INTERNAL COMPRESSION OF FILL MATERIAL ORIGINATING FROM BRINGELLY SHALE

Theva Muttuvel[1], Richard Kelly[2], David Malorey[3], Esteban Litvin[4] and Jubert Pineda[5]

[1]*Principal Geotechnical Engineer, SMEC Australia Email: theva.muttuvel@smec.com Phone: +61 2 9900 7149*

[2]*Chief Technical Principal and General Manager Technical Excellence, SMEC Australia Email: Richard.Kelly@smec.com Phone: +61 7 3029 6625*

[3]*Technical Principal, SMEC Australia Email: David.Malorey@smec.com Phone: +61 2 9925 5558*

[4]*Engineering Services Design Manager, Ferrovial Agroman, Email: elitvin@ferrovial.com Phone: +61 2 8736 9600*

[5]*Senior Lecturer, University of Newcastle Email: jubert.pineda@newcastle.edu.au Phone: + 61 2 4921 7034*

ABSTRACT

A rule of thumb handed down from senior to junior geotechnical engineers in Australia is that internal compression of embankment fill is 0.1% of embankment height and it has been widely used in performance specifications for major infrastructure projects around Australia. It is not clear where this rule of thumb originated from or on what basis it was developed. In Western Sydney this rule of thumb has been adequate for many years because the scale of earthworks in terms of fill thickness has been relatively minor. Large scale earthworks have started to occur over the past decade or so as more significant road and rail development has occurred. Recent experience with higher fills constructed in Western Sydney shows that internal compression strain rate can be greater than 0.1% per log cycle of time. Embankments to about 10 m height on a rail infrastructure projects on Western Sydney were constructed from Bringelly Shale. The fill materials and compaction were compliant with the relevant engineering standards. Comparison of topographic survey about 5 years after construction with design profiles indicated that they had settled between 0.05 and 0.25 m, subject to construction tolerances, and the internal compression strain rate varied between 0.5% and 6.3% per log cycle of time, adopting 1 year as the starting time of post construction settlement. Anecdotal evidence from road embankments of up to 14 m have identified similar magnitudes of settlement response from fill formed of the same Bringelly Shale materials. These values are much higher than the rule of thumb of 0.1% per log cycle of time. Though Bringelly Shale-based fill material has shown such a significant settlement issue, to the authors' knowledge, there are no references found in the technical literature that provides some guidance on assessing the internal compression of Bringelly Shale based fill material. Therefore, a series of laboratory tests have been conducted to understand the settlement behaviour of compacted Bringelly Shale. Compaction tests, particle size distribution, Atterberg limits, and small and large-scale compression tests of compacted fill material have been conducted. This paper summarises some of the findings of the laboratory tests and authors' view on the performance of Bringelly Shale fill material and future studies.

1 INTRODUCTION

Ongoing post construction settlement of compacted fill material originated from Bringelly Shale has been observed in major infrastructure projects in Western Sydney region, causing significant maintenance cost. Embankments to about 10 m height on a rail infrastructure project were constructed from Bringelly Shale. The fill materials and compaction were compliant with the relevant engineering standards. The measured settlement after five years indicates that the internal compression strain rate varied between 0.5% and 6.3% per log cycle of time as shown in Figure 1, assuming that the start time for settlement was 1 year. These values are much higher than a typical internal compression strain rate of 0.1% per log cycle of time for engineered fill. In addition, rates of settlement recorded for settlement markers installed after completion of construction were approximately linear with the logarithm of time. The strain at the location of the settlement markers was up to 1.2% of embankment height over the period of about four years since these markers had been installed after construction. The settlement and rate of settlement appear to reduce as the embankment height increases. This data contradicts the published literature that settlement is proportional to embankment height or applied stress (Hopkins and Beckham, 1998; Waddell and Wong 2005), but is most likely a result of higher fills existing adjacent to structures, hence are more heavily compacted than away from structures and utilise select fill rather than general fill.

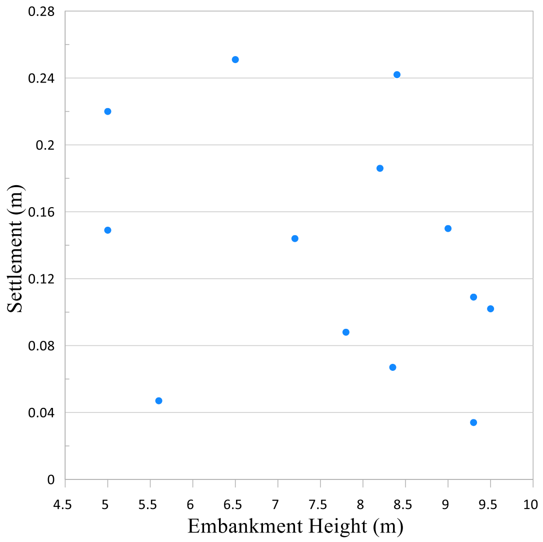


Figure 1(a): Change in embankment level over 5 years

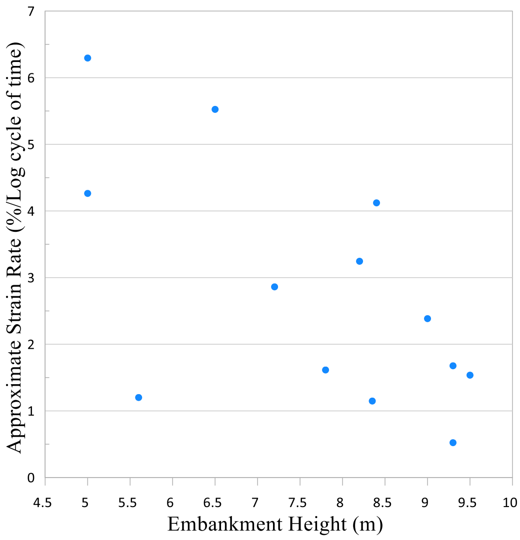


Figure 1b: Approximate strain rate over 5 years

Similarly, embankments constructed for major road infrastructure in Western Sydney have experienced settlement performance well beyond the rule of thumb magnitudes. In one example from two 14 m high embankments, 200 mm of surface settlement was observed in the 8 years post construction of the embankment. As these examples were located beneath pile raft foundations, no external vertical loading of the embankments was influencing the settlement behaviour. Samples extracted from this project were tested in the laboratory and compression of the order of 2 - 4% of the sample height were observed equating to the potential for the overall embankment compression to reach between 280 mm to 560 mm.

Western Sydney is underlain by Bringelly Shale. Bringelly Shale is interpreted by Herbert (1979) as a coastal alluvial plain sequence which grades up from a lagoonal-coastal marsh sequence at the base to increasingly more terrestrial, alluvial plain sediments towards the top of the formation. Lithologically, it comprises sequences that can be listed in order of decreasing volumetric significance as (1) claystone and siltstone (2) laminite (3) sandstone (4) coal and highly carbonaceous claystone.

Very little information is available in the literature regarding the engineering behaviour of Bringelly Shale fills. William (2004) and William & Airey (2005) provide basic properties of Bringelly Shale. They report that the mineralogy of Bringelly Shale comprises illite, mixed illite-smectite, kaolinite and quartz as major constituents and the slake durability of the material ranges from very low when weathered to medium when fresh. William (2004) also provides engineering parameters for intact rock and for reconstituted slurry material.

With the limited information provided in literature and authors' observations, Bringelly Shale particles break down due to changes in moisture content, causing settlement and such change may occur over a period.

Some evidence of long-term degradation of Bringelly Shale is provided in Figure 2. The figure shows a stockpile of Bringelly Shale, extracted from one of the brick quarries along Elizabeth Drive, that originally comprised cobble sized particles and has degraded over time to gravel size particles and finer. This degradation behaviour may contribute to the settlement of Bringelly Shale fill.

Figure 2: Observed degradation of Bringelly Shale

Pineda et al (2014a and b) studied the degradation of claystone due to environmental effects as those induced by stress relief (unloading) and the subsequent exposure of the rock to cyclic changes in relative humidity (RH). They demonstrated that the application of RH (suction) cycles causes a progressive degradation of the claystone. Degradation was quantified in terms of the accumulation of irreversible swelling strains (due to the expansion of clay minerals as well as micro-cracking), increase in the maximum water retention capacity, reduction in rock stiffness, increase in rock compressibility and permeability and reduction in shear strength parameters. Rock degradation increases with the number of RH cycles, increases further with the increase in the amplitude of RH, and reduces with the increase in the stress level. This is supported by observations from quarried batters in Western Sydney that experienced significant degradation of the formed surface in limited periods of time (circa 5 years) as illustrated in Figure 3 below.

Figure 3: Rock mass degradation on exposure in quarried batter

Research findings from Hopkins and Beckham (1998) in claystone and limestones indicated that major problems occurred in Shale fills because in many cases they tend to degrade from a hard, or indurated mass, to a fine-grained mass of soil. The degradation causes settlements and reductions in material strength. They reported that degradation of shale particles in embankments frequently occurred over a long period of time and many problems did not occur until several years after construction. They also reported that slope movement in shear has contributed to long term settlement and recommended to achieve a minimum long-term factor of safety of 1.5 to control settlement.

As discussed above, published literature indicates that shale particles shall break down when moisture movement either by direct wetting or change in humidity around the particles occur. For the purpose of this paper settlement caused by this mechanism has been classified as hydro-compression. It is expected that creep of engineered fill is likely to be minimal. However, it is difficult to differentiate creep and hydro-compression induced settlement in real world application. Therefore, for the purpose of this paper, both hydro-compression and creep has been termed as "internal compression".

