Have we understood enough on consolidation theory?

Myint Win Bo
DST Consulting Engineers Inc., Thunder Bay, Ontario, Canada

ABSTRACT
Theoretical soil mechanics started with consolidation theory in the early 19th century (Terzaghi & Frohlich 1936). Many settlement and soil deformation problems were solved and overcome by this classic theory. However the process of consolidation was simplified in the theory with several necessary assumptions made to solve the problem reasonably. This simplification also led to a serious argument between two prominent academics and the tragic story of the suicide of Fillunger and his wife in 1937 (Gibson et al. 1995). Therefore there is no doubt that this theory and the process of consolidation is not a simple problem but one that is very very complex. Is this theory non-linear or linear? Does the same type of soil have the same unique consolidation behaviour? Is there in fact any such pressure like preconsolidation pressure, and can we really determine this pressure in the laboratory? These questions will be discussed and answered in this paper. In addition to answering these questions, this paper will examine the complexity of this theory's elements, such as time, strain rate, stress, loading rate, and temperature dependent consolidation behaviours. This paper will also explain the magnitude of secondary compression and it's starting time with a few interesting examples drawn from theoretical and experimental studies. Finally, this paper will so explain the factors affecting the consolidation process and the limitations for its application to practical problems.

RÉSUMÉ
La mécanique des sols théorique a débuté avec la théorie de la consolidation, au début du 19ème siècle (Terzaghi & Frohlich, 1936). Plusieurs problèmes d'affaissement et de déformation des sols furent résolus et surmontés par cette théorie classique. Toutefois, le mécanisme de consolidation fut simplifié dans cette théorie par l'application de plusieurs postulats, de façon à résoudre le problème raisonnablement. Cette simplification mena à un sérieux désaccord entre deux importants académiciens et au suicide tragique de Fillunger et de son épouse en 1937 (Gibson et al 1995). Par conséquent, il n'y a peu de doutes que cette théorie, ainsi que le mécanisme de consolidation, n'est pas qu'un simple problème mais plutôt un qui est très complexe. Est-ce que ce mécanisme de consolidation est linéaire ou non-linéaire? Est-ce que le même type de sol aura le même comportement de consolidation singulier? Y a-t-il en réalité une contrainte telle que la contrainte de pré-consolidation, et pouvons-nous réellement déterminer cette contrainte en laboratoire? Ces questions seront examinées et répondues dans cet article. En plus de répondre à ces questions, cet article explorera la complexité des éléments propres à cette théorie, tels que les comportements de consolidation en fonction du temps, du taux de contrainte, d'effort, du taux de chargement et de la température. Cet article expliquera également l'importance de la compression secondaire ainsi que de son temps de départ avec quelques exemples intéressants tirés d'études théoriques et expérimentales. En terminant, cet article expliquera les facteurs influençant le mécanisme de consolidation et les limites de son application aux problèmes pratiques.

1 INTRODUCTION

One dimensional consolidation theory was developed by Terzaghi between 1919 and 1923. It was based on experimental tests conducted on a thin layer of soil and a mathematical formulation which driven with many assumptions was known to simplify the engineering behaviours of compressible soil. However many geotechnical engineers have benefited from applying this theory. With the precise measurement of soil compression behaviour in the laboratory on a thin layer of soil under one dimensional loading, engineers have managed to predict in some cases, very closely the performance, whereas in many cases predictions are off by more than +/- 20 % variation even with very good judgment. This variation is due to many factors, which have an affect on the consolidation process. Some of these factors are; i) the behaviour of consolidation in the field has departed from the assumptions made in the 1-dimensional theory, ii) the environmental conditions differ from the laboratory conditions under which the compression and consolidation parameters are obtained, iii) the rate of loading and the rate of strain differ between laboratory and field conditions iv) more importantly the limitations of the one dimensional theory itself and many others. This paper will discuss the evolution of consolidation theory and its limitations to application in the engineering practice. These limitations are discussed with many examples drawn from laboratory tests carried out on Singapore Marine Clay at Changi. The details on geotechnical characteristics of Singapore marine clay can be found in Bo et al. (2003).

2 HISTORY AND EVOLUTION OF CONSOLIDATION THEORIES

Terzaghi presented his consolidation theory to the world in 1925 with his book entitled "*Erdbaumechanich*", which means soil mechanics in the English language. The one dimensional consolidation theory he proposed has following assumptions:

- Soil is homogeneous
- Soil is 100 % saturated
- Soil skeleton and water are incompressible
- Compression is one dimensional vertically
- Flow of water is only one dimensional as is the same with the direction of gravitational forces
- Flow characteristics comply with Darcy's law
- Soil compression is linearly related to the effective stress gain(But only in a narrow range of stress)
- Stress-strain behaviour of soil is linear and elastic (Again, only over small increments in stress)
- Coefficient of consolidation (C_V) is assumed to be constant throughout the consolidation process

In addition, his theory was based on the compression behaviour of a thin layer of soil under small strain.

