Compression Behavior of Synthetic Lightweight Aggregates

Assem Elsayed[1] and Christopher Swan[1]

[1] Department of Civil and Environmental Engineering, Tufts University, Medford, MA 02155

KEY WORDS: synthetic aggregates, compression, waste plastics, high carbon flyash

ABSTRACT

This paper presents the compression properties of a synthetic lightweight aggregate (SLAs) made of fly ash and mixed plastics. SLAs are manufactured through a thermal-mechanical process that, under heat and pressure, mixes and extrudes the plastic and fly ash into a solid die that is granulated to create various size particles resembling natural sand and gravel. In this paper compression properties are presented for SLAs made with a mixture of plastics and high carbon coal fly ash, two waste streams with little to no value. Aggregate fly ash-to-plastic ratio, by weight, was 80:20 with fly ash carbon content ranging from 15% to 30%. A series of one-dimensional compression tests, at moderate and elevated stresses, were performed on SLAs with their behavior compared with traditional aggregates of normal-weight sand and expanded clay/shale lightweight aggregate. All tests were conducted on specimens with similar initial grain size distributions. Results show that SLAs have relatively substantial elastic deformation compared to the traditional aggregates but also exhibit substantial rebound upon unloading. However, at elevated stresses, the rate of secondary compression for both the SLAs and sand approach similar values, with the SLAs actually exhibiting a slowing rate (i.e., stiffening) of secondary compression. Overall, the compression results show that SLAs will have advantages (e.g., lightweight, stiffening with age) and disadvantages (e.g., large elastic deformation) over the use of traditional granular materials.

1.0 Introduction

Since ocean dumping of municipal solid waste (MSW) was banned in the United States in 1933, virtually all non-recycled waste has been placed in landfills (Kreith, 1994). With available landfills space becoming scarcer, it is essential to find new ways to minimize waste generation and one way to do that is by recycling. Recycling has become an established national industry that is expanding annually. By 1998, almost all urban areas in the United States, approximately 81% of the population, had convenient access to a plastic recycling collection program.[17] The world's annual consumption of plastic materials has increased from around 5 million tons in the 1950s to nearly 100 million tons in 2002.[18]

Recycling is the process of converting waste products into a useful product or materials. It differs from reuse, which simply means using a product or material again. According to the Environmental Protection Agency, about 13 percent of the nation's solid waste (that is, the waste which is normally handled through garbage collection systems) is recycled. Recycling is appealing because it seems to offer a way to simultaneously reduce the amount of waste disposed in landfills and to save natural resources. During the late eighties, as environmental concerns grew, public opinion focused on recycling as a key way to protect the environment. Flyash and waste plastics are typically considered to be waste materials.

1.1 Flyash

Flyash is fine-grained particles consisting of solid spheres and hollow cenospheres and fibrous carbon. The major constituents of flyash are heterogeneous glassy and crystalline silica, alumina, iron, and calcium. The minor constituents of flyash are magnesium, sulfur, sodium, potassium, and carbon.[14] The structures, compositions, and properties of flyash depend upon the structure and composition of the coal and the combustion processes by which the flyash is formed. Particle sizes and physical properties in flyash are extremely variable and dependent on the source of coal. The particle sizes vary from less than 1μm (micron=10^{-6}m) to more than 100 μm with a typical particle size less than 20μm. All flyash particles used in this research study are smaller than 75μm (minus #200 sieve).

Flyash is predominantly generated from coal-powered electric utilities. Almost all flyash is captured by dust collecting systems, such as electrostatic precipitators or bag houses. In the United States, 98.2 million metric tons of CCBs are produced each year with flyash representing 57 million metric tons (58%) of the CCBs produced. Yet, only 34% of this flyash is reused.[2] The remaining material is most often disposed of in landfills. The annual cost of CCBs disposal is approximately \$1.0 billion; this figure will most likely increase as landfill space becomes restricted.

1.2 Recycled Plastics

"Plastics" is the general term used to describe materials composed of carbon-chain macromolecules similar to cellulose. These macromolecules (polymers) are made from building blocks (monomers). Plastics are engineered through specific monomer combination and concentration for a variety of end-uses from tough resilient composite car bodies to flexible plastic wrap. Over 41 million metric tons of thermoplastics are produced in 2006 in the US and Canada.[1] Six plastics account for over 70 percent of all plastic sales. These are low-density polyethylene (#4 LDPE - 17 percent), polyvinyl chloride (#3 PVC - 15 percent), high-density polyethylene (#2 HDPE - 14 percent), polypropylene (#5 PP - 13 percent), polystyrene (#6 PS - 9 percent), and polyethylene terephthalate (#1 PETE - 4 percent).

The plastics that are most commonly recycled are PETE soda bottles and other containers, HDPE milk jugs, and other HDPE containers, which account for 3, 3, and 5 percent, respectively, of all plastics that are discarded (0.2 percent, 0.3 percent, and 0.5 percent by weight of all municipal solid waste). Recycled plastics are typically chipped, washed, and heated to produce pellets or flakes that can be manufactured into secondary products. Some mixed plastic items can be separated by weight into their component resins, but in general, plastics must be separated before they are input into a secondary manufacturing process and these previous costs may limit the reuse of the recyclable material.[7] Mixed plastics can be recycled for relatively non-demanding applications such as " plastic lumber" for decks, traffic stops and park benches.

1.3 Previous Studies on Synthetic Lightweight Aggregates, SLAs

Synthetic lightweight aggregates (SLAs) are created by combining fly ash and plastics though a melt-blending process. The compounding can be done using a variety of traditional, plastics compounding equipment, such as a co-rotating intermeshing twin screw extruder, combining the ash and plastics under heat and pressure to produce an extrudate that is allowed to cool and solidify. The solid extrudate is finally cut into uniformly-sized particles using a rotating knife granulator. Figure 1.1 shows a schematic of the manufacturing process.

