Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)
Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5
https://doi.org/10.53243/ICSMGE2026-426

A Double Compressibility Model for Gassy Soil

Stephen D. Thomas
OGI Groundwater Specialists Ltd, City of Durham, United Kingdom. stephen.thomas@ogi.co.uk

ABSTRACT: Gassy soil is the term that describes a soft marine soil containing methane gas bubbles in an otherwise saturated soil matrix. Multi-stage oedometer tests demonstrate two distinct stages of vertical deformation. The initial, undrained stage, takes place as a consequence of the transfer of total stress loading to the gas bubble pressure, causing gas compressibility and subsequent undrained volume compressibility. However, during this undrained stage, the pore water pressure increases by approximately the same increment as the total stress increase, with no volume change of the saturated soil matrix. After the initial undrained compression deformation, the drained stage begins where the increased pore water pressure dissipates to the drained atmospheric boundary, with the effective stress within the saturated soil matrix ultimately increasing to close to the applied total stress. Laboratory tests demonstrate that the saturated soil matrix behaves similarly to a fully saturated soil, insomuch that the void ratio of the saturated component of the soil is dependent on the difference between the average vertical total stress over the sample and the pore water pressure, coined “operative stress.” However, the volume change of the gas bubbles from compression and dissolution, is governed by the increase in the gas pressure, which is related to the mean total stress. Based on these observations, the volume change of a gassy soil is then governed by two independent stress changes: (i) the change in operative stress (total stress – pore water pressure), and (ii) the change in total stress. In effect, this results in a gassy soil having two independent modes of compressibility, hence the term Double Compressibility Model. Building on the equations of Biot (1941), based on both equilibrium and continuity conditions, transient consolidation equations are developed that govern the initial undrained compression, and the subsequent time-dependent consolidation behaviour of gassy soil.

KEYWORDS: Double compressibility, gassy soil, consolidation.

1 INTRODUCTION

Led by Professor Gilliane Sills of the University of Oxford Soil Mechanics Department, an extensive programme of research was conducted into the influence of undissolved gas bubbles on the behaviour of marine sediments, known as Gassy Soil.

Gassy soil is the term for a soft marine soil containing gas bubbles within an otherwise water-saturated soil matrix, with the gas bubbles being considerably larger than the soil grains.

Figure 1 illustrates a sample of gassy soil with around 20% overall gas saturation (80% water saturation). As a consequence of the surface tension between the gas (normally methane) and the water-filled voids of the saturated soil matrix, the gas pressure, can be different from the water pressure.

Knowledge of the behaviour of gassy soil is important for marine geotechnics, as gassy soil is regularly encountered resulting from biogenic activity, and results in a marine soil that is compressible in undrained conditions. Unlike saturated soil, the seabed level of a gassy soil sediment can rise and fall under tidal conditions. This Oxford research program comprised both laboratory testing and theoretical analysis of the consolidation behaviour of gassy soils. This paper presents the results of the laboratory testing, and describes the theory developed to simulate the undrained and time-dependent drained deformation that occurs as a result of applying stress loading on a gassy soil.

2 TESTING METHODOLOGY

2.1 *Sample preparation*

A specially designed oedometer cell to consolidate gassy soil was manufactured at the Oxford Soil Mechanics Laboratory. This enabled the generation of a gassy soil sample within the cell (Figure 1). The procedure required mixing a saturated marine sediment slurry with a methane charge zeolite powder. Over 24-hours, the zeolite releases small amounts of methane into the sediment. This results in the production of a soft marine silty soil, containing numerous discrete gas bubbles, with the bubbles being significantly larger than the soil particles. A sketch of a typical arrangement of solid particles is depicted in Figure 2a, with an idealised model of spherical occluded gas bubbles, surrounded by an otherwise saturated soil matrix idealised in Figure 2b (Wheeler, 1986).

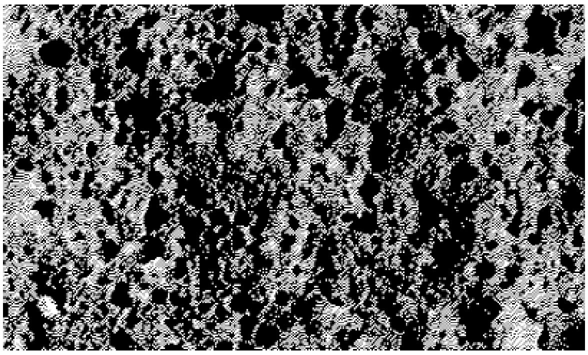

Figure 1. Appearance of Gassy Soil. Magnified x 6 Degree of Saturation=0.8, 1-D Consolidation to 35kPa (Nageswaran, 1938).

