Chapter 16

Geotechnical Properties and Applications Of Expanded Shale, Clay and Slate (ESCS) Structural Lightweight Aggregate

Referred to in this Chapter as ESCS Lightweight Aggregate, or Lightweight Aggregate

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16.1 INTRODUCTION

The Proven Solution

For almost 50 years Rotary Kiln produced Expanded Shale, Clay & Slate (ESCS) Lightweight Aggregate has been effectively used to solve geotechnical engineering problems and to convert unstable soil into usable land. Lightweight aggregate can reduce the weight of compacted geotechnical fills by up to one-half. Where thermal stability is required, lightweight aggregate provides significantly greater thermal resistance compared to soil, sand or gravel fill. It affords permanent economical insulation around waterlines, steam lines, and any other thermally sensitive vessel. This inert, durable, stable, free-draining and environmentally "friendly" aggregate is extremely easy to handle and provides economical long-term solutions for geotechnical challenges.

The Source

ESCS is produced from deposits of shales, clays, and slates. These minerals are principally composed of silica and alumina, similar to those used in brick and other ceramics known to be extremely durable. From the quarry (deposit) the raw material is taken to the preliminary crushing/screening plant, and then expanded in a rotary kiln.

Particle Shape, Color, Surface Texture

Depending on the source and method of production, lightweight aggregates exhibit considerable differences in particle shape, color and texture. Shapes may be cubical, rounded, angular, or irregular. Textures may range from fine pore, relatively smooth skins to highly irregular surfaces with large exposed pores. Particle shape and surface texture directly influence bulk loose as well as compacted densities.

ESCS aggregates are crushed and screened in a manner identical to crushed stone. As such, they contain minimal fines, closely resemble natural granular materials and are classified as “free draining”. In contrast to natural aggregates from borrow pits; the geotechnical performance of ESCS aggregates is very predictable.

The Material

ESCS lightweight aggregate has a long track record of quality and performance. Since its development in the early nineteen hundreds, ESCS produced by the rotary kiln process has been used extensively in asphalt road surfaces, concrete bridge decks, high-rise buildings, concrete precast/prestressed elements, and concrete masonry and geotechnical applications. The quality and low density of lightweight aggregate results from a carefully controlled manufacturing process. In a rotary kiln, selectively mined shale, clay or slate is fired in excess of 2000°F to the point of incipient fusion, causing the creation of a cellular structure of expansion within the particles that is retained upon cooling. The lightweight aggregate material is then processed to precise gradings. The result is a high quality ceramic lightweight aggregate that is inert, durable, tough stable, highly insulative, and free draining, ready to meet stringent structural specifications.

16.2 PHYSICAL PROPERTIES OF STRUCTURAL LIGHTWEIGHT AGGREGATE

Particle Shape and Surface Texture

Depending on the source and the method of production, lightweight aggregates exhibit considerable differences in particle shape and texture. Shapes may be cubical, rounded, angular, or irregular (Fig. 16.1 and 16.2). Textures may range from fine pore, relatively smooth skins to highly irregular surfaces with large exposed pores.

Particle shape and surface texture can directly influence the finished products. For example in concrete shape and texture influences workability, coarse-to-fine aggregate ratio, cement content requirements, and water demand in concrete mixtures, as well as other physical properties.

Figure 16.1 ***Lightweight Particle***

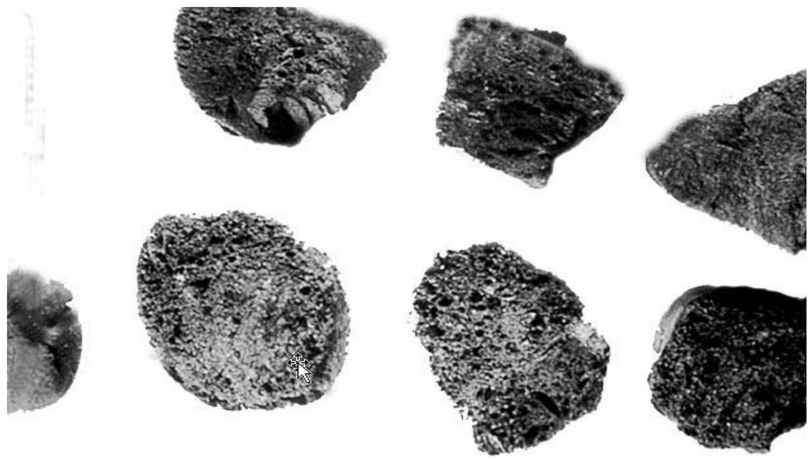


Figure 16.2 ***Structural ESCS lightweight aggregate that is composed of a strong, ceramic, vitreous material encapsulating a system of general non-interconnected pores. Although, the particle density is approximately 1/2 of natural aggregates this aggregate when used in concretes and geotechnical application provide the usual structural strengths, stability and durability.***

Grading Lightweight Aggregate

Grading requirements are generally similar to those provided for normalweight aggregate with the exception that lightweight aggregate particle size distribution permits a higher weight through smaller sieves. This modification recognizes the increase in relative density typical for the smaller particles of most lightweight aggregates, and that while standards are established by weights passing each sieve size, ideal formulations are developed through volumetric considerations.

For normalweight aggregates, the relative density of fractions retained on the different sieve sizes are nearly equal. Percentages retained on each size indicated by weight give a true indication of percentages by volume. However, the relative density of the various size fractions of lightweight aggregate usually increases as the particle size decreases. Some coarse aggregate particles may float on water, whereas material passing a No. 100 sieve (0.015 mm) may have a relative density approaching that of normalweight sand. It is the volume occupied by each fraction, and not the weight of material retained on each sieve, that determines the void content and paste content, and influences workability of the concrete. Percentages retained on each sieve and fineness modulus, by weight and by volume, are computed for comparison in the example illustrated in Table 16.1.

Table 16.1 Comparison of fineness modulus by weight and volume for typical lightweight aggregate.

Sieve Size No.	Opening in. (mm)	Percent Retained By weight	Cumulative Percent Retained by weight	Bulk Specific Gravity, SSD Basis	Percent Retained By Volume	Cumulative Percent Retained by volume
4	0.187 (4.75)	0	0	----	0	0
8	0.0937 (2.38)	22	22	1.55	26	26
16	0.0469 (1.19)	24	46	1.78	25	51
30	0.0234 (0.59)	19	65	1.90	19	70
50	0.0117 (0.30)	14	79	2.01	13	83
100	0.0059 (0.15)	12	91	2.16	10	93
Pan	----	9	100	2.40	7	100

Fineness modulus (by weight) 3.03 fineness modulus (by volume) = 3.23

A fineness modulus of 3.23 by volume in the example indicates a considerably coarse grading than that normally associated with the fineness modulus of 3.03 by weight. Therefore, lightweight aggregates require a larger percentage of material retained on the finer sieve sizes on a weight basis than do normalweight aggregates to provide an equal size distribution by volume.

The use of normalweight sand usually results in some increase in strength and modulus of elasticity. These increases, however, are made at the sacrifice of increase density. The mixture proportions selected, therefore, should consider these properties in conjunction with the corresponding effects on the overall economy of the structure.

Structural lightweight aggregate producers normally stock materials in several standard sizes that include coarse, intermediate, and fine gradings.

By combining size fractions or by replacing some or the entire fine fraction with normalweight sand, a wide range of concrete densities may be obtained. Aggregates for structural lightweight concrete usually have a top size of minus 3/4 in. or minus 1/2 in. Most lightweight concretes use a lightweight coarse aggregate 3/4 inc. to 4 mesh (1/2 - #8) with ordinary sand, minus 4 mesh (minus 4.8 mm), however other combinations of LWA and natural aggregate are used.

Aggregate for lightweight concrete masonry units are normally sized minus 3/8 in. (9.5 mm). This aggregate is usually the crushed variety because of improved machining characteristics and the zero slump concrete mix is drier than that for fresh structural concrete.

The aggregate producer is the best source of information for the proper aggregate combinations to meet fresh concrete density specifications and equilibrium density for dead load design considerations.

ESCS lightweight aggregate is manufactured to meet the ASTM C 330 requirement as shown in Table 16.3.

TABLE 16.3. ASTM C 330 Grading Requirements

Size in (mm)	1 (25)	3/4 (20)	1/2 (13)	3/8 (10)	#4 (5)	#8 (2)
3/4” to #4 (20 to 5 mm)	100	90-100	-	10-50	0-15	-
1/2" -#4 (13 to 5mm)	-	100	90-100	40-80	0-20	0-10
3/8” - #8 (10-2 mm)	-	-	100	80-100	5-40	0-20

Relative Density of Aggregate Particles

Structural Lightweight Aggregate has a low particle density due to the internal cellular pore system. The cellular structure within the particles is developed by heating certain raw materials to high temperatures to the point of incipient fusion, at which time gases are evolved within the pyroplastic mass, causing expansion that is retained upon cooling. Strong, durable, ceramic lightweight aggregates contain a relatively uniform system of pores that have a size range of approximately 5 to 300 μm enveloped in a high-strength vitreous phase. Pores close to the surface are readily permeable and fill within the first few hours of exposure to moisture. Interior pores, however, fill extremely slowly. A fraction of the interior pores are essentially non interconnected and may remain unfilled after years of immersion (Fig. 16.3).

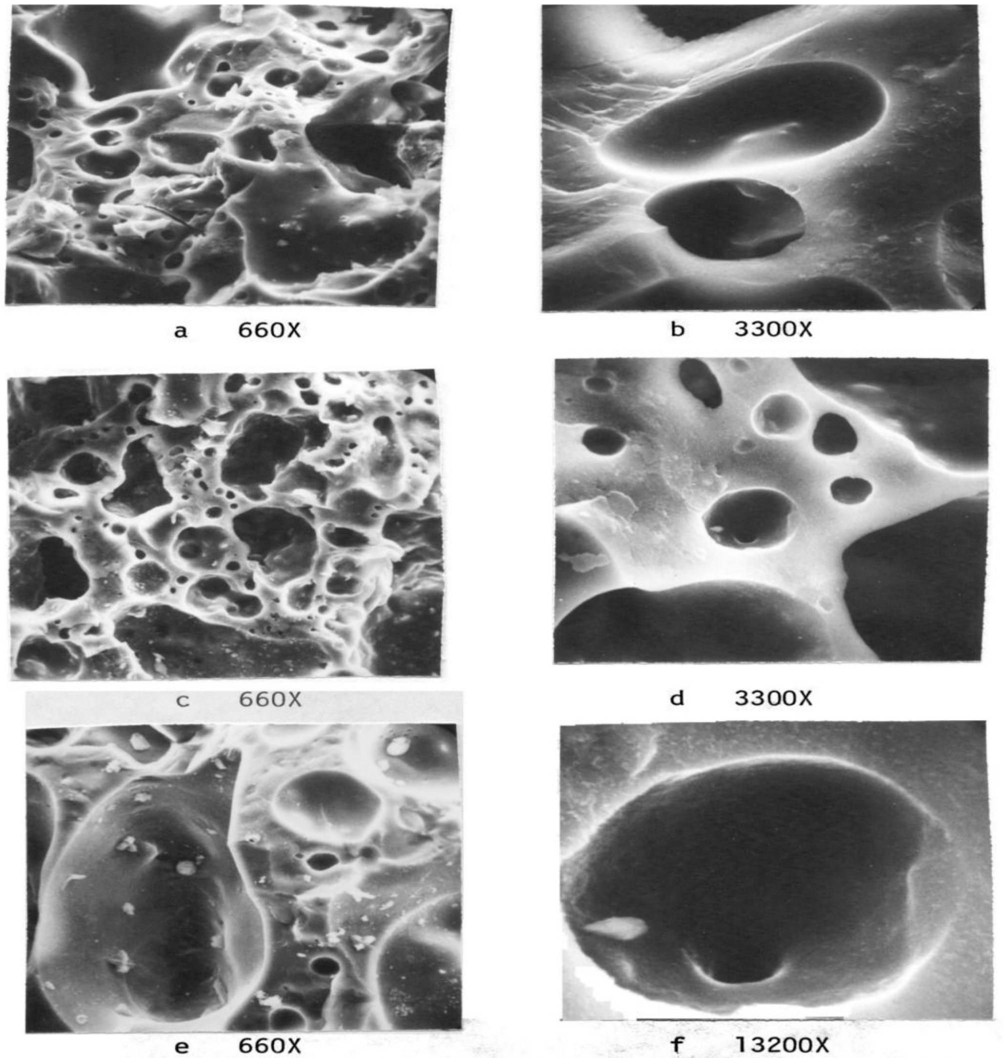


Figure 16.3 ***Scanning Electron Microscopy of Mature Lightweight Concrete showing the lightweight aggregate particle. Sample taken from the Cossakie Bridge deck, New York thruway (Holm et. al. 1984).***

The particle density of an aggregate is the ratio between the mass of the particle material and the volume occupied by the individual particles. This volume includes the pores within the particle, but does not include voids between the particles (Fig. 16.4). In general, the volume of the particles is determined from the volume displaced while submerged in water. Penetration of water into the aggregate particles during the test is limited by the aggregate's previous degree of saturation.

The oven-dry density of an individual particle depends both on the density of the solid vitreous material and the pore volume within the particles, and generally increases when particle size decreases. After pulverizing in a jar mill over an extended period, the relative density of the poreless, solid ceramic material was determined to be 2.60 by methods similar to those used in measuring the relative density of cement.

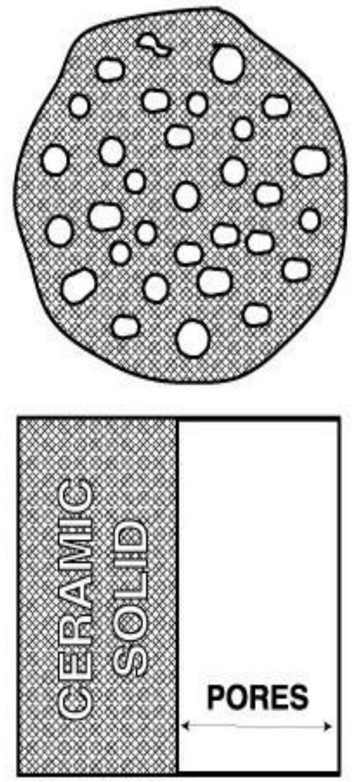


Figure 16.4 ***Schematic of Dry Lightweight Aggregate***

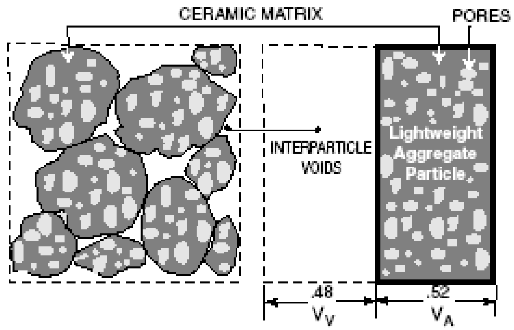
Aggregate Bulk Density

Aggregate bulk density is defined as the ratio of the mass of a given quantity of material and the total volume occupied by it. This volume includes the voids between, as well as the pores within the particles. Bulk density is a function of particle shape, density, size, gradings, and moisture content, as well as the method of packing the material (loose, vibrated, rodded) and varies not only for different materials, but for different sizes and gradations of a particular material. Table 16.2 summarizes the maximum bulk density for lightweight aggregates listed in ASTM C 330 and ASTM C 331. ASTM C 332 provides minimum density requirements for perlite and vermiculite to limit over-expanded, weak particles that would break down in mixing.

TABLE 16.2—*Requirements of ASTM C 330, C 331, and C 332 for Dry Loose Bulk Density of Lightweight Aggregates.*

Aggregate Size and Group	Maximum Dry Loose Bulk Density kg/m^3 (lb/ft^3)	Minimum Dry Loose Bulk Density kg/m^3 (lb/ft^3)
ASTM C 330 AND C 331		
fine aggregate	70 (1120)	...
coarse aggregate	55 (880)	...
combined fine and coarse aggregate	65 (1040)	...
ASTM C 332		
Group 1		
Perlite	12 (196)	7.5 (120)
Vermiculite	10 (160)	5.5 (88)
Group 2		
fine aggregate	70 (1120)	...
coarse aggregate	55 (880)	...
combined fine and coarse aggregate	65 (1040)	...

The relationship between the particle relative density and the bulk density of a aggregate sample is illustrated in Fig. 16.5 for a hypothetical lightweight aggregate.



The following calculations are based on a hypothetical lightweight aggregate sample (illustrated above) that has a bulk loose dry density of 44.6 lb/ft^3 (714 kg/m^3) and a relative density (SSD pychnometer) of 1.52 after a 24-hour soak resulting in a moisture content of 10.5% by weight. The relative density of the ceramic matrix was measured to be 2.60.

$$RD_D \left[\begin{array}{c}\text{Relative}\\ \text{Density,}\\ \text{Dry}\end{array}\right] = \frac{RD_{24}}{(1+M)} \left[\frac{\begin{array}{c}\text{Pychnometer Relative}\\ \text{Density after 24-Hour Soak}\end{array}}{\begin{array}{c}\text{Moisture Content by}\\ \text{Weight after 24-hour Soak}\end{array}}\right] = \frac{1.52}{1+.105} = 1.38\ (1380)$$

$$V_A \left[\begin{array}{c}\text{Fractional Part of Bulk}\\ \text{Volume Occupied by}\\ \text{Aggregate Particles}\end{array}\right] = \frac{D_B}{RD_D} \left[\frac{\begin{array}{c}\text{Measured Bulk Dry}\\ \text{Loose Density}\end{array}}{\begin{array}{c}\text{Relative Density}\\ \text{of Dry Particle}\end{array}}\right] = \frac{714}{1380} = 0.52$$

$$V_V \left[\begin{array}{c}\text{Fractional Part of Bulk}\\ \text{Volume Occupied by Voids}\\ \text{between Particles}\end{array}\right] = 1.00 - 0.52 = 0.48$$

Figure 16.5. ***Schematic Representation of Bulk Volume, Interparticle Voids and Internal Particle Pores Showing Fractional Volumes of the bulk density of lightweight aggregate***

Moisture Dynamics

The non-steady state exchanges of moisture in and out of particles of lightweight aggregate may be separated into two distinctly different processes. The first is when LWA is immersed in water (or another fluid) and continuously absorbs water, initially at a high rate, then at a significantly reduced rate, and then into a rate so slow that it takes years to conclude.

The second mechanism is characterized as "sorption" in which the moisture exchange is between the surface of the lightweight aggregate particle and the surrounding medium (air at differing relative humidity or hydrating cement paste in concrete). ACI 116 defines surface moisture (or adsorbed moisture) as free water retained on the surfaces of aggregate particles and considered to be part of the mixing water in concrete, as distinguished from absorbed water".

Adsorption – "Adsorption is considered to occur when a relatively dry material retains or takes up water in a vapor form from a surrounding atmosphere". and;

Desorbtion – "Desorbtion is the loss of adsorbed water [surface water] to a drying atmosphere" (Landgren, 1964).

If desorbtion is taking place then the internal (absorbed) moisture will gradually move to the surface and behave like surface moisture to further exchange with the surrounding medium.

The mechanism of the absorption of water into immersed or continuously prewet lightweight aggregate is widely understood and accounted for. The loss of moisture from unsealed LWA into the surrounding air (LWA with an extremely high degree of saturation, when laid on the floor of a laboratory will lose all but a small percentage of its absorbed water within two days) is not equally well known or appreciated. Lightweight aggregate is not hydrophilic (having a strong affinity for water).

When surrounded by a fine pore matrix (hydrating cement paste – smaller pores less than one micron) the large sized pores of a structural lightweight aggregate (typically from 5 to 300 microns) will have their moisture content lowered depends on the amount of moisture in the aggregate, due to the "wicking" action of the fine capillary pore system of the somewhat hydrophilic action of the hydrating cement paste. The rate of wicking slows as the cement hydrates and the capillaries close off. See appendices F (Valore, 1988) and G (Landgren, 1964) for sorption curves of lightweight aggregate, hydrated cement paste and bricks.

Therefore, soon after set, when the microporous structure of the hydrated cement paste develops, the moisture in the lightweight aggregate will serve as a reservoir for supplying the moisture necessary for providing the curing conditions essential for full hydration of the cement, this is commonly referred to as internal curing. As shown in Landgren's paper this emptying of water from the LWA will start at relative humidity lower than about 98% which happens just a shortly after hydration begin.

Absorption Characteristics

Due to their cellular structure, lightweight aggregates absorb more water than their ordinary aggregate counterparts. Based upon a 24-hour absorption test conducted in accordance with the procedures of ASTM C 127 and ASTM C 128, structural-grade lightweight aggregates will absorb from 5 to more than 25 percent moisture by mass of dry aggregate. By contrast, ordinary aggregates generally absorb less than 2 percent of moisture. The important distinction in stockpile moisture content is that with lightweight aggregates the moisture is largely absorbed into the interior of the particles, whereas with ordinary aggregates it is primarily surface moisture. Recognition of this difference is essential in mixture proportioning, batching, and control. Rate of absorption is unique to each lightweight aggregate, and is dependent on the characteristics of pore size, continuity, and distribution, particularly for those pores close to the surface.

When the aggregate is used in concrete the internally absorbed water within the particle is not immediately available for chemical interaction with cement as mixing water, and as such, does not enter into water-cement ratio (W/Cm) calculations. However, it is extremely beneficial in maintaining longer periods of hydration (Internal Curing) essential to improvements in the aggregate/matrix contact zone. Internal curing will also bring about a significant reduction of permeability by extending the period in which additional products of hydration are formed in the pores and capillaries of the binder.

As can be seen in Fig. 16.6 the rate of absorption can be divided into four regimes.

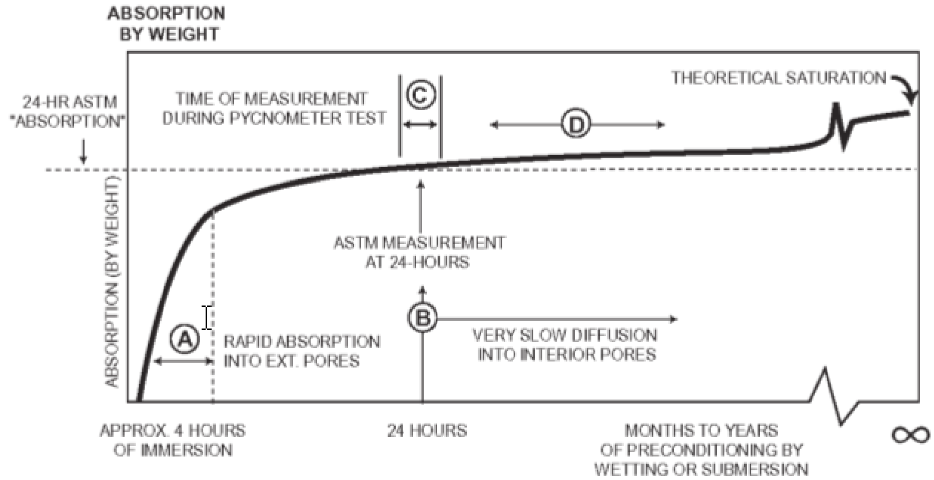
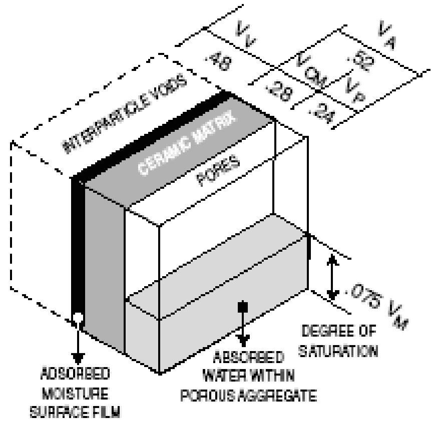


Figure 16.6 Absorption *vs. Time for typical structural grade ESCS lightweight aggregate*

Region A. Rapid entry of water by capillary absorption by close to surface pores within the first few hours.
Region B. Very slow diffusion into interior pores.
Region C. When the moisture content is approximately equal to that obtained by ASTM procedure (24 hour immersion), then the slope of the line reflecting further absorption represents the very slow process of diffusion. This is the basis for providing accurate relative density values during the relatively short time used to conduct pycnomter tests at 24 hours.
Region D. Absorption developed over an extended period of time used to mix, transport, place, and prior to initial set (6-8 hours $\pm$) will be very small, and therefore the W/Cm ration will be ***decreased*** by an equivalent small amount.

For illustrative purpose the water absorption with time and the resulting degree of saturation for a midrange, typical lightweight aggregate are shown in Figs. 16.7, 16.8 and Table 16.3.



$$\text{Fractional Part of Lightweight Aggregate Paticle } (V_A) \text{ occupied by the solid ceramic Matrix} = \frac{RD_D}{RD_{CM}} \left[\frac{\text{Relative Dry Density}}{\text{Relative Density of the Solid Ceramic Matrix}}\right] = \frac{1.38}{2.60} = 0.53$$

Fractional Part of Lightweight Aggregate Particle Occupied by Pores = 1.00 - 0.53 = 0.47

$$V_{CM} \left[\text{Fractional Part of Bulk Volume occupied by the Solid Ceramic Matrix}\right] = V_A \times \left[\text{Fractional Part of Aggregate Particle } (V_A) \text{ occupied by the Solid Ceramic Matrix}\right] = 0.52 \times 0.53 = 0.28$$

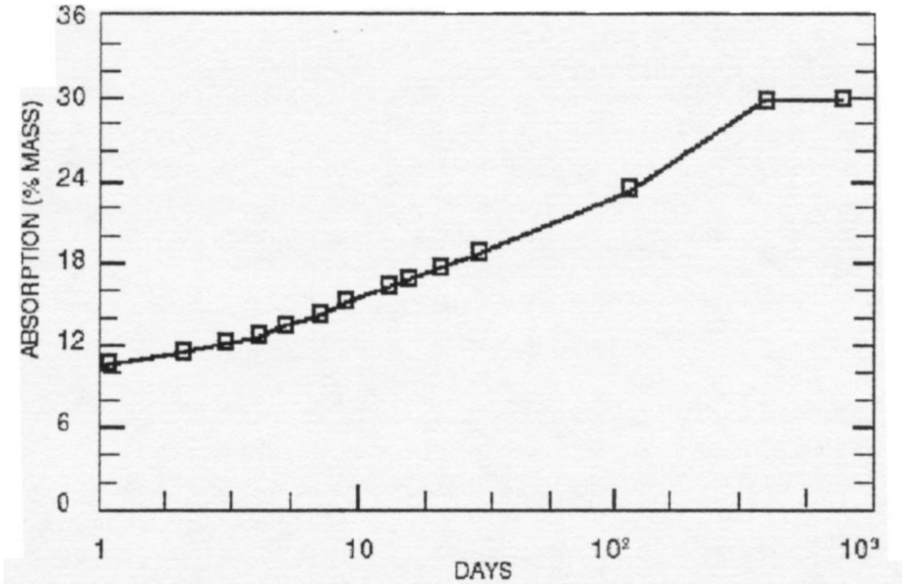
$$\therefore \; V_P \left[\text{Fractional Part of Bulk Volume Occupied by Pores in Aggregate}\right] = 0.52 - 0.28 = 0.24$$

$$V_M \left[\text{Fractional Volume of Bulk Loose Sample Occupied by Moisture}\right] = \frac{\left[\text{Moisture Content by Weight}\right]\left[\text{Bulk Loose Dry Density of Sample}\right]}{1000 \left[\text{Density of Water}\right]} = \frac{0.105 \times 714}{1000} = .075$$

$$DS \left[\text{Degree of Saturation of the Pores Occupied by Moisture}\right] = \frac{0.075}{0.24} = 0.31 \, *$$

* "Saturated Surface Day" after 24-hour submersion for this illustrative sample represents water filling only 31% of the available pore space.

Figure 16.7. ***Schematic representation of volumes occupied By the ceramic matrix, the remaining pores and the degree of saturation of absorbed water.***



Log of time, days

Figure 16.8. ***Water Absorption by Weight of Coarse Lightweight Aggregates during 2-years of Water Immersion***

Table 16.3. Aggregate Absorption and Degree of Saturation (Holm et. al. 2004)

Immersion Time	Water Absorption (% Mass)	Degree of Saturation	% of 24-Hour Soak	Relative Density Factor
0 mins	0	0	0	1.38
2 mins	5.76	.17	55	1.46
5 mins	6.15	.18	59	1.46
15 mins	6.75	.20	64	1.47
60 mins	7.74	.23	74	1.49
2 hours	8.32	.24	79	1.49
1 day	10.5	.31	100	1.52
3 days	12.11	.35	115	1.55
28 days	18.4	.54	175	1.63
4 months	23.4	.69	223	1.70
1 year	30	.88	285	1.79
2 years	30	.88	285	1.79

“Saturated” Surface Dry

ASTM C 127 and C 128 procedure prescribe measuring the “saturated” (*inaccurately named in the case of Lightweight Aggregates; partially saturated after a 24-hour soak is more accurate)* particle density in a pycnometer and then determining the absorbed moisture content on the sample that had been immersed in water for 24 hours. After a 24-hour immersion in water, the rate of moisture absorption into the lightweight aggregate will be so low that the partially saturated particle density will be essentially unchanged during the time necessary to take weight measurements in the pycnometer. After the moisture content is known, the oven-dry particle density may be directly computed. Fig. 16.9 illustrates typical ESCS lightweight aggregate.