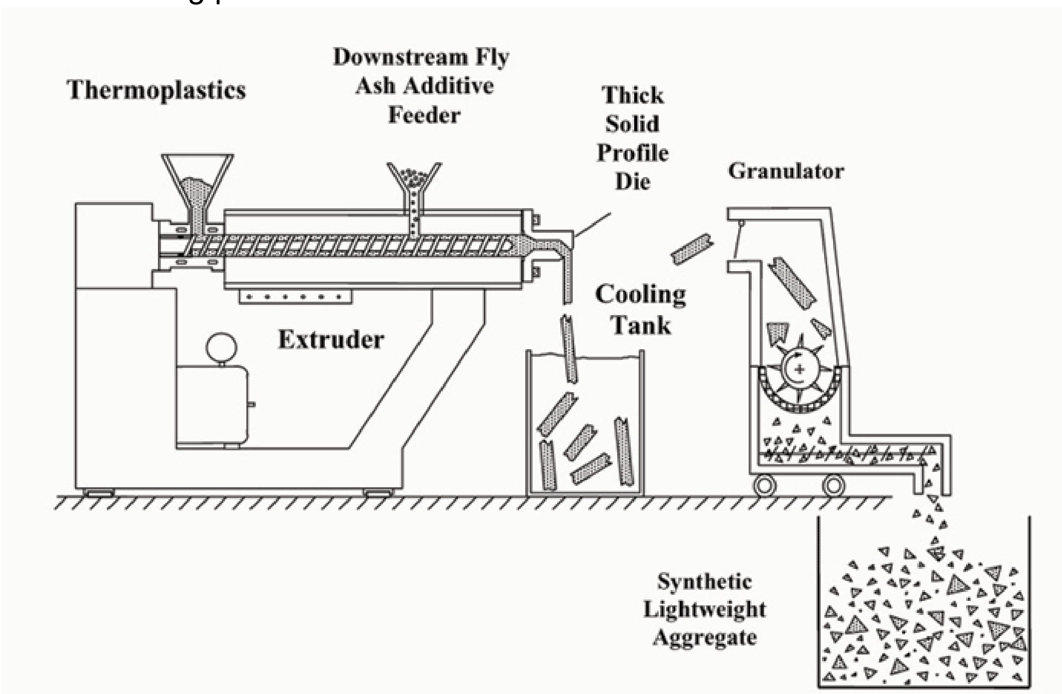

Figure 1.1: Schematic of SLA Manufacturing Process[13]

Researchers have been developing and evaluating synthetic lightweight aggregates developed from commingling coal flyash with waste plastics. Low-carbon coal flyash commingled with HDPE were the first generation of SLAs.[10, 12] Subsequent work by Cook (2000), Gaudreau (2002) and Swan and Sacks (2005) have continued to evaluated the geotechnical properties of various SLAs made from various flyash and plastic mixtures.[5, 8, 21]

The most relevant of these efforts was presented by Swan and Sacks who detailed a study of 15 one-dimensional compression tests on various SLAs that contained PS, LDPE, HDPE, and MP and on a granitic sand. All the materials passed Sieve# 8 (2.36mm) and were retained on Sieve #16 (1.18 mm). The specimens' diameter and height were 63 mm (2.5 inch) and 25mm (1 inch), respectively. The loading program consisted of applying load from 0 to 1240 kPa (180 psi), then 1240 kPa to 12380 kPa (1800 psi) maintaining each stress for 24 hours. The study attempted to have each material have same initial porosity (n_o) of 0.60 ± 0.03.

The paper concluded that 1) SLAs underwent a much greater reduction in volume than natural sand under the same applied stress; 2) SLAs absorb stress through plastic particle deformation over the entire range of applied stresses, whereas natural sand absorbs low stresses through elastic skeleton deformation and particle rearrangement and at high stresses through crushing at particle contact points; 3) because of particle interlock and bonding, SLA particles rebound minimally after undergoing high stresses; and 4) SLAs exhibit significant amounts of creep under both low and high-sustained stresses while the sand exhibits significant creep only under high-sustained stresses.

1.4 Objectives and Scope of the Research Effort

Similar to the effort of Swan and Sacks, the objective of the study presented in this paper was to evaluate the 1-D compression behavior of various synthetic lightweight aggregates and compare the measured behavior to that of normal weight and traditional expanded clay/shale lightweight aggregates. However, the effort was expanded to include eight "aggregates": four SLAs comprised of 80% high-carbon flyash (LOI > 10%) and 20% different polymers (HDPE, LDPE, PS, and Mixed Plastics, a combination of PETE, HDPE, LDPE, PP, and PS); pure PS; pure flyash; a traditional lightweight expanded clay/shale aggregate (EC); and a sand obtained from Cape Cod , MA (CCS).

2.0 Materials and Procedures

2.1 Materials

The SLAs manufacturing process has been previously described. The four SLA mixtures are designated as follows: SLA_{HDPE} for HDPE based SLA, SLA_{LDPE} for LDPE-based SLA, SLA_{PS} for PS-based SLA, and SLA_{MP} for mixed plastics-based SLA. The pure PS was developed in a similar manner as the SLAs except no ash was added during the manufacturing process. The pure fly ash was a high-carbon (LOI > 10%) fly ash from the same source as used in producing the various SLAs.

Two granular soils, expanded clay/shale (EC) and Cape Cod Sand (CCS) are employed in this research study to be tested as control materials. The EC is produced by rapidly heating clays to a semi plastic condition known as "the point of incipient vitrification" in rotary kilns or sintering hearths. Under this condition the material may expand up to 7 times of its original volume. The density of the end product is a function of the virgin material, with some clays and shales having higher residual water contents increasing their potential for expansion. The natural beach sand (CCS) used in the study was obtained from Cap Cod, Massachusetts. CCS consists primarily of angular to sub angular quartz with some feldspar.

2.2 Procedures

A series of consolidation tests were performed on different samples of the presented aggregates; a total of 27 tests were attempted. Two different techniques were employed for the consolidation test, adopted from ASTM D2435-90 "Test Method for One-Dimensional Consolidation Properties of Soils" and for this research study this technique will be called "stress control", and ASTM D4186-89 "Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading" and for this research study this technique will be called "constant rate of strain". The following sections describe the general set of procedures used for all tests.

2.2.1 Sample Preparation

All the specimens were tested in a stainless steel compression ring having an inside diameter of 63 mm; (2.5 inch) and a height of 25 mm; (1 inch). A film of vacuum grease was applied to the interior surface of the ring prior to each test. Test specimens were developed using an air pluviation process where the granular material is “rained” through a series of screens and comes to rest inside the ring. Therefore, the samples were poured into the assembled consolidation cell with dry filter paper placed below and above the sample. A few hand taps to the cell were necessary to make the top surface of the sample level and until the loading cap reached a level indicating a specimen height of 25 mm.

All the samples have the same initial relative density before testing rather than if they all have the same initial void ratio. If the void ratio were used the sand and any SLA would exist in different states of packing. Therefore, an initial relative density of 70% was utilized for all the presented materials. The description of a soil deposit when the relative density is 70% is medium dense[11], which is suitable enough for the most practical application of these materials as a granular soil. The minimum and maximum void ratio (e_{min} and e_{max}, respectively) for each materials were obtained in order to set the desired initial relative density of all the samples to 70%.

2.2.2 Constant Rate of Strain Tests

A total of 24 tests were conducted using constant rates of strains of 0.2, 1, and 5% per min strain rate. All test samples had an initial relative density of 70%. The main goal was to study the behavior of each SLA compared to natural sand and to natural lightweight aggregate. The associated strain rate effects would be investigated as well.

