less than 60, plasticity indices in the 15 to 35 range, and moisture contents that are below or near the plastic limit.

## ACKNOWLEDGMENTS

The authors of this paper would like to thank the management of Aquaterra Engineering, LLC for providing the personnel, laboratory, support and encouragement in completing the work for this paper. We would especially wish to recognize the efforts of Mr. Gomer Wallace, senior engineering technician with Aquaterra Engineering, LLC, who performed, compiled and resolved the bulk of the laboratory testing for this study.

## REFERENCES

Chen, F.H. (1988). *Foundations on Expansive Soils*, American Elsevier Scientific Publications, New York.

Department of the Army (1983). "Foundations in Expansive Soils," TM 5-818-7: 1-5.

Krebs, Robert D. and Walker, Richard D. (1971). *Highway Materials*, McGraw-Hill Companies, New York, NY: 68-84.

Holtz, R.D. and Kovacs, W.G. (1981). *An Introduction to Geotechnical Engineering*, Prentice-Hall, Englewood Cliffs, NJ: 733.

Redus, J.F. (1962). "Experiences with Expansive Clay in Jackson, Mississippi," *Moisture, Density, Swelling and Swell Pressure Relationships*, Highway Research Board Bulletin No. 313: 40-46.

Rollings, Marian P. and Rollings, Raymond S. (1996). *Geotechnical Materials in Construction*, The McGraw-Hill Companies, New York, NY: 380-384.

Snowden, Jr., J.O. and Priddy, Richard R. (1968). *Loess Investigation in Mississippi*, Mississippi Geological, Economic and Topographical Survey, Bulletin 111: 40-46.

Viessman, Jr., Warren and Lewis, Gary L. (1996). *Introduction to Hydrology*, Fourth Edition, Harper Collins College Publishers, New York, NY: 22.

---

# The Influence of Structure on One-Dimensional Wetting Induced Volume Change of Compacted Soil

Amy. B. Cerato[1], A.M.ASCE, Jumanah. A. Hajjat[2], and Gerald. A. Miller[3], P.E., M.ASCE

[1]Assistant Professor, University of Oklahoma, 202 W. Boyd Street, Room 334, Norman, OK 73019: email: acerato@ou.edu
[2]Graduate Research Assistant (graduated August 2007), University of Oklahoma, 202 W. Boyd Street, Room 334, Norman, OK 73019: email: jumanah.hajjat@ou.edu.
[3]Professor, University of Oklahoma, 202 W. Boyd Street, Room 334, Norman, OK 73019: email: gamiller@ou.edu

**ABSTRACT:** Volume changes due to soil wetting may occur in naturally deposited soils (i.e., unsaturated expansive soils) as well as earthen construction (i.e., compacted fills or embankments). Depending on the stress level, some soils exhibit increase in volume upon wetting (swell), while others may exhibit decrease in volume upon wetting (collapse). This paper focuses on wetting-induced volume changes in compacted soils. Even when soils are compacted to engineering specifications (i.e., minimum density and moisture content ranges), some earthen construction still exhibit problematic behavior under wetting conditions. Not only is this problematic behavior itself a concern, but the laboratory tests used to predict settlement of constructed facilities may not properly model the actual behavior of soil compacted under field conditions. A research study was conducted in the laboratory to investigate the influence of variations in structure on the one-dimensional wetting-induced volume change for three different fine-grained soils. The results of the study suggest that the influence of structure in one-dimensional oedometer tests depends on soil type. Soil with medium to high PI and large clay size fraction appears to be influenced more by differences in structure, whereas soil with low PI and low clay size fraction does not appear to suffer from structure effects in one-dimensional oedometer tests.