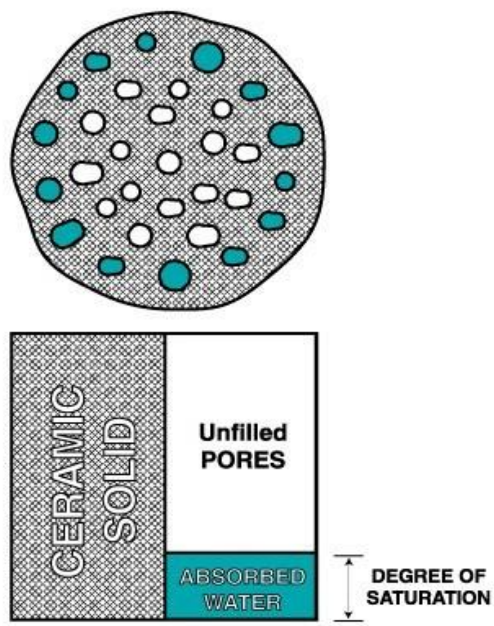


Figure 16.9 ***Schematic of “Saturated” Surface Dry as defined by ASTM C 127 and C 128 after 24-hour submersion***

Following ASTM procedures the measured physical properties of this particular lightweight aggregate are:

Relative Density, RD_{24} = 1.52
Moisture Absorption, M_{24} = 10.5%
Relative density solid, RD_{SOLIDS} = 2.6
Bulk Density, BD = 44.6 pcf (714 kg/m^3)

That after 24-hour immersion in a pychometer, measurements result in a relative density of 1.52 with an "absorption" of 10.5% by mass. The oven-dry particle density (**PD_{OD}**) may be back calculated to be as follows:

$$PD_{OD} = \frac{1.52}{(1 + .105)} = 1.38$$

It follows then that the fractional volume of ceramic solids, $V_S = \frac{1.38}{2.60} = .53$

Fraction Volume of pores, $V_P = 1.00 - .53 = .47$

The degree of saturation (DS: the extent to which the pores are filled)

$$DS = \frac{.105 \; x \, 2.60 \; x \, .53 \; (volume \;\; of \;\; absorbed \;\; water)}{.47 \; (Fractional \;\; volume \;\; of \;\; pores)} = .31$$

Following the prescribed ASTM procedures the DS for ESCS lightweight aggregate will generally be in the range of approximately 25 to 35% of the theoretical saturation. The use of the ASTM expression "saturated surface dry" is therefore, inappropriate for lightweight aggregate because it's theoretically inaccurate and analytically misleading.

Stockpile Moisture Content

From a practical perspective and considering the fact that most lightweight concrete is placed by pumping, the usual practice is to batch the lightweight aggregate at a moisture condition greater than the "Absorption Value" defined by ASTM C 127 procedures (24-hour immersion). In this condition the absorbed (internal) moisture content will be in excess of the 24 hour absorption value defined by ASTM. The degree of saturation (DS) necessary for adequate pumping characteristics, as determined by practical field experience, may be obtained from the ESCS supplier.

Example, assume for this hypothetical lightweight aggregate (Fig. 3.10) that experience has shown that the lightweight concrete will pump efficiently when the lightweight aggregate used has absorption of at least 17% by mass.

At that condition the $DS \text{ (Degree of Saturation)} = \frac{.17\,(2.60 \; x \, .53)}{.47} = .50 .$

Due to the continuous pre-wetting, and because of the very slow further tendency to absorb water into the aggregate, there will invariably be a film of surface (adsorbed) water on the surface of the lightweight aggregate. It is essential to evaluate this quantity of surface water for an accurate determination of the "net" mixing water that influences workability and determines the effective w/cm ratio.

Therefore, it is necessary to run the usual moisture test as follows. Measure the weight of the as-received surface moist sample (W_T). After towel drying, measure the weight of the surface dry sample (W_{TD}) and conduct the drying test.

Sample calculations:

Measured Weights (g)

W_T **(Total sample)** $= 602g$

W_{TD} **(Towel dried)** $= 562g$

W_{OD} **(Oven dried)** $= 480g$

Moisture Content (%)

$$M_T \text{ (Total Sample)} = \frac{602 - 480}{480} x 100 = 25.4\%$$

$$M_{AB} \text{ (Absorbed)} = \frac{562 - 480}{480} x 100 = 17.1\%$$

$$M_S \text{ (Surface)} = \frac{602 - 562}{480} x 100 = 8.3\%$$

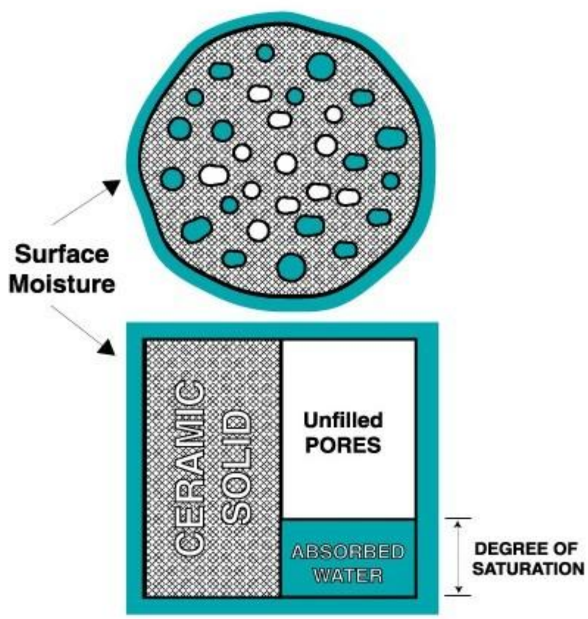


Figure 16.10 ***Schematic of "Partially Saturated" Surface Wet (Moisture Condition of Stockpiled Lightweight Aggregate with Some Surface Water)***

Full Saturation

Lightweight aggregate exposed to moisture in production plants and/or stored in open stockpiles will contain certain moisture content. Lightweight aggregates that are used alone in geotechnical, horticulture or asphalt applications are exposed to the weather, sprinkled or submerged, will continue to absorb water over time.

In the following LWA investigation, the effective particle density of a submerged LWA sample was measured throughout a two-year period to demonstrate long-term weight gain. Long-term absorption and relative density characteristics are also shown in Table 16.3, and Fig. 16.11 and Fig. 16.12. When moisture absorption-versus-time relationships are extrapolated or theoretical calculations used to estimate the total filling of all the lightweight aggregate pores, it can be shown that for this particular lightweight aggregate, the absorbed moisture content at total saturation (M@TS) after an infinite immersion will approach 34% by mass with a totally saturated particle density of 1.85 as can be seen in the following calculations:

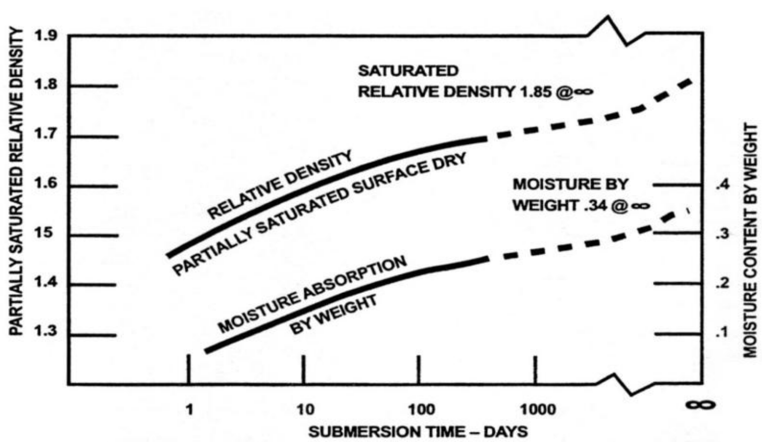


Figure 16.11. ***Moisture absorption (by weight) and relative density of lightweight aggregate versus time of submersion***

Moisture content at total saturation $M@_{TS} = \frac{.47\ x1.0}{.53\ x2.6} = .34$

Relative density at total saturation $RD@_{TS} = (.53\ x2.6) + (.47\ x1.0) = 1.85$

Complete filling of pores in a structural grade LA is unlikely because the non-interconnected pores are enveloped by a very dense ceramic matrix. However, these calculations do reveal a conservative upper limit for the density in submerged design considerations.

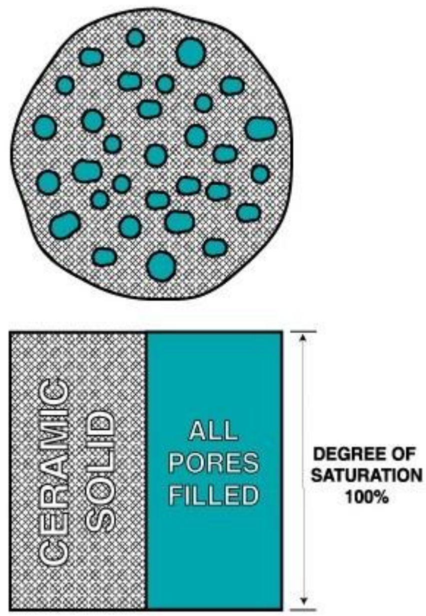


Figure 16.12 ***Schematic of Total Saturation (TS)***
Theoretically All Pores Filled

Chemical Stability

For more than 30-years ESCS aggregates have been successfully used in residential septic tank drain fields and commercial application where there was continuous exposure to waste product leachate. This has been necessary in some cases to replace local carbonate based (limestone & dolomite) aggregates that performed poorly.

In the paper "Expanded Clay and Shale Aggregates for Leachate Systems" presented in the GEO-Environmental Engineer, November 1997, the authors, Bowders et al., reported the results of tests in which lightweight aggregates were immersed in leachate solutions. The conclusions were:

"An additional issue is the durability of an aggregate after eight weeks of immersion in an actual municipal solid-waste landfill leachate, the aggregate showed no sign of deterioration. Gradations remained unchanged and index properties were about the same as those for unimmersed aggregate. Under immersion conditions, MSW (Municipal solid-waste) leachate does not appear to be any more detrimental to the expanded clay and expanded shale aggregates than it would be to other non-carbonate-bearing aggregates".

Durability Characteristics

The durability of structural ESCS lightweight aggregates used in structural concrete applications is well known. More than 600 major U.S. bridges built using structural lightweight concrete have demonstrated low maintenance and limited deterioration. Additionally, lightweight concrete has been used on numerous severely exposed marine structures, including large offshore platforms.

Long-term durability of lightweight aggregate use in geotechnical application was demonstrated in 1991 by reclaiming and testing samples of the aggregates supplied in 1968 to a Hudson River site. Magnesium soundness tests conducted on the reclaimed aggregate sample exposed to long-term weathering resulted in soundness loss values comparable to those measured and reported in routine quality control testing procedures 23 years earlier. There was little long-term deterioration due to continuous submersion and freeze-thaw cycling at the waterline.

ASTM standard specifications C 330 and C 331 for lightweight aggregate have no requirements for corrosive chemicals limitations. The American Concrete Institute Building Code (ACI 318) mandates chloride limitations in the overall concrete mass because of concern for reinforcing bar corrosion, but no limits are specified for individual constituents. Numerous geotechnical projects specifications calling for lightweight aggregates have limited water soluble chloride content in the aggregate to be less than 100 ppm when measure by AASHTO T 291.

All this may seem academic; when the durability of vitrified ceramics is examined from an archeological perspective. Indeed, clay tablets, brick and pottery are in some instances the only remaining vestige of ancient empires. Often all that remains at the sites of ships that sunk centuries ago are ceramic wine vessels. Inspection of the piers at the Roman fish-farming complex in Cosa, Italy revealed a wise choice by the builders that used broken ceramic shards from a nearby pottery, perhaps the first use of a manufactured aggregate. After 2 millennia of exposure to salt water and wave action these piers are still largely intact.

16.3 GEOTECHNICAL PROPERTIES OF LIGHTWEIGHT BACKFILL

In-place Compacted Moist Density

Results of compacted lightweight aggregate density tests conducted in accordance with laboratory procedures (Proctor tests) should be interpreted differently from those for natural soils. Two fundamental aspects of lightweight aggregate backfill will modify the usual interpretation soils engineers place on Proctor test data. The first is that the absorption of lightweight aggregate is greater than natural soils. Part of the water added during tests will be absorbed within the aggregate particle and will not affect interparticle physics (bulking, lubrication of the surfaces, etc.). Second, unlike cohesive natural soils, structural grade lightweight aggregate are designed to contain limited fines, limiting the increase in density due to packing of the fines between large particles. **The objective in compacting lightweight-backfill is not to aim for maximum in-place density, but to strive for an optimum density that combines high stability without unduly increasing compacted density.** Optimum field density is commonly achieved by two to four passes of rubber tire equipment. Excessive particle degradation developed by steel-tracked rolling equipment should be avoided.

A testing program sponsored by ESCSI was conducted at MACTEC (Law) Engineering (April 29, 2003) Herndon, Virginia on four sources of ESCS aggregates as contained in Appendix C. This program compared the results of laboratory compaction tests conducted in accordance with the procedures of:

- ASTM D 4254, "*Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density".*
- ASTM D 4253, "*Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table".*
- With a modification of ASTM D 698, *"Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort".* Modified and referenced here as on point proctor test (OPP).

(These standards are enclosed in Appendix C)

These tests along with similar tests conducted earlier on two ESCS aggregates revealed:

As shown in Table 16.4 the one point proctor test (OPP) produced percent compaction results that vary from 0.95 to 1.05 with and average of 1.00 of the maximum compacted densities achieved by following the procedures of ASTM 4253 Maximum Index Density Method. Table 16.4 shows the effect particle shape has on density.

Table 16.4. Summary of ESCSI Laboratory Testing Programs on the Compacted Density of ESCS Lightweight-Backfill

(4 MACTEC Density Tests (4/2003), plus 2 tests from (3/2001))

Δ Relative Density Max D4254 Min D4253 **Note "A"**	**Δ** One Point Proctor OPP **Note "A"**	Percent Compaction OPP ÷ Max **Note "B"**	Moisture As Received %	Particle Shape SA SR	Grading	Densification Due to Compaction OPP ÷ Min **Note "C"**
U 57.4 Max -50.2 Min +7.2 **Δ**	57.3 -50.2 +7.1 **Δ**	1.00	1.1	SA	3/8-#8	1.14
V 41.5 Max -36.8 Min +4.7 **Δ**	39.4 +2.6 **Δ**	.95	1.2	SR	¾-#4	1.07
W 55.5 Max -51.6 Min +3.9 **Δ**	54.3 +2.7 **Δ**	0.98	2.0	SR	3/8-#8	1.05
X 51.6 Max -49.7 Min +1.9 **Δ**	54.0 +4.3 **Δ**	1.05	3.0	SA	¾-#4	1.05
Y 55.3 Max -52.5 Min +2.3 **Δ**	58.0 +5.5 **Δ**	1.05	21.3	SA	¾-#4	1.10
Z 41.6 Max -38.2 Min +3.4 **Δ**	41.1 +2.9 **Δ**	0.99	0.4	SR	½-#4	1.08

All densities shown are as tested with as received moisture content.
OPP = One Point Proctor (Modified ASTM D 698)
SA = Sub-angular
SR = Sub-rounded

Table 16.5. EFFECT OF PARTICAL SHAPE AND GRADING ON DENSITY

(Analysis of 4 tests MACTEC 4/2002 plus 2 tests 3/2001)

Aggregate Samples	Percent Compaction OPP ÷ Max	Densification Due to Compaction OPP ÷ Min	Density increase Due to Compaction (OPP – Min) lb/ft³
Group I			
Y	1.05	1.10	5.5
X	1.05	1.05	2.8
U	1.00	1.14	7.2
		Ave.1.10	Ave. 5.2
Group II			
Z	0.99	1.08	2.6
W	0.98	1.05	1.7
V	0.95	1.05	2.7
		Ave.1.06	Ave.2.3
Average of All tests	1.00	1.08	3.8
Variation	very small	Small, predictable	small within group

Group I — Are ESCS aggregates with a sub-angular (SA)particle shape and a wide range of sizes (3/4 - #4).

Group II — Are ESCS aggregate with a sub-rounded (SR) particle shape and a relatively small range of sizes (3/8 - #4)

Notes to Table 16.3 and 16.4

A. One point proctor (OPP): 2 measurements (one preliminary lab, one field) Relative density: 3 measurements (two labs, one field-how tested?)
B. OPP / Max(lab) = 1.00 average, OPP ≈ Max(lab) The simple one-point proctor density can be determined in lab, office, field, with usual concrete technology equipment (0.5 cf bucket, rod), and develop densities very close to that obtained with heavy surcharge and vibration in the laboratory.
C. Densification due to compaction 1.08 Average (Range 1.05-1.14)
D. Average increase in density due to compaction 3.8 pcf (1.7 to 7.2)

ESCSI Recommended Compaction Procedure: *Based upon the results of laboratory tests as well as the experience gained in field testing on major lightweight aggregate geotechnical projects, ESCSI recommends the following procedure:*

Compacted moist density shall be determined by a modification of ASTM D 698 (AASHTO T 99) "Standard Test Methods for Laboratory Compaction Characteristics for Soil Using Standard Effort". The aggregate shall be placed in three layers in a standard 0.5 cubic foot bucket, with each layer compacted by 25 blows of a 5.5 pound hammer dropped from a distance of 12 inches. The aggregate is compacted only once at the received moisture content. This procedure is referenced to as the one point proctor (OPP).

Shear Strength

ESCS lightweight aggregate provides an essentially cohesionless, granular fill that develops stability from inter-particle friction. Extensive triaxial compression tests conducted by Stoll and Holm (1985) on large 250 x 600 mm (10 x 24 in. high) specimens have confirmed angles of internal friction of 38 degrees. This comprehensive testing program was completed on ESCS form six production plants. It included variations in grading, moisture content, and compaction levels, and revealed consistently high angles of internal friction.

Additionally, an extensive direct shear testing program conducted by Valsangkar and Holm (1990) confirmed high angle of internal friction measured on large-scale triaxial compression testing procedures.

Triaxial Compression Tests on Lightweight Aggregates: In order to determine the resistance to lateral forces developed by compacted lightweight aggregate, large scale triaxial compression tests were conducted at Columbia University's Geotechnical Laboratory under the direction of Professor Robert D. Stoll (Fig. 16.9). The failure surface developed during the course of the test was always clearly visible.

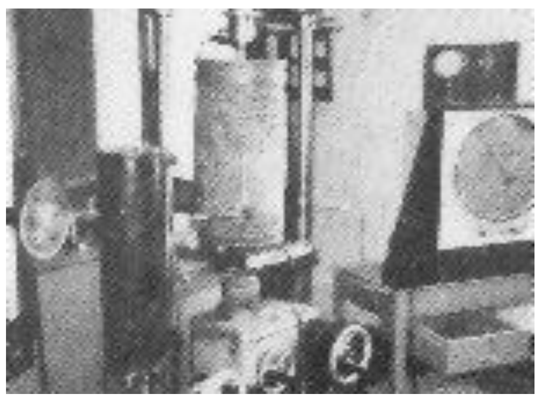


Figure 16.13. Triaxial Compression Testing

Standard laboratory equipment is appropriate when testing samples of small particles (soil) but not for coarse aggregate specimens. This testing program conducted on stockpile samples provided an assurance of repeatability in testing. Further tests evaluated the influence of aggregate moisture conditions on the angle of internal friction (Ø). Finally, a two-year program was conducted on five lightweight aggregates from other rotary kiln plants in other geographic areas to determine the effects of differing aggregate properties (particle strength, shape and grading) on the angle of internal friction. Based on the tests on this particular lightweight aggregate the angle of internal friction was determined to be in excess of 40 degrees in loose condition and slightly higher in a compacted condition.

All of the triaxial tests were run on specimens approximately 25.4 cm (10 in.) in diameter and 61.0 (24 in.) long. Specimens were confined in a rubber sleeve with a wall thickness of approximately 1.5 mm. Isotropic confining stress was applied to specimens by connecting a controlled vacuum through a port in either the tip or bottom platen. All testes were run at a constant rate of axial displacement which was equivalent to an average strain rate of 0.7% per minute.

Tests were run on “loose” and “compacted” specimens for each different material. The loose specimens were prepared by gently placing the aggregate into the forming mold one scoop at a time, with an effort made to avoid vibration or other disturbance. Once in place the aggregate was not leveled or rearranged. In the tests on “compacted” aggregate, each specimen was compacted in five layers with 25 blows of a 24.5 N (5.5 lb.) hammer falling 30.5 cm (12 inches) on each layer. The densities produced by these procedures as well as other information about the source of the samples are given in Table 16.5. The difference in density between the loose and compacted specimens is about the same as the difference between the maximum and minimum dry densities that resulted when the standard ASTM (D4253 and 4254) tests for the relative density of cohesionless soils were performed.

Fig. 16.14 shows the stress-strain curves obtained for six sets of tests. Most of the tests were run at the moisture content “as received” in the lab. Four of the tests (1 through 4) were run on a coarse fraction (passing the 3/4 inch sieve and retained in the No. 4 sieve). The figure shows a difference in response between the aggregates tested. A physical inspection revealed a difference in the particle shape and texture. While there is some variation in the angle of friction determined at the peak stress, a more significant difference may be the amount of strain that is required to develop the full shearing strength.

Table 16.5. ***Physical Properties of Aggregates Used in Tests***

Lightweight Aggregate Sample	Aggregate Grading	Water Content At Test Time (%)		Dry Density (pcf)	
		Compact	Loose	Compact	Loose
ESCS 1	3/4 in/No. 4	5.3	7.1	52.0	46.0
ESCS 2	3/4 in/No. 4	7.2	6.7	53.4	47.6
ESCS 3	3/4 in/No. 4	4.0	6.0	49.2	41.7
ESCS 4	3/4 in/No. 4	8.1	8.4	50.6	46.4
ESCS 5	3/8 to Pan	8.2	8.4	61.9	53.9
ESCS 6	3/8 in/No. 8	.01	1.4	53.0	47.1

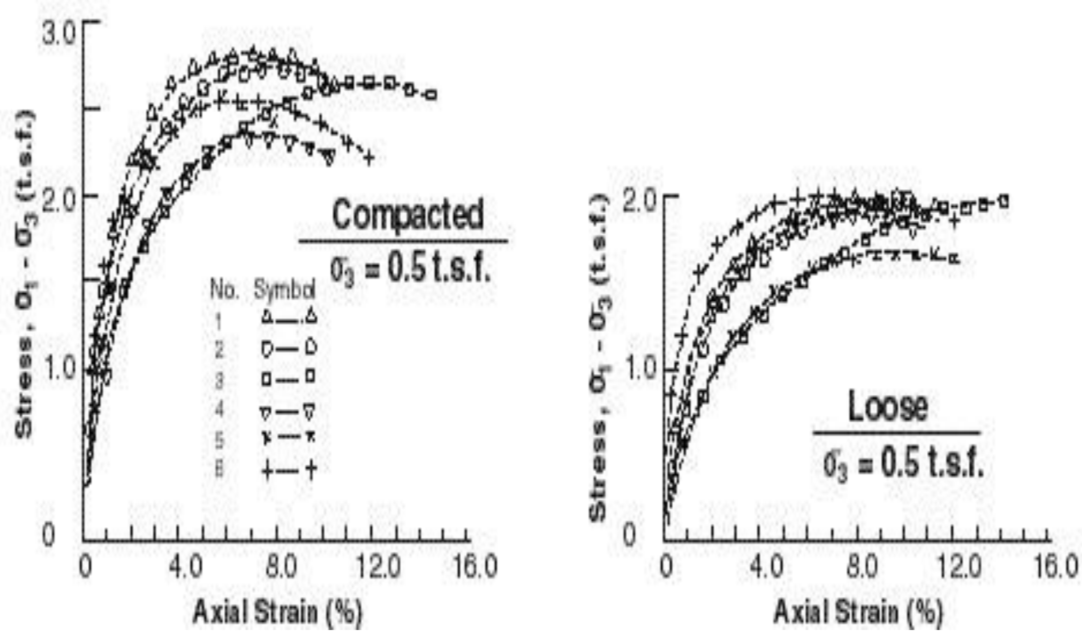


Figure 16.14. ***Stress-Strain Curves for Triaxial Compression Test (1 t.s.f. = 95.8 kPa)***

Aggregates 5 and 6 in Figure 16.14 contain intermediate and fine fractions. Which are also commonly available at most ESCS manufacturing plants. In these materials the coarsest particles are those passing the 3/8 inch sieve, and there is a more noticeable stress drop-off after the peak, as is typical in many well graded granular soils. In general, the curves shown are quite similar to what is obtained for many common gradations of ordinary fill. For the compacted aggregates, the angle of internal friction corresponding to the peak stress difference varies from 44.5° to 48°, whereas for the loose material, the range was 39.5° to 42°.

In the case of sample ESCS 1, aggregate tests were performed at several different confining pressures indicated that the Mohr envelop was essentially a straight line passing through the origin. In addition, tests were run on this material after it had been soaked in water for a period of five weeks. In the tests on water-soaked aggregate, the angle of internal friction was 1° to 2° lower than for the tests on the air dry or slightly moist materials.

Direct Shear Tests on Lightweight Aggregate: The size of the direct shear box is 18 x 12 x 25 inches (450 x 305 x 600 mm) deep. This equipment was developed at University of New Brunswick, Canada for testing coarse materials and has been used to test peat, landfill samples, and coarse aggregates. The upper box is fixed in its position, and the lower box is pushed on specially designed roller bearings using a hydraulic jack (Figure 16.15). A unique feature of the apparatus is the provision of two jacks for application of normal loads. The pressures in the jacks are manipulated during shearing of the soil specimen to prevent lifting and tilting of the shear box, and to counteract moments generated by the nonaligned nature of horizontal forces on the lower and upper boxes of the shear device. The soil specimen is sheared at a fairly constant rate using the hydraulic jacks, and the horizontal loads are measured using either a proving ring or a load cell.

In the series of tests where geotextiles were used, a specially designed clamp was used and attached to the walls of the lower box to hold the geotextiles in place. The clamp consisted of a turnbuckle and wooden rod. In all the tests performed in this series, the geotextile was always located at the interface between the two halves of the shear box.

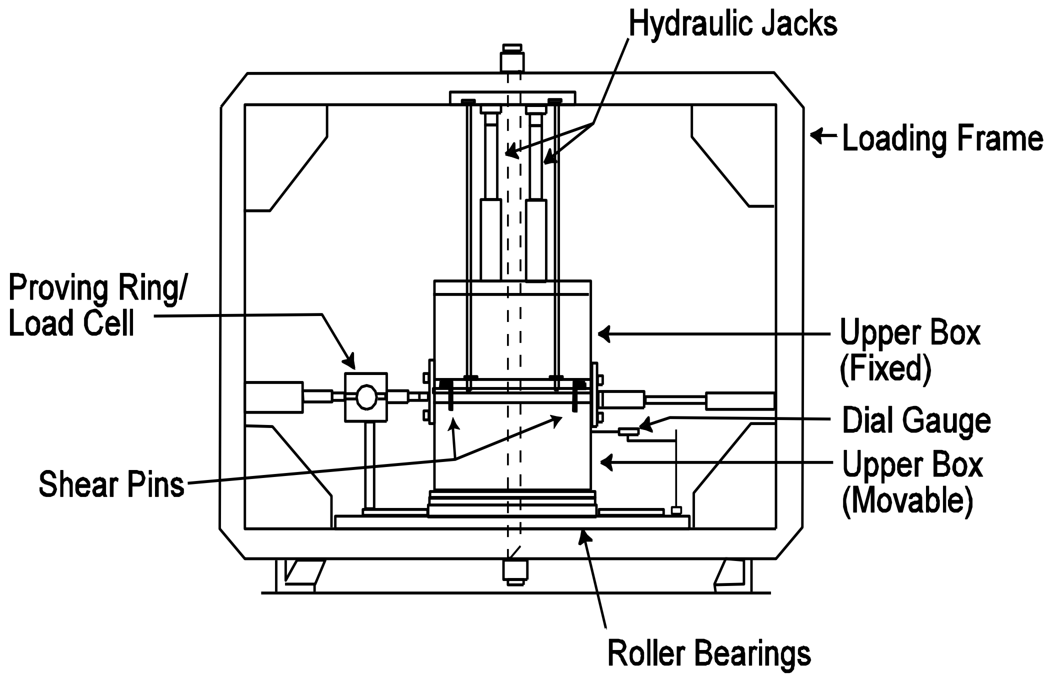


Figure 16.15. ***Large-size direct shear test apparatus***

Table16.6. ***Angle of Internal Friction for Coarse Aggregates***

Material	**Test Method**	**Angle of Internal Friction (Ø) Degrees as Determined by Direct Shear (DS) and Triaxial Compression Tests (T)**			
		Loose		**Compact**	
		Ø	**Density**	**Ø**	**Density**
ESCS AGG A	DS	40.5	58	48	66
ESCS AGG B	DS	40.0	52	45.5	58
Limestone	DS	37.0	107	N/A	118
ESCS AGG B	T*	39.5	46	44.5	52

*** Data from triaxial tests, Stoll and Holm (1985)**
Note: Unit weight for loose and compact specimens are the same as for Consolidation tests.

The first series of direct shear tests was performed on loose and compact specimens of normalweight and lightweight aggregates to investigate the effect of the relative density on the angle of internal friction. Results are presented in Table 16.6 for all the aggregates tested.