2.2.3 Incremental Constant Stress Tests

A total of 3 tests were performed using a stress control technique where a stress of 700 kPa was maintained on the sample for 10 days. The initial relative density for all of these tests was 70%. The stress level of 700 kPa was chosen because this is the stress level that a loaded truck axel of 36 tons exerts on a highway embankment. The main objectives of the incremental tests were to study the secondary compression behavior over both a short and a long period of time, and to compare the results of the two different techniques, stress and strain control.

3.0 Test Results

3.1 Constant Rate of Strain Tests

A series of one–dimensional compression constant rate of strain tests was conducted on the various SLAs as well as CCS and EC. Tests were performed at three strain rates of 5, 1, and 0.2 % per min. For the purpose of comparison, CCS is serving as a reference point. Figures 3.1 and 3.2 show the strain rate effects on CCS at the three strain rates. All the samples started at the same relative density of 70% (initial void ratio was 0.617). The compression of SLAs, pure PS plastic, and pure flyash at the strain rates of 5, 1 and 0.2 % per min are shown in Figures 3.3, 3.4 and 3.5, respectively. These figures also show the compression of CCS and EC for comparison.

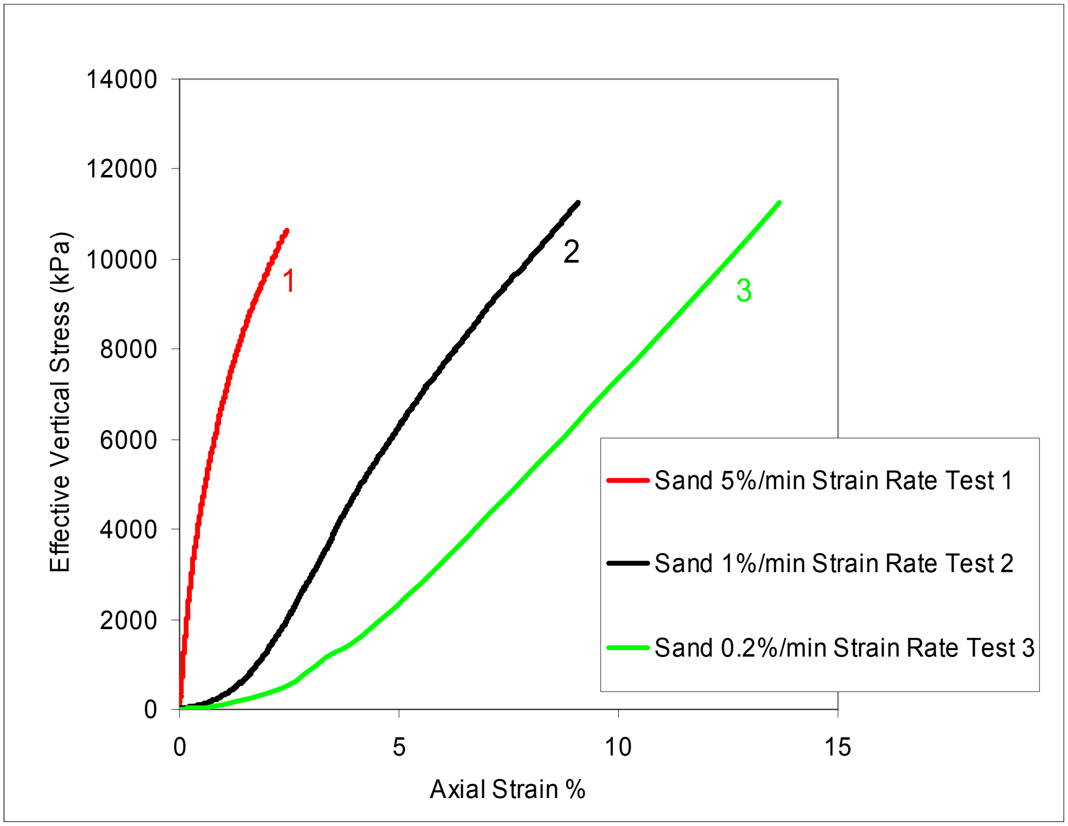

Figure 3.1: Effective vertical stress vs. axial strain for sand (CCS) at 5, 1, and 0.2 % per min rate of loading.

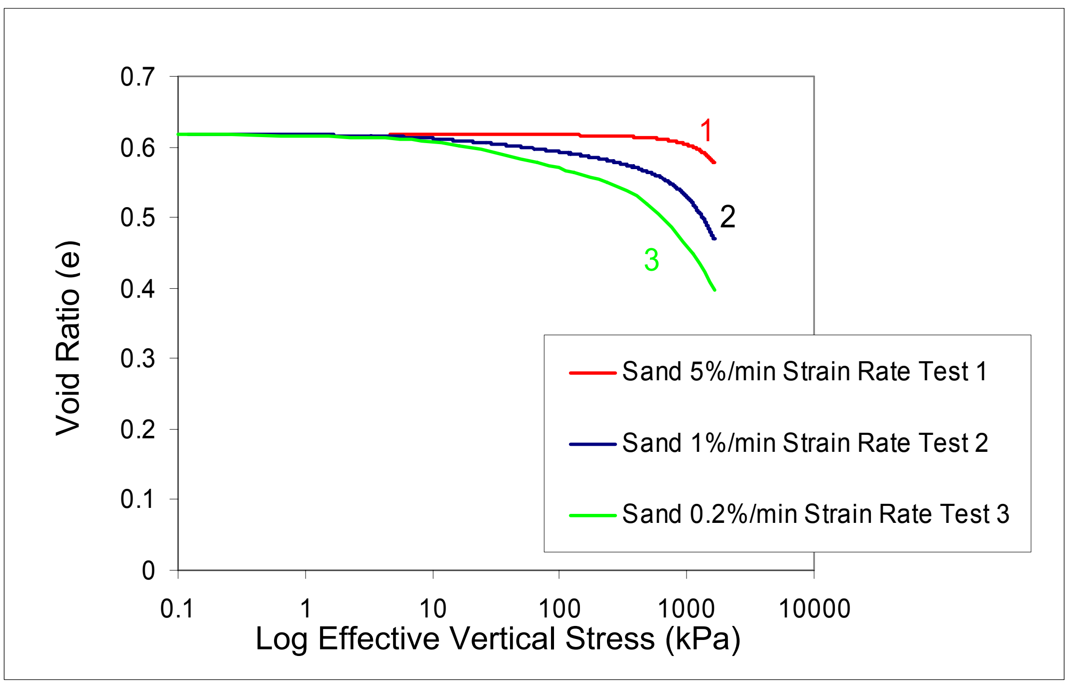

Figure 3.2: Void ratio vs. log effective vertical stress of sand (CCS) at 5, 1, and 0.2 % per min rate of loading.

