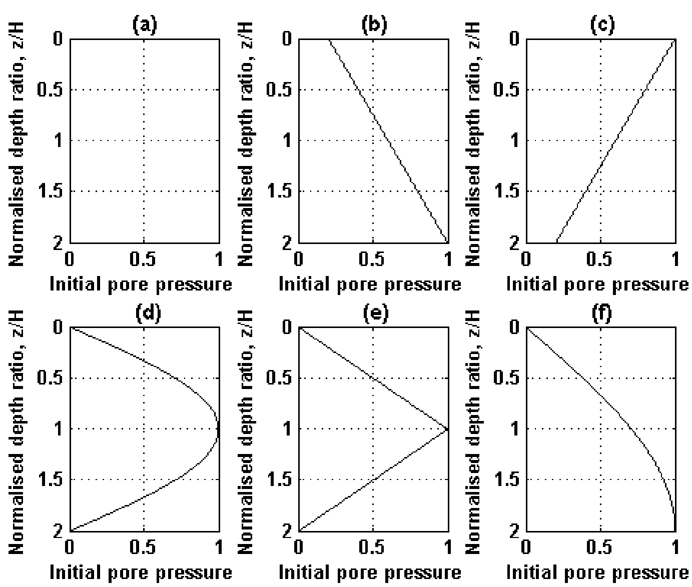

Figure 3.1 - Standard initial excess pore pressure distributions

Consolidation ratio isochrones are only ever provided for cases of constant initial excess pore water distributions. It is the average consolidation curves that are usually used to highlight differences in consolidation behaviour between the various initial distributions shown in Figure 3.1. The absence of accompanying consolidation ratio isochrones for these initial distributions, however, excludes significant information regarding the consolidation behaviour with respect to depth. Therefore, a more comprehensive analysis of consolidation behaviour in terms of consolidation ratio isochrones (and pore pressure isochrones) is proposed.

It is expected that the achievement of this objective will result in two journal papers that address both singly and doubly drained soil strata, and highlight possible relationships between these two cases.

3.1.2. Degree of Consolidation

It is widely accepted that the percentage of excess pore pressure that has dissipated at some time during consolidation (*U*) can be applied to estimate the total settlement at that time;

$$s_t = U s_\infty \tag{3.1}$$

where s_t is the settlement at time *t*, U is the degree of consolidation averaged for the entire depth of soil, and s_∞ is the ultimate (or total) settlement.

Whilst it is probable that Equation (3.1) is applicable to soil strata subjected to constant initial excess pore water pressure, its accuracy in relation to other distributions (Figure 3.1) should be verified. Due to the fundamental assumption of soil and water incompressibility, the degree of settlement is considered equal to the volume of water discharged through the drainage boundaries. Thus, the degree of settlement can be calculated as;

$$\frac{s_t}{s_\infty} = \frac{\int_0^T k \frac{\partial U}{\partial z}}{\int_0^\infty k \frac{\partial U}{\partial z}} \tag{3.2}$$

where *k* is the permeability and $\partial U / \partial z$ is the hydraulic gradient.

3.1.3. Constant Loading

Research conducted by Sivakugan and Vigneswaran (1991) on the average consolidation behaviour of soil strata subjected to a constant rate of loading (rather than instantaneous application of load) will be extended to include various initial distributions of excess pore water pressure. This objective is particularly relevant to consolidation cases where hydraulic fills exist. Due to the high workload and costly nature of surcharge preloading, the application of surcharge to hydraulic fills and slurry is often conducted in stages. Thus, the load under which consolidation occurs is not applied instantaneously, but rather over some period of time. Furthermore, since the program MATLAB will be utilised for this analysis, it is possible to simulate the more realistic staged loading and compare this with a constant rate of loading, as developed by Sivakugan and Vigneswaran (1991).

3.1.4. Non-homogeneity

A fundamental assumption of Terzaghi's one-dimensional consolidation theory is that the soil being analysed is entirely homogeneous. Thus, the material properties of permeability (k) and volume compressibility (m_v) are considered constant for the entire depth of a soil stratum for consolidation analyses. This renders the coefficient of consolidation (c_v) described in Equation (2.3) also constant, and integration is conducted accordingly (see Appendix A).

Whilst the assumption of homogeneous conditions may be valid in most situations, it is not applicable to slurry or dredged mud. In these cases, settling takes place in conjunction with consolidation and as such, the soil undergoing consolidation will not have uniform properties. Coarser particles will have settled first, indicating a higher degree of permeability at the base of the soil. The finer particles that settle towards the surface of the consolidation profile will have a lower degree of permeability. This effect of non-homogeneity on the consolidation behaviour of fine soils has been studied by Davis and Raymond (1965), Gibson et al. (1967), Mesri and Rokhsar (1974), Mesri and Tavenas (1983), Mesri and Choi (1985), Zhuang et al. (2005) and Xie et al. (2005).

The series solution developed in Section 3.1.1 will be modified so that Terzaghi's one-dimensional consolidation equation (Equation 2.2) can be solved for a non-homogeneous soil stratum, where k and/or m_v vary with depth. As suggested in previous literature (Xie et al. 2005), the variation in k has a greater impact on consolidation behaviour than the variation in m_v. By allowing k to vary whilst m_v remains constant (and vice versa), this conclusion can also be validated.

Furthermore, the effect of varying both k and m_v will also be studied, where each varies according to the following expression;

$$k(z) = k_0\left(1+\frac{z}{H}\right)^p \tag{3.3}$$

$$m_v(z) = m_0\left(1+\frac{z}{H}\right)^q \tag{3.4}$$

where k_0 and m_0 denote the permeability and compressibility at the top of the clay layer, H indicates the drainage thickness of the clay layer, and p and q represent parameters that characterise the manner of variation of k and m_v with depth. Gibson et al. (1967) used similar variations of k and m_v for their analysis, the results of which will provide a basis for comparison during this investigation.

All previous analyses of the effect of non-homogeneity on the consolidation behaviour of soils have been conducted assuming instantaneous loading and constant initial excess pore pressure distributions. Thus, the opportunity to extend this analysis to include constant loading and/or varying initial excess pore pressure distributions is available.

3.2 Methodology

Although the bulk of this investigation will be conducted using numerical methods (series solution and its application in MATLAB), a significant component will incorporate experimental work, the results of which are necessary for numerical analysis.

3.2.1. Numerical

Separation of variables will be used to determine a series solution for the consolidation equation (Equation 2.2) and the corresponding boundary conditions will be applied to determine the series coefficients. The series coefficients can be evaluated using an orthogonality relationship (Taylor 1962, Holtz and Kovacs 1981). However, a more efficient procedure is to use pseudo-spectral or collocation approach. The series is truncated after *N* terms and forced to satisfy Eq. (4) at *M (=N)* collocation points where $i = 1,...M$ and;

$$z_i = \left(\frac{i}{N+1}\right)2H. \tag{3.5}$$

The above method will provide the basis for all numerical analyses within this investigation. The program MATLAB will be used to simulate all relevant scenarios.

3.2.2. Experimental

Undisturbed dredge mud samples from the Port of Brisbane will be analysed to determine fundamental properties (specific gravity, Atterberg limits, particle distribution, sensitivity etc.) and establish basic consolidation properties. Oedometer tests with permeability-measuring capabilities will be utilised so that the relationship between consolidation pressure and permeability (as in Figure 2.16) can be established.

More importantly, the degree of non-homogeneity in a slurry sample needs to be defined. The Port of Brisbane dredged mud will be used for this investigation. The settling process that occurs upon placement of the slurry in containment bunds will be replicated on a smaller scale in the laboratory using a settling column (Figure 3.2).