A comparison of the data from the present series with results of triaxial tests (on one material) reported in Stoll and Holm is also presented in Table 16.6. It is seen that results of the direct shear tests are in good agreement with the triaxial testing data. The data also shows that there is a difference in response between the ESCS A and ESCS B aggregate for compact relative density. A detailed visual examination indicated that the lightweight aggregate is relatively more angular, which explains high values of angles of internal friction and less compressible behavior. The results of the direct shear testing indicate that the shear strength characteristics of lightweight aggregate are similar to commonly used normalweight aggregates.

Interaction Between Lightweight Aggregate and Geotextiles

Valsangkar and Holm reported results of testing programs on the interaction between geotextiles and lightweight aggregate fills that included the variables of differing aggregate types and densities, thickness of aggregate layer, and geotextile types. The results indicated that the overall roadbed stiffness is unaffected when lightweight aggregate is used instead of normalweight aggregate for small deflections and initial load applications. These tests were followed by a large-scale test, which reported that the comparison of the friction angles between the lightweight aggregate or the normalweight aggregate and the geotextiles indicate that interface friction characteristics are, in general, better for lightweight aggregate than normalweight aggregates.

Results of the direct shear tests performed with geotextiles incorporated at the interface between the upper and lower shear box are presented in Table 16.7. It should be noted that all the tests in this series were performed on loose aggregate specimens only. For the combination of aggregate/geotextile/peat subgrade, the road structure was inverted in the shear box due to high compressibility of peat. The aggregate was placed in the lower box and peat in the upper box with geotextile located at the interface. With this arrangement, the geotextile remained at the interface between the two halves of the shear box in spite of considerable compression of the peat under normal stress.

Table 16.7. ***Friction Angle Between Geotextiles and Coarse Aggregates***

Material in Lower Box	**Material in Upper Box**	**Fabric**	**Fraction Angle (Degree)**
Limestone	Limestone	Woven	41.0
Limestone	Limestone	Nonwoven	42.0
ESCS (A)	ESCS (A)	Woven	47.8
ESCS (A)	ESCS (A)	Nonwoven	47.0
Peat	Peat	---	31.0
Limestone	Peat	Woven	32.0
Limestone	Peat	Nonwoven	32.0
ESCS (A)	Peat	Woven	32.0
ESCS (A)	Peat	Nonwoven	32.0
Peat	Peat	Woven'	31.0
Peat	Peat	Nonwoven	30.0

Note: Water content of peat = 600%, Unit weight of limestone aggregate = 13.5 kN/m³, Unit weight of ESCS (A) = 8.5 kN/m³
To convert density from kN/m³ to lb/ft³ multiply by 6.37
To convert stress from kPa to lb/in² multiply by .147

Compressibility

Compressibility tests completed on lightweight aggregate fill have demonstrated that the curvature and slope of the ESCS backfill stress-strain curves in confined compression were similar to those developed for companion limestone samples (Addo, 1986). Cyclic plate-bearing tests on lightweight aggregate fills demonstrated vertical subgrade reaction responses that were essentially similar for the lightweight and normalweight aggregate samples tested (Valsangkar and Holm, 1993).

ASTM C 330 specifications required all structural lightweight aggregates to develop concrete strengths above 17 Mpa (2500 psi). All structural ESCS concrete will develop 34.4 Mpa (5000 psi), and small number can be used in concretes that develop compressive strengths greater than 69 Mpa (10,000 psi).

Materials Tested: ESCS aggregates from two sources were studied. The particles tested were subangular in shape, durable and chemically inert. The expanded, vitrified particles are screened to produce the desired gradation for a particular usage. In the geotechnical applications, coarse aggregates with particle sizes between 5 mm to 25 mm are commonly used and materials within this grading were used in this testing program.

Both ESCS aggregates studied had a grain size distribution varying between 3/4-#4 (19 and 4.7 mm). The uniformity coefficient of ESCS lightweight aggregate LWA #1 was 1.4, whereas for the lightweight aggregate LWA #2 the coefficient was 1.5.

One-dimensional compressibility and direct shear tests using normalweight crushed limestone aggregate were performed by Addo in 1986, and these data were used in the test for comparison purposes. This material has a uniformity coefficient of 1.4 and a grain size distribution varying between 3/4-#4 (19 and 4.7 mm).

Test Procedures: One-dimensional compressibility tests were performed in a 550-mm diameter 306-mm-deep floating steel ring. The vertical loads were applied by a 100-ton capacity hydraulic jack and settlements were measured by three dial gages. One of the special features of the large-size consolidometer is the provision of three strainsert bolts attached to the confining steel ring. These bolts are 19 mm in diameter and instrumented with strain gages. The bottom end of the bolt is connected to the outside wall of the floating ring. The top of the bolt is inserted into a slot in the cylindrical housing, which is attached to the bottom plate of the consolidometer (Fig. 16.16). With this arrangement, the frictional forces mobilized on the wall of the consolidometer exert tensile force on the strainsert bolts, which are monitored during the loading of soil specimens. For each load increment, the applied load at the top of the soil specimen is known, and the load at the bottom is calculated from the strainsert bolt data. The average axial stress on the soil specimen is calculated by taking the algebraic mean of the load at the top and bottom.

The friction mobilized along the perimeter of the floating ring increase with the applied axial load and the relative density of the soil specimen. At a maximum load of 150 kN, approximately 30% of the applied load was transmitted in side friction for loose specimens for all aggregates tested. These data indicate the importance of measurements and accounting for the side friction in large-scale one-dimensional compression testing. During each load increment time-dependent settlements were monitored, and the next load increment was applied when the settlement under the previous load was complete. All the tests were done on dry specimens only.

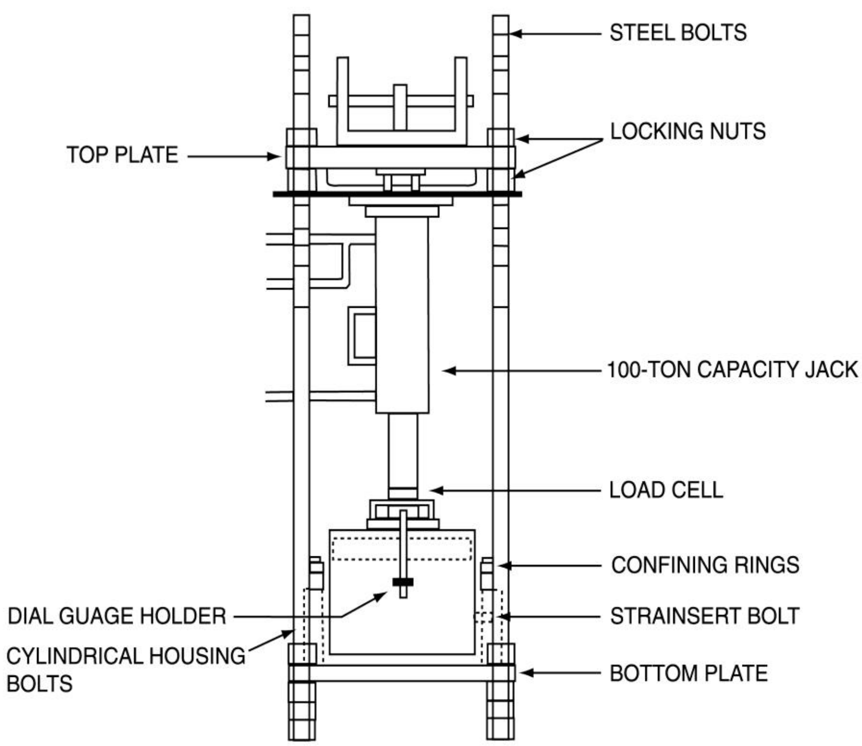


Figure 16.16. ***100-ton capacity consolidometer***

Test Results: Table 16.8 summarizes the materials and densities employed in the compressibility test series. Compressibility test results are present in Figs. 16.17 and 16.18 for compact and loose sample, respectively. In additions to the test data for lightweight aggregates, compressibility curves for normalweight crushed limestone aggregate are also presented

Table 16.8. ***Compressibility Test Series***

Material	Dry Unit Weight kN/m²	
	Loose	**Compact**
LWA # 1	9.11	10.41
LWA # 2	8.24	9.16
Limestone	16.73	18.50

To obtain lb/ft³ multiply kN/m² by 6.36

for comparison purposes. From Figure 16.13 and 16.14 it is seen that the curvature and the slop of the stress-strain curves in confined compression for the first monotonic and subsequent cyclic loadings are similar for crushed limestone and LWA #2 lightweight aggregate. The LWA #1 lightweight aggregate with a similar grain size and relative density appears to be relatively lest compressible. The trend reported by Stoll and Holm 1985 of the increased slope of stress-strain curve sequent to the first monotonic loading is observed both for the normalweight and lightweight aggregates. Also, the unloading and reloading curves are very flat for all the aggregates tested in this series.

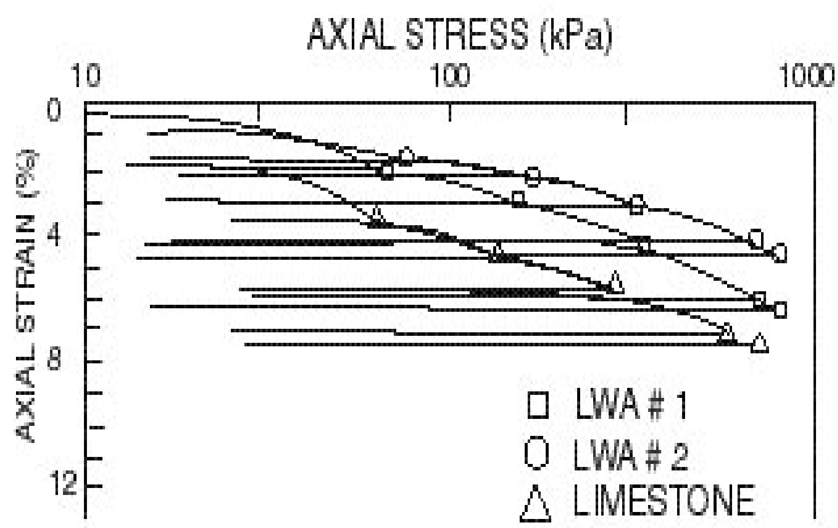


Figure 16.17. ***One-dimensional compression stress/strain curves for compact coarse aggregates***

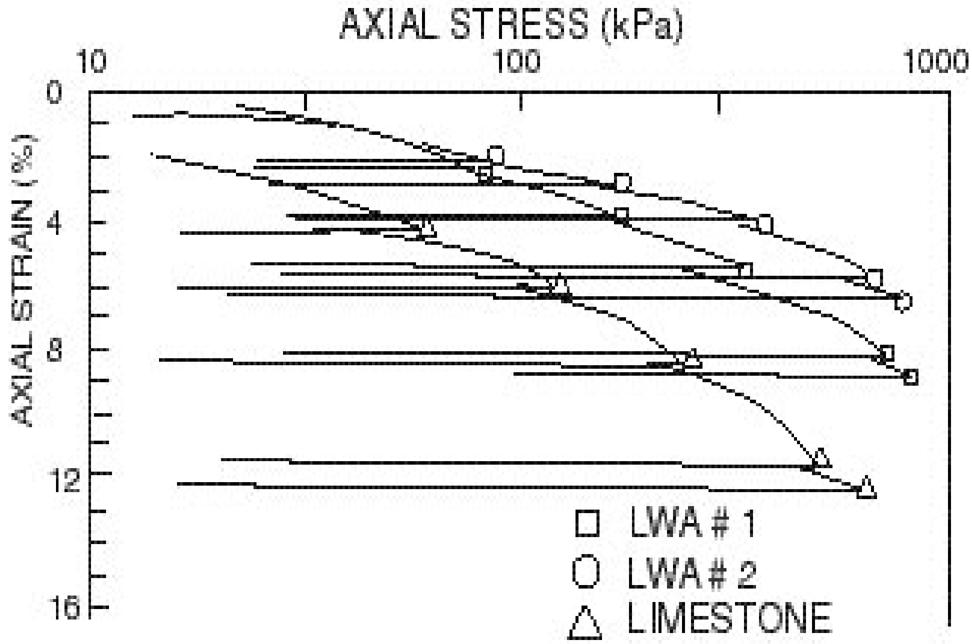


Figure 16.18. ***One-dimensional compression stress/strain curves for loose coarse aggregates***

At the end of five cycles of loading, grain-size analysis tests were performed to investigate the extent of particle breakage due to cyclic loading. Comparison of the grain-side distribution curves before and after cyclic loading indicated that no noticeable degradation had occurred. However, minor degradation occurring during the first few cycles of loading appears to lead to more stabilize interparticle contacts, and the material reacts in a much stiffer manner to subsequent load application.

Cyclic Plate Load Tests on Lightweight Aggregate Beds

A testing program carried out at the University of New Brunswick, Canada consisted of laboratory plate load tests on beds of lightweight aggregate with or without geogrid reinforcement. The variables studied were relative density of the aggregate and location of the goegrid with respect to the base of the plate.

Materials: ESCS Lightweight aggregate was used in this study. The lightweight aggregate tested had a grain size distribution between 3/4 - #4 (19 and 4.7 mm) with a uniformity coefficient of 1.4. Table 16.9 gives the angle of internal friction data for the lightweight aggregates from two sources, along with the data for limestone aggregate.

Table 16.9. ***Angle of Internal Friction for Coarse Aggregates***

Material	Test Method	Angle of Internal Friction (Ø) Degrees as Determined by Direct Shear (DS) and Triaxial Compression Tests (T)			
		Loose		Compact	
		Ø	**Density**	**Ø**	**Density**
ESCS AGG A	DS	40.5	58	48	66
ESCS AGG B	DS	40.0	52	45.5	58
Limestone	DS	37.0	107	N/A	118
ESCS AGG B	T*	39.5	46	44.5	52

*** Data from triaxial tests, Stoll and Holm (1985)**
Note: Unit weight for loose and compact specimens are the same as for Consolidation tests.

Testing: Plate load tests were performed in a test pit 10’6” x 10’6” x 5’3” inches (3.2 x 3.2 x 1.6 m) deep. The facility is equipped with loading frames, and the reaction beam can be adjusted in the vertical position depending on the thickness of the soil in the test pit. Details of the test setup are shown in Figure 16.19. A standard steel plate 12 inch (300 mm) in diameter was used in all the tests. The loads were applied by a hydraulic ram, and the settlements were monitored using two dial gauges. The date from the dial gauges and the level vial mounted on the plate were used to ensure that plate tilting did not occur during testing.

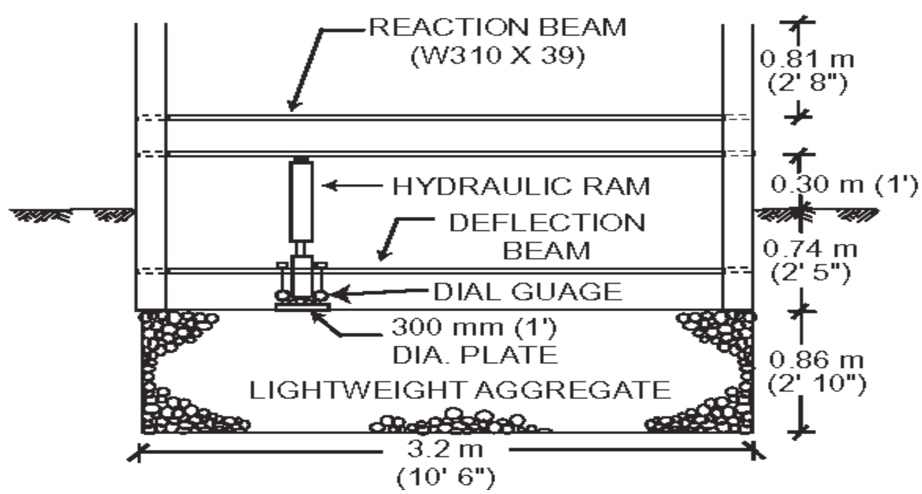


Figure 16.19. ***Setup for plate load test***

In all the tests performed the thickness of the lightweight aggregate was at least 2′10″(900 mm). Loose relative density was achieved by end dumping the aggregate in the test pit. An average dry density of 50 lb/ft³ (800 kg/m³) was achieved when the aggregate bed was prepared by end dumping.

After completion of testing of the loose lightweight aggregate, it was removed from the test pit. A small vibratory plate compactor 21 x 24 inch (530 x 610 mm) plate was then used to compact 6″ (150-mm) -thick lifts of lightweight aggregate (Figure 16.20). Density measurements made after compaction indicated that an average dry density of 59 lb/ft³ (950 kg/m³) was achieved.

When the plate was properly seated, load was applied with the hydraulic ram (Figure 16.17). For loose aggregate beds, the loads were monotonically applied in increments of 225 lb (1 kN) until a settlement of 1/2 inch (12 mm) was achieved. For the compacted aggregate bed, monotonically increasing loads were applied in increments of about 450 to 675 lb (2 to 3 kN) until the place settlement reached 1/2 inch (12 mm). Load increments for reinforced aggregate varied from 900-1350 lb (4 to 6 kN) during the monotonic application of loads. Irrespective of the magnitude of the load increment, each load increment was maintained until the rate of settlement was less than .0008 inch (0.02 mm) /min for a minimum of three successive minutes.

Figure 16.20. ***Test Pit with BOMAG Vibratory compaction (21" x 24" Plate)***

The choice of 1/2 inch (12-mm) settlement as the maximum settlement was adopted on the basis of the ASTM standard for plate load testing (ASTM D 1195-64). However, load cycling before reaching 1/2 inch (12-mm) settlement was not carried out as recommended in ASTM D 1195-65, because the primary objective of the study was to determine the coefficient of subgrade reaction for monotonic loading. The other reason for adopting the 1/2 inch (12-mm) settlement criterion and not cycling the load before this much settlement occurred is found in the work by DeBeer, which concluded that the settlement at the onset of bearing capacity failure of granular soils with high relative density is on the order of 5 percent of the width of the loaded area.

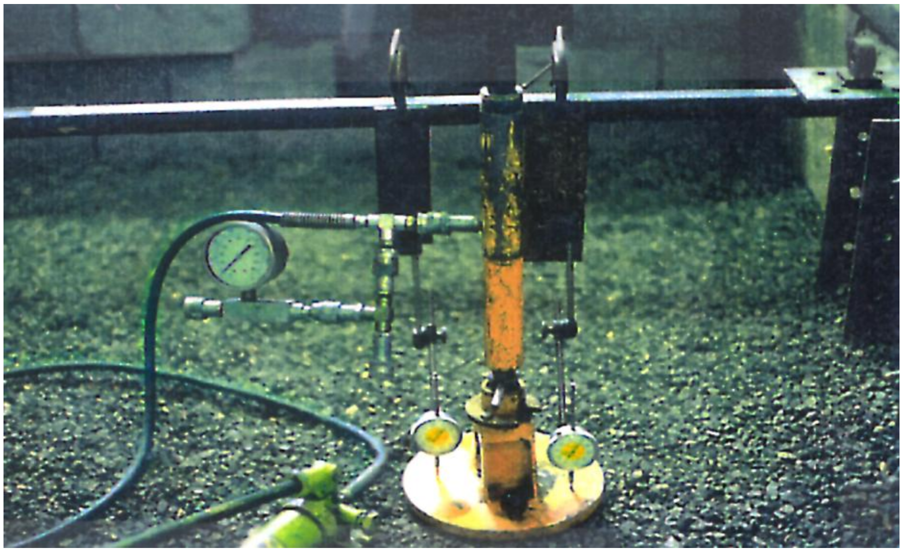


Figure 16.21. ***10-ton hydraulic ram jack, 12" dia. Plate and gages***

In all the tests performed, cyclic loads were applied after the monotonic load was applied to achieve a 1/2 inch (12-mm) settlement. In each case the maximum load corresponding to 1/2 inch (12-mm) settlement was applied six to eight times to study the behavior under cyclic loading. Each test was done at least twice to ensure that data and trends were reproducible.

Results: Plate load test results for unreinforced lightweight aggregate are presented in Figure 16.21 for compact and loose beds. The bearing stress for 1/2 inch (12-mm) settlement increased from 16.8 to 66.2 lb/in² (116 kPa to 456 kPa) because of moderate compaction. The values of coefficient of vertical subgrade reaction were determined from the slope of the bearing stress-versus-settlement data obtained during the monotonic loading. The results are given in Table 16.10. Typically, values of coefficient of vertical subgrade reaction of 30 pci (8 MN/m³) loose and 140 pci (38 MN/m³) compact are used for normalweight coarse grained soils. Thus, the plate loading tests confirm that the behavior of tested coarse lightweight aggregate is similar to that of normalweight aggregates.

Table 16.10. Coefficient of Vertical subgrade Reaction for Coarse Lightweight Aggregate

Test No.	Plate Diameter mm	Relative Density	Coefficient of Subgrade Reaction, MN/m^3
1	300	Loose	9
2	300	Loose	10
3	300	Compact	42
4	300	Compact	38

To convert:

- kPa to lb/in^2 multiply by .145
- MN/m^3 to pci multiply by 3.69
- Mm to inches multiply by .0393
- Kg/m^3 to lb/ft^3 multiply by .016

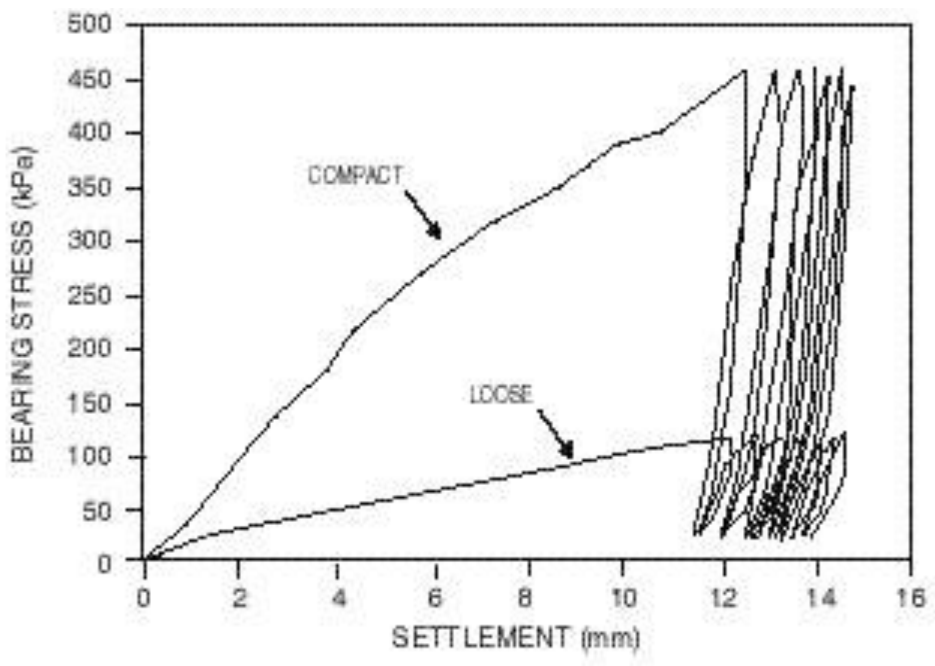


Figure 16.22. ***Effect of relative density on plate settlements***

Conclusions: Results of the preliminary plate load testing program reported in this paper indicate that the coefficient of vertical subgrade reaction values of lightweight aggregates is similar to that of normalweight aggregates used in roadway and engineered fill applications. The inclusion of geogrid as a soil reinforcement enhances the compressibility characteristics of the lightweight aggregate similar to the normalweight aggregate. Even though relatively few tests have been done in this program, the extensive testing done previously at the University of New Brunswick, with the result of the present investigation, indicate that geotechnical behavior of coarse lightweight aggregate is similar to that of normalweight aggregate.

This section was adapted from a paper that was reprinted with permission of the Transportation Research Board, National Research Council, Washington, DC, Transportation Research Record No. 1422; Soils, Geology, and Foundations; Lightweight Artificial and Waste Materials for Embankments Over Soft Soils, January 1993.

This paper as originally published by TRB used only metric notation. For US customary notation us the following: **1 inch = 25.4 mm; 1 pound = 4.45 N; 1 psi = 6.90 kPa; and, 1 pcf = .0624 kg/m³**

Model Test On Peat Geotextile Lightweight Aggregate System

Considerable research has been done in recent years dealing with the interaction of normalweight aggregate and geotextile overlying soft compressible soils. In some instances, lightweight aggregate is used instead of ordinary aggregate to reduce settlements, and in the bridge abutment areas, to minimize lateral forces and to reduce drag loads on piles.

The paper "*Model Tests on Peat-Geotextile-Lightweight Aggregate System*", (Valsangkar and Holm, see appendix E) reports the results of experimental research (testing equipment is shown in Fig. 16.23) dealing with interaction of lightweight aggregate and geotextiles overlying peat subgrades. Variables investigated include differing aggregate types and densities, thickness of the aggregate layer and geotextile types. The results indicate that for small deflections and initial load application the overall roadbed stiffness is unaffected when lightweight aggregate is used instead of normal weight aggregate.

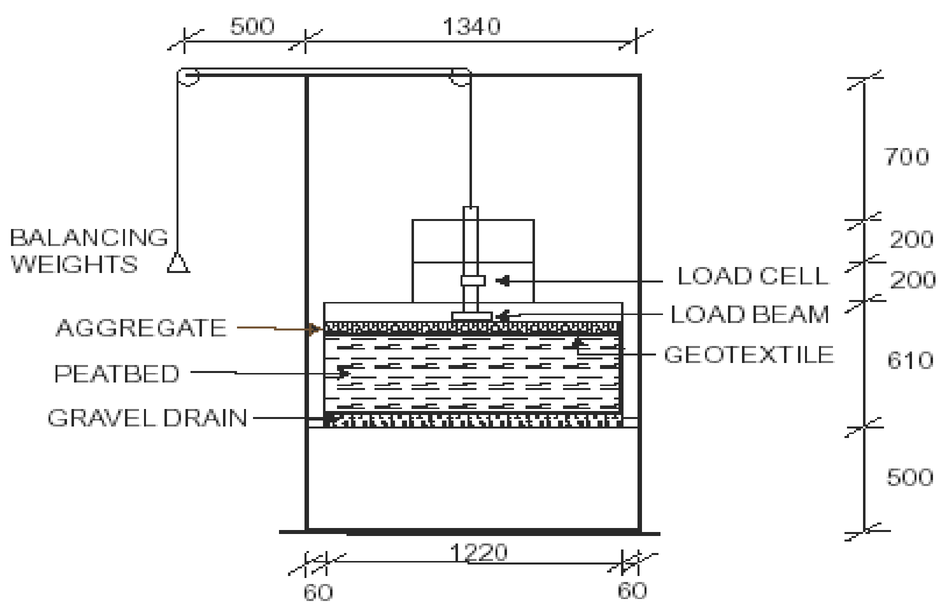


Figure 16.23. ***Small scale model test apparatus (1" = 25.4 mm)***

Permeability of Lightweight Aggregate Backfill

Free draining properties of backfill systems are essential in the prevention of the build-up of hydrostatic pressures. Very high permeability's greater than .39 inch (1 cm) /sec have been measured on samples of ESCS lightweight aggregate tested in several laboratories confirming the fact that granular coarse ESCS are "free draining".

In the paper *"Expanded Clay and Shale Aggregate for Leachate Systems"* presented in the Geo-Environmental Engineer, November 1997, the authors, Bowders et. al. reported the results of permeability tests on four samples of expanded shale and clay. The results are shown below in Table 16.11.

Table 16.11. Permeability (k) Determined Using Hazen Formula, Constant-Head, and Constant Rate-of-Flow Techniques

Permeability	Expanded Shale		Expanded Clay		Leachate Collection Sand
	19 to 6 mm (3/4 in. to no. 4)	10 to 2 mm (3/8 in to no. 10)	9 to 5 mm 3/4 in. to no. 4)	13 to 5 mm (1/2 in to no. 4)	
Hazen Formula					
k (cm/s)	25	6	25	25	0.06
Constant-Head Tests					
k (cm/s) $1 \le i \le 2$	9 to 6	3 to 2	11 to 6	8 to 6	—
k (cm/s) $0.2 \le i \le 0.5$	—	6 to 4	19 to 12	—	—
Constant Rate-of-Flow Tests Δh – 0.7 to 0.3 cm					
k (cm/s)	44	10	39	40	0.02
Constant-Head Tests, Postleachate Immersion					
k (cm/s) $0.2 \le i \le 0.5$	—	2 to 0.2	13 to 8	—	—
Constant-Head Test, under 350 kPa Normal Stress					
k (cm/s) $0.2 \le i \le 0.5$	7 to 3	0.6 to 0.1	12 to 9	6 to 2	0.4 to 0.2

Note: i = hydraulic gradient; Δh = head loss
Due to sidewall leakage during the test, these values are not representative;
Lower hydraulic conductivities are to be expected.

The authors concluded, "A primary issue for any potential leachate collection material is whether the aggregate has a sufficiently high hydraulic conductivity. We measure hydraulic conductivities of 6 cm/s – 40 cm/s when the aggregate was not subjected to external compressive stress, and 0.1 cm/s – 12 cm/s under a 350 kPa (50 psi) compressive stress. Hydraulically, these aggregates, even under a large compressive stress, should perform well in leachate collection systems applications

Thermal Properties of Lightweight Fills

For more than eight decades, design professionals have used lightweight concrete masonry and lightweight structural concrete on building facades to reduce energy losses through exterior walls. It is well demonstrated that the thermal resistance of lightweight concrete is considerably higher than that of ordinary concrete, and this relationship extends to aggregates in the loose state.