Cox (1978) reports results from fully inundated oedometer tests performed on a claystone aimed at assessing how much the fill compresses or swells as a function of density and stress. Data from Cox (1978) is re-interpreted and presented in Figure 4. The data shows that the fill compresses at low density and low load as well as at higher density and higher load. As the density increases the fill has a propensity to swell at all loads. It is not clear whether these data include both hydro-compression and creep as the time-movement histories are not provided in the paper. Data from Waddell and Wong (2005) and unpublished compression test data from relevant construction projects are also plotted on Figure 4. Data from Hopkins and Beckham (1998) from a trial embankment constructed in Kentucky from non-durable shale are also shown in Figure 4. This embankment had internal compression of about 0.8% over about a 10-year period. However, the Hopkins and Beckham (1998) data plot in the swelling section of Figure 4. This demonstrates that accurate definition of compression or swelling is specific to a given material and possibly due to other factors such as moisture content with respect to optimum moisture content and clay mineralogy. Hopkins and Beckham (1998) also reported that relatively high embankment experienced internal compression even when material was well compacted. These settlements are considered partly due to slope movement and may be another reason why material classified under swelling zone in Figure 4 experience settlement.

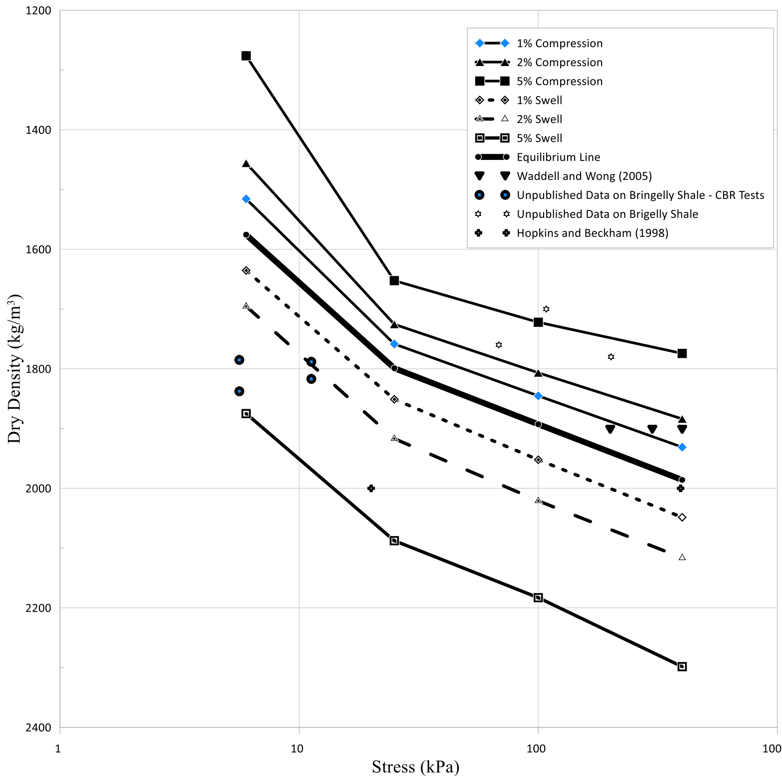


Figure 4: Re-interpretation Cox (1978) data

While Figure 4 appears to provide a means of estimating volume changes in fill it strictly only applies to the conditions adopted in the tests performed by Cox (1978). Different conditions would change the volumetric behaviour. For example, Alonso et al (2011) report soils compacted dry of optimum tend to have a more open void structure than soils compacted at or wet of optimum which can collapse on subsequent wetting. Therefore, two samples compacted to the same density and subject to the same vertical load could deform differently depending on the compaction moisture content.

Cardoso et al (2012) reported experimental test data on a claystone used in construction of a motorway embankment in Portugal. They found, like Cox (1978), that there are zones of compression and swelling when the data is plotted as a function of void ratio and stress. Fills that are heavily compacted such that they exist in the over-consolidated zone with respect to a saturated normal consolidation line tends to swell when inundated and fills that are relatively loose with open void structure that exist above a saturated normal consolidation line tend to collapse. Cardoso et al (2012) explained this behaviour in the context of the unsaturated Barcelona Basic Model (Alonso et al, 1990).

Cardoso et al (2012) also reported that whether or not a material shrinks, or swells is a function of grading as well as density. They found that heavily compacted samples and samples where voids were filled with fines tended to swell whereas uniform grading tended to compress. Intermediate behaviour where swelling initially occurred followed by compression was found at intermediate density and grading states.

Waddell and Wong (2005) performed long term oedometer tests on weathered and fresh Ashfield Shale. Their samples were compacted to 98% standard maximum dry density at 1% dry of optimum moisture content. The samples were then loaded and allowed to compress without inundation for several months. The samples were subsequently inundated and monitored for several more months. Results from these tests showed that compression prior to inundation was a function of applied load recording 4% at 200 kPa and 6% at 400 kPa. Results presented by Waddell and Wong (2005) show that samples compressed on inundation. However, the more lightly loaded sample subsequently swelled. This behaviour is consistent with that observed by Cox (1978) if it is assumed that the lightly loaded sample exists in the swelling side of the line of equilibrium and the more heavily loaded samples exist on the compression side of the equilibrium line. Waddell and Wong (2005) interpret compression post about 100 days as linear with the logarithm of time and report creep strain rate as a function of vertical stress in MPa. The creep coefficient was larger for weathered shale than for fresh shale. The creep coefficient for weathered Ashfield Shale was reported to be 0.027 times the vertical stress in MPa. If this is applied for an embankment height of 5 m to 10 m constructed in the rail infrastructure project discussed previously, internal compression strain rate will be 0.27% to 0.54% per log cycle of time. Whereas the observed movement in this project is not following a typical trend of increasing in settlement with embankment height, the measured internal compression strain rate of Bringelly Shale based on monitoring data from this rail project is several times higher than that reported by Waddell and Wong (2005) for Ashfield Shale.

The literature suggests that whether or not a shale material compresses or swells, their magnitudes and time rates of volume change, is a function of its mineralogy, density, load, grading, compaction moisture content, and subsequent changes in moisture content. While trends in qualitative behaviour are reasonably clear, methods for quantification of the magnitudes of settlement and time rate of settlement for engineering purposes is less clear.

From an engineering point of view, the aim is to control the magnitude and rate of volumetric changes in a fill at least cost. Transport for NSW issued technical note TN033 in 2016 in which they noted that expansive material with similar properties to Bringelly Shale is considered unsuitable and should not be used in fills. The note requires conventional material suitability tests to be performed and allows the use of expansive materials blended with non-reactive soils for use as general fill. Expansive material can also be chemically stabilised to improve its properties. Both of these approaches are expensive and will increase the cost of construction in Western Sydney. An alternative approach is to use earthworks processes to limit deformations. Hopkins and Beckham (1998) report a process of compacting in maximum 200mm thick lifts, compacting at a moisture content of optimum +/-2% using a heavy rotary disc to mix soils, compact to minimum 95% standard maximum dry density using heavy compactors with a minimum of 3 passes of a static roller (minimum 27.4 Tonne) and 2 passes of a vibratory roller (minimum 24.9 Tonne). It is also recommended to apply adequate water to slake the material and promote breakdown and achieve majority of settlement during construction. The aim of this process was to accelerate breakdown of materials through mechanical means and through use of relatively high-water contents with an intention to keep the moisture content on wet of optimum as much practically as possible. In addition, they recommend a batter slope of 2.5H:1V to 3H:1V with a minimum factor of safety of 1.5 required to reduce the long-term settlement caused by slope movement with height more than 9 m. This method was reported to reduce long term settlements of embankments up to 20 m high from 0.3 m - 0.9 m in 10 years to 0.15 m - 0.18 m in 27 years. BRE (1998) provide similar recommendations for low rise buildings constructed on clay fills to reduce the potential for collapse compression. BRE (1998) recommend that fill be placed at densities ranging approximately between standard and modified maximum dry densities with air voids less than 5% (i.e. wet of optimum moisture content).

Although Bringelly Shale-based fill material has shown significant settlement at the rail and road infrastructure projects in Western Sydney as discussed above, to the authors' knowledge, no references have been found in the technical literature that provide some guidance on assessing the magnitude and time rate of internal compression of Bringelly Shale based fill material or how internal compression can be controlled.

Based on observations and literature discussed above, authors consider that internal compression of compacted Bringelly Shale fill is likely to be caused by moisture movement. This moisture movement can be:

- Direct wetting (Soaking): This is applicable to situation where free water movement occurs (e.g. earthworks in touch with water at bottom and top layers such as quarry filling and narrow embankments). This process will change overall moisture content of the fill.

- Moisture movement within compacted mass (i.e. at compacted moisture content – no soaking): This is applicable to situation where overall moisture content doesn't change due to external water movement (e.g. large landform constructed above ground surface). However, moisture content may not be uniform within compacted fill after placement (though overall moisture content of compacted fill is constant), hence moisture movement/humidity changes is likely to occur over time to reach a uniform moisture content.

Observed settlement of embankments in the rail and road infrastructure projects could be due to both mechanisms discussed above. Studying combined effects of these two mechanisms is very complex. Hence, a laboratory program has been developed to understand these mechanisms separately. A series of laboratory tests has been conducted to understand the settlement behaviour of compacted Bringelly Shale fill. Compaction tests, particle size distribution, Atterberg limits, and small and large-scale compression tests of compacted fill material have been performed. This paper summarises and discusses some of the findings of these laboratory tests.