It has been documented by Gibson et al. (1995) that consolidation theory has created the Terzaghi and Fillunger affair, which led Fillunger and his wife to commit suicide.

As Terzaghi's theory was unable to solve for the large strain consolidation process under which the characteristics of compressible soils are changing during the consolidation process, Gibson et.al. (1981) proposed the large strain theory, which takes into consideration the changes of the soil parameter during the consolidation.

Again Carrillo (1942) and Barron (1948) proposed another consolidation theory, which takes into consideration both radial and vertical flow, which is generally much faster than vertical flow alone.

Biot in 1955, (Cryer 1963) presented a more comprehensive theory, which took into consideration the flow towards three dimensions with 3-D deformation.

Consolidation generally starts with the level of effective stress, which is equivalent to the overburden stress, therefore this theory is unable to realistically solve the problem of self-weight consolidation in which natural deposit of soil is still undergoing consolidation with stresses caused by its self-weight. Been and Sills (1981), Lee and Sills (1981), Mikasa (1961) have extensively studied self-weight consolidation and proposed the self-weight consolidation theory.

However in many cases the application of load may be applied before completion of the self-weight consolidation. Compression and deformation of soil upon additional load, which is still undergoing self-weight consolidation, is different from deformation of normally consolidated natural soil. Therefore application of Terzaghi Theory may be invalid (Bo et al. 1999, 2005 & Bo 2002, 2008). Therefore Bo in 2002 & 2008, Bo et al. 2004 proposed a model and a few sets of equations which can solve the compression and consolidation characteristics of ultra-soft soil, which usually have a natural moisture content greater than the liquid limit.

Casagrande in 1936 proposed a method of determining preconsolidation pressure using void ratio (e) vs. log of effective stress (σ') data from oedometer tests, which can differentiate the behaviours between compression and recompression.

Casagrande in 1936 and Taylor & Merchant in 1940 proposed a method for the determination of the end of primary consolidation and the commencement of secondary compression, based on the experimental study of, again, a thin layer of soil.

3 COMPRESSIBILITY

The magnitude of compression may have a few components, such as elastic compression, primary and secondary consolidation. Elastic compression will not be discussed in this paper. In order to be able to predict the magnitude of settlement both in primary and secondary consolidation, laboratory consolidation tests are usually carried out on a thin layer of soil, generally about 19 mm thick under saturated condition. Test results are then processed, in order to obtain an e log σ' curve. From these curves, indices called the compression index, the recompression index, and yield stress are determined. From the settlement vs. log time or root time graph, the coefficient of consolidation (C_V) is determined. A secondary compression index can also be determined from the e log t curve.

3.1 Magnitude of strain

The magnitude of strain, in other words, the magnitude of settlement is increasing with an increase in compression index. However this compression index varies with the types and method of tests being carried out.

3.2 Loading rate

In standard practice, a load increment ratio of unity was applied in laboratory consolidation tests. The compression index generally increases with the ratio of load increment (Fig. 1). Based on experimental tests carried out on Changi Marine clay, it was found that the compression indices were also increasing with the rate of loading. In the field, the rate of loading could vary depending upon the construction schedule. Therefore, the magnitude of settlement can vary depending upon the rate of loading even for the same type of soil with the same initial effective stress condition.

3.3 Strain rate

It is known that the deformation of soil is strain rate dependent. Settlement rates in the field could vary depending upon drainage condition, temperature and also the magnitude and rate of loading. Strain rate effects have been reported by Leroueil et.al (1985 & 86). Based on the experimental tests carried out on the Changi marine clay, it was revealed that the compression index was increasing with the rate of strain and hence the magnitude of settlement (Table 1).

3.4 Stress Range

Compression index is not a unique constant value for a same type of soil. It is stress dependent. Compression indices generally increase with stresses increase. Only at very high stress level compression indices decrease again (fig 2).

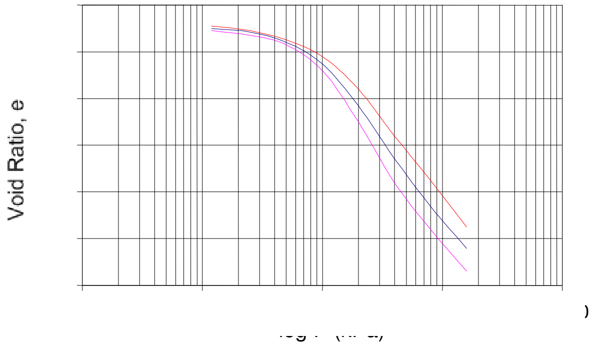


Figure 1: Increasing Compression Index with ratio of loading increment (After Bo and Choa, 2004 also in Das 1983)

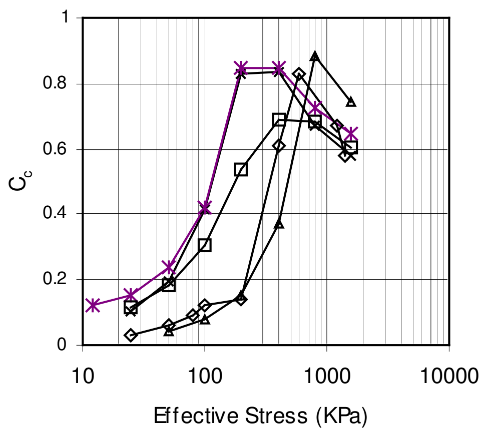


Figure 2: Variation of compression indices with stresses (After Bo and Choa, 2004)

3.5 Loading duration

It has been known that a secondary compression starts after the primary consolidation. However a loading duration of 24 hours was applied in a standard consolidation test, which includes significant magnitude of secondary compression. This results a high compression index (See Figure 3).