Structural lightweight aggregate has been effectively used to surround high-temperature pipelines to lower heat loss. Long-term, high-temperature stability characteristics can be maintained by aggregates that have already been exposed to temperatures of 2012º F (1100° C) during the production process. Other applications have included placing lightweight aggregate beneath heated oil processing plants to reduce heat flow to the supporting soils.

Thermal Conductivity: The following information on Thermal Conductivity Table 16.12 is provided to assist engineers when designing thermal sensitive projects, i.e. frost protection for underground water lines, insulation around or under thermal sensitive vessels, etc.

Moisture has a significant effect on thermal conductivity of granular insulating fills. The thermal conductivity increases about 4% per one-percent moisture for expanded shale, clay and slate lightweight aggregate and increases 7-9% per one-percent moisture for natural sand and gravel.

The practical in-place "k" values for insulating fills depends on the equilibrium moisture content of the fill, which varies depending on the environmental conditions. Where protected conditions exist like core insulation inside concrete masonry units or fills protected by water proof membranes, a "k" value multiplying factor of 1.1 to 1.2 is commonly used. Where unprotected conditions exist like in large geotechnical fills or insulation around underground utility line, a multiplying factor of 1.8 to 1.9 is commonly used.

Table 16.12. Weight and Thermal Conductivity values for Expanded Shale, Clay & Slate Lightweight Aggregate

Dry Density lb/ft^3	Thermal Conductivity, k[1], Btu/hr ft^2 (deg F/in.)	(W/m deg C)
Coarse 3/4 " or 1/2 " to #4[2]		
20	.68	(0.097)
30	.83	(0.119)
40	.93	(0.141)
50	1.13	(0.163)
60	1.29	(0.185)
70	1.44	(0.207)
Natural Granular Fill (Sand with clay and gravel)		
110	7.5 – 8.5	(1.2 – 1.3)
120	9 – 12	(1.35 – 1.7)
130	11 – 15	(1.6 – 2.2)
140	13.5 – 20	(1.9 – 3.0)
160	21 – 35	(2.6 – 5.0)

1. K values were taken from *"The Thermo-physical Properties of Masonry and its Constituents, Part 1, Thermal Conductivity of Masonry Materials",* by Rudolph C. Valore, Jr.
2. The K values for Fine or Coarse/fine blend averages 6% lower.

16.4 APPLICATIONS OF LIGHTWEIGHT AGGREGATE FILLS

Lightweight Aggregate Fills at Waterfront Structures

A classic example of how unusable river front was reclaimed and large industrial site extended by the use of sheet piles and lightweight aggregate fill is shown.

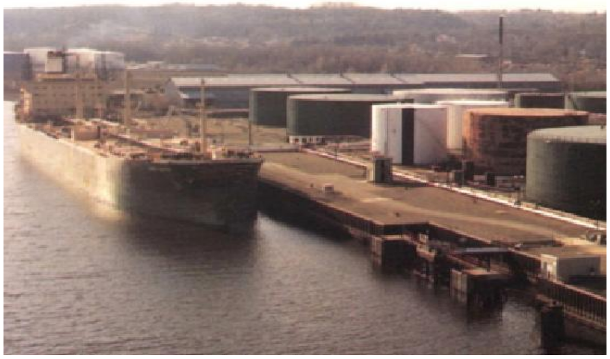
WATERFRONT STRUCTURES

- ❖ Allows economical modification to marine terminals
- ❖ Allows increased dock side draft
- ❖ Reduces Lateral Thrust/Bending moments
- ❖ Allows free drainage and control of in-place density

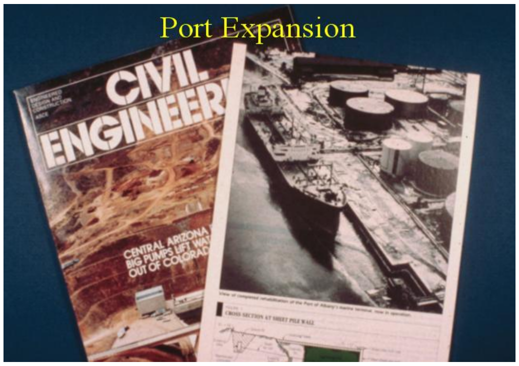
Port of Albany

Lightweight aggregate fill specifications for this project required rotary kiln expanded shale to have a controlled coarse aggregate grading and laboratory test certification of an angle of internal friction greater than 40 degrees. No constructability problems were experienced by the contractor while transporting, placing and compacting the lightweight soil fill. Peak shipments were more than 1,000 tons per day without any logistical difficulties. The material was trucked to the point of deposit at the job site and distributed by front-end loaders. This project used approximately 27,000 yds^3 (20,000 m^3) of compacted lightweight and resulted in overall savings by reducing sizes of sheet piling and lowering costs associated with the anchor system.

Modifications to the Port of Albany marine Terminal reclaimed an area of approximately 1,500 x 80 ft. in and unstable slope area and provided increased dockside draft to permit service by large oil tankers. Lightweight aggregate backfill minimized lateral earth pressures, while also reducing overburden pressures on the sensitive silts. Transportation, placement and compaction of the lightweight aggregate soil fill was not weather sensitive and was readily accomplished in a minimum time frame and without logistic difficulties.

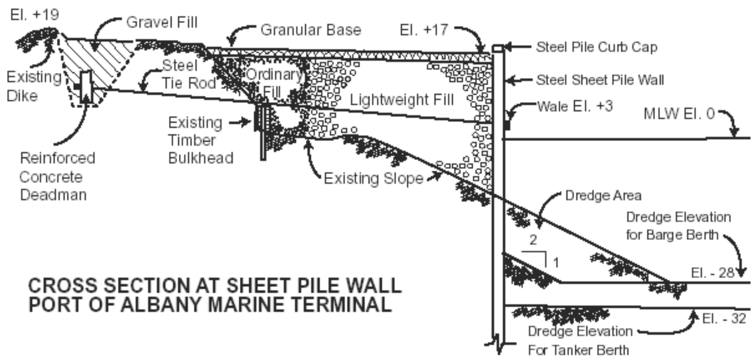


Port of Albany marine Terminal Expansion, Albany, New York.
Engineer: Childs Engineering, Inc., Medfield, Mass.



The reclamation of useable space, to extend the property of this
Major petroleum tank farm was demonstrated in this project
Which was reported on Civil Engineering Magazine.

To permit ocean going tankers to dock very long sheet
Piling was driven into a soft seam of soil.



Cross Section At Sheet Pile Wall
Port of Albany Marine Terminal

At times, more than 50 truck loads of ESCS were delivered daily,
Leveled and compacted by rubber tired loaders in lifts of
Approximately 12 to 18 inches. The process is simple, not limited
By weather and comparable to the use of ordinary granular aggregates.

...The project was finished on time and within budget,
without any unusual construction procedure.

Lightweight Aggregate Fill Behind Retaining Walls

Bulkheads and Retaining Walls
The use of lightweight Aggregate fill

- ❖ Reduces soil thrust as well as bending moments
- ❖ Reduces forces against abutment and end slope
- ❖ Allows free drainage
- ❖ Improves embankment stability

Retaining Wall Backfill, Providence Rhode Island
Engineer: C.E. Maguire Engineers, Mansfield, Mass.

Rhode Island State House at Providence River

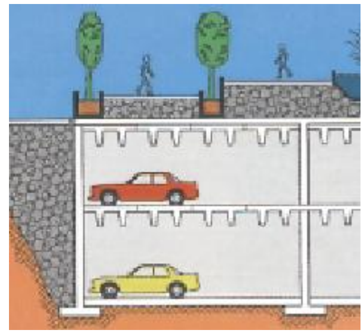
This project involved the construction of a retaining wall behind the Rhode Island State House at the Providence River. The weight of the entire project, including the wall, the backfill, and a future roadway at the top of the wall, was quite significant. With the area's soft clay strata, there were engineering concerns that too much weight might force the existing bulkhead toward the river. The use of approximately 3,600 cubic yards of lightweight aggregate fill reduced the total project weight so dramatically that the probability of deep seated bulkhead failure was virtually eliminated.

Lightweight Aggregate Fill on Elevated Structures

Elevated Structures The use of lightweight Aggregate fill

Landscape & Plaza Fills

- Minimizes dead loads
- Free draining helps minimize hydrostatic potential
- More plants and levels can be added
- Easy to transport and install



Barney Allis Plaza, Kansas City, Missouri
Architect/Engineer: Marshall & Brown Incorporated

Barney Allis Plaza

6,000 cubic yards of lightweight aggregate (expanded shale) was used as loose granular fill on top of an existing underground parking garage. The material provided subsurface drainage, weight reduction and long term stability. In addition, the lightweight aggregate material established the grade and contour for a plaza area which was built on top of the parking structure. The lightweight aggregate material was graded ASTM C 330 3/4 " x No. 4.

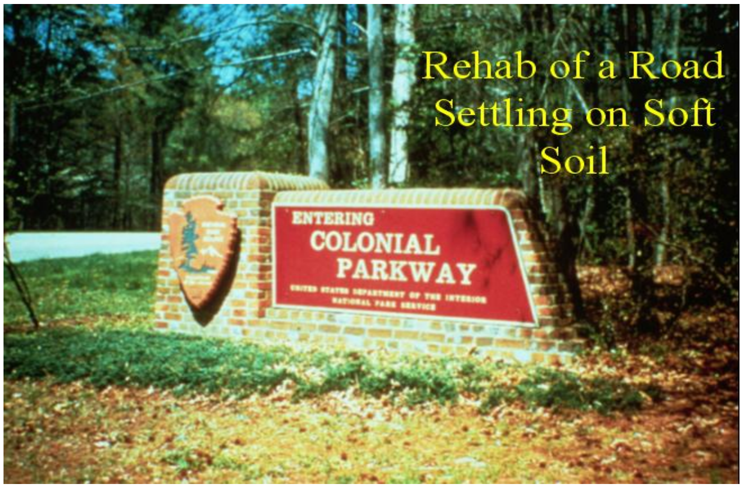
Lightweight Aggregate Fills Over Soft Soils – Load Compensation

Load Compensation for Sinking Road Bed, Colonial Parkway, VA.

In numerous location through North America, design of pavements resting on soft soils has been facilitates by "load compensation" replacement of heavy soils with a free-draining structural lightweight aggregate with low density and high stability. Replacing existing heavy soil with lightweight aggregate permits raising elevations to necessary levels without providing any further surcharge loads to the lower level soils.

Rehabilitation of Colonial Parkway near Williamsburg, Virginia, built alongside the James and York Rivers, provides a representative example of the procedure. Soft marsh soil sections of this roadway had a low load-bearing capacity, and had experienced continuous settlement. The concrete roadway slabs were removed along with the soil beneath to a depth of more than 3 feet. The normalweight soil was then replaced with structural lightweight aggregate with a compacted moist density of less than 60 lb/ft^3 (960 kg/m^3). This provided effective distribution of load to the soft soil layer, load compensation, and side slope stability.

Reconstruction was completed in two stages by first completely rehabilitating in one direction, followed by excavation of the opposing lane with delivery, compaction, and slab construction routinely repeated.



The Colonial Parkway between historic Williamsburg, Yorktown, and Jamestown, VA is constructed over the soft soils in the swamp area between the James and York Rivers.

The settlement of heavy, soft soils particularly at the bridge locations, which were supported on piles, caused an unacceptable road bump.

The heavy soils were removed and replaced with a greater Volume (equal weight) of lightweight aggregate that allowed raising the grade level. This project required no special construction equipment and no waiting for insulating concrete formwork.

And compacted by rubber tired traffic to an in place density of less than 60 lb/ft³.

After completion of one lane, the second lane was opened up. This slide clearly demonstrates the in place stability of the Compacted lightweight aggregate previously placed. Because Structural lightweight aggregate is manufactured to stringent Standards, the Angle of internal friction will be assured. [Typically 38 degrees +- 2 loose and > 40 degrees compact

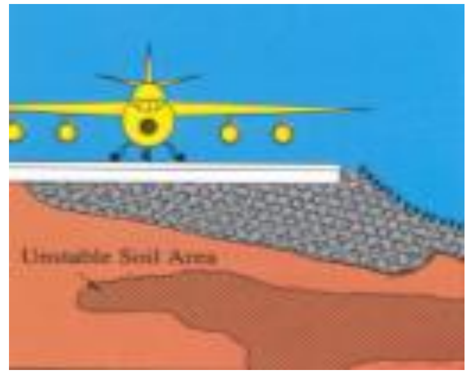
Lightweight Aggregate Fill Reduces Settlement Over Unstable Soils, Morgan City, LA.

Highway embankment fills over unstable soils present particularly difficult problems. Uneven settlement can produce a "roller coaster" ride, as well as significant maintenance problems. The Louisiana Department of Transportation and Development constructed a series of roadway test sections with sand fill 9.5 feet in depth. In one section, 2.5 feet of sand was replaced with 2.5 feet of lightweight aggregate fill. The reduction in weight, coupled with the increase in long term stability provided by the lightweight aggregate's high angle of internal friction, reduced settlement 40% to 60% as compared to the all-sand fill. Considerable savings in highway maintenance, repairs and replacement can be realized if differential settlement is reduced.

Embankment Fill, Louisiana DOT D Test Project
Morgan City, Louisiana

Lightweight Fill For Airport Runway Repair, Norfolk, VA.

- ❖ Allows otherwise un-useable land to be reclaimed and developed
- ❖ Design elevations are achieved with low fill weight
- ❖ Low fill weight increases slope stability
- ❖ Controlled gradings assure uniform and consistent in-place density
- ❖ Long-term settlement is controlled and reduced
- ❖ Controlled fill allows uniform load distribution



Runway Repair, Norfolk Naval air Station, Norfolk, Virginia
Engineer: Patton, Harris, Rust & Associates

Much of this facility was built on marsh land. Poor soil conditions and intense traffic loads produced differential settlements and "alligator" cracking of the taxiway after only 3 years. High soil stability and relief from overburden pressures were provided by substituting compacted lightweight aggregate for heavy, unstable soil to a depth of 4 feet. Lightweight aggregate material was placed at 6 inch lifts and hand compacted with a vibratory plate. Field compaction and projected yields were monitored using a nuclear densometer. The compacted base was then paved and air traffic restored in a timely manner. Differential settlement was economically solved.

Lightweight Aggregate Backfill for Reduced Settlement of Levees

Reduced submerged density will contribute too significantly reduced settlement as well as lower maintenance costs on levee structures built over soft soils.

Lightweight Aggregate Fills For Bridge Applications

Charter Oak Project

The following data has been excerpted from "*Lightweight Fill Solutions to Settlement and Stability Problems on Charter Oak Bridge Project, Hartford, Connecticut,*" by John P. Dugan, Jr., Halley & Aldrich, presented to the Transportation Research Meeting and reported in TRB No. 1422, TRB, Washington, DC 1993.

Project Description: The new Charter Oak Bridge, which links Hartford and East Hartford, Connecticut, was opened to traffic in August 1991, 72 months from the start of design and 40 months from the start of construction. The 6-lane, 1,037-m (3,400-ft.) -long, $90 million multigirder steel structure, built 61 m (200 ft.) south of the old bridge, carries US Route 5 and State Route 15 over the Connecticut River and its flood plain. The project included extensive construction of approach roads and bridges, valued at $110 million.

Lightweight Fill: For this project the following ESCS grading was specified:

Square Mesh Sieve Size	*Percent Passing by Weight*
1 in. (25.4 mm)	100
3/4 in. (19.0 mm)	80 - 100
3/8 in. (9.5 mm)	10 - 50
No.4	0 – 15

For design, a unit weight of 60 lb/ft³ (961 kg/m3) and an angle of internal friction of 40 degrees were used.
The lightweight fill was placed in 2 ft. (0.61 m) -thick lifts and compacted with four passes of a relatively light 5 ton (4.5 Mg) vibratory roller operating in vibratory mode.

Sub-Surface Conditions: The site is in the floodplain of the Connecticut River. Sub-surface conditions, in the order of increased depth, are:

Embankment Stabilization: If constructed of earthen material 125 lb/ft³ (2,002 kg/m³), the maximum 46 ft. (14.0 m) high embankment for the Charter Oak Bridge's east approach would not have an acceptable safety factor against slope instability. The safety factor against slope failure toward the adjacent Hockanum River, using earth fill, was estimated to be only 1.0 to 1.1 (Fig. 16.20).

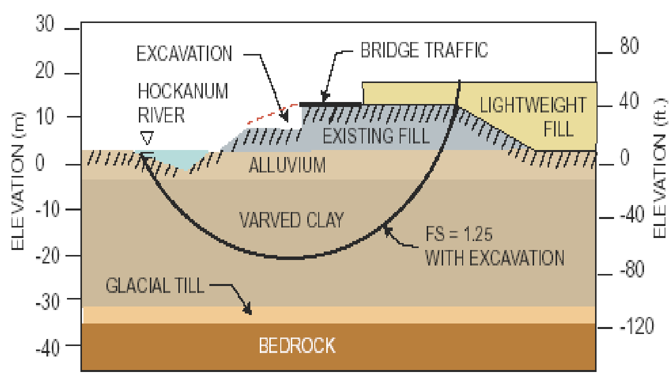
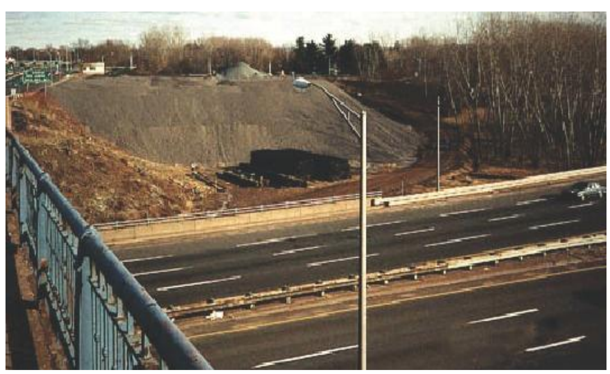


Figure 16.20. ***Slope stability for east abutment. Final conditions with lightweight fill***

.

Many stabilization alternatives were considered. A toe berm placed in the river was the most economical but was rejected to avoid delays that would occur because of the time required to obtain environmental permits. Therefore, it was decided to construct the embankment of lightweight fill. The 82,000 yds. (52,730 m) of lightweight fill is one of the largest quantities of lightweight fill placed for one project in the United States. Lightweight fill significantly reduced stress in the weak varved clay. Even so, it was necessary to excavate a portion of the approach fill to the existing bridge to provide the design safety factor of 1.25. The light weight fill's 40 degree angle of internal friction was higher than provided by earth fill, which increased resisting forces along the potential failure plane. Another benefit of the lightweight fill was the significantly reduced settlement, compared with an earth fill. The total settlement, over the first 15 years, of a lightweight embankment was predicted to range from 1.4 to 2.1 ft (0.43 to 0.64 m), compared with estimates of up to 6.5 ft. (1.98 m) for earth fill. Observed settlement at the east abutment over a year is in line with the predicted values. Hence, the surcharge fill and vertical drains that were planned to speed consolidation of an earth fill were unnecessary. Nevertheless, the lightweight technique cost an additional $2 million in construction compared with the more conventional earth fill/berm/surcharge design. [However, this design provided the most timely and cost effective solution to the problem]

Lightweight fill was placed in approach embankments for a replacement bridge to reduce settlements of the adjacent exiting bridge. Project specifications called for an in-place, compacted, moist bulk density of less than 60 lb/ft^3. Results of tests on ½ of steel buckets placed in fill, covered with compacted aggregated, then retrieved and weighed, demonstrated in place compacted moist densities less then specified maximums, and in agreement with a one point proctor test conducted in the lab.



Although, many stabilization alternatives were considered, it was decided to construct embankment using structural lightweight fill. Approximately 82,000 cyds of lightweight fill was used at this bridge abutment location.

Settlement Reduction at Existing Bridge: A part of the overall project was replacement of Route 15 over Main Street in East Hartford, Connecticut, with a new bridge — a single-span structure 183 ft. (55.8 m) wide, at the existing bridge, but extending 70 ft. (21.4 m) north and 25 ft. (7.6 m) south. Plans called for stage construction, with traffic maintained on the existing bridge while the north section of the new bridge was built. Then traffic was carried entirely on the north half of the new bridge while the existing bridge was being demolished and the south half being built. Lightweight fill made it possible to keep the existing bridge in service while the north portion of the new bridge was being built, and to avoid more expensive alternatives to prevent settlement. The existing bridge is supported on spread footings bearing on a sand layer over approximately 140 ft. (42.7 m) of soft varved clay. A recent inspection had reported 3 in. (7.6 cm) settlement of the west abutment and rotation and horizontal movements of both abutments of the single-span bridge. Temporary corrective repairs were planned; however, there was little tolerance for additional deflections. Although the new bridge was designed to be supported on deep end-bearing piles, the 25 ft. (7.6-m) -high approach fills would increase stresses and lead to settlement in the clay beneath the existing bridge. If an earthen embankment was used, predicted bridge settlements ranged from 1/2 to 2 in. (1.3 to 5.1 cm), which were considered intolerable. The project was therefore designed using lightweight fill for portions of the approach embankments within 75 ft. (22.9 m) of the existing bridge. The lightweight fill reduced stress increases in the clay, lowering predicted settlements of the existing bridge to tolerable limits, to approximately half the magnitudes for earth fill. Measured settlements of the two bridge abutments, during the 1 1/2-year period between embankment placement and demolition of the bridge, were 3/4 in. (0.16 cm) and 1 in. (0.22 cm), which are within the range expected for the lightweight fill. The lightweight fill option was significantly less expensive than underpinning the existing bridge and lengthening the new bridge to provide greater distance between the approach fills and the existing structure.

The lightweight fill was placed in 2.0 ft thick lifts and compacted with four passes of relatively light roller. This project also used Structural Lightweight Aggregate Backfill in several different applications including at waterfront structures and over very old but functional brick water tunnels.

Compared with an earth fill a major benefit of the lightweight fill was the significantly reduced settlement. The total settlement, over the first 15 years, of a lightweight fill embankment was predicated to range from 1.4 to 2.1 ft compared with estimates of up to 6.5 ft for earth fill. Considering all applications, more than 100,000 tons of structural lightweight aggregate backfill were used on the project.

Settlement Prevention at Existing Sewer: A 6.5 ft. (2.0-m) -diameter sewer crosses the existing and new bridge alignments between the west abutment and Pier 1. This 60-year-old cast-in-place concrete pipe founded in the loose silty alluvium is underlain by varved clay (Figure 16.21). Preload fill for construction of the bridge, adjacent pile driving, and new alignment of I-91 northbound required up to 20 ft. (6.1 m) of fill over the sewer and would cause settlements in the varved clay and unacceptable movements in this old pipe. The most severe settlement problem was solved by designing a pile-supported bridge to carry I-95 over the sewer pipe. Nevertheless, stress increases in the clay from the adjacent approach fills and the effects of the pile driving were estimated to cause 1 to 2 in. (2.5 to 5.1 cm) of settlement beneath the pipe. To prevent pipe settlement, 5 ft. (1.5 m) of alluvium from above the pipe was replaced with lightweight fill. This decreased the effective stress in the clay below the pipe by approximately 300 lbs/ft² (300 *P*) and counteracted settlement effects from the other sources. No significant pipe settlement was measured.

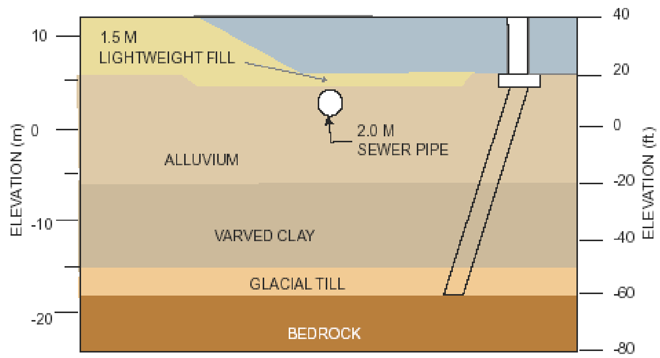


Figure 16.21. ***Lightweight fill above MDC sewer pipe***

Wharf Stabilization: The project included construction of a wharf and boat launch ramp along the west shore of the Connecticut River south of the Charter Oak Bridge. Lightweight fill was designed to provide stability for the wharf's anchored sheet pile bulkhead. The bulkhead retains 25 ft. (7.6 m) of soil above dredge level in the river (Figure 16.22). Stability analyses of circular failure surfaces indicated an unacceptably low factor of safety. As an alternative to anchoring a stiffer wall into underlying bedrock, a layer of lightweight fill was designed to reduce stresses in the weak varved clay and alluvium deposits and increase the factor of safety for overall slope stability to 1.25. The design called for replacing existing soil with a 5 ft. (1.5-m) thickness of lightweight fill. The 8 in. (0.2-m) -thick reinforced concrete wharf slab was placed on a 12 in. (0.3-m) -thick layer of compacted gravel fill over the lightweight fill.

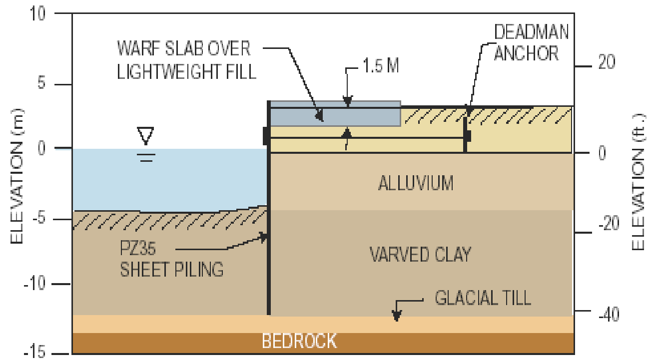


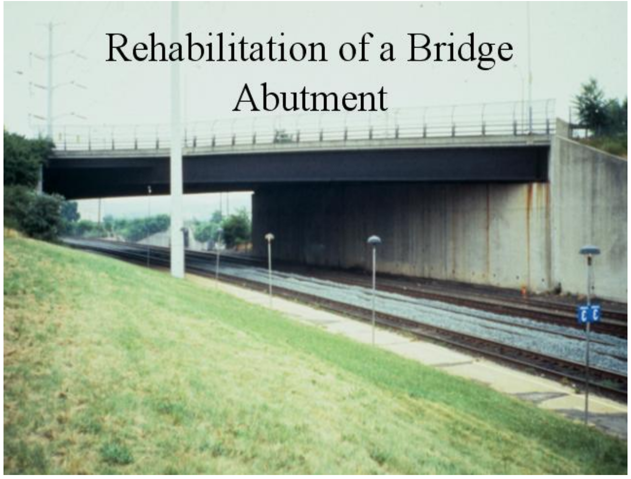
Figure 16.22. ***Lightweight fill placed to improve stability for wharf's sheet pile bulkhead.***

Closing: Design and construction of the Charter Oak Bridge and approaches over soft soil proved to be complex and challenging. Lightweight fill was an invaluable tool to increase slope stability and reduce settlements, both for facilitating the new construction and protecting sensitive existing structures.

References: Smith, A.D. Design of the Charter Oak Bridge Embankment. *Proc., ASCE Specialty Conference on Stability and Performance of Slopes and Embankments*, 1992

Rehabilitation of Existing Bridge Abutments, Duke Street Bridge, VA.

Project name: Duke Street Bridge **Location: Virginia**



Rotation of this bridge abutment of more than 8 inches caused the plate girders on this skewed bridge to move away from their bridge bearing seal.

...which was witnessed by the characteristic "dip at the end of the bridge", caused by the settling of the subgrade.

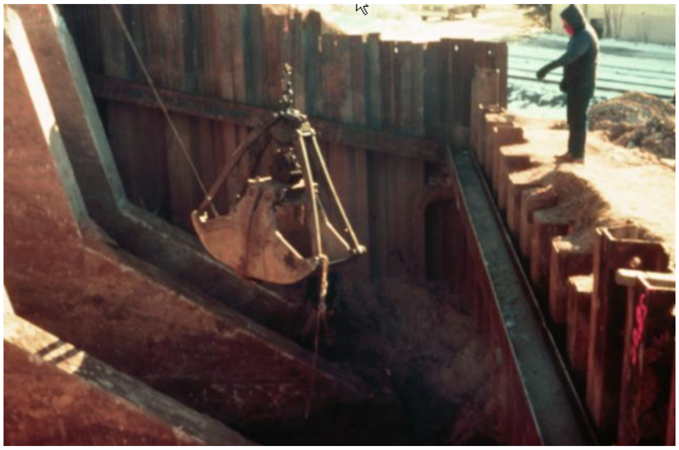
...Relative movement of the different parts of the structure were obvious.

The heavy soil was excavated from behind two of the four lane bridge abutment (maintaining service on this critical connection that split a major city)

And rolled and compacted on both sides of the bridge.

With the remedial lightweight backfill rapidly following the excavation

With continuous progress despite inclement weather. Approximately 20,000 tons of lightweight backfill were placed on this structure, within budget, while meeting the critical completion time requirements.