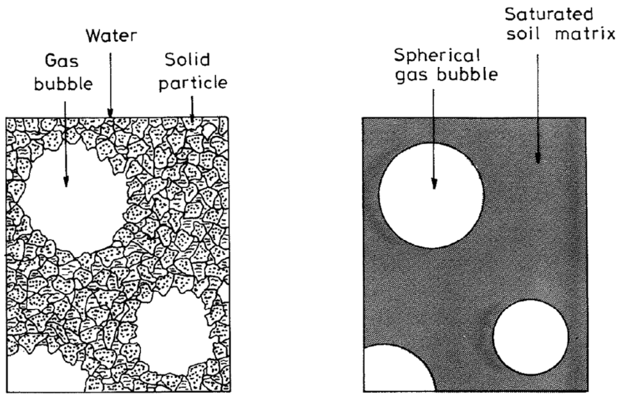

Figure 2a) large gas bubbles, 2b) the idealized model (Wheeler, 1986).

2.2 *Experimental apparatus*

The gassy soil consolidation cell was first developed by Nageswaran (1983), then modified by Thomas (1987). The cell enables application of a total vertical stress from beneath via a hydraulic piston, on a gassy soil sample (Figure 3). Drainage of the pore water and both free and dissolved gas is to the surface porous drainage boundary. Also enabled are accurate measurements of (i) the gassy soil sample volume, (ii) pore water pressure at the base of the soil sample, (iii) vertical stress acting directly on the soil sample, and (iv) horizontal total stress at the side of the soil sample.

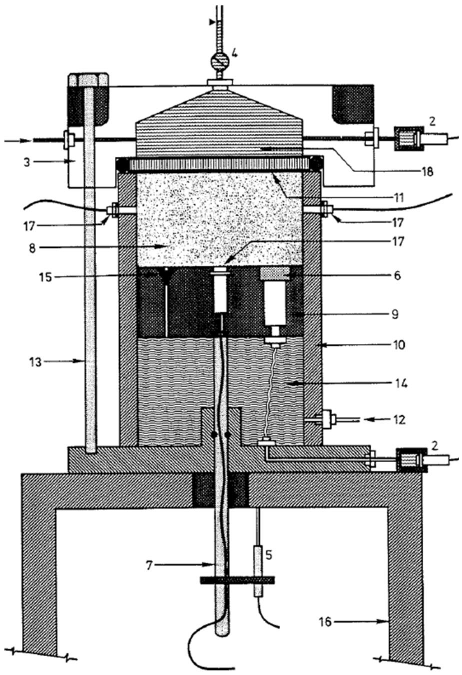

Figure 3. Schematic description of the oedometer cell. 1. Drainage port, 2. Pressure transducer, 3. Top cap, 4. Tap valve, 5. Displacement transducer, 6. Porous stone, 7. Guide rod, 8. Soil sample, 9. Piston, 10. Cylinder wall, 11. Filtering system, 12. Water reservoir inlet, 13. Holding down bolt, 14. Water reservoir, 15. Bleed screw, 16. Support frame, 17. Total stress transducer, 18. Drainage water (Thomas, 1987).

2.3 *Experimental testing*

Two main series of tests were conducted, Series A and Series B. The first series (A) was focused on measuring the accurate value of void ratio, both water and gas, over an increasing total stress regime, with the pore water pressure constantly draining. Series B comprised three stages of stress increase, with measurements of changes in displacement and pore pressure made over the duration of the undrained and drained stages.

2.3.1 *Test Series A*

A series of oedometer tests was conducted in which the vertical stress, σ_v, increased linearly from 25 kPa to a little over 435 kPa over a period of four hours. Drainage was allowed from the top surface of the soil sample, with pore water pressure measured on the bottom, undrained face. Seven tests were carried out, with initial degrees of saturation ranging from 0.87 to 1.0. A typical response of the individual components, either measured or calculated, are presented in Figure 4.

The gas pressure shown in Figure 4 is back-calculated from Boyle's Law and Henry's Law applied to the volume change of gas measured within the cell. This demonstrates that the free gas pressure continues to rise with total stress and does not appear to have a direct relationship with the pore water pressure. Counterintuitively, it suggests that the gas pressure has a direct relationship with the vertical total stress.

The traditional way of plotting volume change from a test like this on a saturated soil would be in terms of void ratio against effective stress, defined by Terzaghi (1936) as the difference between the total stress and pore water pressure, $\sigma_v - u_w$. For a gassy soil, however, the term effective stress is ambiguous, since it cannot satisfy all components of the Terzaghi definition.

"All the measurable effects of a change of the stress, such as compression, distortion and a change in shearing resistance, are exclusively due to changes in the effective stresses…".

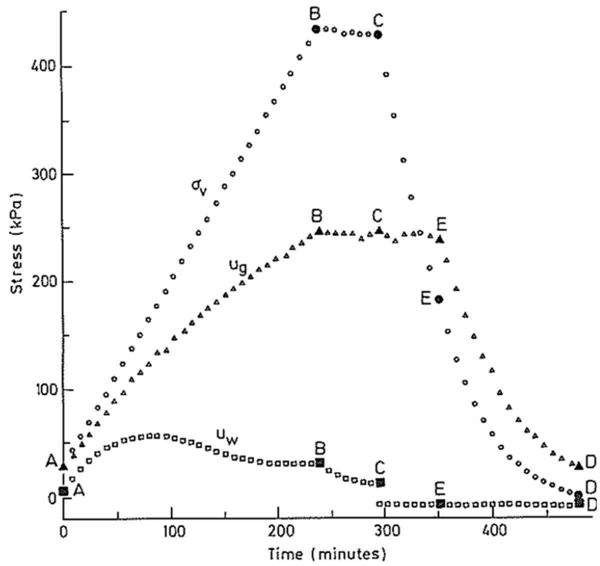

Figure. 4. Results from consolidation test with load increasing linearly with time, showing vertical stress, gas pressure and pore water pressure.