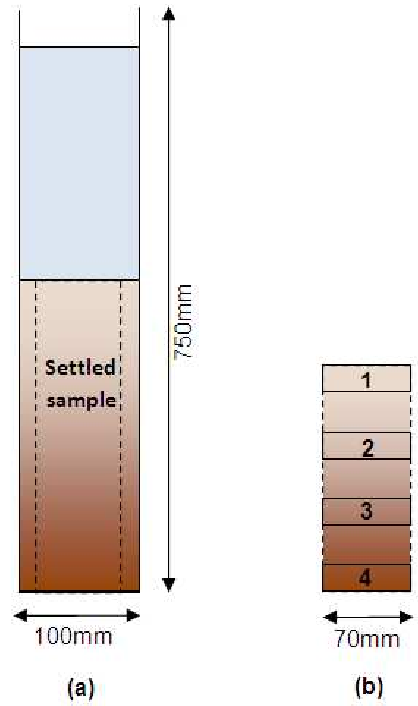

Figure 3.2 – Settling column

The reconstituted dredge mud will be poured into the top of the settling column and allowed to settle accordingly (Figure 3.2 a). Once all settling has taken place, a sample approximately 70 mm in diameter will be extruded, and a hydraulic gradient applied to consolidate the sample to working conditions. Three to four samples of approximately 40 mm height will then be tested in an oedometer to determine the permeability and coefficient of volume compressibility. Thus, a relationship between permeability/compressibility and depth in the form of Equations (3.3) and (3.4) can be established. These relationships will be utilised in subsequent numerical analyses. A modified oedometer apparatus is also proposed, through which pore pressures can be directly measured.

3.3 Reporting and Publication

Numerous papers are expected to come from research associated with this investigation. Since these papers will be submitted and published as the data becomes available, the report writing for the final thesis will take place simultaneously. Accepted journal paper submissions will further validate thesis reporting.

3.3.1. Publications

A summary of expected journal papers is provided in Table 3.1.

Table 3.1 – Future journal paper submissions

Project Objective	Paper Topic	Number of Papers
Initial distribution of excess pore water pressure	• One-way drainage • Two-way drainage	2
Degree of consolidation	• Validation of current settlement assumptions	1
Constant loading	• Validation of Sivakugan and Vigneswaran (1991) research, and elaboration on loading styles • Varying initial distributions of excess pore pressure	2
Non-homogeneity	• Vary k, whilst m_v is constant • Vary m_v, whilst k is constant • Variation of k and m_v • Effect of varying initial distributions of excess pore pressure • Effect of constant loading	5
Modified oedometer apparatus		1

The possible journal papers above will be evaluated based on content and submitted to the relevant journal on the list below;

- Geotechnique
- ASCE Journal Geotechnical and Geomechanical Engineering
- ASCE International Journal of Geomechanics
- ASTM Geotechnial Testing Journal
- Canadian Geotechnical Journal
- Soils and Foundations
- Geotechnical and Geological Engineering

3.3.2. Progress Report

Annual progress reports will be submitted to the School of Engineering and Physical Sciences as per postgraduate degree requirements.

Currently, standard oedometer tests are being conducted on undisturbed dredged mud samples approximately 3 years old from the Port of Brisbane reclamation paddocks. Basic tests such as particle size distribution, specific gravity and sensitivity are also underway.

To date, a journal paper on the effects of initial distributions of excess pore water pressure has been submitted to the ASCE International Journal of Geomechanics.

3.4 Timetable

The following timetable outlines project goals and their respective deadlines.

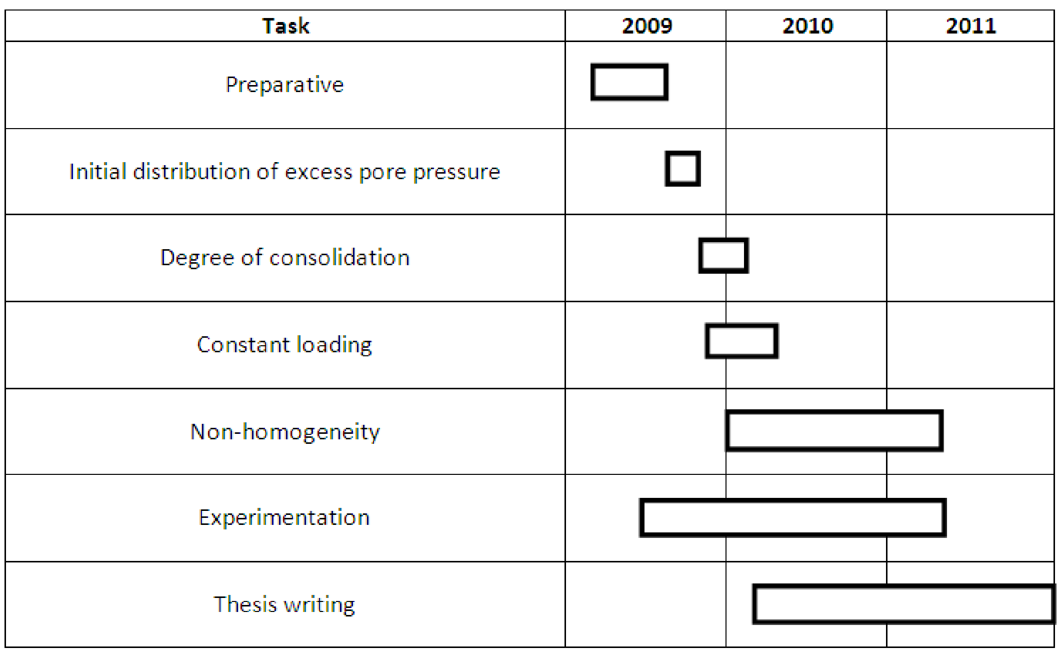

Figure 3.3 - Timetable

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Appendix A: Series Solution for Terzaghi's One-dimensional Consolidation Theory

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

Boundary Condition	Mathematical Expression
There is complete drainage at the top of the soil layer.	When $z = 0,\ u = 0$
There is complete drainage at the base of the soil layer.	When $z = H, u = 0$
The initial hydrostatic excess pressure (u) is equal to u_i.	When $t = 0, u = u_i$

$$\text{let } u = F(z)G(t)$$

$$\therefore \frac{F''}{F} = \frac{G'}{G}\frac{1}{c_v}$$

The left hand side of the equation is only dependent on *z*, and the right hand side is only dependent on *t*. Since *z* and *t* are independent of each other, the above equations equal some constant.

$$\frac{F''}{F} = \frac{G'}{G}\frac{1}{c_v} = k$$

<table>
<tr><th>F</th><th>G</th></tr>
<tr><td>Boundary conditions:
$u(0,t) = 0$
$u(2H,t) = 0$

Form of the equation:
$F''-Fk = 0$
$F = e^{\lambda z}; F' = \lambda e^{\lambda z}; F'' = \lambda^2 e^{\lambda z}$

$\lambda^2 - k = 0$

CASE I: k>0
$\lambda = \pm\sqrt{k}$
$F = Ae^{\sqrt{k}z} + Be^{-\sqrt{k}z}$
$A = 0; B = 0$
No non-trivial solutions</td><td>Boundary conditions:
$u(0,t) = 0$
$u(2H,t) = 0$

Form of the equation:
$G'-Gkc_v = 0$

$\therefore G = A\exp\left(-c_v\left(\frac{n\pi}{2H}\right)^2 t\right)$</td></tr>
</table>