## INTRODUCTION

Most compacted clayey soils will experience swelling under low confining stress and collapse under higher confining stresses; the degree of which depends on soil type and state. In general, all soils are susceptible to wetting-induced collapse when inundated with water under sufficient confining pressure. Precipitation, capillary water from foundation soils, and flooding may cause changes in the moisture content of soils and lead to collapse (Lim and Miller 2004). Several factors influence the amount of collapse potential: initial moisture content, initial dry density, soil type, and confining pressure (ie., Basma and Tuncer 1992, and Lim and Miller 2004). These factors, influencing collapse potential in the field, are taken into consideration when 1-D oedometer tests are performed in the laboratory in order to predict settlement behavior of a soil. To achieve relevant settlement information from 1-D oedometer tests, it is important to test "real" specimens from the field. However, compacted soils are generally evaluated in the laboratory prior to field construction and the engineering behavior of compacted fills in the field can be different from the engineering behavior predicted by laboratory tests. Therefore, an additional factor must be taken into consideration in 1-D oedometer test predictions in an attempt to model actual field compacted behavior.

This factor is the soil's 'structure,' which has been shown to influence the behavior of soils when subjected to either static or dynamic loads (Vallejo 1995). Consequently, evaluation of the soil structure and its changes as a result of loading is important in the field and the laboratory to understand the behavior of the soil. The term "soil structure" includes the combined effects of soil fabric and interparticle forces. "Fabric" generally refers to the geometric arrangement of particles whereas interparticle forces include physical and physico-chemical interactions between particles. The soil structure in this case, is associated with specimen preparation methods and is influenced by five factors: compaction method, clod size, clod size distribution at compaction, dry density or void ratio and compaction moisture content.

Research described herein is focused primarily on studying the influence of soil structure on one-dimensional behavior observed during oedometer tests for two natural soils from Oklahoma. The primary objectives of this research were to (1) examine the influence of sample preparation method on soil structure and (2) examine the influence of soil structure on collapse potential. Understanding these influences and how they can potentially impact predictions of fill behavior is important to appreciate long term behavior of earthen structures. Further, the understanding may impact the approach to design and quality control for important earthen structures.

## BACKGROUND

Observations of soil volume change in the field may be different from predictions provided by one-dimensional (1-D) oedometer tests, even when the initial moisture content, initial dry density, soil type, and confining pressure are kept the same (e.g. Miller and Cleomene 2007). Therefore, the soil behavior under loading should be studied with consideration given to differences in structure between laboratory and field compacted specimens. It was found previously that the soil fabric strongly affects the collapsibility of soils (Rizkallah and Keese 1989); however, some soils may not exhibit variations in collapsibility by changing the internal fabric (i.e., silts).

Based on the compaction moisture content, there are two modes of soil fabric (Leroueil et al. 2002). When the soil is compacted at a moisture content lower than the optimum value, the soil fabric is aggregated; consequently, the soil exhibits more of a clod structure. On the other hand, when the soil is compacted at a moisture content higher than the optimum value, the soil fabric is homogenous and exhibits little to no clods (Benson and Daniel 1990, Leroueil et al. 2002). The soil clods compacted in the laboratory are often smaller than the soil clods compacted in the field. So, an important question arises: is it acceptable to predict volume change of compacted fills from standard 1-D oedometer tests?

Prior to the current study, Cleomene (2005) conducted research at the University of Oklahoma to examine differences in the 1-D volume change behavior of laboratory versus field compacted soil. Four soils from Oklahoma were used; three lean clays and one silty sand. Two compaction procedures were used to prepare standard oedometer samples in the laboratory: 1) air dried soil passing a #4 sieve was mixed with water, compacted in a standard Proctor mold, and then a sample was trimmed directly into the 60-mm oedometer ring; and 2) air dried soil passing a #10 sieve was mixed with water and then compacted (tamped) directly into an oedometer ring. The sample preparation methods did affect the behavior for the clayey soils, even though the samples had nominally similar dry density and moisture content. The fine sandy soil behavior was much less affected by preparation methods. Thus, it appeared that differences in the soil structure created during field and laboratory compaction could result in differences in the oedometer test results. Because only four similar soil types were tested, the results were somewhat inconclusive. Thus, the current study was developed to provide additional information on the influence of soil structure for a wider range of soil types..