3.6 Temperature affect

It has been demonstrated by Darcy's law that the rate of flow is increasing with a reduction in viscosity and that the viscosity of pore water is decreasing with an increasing temperature. Hence the consolidation rate or strain rate increases with increasing temperature.

This leads to an increased compression index, in other words, the magnitude of settlement increases with an increasing temperature. This temperature effect is shown in Figure 4 (Leroueil 1999, Marques 1996).

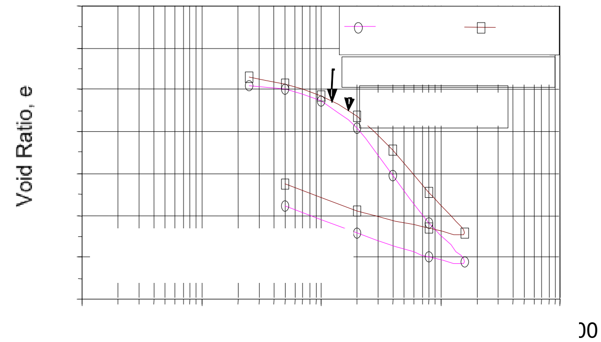


Figure 3: Loading Duration Effects (After Bo and Choa, 2004)

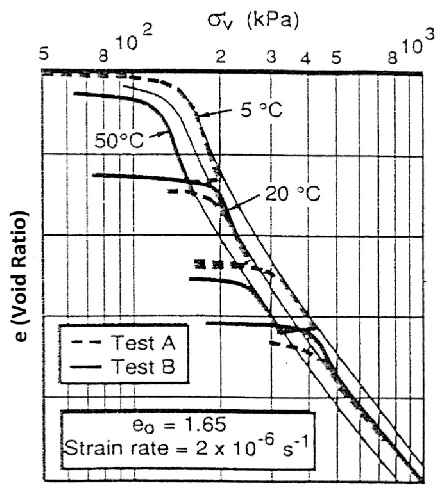


Figure 4: Effects on one-dimensional compression of St-Polycarpe clay of temperature (after Marques 1996)

4 YIELD STRESS

Yield stress is the transition stress, which divides the recompression range and the virgin compression range. This yield stress can be determined by applying the Casagrande graphical method. However it has been known that this Casagrande method is not true representative of the yield stress. Many alternative methods have been proposed by various researchers. Among others, methods proposed by Butterfield (1979), Janbu et.al (1981), and Sridharan & Sreepada (1981) are used in the industry. However these methods do not yield the same result for the same set of data from the same test (Nash et.al 1992, Bo et.al 2003, Bo & Choa 2004) (Table 2).

4.1 Loading rate

As like compression indices, yield stress also increases with the loading rate.

Table 1: Comparison of Compression and Recompression Indices from Various Tests. (After Bo & Choa 2004)

	24 hrs	EOP	RC(1)	RC(3)	RC(4)	CRS (0.5 mm/h)	CRS (1 mm/h)	CRS (1.5 mm/h)	CRS (2 mm/h)
Cc	0.86 (0.95)	0.76 (0.79)	0.98	0.89	0.89	0.91	1.15	0.94	1.23
Cr	0.07 (0.07)	0.06 (0.06)	0.12	-	-	-	-	-	-

Note: EOP = End of primary, RC = Radial drainage, CRS = Constant Rate of Strain, 1 = Radial inward, 3 = radial outward, 4 = both radial inward and outward

4.2 Strain rate

Yield stress is known to be affected by the strain rate as shown in Table 2 and Fig 5. This effect was also extensively discussed by Leroueil et.al (1985 & 86).

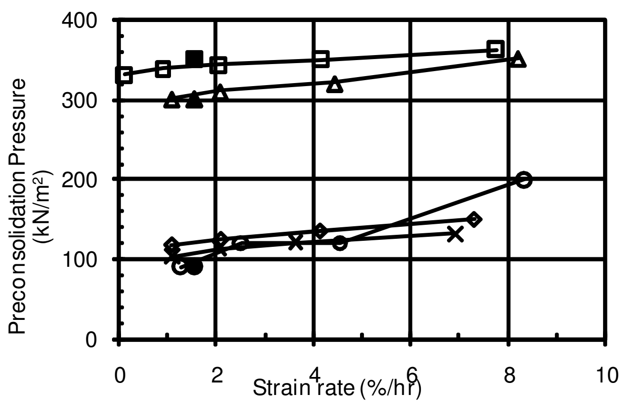


Figure 5: Strain Rates Effect on Yield Stress (After Bo and Choa, 2004)

4.3 Temperature affect

Leroueil et.al has carried out extensive laboratory tests on soft clay, and demonstrated that the e log σ' curve is affected by temperature variation. Yield stress is increasing with a reducing temperature (Fig. 4).