CASE II: *k=0* $$\lambda = 0$$ $$F = A + Bz$$ $$A = 0; B = 0$$ No non-trivial solutions CASE III: *k<0* $$\lambda = \pm i\sqrt{-k}$$ $$F = A\cos\sqrt{-k}z + B\sin\sqrt{-k}z$$ $$A = 0$$ $$B = 0 \text{ or } B = \sin\sqrt{-k}\,2H = 0$$ $$k = -\left(\frac{n\pi}{2H}\right)^2$$ $$\therefore F = B_n \sin\left(\frac{n\pi z}{2H}\right)$$	

$$u(z,t) = \sum_{n=1}^{\infty} D_n \sin\left(\frac{n\pi z}{2H}\right)\exp\left(-c_v\left(\frac{n\pi}{2H}\right)^2 t\right)$$

$$= \sum_{n=1}^{\infty} D_n \sin\left(\frac{n\pi Z}{2}\right)\exp\left(-T\left(\frac{n\pi}{2}\right)^2\right)$$

where,

$$Z = \frac{z}{H}$$

$$T = \frac{c_v t}{H^2}$$

$$D_n = AB_n$$

Boundary condition:

$$u(z,0) = \sum_{n=1}^{\infty} D_n \sin\left(\frac{n\pi z}{2H}\right)$$

Multiply both sides by $\sin\left(\frac{n\pi z}{2H}\right)dz$ and integrate from 0 to 2H.

$$\int_0^{2H} u_i \sin\left(\frac{n\pi z}{2H}\right)dz = D_n \int_0^{2H} \sin^2\left(\frac{n\pi z}{2H}\right)dz$$

Definite integrals:

$$\int_0^{2H} \sin\left(\frac{m\pi z}{2H}\right)\sin\left(\frac{n\pi z}{2H}\right)dz = 0$$

$$\int_0^{2H} \sin^2\left(\frac{n\pi z}{2H}\right)dz = H$$

$$\therefore D_n = \frac{1}{H}\int_0^{2H} u_i \sin\left(\frac{n\pi z}{2H}\right)dz$$

$$\therefore u(z,t) = \sum_{n=1}^{\infty}\left(\frac{1}{H}\int_0^{2H} u_i \sin\left(\frac{n\pi Z}{2}\right)dz\right)\sin\left(\frac{n\pi Z}{2}\right)\exp\left(-T\left(\frac{n\pi}{2}\right)^2\right)$$

This is a general equation for the assumed conditions, and allows the excess pore water *u* to be calculated for a soil mass under any initial stress distribution u_i, at any depth *z*, and at any time *t*.

Appendix B:

Consolidation behaviour of soils subjected to asymmetric initial excess pore pressure distributions

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Abstract

Although the consolidation settlements beneath foundations and embankments are rarely one-dimensional, Terzaghi's one-dimensional consolidation theory is often applied to these situations to approximate consolidation behaviour. This paper investigates the consolidation behaviour of a soil strata subjected to various initial excess pore pressure distributions, most of which may occur under foundation and embankment loading. The results show that analysis in terms of average consolidation provides an incomplete representation of the consolidation behaviour. Whilst the average consolidation curves for all constant and linearly varying initial distributions are identical, the isochrones for each distribution are unique. Furthermore, the application of a bottom-skewed excess pore pressure distribution results in a redistribution of pore pressures toward the skewed region so that an increase in excess pressure occurs at some depths after consolidation has already commenced. As a result, conventional consolidation relationships are considered redundant for these cases, and an alternative method of consolidation analysis in terms of normalised pore pressures is proposed.

CE Database subject headings: Pore Pressure; Soil Consolidation; Clays.

Introduction

The consolidation settlements beneath foundations and embankments are rarely one dimensional. Due to the limited extent of loading, in reality, the consolidation is often two-dimensional, in the case of long embankments, or three-dimensional in the case of isolated footings. Common practice is to assume one-dimensional consolidation theory and apply corrections as appropriate. The correction factor, μ, which is dependent upon the shape of the loaded area, was developed by Skempton and Bjerrum (1957) to refine the prediction of ultimate consolidation settlement (Berry and Reid 1988).

Consolidation is inherently linked to the changes in effective stress, which result from changes in pore water pressure as seepage flow progresses towards the drainage boundaries. Upon application of an external load, there is an initial increase in pore water pressure throughout the sample known as the initial excess pore water distribution. According to Darcy's Law, the excess pore water pressures (i.e. pressures in excess of hydrostatic) are the driving force of seepage flow. Flow takes place due to the hydraulic gradient generated by the initial excess pore pressure distribution. At any stage of the consolidation process, the pore water pressures will vary within the soil layer. The distribution of excess pore water pressure at any given time after loading can be represented by an isochrone (Powrie 1997). Thus, the process of consolidation can be expressed as a series of isochrones that graphically represent the relationship between time and the degree of consolidation over the depth of the soil stratum. Consolidation can be further characterised by the average degree of consolidation, which represents the consolidation of the stratum as a whole, and eliminates the variable of depth (Taylor 1962).

A soil subject to constant initial excess pore water pressure is most representative of one-dimensional consolidation, and as such is commonly described in soil literature in terms of both isochrones and average consolidation curves (Taylor 1962, Holtz and Kovacs 1981, Berry and Reid 1988, Powrie 1997, Atkinson 2007, Lancellotta 2009). However, the average consolidation behaviour of other initial excess pore water pressure distributions (such as those in Figure 1) has also been analysed by Taylor (1962), Powrie (1997) and Lancellotta (2009), despite the inherent two- or three-dimensional consolidation behaviour associated with such distributions.

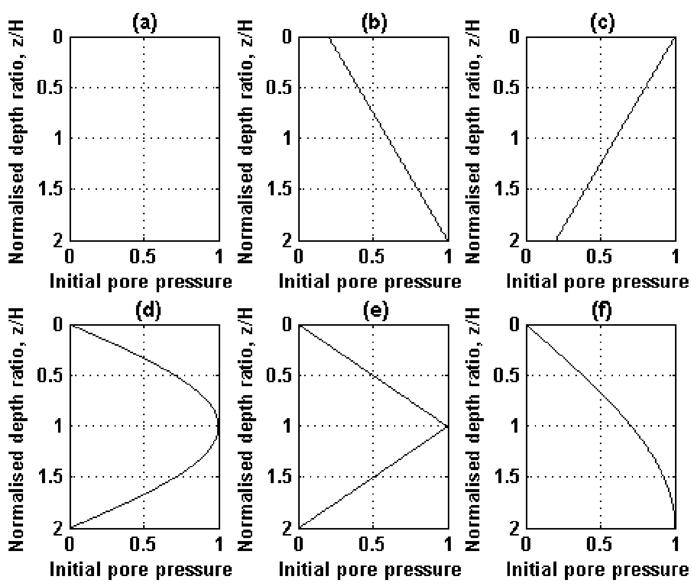

Figure 1 - Standard initial excess pore pressure distributions

Such initial distributions are uncommon under one-dimensional loading, spread to a large lateral extent. Although technically incorrect in these cases, Terzaghi's one-dimensional consolidation theory still provides fundamental insight into the basic consolidation behaviour of soils.

A constant or linear distribution of excess pore pressure is commonly adopted in most consolidation analyses. Janbu et al. (1956) analysed the average consolidation behaviour of a consolidating soil layer with a freely drained upper surface and impermeable base, for linearly increasing and decreasing initial excess pore pressure distributions. The assumption of uniform initial excess pore pressure was adopted by Mesri (1973) for calculations involving the settlement of a consolidating layer separated from freely draining upper and lower surfaces by incompressible layers of finite permeability. Kim et al. (2007) investigated the spatial distribution of excess pore-water pressure induced by piezocone penetration into overconsolidated clays. Simple equations for estimating the consolidation coefficient and final settlement based on any type of linear loading with one-way or two-way drainage have also been proposed by Singh (2008).