## METHOD OF INVESTIGATION

### Test Soils

Three natural soils collected from Oklahoma were used in this work, including one lean clay (CL), one fat clay (CH) and one silt (ML). All the test soils were subjected to classification and physical property tests including: Atterberg Limits (ASTM D4318-00), grain size distribution (ASTM D422-00), specific surface area (Lutenegger and Cerato 2002), carbonate content (Dreimanis 1962), cation exchange capacity (Rhoades 1982), and standard Proctor compaction (ASTM D698-00) tests. Properties of the test soils are listed in Table 1.

### Oedometer Sample Preparation

To create oedometer samples with different structure, two different methods of sample preparation were utilized in this study: 1) soil samples compacted directly into 63.5-mm diameter oedometer rings with a nominal maximum clod size of 2 mm, and 2) soil samples trimmed from soil compacted in a 101.6-mm diameter compaction molds with a nominal maximum clod size of 38 mm. The latter, while not the same as field-compacted soil, is more representative of larger clod sizes associated with field compaction. It was expected that if differences were observed in the behavior of soil prepared using these two different methods, then even greater differences may be possible between lab and field tests where clod sizes can be considerably larger than 38 mm (limited to 75 mm by specification but larger chunks are common).

**Table 1: Properties of the Test Soils**

| Soil # | Soil Name | AASHTO | USCS | LL (%) | PI (%) | Fines (%) | Clay Fraction (%) | $\gamma_{dmax}$ (kN/m$^3$) | OMC (%) |
|---|---|---|---|---|---|---|---|---|---|
| 3 | Burford-Vernon Complex | A-6 (15) | CL | 36 | 17 | 91 | 36 | 17 | 17.3 |
| 7 | Devol Soil | A-4 (0) | ML | ---- | NP | 80 | 7 | 17.6 | 14.7 |
| 10 | Hollywood Soil | A-7-6 (45) | CH | 65 | 43 | 93 | 61 | 15.2 | 24 |

Note: AASHTO: American Association of State Highway and Transportation Officials; USCS: Unified Soil Classification System; LL: Liquid Limit; PI: plastic index; OMC: optimum moisture content; CL: Inorganic clays of low to medium plasticity lean clays; CH: Inorganic high plasticity, fat clays; ML: inorganic silts; and SC: inorganic clayey sand.

**Table 1. (cont.) Properties of the Test Soils**

| Soil # | Total SSA (m$^2$/g) | Ext. SSA (m$^2$/g) | Int. SSA (m$^2$/g) | CEC (meq/100g) | Activity (PI/CF) | Carbonate Content (%) | Dolomite Content (%) | Calcite Content (%) |
|---|---|---|---|---|---|---|---|---|
| 3 | 92 | 29 | 63 | 23.6 | 0.47 | 3.9 | 0.5 | 3.5 |
| 7 | 54 | 10 | 44 | 16.5 | --- | 5.2 | 1.6 | 3.6 |
| 10 | 220 | 44 | 176 | 50.0 | 0.7 | 4 | 0.9 | 3.1 |

Note: SSA: Specific surface area, CEC: Cation Exchange capacity

### Oedometer Testing Procedures

All soil samples were subjected to single- and double-oedometer tests, although only the single-oedometer (collapse) method results will be presented herein due to page limitations. The single-oedometer method was performed in general accordance with American Society for Testing and Materials (ASTM) D 5333 "Standard test method for measurement of collapse potential of soils." It consists of incrementally loading a test specimen at its 'as-compacted' water content, up to a target stress level followed by inundation with water. Each loading increment is maintained for one hour or until no further significant deformation occurs. The sample is then inundated with water to induce collapse at a vertical stress of approximately 200 kPa. The collapse potential is determined when there is no more noticeable deformation in the specimen, which generally occurs overnight. The collapse index, $I_e$, is the collapse potential, or vertical strain change due to wetting the test specimen at an applied vertical stress of 200 kPa. Loads continue to be added after 200 kPa to determine the compression index and compare the results from the ring-compacted (RC) and Proctor-trimmed (PT) samples.