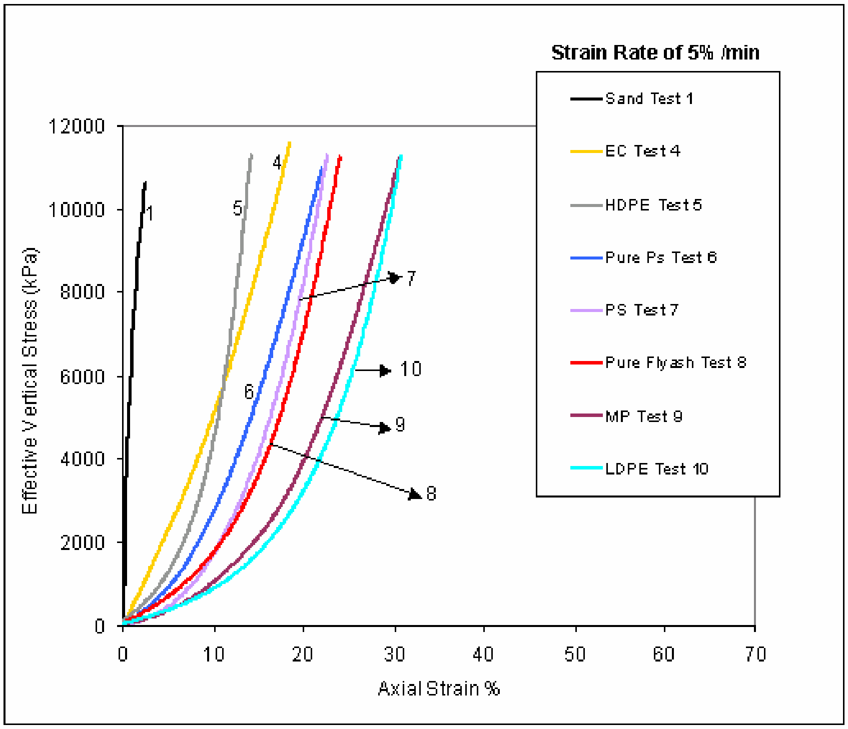

Figure 3.3: Effective Vertical Stress vs. Axial Strain for Sand (CSS), EC, Various SLAs, Pure PS, and Flyash at a Strain Rate of 5 % per min

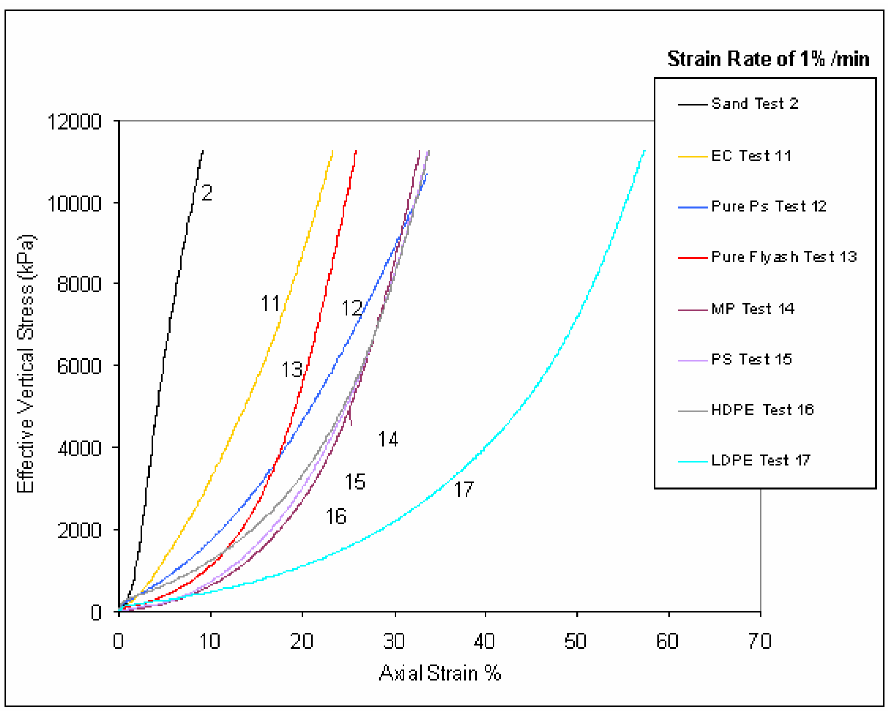

Figure 3.4: Effective Vertical Stress vs. Axial Strain for Sand (CCS), EC, Various SLAs, Pure PS, and Flyash at a Strain Rate of 1 % per min

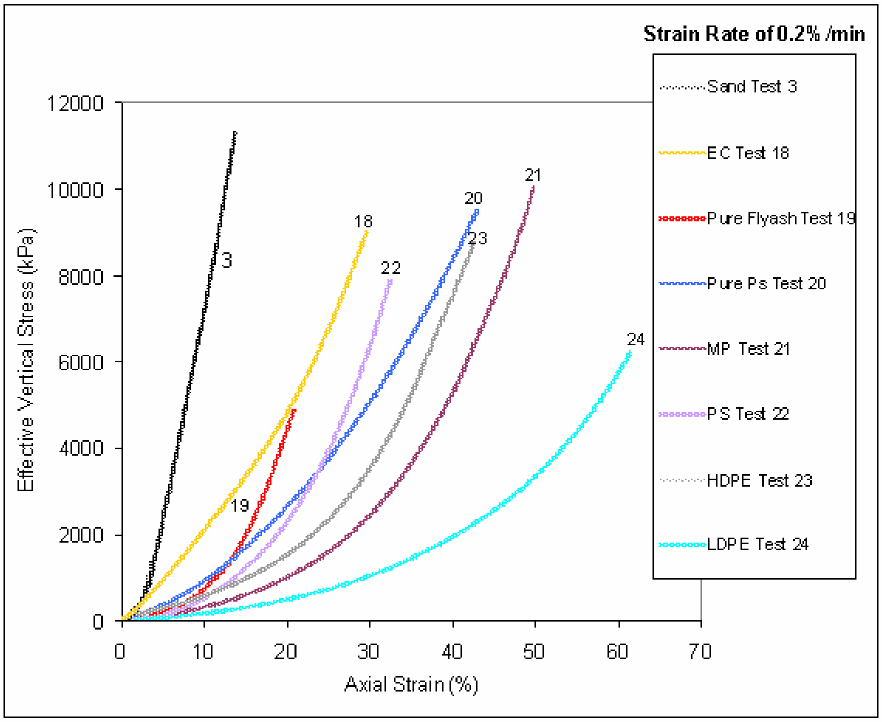

Figure 3.5: Effective Vertical Stress vs. Axial Strain for Sand (CCS), EC, Various SLAs, Pure PS, and Flyash at Strain Rate of 0.2 % per min.