However, it is seen from Figure 1 that gassy soil appears to be a saturated soil matrix containing occluded gas voids, and it is seen that the stress difference $\sigma_v - u_w$ is still important in determining the behaviour of this saturated soil matrix, but does not govern the total compression of the sample. Therefore, the term operative stress and the symbol σ'' have been coined (Sills et al. 1991) for the difference between the total stress and the pore water pressure. For this series of tests (Test Series A), the total void ratio e was plotted against the vertical operative stress $\sigma_v'' = \sigma_v - u_w$, where u_w is an average value of pore water pressure calculated from the measured value of pore water pressure on the undrained face, and the condition of a parabolic distribution through the sample.

So, in addition to the usual measurement of the volume of water draining during consolidation, the free gas volume in the sample was also known at any time, by subtraction of the solids volume and water volume from the overall sample volume. Figure 5 depicts the variation of the total void ratio e, versus vertical operative stress, with the water matrix void ratio e_w, versus vertical operative stress depicted in Figure 6.

This demonstrates that when the water void ratio e_w was plotted against the vertical operative stress (Figure 6) the experimental results of water void ratio versus operative stress, all reduced to a unique line. Referring back to Figure 2b, this suggested that for any initial gas content, the water void ratio, or, the void ratio of the saturated soil surrounding the gas bubbles, is solely a function of the operative stress.

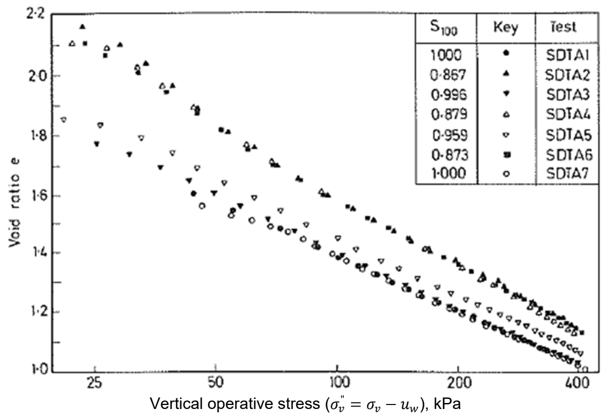

Figure 5. Total void ratio plotted against vertical operative stress for consolidation tests with load increasing linearly over time.

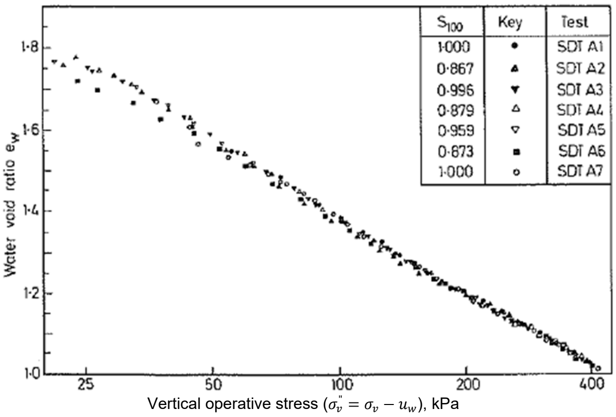

Figure 6. Water void ratio plotted against vertical operative stress for consolidation tests with load increasing linearly over time.

2.3.2 *Test Series B*

The second series of tests (Test Series B) was conducted using a more traditional testing approach. This comprised an increment of load applied instantaneously, with the drainage line initially held closed for a short period to measure the undrained increase in pore water pressure and undrained deformation.

The drainage line is later opened to enable the drainage of pore water, so enabling the lowering of pore water pressure, and then a subsequent drained consolidation deformation.