2 LABORATORY TESTING PROGRAMME

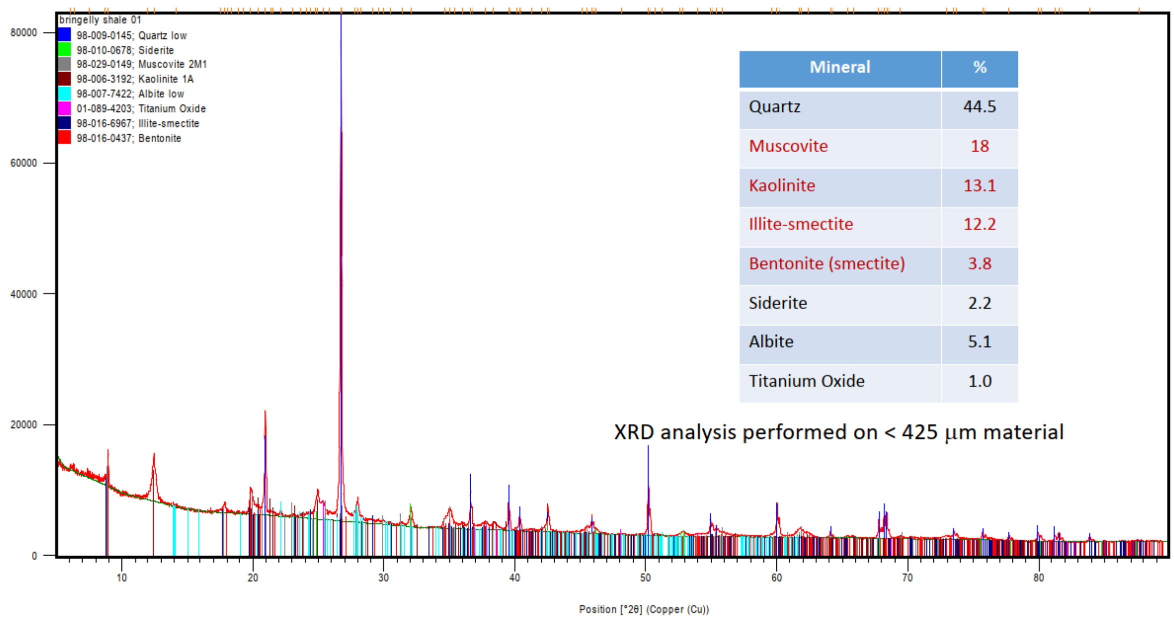
2.1 MATERIAL PROPERTIES

Shale material for testing has been sourced from a site in the Western Sydney and is classified as shown in Table 1. The shale was classified into BS2 and BS3 type weathered rock materials and residual clay.

Table 1: Material classification of Bringelly Shale

Strength Description	**Point load index Is (50) MPa**		**Material Type**	**Material description**
	From	**To**		
Residual clay	-	-	Soil	Soil derived from complete weathering of Bringelly Shale or dyke material
Soil Strength to Very Low Rock	-	0.03	BS3	Bringelly Shale: Interbedded/interlaminated sandstone and siltstone, and claystone
Low to medium	0.1	0.3	BS2	

A quantitative X-Ray diffraction analysis was carried out on BS2 material using rock powder sieved through 75 μm sieve. The X-Ray spectrogram is shown in Figure 5. A quantitative analysis indicates that it is composed by quartz (44.5 %), muscovite (18%), kaolinite (13.1 %), mixed illite-smectite (12.2 %), smectite (3.8 %), siderite (2.2 %), and albite (5.5 %). This represents around 47.1 % of clayey minerals. The results indicate less clay minerals than reported by William (2004).



Mineral	%
Quartz	44.5
Muscovite	18
Kaolinite	13.1
Illite-smectite	12.2
Bentonite (smectite)	3.8
Siderite	2.2
Albite	5.1
Titanium Oxide	1.0

Figure 5: X-Ray diffraction test conducted on BS2 material

Particle size distribution of residual clay, BS3 and BS2 are provided in Figure 6. In general, residual clay and BS3 consists of higher fine content than BS2. Particle size distribution of residual clay, BS3 and BS2 materials have been estimated according to the AS 1289.3.6.1. Particle size distribution curves are provided in Figure 6 where two PSD curves are shown for BS2. They represent the two different batches of samples used in the laboratory testing campaign. Sample 1 displays a fine fraction (< 75 µm) around 4% whereas it increases to 27% in Sample 2. Material BS3 and residual soil show fine fractions of 84% and 80%, respectively.

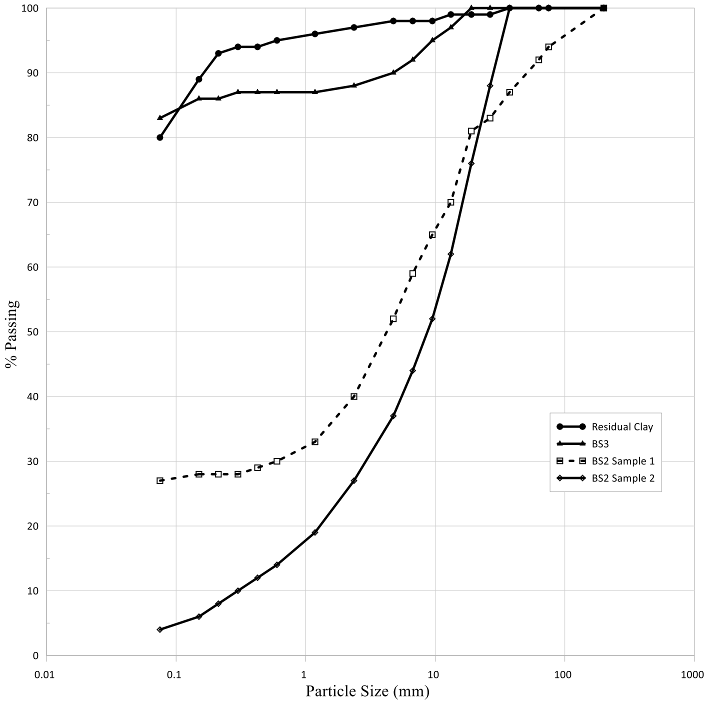


Figure 6: Particle size distribution of residual clay, BS3 and BS2

Liquid limit and plastic limit tests have been carried out to evaluate the plasticity of the tested materials. Atterberg limits were estimated following the procedure described in AS 1289.3.1.1, AS1289.3.2.1 and AS1289.3.3.1. Test results are summarised in Figure 7. Results indicate that plasticity of BS2 material is lower than BS3 and residual soil. This is attributed to the fact that BS2 has lesser fines than BS3 and residual clay. The residual soil classifies as high plasticity clay (CH) whereas materials from BS2 and BS3 are both low plasticity clays (CL).

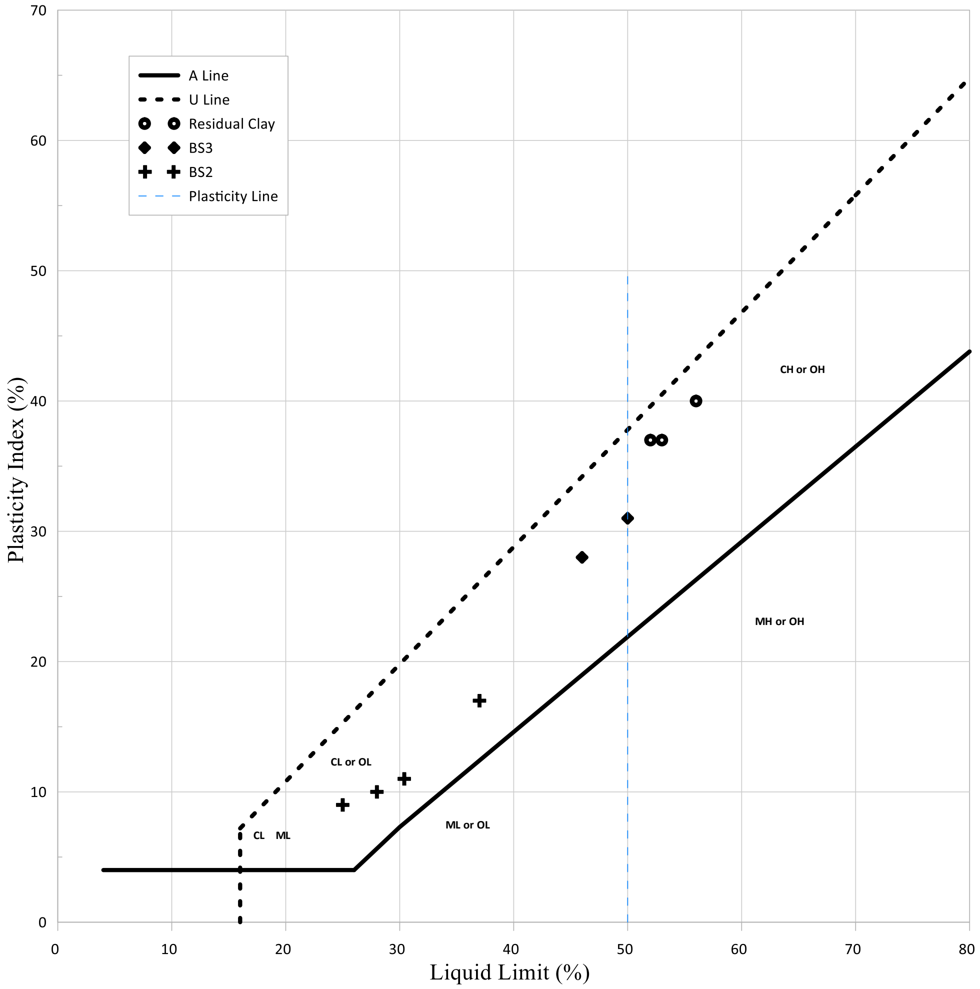


Figure 7: Atterberg limits of residual soil, BS3 and BS2

Standard and Modified Proctor compaction tests have been conducted on residual clay, BS3 and BS2 materials. Standard and Modified Proctor compaction curves were estimated following the procedure described in AS 1289.5.1.1 and AS1289.5.2.1, respectively to interpret maximum dry density (MDD) and optimum moisture content (OMC). The maximum dry density and optimum moisture content values are provided in Table 2. Maximum dry densities of BS3 and residual clay are lower than BS2 material while the optimum moisture content is higher than BS2 material. This is expected considering the increased fine content in BS3 and residual clay than in BS2.

Table 2: Compaction test results for residual clay, BS3 and BS2

Material Type	**Number of tests**	**Compaction type**	**Maximum Dry Density, t/m^3**	**Optimum moisture content (%)**
Residual Clay	3	Standard Compaction	1.74 ± 0.006	17.2 ± 0.60
	3	Modified Compaction	1.91 ± 0.010	11.95 ± 0.55
BS3	3	Standard Compaction	1.81 ± 0.035	15.35 ± 1.45
	3	Modified Compaction	1.99 ± 0.007	9.95 ± 0.45
BS2	2	Standard Compaction	2.09 ± 0.015	10.0 ± 1.5
	2	Modified Compaction	2.16 ± 0.020	6.65 ± 0.35

2.2 STUDY VOLUMETRIC BEHAVIOUR OF BRINGELLY SHALE

Two testing regimes have been carried out to study the volumetric behaviour of Bringelly Shale fill material a) under direct soaking (external water application) and at as-compacted moisture content (no external water application) as discussed below.