3.1.1 Analysis and Discussion

It is apparent that as specimens are tested at higher strain rates they exhibit a stiffer response. For example, from Figure 3.1, at the same vertical stress level, the sand samples exhibited less deformation at higher strain rates. Also, from Figure 3.2 at higher strain rate the sand samples yielded less response in terms of change in void ratios. This effect of loading rate has been experimentally studied by a number of researchers. [4, 6, 16, 19, 22] Figures 3.3, 3.4, and 3.5 show that each material has a unique response for different strain rates. The following observations can be noted based on these results:.

1) Sand is the stiffest among all the presented aggregates showing the stiffest response for each strain rate.
2) The sand samples exhibited small strains to large stresses compared to the other aggregates. The shape of the curves for the sand samples at 5 and 1% per min strain rate was concave downward (Figure 3.1). Thus the tangent constrained modulus is decreasing with increasing stress level. At 0.2% per min strain rate, the sand sample exhibited slight increase in the tangent modulus of elasticity until it came to a constant value “residual” after stress level of 1.0 MPa.
3) The responses of the other aggregates (EC, SLA_{HDPE}, SLA_{LDPE}, SLA_{PS}, SLA_{MP}, Flyash, and Pure PS) indicate similar behavior; i.e., the higher the strain rate the stiffer the response except that these aggregates exhibited large strains to large stresses.

4) The shape of the stress-strain curves for SLAs samples, their components and EC, Figures 3.3 to 3.5, at 5, 1,and 0.2% per min strain rate were concave upward. Thus, the tangent constrained modulus of elasticity is increasing with increasing stress level (i.e. gain in stiffness).
5) The materials have unique transition zones from elastic to plastic state, which depended upon the type of aggregates. At 0.2% per min strain rate, for the pure PS and SLA_{LDPE}, the volume of voids had reduced to zero (theoretically) at vertical stresses of 9 and 6 MPa, respectively. [Thus, these tests were stopped.] At this condition the aggregates have compressed and re-shaped themselves to displace all the void space and became monolithic, a condition confirmed upon visual observation during test dismantling. Other SLAs also transformed into monolithic samples at the end of the test at 11.0 MPa (though they did not meet the theoretical zero void space condition). In contrast, Sand, EC and Pure flyash samples were dismantled easily and they did not stick together.

3.2 Secondary Compression of SLAs via Stress Control Tests

Using incremental loading procedures, samples of SLA_{PS} pure PS, and Cape Cod Sand (as a control) were loaded to a stress of 700 kPa maintained for 10 days. Figure 3.6 shows the void ratio vs. log of time for these tests. Figure 3.7 shows the normalized void ratio vs. log of time and Figure 3.8 shows the normalized void ratio vs. square root of time. Also, Figure 3.9 depicts the calculated strain rate vs. log of time for the same aforementioned samples.

3.2.1 Analysis and Discussion

SLA_{PS}, CCS, and pure PS have different void ratios at the beginning of the test because of fixing the initial relative density to 70 % for all the samples. Therefore, Figure 3.6 does not present a clear trend of the creep behavior due to scale limitation. Thus, Figure 3.7 presentation of a normalization of the void ratio as a function of time provides a clearer indication that each material has a different behavior under sustained stress of 700 kPa.

Gibson and Lo (1961) identified three types of secondary compression curves, which are common in soils exhibiting secondary compression.[9] The three types are shown in Figure 3.10. These three types are summarized as:

1) Type 1 curve shows a gradual decrease in the rate of secondary compression until ultimate settlement is finally reached.
2) Type 2 curve, which is characterized by the proportionality of secondary compression with the log time for an appreciable time period before the final settlement is reached.
3) Type 3 curve shows an acceleration of the rate of secondary compression. This acceleration can be either gradual (curve 3 b) or abrupt (curve 3 a), and is believed to be due to the breakage of bonds between particles.

Applying this classification to Figure 3.7 suggests that, SLA_{PS} and pure PS followed Type 2 behavior where the CCS follows a Type 3b behavior.

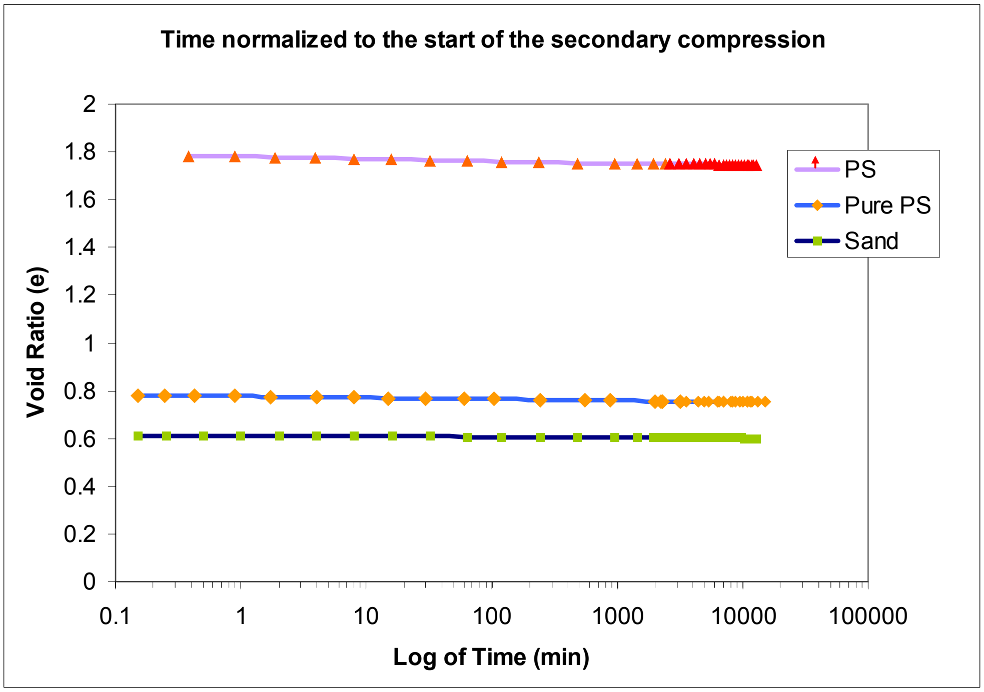

Figure 3.6: Void Ratio vs. Log Time for SLA$_{PS}$, Pure PS, and CCS at 700 kPa.