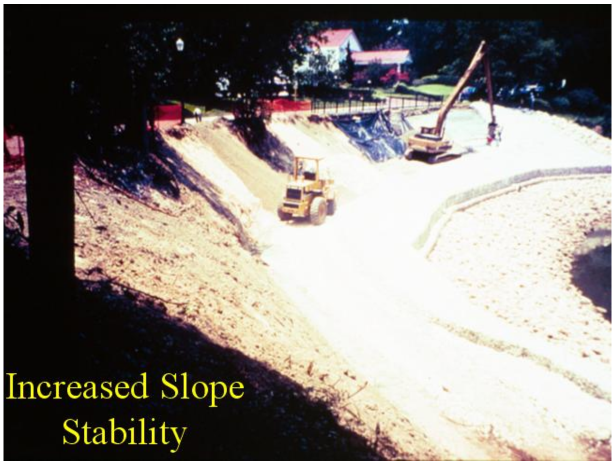


In similar fashion on a Connecticut project lightweight fill was used behind relatively small bridge abutments that required rehabilitation.

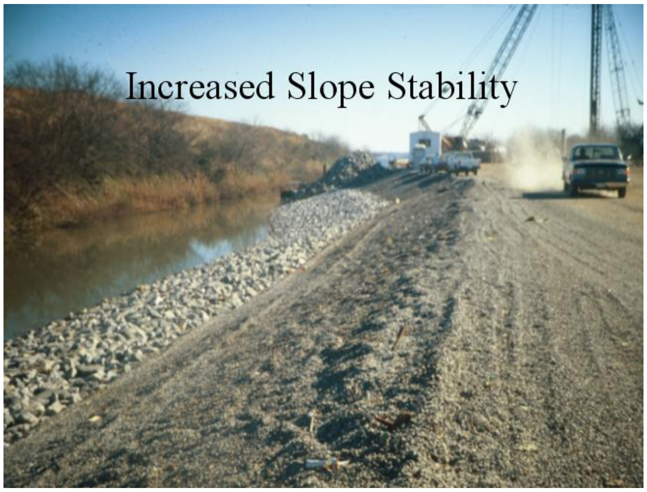
And compacted by hand held small plate tamping equipment.

Lightweight Aggregate Fill For Slope Stability

Improvement of slope stability has been facilitated by lightweight aggregate in a number of projects prone to sliding. Waterside railroad tracks paralleling the Hudson River in the vicinity of West Point, New York, has on several occasions suffered serious misalignment due to major subsurface sliding because of soft clay seams close to grade level. After riverbank soil was excavated by a barge-mounted derrick, lightweight aggregate was substituted and the railroad track bed reconstructed. Reduction of the gravitational force driving the slope failure combined with the predictable lightweight aggregate fill frictional stability provided the remedy for this problem. Troublesome subsoil conditions in many other marine applications in Harbors throughout the United States have also been similarly remedied.



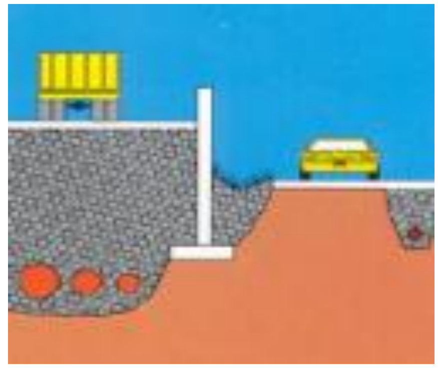
Predictably low densities coupled with and assured angle of internal frictions allowed more efficient use of housing adjacent to the slope in this photo.



Reducing the driving force of the weight of the embankment, coupled with a high angle of internal friction will increase the factor of safety involved in slope stability calculations.

Lightweight Aggregate Backfill Over Buried Pipes

Underground Conduits & Pipelines



- ❖ Reduces dead loads on buried structures
- ❖ Allows construction of higher fills
- ❖ Provides thermal insulation to underground facilities

This outfall pipe from a major institution traversed a soft organic soil that had created numerous settlement related problems. By replacing the heavy marine clay with lightweight aggregate the problem was substantially reduced.

The procedure is simple, economical, and insensitive to inclement weather. No new technology is required, only a backfill that weighs approximately ½ of the original soil.

The lightweight backfill material is easier to handle and encourages increased productivity of on site labor. Another Midwestern project utilized approximately one cubic yard of lightweight backfill per foot of pipeline...for about one mile.

Lightweight Fill for Intermediate Layers

Structure Repair & Rehabilitation

- Reduces dead load on existing structures
- Easy transportation and installation increase productivity
- Precise gradings allow for a uniform and controlled in-place density

Rehabilitation of an industrial workplace built over a former waste site was accomplished by removing several feet of uncontrolled fill and replacing this volume with lightweight fill. Soil settlement of the high organic original fill had caused numerous problems....cracked slabs, machine misalignment, bumpy forklifts, etc.

Because of soft subsoil the slab and subgrade was removed. Lightweight aggregate was installed as the new subgrade. The concrete floor was constructed allowing the industrial facility to be quickly returned to service. Construction was simple, fast, economical, and involved no new special construction techniques.

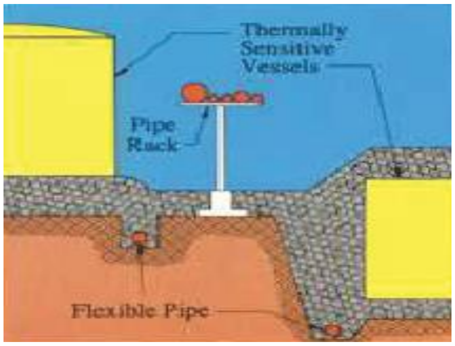
Lightweight Aggregate Backfill Behind Concrete Masonry Segmental Retaining Walls

Lightweight aggregate application to restore the eroded area for the Clifton Avenue property owners, a Keystone Retaining Wall was utilized. This type of wall can be constructed with very little batter (slope). Since the eroded area had to be filled for the restoration, the Keystone Retaining Wall was a logical choice. This mechanically stabilized earth (MSE) wall system used Tensar Geogrid reinforcement and 15,600 cubic yard of Expanded Clay Lightweight Aggregate backfill in the reinforced zone of the wall. Construction of the Keystone Wall on top of the soil-nailed wall presented design challenges since the soil-nailed wall was not designed for the additional weight created by the restored property above. In order to minimize the stress on the lower soil-nailed section, the near vertical Keystone Retaining wall was backfilled with Lightweight Aggregate. Even though the cost for Lightweight Aggregate was greater than conventional fill, lightweight aggregate has a much lower density and a higher degree of internal stability. This combination of physical properties made lightweight aggregate the perfect fill and most economical solution for this challenging site.

The Keystone Retaining Wall, backfilled with Lightweight Aggregate,
Provided an economical and aesthetically pleasing method to
Restore the eroded frontage property.
Natchez, Mississippi, Design/Build Team, Howard Baker Inc.
Ogdon Environmental & Energy
Burns, Cooley, Dennis, U.S. Corps of Engineers

Lightweight Fills for High Thermal Resistance (Below Frost Line)

- Insulating Backfill Substantially reduces ground movement-induced stresses on buried pipes and structures
- Counteracts frost heaving, resists freeze/thaw cycles and highly insulative
- Inert, non-corrosive and stable



Calgary Pipeline, Calgary, Canada
Engineers: City of Calgary, Phildysh & Associates Consultants, Ltd.

Water mains must be installed below the level of frost penetration. In Calgary this requires deep, wide trenches. Such trenches are expensive and often dangerous to workers. The insulating properties of lightweight aggregate fill allowed engineers to reduce trench depth from approximately 11 to 7 ft. (3.3 to 2.1 m). This provided safer working conditions and reliable freeze protection with an environmentally "friendly" material. Lightweight aggregate backfill will also afford easier winter excavation for pipe repair, reduce disruption of water supply and street traffic by decreasing construction time, and eliminate the need for synthetic insulating board and wide trenches. With lightweight aggregate backfill, present and future savings in capital costs alone are expected to be substantial.

Lightweight Aggregate Backfill Provides Free Draining for Leachate In Waste Land Fill

Landfill Drainage

- ❖ Inert; High chemical stability
- ❖ Reduces dead loads on pipes
- ❖ Allows free drainage of leachate/water
- ❖ Acid insoluble

To assure long term resistance to leachate acids that would decompose calcium based aggregates, lightweight aggregate was used to provide predictably high hydraulic conductivity and lower loads to the waste fill.

16.5 ECONOMIC IMPLICATIONS

An economic solution provided by a design that calls for a specially manufactured lightweight aggregate requires some elaboration. In many geographical areas, structural lightweight aggregates are sold on the basis of a price per ton, FOB the plant. However, the contractor responsible for the construction of the project needs in-place total cost on the compacted material necessary to fill a prescribed volume. To illustrate that point, one may presume that if a lightweight aggregate is available at \$X/ton, FOB the production plant, and trucking costs to the project location is \$Y/ton, the delivered job site cost will be \$(X + Y)/ton. As mentioned previously, many projects have been supplied with structural lightweight aggregates delivered with a moist loose density of about 48 lb/ft^3 (770 kg/m^3) and compacted to a moist in-place density of about 55 lb/ft^3 less than the typically specified 60 pcf (960 kg/m^3). This would result in an in-place, compacted moist density material cost (not including compaction cost) of

$$\$/yd^3 = [(X + Y) \times 55 \times 27] / 2{,}000$$

Additional Economic Benefits:

- Approximately twice as much volume of lightweight aggregate can be transported per load as compared to normalweight.
- In restricted or commercial areas, cutting the number of trucks by half is environmentally significant.
- Loader or crane bucket volume can be increased to allow faster placement and longer reaches.
- In tight spaces where hand placement and compaction is required, lightweight aggregate is much easier to handle and offers considerable labor savings.

APPENDIX A

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APPENDIX B

- ESCSI Sponsored Geotechnical Electrochemical and Resistivity Testing Report, August 15, 2001, pp. B1 – B8
- Froehling and Robertson Report of Resistivity Tests on Lightweight Aggregate Stockpiles Using ASTM G 57 Four Electrode Method, pp. B9 – B13
- Extract from FHWA – NHI – 00 – 043 Report March 2001, "*Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*", pp. B14 – B16
- ASTM G 57 "*Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method*", pp. B17 – B21
- AASHTO "*Highway Bridges*", section 7.3.6 Structural Backfill Material MSE Walls, pp. B22
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- AASHTO T 290 (1999), "*Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil*", pp. B35 – B43
- AASHTO T 291 (2000), "*Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil*", pp. B44 – B53

ESCSI SPONSORED
GEOTECHNICAL ELECTROCHEMICAL AND RESTIVITY
TESTING REPORT

August 15, 2001

I. INTRODUCTION

This ESCSI coordinated geotechnical electrochemical/restivity testing program was approved at the midyear meeting in Boulder, Colorado (May 2000). The program was primarily conducted at the Law Engineering Laboratory in Herndon Virginia and included eight companies submitting 21 separate samples. One company entered the program later and 4 member companies had already completed similar type testing programs. All participating companies were mailed hard copies of the results of the tests on their submitted samples and were billed directly by Law Engineering.

II. SUMMARY OF TEST RESULTS

- All 21 samples exceeded the AASHTO minimum restivity requirements of 3,000 ohm-cm when tested in accordance with AASHTO test method T 288.
- All 21 samples had less than the AASHTO maximum chloride ion concentration of 100 parts per million when tested in accordance with AASHTO test method T 290
- Eight of the 21 samples tested exceeded the AASHTO maximum sulfate ion concentration of 200 ppm when tested in accordance with the procedures of AASHTO test method T 290.
- Four of the 21 samples tested exceeded the AASHTO maximum pH limit of 10 when tested in accordance with the procedures of AASHTO test method T 289.
- A summary of the test results are shown in Table I

Table I Summary of Test Results

Sample No.	Restivity x 10^3 ohm-cm	@Moisture Content % by Weight*	Chloride (ppm)	Sulfate (ppm)	pH
AASHTO Spec	>3	__	100	200	5-10
FHWA	>3	__	100	200	5-10

If restivity >5 x 10^3 then chloride and sulfates may be waived.

1	5.4	57	10/11	625/219	9.1
1				281/198/181	
2	15.3	83	19	72	8.6
3	25.2	64	12	18	10.5
4	11.7	57	8	170	8.8
5	18.0	66	47	1	9.8
9	21.6	44	76	472	10.6/9.25/9.0
10	18.0	66	10	49	8.8
11	5.4	75	37/24	607/428	8.2
23	14.4	54	17	119	9.9
23a	12.6	43	13/10	30/24	9.5
25	11.7	55	31	143	8.6
28	18.0	71	7	16	8.8
31	18.0	47	28	215	10.6
38	8.1	62	57	910	9.9
38				754/674/640	
40	7.2	62	23	176	8.8
42	18.0	41	51	14	8.5
45	4.0	52	57	550	10.5
50	5.0	54	2	234	9.5
51	13.5	67	36	125	7.8
53	9.0	57	4	323	8.7
15	5.0	56	12	217	9.6

* The AASHTO T 288 test is directed towards soil size particles (minus 10 mesh) and not granular, coarse aggregate. With repeated testing after incrementally adding water, the moisture content reported is at the point of minimum resistivity. At these high moisture contents the sample is a slurry, resembling mud and not a manufactured coarse aggregate

III. ANALYSIS OF TESTING PROCEDURES

Size of sample

Variation in results as shown in Table I, may not be unusual because the sample size is very small (100 – 250 grams). A larger sample size, particularly in testing for restivity, may be more representative of field conditions.

Grading of sample

All of the AASHTO test procedures used above are directed at soil size particles (minus # 10 mesh) and not for a granular coarse aggregate. To compare the effect of aggregate size on test results two samples (No.1 and No. 11) were split and tested in different ways. One was strictly in keeping with AASHTO test procedures and the second was tested using exactly the same procedure except that the grading was a coarse aggregate as delivered (e.g. ½ - #4). It was obvious that the coarse aggregate sample provided significantly less surface area for transfer of ions. The results of the comparison (listed below) show a reasonable correlation between the chloride test results, but a considerable difference in the sulfate values. The difference in sulfate values appears to be directly related to the surface area of the aggregate, with the smaller grading having considerably more surface area.

	Chlorides	Sulfates
ESCS Aggregate Sample 1 minus #10 mesh	10	625
As delivered coarse aggregate (½-#4)	11	219
ESCS Aggregate Sample 11 minus #10 mesh	37	607
As delivered coarse aggregate (½-#4)	24	428

IV. SULFATES: LAB TO LAB REPRODUCABILITY – AASHTO T 290

In order to determine the lab-to-lab repeatability of test results using the same material, samples were taken from the same bags as Sample No. 1 and No. 38 as reported in table 1 and tested at a second independent testing laboratory. This laboratory was instructed to separate the sample and test it using 3 different procedures with the results as shown below.

Test on Sample 1	Sulfates Lab #2	Sulfates Lab #1
A. AASHTO T 290 (minus #10 mesh)	281 ppm	625 ppm
B. AASHTO T 290 (Coarse aggregate as delivered)	198 ppm	219 ppm
C. AASHTO T 290 (Coarse aggregate as delivered adjusted weight)*	181 ppm	___

Test on Sample 38	Sulfates Lab #2	Sulfates Lab #1
A. AASHTO T 290 (minus #10 mesh)	754 ppm	910 ppm
B. AASHTO T 290 (Coarse aggregate as delivered)	674 ppm	___
C. AASHTO T 290 (Coarse aggregate as delivered, adjusted weight) *	640 ppm	___

*Sample "C" used a smaller reduced weight of material, adjusted for specific gravity (i.e. 1.50/2.60 x 100 grams = 58 grams) on the assumption that an equal volume of ESCS used behind a wall would require less weight.

V. EFFECT OF PARTICLE SIZE ON SULFATE RESULTS

Approximately 10 years ago one member company separated out individual screen size particles from one sample and had an independent testing laboratory measure water-soluble sulfate concentration as a function of particle size. The results as listed below support the finding as described in Section III that aggregate size (surface area) has a significant effect on the soluble sulfates measured by the test method.

Aggregate size	Water-soluble sulfate (ppm)
¾"	60
⅜	80
#10	120
#30	280
#200	650

VI. LAB TO LAB REPEATABILITY OF pH – AASHTO T 289

In a similar fashion, two samples from the same bag were tested for pH at both labs.

Lab #1	pH 10.6
Lab #2	pH 9.25

It should be noted that since pH is determined on a logarithmic scale, the pH level of Lab #1 is more than 10 times that reported by Lab #2.

VII. FIELD RESITIVITY TESTING OF ESCS STOCKPILES

Soil restivity tests reported in the Norlite Corporation, geotechnical brochure conducted on full size stockpiles using the procedures of ASTM G 57-95a "*Standard Test Method for Field Measurement of Soil Restivity Using the Wenner Four-Electrode Method*" resulted in restivity values at least one order of magnitude greater than that developed on 4"x6"x1.8" size laboratory samples.

Norlite Data:	Laboratory restivity tests (small sample. Minus #10 mesh)	32,234 ohm cm
	Field test on Norlite stockpile, Four-terminal method	530,000 ohm cm

In order to directly compare the resistivity values obtained by Laboratory tests with those obtained when measuring large stockpiles, Froehling & Robertson, Richmond, VA was commissioned to conduct ASTM G 57 tests on 3 stockpiles of ESCS located in a Richmond, VA readymix concrete plant. The ASTM G 57 procedure is widely used to measure electrical fields in the soils around high voltage transmission lines. These test results confirmed the high values previously arrived at by Norlite corporation.

¾ - #4 Solite @ 6.88% moisture – 447,000 ohm cm

⅜ - #8 Solite @ 4.50% moisture – 548,000 ohm cm

¾ - #4 Solite @ 11.4% absorbed and 2.7 free moisture – 528,000 ohm cm

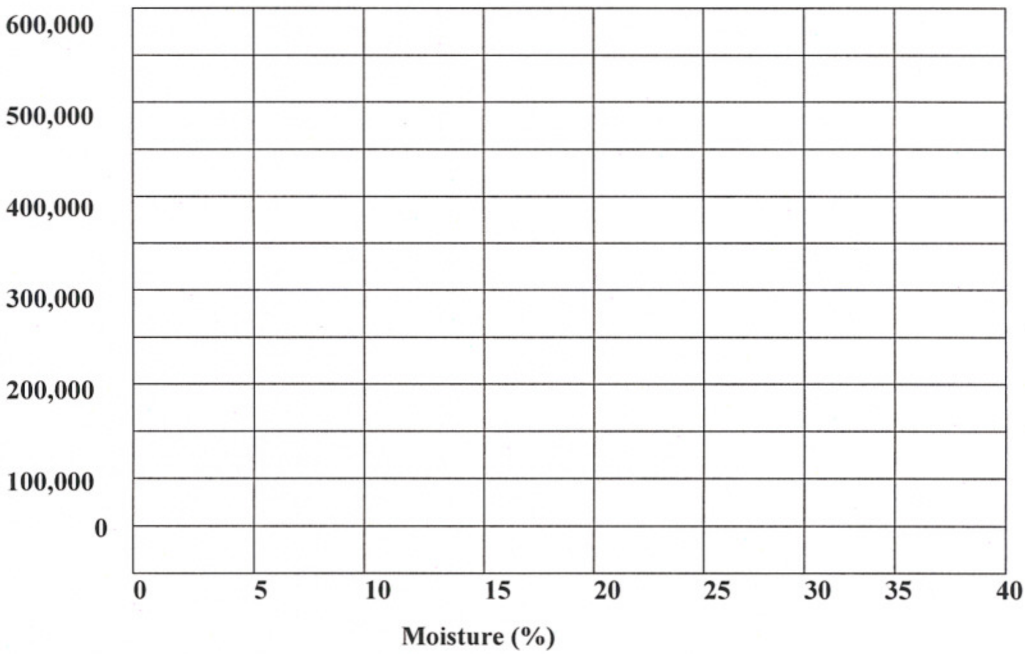
A sample obtained from the ¾ - #4 Solite stockpile was taken to Law Engineering for testing strictly in accordance with the procedures of AASHTO T 288 with the exception the sample was not crushed to minus #10 mesh, with the results shown below in Table 2 and Figure 1.

Table 2

SAMPLE			RESISTIVITY (ohm cm)	MC (%)	pH
ID	DEPTH (ft)	TYPE			
¾ to #4	Stockpile	LWA	62,780,000	2.9	7.2
			564,000	9.9	
			219,450	15.9	
			188,100	21.4	
			163,020	27.8	
			131,670	33.6	
			156,750	39.4	

Figure 1

Resistivity vs. Moisture Content



It appears that when the grading of the test sample is not changed from the stockpile material, that the results of field and lab resistivity measurements are reasonably comparable.

A. Field Stockpile measurements (4 terminal method)

		Resistivity
ASTM G 57-95a	Solite ¾ - #4, 8.88% moisture	447,000 ohm-cm
	Solite ¾ - #4, 11.4% moisture	528,000 ohm-cm

B. Lab measurements (4x6x1.8″ box)

AASHTO T 288	Solite ¾ - #4, 9.9% moisture	564,300 ohm-cm

RECOMMENDATION

1. Considering the range of test results observed in this series of tests it is recommended that for the ESCS materials that did not meet all of the typical AASHTO/FHWA specifications, new tests should be conducted at a local independent testing laboratory.
2. Based upon the fundamentally differing results obtained when the sample is tested using the AASHTO T 288 procedure, or when tested as shipped and placed in actual geotechnical projects, ESCSI will initiate inquiries with appropriate ASTM, AASHTO and transportation research board committees, and propose revisions to existing specifications and procedures.

ATTACHMENTS

Froehling & Robertson testing results of field stockpiles of ESCS, May 7, 2001.

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May 7, 2001

DAILY OBSERVATION REPORT

DATE OF OBSERVATION: April 26, 2001 **RECORD NO:** C60-0092T

ESCSI – Expanded Shale, Clay and Slate Institute
Attn: Mr. Thomas A. Holm, P.E.
7580 Rockfalls Drive
Richmond, Virginia 23225

PROJECT: Ready Mixed Concrete Plant – (3) Three Lightweight Coarse Aggregate Stockpiles

TYPE OF OBSERVATION: Field Resisitivity - Line Traverses and Laboratory Moisture Content Testing

FIELD NOTES

A geotechnical-engineering representative visited the above referenced project site with the ESCSI representative, Mr. Holm, on April 26, 2001. Froehling & Robertson, Inc., completed resistivity testing and laboratory moisture contents for the (3) three designated coarse aggregate stockpiles, as requested. The ESCSI representative, Mr. Holm, requested the site visit.

Our services have been performed in general accordance with the ASTM designations D 6431 – 99 and G 57 – 95a using the Wenner Four-electrode method and locally accepted geotechnical engineering practices.

Froehling & Robertson, Inc., appreciates the opportunity of working with you on this project. If you have any questions regarding any aspect of this report or if we can be of further service to you, please feel free to contact the undersigned.

Very Truly Yours,
Froehling & Robertson, Inc.

Ben A. Uteir, P.E.
CMT Manager

Michael K. Whanger, E.I.T.
Engineering Staff

Enclosure : (3) attachments

HEADQUARTERS: 3015 DUMBARTON ROAD • BOX 27524 • RICHMOND, VA 23261-7524
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SINCE F&R 1881

Soil Resistivity Data Sheet

Site Name: Ready Mixed Plant
Site Address: Bethleham Road, Richmond, VA

Site Number: Solite 3/4" to #4 Stone
Test Date: April 26, 2001
F&R No.: C60-0092T

Weather Conditions: Sunny, Clear, 74 degrees F.

Performed by: MKW/ TAH

R = Measured Resistance Δ = Resistivity

Instrument Settings: Test Current: 2 mA Range: 2-200 W

L (ft)	3	4	6	8	10		
Δ (Ω–cm)	574.5 * R	766 * R	1149 * R	1532 * R	1915 * R		
Area 1 Measured R	998	560	386.00	298.00	173.00	Area 1 Average R	483
Area 1 Calculated Δ	573,351	428,960	443,514	456,536	331,295	Area 1 Average Δ	446,731

Average R for Site =	483	Ω
Average Δ for Site =	446,731	Ω - cm

Model number of test instrument: Det 5/4

Following are the Corresponding Test Sample Locations & Laboratory Moisture Content Determinations, as requested:

Sample No. 1 - 3/4" to #4 stone stockpile - Solite

Total - Moisture Content = **6.88 %**

B-10

SINCE F&R 1881

Soil Resistivity Data Sheet

Site Name: Ready Mixed Plant
Site Address: Bethleham Rd., Richmon

Site Number: Solite 3/8" to No.8 Stone
Test Date: April 26, 2001
F&R No.: C60-0092T

Weather Conditions: Sunny, Clear, 74 degrees F.

Performed by: MKW/TAH

R = Measured Resistance Δ = Resistivity

Instrument Set Test Current: 2 mA Range: 2-200 W

L (ft)	4	6	8	10		
Δ (Ω–cm)	766 * R	1149 * R	1532 * R	1915 * R		
Area 1 Measured R	1435	411.00	235.00	137.00	Area 1 Average R	555
Area 1 Calculated Δ	1,099,210	472,239	360,020	262,355	Area 1 Average Δ	548,456

Average R for	555	Ω
Average Δ for	548,456	Ω - cm

Model number of test instru Det 5/4

Following are the Corresponding Test Sample Locations & Laboratory Moisture Content Determinations, as requested:

Sample No.2 - 3/8" to #8 stone stockpile - Solite

Total - Moisture Content = **4.50 %**

Soil Resistivity Data Sheet

Site Name:	Ready Mixed Plant	Site Number:	Solite-Ready Mix 3/4" to #4
Site Address:	Bethleham Rd. Richmond	Test Date:	April 26, 2001
		F&R No.:	C60-0092T

Weather Conditions: Sunny, Clear, 74 degrees F.

Performed by: MKW/ TAH

R = Measured Resistance   Δ = Resistivity

Instrument Set Test Current: 2 mA   Range: 2-200 W

L (ft)	4	6	8	10		
Δ (Ω–cm)	766 * R	1149 * R	1532 * R	1915 * R		
Area 1 Measured R	875	512.00	311.00	196.00	Area 1 Average R	474
Area 1 Calculated Δ	670,250	588,288	476,452	375,340	Area 1 Average Δ	527,583

Average R for	474	Ω
Average Δ for	527,583	Ω - cm

Model number of test instru Det 5/4

Following are the Corresponding Test Sample Locations & Laboratory Moisture Content Determinations, as requested:

Sample No. **3** - 3/4" to #4 stone stockpile - Solite (Ready Mixed Co. owns)
High Moisture Sprinklered

Total - Moisture Content = **13.18 %** ; Absorbed - Moisture Content = **9.12 %**

Sample No. **4** - 3/4" to #4 stone stockpile - Solite (Ready Mixed Co. owns)
High Moisture Sprinklered

Total - Moisture Content = **9.66 %** ; Absorbed - Moisture Content = **8.38 %**

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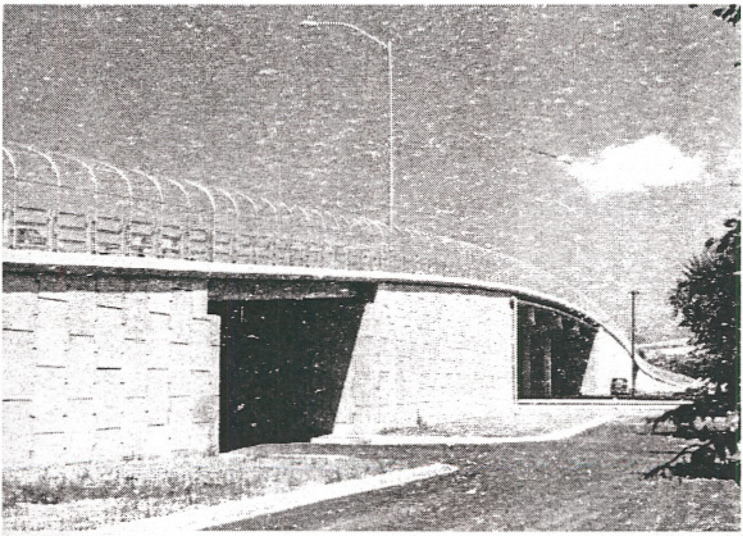
CALLY STABILIZED EARTH WAI
REINFORCED SOIL SLOPES
GN & CONSTRUCTION GUIDELIN



Highway Institute
Technology

B14

reasonable for the backfill requirements listed. A significant economy could again be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value. Likewise, soils outside the gradation range listed should be carefully evaluated and monitored.

c. Retained Fill

The key engineering properties required are strength and unit weight based on evaluation and testing of subsurface data. Friction angles (ϕ) and unit weight (γ_T) may be determined from either drained direct shear tests or consolidated drained triaxial tests. If undisturbed samples cannot be obtained, friction angles may be obtained from in-situ tests or by correlations with index properties. The strength properties are required for the determination of the coefficients of earth pressure used in design. In addition, the position of groundwater levels above the proposed base of construction must be determined in order to plan an appropriate drainage scheme. For most retained fills lower bound frictional strength values of 28 to 30 degrees are reasonable for granular and low plasticity cohesive soils. For highly plastic retained fills (PI>40), even lower values would be indicated and should be evaluated for both drained and undrained conditions.

d. Electrochemical Properties

The design of buried steel elements of MSE structures is predicated on backfills exhibiting minimum or maximum electrochemical index properties and then designing the structure for maximum corrosion rates associated with these properties. These recommended index properties and their corresponding limits are shown in table 6.