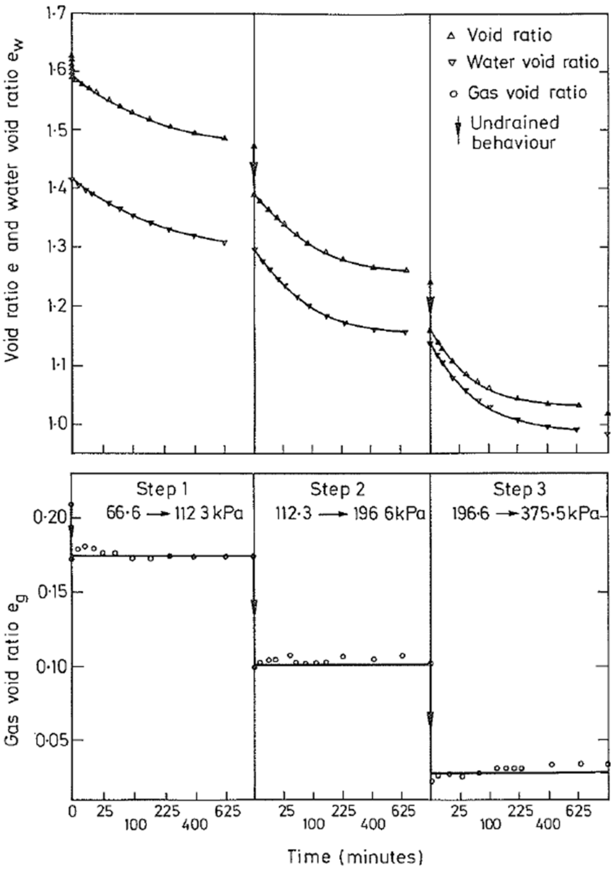

Figure 7. Components of void ratio versus time for test SDTB2.

Each incremental loading test comprised three individual increments (Thomas, 1987). Initially, at zero time, no drainage was allowed, no change occurred in the water void ratio e_w, and the changes in the total void ratio e and gas void ratio e_g were due solely to gas compression or dissolution of the gas into the pore water. During subsequent drainage in the consolidation stage, as depicted in Figure 7, it was observed that the changes in e and e_w were almost equivalent, with very little change in e_g. A small amount of free gas was collected from the drainage face of the sample, some of which would have left the sample in solution in the pore water. This suggested that the gas volume changed only when the total stress changed, and was not altered by the dissipation of pore water pressure during consolidation. Therefore, there was very little flow of free gas during the drained stage of the test, given the small volume that was collected and the constant volume that remained in the sample.

During the undrained stage after the application of the initial load with the drainage line closed, it was feasible to more accurately measure the increase in pore water pressure. The results of the pore water pressure measurements were most unexpected. When a typical unsaturated soil with air/water filled voids is compressed under undrained conditions during an oedometer test, the pore water pressure usually does not increase beyond the total stress increment. This was not observed during the undrained tests on gassy soil. Instead, there was an increase in pore water pressure increment that was at most times greater than the total stress increment. Figure 8 presents the findings of normalised pore water pressure increase $(\Delta u_w/\Delta\sigma_m)$ versus the free gas porosity (n_g). Whilst it can be seen that there is indeed a scatter of results, it is clear that there is a trend of increasing normalised pore pressure change vs with greater occluded free gas content.

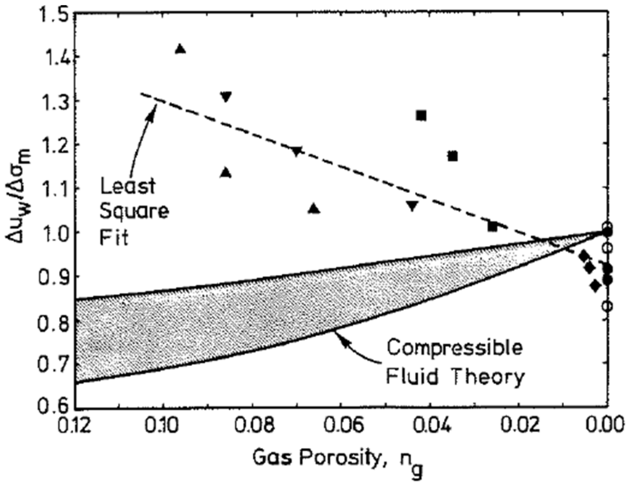

Figure 8. Undrained pore water pressure change vs gas porosity.

Test Series B also enabled the identification of total void ratio at several stages of vertical operative stress. The results are presented in Figure 9, demonstrating a similar response to Test Series A, with there being no unique void ratio line.

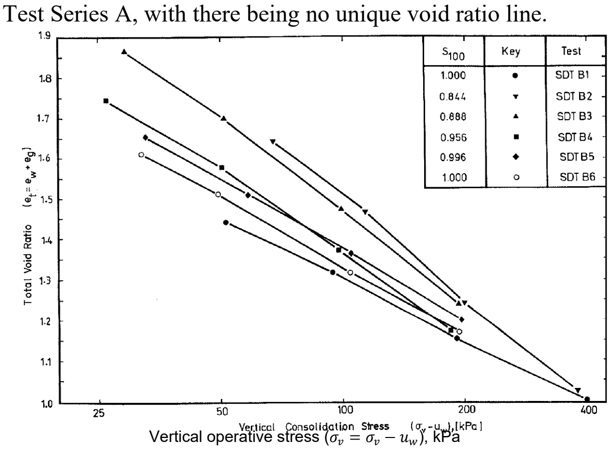

Figure 9. Total void ratio versus vertical operative stress (Test Series B)

Figure 10 presents a plot of the water-saturated matrix void ratio plotted against operative stress. Similar to Test Series A, this produces a more unique line of void ratio vs operative stress.