The vertical stress increase beneath a loaded area generally peaks at some depth relatively close to the ground level, and decays with depth from then on. Consequently, the resulting initial excess pore water pressure distributions with depth would also be similar to those shown in Figure 2.

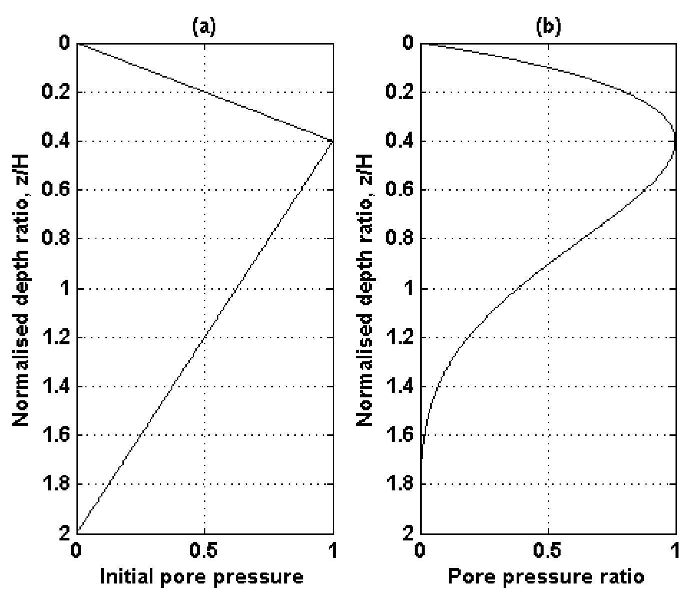

Figure 2 – Initial excess pore pressure distributions with g_{max} at z/H = 0.4

(a) triangular and (b) bottom skewed

One of the objectives of this paper is to investigate the pore water pressure dissipation pattern for this special case, where the initial maximum pore water pressure is represented by a triangle (Figure 2a) with maximum initial pressure occurring at depths of z/H = 0, 0.5 and 1.0. The effects of asymmetric initial distributions are also further investigated by varying the spread of maximum initial pore pressure (Figure 2b).

Series Solution Method

The differential equation governing one-dimensional consolidation and the dissipation of excess pore water pressures is as follows;

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{1}$$

where c_v is the coefficient of consolidation, u is the excess pore pressure, z represents the distance measured downward from the surface of the clay layer and t represents time. The thickness of the stratum is $2H$, with H being the longest drainage path length if doubly drained, as will be assumed for all cases within this paper. In the cases of asymmetric initial excess pore pressure distributions, the dual advantage of analysing singly or doubly drained conditions is no longer available – they must be investigated as two separate cases. The boundary conditions in Table 1 were applied to develop a series solution for the consolidation equation in Eq. (1).

Table 0.1 - Boundary Conditions

Boundary Condition	Mathematical Expression
There is complete drainage at the top of the soil layer.	When $z = 0,\ u = 0$
There is complete drainage at the base of the soil layer.	When $z = 2H, u = 0$
The initial excess pore water pressure distribution is specified as a function of depth.	When $t = 0, u = g(z)$

Upon application of boundary conditions one and two to Eq. (1), the following general solution was obtained:

$$u(z,t) = \sum_{n=1}^{\infty} A_n \sin\left(\frac{n\pi z}{2H}\right) \exp\left(\frac{-n^2\pi^2 T}{4}\right) \tag{2}$$

where,

$$T = \frac{c_v t}{H^2} \tag{3}$$

Series Coefficient Evaluation

Further utilisation of boundary condition three in Table 1 yields the following equation, through which collocation can be applied to solve for coefficient A_n.

$$u(z,0) = g(z) = \sum_{n=1}^{\infty} A_n \sin\left(\frac{n\pi z}{2H}\right) \tag{4}$$

The series coefficients can be evaluated using an orthogonality relationship (Taylor 1962, Holtz and Kovacs 1981). However, a more efficient procedure is to use pseudo-spectral or collocation approach. The series is truncated after *N* terms and forced to satisfy Eq. (4) at *M (=N)* collocation points where $i = 1,...M$ and:

$$z_i = \left(\frac{i}{N+1}\right) 2H. \tag{5}$$

Eq. (4) can be simplified and represented as:

$$g(z_i) = \sum_{j=1}^{N} A_j u_j(z_i) \tag{6}$$

where,
$$u_j(z_i) = \sin\left(\frac{j\pi z_i}{2H}\right). \tag{7}$$

In practice, *M* can be chosen larger than *N*, and this then reduces the collocation to discrete least squares. The system of equations can be solved after multiplication by transpose of the coefficient matrix:

$$U^T \tilde{g} = U^T U \tilde{a} \tag{8}$$

where $[\tilde{g}]_i = g(z_i); [U]_{ij} = u_j(z_i); [\tilde{a}]_j = A_j$. Values of *M* chosen in the range *2N* to *3N* work well, in practice.

Gibbs Phenomenon

The Gibbs phenomenon refers to the erratic behaviour of the Fourier series of a piecewise continuously differentiable periodic function at a discontinuity (Carslaw 1930). When initial excess pore pressure distributions contain discontinuities (i.e. are not zero at the stratum boundaries), the n^{th} partial sum of the Fourier series exhibits oscillations near the discontinuity. This is evident in Figure 3 (a), where a discontinuity of the initial excess pore pressure distribution exists at the base of the soil stratum.

Gibbs phenomenon occurs at a discontinuity when an orthogonality relationship is used to determine the series coefficients. However, this phenomenon can be avoided when using the collocation approach by removing collocation points from the immediate vicinity of the discontinuity, thus allowing the series approximation to vary smoothly over the discontinuity, for example, caused by *g(x)* with discontinuities at *z* = 0 and *z* = 2H. The collocation points are chosen from the interval (*Δ, 2H- δ)* instead of *(0, 2H):*

$$z_i = \Delta + \frac{i(2H - \Delta - \delta)}{N+1}, i = 1,..., N \tag{9}$$

where *Δ* and *δ* are small intervals.

To utilise the properties of the series solution whilst avoiding any error associated with Gibbs phenomenon, boundary values of *z* where a discontinuity occurred were truncated by 3.5%. This virtually eliminated all oscillatory effects of Gibbs phenomenon as demonstrated in Figure 3 (b). To ensure confidence in results based on allocation of M and N number of points, the following RMS error was calculated for each sequence.

$$\varepsilon^2 = \frac{\int_{\Delta}^{2H-\delta} \left(u(z,0) - \sum_{n=1}^{N} A_n u_n(z,0) \right)^2 dz}{\int_0^{2H} dz} \tag{10}$$

The constants, *Δ* and *δ* are small non-zero increments when there is a discontinuity present at *z = 0* or *z = 2H*. When there is no discontinuity present, *Δ* and *δ* are zero.