4.4 Scale effect

The determined yield stress value can be affected by the scale used, it has been demonstrated by Mikasa (1995) and Bo and Choa (2004). Even for the same set of results plotted on different scales, the results will yield varied stress values. Even with the same results plotted with different engineering units, the same yield stress will not be obtained.

It has been known that the preconsolidation pressure is a pseudo pressure and it is uncertain that it will truly represent the yield stress of the soil. Even the soil, which is settling linearly with an applied load could result in a so called preconsolidation transition when it is plotted on a log scale and determined by applying the Casagrande method (Fig 6).

Geotechnical engineers generally have a misconception that soils will settle only to a small magnitude in the stress range below pre-consolidation pressure and to a high magnitude in the high stress range. In reality if the void ratio changes vs. stresses are plotted on a mathematical scale it is obvious that soils settle at a greater magnitude in the low stress range and at a lower magnitude in the high stress range.

5 OTHERS

5.1 Large strain effect

In practice, a set of initial void ratios and compression and recompression indices are applied in predicting the magnitude of settlement. In reality void ratios and compression indices change during the consolidation process, thus makes the magnitude of settlement lower than the predicted settlement using the initial void ratios.

5.2 Submergence effect

In many cases, additional fill is placed just above the groundwater level. Therefore initially the placed fill has a full load as the fill profile is above the water level.

However during the consolidation process the fill will sink below the water level and part of the fill load will become a submerged load.

This fill load reduction will affect the final magnitude of the settlement and time rate of settlement.

5.3 Non-linear effect

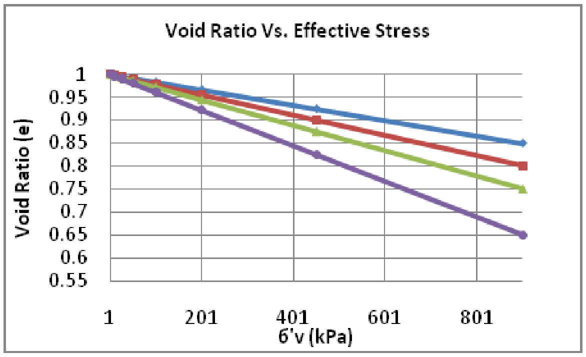
In many design processes, the time rate of consolidation is calculated using a single value of C_V, as conventional theory cannot handle variations of the soil parameters within the layer as well as variation of the values during the process of consolidation.

The effective stresses and void ratios of the soil vary with depth and hence the C_V will also vary with depth. Bo & Choa (2004) described an equivalent thickness method to treat multi-layer soils with variations in C_V values.

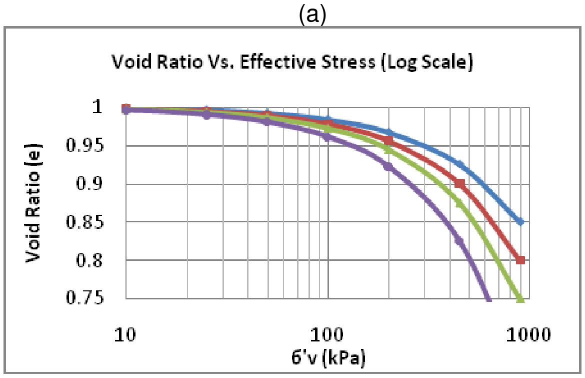
C_V values reduce with effective stress, as well as there are a tenfold differences between the magnitude of C_V in the compression range and the recompression range. Therefore during the consolidation process the soil will settle at a much faster rate following higher rate of consolidation in the recompression range and the settlement rate will become much slower following lower rate of consolidation in the compression range. This non-linearity was not taken into consideration in predicting time rate of settlement using conventional calculation. The difference in the predicted time rate of consolidation due to these various factors is shown in Fig. 7.

Table 2: Comparison of preconsolidation Pressure from various types of tests and various interpretation methods (After Bo & Choa 2004)

Method	24 hrs	EOP	CRS (0.5 mm/h)	CRS (1 mm/h)	CRS (1.5 mm/h)	CRS (2 mm/h)
Casagrande	65 (201)	71 (256)	90	120	120	200
Janbu	94 (436)	92 (454)	-	-	-	-
Butterfield	46(203)	57 (222)	-	-	-	-
Sridharan	58(222)	69	-	-	-	-



(a)



(b)

Figure 6: Preconsolidation Transition due to Logarithmic Scale (a) Mathematical scale (b) Log scale

5.4 Sub-division effect

Many engineers have estimated the magnitude of settlement by crudely applying the effective stress gain at the centre of the soil layer.