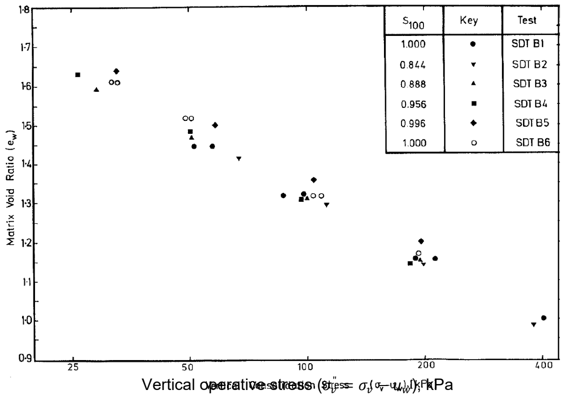

Figure 10. Matrix void ratio vs vertical operative stress - test series B.

Modification of the oedometer cell was made for Test Series B, to enable the measurements of total horizontal stress. This enabled the calculation of mean operative stress. In turn, this enabled the matrix void ratio to be plotted against mean operative stress is depicted in Figure 11.

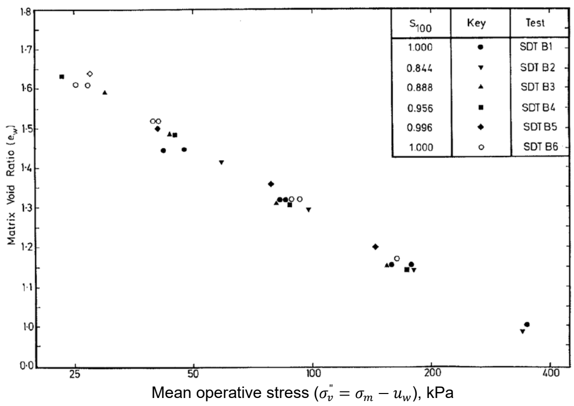

Figure 11. Matrix void ratio vs mean operative stress - test series B.

3 A DOUBLE COMPRESSIBILITY MODEL

3.1 *The double compressibility hypothesis*

These observations led to the hypothesis that the volume compressibility of the saturated soil matrix, surrounding the gas bubbles, is a function of the change in operative stress.

$$\epsilon'' = \Delta\sigma''/D'' = (\Delta\sigma^o - \Delta u_w)/D'' \quad (1)$$

Where ϵ'' is the vertical strain of the saturated soil matrix, D'' is the vertical drained constrained modulus, $\Delta\sigma''$ is the change in vertical operative stress, $\Delta\sigma^o$ is the change in total vertical stress, and Δu_w is the change in pore water pressure. The results also led to the hypothesis that the gas pressure within the gas bubbles, was dependent on, and possibly proportional to, the total stress applied to the soil sample, and not related to the pore water.

This hypothesis means that the strain resulting from the compression of the gas bubbles, ϵ^o, as produced by an increase in total stress, $\Delta\sigma^o$, is related to the gassy component of the constrained modulus D^o i.e.

$$\epsilon^o = \Delta\sigma^o/D^o \quad (2)$$

This hypothesis was also extended to the gas bubbles behaving akin to a compressible solid, rather than a compressible pore fluid. At this point in the development of the governing equations, it was considered that during the initial undrained phase, whilst a gas bubble reduces in volume due to an increase in gas pressure caused by the increase in total stress, the surrounding saturated soil matrix maintains volume incompressibility, but simply changes shape to accommodate the changing volume of the gas bubble.

3.2 *A Double Compressibility Model for Gassy Soil*

The laboratory tests A and B on gassy soils of variable gas content, demonstrated that it was likely that the deformation of the soil takes place as a result of two independent processes. The first undrained stage is due to the gas bubbles reducing in volume from a combination of compression caused by an increase in gas pressure, together with a loss in volume caused by dissolution of the gas into the surrounding saturated soil matrix, again caused by an increase in gas pressure. Note that whilst this is an undrained process, it is not necessarily instantaneous because of the time dependent process of the gas dissolution into the surrounding pore water pressure.

During the undrained period, the water-saturated soil matrix surrounding the gas bubbles does not change in volume, but it is able to deform by shear and changes shape to accommodate the change in shape and size of the gas bubbles.

The second mode of deformation takes place as a result of the draining of the saturated soil matrix through the upper drainage boundary. With the total stress remaining constant, and the pore water pressure reducing, this creates an overall increase in operative stress over the gassy soil sample, together with an associated increase in effective stress within the saturated soil matrix surrounding the gas bubbles. This results in two independent strain components caused by:

(i) an increase in the total stress $\Delta\sigma^o$, and

$$\epsilon^o = \frac{1}{D^o}\Delta\sigma^o \quad (3)$$

(ii) an increase in the operative stress ($\Delta\sigma'' = \Delta\sigma^o$ - Δu_w),

$$\epsilon'' = \frac{1}{D''}\Delta\sigma'' = \frac{1}{D''}(\Delta\sigma^o - \Delta u_w) \quad (4)$$

Where ϵ'' is the vertical strain of the saturated soil matrix, D'' is the vertical constrained drained modulus, $\Delta\sigma''$ is the change in vertical operative stress, $\Delta\sigma^o$ is the change in total vertical stress, and Δu_w is the change in pore water pressure.