## RESULTS

Soil #3 collapse test results are shown in Figure 1. Eight ring-compacted (RC) and two Proctor-trimmed (PT) tests were performed to study the repeatability of the procedure as well as to lend strength to the comparison of RC and PT structure influences on volume change behavior. As can be seen, there is some variability in collapsibility within RC tests performed with an $I_e$ range from 0.5-2.9%, even though every effort was made to ensure the water content and dry density were initially the same. The differences in water content seen between the tests could be the cause in the $I_e$ variability, thus only four of the eight tests were used to determine the average curve to compare with the PT tests, although the average curve using four tests was very similar to the overall average of eight tests. There were only two PT tests performed on Soil #3, and the average curve and range is shown along with the average curve and range for the RC tests.

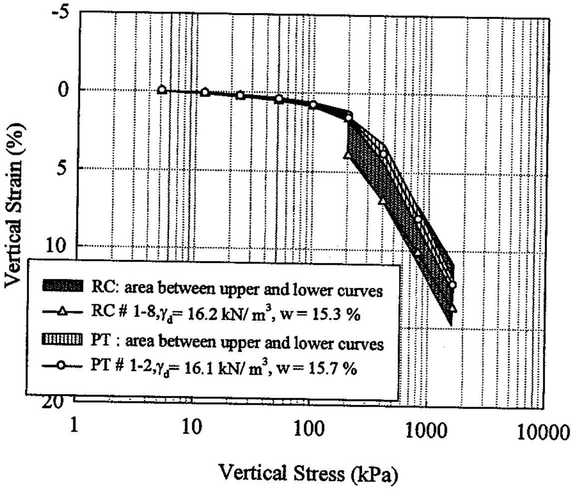

**Figure 1. Soil # 3 - Comparison of test results from RC and PT specimens.**

For Soil # 7, one RC and PT test were performed (Figure 2). There is no difference in collapse index between the RC and PT samples. Figure 3 shows the collapse test curves for Soil #10 (CH). The RC test showed a large collapse of 2.7% where the PT test showed swelling (-1.4%). This result was not expected, but can be somewhat explained by the slow movement of water inside the large clods in PT specimens causing a simultaneous slow loss of matric suction. This caused the soil particles inside the large clods in the PT specimen to swell and then occupy the large pores (inter-aggregate). Therefore, the overall behavior of the specimen was an expansion (swelling). It appears that the differences in soil structure of the two cases (RC vs. PT) influences behavior in clayey soils.

Comparison between average collapse indices for RC and PT specimens of all three soils are presented in Figure 4. As can be seen, there is a significant structure influence on collapse behavior in clayey soil, but not in silty soil, which exhibited very little collapse behavior. The lack of collapse and structure effect in the silty soil is most likely due to clod breakdown under both compaction methods that resulted in a similar structure. In both clayey samples, the RC specimens exhibited larger collapse than the PT specimens.

For the same moisture content and dry unit weight, the structure of clayey specimens compacted in a proctor compaction mold is different from the structure compacted directly in small rings. The differences in structure between the two cases are distinguished by clod size, pore size distribution and inter-particle forces. Because of the different clod sizes used in both approaches, different pore size distributions were formed. The soil clod consists of soil particles grouped in clusters and separated by individual or small pores (intra-aggregate). The number of the small pores in one clod in the PT specimens may be much higher than that in the RC specimens. Based on the pore size distribution in a compacted soil, the large pores (inter-aggregate) change in size and shape, whereas there are no significant changes in the size and shape of the smaller pores (intra-aggregate) (Vallejo 1995). Consequently, the clods in the PT specimens, where the small pores (intra-aggregate) remain unchanged, may deform less than those of the RC specimens (Cleomene 2005).