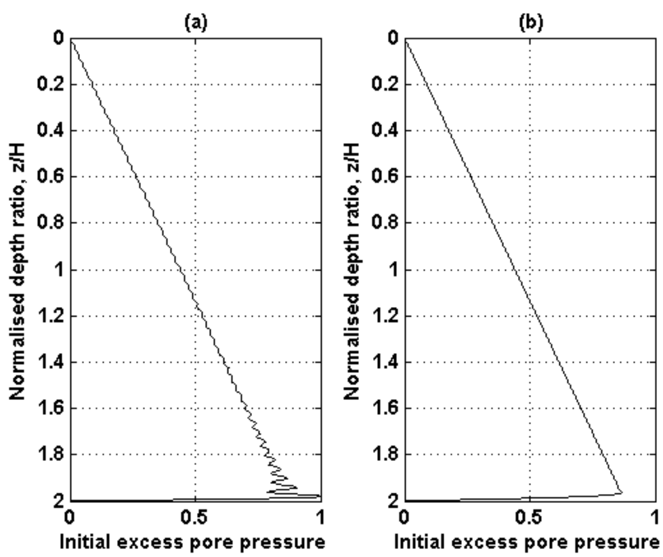

Figure 3 – Gibbs phenonmenon

(a) with and (b) without correction factor

Isochrone Expressions

The consolidation ratio U_z is a measure of the degree to which consolidation has progressed at any depth within a consolidating soil stratum, and is expressed as follows.

$$U_z = 1 - \frac{u(z,t)}{g(z)} \tag{11}$$

Values of U_z range from 0, indicating no consolidation has taken place, to 1 where the soil has completed consolidation under the applied load, which is theoretically only possible at $t = \infty$.

However, the isochrones produced by Eq. (11) do not adequately represent the pore pressure dissipation process when the initial excess pore pressure distribution is also a function of depth, as information is 'lost' during normalisation, where the denominator *g(z)* varies with *z*. Thus, a supplementary expression is proposed in order to capture all information regarding excess pore pressure dissipation and consolidation progress.

$$P_z = \frac{u(z,t)}{g_{\max}} \tag{12}$$

Here, P_z represents normalised excess pore pressure varying the range of 0 to 1, and $g_{\max}$ is the maximum value obtained from the initial excess pore pressure distribution due to the applied load.

Average Consolidation Relationship

The average consolidation U represents the consolidation of the stratum as a whole and is calculated as the average value of U_z over the depth of the soil layer.

$$U = 1 - \frac{\int_0^{2H} u(z,t)dz}{\int_0^{2H} g(z)dz} \tag{13}$$

Standard Initial Excess Pore Pressure Distributions

To date, consolidation ratio isochrones are only provided for the case of constant initial excess pore pressure. Powrie (1997) does, however, highlight the dissipation of excess pore pressure that results from a triangular initial distribution where one impermeable boundary exists at the base of the soil layer. Consequently, past analysis of excess pore pressure dissipation based on initial distributions described in Figure 1 have only been in terms of average consolidation ratios (with the exception of the constant initial distribution in Figure 1a). Figures 4 through 10 show the pore pressure and consolidation ratio isochrones corresponding to initial excess pore pressure distributions (a) to (f) in Figure 1 for T = 0.1, 0.2 through to 1.0. The error (Eq. 10) associated with each isochrone figure is of the order

10^{-7}. The dashed line in each figure represents the initial excess pore pressure distribution, normalised by its maximum value.

Isochrones

Figure 4 shows the pore pressure and consolidation isochrones where the initial excess pore pressure is constant at all depths.

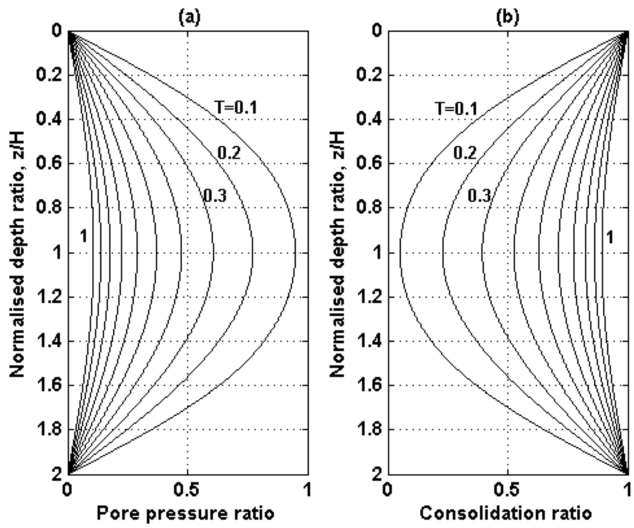

Figure 4 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(a)

Since the initial distribution is constant for the entire stratum depth, the normalised pore pressure isochrones are the same as those reported in textbooks. However, in Figures 5 and 6, where the initial excess pore pressure distributions are linear, the normalised pore pressure isochrones provide information regarding the skewed dissipation of excess pore pressure which is not clearly shown in the accompanying consolidation ratio isochrones.

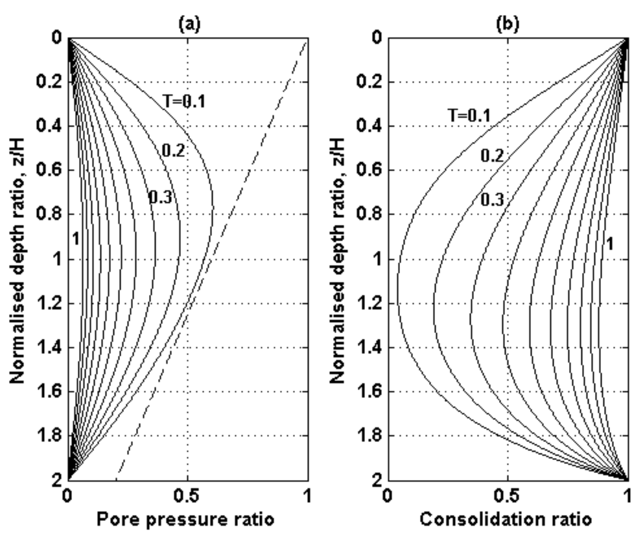

Figure 5 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(b)

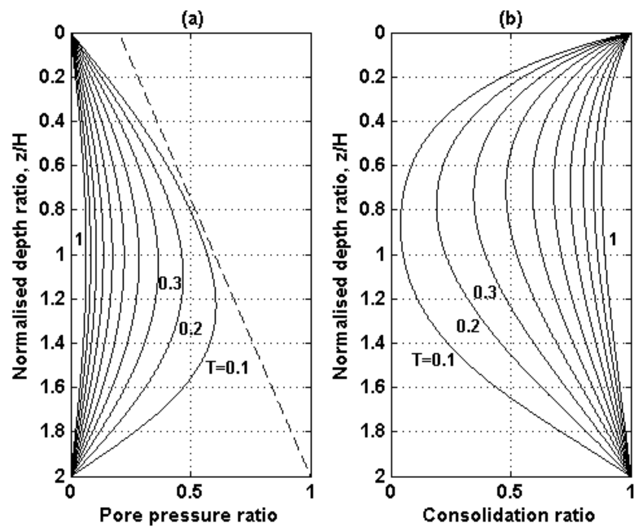

Figure 6 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(c)

Thus, in the case of non-uniform initial pore pressure distributions, the term consolidation ratio is of little use. It simply states the percentage of initial pore pressure that has dissipated – a greater consolidation ratio does not always indicate higher pore water pressures. Therefore, it is more suitable to present consolidation ratio isochrones in conjunction with normalised pore pressure isochrones for a comprehensive understanding of the pore pressure dissipation process.

In reality, the consolidation behaviour of a soil stratum will not always be analysed after immediate application of the design load. If analysis takes place after some unknown time has elapsed, the 'new' initial distribution will be sinusoidal, and consolidation analysis would take place accordingly. Interestingly, the consolidation isochrones generated by a sinusoidal initial pore pressure distribution are independent of depth (Figure 7). This is obvious upon examination of Eq. (11) – the sinusoidal component of time-dependent and initial excess pore pressures is eliminated so that the consolidation ratio becomes a function of time only.