As it is not feasible to separate the individual strain components ϵ^o and ϵ'' these must be added together to give the overall strain ϵ , giving:

$$\epsilon = \epsilon^o + \epsilon'' = \frac{1}{D^o}\Delta\sigma^o + \frac{1}{D''}\Delta\sigma'' \quad (5)$$

$$= \frac{1}{D^o}\Delta\sigma^o + \frac{1}{D''}(\Delta\sigma^o - \Delta u_w) \quad (6)$$

$$= \left(\frac{1}{D^o} + \frac{1}{D''}\right)\Delta\sigma^o - \left(\frac{1}{D''}\right)\Delta u_w \quad (7)$$

from which

$$\Delta\sigma^o = \left(\frac{1}{D^o} + \frac{1}{D''}\right)^{-1}\left(\epsilon + \frac{1}{D''}\Delta u_w\right) \quad (8)$$

or

$$\Delta\sigma^o = \overline{D}\epsilon + \overline{B}\Delta u_w \quad (9)$$

where

$$\overline{D} = \left(\frac{1}{D^o} + \frac{1}{D''}\right)^{-1} = \left(\frac{D^o\,D''}{D'' + D^o}\right) \quad (10)$$

and

$$\overline{B} = \left(1 + \frac{D''}{D^o}\right)^{-1} = \left(\frac{D^o}{D'' + D^o}\right) \quad (11)$$

The distribution of total vertical stress in a homogeneous soil can be written:

$$\frac{\partial\sigma_z^o}{\partial z} = F_z \quad (12)$$

From which substituting the double compressibility stress-strain relationship for gassy soil gives:

$$\frac{\partial}{\partial z}\left(\overline{D}\epsilon + \overline{B}\Delta u_w\right) = F_z \quad (13)$$

Further substituting:

$$\epsilon = -\frac{\partial w_z}{\partial z} \quad (14)$$

gives the governing displacement equation:

$$\overline{D}\frac{\partial}{\partial z}\left[\frac{\partial w_z}{\partial z}\right] - \overline{B}\frac{\partial u_w}{\partial z} = -F_z \quad (15)$$

with the governing continuity equation as:

$$\frac{\partial}{\partial z}\left[\frac{k_z}{\gamma_w}\frac{\partial u_w}{\partial z}\right] = \overline{C}\frac{\partial u_w}{\partial t} + \overline{B}\frac{\partial}{\partial t}\left[\frac{\partial w_z}{\partial z}\right] \quad (16)$$

where

$$\overline{C} = (D^o + D'')^{-1} + (1 - H)c_w n_w \quad (17)$$

For the condition that there is no free gas content, then the value of $\overline{B}$, is 1.0, and the value of $\overline{C}$ reduces to the value of the compressibility of the pore fluid, the above equations (15 & 16) reduce to the Biot (1941) poro-elastic consolidation equations.

3.3 *Verification of theory with laboratory results*

Finite element model DCFEM2 (Thomas, 1987) was developed to solve the above differential equations. This model could then simulate the displacement and pore-water pressures observed in Test Series A and B. Before the double compressibility model was developed, an attempt was made to model the observed pore water pressure using a compressible fluid theory; that is, when the pore fluid contains many small compressible gas bubbles, and the soil deforms due to an increase in effective stress, so following the Bishop model (1959).

Figure 12 presents the best attempt to simulate pore water pressure response of Test A6 using a compressible fluid theory. However, this is a poor match of the pore water pressure measured at the undrained face of the consolidation cell.

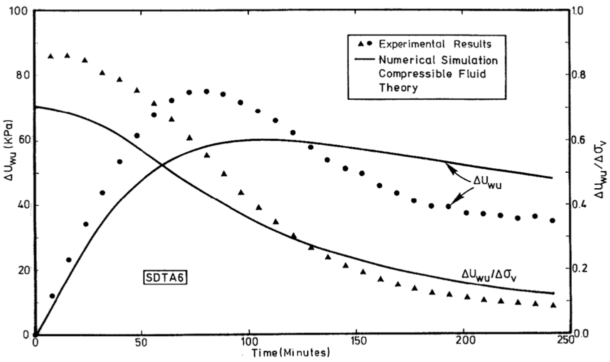

Figure 12. Numerical simulation of undrained face pore water pressure using compressible fluid theory for test SDTA6.

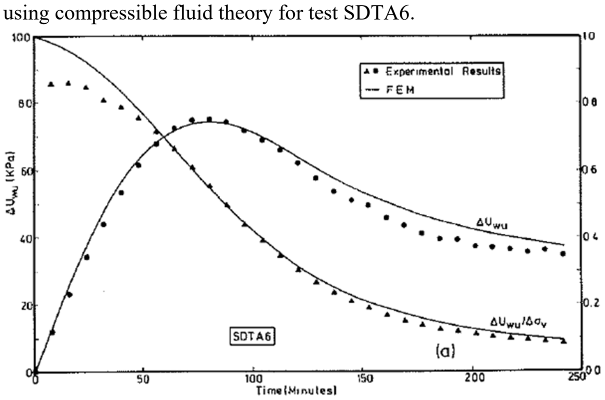

Figure 13. Numerical simulation of undrained-face pore water pressure using compressible gas void theory.