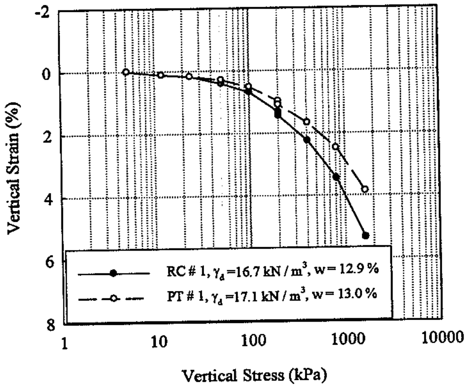

**Figure 2. Soil # 7 - Comparison of RC and PT specimens.**

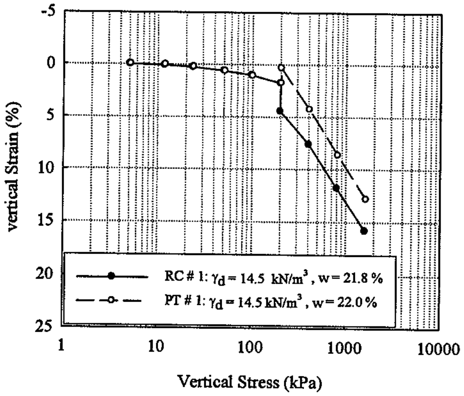

**Figure 3. Soil # 10 - Comparison of RC and PT specimens.**

In addition, it appears that the compactive effort used may affect the wetting-induced collapse behavior; the compaction effort used in the proctor-trimmed specimens was higher than that of the ring-compacted specimens. For the proctor-trimmed specimens, higher compaction effort may cause a change in the stress history of the soil particles. When a soil is compacted with a moisture content on the dry side of the OMC, it possess a flocculent structure where the diffuse double layer of ions surrounding the clay particles cannot be fully developed and interparticle repulsion forces are reduced. This results in a more random orientation of clay particles. At the same moisture content, higher compaction effort tends to give a more parallel orientation to the clay particles; consequently, the soil structure will be more dispersed. Therefore, the proctor-trimmed specimens exhibited smaller collapse indices compared to those of the ring specimens.

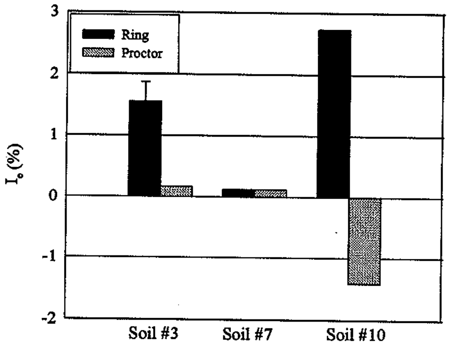

**Figure 4. Comparison of Collapse from RC and PT Specimens.**

## CONCLUSIONS

Based on the comparison of results from RC and PT oedometer tests for three fine-grained soils, the following conclusions can be drawn:

1. Structure effects can be significant for some cohesive soils compacted on the dry side of the OMC.
2. Collapse potential was affected by clod size and initial specimen preparation method in clayey soils. Consequently, it is important to establish corrections between collapse indices in the laboratory and those in the field to predict real wetting–induced compression of embankments and compacted fills.
3. Cohesionless and low cohesive soils (non-plastic and low plastic) were not collapse susceptible and were not affected by sample preparation, and compaction method.

## REFERENCES

Basma. A. A. and Tuncer. E. R. (1992) "Evaluation and control of collapsible soils," Journal of Geotechnical Engineering, Vol. 118, No.10, pp. 1491-1504.

Benson, C. H., and Daniel, D. E., (1990). "Influence of clods on hydraulic conductivity of compacted clay." Journal of geotechnical Engineering, ASCE, Vol. 116, No. 8, pp. 1231-1248.

Cleomene, E. (2005). "Scale and fabric effect on one-dimensional compression behavior of compacted soils". Master Thesis, University of Oklahoma., Norman.

Cerato, A.B. and Lutenegger, A.J. (2002), "Determination of Surface Area of Fine-Grained Soils by the Ethylene Glycol Monoethyl Ether (EGME) Method," ASTM, Geotechnical Testing Journal (GTJ), Vol. 25, No. 3, pp. 315-321.