The symmetrical triangular initial distribution highlighted in Figure 8 produces consolidation isochrones that indicate that consolidation proceeds fastest at the centre of the soil stratum. Although unclear near the top and bottom boundaries in Figure 8, the consolidation isochrones do comply with the initial boundary conditions that specify two drainage boundaries – each isochrone tends to 100% consolidation exactly at the top and bottom of the stratum. This immediate tendency toward 100% consolidation is observed in every case where the initial excess pore pressure is zero at the drainage boundary, as in Figures 1 (d) through (f). This trend is further highlighted by results in Figure 9, where consolidation ratio isochrones are provided for three linearly increasing cases of initial excess pore pressure distribution. The linear distributions are adjusted to approach an origin boundary at the top of the soil stratum. Figure 9 (a) shows linearly increasing pore pressure based on an initial normalised pore pressure value of 0.2 whilst figure 9 (b) demonstrates the skewed isochrones that result from a reduction in initial value to 0.001. Based on Figure 9 (b), it can be concluded that the isochrones in (c) each tend toward 100% consolidation exactly at the top stratum boundary. The half-sine initial excess pore pressure distribution in Figure 10 contains one origin boundary so that at the surface of the soil stratum, consolidation is complete, in compliance with boundary conditions.

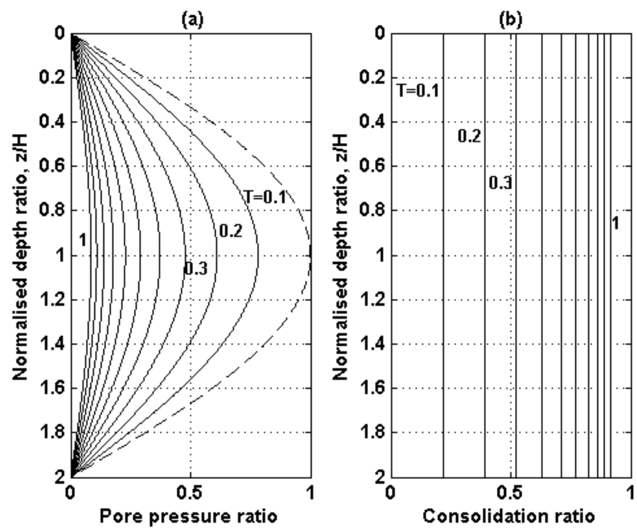

Figure 7 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(d)

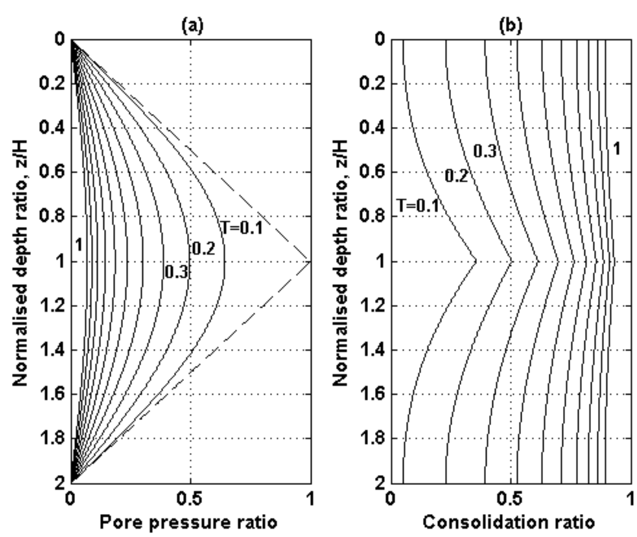

Figure 8 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(e)

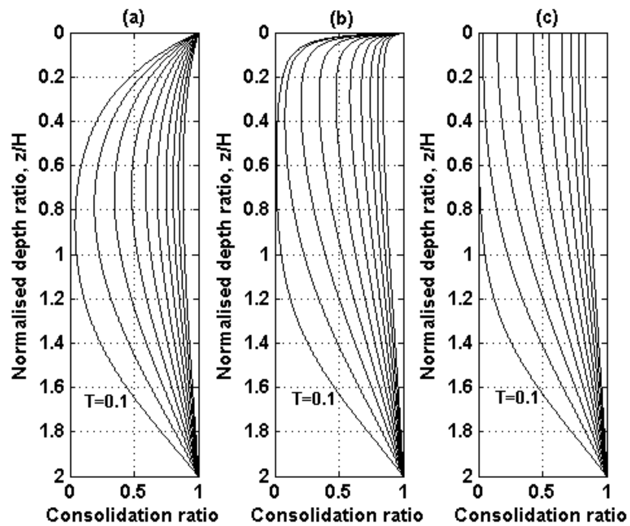

Figure 9 – Consolidation ratio isochrones for linearly increasing excess pore pressure distributions

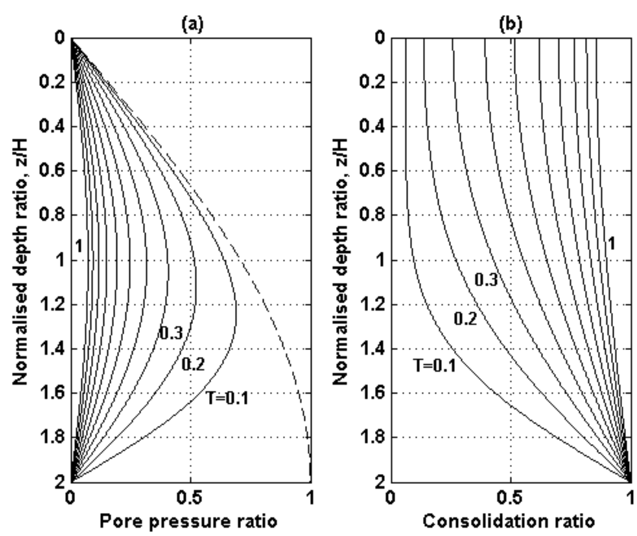

Figure 10 – Pore pressure and consolidation ratio isochrones for initial distribution in Figure 1(f)

Average Degree of Consolidation

The average degree of consolidation curves for six standard initial distributions of excess pore pressure are shown in Figure 11. The consolidation behaviour of the soil stratum as a whole proceeds much slower when an initial sinusoidal variation is applied, and only slightly surpasses the behaviour of a symmetric triangular distribution. Most of the initial excess pore pressure is distributed within the mid-layer and hence it is understandable that these two cases will take longer to consolidate.

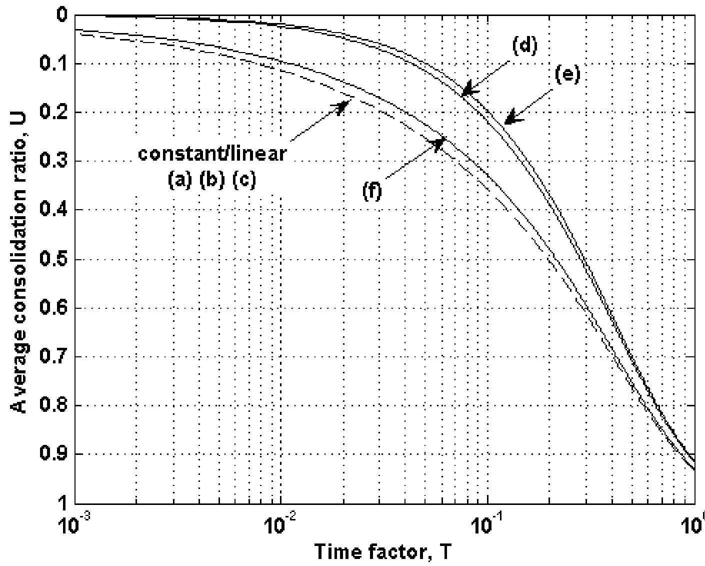

Figure 11 – Average consolidation curves for standard initial distributions

As reported in literature (Taylor 1962), there is a unique average consolidation curve that encapsulates the consolidation behaviour for all constant or linear initial pore pressure distributions (shown in Figure 1 by cases a, b and c). However, the isochrones for these cases are far from identical, and it is therefore recommended that average consolidation curves be considered an additional rather than absolute source of information regarding consolidation behaviour.