Reinforced fill soils must meet the indicated criteria to be qualified for use in MSE construction using steel reinforcements.

Where geosynthetic reinforcements are planned, the limits for electrochemical criteria would vary depending on the polymer. Tentative limits, based on current research are shown in table 7.

Table 6. Recommended limits of electrochemical properties for backfills when using steel reinforcement.

Property	Criteria	Test Method
Resistivity	>3000 ohm-cm	AASHTO T-288-91
pH	>5<10	AASHTO T-289-91
Chlorides	<100 PPM	AASHTO T-291-91
Sulfates	<200 PPM	AASHTO T-290-91
Organic Content	1% max.	AASHTO T-267-86

Table 7. Recommended limits of electrochemical properties for backfills when using geosynthetic reinforcements.

Base Polymer	Property	Criteria	Test Method
Polyester (PET)	pH	>3<9	AASHTO T-289-91
Polyolefin (PP & HDPE)	pH	>3	AASHTO T-289-91

3.5 ESTABLISHMENT OF STRUCTURAL DESIGN PROPERTIES

The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The two most commonly used reinforcement materials, steel and geosynthetics, must be considered separately as follows:

a. Geometric Characteristics

Two types can be considered:

- **Strips, bars, and steel grids**. A layer of steel strips, bars, or grids is characterized by the cross-sectional area, the thickness and perimeter of the reinforcement element, and the center-to-center horizontal distance between elements (for steel grids, an element is considered to be a longitudinal member of the grid that extends into the wall).

- **Geotextiles and geogrids**. A layer of geosynthetic strips is characterized by the width of the strips and the center-to-center horizontal distance between them. The cross-sectional area is not needed, since the strength of a geosynthetic strip is expressed by a tensile force per unit width, rather than by stress. Difficulties in measuring the thickness of these thin and relatively compressible materials preclude reliable estimates of stress.

The coverage ratio R_c is used to relate the force per unit width of discrete reinforcement to the force per unit width required across the entire structure.

$$R_c = b/S_h \tag{8}$$

where: b = the gross width of the strip, sheet or grid; and

S_h = center-to-center horizontal spacing between strips, sheets, or grids

(R_c = 1 in the case of continuous reinforcement, i.e., each reinforcement layer covers the entire horizontal surface of the reinforced soil mass.)

Designation: G 57 – 95a (Reapproved 2001)

Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method[1]

This standard is issued under the fixed designation G 57; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope

1.1 This method covers the equipment and procedures for the field measurement of soil resistivity, both *in situ* and for samples removed from the ground, for use in the control of corrosion of buried structures.

1.2 To convert cm (metric unit) to metre (SI unit), divide by 100.

1.3 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Terminology

2.1 *Definition:*

2.1.1 *resistivity*—the electrical resistance between opposite faces of a unit cube of material; the reciprocal of conductivity. Resistivity is used in preference to conductivity as an expression of the electrical character of soils (and waters) since it is expressed in whole numbers.

2.1.2 Resistivity measurements indicate the relative ability of a medium to carry electrical currents. When a metallic structure is immersed in a conductive medium, the ability of the medium to carry current will influence the magnitude of galvanic currents and cathodic protection currents. The degree of electrode polarization will also affect the size of such currents.

3. Summary of Test Method

3.1 The Wenner four-electrode method requires that four metal electrodes be placed with equal separation in a straight line in the surface of the soil to a depth not exceeding 5 % of the minimum separation of the electrodes. The electrode separation should be selected with consideration of the soil strata of interest. The resulting resistivity measurement represents the average resistivity of a hemisphere of soil of a radius equal to the electrode separation.

3.2 A voltage is impressed between the outer electrodes, causing current to flow, and the voltage drop between the inner electrodes is measured using a sensitive voltmeter. Alternatively, the resistance can be measured directly. The resistivity, ρ, is then:

$$\rho, \Omega\cdot\text{cm} = 2\pi\, aR \; (a \text{ in cm})$$

$$= 191.5\, aR (a \text{ in ft})$$

where:
a = electrode separation, and
R = resistance, Ω.

Using dimensional analysis, the correct unit for resistivity is ohm-centimetre.

3.3 If the current-carrying (outside) electrodes are not spaced at the same interval as the potential-measuring (inside) electrodes, the resistivity, ρ is:

$$\rho, \Omega\cdot\text{cm} = 95.76\, b\, R/\left(1 - \frac{b}{b+a}\right)$$

where:
b = outer electrode spacing, ft,
a = inner electrode spacing, ft, and
R = resistance, Ω.

or:

$$\rho, \Omega\cdot\text{cm} = \pi\, b\, R/\left(1 - \frac{b}{b+a}\right)$$

where:
b = outer electrode spacing, cm,
a = inner electrode spacing, cm, and
R = resistance, Ω.

3.4 For soil contained in a soil box similar to the one shown in Fig. 1, the resistivity, ρ, is:

$$\rho, \Omega\cdot\text{cm} = R\, A/a$$

where:
R = resistance, Ω,
A = cross sectional area of the container perpendicular to the current flow, cm^2, and
a = inner electrode spacing, cm.

NOTE 1—The spacing between the inner electrodes should be measured from the inner edges of the electrode pins, and not from the center of the electrodes.

[1] This method is under the jurisdiction of ASTM Committee G01 on Corrosion of Metals, and is the direct responsibility of Subcommittee G01.10 on Corrosion in Soils.

Current edition approved April 15, 1995. Published June 1995. Originally published as G 57 – 78. Last previous edition G 57 – 95.

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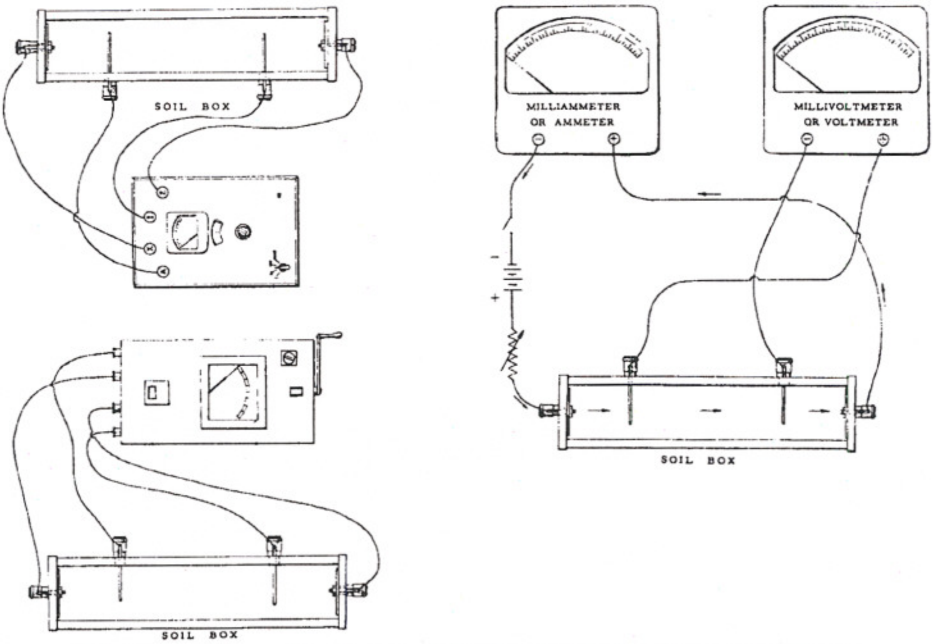

FIG. 1 Typical Connections for Use of Soil Box with Various Types of Instruments

4. Apparatus

4.1 *At-Grade Measurements in situ*:

4.1.1 The equipment required for field resistivity measurements to be taken at grade consists of a current source, a suitable voltmeter, ammeter, or galvanometer, four metal electrodes, and the necessary wiring to make the connections shown in Fig. 2.

4.1.2 *Current Source*—An ac source, usually 97 Hz, is preferred since the use of dc will cause polarization of most metal electrodes, resulting in error. The current can be provided by either a cranked ac generator or a vibrator-equipped dc source. An unaltered dc source can be used if the electrodes are abraded to bright metal before immersion, polarity is regularly reversed during measurement, and measurements are averaged for each polarity.

4.1.3 *Voltmeter*—The voltmeter shall not draw appreciable current from the circuit to avoid polarization effects. A galvanometer type of movement is preferred but an electronic type instrument will yield satisfactory results if the meter input impedance is at least 10 megaohm.

4.1.4 *Electrodes* fabricated from mild steel or martensitic stainless steel 0.475 to 0.635 cm (3⁄16 to 1⁄4 in.) in diameter and 30 to 60 cm (1 to 2 ft) in length are satisfactory for most field measurements. Both materials may require heat treatment so that they are sufficiently rigid to be inserted in dry or gravel soils. The electrodes should be formed with a handle and a

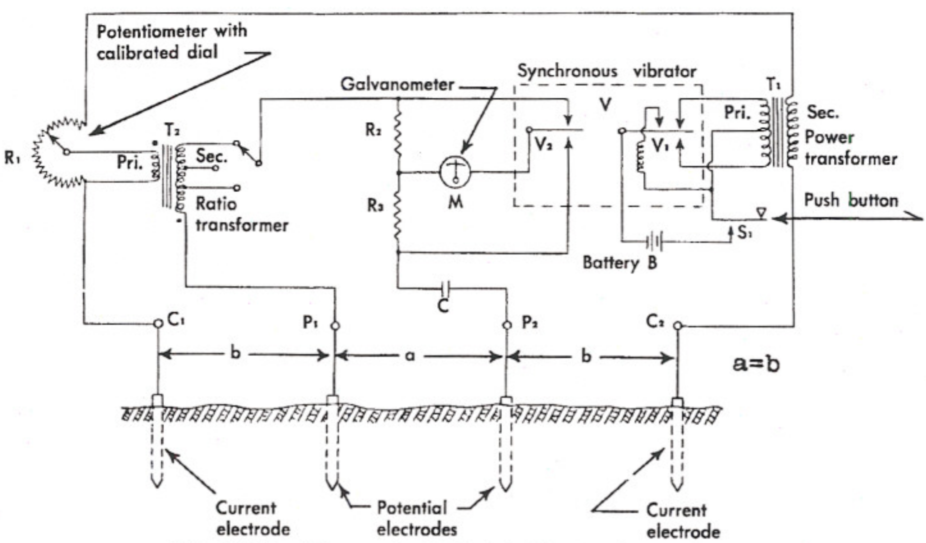

FIG. 2 Wiring Diagram for Typical dc Vibrator-Current Source

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terminal for wire attachment.

4.1.5 *Wiring*, 18 to 22-gage insulated stranded copper wire. Terminals should be of good quality to ensure that low-resistance contact is made at the electrodes and at the meter. Where regular surveys are to be made at fixed electrode spacing, a shielded multiconductor cable can be fabricated with terminals permanently located at the required intervals.

4.2 *Soil Sample Measurement*:

4.2.1 The equipment required for the measurement of the resistivity of soil samples, either in the field or in the laboratory, is identical to that needed for at-grade measurements except that the electrodes are replaced with an inert container containing four permanently mounted electrodes (see Fig. 1).

4.2.2 If the current-carrying (outside) electrodes are not spaced at the same interval as the potential-measuring (inside) electrodes, the resistivity, ρ, is:

$$\rho,\Omega\cdot\text{cm} = 95.76\, b\, R / \left(1 - \frac{b}{b+a}\right)$$

where:
b = outer electrode spacing, ft,
a = inner electrode spacing, ft, and
R = resistance, Ω.

or:

$$\rho,\Omega\cdot\text{cm} = \pi b\, R / \left(1 - \frac{b}{b+a}\right)$$

where:
b = outer electrode spacing, cm
a = inner electrode spacing, cm, and
R = resistance, Ω.

4.2.3 The dimensions of the box can be established so that resistivity is read directly from the voltmeter without further calculation. The box should be readily cleanable to avoid contamination by previous samples.

5. Standardization

5.1 Periodically check the accuracy of resistance meters using a commercial resistance decade box. Meter error should not exceed 5 % over the range of the instrument. If error exceeds this limit, prepare a calibration curve and correct all measurements accordingly. A soil box can be calibrated using solutions of known resistivity. Solutions of sodium chloride and distilled water with resistivities of 1000, 5000, and 10 000 Ω·cm are recommended for this purpose. These solutions should be prepared under laboratory conditions using a commercial conductivity meter, itself calibrated to standard solutions at 20°C (68°F).[2]

6. Field Procedures

6.1 *At-Grade Measurements*:

6.1.1 Select the alignment of the measurement to include uniform topography over the limits of the electrode span. Do not include large nonconductive bodies such as frozen soil, boulders, concrete foundations, etc., which are not representative of the soil of interest, in the electrode span. Conductive structures such as pipes and cables should not be within ½ a of the electrode span unless they are at right angles to the span.

6.1.2 Select electrode spacings with regard to the structure of interest. Since most pipelines are installed at depths of from 1.5 to 4.5 m (5 to 15 ft), electrode spacings of 1.5, 3.0, and 4.5 m (5, 10, and 15 ft) are commonly used. The a spacing should equal the maximum depth of interest. To facilitate field calculation of resistivities, spacings of 1.58, 3.16, and 4.75 m (5.2, 10.4, and 15.6 ft), which result in multiplication factors of 1000, 2000, and 3000, can be used when a d-c vibrator-galvanometer instrument is used.

6.1.3 Impress a voltage across the outer electrodes. Measure the voltage drop across the inner electrodes and record both the current and voltage drop if a separate ammeter and voltmeter are used. Where a resistivity meter is used, read the resistance directly and record.

6.1.4 Make a record of electrode spacing, resistance or amperes and volts, date, time, air temperature, topography, drainage, and indications of contamination to facilitate subsequent interpretation.

6.2 *Soil Sample Measurement*:

6.2.1 Soil samples should be representative of the area of interest where the stratum of interest contains a variety of soil types. It is desirable to sample each type separately. It will also be necessary to prepare a mixed sample. The sample should be reasonably large and thoroughly mixed so that it will be representative. The soil should be well-compacted in layers in the soil box, with air spaces eliminated as far as practicable. Fill the box flush to the top and take measurements as previously detailed (6.1.3). The meter used may limit the upper range of resistivity, which can be measured. In such cases, the resistivity should be recorded as <10 000 Ω·cm, etc.

6.2.2 The measured resistivity will be dependent on the degree of compaction, moisture content, constituent solubility, and temperature. The effect of variations in compaction and moisture content can be reduced by fully saturating the sample before placing it in the box. This can be done by preparing a stiff slurry of the sample, adding only sufficient water to produce a slight amount of surface water, which should be allowed to evaporate before the slurry is remixed and placed in the box. Where available, use ground water from the sample excavation for saturation. Otherwise, use distilled water. If the soil resistivity is expected to be below 10 000 Ω·cm, local tap water can be used without introducing serious error. Some soils absorb moisture slowly and contain constituents that dissolve slowly, and the resistivity may not stabilize for as much as 24 h after saturation. The saturated measurement will provide an approaching minimum resistivity, and can be usefully compared with "as-received" resistivity measurements. Surplus water should not be poured off as this will remove soluble constituents.

6.2.3 Temperature correction will not be required if measurement is made in-the-ditch or immediately after the sample is taken. If samples are retained for subsequent measurement, correct the resistivity if the measurement temperature is substantially different from the ground temperature. Correction

[2] *Handbook of Chemistry and Physics*, 41st ed., The Chemical Rubber Co., p. 2606.

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to 15.5°C (60°F) is recommended if the sample temperature exceeds 21°C (70°F).

$$R_{15.5} = R_T\left(\frac{24.5 + T}{40}\right)$$

where:

T = soil temperature, °C, and

R_T = resistivity at T °C.

A nomograph for this correction is shown in Fig. 3.[3]

7. Planning and Interpretation

7.1 *Planning*:

7.1.1 Surveys may be conducted at regular or random intervals. The former method is suited to graphical presentation and plotting resistivity versus distance, and will identify gradients and abrupt changes in soil condition. The latter method permits precise mathematical treatment, such as cumulative probability analysis. This method permits the determination of the probability of the presence of a soil with a resistivity equal to or greater than a particular value.[4] Where random resistivities are measured over a plant site, these can best be displayed on a plot plan or similar layout. In either case, use pedological surveys in the planning and interpretation of any extensive survey. Measurements could be made in each soil classification under a variety of drainage conditions to simplify survey planning.

7.1.2 If resistivity information is required to assess the requirement for corrosion control measures, it is recommended that the tests be made on a true random basis. Since the number of soil sections that could be inspected is essentially unlimited, infinite population characteristics can be used to simplify statistical treatment. Risk and error must be arbitrarily selected to allow determination of the number of measurements. A risk of 5 % of an error greater than 100 Ω·cm should be suitable for most situations. The error limit should be about 10 % of the anticipated mean resistivity. Where mean or median values cannot be estimated with reasonable accuracy, sequential sampling techniques can be employed.

7.2 *Interpretation*—Interpretation of the results of resistivity surveys will largely depend on the experience of the persons concerned. The mean and median resistivity values will indicate the general corrosivity of the soil. Sharp changes in resistivity with distance and appreciable variations in moisture content and drainage are indicative of local severe conditions.

[3] National Institute of Standards and Technology *Circular No. 579*, p. 157.

[4] Scott, G. N., "Corrosion," *National Association of Corrosion Engineers*, Vol 14, No. 8, August 1958.

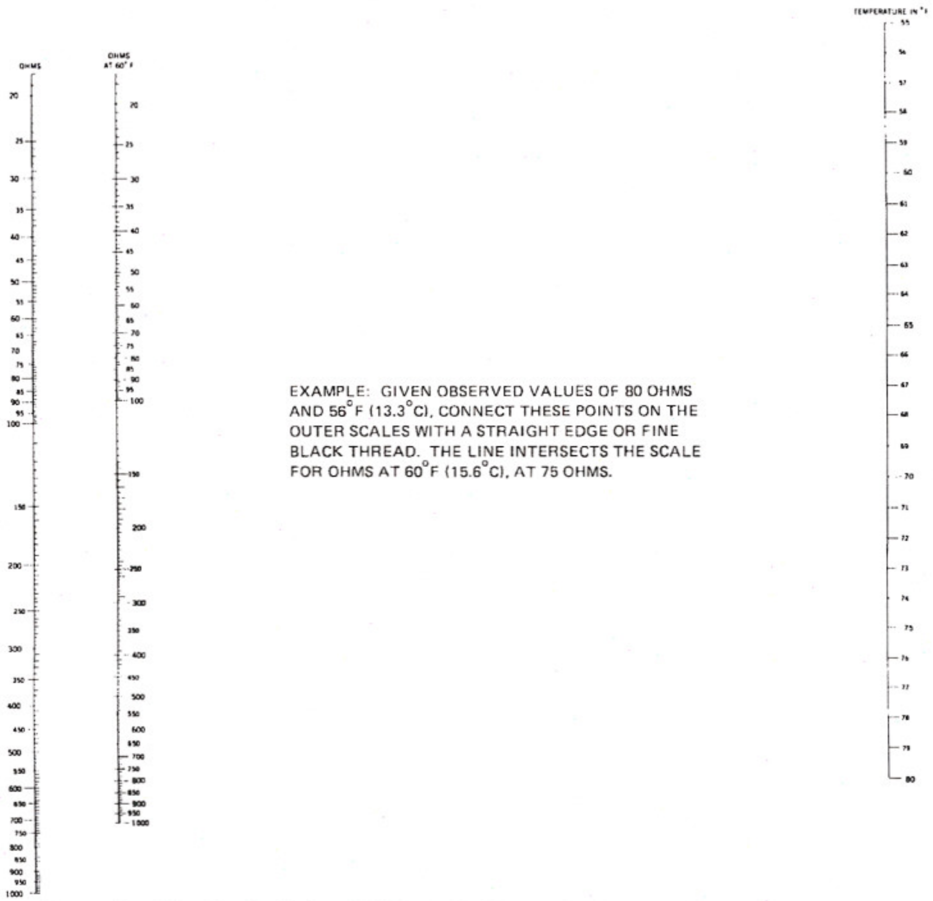

FIG. 3 Nomogram or Conversion Chart for Reducing Soil Paste Resistance in ohms at a Particular Temperature as Measured in the Bureau of Soils Cup, to Resistance at 15.6°C (60°F)

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Cumulative probability plots will indicate the homogeneity of the soil over the area or route and will indicate the probability of severe, moderate, and minimal corrosion of the various construction materials. Available pedological data should be used to facilitate interpretation.

8. General

8.1 It should be recognized that subsurface conditions can vary greatly in a short distance, particularly where other buried structures have been installed. Surface contamination tends to concentrate in existing ditches with surface run-off, appreciably lowering the resistivity below the natural level. Since a pipeline ditch cannot be included in the span of at-grade measurements, soil box samples should be obtained where the opportunity exists. To evaluate contamination effects when a new route is being evaluated, soil samples can be obtained at crossings of existing pipelines, cables, etc, or by intentional sampling using soil augers.

8.2 Other field resistivity measurement techniques and equipment are available. These commonly use two electrodes mounted on a prod that is inserted in the soil-at-grade in an excavation or a driven or bored hole. The two-electrode technique is inherently less accurate than the four-electrode method because of polarization effects, but useful information can be obtained concerning the characteristics of particular strata. More precise procedures may be employed in laboratory investigations and these should be defined in reporting the results. Where resistivity information is included in published information, the measurement techniques used should be defined.

9. Precision and Bias

9.1 *Precision*—The precision of this test method was determined by a statistical evaluation of a multi-participant evaluation with each participant using a different meter. The data from this evaluation are available from ASTM in a research report. A summary of these data is given in Table 1.

TABLE 1 Statistics from Multi-participant Evaluation of Wenner Four Electrode Soil Resistivity Measurement[A]

	Site No. 1	Site No. 3
Electrode spacing, m	6.1	1.5
Average measured resistance	10.9	62.6
Average resistivity, Ω - cm	41 700	59 900
Repeatability standard deviation, Ω - cm	2 300	4 700
Repeatability coefficient variation, Cv, %	5.5	7.8
Reproducibility standard deviation, S, Ω - cm	6 900	10 000
Reproducibility coefficient of variation, Cv, %	16.5	16.6

[A] Evaluation in Chester, New Jersey on May 28, 1993. Triplicate soil resistivity measurements by seven participants each using different meters.

9.1.1 *Repeatability*—Repeatability refers to the variation in results obtained by the same operator with the same equipment and same operating conditions in successive runs. In the case of soil resistivity measurements, the repeatability may be characterized by a coefficient of variation, Cv, representing the repeatability standard deviation divided by the average result and expressed in percent. The multi-participant test program results indicate a repeatability Cv of 6.7 %. The 95 % confidence interval is 2.8 Cv or 18.8 %.

9.1.2 *Reproducibility*—Reproducibility refers to the variation in results that occurs when different operators measure the same soil. In the case of soil resistivity measurements reproducibility may be characterized by a coefficient of variation, Cv, representing the reproducibility standard deviation divided by the average result and expressed in percent. The multi-participant test program results indicate a reproducibility Cv of 16.6 %. The 95 % confidence interval is 2.8 Cv or 46.5 %.

9.2 *Bias*—The procedure in Test Method G 57 for measuring soil resistivity by the Wenner Four Pin Method has no bias because the value of Wenner Four Pin soil resistivity is defined only in terms of this test method.

10. Keywords

10.1 four electrodes method; soil resistivity

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AASHTO SPECS
MSE WALLS
SEC. 7.3.6

7.3.4 Timber

Timber shall conform to the requirements of Section 16, "Timber Structures" and Article 4.2.2, "Timber Piles."

7.3.5 Drainage Elements

7.3.5.1 Pipe and Perforated Pipe

Pipe and perforated pipe shall conform to subsections 708 and 709 of the AASHTO *Guide Specifications for Highway Construction.*

7.3.5.2 Filter Fabric

Filter fabric shall conform to subsection 620 of the AASHTO *Guide Specifications for Highway Construction.*

7.3.5.3 Permeable Material

Permeable material shall conform to subsection 704 of the AASHTO *Guide Specifications for Highway Construction* unless otherwise specified in the contract or the approved working drawings.

7.3.5.4 Geocomposite Drainage Systems

Geocomposite drainage systems shall conform to the requirements specified in the special provisions or the approved working drawings.

7.3.6 Structure Backfill Material

7.3.6.1 General

All structure backfill material shall consist of material free from organic material or other unsuitable material as determined by the Engineer. Gradation will be determined by AASHTO T 27. Grading shall be as follows unless otherwise specified.

Sieve Size	Percent Passing
3″	100
No. 4	35–100
No. 30	20–100
No. 200	0–15

7.3.6.2 Crib and Cellular Walls

Structure backfill material for crib and cellular walls shall be of such character that it will not sift or flow through openings in the wall. For wall heights over 20 feet the following grading shall be required:

Sieve Size	Percent Passing
3″	100
No. 4	25–70
No. 50	5–20
No. 200	0–5

7.3.6.3 Mechanically Stabilized Earth Walls

Structure backfill material for mechanically stabilized earth walls shall conform to the following grading, internal friction angle, and soundness requirements:

Sieve Size	Percent Passing
4″	100
No. 40	0–60
No. 200	0–15*

*Plasticity Index (PI), as determined by AASHTO T 90, shall not exceed 6.

The material shall exhibit an angle of internal friction of not less than 34 degrees, as determined by the standard Direct Shear Test, AASHTO T 236, on the portion finer than the No. 10 sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T 99, Methods C or D (with oversized correction as outlined in Note 7) at optimum moisture content. No testing is required for backfills where 80 percent of sizes are greater than ¾ inch.

The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles.

Additionally, the backfill material shall meet the following electrochemical requirements when steel soil reinforcement is to be used:

pH of 5 to 10
Resistivity not less than 3,000 ohm centimeters
Chlorides not greater than 100 ppm
Sulfates not greater than 200 ppm

7.4 EARTHWORK

7.4.1 Structure Excavation

Structure excavation for earth retaining systems shall conform to the requirements of Section 1, "Structure Excavation and Backfill," and as provided below.

7.4.2 Foundation Treatment

Foundation treatment shall conform to the requirements of Article 1.4.2, "Foundation Preparation and Con-

Standard Method of Test for

Determining Minimum Laboratory Soil Resistivity

AASHTO Designation: T 288-91 (2000)

1. SCOPE

1.1. This test method covers the laboratory determination for the minimum resistivity of a soil.

1.2. The principal use of this test method is to determine a soil's corrosivity and thereby identify the conditions under which the corrosion of metals in soil may be sharply accentuated. This standard is divided into two parts. The first part involves obtaining and preparing the sample to size for testing and the second part describes the test method for determining the minimum laboratory soil resistivity.

1.3. The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 92, Wire-Cloth Sieves for Testing Purposes
- M 231, Weighing Devices Used in the Testing of Material
- R 11, Indicating Which Places of Figures Are to Be Considered Significant in Specified Limiting Values
- T 2, Sampling of Aggregates
- T 248, Reducing Samples of Aggregate to Testing Size

PART I—INITIAL PREPARATION OF TEST SAMPLES

3. SCOPE

3.1. This method covers the dry preparation of soil and soil-aggregate samples, as received from the field, for soil resistivity determination.

3.2. The following applies to all specified limits in this standard: For the purpose of determining conformance with these specifications, an observed value or calculated value shall be rounded off "to the nearest unit" in the last right hand place of figures used in expressing the limiting value, in accordance with R 11.

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4. APPARATUS

4.1. *Balance*—The balance shall have sufficient capacity and be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

4.2. *Drying Apparatus*—Any suitable device capable of drying samples at a temperature not exceeding 60°C (140°F).

4.3. *Sieves*—A series of the following sizes: 6.3 mm ($^1/_4$ in.), 4.75 mm (No. 4), 2.00 mm (No. 10) and pan as required for preparing the sample for the minimum soil resistivity test. The sieves shall conform to M 92, Sieves for Testing Purposes (Note 1).

4.4. *Pulverizing Apparatus*—Either a mortar and rubber-covered pestle or any device suitable for breaking up the aggregations of soil particles without reducing the size of the individual grains of soil (Note 2).

4.5. *Sample Splitter*—A suitable riffle sampler or sample splitter for proportional splitting of the sample and capable of obtaining representative portions of the sample without appreciable loss of fines. The width of the container used to feed the riffle sampler splitter should be equal to the total combined width of the riffle chutes. Proportional splitting of the sample on a canvas cloth is also permitted.

Note 1—The sieve sizes which have an opening size of 6.3 mm ($^1/_4$ in.) or larger shall conform to the requirements specified in M 92 excluding column No. 7. This exclusion permits the use of heavier screens in non-standard frames which are larger than the 203.2 mm (8 in.) round frames.

Note 2—Other types of apparatus are satisfactory if the aggregations of soil particles are broken up without reducing the size of the individual grains.

5. SAMPLE SIZE

5.1. The amount of soil material required to perform the minimum soil resistivity test is as follows:

Test	Approx Mass (g)	Sieve Size Finer Than:
Resistivity	1500	2.00 mm (No. 10)

6. INITIAL PREPARATION OF TEST SAMPLES

6.1. The sample as received from the field shall be dried in air or a drying apparatus not exceeding 60°C (140°F). A representative test sample of the amount required to perform the minimum soil resistivity test shall then be obtained with the sampler, or by splitting or quartering. The aggregations of soil particles shall then be broken up in the pulverizing apparatus until the aggregation of soil particles is separated into individual grains in such a way as to avoid reducing the natural size of the individual particles (Note 3).