Although it may not have a significant affect on a thin layer of soil, it has significant affect on the magnitude of settlement predicted on thick layer of soil. Terzaghi's effective stress gain theory has a multiplier of stress increment ratio. The higher the stress increment ratio, the greater is the magnitude of settlement for the same soil with the same thickness. Even for a homogeneous soil with the same compressibility parameters, the predicted magnitude of settlement is increasing with an increasing number of divided sub-layers Bo & Choa (2004) (Fig.8).

5.5 Self-weight consolidation

Settlement and pore pressure behaviours of ultra-soft soil upon application of additional load are different from natural soil. These types of ultra-soft soils can be found in either man-made fill, mine tailing or even in recently deposited young clay, which are still undergoing self-weight consolidation. Upon application of additional load, ultra-soft soil generally undergoes excessive settlement without dissipation of pore pressure or in other words without gaining an effective stress (Bo et.al 1999, 2005, Bo 2002, 2008). Bo 2002 & 2008 and Bo et.al 2005 has proposed a set of equations to predict magnitude and time rate of settlement for such soils upon additional load. These becomes supplement of self weight consolidation studied by Been & Sills (1981) and extension of Terzarghi effective stress gain theory on natural soil (Terzaghi 1925 & Terzaghi & Frohlich 1936).

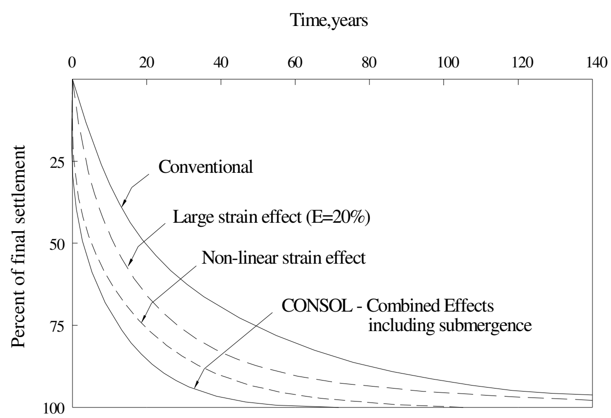


Figure 7: Differences in Predicted Time Rate (After Wong and Choa 1987, Bo and Choa, 2004)

6 SECONDARY COMPRESSION

6.1 End of primary consolidation

Taylor and Casagrande both proposed methods for the determination of the end of primary consolidation based on the experimental results conducted on a thin layer of soil consolidated within an oedometer. It was known that secondary compression started after the primary consolidation, however that is only true for a single soil element or a thin layer of soil. For a thick layer of soil Leroueil (1999) pointed out the secondary compression could start as early as when the primary consolidation reaches 60%. It is true that for a thick layer of soil, when the soil elements at the midpoint of thickness reaches 50 to 60 % degree of consolidation, the soil elements near the drainage layer would have completed their primary consolidation and might have been undergoing secondary compression. Therefore during the process of consolidation soil elements are undergoing different degree of consolidation and different stages of compressions such as primary and secondary.

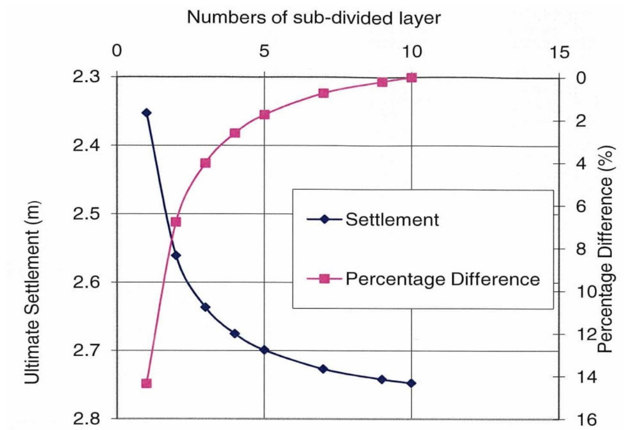


Figure 8: Sub-Division of Layers Effect on Normally Consolidated Clay (After Bo and Choa, 2004)

6.2 Stress dependent secondary compression

As like compression index, secondary compression index (C_α) is also stress dependent. Usually C_α values increase with increasing stresses until yield stress level and reduce with stress levels beyond that (Fig. 9).

6.3 Tertiary compression

The secondary compression index is usually determined from a void ratio change after primary consolidation from the subsequent log cycle of e log t curve. In reality many soils will settle more than a log cycle after primary consolidation. In the next log cycle, the soil will settle at a slower rate with a steeper gradient in the next log cycle. This higher index in the next log cycle has been termed "tertiary compression" by Dhowian and Edil, 1980 & Candler & Chartres 1988. This phenomenon is shown in Fig. 10. Therefore secondary compression is a time dependent.