After the double compressibility model was developed, the finite element model was then used to simulate the same test SDTA6. Figure 13 presents a much better simulation of the normalized increase in pore water pressure and normalised pore water pressure on the undrained face, with Figure 14 delivering a good match of the decrease in total, water and gas void ratio.

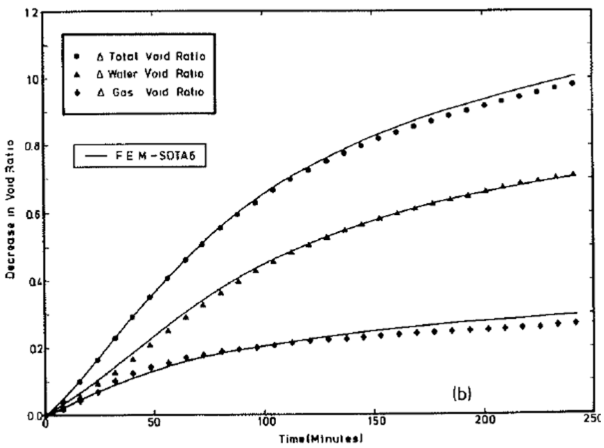

Figure 14. Matched decrease in total, water and gas void ratio over time

3.4 *Verification of theory with a composite test*

To verify the hypothesis that a gassy soil appears to behave as a saturated soil containing discrete compressible inclusions remaining in the same location with respect to the soil particles, a laboratory test was conducted in which the gas voids were replaced by compressible solid inclusions. In order to maintain the same soil matrix behaviour as that of a gassy soil, 15% by volume of water-saturated zeolite was included. The compressible solid inclusions used for this test were polystyrene balls with diameters ranging between 1 and 3 mm.

Figure 15 depicts the principle of mixing the saturated soil with the compressible polystyrene balls. This was placed in the same oedometer as for Test Series A and B. This Test C contains no gas content, but it did have a compressible solid content.

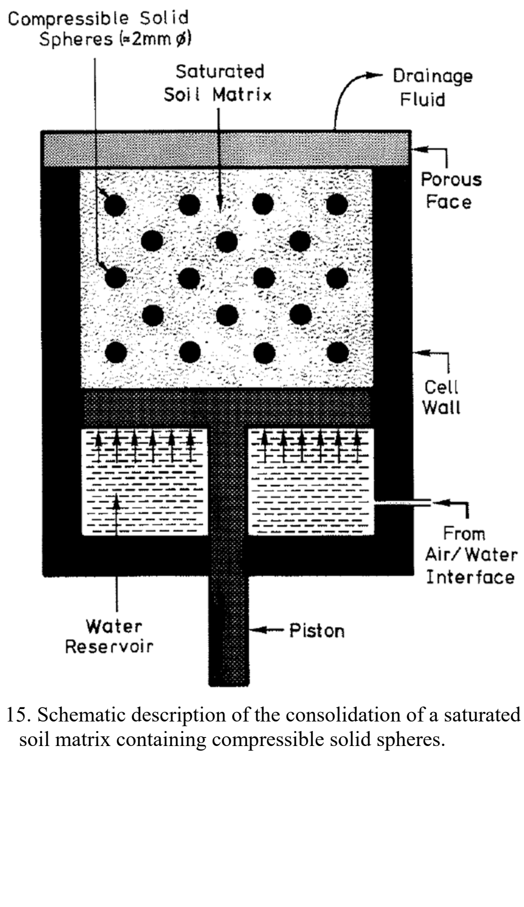

Figure 15. Schematic description of the consolidation of a saturated soil matrix containing compressible solid spheres.

The test results of the change in total, water and polystyrene void ratio presented in Figure 16 demonstrates almost identical behaviour to the results of Test series B.

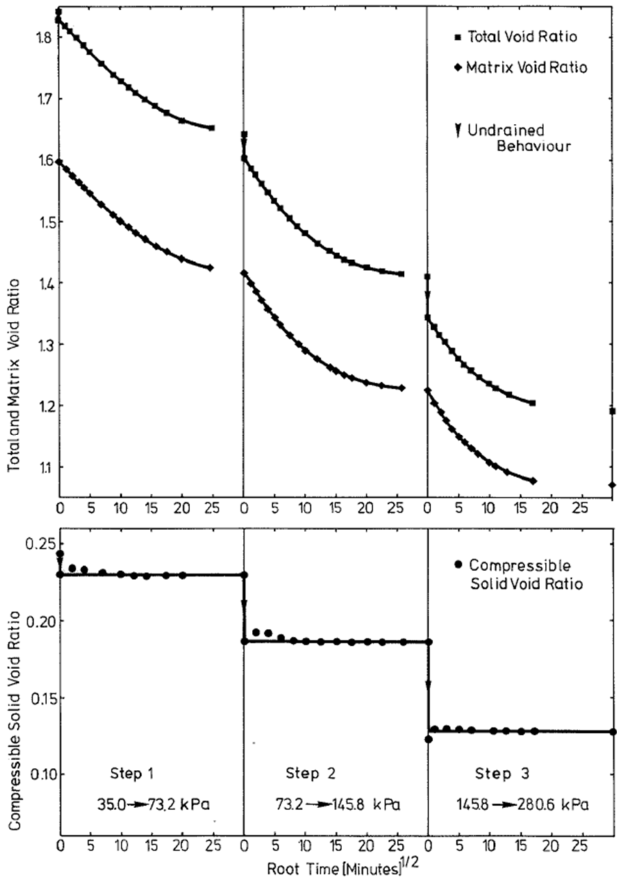

Figure 16. Illustration of the phase void ratio behaviour for a saturated soil containing compressible solid spheres (Test SDTC1).