Dreimanis, A. (1962). Quantitative Gasometric Determination of Calcite and Dolomite by Using Chittick Apparatus. Journal of Sedimentary Petrology. Vol. 32. No. 3. pp. 520-529.

Leroueil, S. (1988). "Tenth Canadian geotechnical colloquium: Recent development in consolidation of natural clays". Canadian Geotechnical Journal, Vol. 25, No. 1-2, pp. 85-107.

Lim, Y.Y.and Miller. G.A. (2004). "Wetting-induced compression of compacted Oklahoma soils", Journal of Geotechnical and Geoenviromental Engineering, Vol. 130, No.10, pp.1014-1023.

Miller, G. A. and Cleomene, E. (2007). "Influence of fabric and scale effect on wetting-induced compression behavior of compacted soils." GeoDenver: New Peaks in Geotechnics. GSP 162: Denver, CO, Feb. 18-21, 2007, pp. 1-10.

Rhoades, J.D. (1982). Cation Exchange Capacity. In AL. Page (ed.) Methods of Soil Analysis, Part 2, Second Edition. Agronomy Monograph 9, American Society of Agronomy, Madison, WI.

Rizallah, V. and Keese, K. (1989). "Geotechnical properties of collapsible soils." Proceedings of the 12th International conference on Soil Mechanics and Foundation Engineering, August, 13-19, Rio de Janeiro, Brazil. pp. 101-104.

Vallejo, L. E. (1995). "Fractal analysis of the fabric changes in a consolidating clay." Engineering Geology, Vol. 43, No. 4, pp. 281-290.

---

# Performance of Expanded Clay Shale (ECS) as an Embankment Backfill

[1] Sireesh Saride, [2]Sunil K. Sirigiripet, [3]Anand J. Puppala, [4]Richard Williammee

[1] Post Doctoral Fellow, Department of Civil and Environmental Engineering, The University of Texas at Arlington, Texas 76019, sireesh@uta.edu

[2]Graduate Research Assistant, Department of Civil and Environmental Engineering, The University of Texas at Arlington, Texas 76019, sunil110@gmal.com

[3]Professor, Department of Civil and Environmental Engineering, The University of Texas at Arlington, Texas 76019, anand@uta.edu

[4]Materials Engineer, Texas Department of Transportation, Fort Worth, Texas 76137, rwillia@ @dot.state.tx.us

**ABSTRACT:** Light weight aggregates (LWA) have been extensively used throughout North America for more than 80 years in cast-in-place structural lightweight concretes for high-rise buildings and bridges. They are now being used for several geotechnical applications including as a backfill in the embankment laid over soft soils, or as a fill material in the retaining wall structures. A research study currently focuses on the use of LWA derived from the manufacturing of Expanded Clay Shale (ECS) as an embankment fill material along State Highway (SH) 360 in Arlington, Texas. This paper first presents the laboratory results obtained by testing Expanded Clay Shale (ECS) material to evaluate its function as embankment backfill. Both direct shear and consolidation tests were performed to evaluate its strength characteristics. This embankment was instrumented with vertical inclinometers to monitor fill movements. Based on the monitored data, it can be mentioned that the use of ECS resulted in less settlements of embankment at the site.

## INTRODUCTION AND BACKGROUND

For nearly a century, expanded clay shale aggregates (ECS) have been used successfully around the world for various applications (Expanded Shale, Clay & Slate Institute (ESCS), 2004). ECS is a light weight aggregate prepared by expanding select minerals in a rotary kiln at temperature of over 1000°C (Holm and Ooi, 2003). The production and raw material selection processes are strictly controlled to insure uniform, high-quality product that is structurally strong, stable, durable and inert, yet also lightweight and insulative (ESCS, 2004).

Millions of tons of ECS produced annually are used in many geotechnical applications. Its availability is currently widespread throughout most industrially developed countries. Consideration of ECS as a remedy to geotechnical problems stems primarily from the