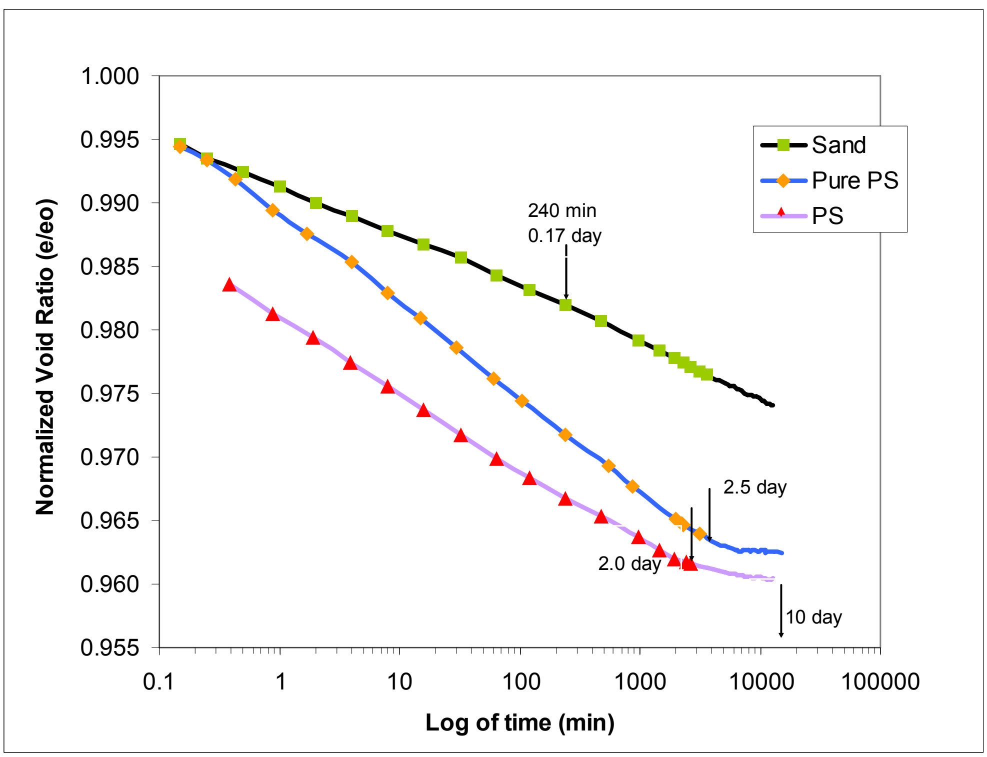

Figure 3.7: Normalized Void Ratio vs. Log Time for CCS, Pure PS, and SLA$_{PS}$ at 700 kPa

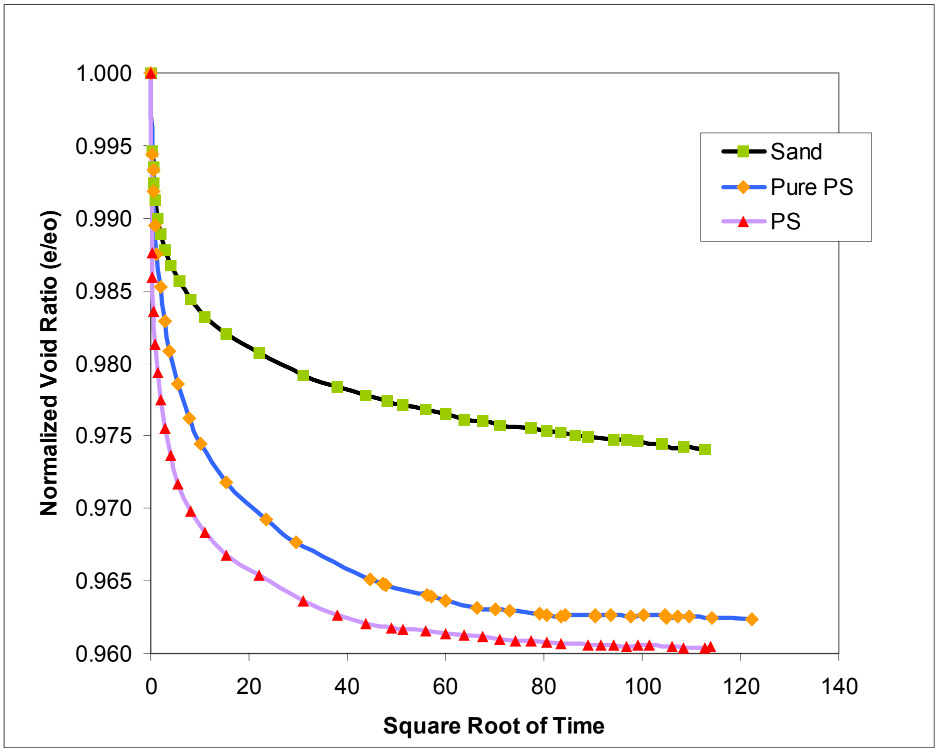

Figure 3.8: Normalized Void Ratio vs. Square Root of Time for CCS, Pure PS, and SLA$_{PS}$ at 700 kPa.

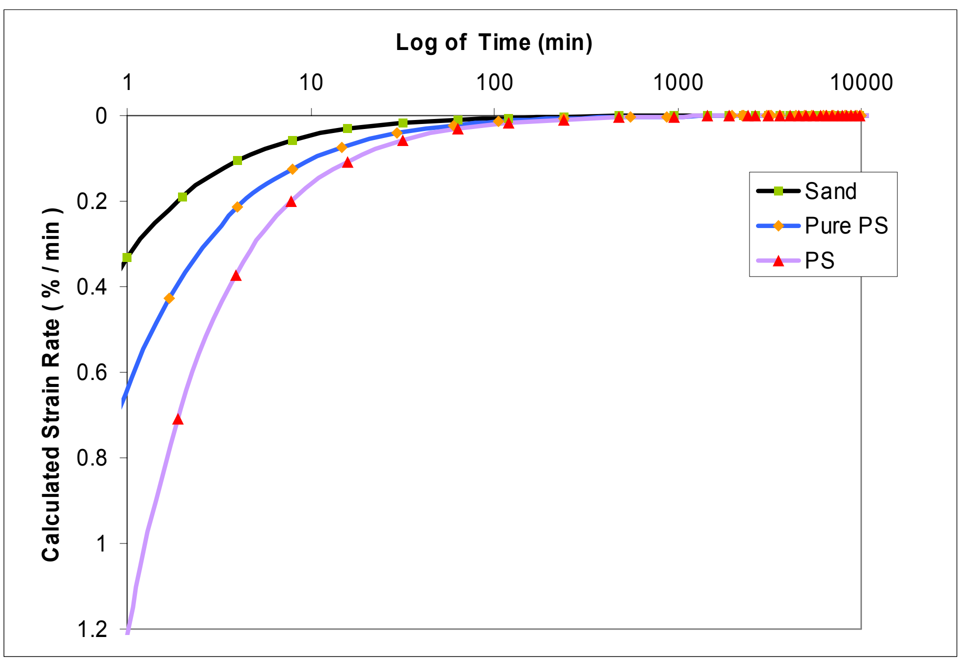

Figure 3.9: Strain Rate vs. Log Time for CCS, Pure PS, and SLA$_{PS}$ at 700 kPa.