2.2.1 Testing Regime 1 – Study on volumetric behaviour under direct wetting

Two mechanical tests have been used to evaluate the influence of the compaction state, stress level and saturation state on the volumetric behaviour of compacted Bringelly Shale specimens. Small-scale and large-scale one-dimensional compression tests were used for this purpose. Small-scale tests were performed in a stainless steel oedometer apparatus with sample dimensions of 48 mm in diameter and 20 mm in height. In this device, top and bottom drainage lines are connected to sealed containers which may be filled with specific salt solutions to maintain a required relative humidity during the test. This feature reduces the possibility for moisture losses due to evaporation.

On the other hand, a 300 x 300 mm direct shear box device was used to evaluate the volume change of compacted Bringelly Shale in large scale tests. Squared specimens with 300 mmx300 mm plan area and 51.5 mm in height were compacted statically in the shear box until achieving the required density. The direct shear box was covered with plastic film to isolate the specimen from the atmosphere and reduce moisture losses due to evaporation.

To study the volumetric behaviour of compacted Bringelly Shale due to an increase in moisture content, a series of wetting upon loading tests has been conducted on material from BS2. Small scale and large-scale compression tests were carried out as summarized in Table 4. Samples for small scale tests 1 to 9 noted in Table 3 have been prepared by statically compacting material in standard oedometer cells to a required density. In all tests, specimens have been statically compacted using a displacement rate of 1 mm/min. After compaction, load has been applied incrementally to a required vertical stress before wetting the samples. To apply 30 kPa vertical stress on sample, load has been applied incrementally at 5 kPa, 15 kPa and 30 kPa. Similarly, to apply 300 kPa vertical stress on sample, load has been applied incrementally at 5 kPa, 15 kPa, 30 kPa, 80 kPa, 160 kPa and 300 kPa. Each load increment has been applied typically over a period of 24 hours. Samples were inundated for a minimum of 3 days and up to 6 days to observe swell and compression behaviour.

Various stress levels, compaction effort and moisture content with respect to optimum moisture content have been used to study the influence on volumetric behaviour.

Table 3: Hydro-compression laboratory tests on BS2 under direct wetting

Test ID	Applied stress at soaking, kPa	Maximum particle size	Actual compaction conditions
1	30	2.36 mm	Modified proctor, dry density of 2.07 t/m^3 and 7.2% water content (+0.5% of optimum)
2	300		
3	30		Modified proctor, dry density of 2.07 t/m^3 and 4.7% water content (2 % dry of optimum)
4	300		Modified proctor, dry density of 2.07 t/m^3 and 6.0% water content (0.7% dry of optimum)
5	30		Modified proctor, dry density of 2.07 t/m^3 and +10.9% water content (4.2% wet of optimum)
6	300		
7	30		Standard proctor, dry density of 1.94 t/m^3 and +12.7% water content (1.2% wet of optimum)
8	300		
9	300		Standard proctor, dry density of 1.94 t/m^3 and 10.6% water content (0.9% dry of optimum)
10	30	19 mm	Standard proctor, dry density of 1.97 t/m^3 and moisture content of 9.3% (wet of optimum)

2.2.2 Testing Regime 2 – Compression tests under as-compacted water content

As discussed previously, post-compaction moisture transfer within the fill could still occur as part of the moisture equilibrium process, leading to similar hydro-compression phenomenon as direct wetting. In order to explore this phenomenon, a series of small-scale compression tests and a large-scale compression test have been conducted (i.e. without inundation). As part of sample preparation for small scale compression tests, moisture content of in-situ material has been measured and compared with target moisture content for each compression test. Where the target moisture content for compression test was higher than in-situ moisture content, additional water has been added. Where the target moisture content for compression test was less than in-situ moisture content, in-situ material has been air-dried. Small-scale test samples were then compressed in a stainless steel oedometer apparatus with sample dimensions of 50 mm in diameter and 20 mm in height to achieve target density. The test specimen including the cell was covered with plastic film to isolate the specimen from the atmosphere and reduce moisture losses due to evaporation.

Small-scale samples prepared using particles passing through 2.36 mm and compacted to 100% standard maximum dry density (SMDD) or 97% of modified maximum dry density (MMDD) have been tested for an extended period. A summary of tests conducted at 100% standard maximum dry density is provided in Table 4.

Table 4: One dimensional compression tests on Residual clay, BS3 and BS2 material compacted to 100% standard maximum dry density – (Small scale samples)

Material	Stress Level (kPa)	Density t/m^3	Moisture content (%)	Period of test (Days)
Residual soil	200	1.73	15.8 (Optimum -2%)	22
	200	1.74	19.7 (Optimum+2%)	22
	200	1.74	16.5 (Optimum)	98
	100	1.72	19.1 (Optimum+1.2%)	19
BS3	160	1.82	16.4 (Optimum +2%)	96
	80	1.84	13.6 (Optimum)	19
	160	1.82	16.7 (Optimum +2.5%)	18
	160	1.82	14.7 (Optimum +0.8%)	18
BS2	160	2.09	9.0 (Optimum)	97
	80		9.4 (Optimum)	92
	160		11.8 (Optimum + 2%)	24
	160		7.5 (Optimum -2%)	24

In addition, one compression test with a large-scale sample (300 mm x 300 mm x 51.5 mm sample) and three compression tests with small-scale samples (compacted in oedometer cells) compacted to 97% modified maximum dry density and optimum moisture content have been conducted for BS2. A summary of tests conducted is provided in Table 5.

Table 5: One dimensional compression tests on Residual clay, BS3 and BS2 material compacted to modified maximum dry density at – (Small scale and large scale samples)

Material	Stress Level (kPa)	Degree of Compaction	Maximum particle size	Moisture content (%)	Period of test (Days)
Residual clay	100	Dry density of 1.9 t/m^3 (100% Modified proctor)	2.36 mm	12 (Optimum)	68
	200			11.5 (Optimum)	68
BS3	80	Dry density of 2.0 t/m^3 (100% Modified proctor)		9.9 (Optimum)	68
	160			9.8(Optimum)	68
BS2	50	Dry density of 2.1 t/m^3 (97% Modified proctor)		6.4 (Optimum)	43
	100	Dry density of 2.11 t/m^3 (97% Modified proctor		6.7 (Optimum)	42
	200			6.2 (Optimum)	43
	30		19 mm	7.1 (Optimum)	6

3 RESULTS

Results of the laboratory tests are summarised in the following sections.

3.1 TESTING REGIME 1 – STUDY ON VOLUMETRIC BEHAVIOUR UNDER DIRECT WETTING

A typical test result is shown in Figure 8 where volumetric strain is plotted against time. A small amount of compression occurred while loading prior to inundation. Swelling then typically occurred after inundation at most stress levels with the exception of a few samples loaded to 300kPa. Further swelling then occurred during unloading. The compression stage was generally completed within about 100 minutes while small amounts of swelling were still occurring up to 5,700 minutes.

The results of the testing programme are summarised in Figure 9 where volumetric strain is plotted against vertical stress as a swell-pressure plot for the inundation stage of the tests. Samples compacted to modified density swelled more than samples compacted at standard density. Samples compacted dry of optimum swell more than those compacted at optimum and greater moisture content. Swelling generally reduced as compaction moisture increased except for samples compacted to standard density and dry of optimum. Swell reduced as the vertical stress increased. Compression occurred in two of the samples compacted to standard density and one of the tests compacted to modified density when loaded to 300kPa.