Note 3—Samples dried in an oven or other drying apparatus at a temperature not exceeding 60°C (140°F) are considered to be air dried.

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6.2. The portion of the dried sample selected for minimum soil resistivity testing shall be separated into fractions by one of the following methods:

6.2.1. *Alternate Method Using 2.00 mm (No. 10) Sieve*—The dried sample shall be separated into two fractions using a 2.00 mm sieve. The fraction retained on the sieve shall be ground with a pulverizing apparatus until the aggregations of the soil particles are broken into separate grains. The ground soil shall then be separated into two fractions using the 2.00-mm sieve.

6.2.2. *Alternate Method Using 4.75-mm and 2.00-mm (No. 4 and No. 10) Sieves*—The dried sample shall first be separated into two fractions using a 4.75-mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 4.75-mm sieve. The fraction passing the 4.75-mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00 mm sieve, and processed as in Section 6.2.1.

6.2.3. *Alternate Method Using 6.3 mm and 2.00 mm ($^1/_4$ in. and No. 10) Sieves*—The dried sample shall first be separated into two fractions using a 6.3 mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 6.3 mm sieve. The fraction passing the 6.3 mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00-mm sieve, and processed as in Section 6.2.1.

PART 2—MINIMUM SOIL RESISTIVITY DETERMINATION

7. SCOPE

7.1. This method covers the laboratory procedure for determining the minimum resistivity of soil samples. The values obtained from this method are relatable to the corrosion potential that a soil may exhibit.

8. APPARATUS AND MATERIALS

8.1. *Resistivity Meter*—An alternating current (AC) meter or a 12 volt direct current (DC) meter utilizing a Wien Bridge (AC bridge) with a phase sensitive detector and a square wave inverter that produces a nominal alternating signal at 97 Hz. (Note 4).

8.2. 100, 200, 500, and 900 ohm resistors with a 1 percent tolerance.

8.3. *Soil Box*—See Figure 1 and Figure 2.

8.4. 2.00-mm (No. 10) sieve conforming to the requirements of M 92.

8.5. Mixing Pans (non-corrosive; e.g., stainless steel, plastic, etc.).

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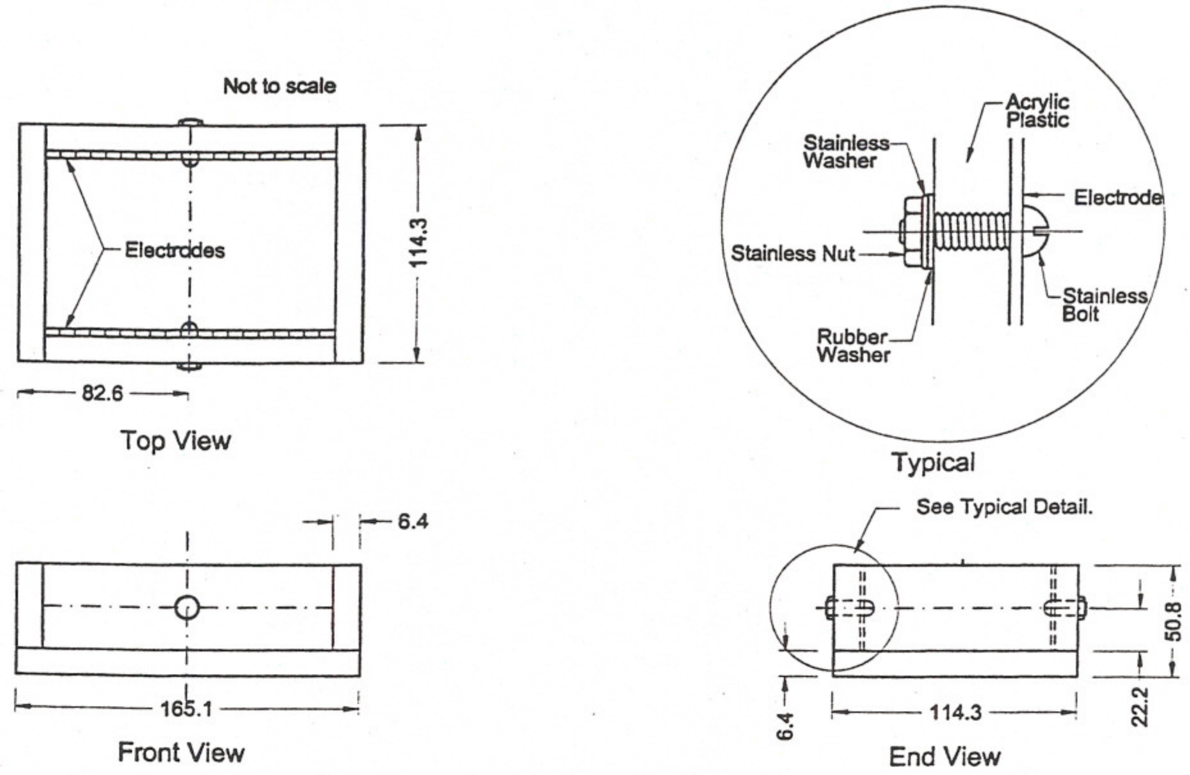
8.6. Graduated cylinder 100 mL capacity.

8.7. Distilled water or deionized water that has a resistivity greater than 20000 (ohm) × (cm).

8.8. Straightedge, 305 mm (12 in.) length.

Figure 1—Soil Box for Laboratory Resistivity Determination



Note: All dimensions are in millimeters.

<u>Materials</u>

Bottom - 1 Pc 165.1 mm x 114.3 mm x 6.4 mm

Ends - 2 Pcs 114.3 mm x 44.45 mm x 6.4 mm

Sides - 2 Pcs 152.4 mm x 44.45 mm x 6.4 mm

Electrodes - 2 Pcs 0.9-mm stainless steel 152.4 mm x 44.45 mm

2 Each - M 4 x 0.5 x 19.0 mm (or longer)
Round head stainless steel
machine bolt with rubber
washer and stainless steel
washer and nut

Figure 2—Soil Box for Laboratory Resistivity Determination

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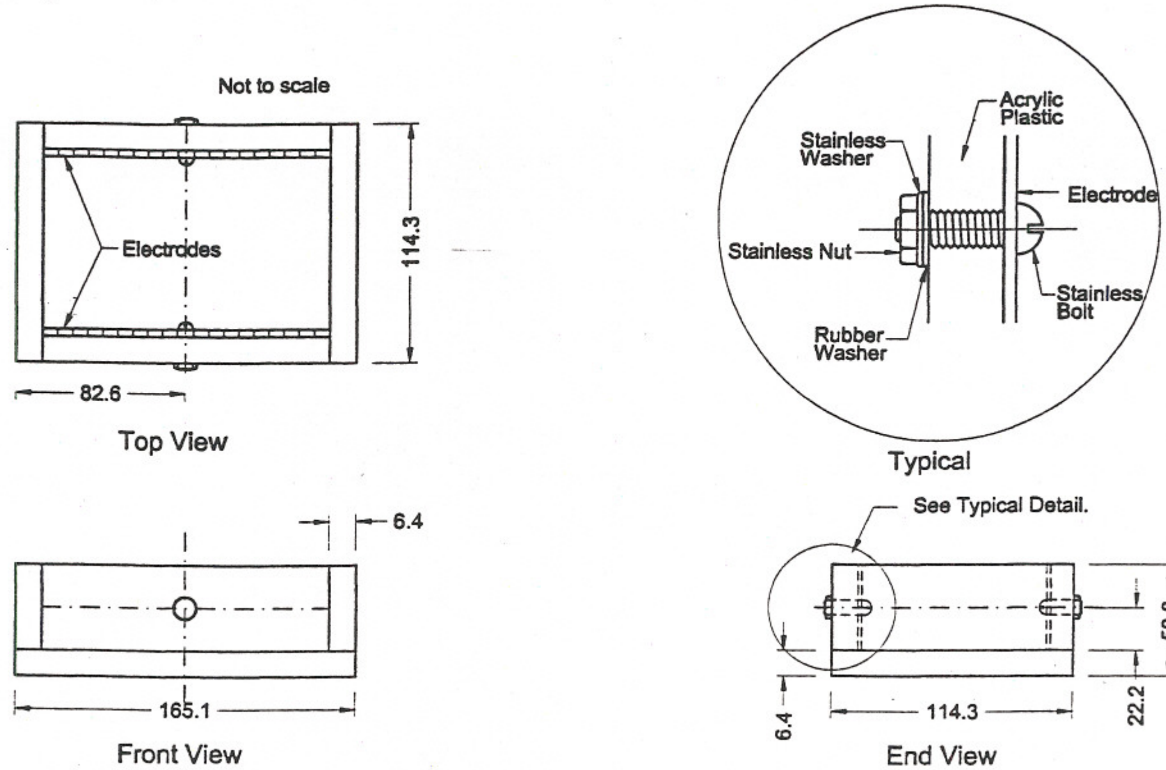

Note: All dimensions are in millimeters.

Materials

Bottom - 1 Pc 165.1 mm x 114.3 mm x 6.4 mm

Ends - 2 Pcs 114.3 mm x 44.45 mm x 6.4 mm

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Electrodes - 2 Pcs 0.9-mm stainless steel 152.4 mm x 44.45 mm

2 Each - M 4 x 0.5 x 19.0 mm (or longer)
Round head stainless steel
machine bolt with rubber
washer and stainless steel
washer and nut

Figure 2—Soil Box for Laboratory Resistivity Determination

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9. CALIBRATION OF RESISTIVITY METER

9.1. *Calibrate resistivity meter (follow manufacturer's instructions):*

9.1.1. Zero the resistivity meter by clamping the two leads together, and adjusting the meter (if necessary).

9.1.2. Connect the leads of the resistivity meter to the 100 ohm resistor and read the meter. Repeat this process with the 200, 500, and 900 ohm resistors.

9.1.3. If the readings are within 10 percent of the resistance of the resistor, the meter is functioning satisfactorily.

10. SOIL RESISTIVITY DETERMINATION

10.1. Select the material for testing in accordance with T 248 and separate on a 2.00 mm (No. 10) sieve. Approximately 1500 grams of the material passing the 2.00 mm (No. 10) sieve will be required for testing.

10.2. Add 150 mL of distilled water to the prepared soil. Mix the sample thoroughly and cover the test sample with a damp cloth and allow the sample to stabilize until equilibrium has been reached, or allow to cure a minimum of 12 hours.

10.3. Zero the meter as per manufacturer's instructions.

10.4. Clean the soil box thoroughly with distilled water.

10.5. Thoroughly mix and place the sample in the soil box in layers and compact (moderate compaction with the fingers is sufficient). Trim off the excess material with the straightedge.

10.6. Measure the resistance and calculate the resistivity of the soil in accordance with the instructions furnished with the meter and record the test value.

10.7. Remove and retain the soil from the box, add 100 mL of distilled water to the sample and mix thoroughly. Clean the soil box with distilled water prior to performing the next test.

10.8. Repeat the process of placing, compacting the soil in the box, then measure the resistance and calculate the soil resistivity (Note 5).

10.9. Repeat steps in Sections 10.4 to 10.8 until a minimum value can be determined.

10.10. The minimum value is used for computing the minimum soil resistivity and reporting (Note 6).

11. REPORT

11.1. The minimum soil resistivity value which was determined above should be reported in units of (ohm) × (cm).

11.2. The Minimum Soil Resistivity utilizing the typical soil box is:

$$\text{Minimum Soil Resistivity} = [\text{minimum reading (ohms)}] \times [6.67\ \text{cm}] \quad (1)$$

(See Note 7.)

Note 4—Most resistance meters without an inverting circuit allow the sample under test to polarize during measurement causing the reading to vary (i.e., drift).

Note 5—In some soils the minimum soil resistivity occurs when the specimen is in a slurry condition. When this occurs it is necessary to thoroughly mix the soil slurry and then pour the slurry water into the soil box until full. If the soil box doesn't reach its capacity with the addition of the slurry water then add just enough of the mixed soil into the box until the soil box is filled and then take the reading.

Note 6—The minimum soil resistivity can occur at any moisture content.

Note 7—Multiplying Constant for each Soil Box is derived by:

$$\frac{\text{Surface Area of One Electrode}\ (\text{cm}^2)}{\text{Measured Average Distance between Electrodes (cm)}} = \frac{cm^2}{cm} = cm \quad (2)$$

Typical Soil Box

$$\frac{15.24\ \text{cm} \times 4.445\ \text{cm}}{10.16\ \text{cm}} = 6.67\ \text{cm} \quad (3)$$

The soil box may be constructed of either 6.4 or 12.7 mm ($^1/_4$ or $^1/_2$ in.) acrylic plastic. If other size soil boxes are used, it will be necessary to determine the correct multiplier. It should also be noted that it may be necessary to prepare extra soil for testing to fill the soil box.

12. PRECISION AND BIAS

12.1. Data not available at this time.

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Standard Method of Test for

Determining pH of Soil for Use in Corrosion Testing

AASHTO Designation: T 289-91 (2000)

1. SCOPE

1.1. This test method describes procedures and apparatus for determining a pH value for corrosion testing by use of a pH meter.

1.2. The principal use of the test is to supplement soil-resistivity measurements and thereby identify conditions under which the corrosion of metals in the soil may be sharply accentuated. This standard is divided into two parts. The first part involves obtaining and preparing the sample to size for testing. The second part describes the test method for determining the pH of soil.

1.3. The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 92, Wire-Cloth Sieves for Testing Purposes
- M 231, Weighing Devices Used in the Testing of Materials
- R 11, Indicating Which Places of Figures Are to Be Considered Significant in Specified Limiting Values
- T 2, Sampling of Aggregates
- T 248, Reducing Samples of Aggregate to Testing Size

PART I—INITIAL PREPARATION OF TEST SAMPLES

3. SCOPE

3.1. This method covers the dry preparation of soil and soil-aggregate samples for determining a soil's pH.

3.2. The following applies to all specified limits in this standard: For the purpose of determining conformance with these specifications, an observed value or calculated value shall be rounded off "to the nearest unit" in the last right hand place of figures used in expressing the limiting value, in accordance with R 11.

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4. APPARATUS

4.1. *Sieves*—A series of sieves of the following sizes: 6.3 mm (1/4 in.), 4.75 mm (No. 4), 2.00 mm (No. 10) and a pan. The sieve shall conform to M 92, Sieves for Testing Purposes (Note 1).

4.2. *Balance*—The balance shall have sufficient capacity, be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

4.3. *Drying Apparatus*—Any suitable device capable of drying samples at a temperature not exceeding 60°C (140°F).

4.4. *Pulverizing Apparatus*—Either a mortar and rubber-covered pestle or any device suitable for breaking up the aggregations of soil particles without reducing the size of the individual grains (Note 2).

4.5. *Sample Splitter*—A suitable riffle sampler or sample splitter for proportional splitting of the sample and capable of obtaining representative portions of the sample without appreciable loss of fines. The width of the container used to feed the riffle sampler splitter should be equal to the total combined width of the riffle chutes. Proportional splitting of the sample on a canvas cloth is also permitted.

Note 1—The sieve sizes which have an opening size of 6.3 mm (1/4 in.) or larger shall conform to the requirements specified in M 92 excluding column no. 7. The exclusion of Column 7 permits the use of heavier screens in nonstandard frames which are larger than the 203.2 mm (8 in.) round frames.

Note 2—Other types of apparatus are satisfactory if the aggregations of soil particles are broken up without reducing the size of the individual grains.

5. SAMPLE SIZE

5.1. The amount of soil material required to perform the test is as follows:

Test	Approx Mass (g)	Sieve Size Finer Than:
pH	100	2.00 mm (No.10)

6. INITIAL PREPARATION OF TEST SAMPLES

6.1. The sample as received shall be in a moist condition for pH testing purposes. If the sample is too wet, it may be dried to a moist condition in air or a drying apparatus not exceeding 60°C (140°F) prior to sample selection (Note 3). A representative test sample to perform the pH test shall then be obtained with the sampler, or by splitting or quartering as per T 248.

Note 3—Samples dried in an oven or other drying apparatus at a temperature not exceeding 60°C (140°F) are considered to be air dried.

6.2. The portion of the sample selected for pH testing shall be separated into fractions by one of the following methods:

6.2.1. *Alternate Method Using 2.00-mm (No. 10) Sieve*—The sample shall be separated into two fractions using a 2.00-mm sieve. The fraction retained on the sieve shall be ground with a

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pulverizing apparatus until the aggregations of soil particles are broken into separate grains. The ground soil shall then be separated into two fractions using the 2.00-mm sieve.

6.2.2. *Alternate Method Using 4.75-mm and 2.00-mm (No. 4 and No. 10) Sieves*—The sample shall first be separated into two fractions using a 4.75 mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 4.75-mm sieve. The fraction passing the 4.75-mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00-mm sieve, and processed as in Section 6.2.1.

6.2.3. *Alternate Method Using 6.3-mm and 2.00-mm ($^1/_4$-in. and No. 10) Sieves*—The sample shall first be separated into two fractions using a 6.3-mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 6.3-mm sieve. The fraction passing the 6.3-mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00-mm sieve, and processed as in Section 6.2.1.

PART 2—DETERMINATION OF SOIL PH

7. SCOPE

7.1. This section describes the procedure for determining soil pH.

8. APPARATUS

8.1. *pH Meter*—suitable for laboratory or field analysis, with either one or two electrodes.

8.2. A 50 mL wide-mouth glass beaker or other suitable container with a watch glass for cover. If lightweight material is to be tested, it may be necessary to increase the beaker size up to a maximum of 250 mL.

8.3. *Standard Buffer Solutions of Known pH Values*—standards to be used are pH of 4.0, 7.0, 10.0.

8.4. Distilled water.

8.5. A teaspoon or small scoop.

8.6. A thermometer capable of reading 25 ± 10° C, to the nearest 0.1°C.

8.7. *Sieves*—A 2.00 mm (No. 10) for preparing the sample and a pan. The sieve shall conform to M 92, Sieves for Testing Purposes.

8.8. A glass stirring rod.

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9. PROCEDURE

9.1. Of the material selected for testing, place a mass of 30.0 ± 0.1 grams of soil into the glass beaker or other suitable container.

9.2. Add 30.0 ± 0.1 grams of distilled water to the soil sample. Stir to obtain a soil slurry and then cover with a watch glass.

9.3. The sample must stand for a minimum of one hour, stirring every 10 to 15 minutes. This is to allow the pH of the soil slurry to stabilize.

9.4. Measure the temperature of the sample and adjust the temperature controller of the pH meter to that of the sample temperature. This adjustment should be done just prior to testing. On meters with an automatic temperature control, follow the manufacturer's instructions.

9.5. Standardize the pH meter by means of the standard solutions provided. Temperature and adjustments must be performed as stated under Section 9.4 (See Note 4).

9.6. Immediately before immersing the electrode(s) into the sample, stir well with a glass rod. Place the electrode(s) into the soil slurry solution and gently turn the beaker or container to make good contact between the solution and the electrode(s). DO NOT place the electrode(s) into the soil, only into the soil slurry solution. (See Note 5).

9.7. The electrode(s) require immersion 30 seconds or longer in the sample before reading to allow the meter to stabilize. If the meter has an auto read system, it will automatically signal when stabilized.

9.8. Read and record the pH value to the nearest tenth of a whole number. If the pH meter reads to the hundredth place it is necessary to round off the result in accordance with the rounding-off method R 11.

9.9. Rinse off the electrode(s) well with distilled water, then dab lightly with tissues to remove any film formed on the electrode(s). *Caution*: Do not wipe the electrode(s) as this may result in polarization of the electrode(s) and consequent slow response. (See Note 6).

Note 4—To standardize the pH meter, use the 7.0 pH buffer standard solution plus the other standard solution which is nearest the estimated pH value of the sample to be tested. If the manufacturer's instructions indicate a method other than that noted above, then those instructions must be followed.

Note 5—When immersing electrode(s) into the glass beaker or container, care should be taken not to hit the bottom or side, causing damage to electrode(s).

Note 6—If polarization does occur, as indicated by a slow response, rinse the electrode(s) and dab lightly again.

10. PRECAUTIONS

10.1. Periodically check for damage to electrode(s).

10.2. Electrode tip should be kept moist during storage. Follow the manufacturer's instructions.

11. REPORT

11.1. As specified in Section 9.8, report the pH value to the nearest tenth of a whole number in accordance with R 11.

12. PRECISION AND BIAS

12.1. Data are not available at this time.

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Standard Method of Test for

Determining Water-Soluble Sulfate Ion Content in Soil

AASHTO Designation: T 290-95 (1999)

1. SCOPE

1.1. This test method covers the determination of the water-soluble sulfate ion content in soil. This standard is divided into two parts. The first part specifies the procedure for sampling and preparing the sample to size for testing. The second part delineates two test procedures (Methods A or B) for the determination of the sulfate ion content in soils. The selection of the method is dependent on the concentration of sulfate ion and the accuracy desired. Two methods are given as follows:

	Section
Method A:	
(Gravimetric Method)	(1 to 7) and (8 to 16)
Method B:	
(Turbidimetric Method)	(1 to 7) and (17 to 26)

1.2. Method A is a primary measure of sulfate ion. Method B is less time-consuming, but often more liable to interference than Method A. It is particularly useful in the lower sulfate range and can be used as a screening test. This method is directly applicable over the range of 10 to 100 mg/kg.

1.3. The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 92, Wire-Cloth Sieves for Testing Purposes
- M 231, Weighing Devices Used in the Testing of Materials
- R 11, Indicating Which Places of Figures Are to Be Considered Significant in Specified Limiting Values
- T 2, Sampling of Aggregates
- T 248, Reducing Samples of Aggregate to Testing Size

2.2. *ASTM Standards*:

- D 859, Test Method for Silica in Water
- D 1129, Standard Terminology Relating to Water
- D 1193, Standard Specification for Reagent Water

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- E 60, Standard Practice for Analysis of Metals, Ores, and Related Materials by Molecular Absorption Spectrometry
- E 275, Standard Practice for Describing and Measuring Performance of Ultraviolet, Visible, and Near-Infrared Spectrophotometers

3. DEFINITIONS

3.1. For definitions of terms used in these methods, refer to ASTM D 1129.

PART I—INITIAL PREPARATION OF TEST SAMPLES

4. SCOPE

4.1. This method covers the dry preparation of soil and soil-aggregate samples, as received from the field, for use in determining the sulfate ion content in soils.

4.2. The following applies to all specified limits in this standard: For the purpose of determining conformance with these specifications, an observed value or calculated value shall be rounded off "to the nearest unit" in the last right-hand place of figures used in expressing the limiting value, in accordance with R 11.

5. APPARATUS

5.1. The balance shall have sufficient capacity, be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

5.2. *Drying Apparatus*—Any suitable device capable of drying samples at a temperature not exceeding 60°C (140°F).

5.3. *Sieves*—A series of sieves of the following sizes: 6.3 mm ($^1/_4$ in.), 4.75 mm (No. 4), 2.00 mm (No. 10) and a pan. The sieve shall conform to M 92, Wire-Cloth Sieves for Testing Purposes (Note 1).

5.4. *Pulverizing Apparatus*—Either a mortar and rubber-covered pestle or any device suitable for breaking up the aggregations of soil particles without reducing the size of the individual grains (Note 2).

5.5. *Sample Splitter*—A suitable riffle sampler or sample splitter for proportional splitting of the sample and capable of obtaining representative portions of the sample without appreciable loss of fines. The width of the container used to feed the riffle sampler splitter should be equal to the total combined width of the riffle chutes. Proportional splitting of the sample on a canvas cloth is also permitted.

Note 1—The sieve sizes which have an opening size of 6.3 mm ($^1/_4$ in.) or larger shall conform to the requirements specified in M 92 excluding column No. 7. This exclusion permits the use of heavier screens in non-standard frames which are larger than the 203.2 mm (8 in.) round frames.

Note 2—Other types of apparatus are satisfactory if the aggregations of soil particles are broken up without reducing the size of the individual grains.

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6. SAMPLE SIZE

6.1. The amount of soil material required to perform the individual test is as follows:

Test	Approx Mass (g)	Sieve Size Finer Than:
Sulfates	250	2.00 mm (No. 10)

7. INITIAL PREPARATION OF TEST SAMPLES

7.1. The sample as received from the field may be dried in air or a drying apparatus not exceeding 60°C (140°F) prior to sample selection (Note 3). A representative test sample of the amount required to perform the tests shall then be obtained with the sampler, or by splitting or quartering. The aggregations of soil particles shall then be broken up in the pulverizing apparatus in such a way as to avoid reducing the natural size of the individual particles.

Note 3—Samples dried in an oven or other drying apparatus at a temperature not exceeding 60°C (140°F) are considered to be air dried.

7.2. The portion of the sample selected for sulfate testing shall be separated into fractions by one of the following methods:

7.2.1. *Alternate Method Using 2.00 mm (No. 10) Sieve*—The dried sample shall be separated into two fractions using a 2.00 mm sieve. The fraction retained on the sieve shall be ground with the pulverizing apparatus until the aggregation of soil particles is separated into individual grains. The ground soil shall then be separated into two fractions using the 2.00 mm sieve.

7.2.2. *Alternate Method Using 4.75 mm and 2.00 mm (No. 4 and No. 10) Sieves*—The dried sample shall first be separated into two fractions using a 4.75 mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 4.75 mm sieve. The fraction passing the 4.75 mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00 mm sieve, and processed as in Section 7.2.1.

7.2.3. *Alternate Method Using 6.3 mm and 2.00 mm ($^1/_4$ in. and No. 10) Sieves*—The dried sample shall first be separated into two fractions using a 6.3 mm sieve. The fraction retained on this sieve shall be ground with a pulverizing apparatus until the aggregations of soil particles are broken into separate grains and again separated on the 6.3 mm sieve. The fraction passing the 6.3 mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00 mm sieve, and processed as in Section 7.2.1.

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PART 2—DETERMINATION OF SULFATE CONTENT

8. METHOD A—GRAVIMETRIC METHOD (SECTIONS 8 TO 16)

8.1. *Scope:*

8.2. This method is utilized to determine the amount of water-soluble sulfate ion in soil. It is directly applicable to samples containing approximately 20 to 100 mg/kg of sulfate ion. It can be extended to higher or lower ranges by adjusting the sample size.

8.3. This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

9. SUMMARY OF METHOD

9.1. Sulfate ion is precipitated and the mass determined as barium sulfate after removal of silica and other insoluble matter.

10. INTERFERENCES

10.1. Sulfites and sulfides may oxidize and precipitate with the sulfate. Turbidity caused by silica or other insoluble material would interfere if allowed to be present, but removal of such interference is provided in this method.

10.2. Other substances tend to be occluded or adsorbed on the barium sulfate, but these do not significantly affect the precision and accuracy of the method.

11. PURITY OF REAGENTS

11.1. Reagent grade chemicals shall be used in all tests. Unless otherwise indicated, it is intended that all reagents shall conform to the specifications of the Committee on Analytical Reagents of the American Chemical Society, where such specifications are available. Other grades may be used, provided it is first ascertained that the reagent is of sufficiently high purity to permit its use without lessening the accuracy of the determination.

11.2. Unless otherwise indicated, references to water shall be understood to mean Type II reagent water conforming to Specification D 1193. In addition, reagent water used for these methods shall be sulfate-free.

12. REAGENTS

12.1. *Ammonium Hydroxide (sp gr 0.90)*—Concentrated ammonium hydroxide (NH_4OH).

12.2. *Barium Chloride Solution (100 g/L)*—Dissolve 100 g of barium chloride ($BaCl_2{\cdot}2H_2O$) in water and dilute to 1 L.

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12.3. *Hydrochloric Acid (1 + 9)*—Mix 1 volume of hydrochloric acid (HCl, sp gr 1.19) with 9 volumes of water.

12.4. *Hydrofluoric Acid (48 to 51 percent)*—Concentrated hydrofluoric acid (HF).

12.5. *Methyl Orange Indicator Solution (0.5 g/L)*—Dissolve 0.05 g of methyl orange in water and dilute to 100 mL.

12.6. *Nitric Acid (sp gr 1.42)*—Concentrated nitric acid (HNO_3).

12.7. *Picric Acid (saturated aqueous solution).*

12.8. *Silver Nitrate Solution (100 g/L)*—Dissolve 10 g of silver nitrate ($AgNO_3$) in water and dilute to 100 mL.

12.9. *Sulfuric Acid (sp gr 1.84)*—Concentrated sulfuric acid (H_2SO_4).

13. TEST PROCEDURE

13.1. Weigh 100 grams of the soil sample for testing. Put the test sample into a 500-mL Erlenmeyer flask.

13.2. Add 300 mL of distilled water. Stopper the flask and shake the mix.

13.3. Centrifuge the sample; if the sample exhibits turbidity then filter the sample through a 0.45 micron membrane filter. A drop of concentrated nitric acid may be added, if needed, to precipitate finely divided suspended matter (Note 4).

13.4. Pipet 30.0 mL, or measure a quantity of the clear sample containing sulfate ion equivalent to 20 to 50 mg of barium sulfate ($BaSO_4$) into a 250 mL beaker. Adjust the volume by evaporation, or dilution with water, to approximately 200 mL. Adjust the acidity of the sample to the methyl orange end point and add 10 mL excess HCl (1+9). If a quantity other than 30 mL is used, substitute the volume of the aliquot into the calculation in place of 30 mL.