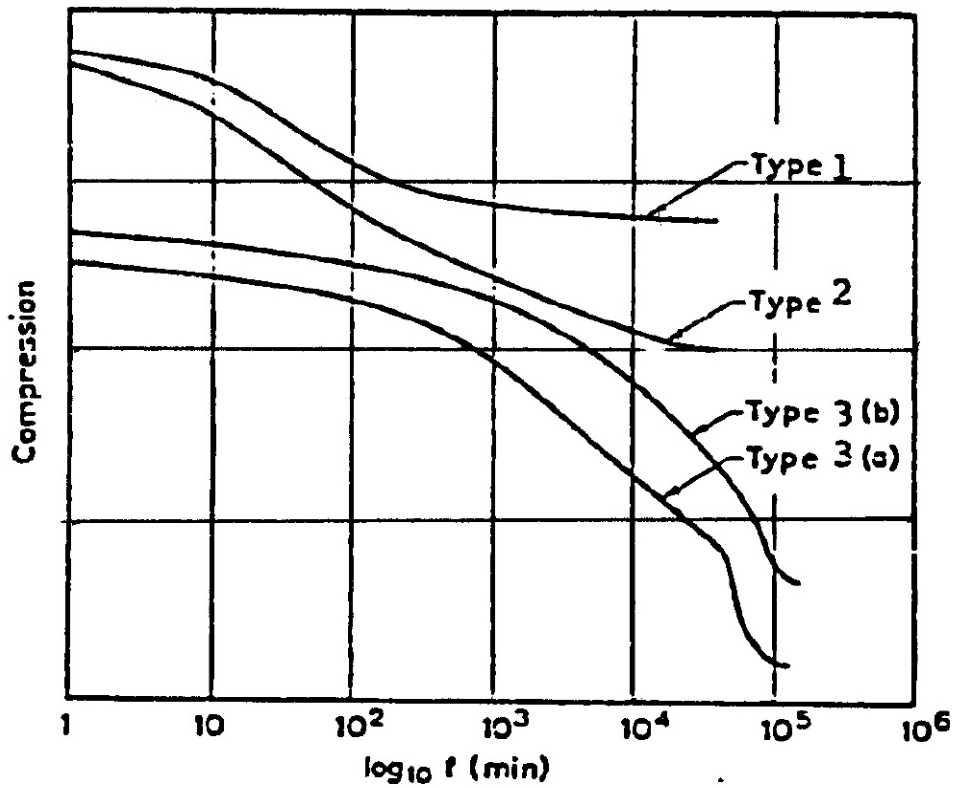

Figure 3.10: Types of secondary compression curves (After Gibson and Lo, 1961).

Each soil exhibits two stages of creep. Table 3.1 shows the different stages of secondary compression and the corresponding different magnitudes of the coefficient of secondary compression (C_α), where C_α is defined as the following:

$$C_\alpha = \frac{\Delta e / \Delta \log t}{1 + e_o}$$

Where C_α is the slope of line in (e-log t space) normalized to the initial void ratio. As seen in Table 3.1, values of C_α for SLA_{PS} range from 0.0008 to 0.0039. Various values of the coefficient of secondary compression have been reported from the literature. For example, C_α for normally-consolidated clays ranged from 0.005 to 0.02. For very plastic and organic soils, values of C_α of 0.03 or higher are common. For pre-compressed clays with an overconsolidation ratio (OCR = maximum previously applied pressure/existing applied pressure) > 2, a value of less than 0.001 is expected. Therefore, the values in Table 3.1 indicate that SLAs would behave similarly to the lower limit of normally-consolidated clay, with respect to secondary compression behavior.

The results presented in Figure 3.7 suggest that sand exhibits a transition in slope of e-log t curve at 240 min; SLA_{PS} also had a transition but at 2.0 days, while Pure PS started a different slope at 2.5 days. It is clear that, as the time elapses SLA_{PS} gain strength, as the density of the material increases forming a stiffer skeleton. On the other hand, sand particles have undergone continuous rearrangement and crushing and this results in a weaker skeleton; i.e., gradual acceleration in secondary compression. This is counter intuitive to the observation that the CCS is stiffer than either the PS or SLA_{PS}, which is illustrated in Figure 3.9 that shows shows the calculated strain rate vs. log of time for the applied stress of 700 kPa for all three materials. Based on this figure, it can be concluded that strain rate is decreasing dramatically as the time elapses and that the strain rate for CCS < the strain rate for Pure PS < the strain rate for SLA_{PS}, which indicates that sand is initially stiffer than either pure PS or SLA_{PS}.

Table 3.1: Coefficients of Secondary Compression for CCS, Pure PS, and SLA_{PS}

	Sand	Pure PS	SLA_{PS}
C_α	0.0012 (up to 0.17 day) 0.0018 (0.17 to10 days)	0.0035 (up to 25 days) 0.0004 (2.5 to 10 days)	0.0039 (up to 2.0 days) 0.0008 (2.0 to 10 days)
Notes: • The reported C_α is normalized to $(1+e_o)$, where e_o is the void ratio at the beginning of each stage • The time at the end of primary consolidation, t_p (elastic) for the first stages is 1 minute			

4.0 Conclusions

Based on the constant rate of strain testing, the following conclusions can be made:

- Generally, the rigidity or stiffness of the various materials increase with increasing rate of loading.
- Sand exhibits small strains (up to 10 %) at large stresses, while SLAs, flyash, PS, and EC exhibited large strains (10 to 60 %) at large stresses.
- The sand (CCS), SLAs, Pure PS, Flyash, and EC showed increase in strength and stiffness of their particles with increase in the rate of loading.
- SLAs exhibited increase in stiffness with increasing stress level.
- SLAs are highly compressible compared to CCS.
- The primary mechanism of particle compression in natural granular soils is particle rearrangement and breakage at contact points. For sand the particles are stiff and as the sample is experiencing increase in stress level, the breakage at contact points occurs. This results in a new void space and fabric between the particles that adapts to the increase in stress redistribution since the volume of the solid particles remains constant during the load application. In contrast, from the deformation curves, SLAs have highly compressible, ductile particles, and the main mechanism that governs the compression behavior of SLAsis particle deformation. Thus, as the SLAs

samples are experiencing increase in stress, particle deformation occurs resulting in a faster change in void ratios compared to sand particles.