Asymmetric Initial Excess Pore Pressure Distributions

The consolidation behaviour of soils is commonly analysed based upon symmetric initial excess pore pressure distributions, where the added advantage lies in the ability to analyse in terms of single or doubly drained strata, using the same graphs. However, the practical relevance of consolidation analysis in terms of asymmetrical excess pore pressure distributions must be considered.

The vertical stress increase beneath a loaded area generally peaks at some depth comparatively close to the soil surface, and subsequently decays with depth (Ranjan and Rao 1991). As a result, the excess pore pressure distribution with depth would resemble Figure 2 (b). The effect of varying the point of maximum excess pore pressure when the distribution is represented by a triangle (Figure 2a) was investigated. Pore pressure and consolidation ratio isochrones where the maximum initial pressure occurs at depths of z/H = 0, 0.5 and 1.0 are shown in Figures 12, 13 and 8, respectively. As expected, consolidation proceeds fastest at the depth at which the peak initial pressure is located. The accompanying average consolidation curves for these cases are shown in Figure 14, with the additional case of maximum pore pressure at z/H = 0.3 also included to further emphasize the trend. As the initial excess pore pressure peaks further from a drainage boundary, the consolidation progresses much slower. The case with z/H = 0 is the same as the average consolidation for constant/linear distributions in Figure 1 (a), (b) and (c); the case with z/H = 1 is the same as Figure 1 (e).

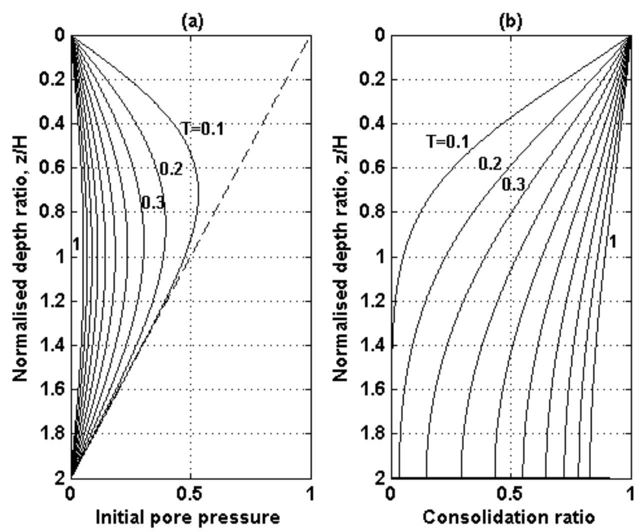

Figure 12 – Pore pressure and consolidation ratio isochrones for initial distribution (a) $z_{max}/H = 0$

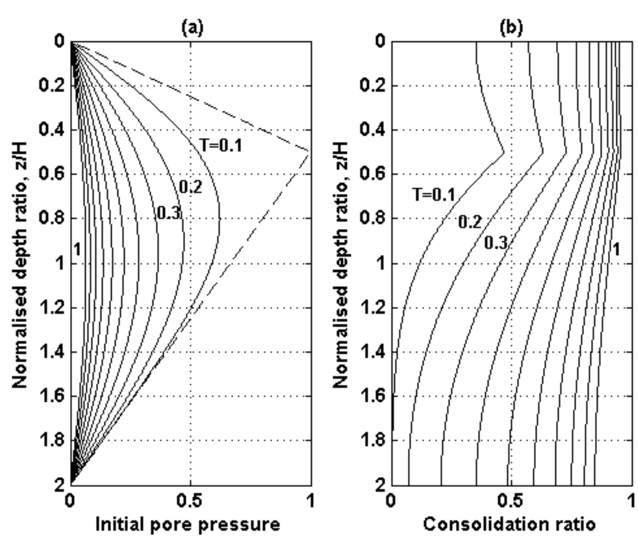

Figure 13 – Pore pressure and consolidation ratio isochrones for initial distribution (a) $z_{max}/H = 0.5$

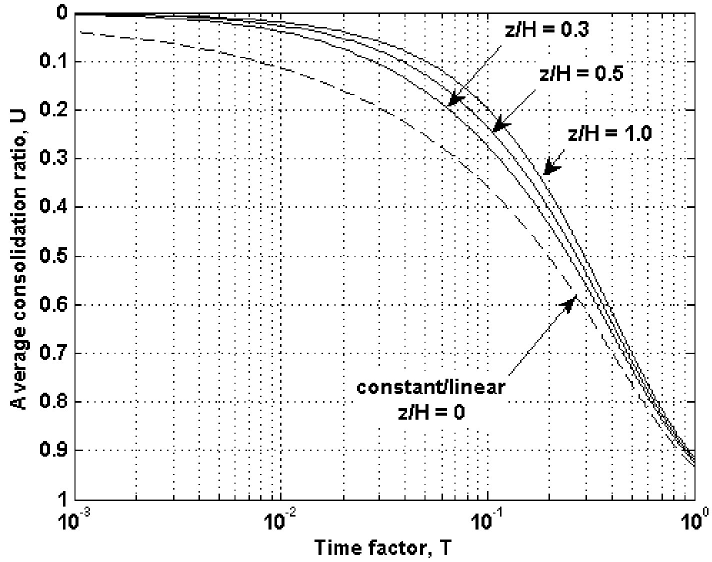

Figure 14 – Average consolidation curves for triangular initial excess pore pressure distributions

To further investigate the effects of asymmetric initial distributions, the degree of spread in the stress distribution shown in Figure 2 (b) was varied. The depth of maximum initial pore pressure and the normalised depth at which this occurred are held constant at *z/H* = 0.4 so that the degree of spread (*b*) is the only variable analysed. The initial pore pressure variation of the bottom-skewed distribution was represented by the following equation;

$$g(z) = \frac{z^a (2-z)^b}{c} \tag{14}$$

where constants $a = 0.25b$ and $c = 10^{0.105b}$ ensure that the initial distribution maintains a constant maximum pore pressure at *z/H* = 0.4. As demonstrated in Figure 2 (b), the majority of excess pore pressure is concentrated within a relatively narrow region from *z/H* = 0.2 to 0.8. The spread or skewness of this region is controlled by increasing/decreasing the variable *b*.

The pore pressure isochrones for initial distributions where *b* = 2, 6 and 12 are shown in Figure 15. Interestingly, a redistribution of excess pore pressures occurs toward the region of minimal initial excess pore pressure, which is a direct result of the concentrated nature of the initial excess pore pressures toward the surface drainage boundary. This phenomenon is briefly mentioned in Taylor (1962) for a decreasing linear initial distribution with an impermeable base layer – during the initial stages of consolidation, downward flow results in a transient increase in excess pore pressure near the impermeable base of the stratum. This increase gradually decays as the pore water eventually drains through the top of the layer.

However, the results in Figure 15 for bottom-skewed initial pore pressure distributions within a doubly drained system, demonstrate the prevalence of pore pressure redistribution for the majority of the consolidation process. This effect becomes more pronounced as the spread of maximum initial pore pressure decreases (i.e. *b* increases). Near the bottom boundary, the pore pressures exceed the initial pore pressure, due to the redistribution.