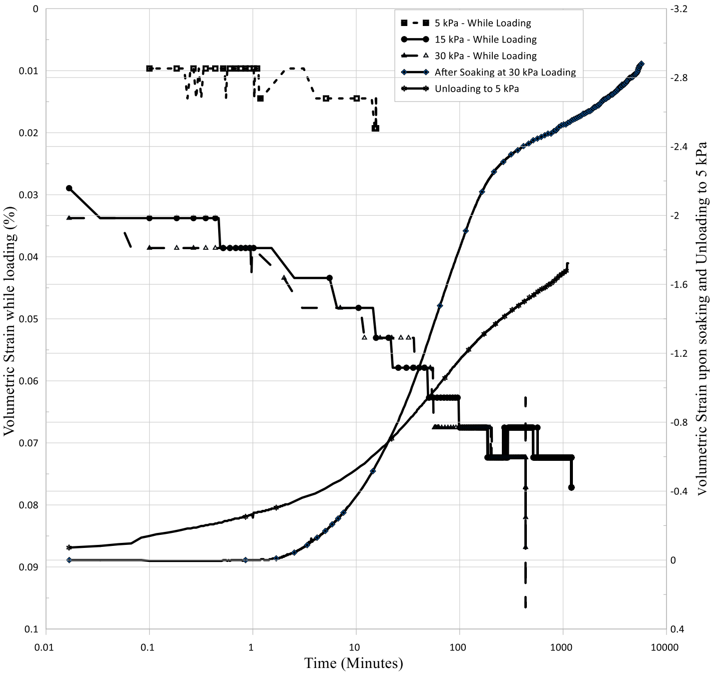


Figure 8. Typical test result

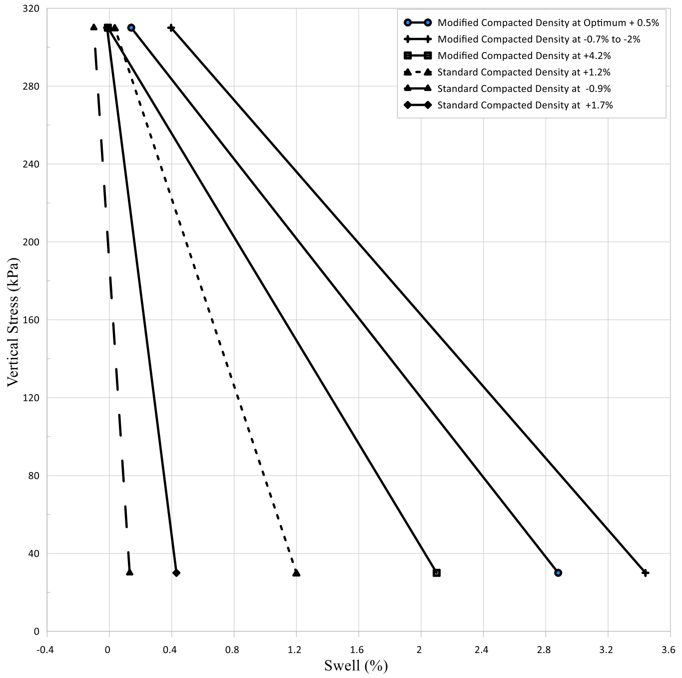


Figure 9: Summary of inundation testing

3.2 TESTING REGIME 2 – COMPRESSION TESTS UNDER AS-COMPACTED WATER CONTENT CONDITIONS

Time-strain plots for the tests on residual soil are presented in Figures 10 and 11. Four tests were performed up to 32,000 minutes and three tests were performed between 98,000 and 160,000 minutes. The tests performed to 32,000 minutes were compacted to standard density at different moisture contents. The strain at 32,000 minutes was greatest for the sample compacted wet of optimum and reduced as the compaction moisture content reduced. The test performed to 160,000 minutes with sample compacted at 100% standard density and optimum moisture content appeared to reach an asymptote compressive strain of 7%. The tests compacted to modified density initially swelled and then started to compress. The swelling increased in inverse proportion to the applied load. The time to start of compression increase also in inversely proportional to the applied load.

The moisture content at the start of the tests was typically a few percent higher than the moisture content at the end of the tests. Some evaporation appears to have occurred and that could have contributed to the compressive strains and is considered as a limitation of the study. While evaporation complicates the interpretation of laboratory test data, it will occur naturally in the field which could contribute to real world performance.

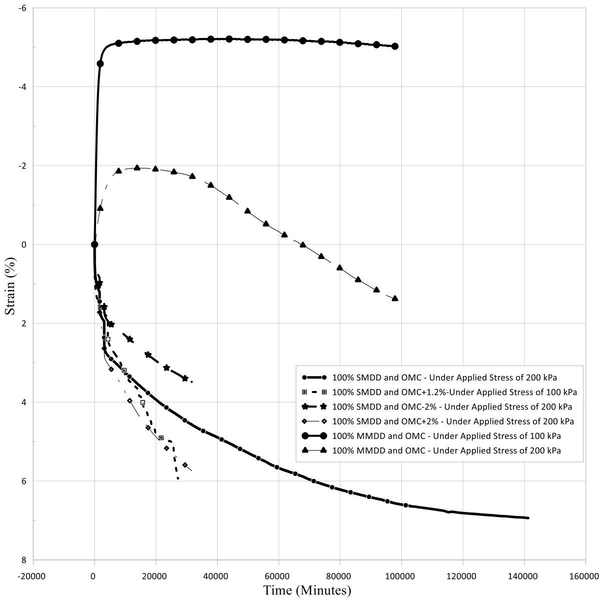


Figure 10: Strain versus time for Residual soil

The data in Figure 10 plotted against the logarithm of time shows that the rate of strain varies with time. The strain in the long-term test at 100% standard compaction has a shape broadly similar to a consolidation curve where the initial rate is relatively small then increases with the logarithm of time and then reduces at large times. The long-term tests at 100% modified compaction have shapes similar to unloading creep tests in soft soils. The time to the start of compressive strain varies with load and the rate of strain increases with time. At the end of these tests the rate of strain is approximately linear with the logarithm of time, however that might have changed if the tests were allowed to run for a longer period of time.

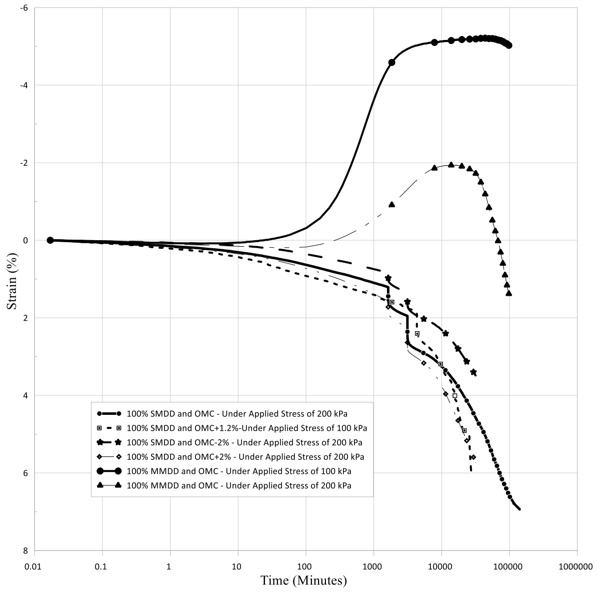


Figure 11: Strain versus log time for Residual soil

Time versus strain plots for BS3 and BS2 materials are shown in Figures 12 to 15. The data shows similar trends to residual soil. Compression increases as density reduces and as moisture increases, although all BS3 test wet of optimum and the BS2 test wet of optimum do not follow these trends. The magnitude of compression at 35,000 minutes of BS3 is less than that for residual soil but is similar to the BS2 sample. Swelling of the BS3 material compacted at 100% modified density is less than residual soil compacted at 100% modified density and compression of BS2 material occurred when compacted at 97% modified density. The trends of swelling / compression with load for the tests at modified compaction are the reverse of what would be expected. However, the magnitude of the movements is small and could be explained by slight variations in grading and density.