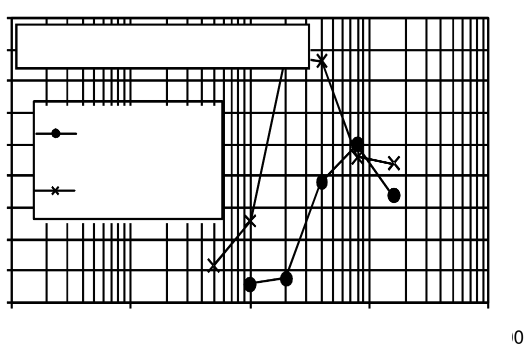


Figure 9: Stress Dependency of Secondary Compression Ratio (After Bo and Choa 2004)

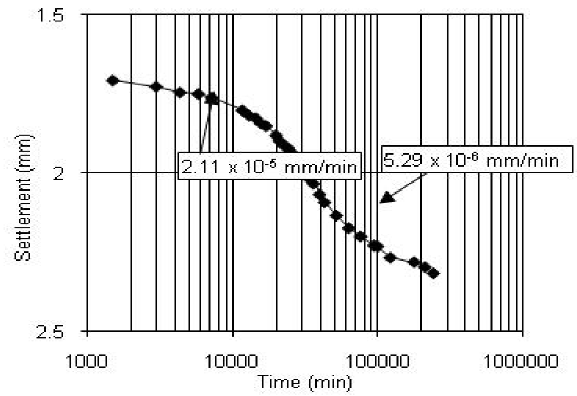


Figure 10: Tertiary Compression (After Bo and Choa, 2004)

7 COEFFICIENT OF CONSOLIDATION AND TIME RATE OF CONSOLIDATION

7.1 Non-linearity between effective stress gain and deformation

Conventional consolidation theory is based on the linear elastic model and vertical deformation increases linearly with effective stress. However in reality pore pressure dissipation in other words effective stress gain always lacks behind the deformation despite they merge together at the 100 % degree of consolidation (Fig. 11). The degree of such non-linearity is increasing with ratio of applied stress to in-situ initial stress (Mikasa, 1981 & 1995).

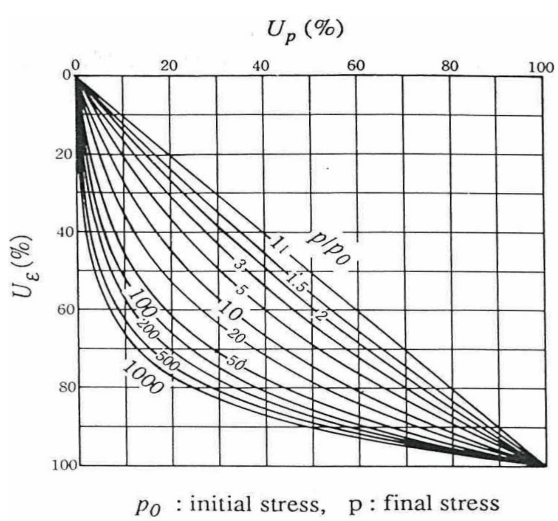


Figure 11: U_E – U_P relations for clays varying p/p_o (from Mikasa et al., 1981, 1995)

7.2 Non-uniform strain Profile

Time rate of consolidation is conventionally calculated using a time factor curve. However when time rate of consolidation are calculated for the same type of soil with varying thickness applying the curve available for uniform strain, lager settlements are predicted for thinner layer of soil in the earlier time step as shown in (Fig. 12a). This shows that conventional consolidation theory to predict settlement rates can lead to in correct results (Duncan 1994). When predictions are made applying non-linear stress-strain and non-uniform strain profile, more realistic settlement rates are resulted (Fig. 12b).

7.3 Taylor and Casagrande

While the Taylor method determines C_v values from a square root time method, the Casagrande method determines C_v value from an e log t curve. However neither of these two methods provide the same C_v for the same set of laboratory data on the same soil (Fig. 13).

7.4 Scale effect

The Taylor method determines the straight-line portion of an e square root t hyperbolic curve to determine the t_{90} value, which is affected by scale selected.