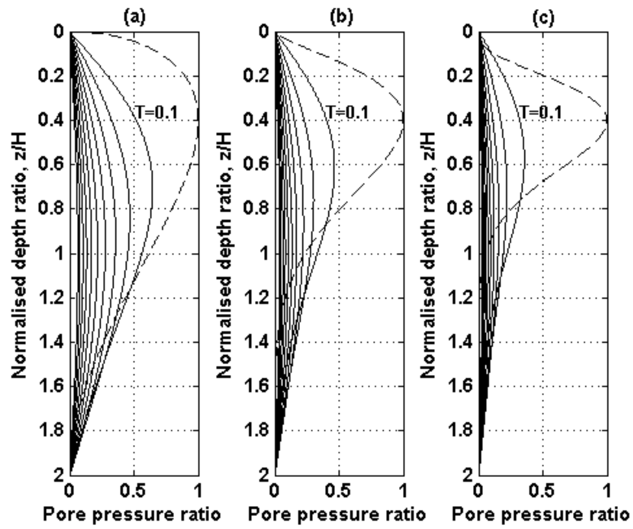

Figure 15 – Pore pressure isochrones for bottom skewed initial excess pore pressure distribution (a) b = 2, (b) b = 6 and (c) b = 12

The phenomenon of excess pore pressure redistribution when a soil stratum is subjected to a bottom-skewed initial distribution is of significance when calculating the degree of consolidation. As evidenced by Eq. (11), the degree of consolidation relationship is dependent upon a fundamental assumption that the initial excess pore pressure will always be greater than the corresponding pore pressure isochrones – a negative degree of consolidation is not feasible. Here, the importance of describing the consolidation process in terms of pore pressure isochrones rather than consolidation ratio isochrones becomes evident.

The average consolidation curves for increasing values of *b* are provided in Figure 16. A soil stratum subject to bottom-skewed initial excess pore pressure distributions consolidates slower than that subject to constant/linear distributions. Furthermore, an increase in *b* results in a decrease in rate of consolidation for time factors approximately less than 0.04 – the rate of consolidation then appears to increase as *b* increases.

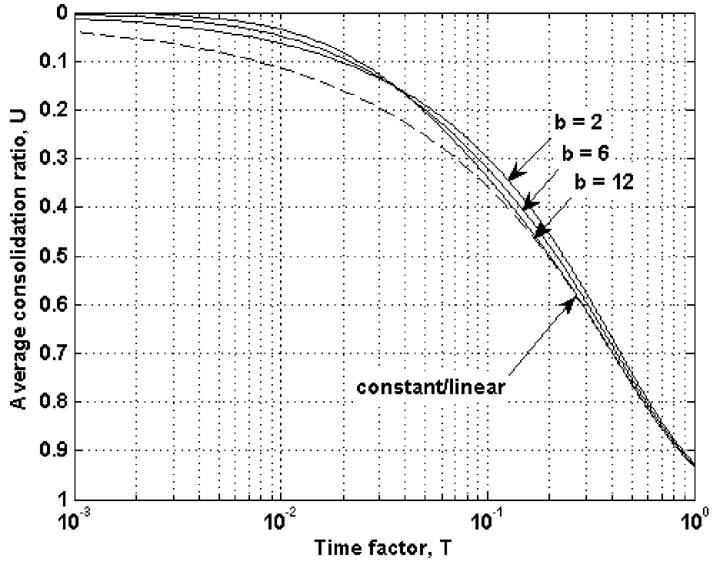

Figure 16 – Average consolidation curves for bottom skewed initial excess pore pressure distributions

The phenomenon of pressure redistribution is not isolated to bottom-skewed pore pressure distributions, as demonstrated by Figure 17, where excess pore pressure isochrones are shown for symmetric cases where b = 2, 6 and 12. Thus, it is evident that pressure redistribution occurs in regions of minimal initial excess pressure, regardless of the location of concentrated initial excess pore pressure.

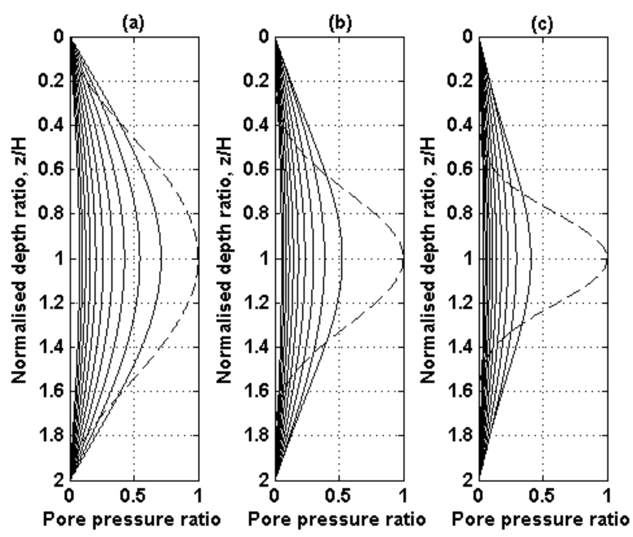

Figure 17 – Average Pore pressure isochrones for skewed initial excess pore pressure distribution

(a) b = 2, (b) b = 6 and (c) b = 12

Conclusion

An analysis of consolidation behaviour of a soil stratum subject to varying distributions of initial excess pore pressure was conducted in terms of normalised pore pressure and degree of consolidation isochrones. The application of the series solution method with assistance from the program MATLAB was suitable for this analysis. The results show that the average consolidation ratio curves used to assess the consolidation behaviour of standard initial pore pressure distributions in common soil literature exclude important information that is only available upon examination of the isochrones.

The six cases of standard initial pore pressure distributions discussed in literature were revisited and results indicate that the term consolidation ratio alone is inadequate in fully describing the consolidation process. Thus, for a complete description of consolidation behaviour, it is suggested that consolidation ratio isochrones be viewed in conjunction with pore pressure isochrones, both of which take quite different shapes. The consolidation ratio and pore pressure ratio isochrones are only identical for the case of initial uniform pore pressure distributions, one being the mirror image of the other. For soil subject to an initial sinusoidal pore pressure distribution, the consolidation ratio remains constant at all depths.

The average consolidation ratio plots were developed for several symmetric and asymmetric initial pore pressure distributions. While these plots do not provide information regarding consolidation at a definitive depth, they are useful in estimating the consolidation settlement at a specific time. Although a wide range of pore pressure distributions were investigated, all average consolidation curves fell within a relatively narrow band, with maximum spread near T = 0.1. Furthermore, it is evident that when the initial pore pressure is concentrated near the mid-layer, the rate of consolidation is slower. When the majority of initial pore pressure is located near the drainage boundaries, the dissipation is comparatively faster.

Upon investigation of the effects of asymmetric initial pore pressure distributions that resemble the stress distribution underneath a loaded footing, the phenomenon of pressure redistribution during consolidation was observed. When the majority of initial excess pore pressure was concentrated within one region, a redistribution of excess pore pressures toward the region of minimal pore pressure occurred during initial stages of consolidation. This increase gradually decayed and subsequent consolidation conformed to traditional dissipation models. As a result, the conventional relationship for degree of consolidation was considered inappropriate for cases of skewed initial pore pressure distributions, and analysis is recommended in terms of normalised pore pressure isochrones and average consolidation curves.

Notation

The following symbols are used in this paper:

A_n = series coefficient;

a = constant;

$\tilde{a}$ = coefficient matrix;

b = degree of spread;

c = constant;

c_v = coefficient of consolidation;

g = initial excess pore pressure;

$\tilde{g}$ = initial excess pore pressure matrix;

H = length of drainage path for doubly-drained strata;

M = number of z terms;

N = number of terms;

P_z = normalized pore pressure;

T = time factor;

t = time;

U = average consolidation;

U_z = degree of consolidation along depth of soil strata;

u = excess pore pressure;

z = depth;

ε = RMS error; and

μ = settlement correction factor based upon shape of loaded area.

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