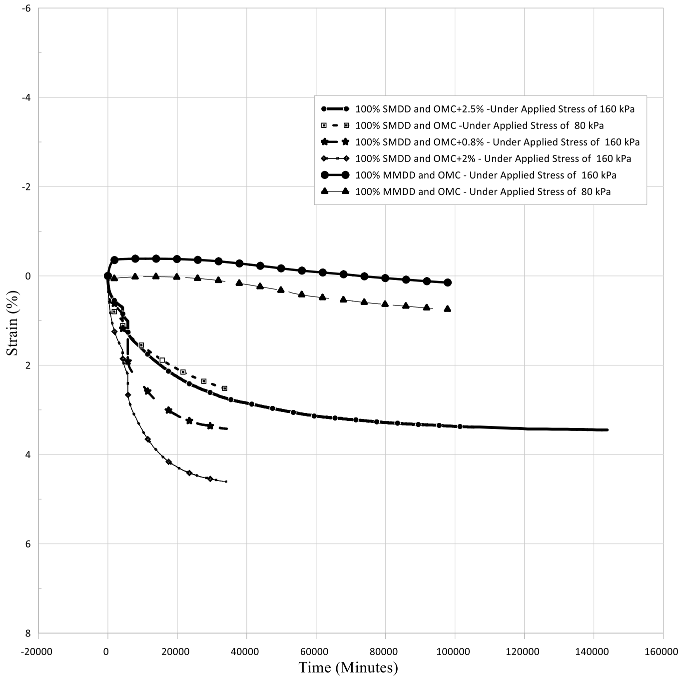


Figure 12: Strain versus time for BS3

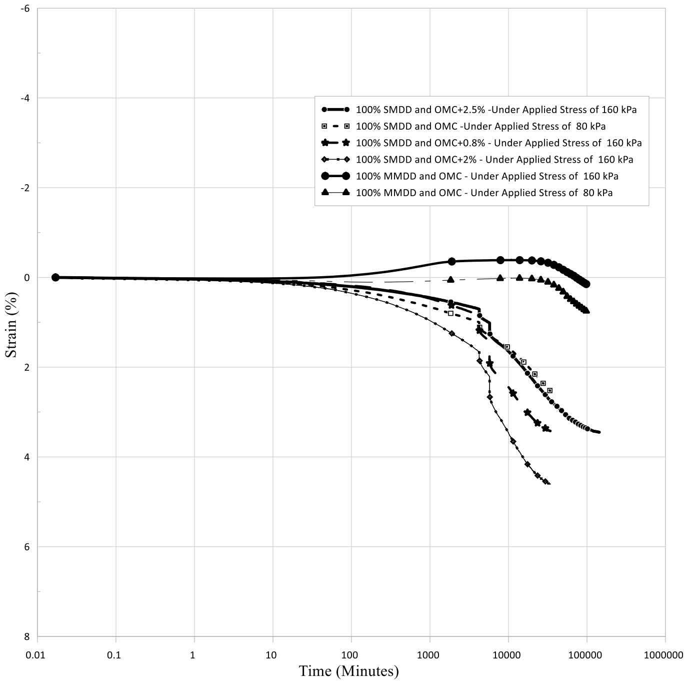


Figure 13: Strain versus log time for BS3

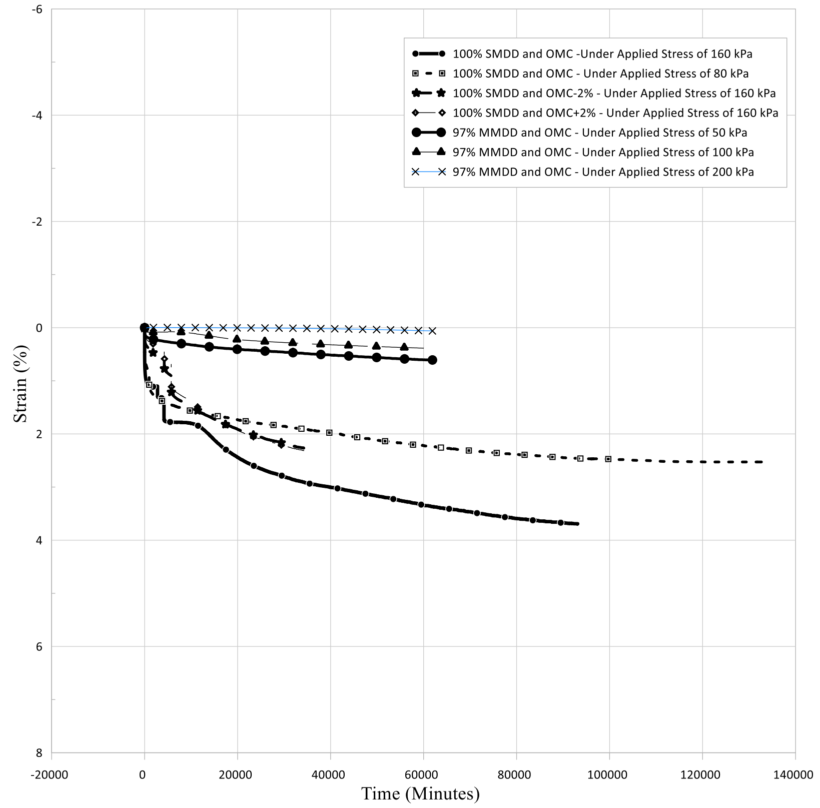


Figure 14: Strain versus time for BS2

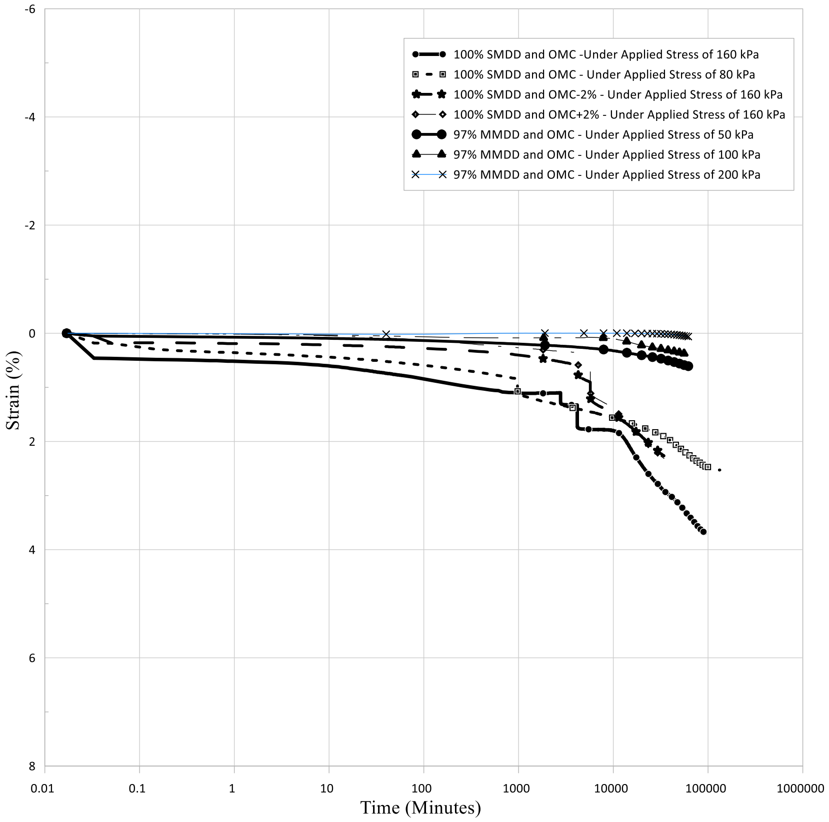


Figure 15: Strain versus log time for BS2

3.2.1 Rate of Internal Compression

The time-settlement behaviour of a compacted sample indicates that the rate of internal compression of fill appears to vary with time, density, stress and moisture content as discussed above for small-scale samples.

A typical settlement behaviour of a compacted sample of BS2 indicates that the internal compression of fill appears to occur at two different stages as shown in Figure 16 for small-scale samples at applied load of 160 kPa.

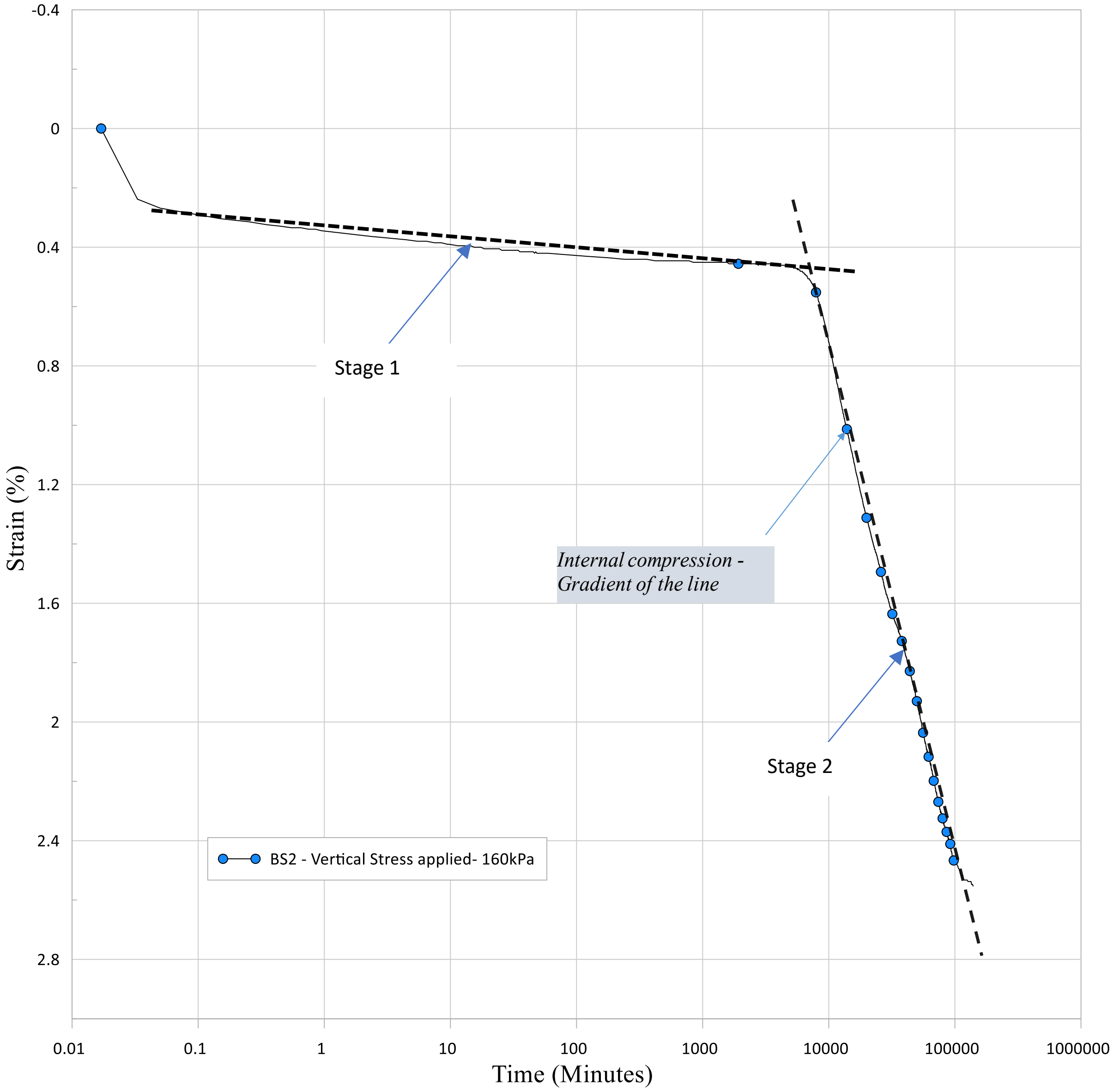


Figure 16: Interpretation of internal compression from compression test on material Type BS2

Authors' interpretation of Figure 16 is that the soil and shale particles act like heavily over-consolidated clays i.e. having high suction. The free water used during compaction moves through the soil and shale particles which causes softening of particle contacts as well as breakdown of particles. This softening of interparticle contacts, and breakdown of unsaturated particles with high suction causes settlement. Based on the compression test results, typical internal compression behaviour is a time dependent phenomenon and varying approximately linearly in log time scale at two different rates, namely Stage 1 and Stage 2 as shown in Figure 16.

Rates of internal compression at large time for BS2 samples compacted to 97% of modified maximum dry density is 2.11 t/m3 and standard maximum dry density of 2.1 t/m3 are shown in Figure 17. Rates of internal compression in percentage are approximately linear with applied stress and are 0.012 x vertical stress in kPa per log cycle of time. This rate is approximately 4 to 5 times greater than the value of 2.7 times vertical stress in MPa (in percentage) reported by Waddell and Wong (2005) for weathered Ashfield Shale. Rates of internal compression for residual soil and BS3 soils are also shown in Figure 18. Trendlines fitted through the origin also show high rates of internal compression.

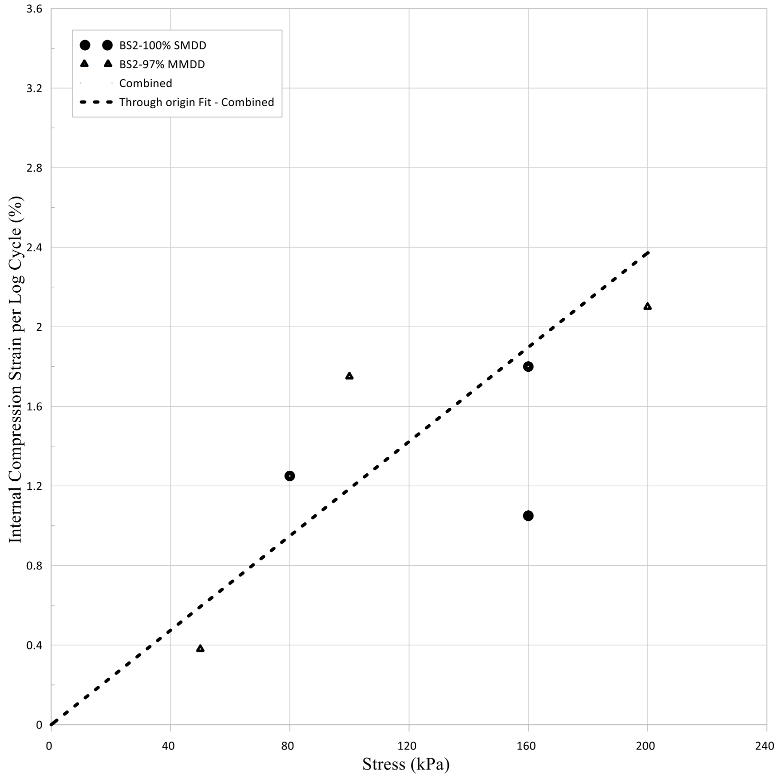


Figure 17: Internal compression strain rate with stress - BS2 (Small scale samples)