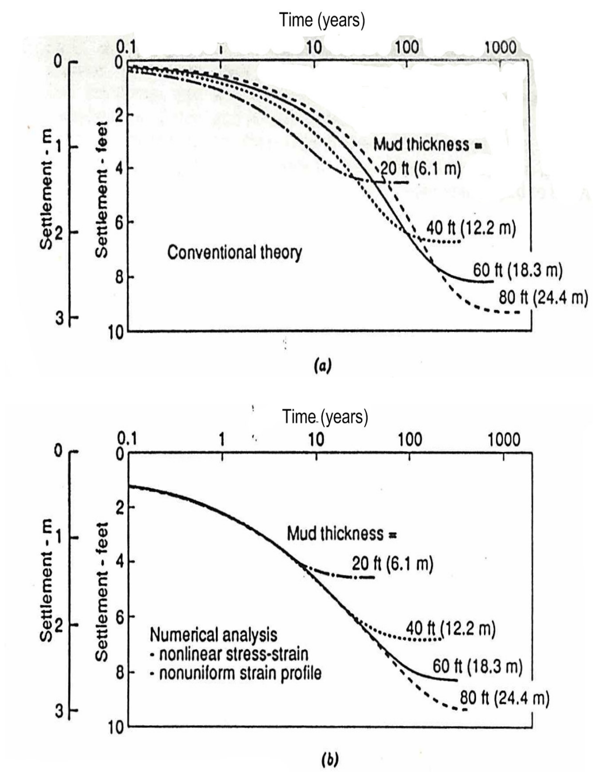


Figure 12: Calculated Variations of Settlement with Time for Mud Thicknesses of 6.1 m, 12.2 m, 18.3 m, and 24.4 m: (a) Calculated Using Conventional Consolidation Theory; and (b) Calculated Using Numerical Analysis (After Duncan 1993, Taylor 1948).

8.0 CONCLUSION

- The subject of soil mechanics started with consolidation theory.

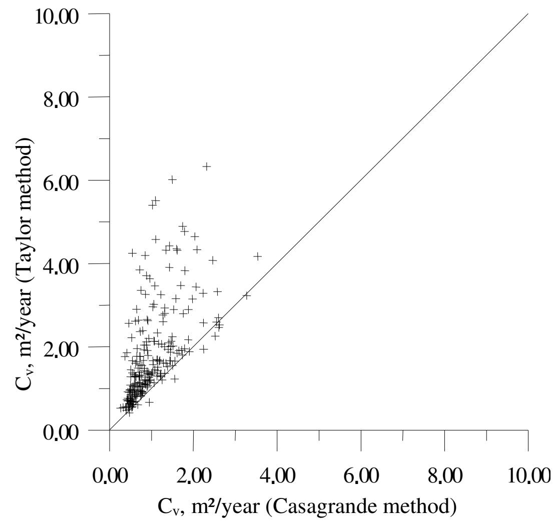


Figure 13: Comparison of Taylor's Versus Casagrande's method (After Bo and Choa 2004)

- Many assumptions and simplifications were made in the theory to simplify the complex behaviour of soil consolidation.
- Such a complex consolidation theory has led one of the potential academic and his wife to give up their life.
- Nevertheless, many practicing geotechnical engineers have benefited from consolidation theory and managed to eliminate the future settlement likely to be caused by consolidation process.
- However there are many limitations in the one dimensional consolidation theory.
- Consolidation behaviour is not unique and is stress strain dependent. They are also affected by temperature, environment.
- The deformation of soil is much more complex than theory can predict.
- Therefore engineers applying consolidation theory should be aware of limitations and treat the problems with cautious.

ACKNOWLEDGEMENT

The author would like to acknowledge Mr. Stephen Prime and Ms. Patricia Godin of DST Consulting Engineers Inc. for their assistance in the preparation of this paper.

REFERENCES

Barron, R.A. 1948. Consolidation of fine grained soils by drain wells, Trans. ASCE, 113: 718-734.

Been, K. and Sills, G. C. 1981. Self-weight Consolidation of Soft Soils, An Experimental and Theoretical Study, *Géotechnique* 31:(4), 519-535.

Biot, M. A. 1955. Theory of Elasticity and Consolidation for a Porous Anisotropic Solid, *Journal of Applied Physics,* 20:(2), 182-185.

Bo Myint Win, Arulrajah, A., Choa, V. and Na, Y. M. 1999. One-dimensional Compression of Slurry with Radial

Drainage, *Soils and Foundations*, Japanese Geotechnical Society, 39:, 9-17,.

Bo, M W 2002. Deformation of Ultra-Soft Soil" Ph.D Thesis, Nanyang Technological University, Singapore.

Bo, M.W., Choa, V & Hong, K.H. 2003 Material characterization of Singapore Marine Clay at Changi, *Quarterly Journal of Engineering Geology and Hydrologeology*, The Geological Society, London, 36☹4), 305-320.

Bo, M.W., Chu, J., Low, B.K., Choa, V. 2003. Soil Improvement, Prefabricated Vertical Drain Techniques, Thompson Learning, Singapore.

Bo, M. W & Choa. V 2004. RECLAMATION AND GROUND IMPROVEMENT, Thomson learning.

Bo, M. W**.,** Wong, K.S & Choa, V. 2005. Reclamation and Ground Improvement on Ultra-soft soils, *International Journal of Ground Improvement*. 9:(1), 23-31.

Bo, M. W 2008. COMPRESSIBILITY OF ULTRA-SOFT SOIL, World Scientific.

Butterfield, R. 1979. A natural compression law for soil (an advance on e-log I'), Geotechnique, 29:(4), 469-480.

Candler, C. J., and Chartres F.R.D. 1988. Settlement and analysis of three trial embankments on soft peaty ground. Pro. And Baltic Conf. On Soil Mech. And Foundation Engineering, International society of Soil Mechanics and Foundation Engineering, I, 268-272.