- Due to the ductile behavior exhibited by the SLAs, these aggregates cannot be used as a load–bearing material where deformation needs to be minimized.

Based on the incremental consolidation test results, the following conclusions can be made for the secondary compression behavior of SLA_{PS}:

- SLA_{PS} exhibits secondary compression; the rate of which diminishes after an appreciable time period (2 to 2.5 days).
- The magnitude of secondary compression of SLA_{PS} was comparable to those of sand; with the advantage that the creep rate of SLA reduces dramatically while the creep rate of sand does not.
- The magnitude of secondary compression of SLAs is higher compared to those of sand, the fact that SLAs' creep decreases with time while the creep of sand does not (Figure 3.7) may provide SLAs with an advantage.

Based on this work, it can be concluded that a potential application of SLAs is in embankment fill over soft soils. Also, SLAs may be used anywhere where good drainage is desired and light loads (< 160 kPa) are expected, such as in fill behind retaining walls, fill around basement walls, and highway French drains.

5.0 Recommendations

Future work on this project should be directed towards a further analysis of the mechanical properties of the synthetic lightweight aggregates when subjected to cyclic loads. The following are some specific recommendation for further research.

1. Investigate the effect of elevated temperature on the mechanical behaviors of SLAs (e.g., above 200°F because this is the most expected temperature in hot regions)
2. Perform series of consolidation tests on other soft aggregates such as rubber and asphalt shingles, under the same conditions of testing, for comparison purposes.
3. Perform consolidation tests on SLAs made with lower (or higher) carbon content flyash to see if the different chemical composition of the flyash has any effect on the properties of the aggregates.
4. Perform consolidation tests on SLAs for a longer period of time to investigate the variation in secondary compression for different resins.
5. Evaluate SLAs with particle sizes above 4.75 mm to investigate SLAs as a gravely soil. A series of consolidation tests are needed to study the mechanical behavior of these new SLAs and further validate the current work.
6. Investigate the mechanical behaviors of SLAs in consolidation tests visually by employing a video microscope. The goal is to capture "visually" the real mechanism during consolidation and shearing phases.

6.0 Acknowledgements

The authors would like to thank Tufts University for providing the laboratory facilities used to do most of the testing. The authors also thank Professor Robert Malloy and the

Plastics Engineering Department at the University of Massachusetts at Lowell for developing and manufacturing the various SLAs used in this study.

7.0 References

1. American Chemistry Council Plastics Statistics. Accessed April 2007. http://www.americanchemistry.com/s_acc/sec_rss_link.asp?CID=996&DID=3965

2. American Coal Ash Association (ACAA) 2000 Coal Combustion Product (CCP) Production and Use. http://www.acaa-usa.org/CCP%20Survey/PDF/00SurveyComplete.PDF

3. American Society for Testing Materials (1994). *Annual book of ASTM Standards,* Vol. 8. *Soil and Rock,* pp. 530-550. Philadelphia: ASTM.

4. Casagrande, A., and Wilson, S.D. (1951). "*Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content*", Geotechnique, 2(3), 251-263.

5. Cook, G.J., (2000). "*Physical and Mechanical Properties of Composite Lightweight Aggregates*", Thesis submitted to Tufts University for partial fulfillment of the requirement of Master of Science from the Department of Civil and Environmental Engineering.

6. David, Y.P., and Campanella, R. G. (1997). "*Time-Dependent Behavior of Undisturbed Clay*", J. Geotech. Eng., 103 (7). 693-709.

7. Ehrig, R.J., (1989). *Plastics Recycling*, Hanser Publishers, New York, NY.

8. Gaudreau, A. J. (2002), "*Stress-Strain-Strength Behavior of Synthetic Lightweight Aggregates in Triaxial Compression*," Thesis submitted to Tufts University for fulfillment of the requirements for degree of Master of Science from the Department of Civil and Environmental Engineering.

9. Gibson, R.E. and Lo, K. Y., (1961). " *A Theory of Consolidation of Soils Exhibiting Secondary Compression*", Acta Polytechnica Scandinavia, Ci. 10296, 1-16.

10. Holmstrom, O.C., and Swan, C.W., (1999). "*Geotechnical Properties of Innovative Synthetic Lightweight Aggregates,*" Proceedings of the 1999 International Ash Utilization Symposium, Lexington, KY.

11. Holtz, R.D., and Kovacs, W.D. (1981). "*An Introduction to Geotechnical Engineering*" Prentice Hall, Englewood Cliffs, N.J.

12. Kashi, M.G., Swan, C.W., Holmstrom, O., and Malloy, R.A., (1999). "*Innovative Lightweight Synthetic Aggregates Developed from Coal Flyash,*" Proceedings from

the 13th International Symposium on Use and Management of Coal Combustion Products (CCPs), Orlando, Florida.

13. Kashi, M.G., Malloy, R.A., Swan, C.W., "Development Of Synthetic Aggregate For Construction Material" final report for Chelsea Center For Recycling And Economic Development, February 2001, 36pp.

14. Kosmatka,. S.H., and Panarese, W.C., (1988). "*Design and Control of Concrete Mixtures,*" 13th Ed., Portland Cement association, Skokie, Illinois.

15. Kreith, Frank (1994) Handbook of *Solid Waste Management*, McGraw-Hill, Inc.

16. Lefebvre, G., and LeBouef, D. (1987). "*Rate Effects and Cyclic loading of Sensitive Clays*", J. Geotech. Eng., 113(5), 476-489.

17. Plastics Resource: Recycling Facts from the American Plastics Council. http://www.plasticsresource.com/recycling/recycling_backgrounder/bk_1998.html

18. Plastics Resource: State of Plastics Recycling, http://www.plasticsresource.com/recycling/recycling_backgrounder/state_of_recycling.html

19. Richardson, Jr., M., and Whitman, R. V. (1963). " *Effect of Strain-Rate Upon Undrained Shear Resistance of a Standard Remolded Fat Clay*", Geotechnique, 13, 310-324.

20. Sheahan, T.C., Ladd, C.C., and Germaine, J.T. (1996). " *Rate-Dependent Undrained Shear Behavior of Saturated Clay*", J. Geotech. Eng., 122(2), 99-108.

21. Swan, C.W., and Sacks, A., (2005). "*Properties of Synthetic Lightweight Aggregates for Use in Pavement Systems,"* GSP 130 Advances in Pavement Engineering.

22. Taylor, D.W. (1943). "*Cylindrical Compression Research Program on Stress-Deformation and Strength Characteristics of Soils*", 9th progress report, U.S. Army corps. Of engineers, waterways expt. station, MIT, Mass.