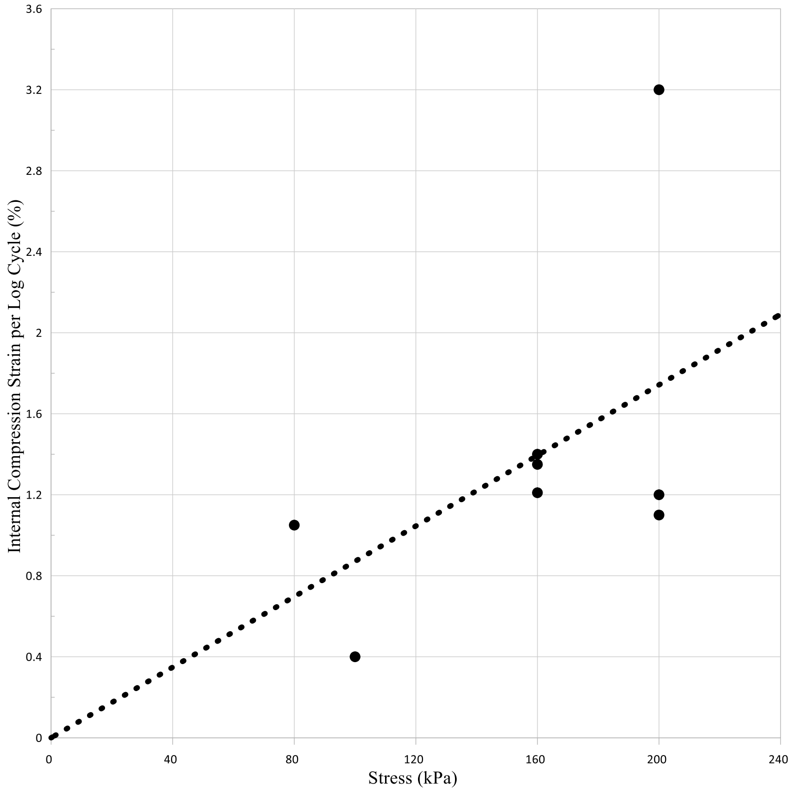


Figure 18: Internal compression strain rate with stress – BS3 and Residual Clay (Small scale samples)

3.3 LARGE SCALE TESTS

Time strain plots for BS2 material compacted to 100% maximum standard dry density and 100% maximum modified dry density at the optimum moisture content are shown in Figure 19. The samples were initially loaded to 30 kPa, allowed to compress for a period and then inundated with water. On loading, the test compacted to standard density compressed much more than the test compacted at modified density. On inundation the test compacted to modified density swelled slightly more than the test compacted at standard density but swelling was small in both cases. These data are compared with similar small scale oedometer tests in Figure 19. Densities and moisture contents for these tests are provided in Table 6 and it can be seen that they are similar. The compression on loading in the oedometer tests are broadly similar to the shear box test compacted to modified density but are much smaller than the test compacted to standard density. Swelling on inundation in the oedometer tests is much larger than swelling on inundation in the shear box. In general, results show that the behaviour of Bringelly Shale varies with the particle size distribution and scale of the laboratory test.

Table 6: Densities and moisture contents for shear box and oedometer tests on BS2 material

Test	Moisture content (%)	Dry density (t/m^3)
Modified Shear box	7.1	2.11
Standard Shear box	9.3	1.97
Modified Oedometer	7.2	2.07
Standard Oedometer	10.6	1.94

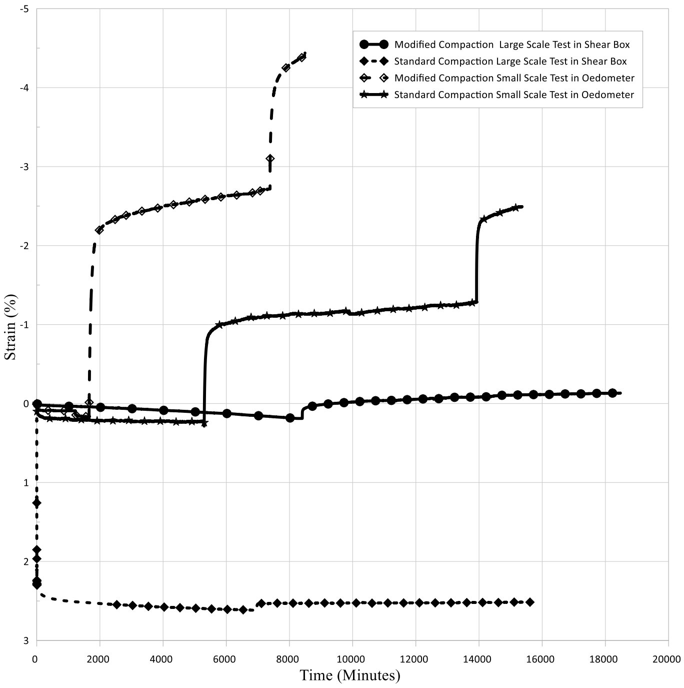


Figure 19: Comparison of oedometer and shear box inundation tests in BS2 materials

4 DISCUSSION

4.1 PREDICTING THE MAGNITUDE OF VOLUMETRIC CHANGE – WHEN INUNDATED

The results of the laboratory tests indicate that the volumetric changes in compacted Bringelly shale are a function of density, stress and moisture content (suction). This is in agreement with the behaviour of other compacted fills reported in the literature (e.g. Brandon et al., 1990; Noorany and Stanley, 1994; Cardoso et al., 2012). Predicting the magnitude of volume changes should take all of these factors into account. Use of a contour plot similar to Cox (1978) only takes stress and density into account but it has been shown that samples compacted to the same density with the same applied load have different magnitudes of volume change depending on their compaction moisture content and particle size. Similarly, attempting to develop strain contours in the compaction plane can capture effects of density and moisture content but not the effects of applied stress. Unsaturated soil mechanics provides a framework that can take all three factors into account. Available experimental evidence can be studied from the Unsaturated Soil Mechanics perspective (e.g., Alonso et al, 1990).

A simplified version of the elasto-plastic Sheng, Fredlund, Gens (SFG) model (Sheng et al, 2008) has been adopted. It is assumed that the density and stress level adopted in the laboratory tests create a soil state that is over-consolidated in comparison with a saturated normal compression line and collapse type mechanisms do not occur. The SFG (2008) model is expressed in terms of normal compression lines but it also applies to swelling lines (Prof Sheng, personal communication).

In this model, volumetric strains are estimated from Equation 1 where $d\varepsilon_v$ is the increment of volume strain, κ_{vp} is the swelling line for saturated soils, κ_{vs} is the swelling line for unsaturated soils, P is the mean total stress and S is suction. The volumetric change has been assessed using one dimensional analysis. Changes in swelling line with suction are described in Equation 2. In Equation 2 S_a is the air entry suction.

$$d\varepsilon_v = -\kappa_{vp}\left(\frac{dP}{P+S}\right) - \kappa_{vs}\left(\frac{dS}{P+S}\right) \qquad (1)$$

$$\kappa_{vs} = \kappa_{vp} \qquad S \leq S_a \qquad (2a)$$

$$\kappa_{vs} = \kappa_{vp}\frac{S_a+1}{S+1} \qquad S > S_a \qquad (2b)$$

The parameters κ_{vp} and κ_{vs}were obtained from each individual inundation test as shown in Figure 20. The κ_{vs} parameters were taken as the secant line from initial stress to target vertical stress prior to inundation. The κ_{vp} parameters were obtained from the slope of the unloading line after inundation in each of the individual tests. Soil suction can be obtained using Equation 2 as a function of the stiffness parameters and the air entry suction. Volume strain is then obtained from Equation 1 adopting incremental steps in suction. A least squares data fit between measured and predicted parameters is then performed to obtain the air entry suction and optimise the predicted volume strain. A comparison of predicted swell during the inundation phase of the test with measured values shown in Figure 21 corresponding to an air entry suction of 39 kPa. There is reasonable agreement between measured and predicted swell. It is not known whether the adopted air entry suction of 39 kPa is accurate, however a low value is consistent with a granular material such as BS2.