Carillo, N. 1942. Simple Two and Three Dimensional Cases in the Theory of Consolidation of Soils, *Journal of Mathematics and Physics*, 21:, 1-5.

Casagrande, A. 1936. The Determination of the Pre-Consolidation Load and Its Practical Significance, *Proceedings, First International Conference on Soil Mechanics, Cambridge, Mass, 3:, 60-64.*

Cryer, C. W. 1963. A Comparison of the Three-Dimensional Consolidation Theories of Biot and Terzaghi, *Quarterly Journal of Mechanics and Applied Mathematics*, 16:, 401-412.

Das, B M., 1983. Advanced Soil Mechanics. McGraw-Hill International Editions.

Dhowian A. W. and Edil T.B. 1980. Consolidation behaviour of peats, Geotechnical Testing J. 3:(3), 105-114.

Duncan, J. M. 1993. Limitation of Conventional Analysis of consolidation Settlement, Journal of Geotechnical Engineering, ASCE ,119:(9), 1333 – 1359.

Gibson, R. E., Schiffman, R. L. and Cargill, K. W. 1981. The Theory of One-dimensional Consolidation of Saturated Clays, II. Finite non-linear consolidation of thick homogeneous layers, *Canadian Geotechnical Journal*, 18: 280-293.

Gibson, R.E., Potter L. J., Savridon C. and Schiffman, R.L. 1995. Some aspects of one-dimensional consolidation and contaminant transport in wastes. Compression and Consolidation of Clayey Soils, Yoshikuni, H. and Kusakabe, O. (eds.), Balkema, Rotterdam, 815-832

Janbu, N., Tokheim, O., Senneset, K. 1981. Consolidation Tests with Continuous Loading, *Proc. 10th ICSMFE.*, Stockholm, 1:, 645-654.

Lee, K. and Sills, G. C. 1981. The Consolidation of a Soil Stratum, Including Self-weight Effects and Large Strains, *International Journal for Numerical and Analytical Methods in Geomechanics*, 5:, 405-428.

Leroueil, S., M. Kabbaj, F. Tavenas, and R. Bouchard. 1985. Stress-strain-Strain Rate Relation for the Compressibility of Sensitive Natural Clay, *Géotechnique*, 35:(2), 159-180.

Leroueil, S., Kabbaj, M., Tavenas, F. and Bouchard. 1986. Stress-strain-strain Rate Relation for the Compressibility of Sensitive Natural Clays Reply, *Géotechnique* 36(2), 288-290.

Leroueil, S. 1999. Geotechnical characteristics of Eastern Canada clays, in *Characterization of Soft Marine Clays*, eds. Tsuchida and Nakase, Balkema, Rotterdam, 3-32.

Marques, M.E.S. 1996. Influencia da velocidade de deformacao e da temperature no adensamento de argilas naturais. M.Sc. Thesis. Research performed at Universite Laval, Quebec, in cooperation with COPPE, Federal University of Rio de Janeiro, Brazil.

Mikasa, M. 1961. Consolidation Analysis of Soft Clay Considering Self-weight of the Clay and its Application to the Reclamation Work*, Proceedings of Annual convention of JSCE*, 7-9 (in Japanese).

Mikasa, M. & Ohnishi, H. 1981. *Soil improvement by dewatering in Osaka South Port, Appendix-5. Determination of soil parameters from consolidation tests. Geotechnical aspects of coastal reclamation project in Japan*. Proc. of 9th ISSMFE. Case history volume. 661 - 664.

Mikasa, M. 1995. Two Basic Questions on Consolidation, *Compression and Consolidation of Clayey Soil, Yoshikuni and Kusakabe (eds),* Balkema, 1097-1098.

Nash, D.F.T., Sills, G.C & Davison, L.R. 1992. One-dimensional consolidation testing of soft clay from Bothkennar. *Geotechnique*. Vo 42: (2), 241-256

Sills, G. C. 1995. Time Dependent Processes in Soil Consolidation, *Compression and Consolidation of clayey soils*, Yoshikuni & Kusakabe (eds) Balkema, Rotterdam, 875-890.

Sridharan, A. and Sreepada Rao, A. 1981. Rectangular Hyperbolar Fitting Method for One-dimensional Consolidation, *Geotech Testing J. C4:;(4),* 161-168.

Taylor , D.W. 1948. *Fundamentals of Soil Mechanics*, John Wiley & Sons, Inc., New York, 700 pp.

Taylor, D.W. and Merchant, W. 1940. A theory of clay consolidation accounting for secondary compressions, J. Math. Phys., 19:, 167-185.

Terzaghi, K. 1925. Erdbaumechanich, Vienna: F. Deuticke.

Terzaghi, K.v. and O.K. Frohlich. 1936. Theorie der Setzung von Tonshichten: eine einfuhrung in die analytische Tonmechanik. Leipzig: F. Deuticke.

Wong, K.S. and Choa, V. 1987. Settlement analysis of soft clay by finite difference method," 5th International Geotechnical Seminar, Case Histories in Soft Clay, Nanyang Technological Institute, Singapore.