To further illustrate the similarity of the behaviour between the compression behaviour of a gassy soil with a composite soil containing compressible solid polystyrene balls, Figure 17 presents the saturated matrix void ratio versus the vertical operative stress for both cases. This comparison illustrates that the mechanical behaviour of the gas bubbles can be treated as if they are compressible solid inclusions.

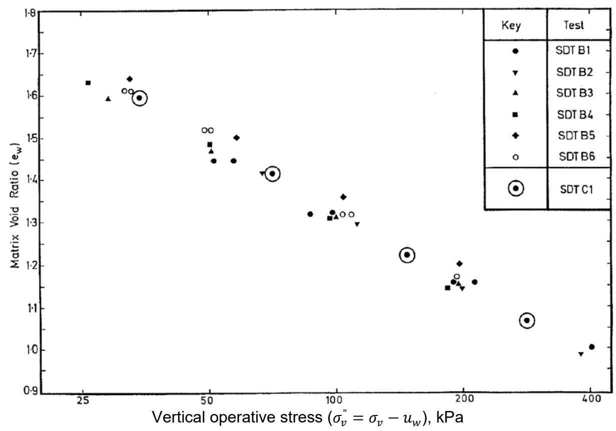

Figure 17. Matrix void ratio versus vertical consolidation stress for test SDRC1 and test series B.

4 CONCLUSIONS

The research programme at the University of Oxford comprised laboratory testing, equation development and finite element modelling on the consolidation behaviour of gassy soil. This has resulted in the following conclusions.

(a) Gassy soil comprises a marine sediment containing discrete, initially spherical gas bubbles, in an otherwise saturated soil matrix.

(b) The change in total void ratio is not governed by a change in "operative stress", i.e. $\sigma_v^{"} = \sigma_v - u_w$.

(c) The change in the void ratio of the saturated soil matrix is observed to be governed by the "operative stress."

(d) The change in gas void ratio is strongly influenced by the change in total stress on the sample, with the gas pressure being dependent on the total stress on the gassy soil.

(e) During undrained compression tests on a gassy soil, there is immediate deformation due to the gas compression.

(f) During undrained compression tests on a gassy soil, the pore water pressure increases in most cases to a value that is greater than the increase in total mean stress.

(g) A "Double Compressible Model" was developed to simulate the deformation of a gassy soil, where the total strain is the summation of strain caused by compression of the gas, and the strain caused by the volume change of the saturated soil matrix.

(h) An experiment was conducted with gas bubbles replaced by small compressible polystyrene balls. The observed behaviour was effectively identical to that of a gassy soil, reinforcing the hypothesis that gassy soil can be viewed as soil containing discrete compressible solid inclusions, surrounded by an otherwise saturated soil matrix.

5 ACKNOWLEDGEMENTS

This research was only possible due to the dedication of my supervisor, mentor and colleague, Professor Gilliane Sills. Over two decades at the end of the last millennium, she led a highly dynamic research team in the basements of the soft-soil research laboratories at the Department of Engineering Science. I express my profound gratitude for her continued guidance and encouragement.

Finally, deep gratitude must go to OGI's Açelya Balli, who provides wonderful positive support in preparing the scripts.

6 REFERENCES

Biot, M.A. 1941. General Theory of Three-Dimensional Consolidation. J. Appl. Phys., Vol. 12, pp. 155-164.

Bishop, A.W. 1959. The Principle of Effective Stress. Tek. Ukeblad, Vol. 39, pp. 859-863.

Nageswaran, S. 1983. Effect of gas bubbles on the sea bed behaviour [PhD thesis]. University of Oxford.

Sills, G.C., Wheeler, S.J., Thomas S.D., & Gardner, T.N. 1991. Behaviour of Offshore Soils Containing Gas Bubbles. *Geotechnique* 41, No.2, 227-241.

Terzaghi, K., 1936. The shearing resistance of saturated soils, and the angle between the planes of shear, in Proceedings of the [1st] International Conference on Soil Mechanics and Foundation Engineering, held June 22–26, 1936, at *Harvard University*, Cambridge, Mass., vol. 1, paper D-7, pp. 54–56.

Thomas, S. D. 1987. The consolidation behaviour of gassy soil [PhD thesis]. *University of Oxford.*

Wheeler, S. J. 1986. The stress-strain behaviour of soils containing gas bubbles [PhD thesis]. *University of Oxford.*