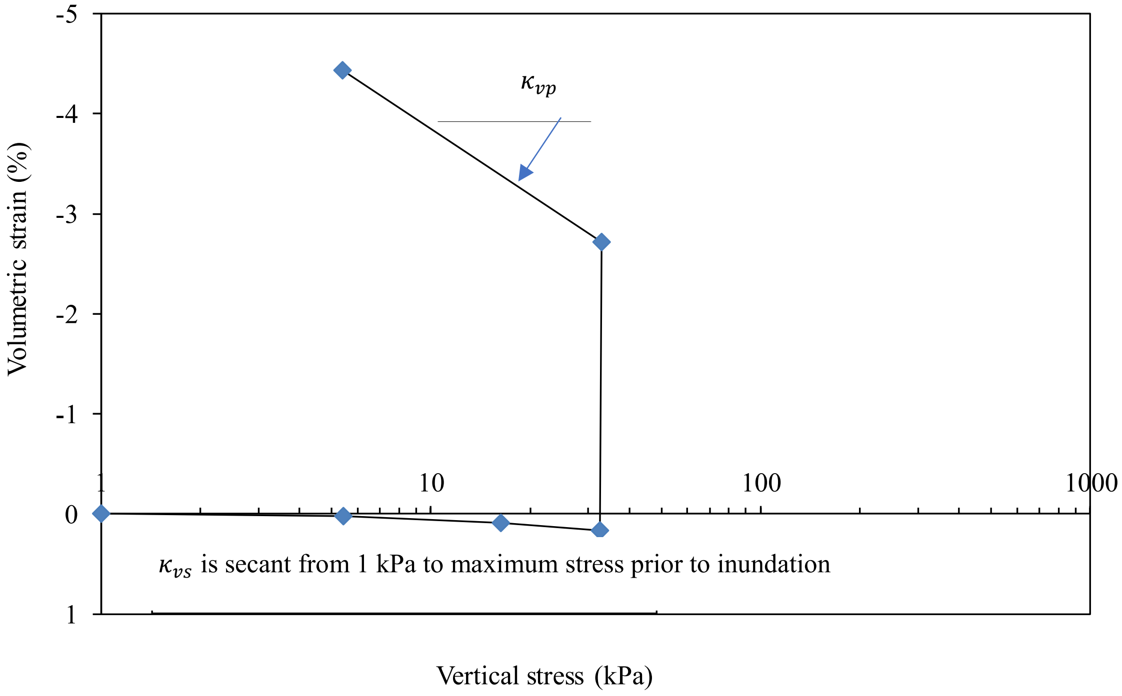


Figure 20: Typical test result

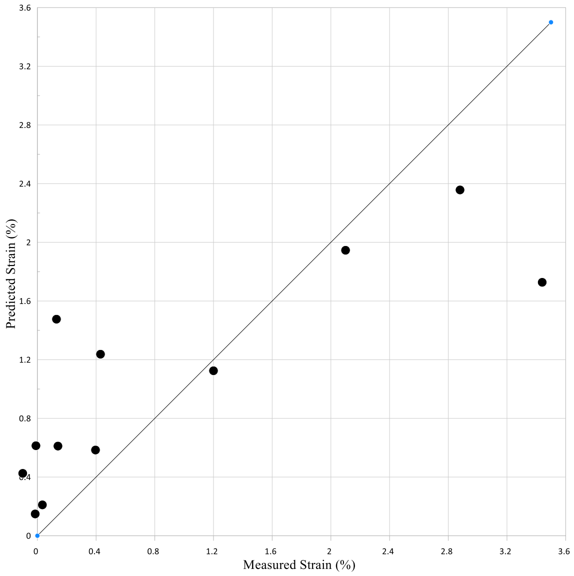


Figure 21: predicted versus measured swell

The inundation tests effectively provide the soil with access to a large amount of liquid water capable of inducing full saturation. This, in turn, leads to the samples to swell.

In principle, a suite of laboratory tests may be performed to facilitate prediction of the internal compression of Bringelly Shale fill. The laboratory tests would have to be performed at sufficiently large scale to capture the particle size distribution expected in practice. A soil water retention curve would be required to relate moisture content to suction and to obtain the air entry suction. One dimensional test where samples at field moisture content and target dry density are loaded to a target pressure and then inundated would provide the κ_{vp} and κ_{vs} values. The rate of compression could be estimated if the permeability of the intact soil particles was known.

The above approach can be used to predict settlement for a scenario where fills are likely to be inundated during the design life such as quarry fills.

4.2 PREDICTING THE MAGNITUDE OF VOLUMETRIC CHANGE – UNDER AS-COMPACTED MOISTURE CONTENT

As presented previously, laboratory test results at as-compacted moisture content indicated that Bringelly Shale fill settles with time (even without inundation). This scenario is applicable to a large landform with a minimal likelihood of change in overall moisture content due to external factors such as groundwater rise and rainfall.

When samples were prepared to study volumetric behaviour under as compaction moisture content (i.e. constant water content tests), required amount of water has been added when moisture content of in-situ material was less than target moisture content or air dried when the moisture content of in-situ material was higher than target moisture content. This process is generally similar to what will be adopted during construction. The test results show that samples compacted to standard density tended to compress while samples compacted to modified density either compressed, swelled or did both. Considering material preparation method (i.e. either air drying or adding water air dried material) it is likely that water content will be non-uniform within the sample. Moisture will therefore move within the sample to come into moisture equilibrium, leading to break down of shale particles. In order to study this behaviour in unsaturated soil mechanics discussed above, variation of moisture within a sample should be studied rigorously. In practice tests of this nature are unlikely to be performed unless the infrastructure being constructed on the fill is particularly expensive and sensitive to movement. A less involved method of estimating internal compression is to perform laboratory tests to obtain the rate of settlement with the logarithm of time at various applied stress levels and use the correlation to estimate compression over a time period. As an example, the BS2 data provided in Figure 17 has been used to estimate settlement over time. The method of calculation was based on the following:

- Fill is subdivided into 1m thick layers and the stress at the centre of each layer estimated assuming a bulk unit weight of 21 kN/m^3;
- Start time for calculations is assumed to be 1 year;
- Calculations were extended to 40 years; and
- Strain calculated using the conventional equation for creep in logarithmic time.

The assessed internal compression rate with respect to embankment height is plotted in Figure 21.

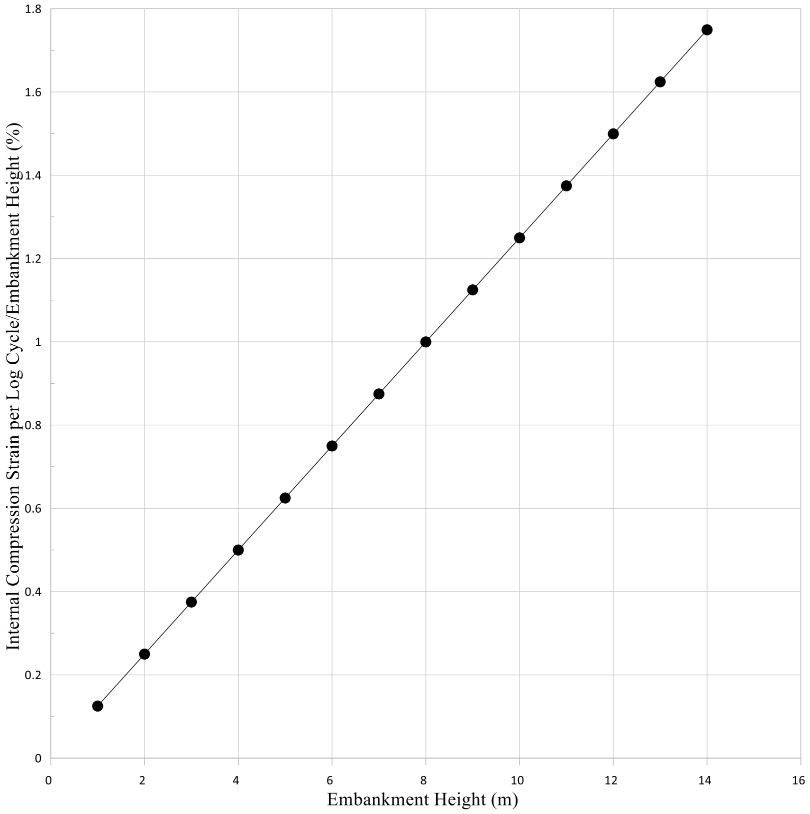


Figure 21: Strain estimated for Weathered Bringelly Shale

4.3 PRACTICAL MEANS OF CONTROLLING VOLUME CHANGES IN BRINGELLY SHALE IN DEEP FILLS

A typical cut to fill operation will start by removing residual soil and placing at the base of the fill then progressively winning and placing less weathered material. That means that the soils most prone to swelling and compression in the tests reported in this paper will be the most heavily loaded. In order to reduce internal compression of the fill the following process could be adopted:

1. Compact the lowest soil layers to modified maximum dry density. Increasing density has been shown to reduce compression;
2. Progressively reduce the compactive effort as the height of the fill increases and load applied to the fill layer reduces. However, higher compactive effort in the upper layers of fill provides some risk management as it would be more prone to swell hence counteracting any compression that occurs deeper in the fill.
3. Compact soils at optimum moisture content or wet of optimum. The intention is to maximise compression on loading by reducing the unsaturated stiffness of the soil and to maximise volume changes during construction rather than post construction. Compacting dry creates a stiff fill but long-term changes in moisture content and humidity will increase post construction internal compression.
4. Rigorously mix the water into the fill to maximise its distribution. The rotary hoe method adopted by Hopkins and Beckham (1998) aims to achieve a high degree of mixing.
5. Adopt thin layers and use heavy compactors in order to reduce the average particle size, maximise density and fill voids with particles.

These recommendations are essentially the same as those of Hopkins and Beckham (1998) except that compaction to higher density is recommended here.

4.4 LIMITATIONS OF THIS STUDY AND OPPORTUNITIES FOR FURTHER RESEARCH

This study has largely been based on limited number of small-scale laboratory tests. Data has been provided that shows fills constructed using a larger particle size distribution behave differently from the small-scale tests. Use of small-scale tests to estimate rates of compression with the logarithm of time are unlikely to accurately represent the behaviour of fills constructed from a wider particle size distribution. Future work would be better performed using field scale particle size distributions.

An attempt to use the Unsaturated Soil Mechanics principles to explain the volumetric behaviour of Bringelly Shale upon direct wetting. Key parameters required by the theory have been inferred through correlating prediction and measurements. A thorough suite of unsaturated soil tests should be performed to investigate the use of the theory for fills constructed from Bringelly Shale.

The concept that there is a state of density, stress and moisture where the volumetric behaviour changes from compression to swelling does not appear to exist in the literature. Further study of this phenomenon would contribute to more accurate prediction of volumetric changes.

5 CONCLUSIONS

The results of this study have demonstrated that the internal compression of Bringelly Shale fill can exceed the rule of thumb strain rate of 0.1% per log cycle of embankment height, particularly when fills are high. A better estimate for higher fills can be obtained through large scale, long term, laboratory tests.

Methods for controlling the internal compression of fill constructed from Bringelly Shale have been developed. In-lieu of some directions provided in Transport for New South Wales (TfNSW) TN033 such as classifying expansive soil as unsuitable or use of chemical treatment and blending non-reactive material with expansive soil to use it as general fill, these methods could be adopted in order to allow greater use of Bringelly Shale hence reduce the costs of transport infrastructure construction in Western Sydney.

6 ACKNOWLEDGEMENTS

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