Note 4—Silica may be removed before applying this method by dehydration with HCl or perchloric acid ($HClO_4$) in accordance with the respective procedures in Method D 859. In this case, the ignition described in Section 13.8 need not be done in a platinum crucible.

13.5. Measure into the beaker a quantity of the clear sample containing sulfate ion equivalent to 20 to 50 mg of barium sulfate ($BaSO_4$). Adjust the volume by evaporation or dilution with water to approximately 200 mL. Adjust the acidity of the sample to the methyl orange end point and add 10 mL excess of HCl (1 + 9).

13.6. Heat the acidified solution to boiling and slowly add to it 5 mL of hot $BaCl_2$ solution (Note 5). Keep the temperature just below boiling until the liquid has become clear and the precipitate has settled out completely. In no case shall this settling period be less than two hours.

Note 5—Faster precipitation and a coarser precipitate can be obtained by adding 10 mL of saturated picric acid solution and boiling the sample five minutes before adding $BaCl_2$.

13.7. Filter the suspension of $BaSO_4$ on a fine, ashless filter paper, and wash the precipitate with hot water until the washings are substantially free of chlorides, as indicated by testing the last portion

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of the washings with $AgNO_3$ solution (Note 6). Avoid excessive washing. If any $BaSO_4$ passes through the filter, pour the filtrate through the paper a second time (Note 7).

Note 6—Do not attempt to obtain a completely negative test for chloride. Discontinue washing when no more than a faint opalescence is produced in the test. Using a small amount of ashless filter paper pulp in the suspension will aid in filtration and reduce the tendency for $BaSO_4$ to pass through the filter.

Note 7—If the filtrate is poured through the paper a second time, $AgNO_3$ must not be present in the filtrate.

13.8. Place the filter paper and contents in a tared platinum crucible (Note 4), and char and consume the paper slowly without flaming. Ignite the residue at approximately 800°C for 1 hour, or until it is apparent that all carbon has been consumed.

13.9. Add a drop of H_2SO_4 and a few drops of HF, and evaporate under a hood to expel silica as silicon tetrafluoride (SiF_4). Reignite at about 800°C, cool in a desiccator, and determine the mass of the $BaSO_4$.

14. CALCULATION

14.1. Calculate the concentration of sulfate ion (SO_4) in milligrams per kilogram, as follows:

$$\text{Sulfate, mg/kg} = (W \times 411500)/S \qquad (1)$$

where:

W = grams of $BaSO_4$ and

S = grams of sample used, e.g.;

$$\frac{100\text{ g soil}}{S} = \frac{300\text{ mL water}}{30\text{ mL aliquot}}$$

S = 10 g

14.2. Sulfate Ion Content in Soil

(mg/kg moisture free) = [Sulfates (mg/kg as determined above) × 100]/ (100 – percent moisture) (2)

15. REPORT

1.1. Report the sulfate content as computed in Section 14.2 on a moisture-free basis in units of milligrams per kilogram (mg/kg). Report this value to the nearest whole number in accordance with the rounding-off method R 11.

16. PRECISION AND BIAS

16.1. Data are not available at this time.

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17. SCOPE

17.1. This method is intended for rapid routine or control tests for the water-soluble sulfate ion in soil where extreme accuracy and precision are not required. It is directly applicable over the range of 10 to 100 mg/kg of sulfate ion (SO_4).

18. SUMMARY OF METHOD

18.1. Sulfate ion is converted to a barium sulfate suspension under controlled conditions. Glycerin solution and a sodium chloride solution are added to stabilize the suspension and minimize interferences. The resulting turbidity is determined by a photoelectric colorimeter or spectrophotometer and compared to a curve prepared from standard sulfate solutions.

19. INTERFERENCES

19.1. Insoluble suspended matter in the sample must be removed. Dark colors that cannot be compensated for in the procedure interfere with the measurement of suspended barium sulfate ($BaSO_4$).

19.2. Although other ions normally found in water do not appear to interfere, the formation of the barium sulfate suspension is very critical. This method is more suitable as a control procedure where concentration and type of impurities present in the water are relatively constant. Determinations that are in doubt should be checked by Method A in some cases, or by the procedure suggested in Note 10.

20. APPARATUS

20.1. *Photometer*—A filter photometer or spectrophotometer suitable for measurements between 350 and 425 nm, the preferable wavelength range being 380 to 400 nm. The cell for the instrument should have a light path through the sample of approximately 40 mm, and should hold about 50 mL of sample. Filter photometers and photometric practices prescribed in this method shall conform to ASTM E 60; spectrophotometers shall conform to ASTM E 275.

21. REAGENTS

21.1. *Barium Chloride*—Crystals of barium chloride ($BaCl_2 \cdot 2H_2O$) screened to 20- to 30-mesh.

21.2. *Glycerin Solution (1 + 1)*—Mix 1 volume of glycerin with 1 volume of water.

Note 8—A stabilizing solution containing sodium carboxymethylcellulose (10 g/L) may be used instead of the glycerin solution.

21.3. *Sodium Chloride Solution (240 g/L)*—Dissolve 240 g of sodium chloride (NaCl) in water containing 20 mL of concentrated hydrochloric acid (HCl, sp gr 1.19), and dilute to 1 L with water. Filter the solution if turbid.

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21.4. *Sulfate, Standard Solution (1 mL = 0.100 mg SO_4)*—Dissolve 0.1479 g of anhydrous sodium sulfate (Na_2SO_4) in water, and dilute with water to 1 L in a volumetric flask. Standardize by the procedure prescribed in Section 13.

22. CALIBRATION

22.1. Follow the procedure given in Section 23, using appropriate amounts of the standard sulfate solution prepared in accordance with Section 21.4, and prepare a calibration curve showing sulfate ion content in mg/L plotted against the corresponding photometer readings (Note 9). Prepare standards by diluting with water 0.0, 2.0, 5.0, 10.0, 15.0, 20.0, 30.0, 40.0, and 50.0 mL of standard sulfate solution to 50-mL volumes in volumetric flasks. These solutions will have sulfate ion concentrations of 0.0, 4.0, 10.0, 20.0, 30.0, 40.0, 60.0, 80.0, and 100.0 mg/L (ppm), respectively.

Note 9—A separate calibration curve must be prepared for each photometer and a new curve must be prepared if it is necessary to change the cell, lamp, or filter, or if any other alterations of instrument or reagents are made. Check the curve with each series of tests by running two or more solutions of known sulfate concentrations.

23. TEST PROCEDURE

23.1. Weigh 100 g of the soil sample for testing. Put the test sample into a 500-mL Erlenmeyer flask.

23.2. Add 300 mL of distilled water. Stopper the flask and shake the mix.

23.3. Centrifuge the sample; if the sample exhibits turbidity then filter the sample through a 0.45 micron membrane filter. A drop of concentrated nitric acid may be added, if needed, to precipitate finely divided suspended matter (Note 4).

23.4. Pipet into a 200 mL beaker 50 mL of the clear sample containing between 0.5 and 5 mg of sulfate ion (Note 10). Dilute to 50 mL with water if required, and add 10.0 mL of glycerin solution (Note 8) and 5.0 mL of NaCl solution. If less than 50 mL of sample is used, use the appropriate dilution factor.

Note 10—The solubility of $BaSO_4$ is such that difficulty may be experienced in the determination of sulfate concentrations below 10 mg/kg. This can be overcome by concentrating the sample or by adding 5 mL of standard sulfate solution (1 mL = 0.100 mg SO_4) to the sample before diluting to 50 mL. This will add 0.5 mg SO_4 to the sample, which must be subtracted from the final result.

23.5. Fill a 40 mm sample cell with sample solution, wipe it with a clean, dry cloth, and place it in the cell compartment. Set the colorimeter to zero absorbance (100 percent transmission) for a blank. This compensates for any acidinsoluble matter that has not been filtered out, or for color present, or for both.

23.6. Pour the sample solution from the cell back into the beaker and add, with stirring, 0.3 g of $BaCl_2 \cdot 2H_2O$ crystals (Note 11). Continue gently stirring the solution for 60 seconds. Let it stand for four minutes, and stir again for 15 seconds. Fill the sample cell as before, and immediately make a reading with the photometer.

Note 11—The stirring should be at a constant rate in all determinations. The use of a magnetic stirrer has been found satisfactory for this purpose.

23.7. If interferences are suspected, dilute the sample with an equal volume of water, and determine the sulfate concentration again. If the value so determined is one-half that in the undiluted sample, interferences may be assumed to be absent.

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24. CALCULATION

24.1. Convert the photometer readings obtained with the sample to mg/L sulfate ion (SO_4) by use of the calibration curve described in Section 22.

Sulfate Ion Content in Soil (mg/kg not corrected for moisture)

$$= \frac{50 \text{ mLs solution } (\text{ppm curve})}{S} \quad (3)$$

where:

S = grams of samples used

$$\text{e.g., } \frac{100}{S} = \frac{300 \text{ mL water}}{50 \text{ mL aliquot}}$$

$$S = 16.6666 \text{ g}$$

24.2. Sulfate Ion Content in Soil

(mg/kg moisture free) = [Sulfates (mg/kg with moisture) × 100]/
(100 – percent moisture) *(4)*

25. REPORT

25.1. Report the sulfate content as computed in Section 24.2 on a moisture-free basis in units of milligram per kilogram (mg/kg). Report this value to the nearest whole number in accordance with the rounding-off method in R 11.

26. PRECISION AND BIAS

26.1. Data are not available at this time.

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Standard Method of Test for

Determining Water-Soluble Chloride Ion Content in Soil

AASHTO Designation: T 291-94 (2000)

1. SCOPE

1.1. This test method describes the procedures for sampling and testing a soil for chloride ion content. This standard is divided into two parts. The first part specifies the procedure for sampling and preparing the sample to size for testing. The second part delineates two test procedures (Methods A or B) for the determination of the water-soluble chloride ion content in soil. Two methods are given as follows:

	Section
Method A:	
(Mohr Titration Method)	(1 to 7) and (8 to 16)
Method B:	
(pH/mV Meter Method)	(1 to 7) and (17 to 28)

1.2. Method A is based upon the Mohr procedure for determining chloride ion with silver nitrate. Method B utilizes a pH/mV Meter. By comparing the mV readings to the calibration curve determine the chloride ion content.

1.3. The values stated in SI units are to be regarded as the standard.

2. REFERENCED DOCUMENTS

2.1. *AASHTO Standards*:

- M 92, Wire-Cloth Sieves for Testing Purposes
- M 231, Weighing Devices Used in the Testing of Materials
- R 11, Indicating Which Places of Figures Are to Be Considered Significant in Specified Limiting Values
- T 2, Sampling of Aggregates
- T 248, Reducing Samples of Aggregate to Testing Size

2.2. *ASTM Standards*:

- D 1129, Standard Terminology Relating to Water
- D 1193, Standard Specification for Reagent Water
- D 2777, Standard Practice for Determination of Precision and Bias of Applicable Methods of Committee D-19 on Water
- D 3370, Practices for Sampling Water from Closed Conduits

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3. DEFINITIONS

3.1. For definitions of terms used in this test method, refer to ASTM D 1129.

PART 1—INITIAL PREPARATION OF TEST SAMPLES

4. SCOPE

4.1. This method covers the dry preparation of soil and soil-aggregate samples, as received from the field, for use in determining the chloride content.

4.2. The following applies to all specified limits in this standard: For the purpose of determining conformance with these specifications, an observed value or calculated value shall be rounded off "to the nearest unit" in the last right-hand place of figures used in expressing the limiting value, in accordance with R 11.

5. APPARATUS

5.1. *Balance*—The balance shall have sufficient capacity, be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

5.2. *Drying Apparatus*—Any suitable device capable of drying samples at a temperature not exceeding 60°C (140°F).

5.3. *Sieves*—A series of sieves of the following sizes: 6.3-mm ($^1/_4$-in.), 4.75-mm (No. 4), 2.00-mm (No. 10) sieve and a pan. The sieve shall conform to M 92, Wire-Cloth Sieves for Testing Purposes (Note 1).

5.4. *Pulverizing Apparatus*—Either a mortar and rubber-covered pestle or any device suitable for breaking up the aggregations of soil particles without reducing the size of the individual grains (Note 2).

5.5. *Sample Splitter*—A suitable riffle sampler or sample splitter for proportional splitting of the sample and capable of obtaining representative portions of the sample without appreciable loss of fines. The width of the container used to feed the riffle sampler splitter should be equal to the total combined width of the riffle chutes. Proportional splitting of the sample on a canvas cloth is also permitted.

Note 1—The sieve sizes which have an opening size of 6.3 mm ($^1/_4$ in.) or larger shall conform to the requirements specified in M 92 excluding column No. 7. This exclusion permits the use of heavier screens in non-standard frames which are larger than the 203.2 mm (8 in.) round frames.

Note 2—Other types of apparatus are satisfactory if the aggregations of soil particles are broken up without reducing the size of the individual grains.

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6. SAMPLE SIZE

6.1. The amount of soil material required to perform the individual test is as follows:

Test	Approx Mass (g)	Sieve Size Finer Than:
Chlorides	250	2.00 mm (No. 10)

7. INITIAL PREPARATION OF TEST SAMPLES

7.1. The sample as received from the field may be dried in air or a drying apparatus not exceeding 60°C (140°F) prior to sample selection (Note 3). A representative test sample of the amount required to perform the tests shall then be obtained with the sampler, or by splitting or quartering. The aggregations of soil particles shall then be broken up in the pulverizing apparatus in such a way as to avoid reducing the natural size of the individual particles.

Note 3—Samples dried in an oven or other drying apparatus at a temperature not exceeding 60°C (140°F) are considered to be air dried.

7.2. The portion of the sample selected for chloride testing shall be separated into fractions by one of the following methods:

7.2.1. *Alternate Method Using 2.00-mm (No. 10) Sieve*—The dried sample shall be separated into two fractions using a 2.00 mm sieve. The fraction retained on this sieve shall be ground with the pulverizing apparatus until the aggregation of soil particles is separated into individual grains. The ground soil shall then be separated into two fractions using the 2.00 mm sieve.

7.2.2. *Alternate Method Using 4.75-mm and 2.00-mm (No. 4 and No. 10) Sieves*—The dried sample shall be separated into two fractions using a 4.75 mm sieve. The fraction retained on this sieve shall be ground with the pulverizing apparatus until the aggregation of soil particles is separated into individual grains and again separated on the 4.75 mm sieve. The fraction passing the 4.75 mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00 mm sieve, and processed as in Section 7.2.1.

7.2.3. *Alternate Method Using 6.3-mm and 2.00-mm (¹/₄-in. and No. 10) Sieves*—The dried sample shall be separated into two fractions using a 6.3-mm sieve. The fraction retained on this sieve shall be ground with the pulverizing apparatus until the aggregation of soil particles are separated into individual grains and again separated on the 6.3-mm sieve. The fraction passing the 6.3-mm sieve shall be mixed thoroughly and, by the use of the sampler or by splitting and quartering, a representative portion adequate for testing shall be obtained. This split-off portion shall then be separated on the 2.00-mm sieve, and processed as in Section 7.2.1.

PART 2—DETERMINATION OF WATER-SOLUBLE CHLORIDE ION CONTENT BY MOHR TITRATION METHOD (METHOD A)

8. SCOPE

8.1. This method covers the test procedure for the determination of water-soluble chloride content of soils.

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8.2. Samples containing from 10 to 150 mg/kg of chloride can be analyzed by this test method. These levels are achieved by dilution as described in the test method.

8.3. This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of whoever uses this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

9. SUMMARY OF METHOD

9.1. This test method is based upon the Mohr procedure for determining chloride ion with silver nitrate. The chloride reacts with the silver ion before any silver chromate forms, due to the lower solubility of silver chloride. The potassium chromate indicator reacts with excess silver ion to form a red silver chromate precipitate. The end point is the appearance of the first permanent orange color.

9.2. This test method is suitable for analyzing solutions with a pH between 6.0 and 8.5.

10. INTERFERENCES

10.1. Sulfide, bromide, iodide, thiocyanate, cyanide, phosphate, sulfite, carbonate, hydroxide, and iron interfere in this test method. Sulfide, sulfite, and thiosulfate can be removed with a peroxide treatment, but usually no attempt is made to remove bromide and iodide because they are usually present in insignificant quantities compared to chloride. If necessary, the pH can be raised and the hydroxides of several metals, including iron, can be filtered off. Iron, barium, lead, and bismuth precipitate with the chromate indicator.

11. APPARATUS

11.1. Buret, 25 mL capacity.

11.2. Hotplate.

11.3. Magnetic stirrer and TFE-fluorocarbon-coated stirring bars.

11.4. Buret, 50 mL capacity, 0.1 mL gradations.

11.5. Pipets, 1, 5, 10, 25, 30, and 50 mL.

11.6. Beaker, 250 mL.

11.7. Erlenmeyer flask, 500 mL.

11.8. Centrifuge with tubes capable of holding at least 50 mL.

11.9. Phydrion papers covering pH 1 through 11 in 1 pH units. A pH meter is preferable if available.

11.10. *Balance*—The balance shall have sufficient capacity, be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

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12. REAGENTS

12.1. *Purity of Reagents*—Reagent grade chemicals shall be used in all tests. Unless otherwise indicated, it is intended that all reagents shall conform to the specification of the Committee on Analytical Reagents of the American Chemical Society, where such specifications are available. Other grades may be used, provided it is first ascertained that the reagent is of sufficiently high purity to permit its use without lessening the accuracy of the determination.

12.2. *Nitric Acid Solution (1 + 19)*—Add 1 volume of HNO_3 (sp. gr. 1.42) to 19 volumes of water.

12.3. *Purity of Water*—Unless otherwise indicated, references to water shall be understood to mean reagent water conforming to ASTM D 1193, Type III.

12.4. *Silver Nitrate Solutions*:

12.4.1. Silver nitrate solution (1 mL of solution is equivalent to 1 mg of chloride). Dissolve 4.79 grams of $AgNO_3$ in distilled water. Dilute to one liter. Add one drop concentrated nitric acid (HNO_3) and dilute to 1 L in a volumetric flask. The HNO_3 will eliminate any precipitation of silver hydroxide which would change the concentration. Standardize against sodium chloride (NaCl). Store in an amber-brown bottle to protect the solution from light.

12.4.2. *Silver Nitrate Solution, Standard (equivalent to 2 mg Cl/mL)*—For chloride concentrations slightly higher than specified in Section 12.4.1, this is a more concentrated standard. Dissolve 9.5834 g of $AgNO_3$ in approximately 700 mL of water. Add one drop concentrated nitric acid (HNO_3) and dilute to 1 L in a volumetric flask. Standardize against sodium chloride (NaCl). Store in an amber-brown bottle to protect the solution from light.

12.4.3. *Silver Nitrate Solution, Standard (equivalent to 5 mg Cl/mL)*—For chloride concentrations higher than specified in Section 12.4.2, dissolve 23.9582 g of $AgNO_3$ in approximately 700 mL of water. Add 1 drop concentrated nitric acid (HNO_3) and dilute to 1 L in a volumetric flask. The HNO_3 will eliminate any precipitation of silver hydroxide which would change the concentration. Standardize against sodium chloride (NaCl) by procedure described below. Store in an amber-brown bottle to protect the solution from light.

12.5. *Sodium Chloride Solution*:

12.5.1. Dry 2 to 6 g of high purity (minimum 99.5 percent) sodium chloride crystals at 110 ± 5°C for 1 hour and cool in a desiccator to room temperature.

12.5.2. Weigh 1.6484 g of the NaCl crystals. Transfer the crystals into a 1-L volumetric flask, dissolve, dilute, and mix well. A quantity of 1 mL of this solution provides 1 mg of Cl.

12.5.3. A 0.2 mL blank can be utilized or determine the indicator blank by substituting 100 mL of reagent grade water for the sample and perform the following:

12.5.3.1. Check pH with a meter if available, or with phydrion paper. If pH is in the range of 6 through 8, proceed immediately to Step B. If the pH is below 6 add sodium bicarbonate to adjust to the above range; if the pH is above 8, add nitric acid to adjust to the above range.

12.5.3.2. Add two drops of potassium chromate solution.

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12.5.3.3. Titrate with silver nitrate solution until the indicator begins to turn from yellow to red.

12.5.3.4. Indicator blank is the volume of $AgNO_3$ required to the end point. Record this value as the blank.

12.5.4. The titre (T) of the solutions is given in the following equation:

$$T = \frac{mg\ \text{CL used}}{\text{mL AgNO}_3\ required - Blank} \qquad (1)$$

12.5.5. If the titre (T) of the solutions are not exactly 1.0, 2.0, or 5.0 mg of Cl per mL $AgNO_3$, it may be desirable to dilute the solutions if they are too concentrated or add more silver nitrate if too weak. In either case, restandardize the resulting solution.

12.6. *Potassium or Sodium Chromate, Indicator Solution*—Prepare a 5 percent solution (5 g/100 mL) and adjust the pH to 7.0 with HNO_3 (1 + 19) or $NaHCO_3$ powder described in Sections 12.2 and 12.7.

12.7. *Sodium bicarbonate ($NaHCO_3$)*— Powder to adjust the sample pH to 8.3.

12.8. *Standardization of Silver Nitrate Solutions*:

12.8.1. Pipet 25 mL aliquots of NaCl solution to standardize the $AgNO_3$ solution (1 mg Cl/mL) and dilute to 100 mL with water. For the higher concentrations use 50 mL aliquots of the NaCl solution to standardize the $AgNO_3$ solution (2 mg Cl/mL) and dilute to 100 mL with water. Use 100 mL of the NaCl solution to standardize the more concentrated $AgNO_3$ solution (5 mg of Cl/mL).

12.8.2. Add 1 mL of five percent indicator solution (see Section 12.6), 1 g of sodium bicarbonate powder, and titrate to the appearance of a permanent orange color preceding a red precipitate. The analyst must practice this titration to become familiar with this color.

13. TEST PROCEDURE

13.1. Weigh 100 grams of soil into a 500 mL Erlenmeyer flask. Add 300 mL of distilled water. Stopper and shake vigorously for 20 seconds. After 1 hour repeat shaking. Centrifuge the sample. If the sample exhibits turbidity then filter the sample through a 0.45 micron membrane filter.

13.2. Check pH and titrate a 30 mL aliquot for chlorides as follows:

13.2.1. Check pH with a meter if available, or with phydrion paper. If pH is in the range of six through eight, proceed immediately to Step B. If the pH is below six add sodium bicarbonate to adjust to the above range; if the pH is above 8, add nitric acid to adjust to the above range.

13.2.2. Add two drops of potassium chromate solution.

13.2.3. Titrate with silver nitrate solution until the indicator begins to turn from yellow to red. If the titration is over 30 mL, take a smaller sample so as to keep the titration under this value. Dilute any smaller sample to about 50 mL with distilled water before titrating.

13.2.4. Record the volume of $AgNO_3$ required to the end point and calculate the chloride content.

13.2.5. Deduct the 0.2 blank or the blank volume as determined in Section 12.5.5.

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14. CALCULATIONS

14.1. Calculate the chloride content as follows:

$$\text{Chloride content (mg/kg)} = (mL\ \text{AgNO}_3\ \text{used} - B) \times T \times 1000/S \qquad (2)$$

T = titre, mg Cl/mL of $AgNO_3$; and

B = indicator blank, 0.2 or as determined in Section 12.5.5; and

S = g of sample titrated as diluted in steps described in Sections 13.1 and 13.2, e.g.;

$$\frac{100\text{ g soil}}{S} = \frac{300\text{ mL water}}{30\text{ mL aliquor}}$$

S = 10 gr

14.2. Chloride content on a moisture-free basis: Determine percent moisture by drying at 110 ± 5°C and adjust above result as:

Chloride content mg/kg (moisture-free basis) = [mg/kg (as received) × 100]/(100 – percent moisture)

15. REPORT

15.1. The chloride content will be reported as in Section 14.2 on a moisture-free basis in milligrams per kilogram (mg/kg). This result will be reported to the nearest whole number in accordance with R 11.

16. PRECISION AND BIAS

16.1. Data are not available at this time.

DETERMINATION OF WATER-SOLUBLE CHLORIDE ION CONTENT UTILIZING A PH/MV METER (METHOD B)

17. SCOPE

17.1. This method covers the test procedure and apparatus for the determination of water-soluble chloride ion content of soils by the use of pH/mV meter equipped with chloride ion electrode(s).

17.2. Samples containing from 10 to 1000 mg/kg of chloride can be analyzed by this test method. The range is based on the calibration curve that is developed (Note 5).

17.3. This standard may involve hazardous materials, operations, and equipment. This standard does not purport to address all of the safety problems associated with its use. It is the responsibility of the user of this standard to consult and establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

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18. SUMMARY OF METHOD

18.1. This test method utilizes a pH/mV meter equipped with chloride ion selective electrode(s). This apparatus can be used directly to measure concentrations after the addition of mV ionic strength adjustment buffer to sample(s) and standards. This procedure brings all solutions to the same ionic strength so that the activity coefficients are equal in all cases. The activity of an ion is a function of the change and concentration of all ions present.

19. INTERFERENCES

19.1. Sulfide, bromide, iodide, thiocyanate, cyanide, phosphate, sulfite, carbonate, hydroxide, and iron interfere in this test method. Sulfide, sulfite, and thiosulfate can be removed with a peroxide treatment, but usually no attempt is made to remove bromide and iodide because they are usually present in insignificant quantities compared to chloride.

20. APPARATUS

20.1. pH/mV Meter.

20.2. Electrodes.

(a) Reference Electrode Ag/AgCl Double Junction.
(a) Chloride Electrode.

20.3. Centrifuge with tubes capable of holding at least 50 mL.

20.4. *Glassware, Assorted*—As required in the procedure.

20.5. *Balance*—The balance shall have sufficient capacity, be readable to 0.1 percent of the sample mass, or better, and conform to the requirements of M 231.

20.6. *Drying Apparatus*—An oven capable of drying samples at a temperature of 110 ± 5°C.

21. REAGENTS/SOLUTIONS

21.1. *Purity of Reagents*—Reagent grade chemicals shall be used in all tests. Unless otherwise indicated, it is intended that all reagents shall conform to the specification of the Committee on Analytical Reagents of the American Chemical Society, where such specifications are available. Other grades may be used, provided it is first ascertained that the reagent is of sufficiently high purity to permit its use without lessening the accuracy of the determination.

21.1.1. 4 M KC1 Solution saturated with AgCl (Purchased, reference electrode internal chamber fill solution).

21.1.2. 1 M KNO_3 Solution. Dissolve 101.11 g of Reagent grade potassium nitrate in deionized water and dilute to one liter. (Reference electrode external chamber fill solution).

21.1.3. 0.2 M potassium nitrate (KNO_3) buffer. Dissolve 20.22 g of potassium nitrate in deionized water and dilute to one liter (Note 4).

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Note 4—Other strengths of the buffer solution may be substituted in lieu of the 0.2 M buffer. However, it should be noted that the buffer solution being substituted must be used to develop the calibration curve as well as for performing the test.

21.1.4. Potassium Chloride Stock Solution. Dry the potassium chloride in an oven at 110 ± 5°C for a minimum of 12 hours and allow to cool in a desiccator before determining the mass. Dissolve 2.103 g of Reagent potassium chloride in deionized water and dilute to one liter (Note 5). This produces a stock solution of 1000 mg/L.

Note 5—Other strengths of stock solution may be prepared which will span the range of the material being tested.

21.1.5. Standard Solutions. Standard solutions are prepared by diluting potassium chloride stock solution.

22. SAMPLE PREPARATION

22.1. Select a representative portion of the material passing the 2.00 mm (No. 10) sieve and dry a minimum of 12 hours at 110 ± 5°C.

22.2. To 100 g of the prepared soil, add 100 mL of deionized water and agitate for fifteen (15) minutes on a small paint shaker. If less vigorous means of agitation is used, a longer period of time is necessary.

22.3. Centrifuge a 50 mL portion of the mixture at approximately 10000 r/min for ten (10) to fifteen (15) minutes and then transfer 20 mL or 25 mL aliquot into a 100 mL beaker. If the sample is turbid then filter the sample through a 0.45 micron membrane filter.

23. CALIBRATION OF METER—MILLIVOLT MEASUREMENT

23.1. *Calibrate the pH/mV Meter*—Follow the instruction supplied with the meter and the electrodes.

24. PROCEDURE

24.1. *Preparation of Calibration Curves*:

24.1.1. Calibration solutions should be prepared by serial dilution of the potassium chloride stock solution. The range of the standards should be from 10 to 1000 mg/L. Solutions of other strengths may be prepared which span the range of the material being tested.

24.1.2. Standards and samples should be ionic strength buffered.

24.1.3. Construct a calibration graph on semi-log paper by plotting the value of the concentration standards on the long axis (mg/L) versus the mV reading obtained with these standards on the linear axis. (Ion selective electrodes give a logarithmic response to the activities of ions rather than to their concentrations. With the addition of an ionic strength adjustment buffer, the electrodes can be used directly to measure concentrations or millivolts.)

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25. CHLORIDE CONCENTRATION MEASUREMENT

25.1. Transfer an aliquot (25 mL to 40 mL) of liquid from the centrifugation of the sample into a 100 mL beaker.

25.2. Buffer the samples (or standards) by the addition of an equal volume of the 0.2 M KNO_3 solution (1 to 1 by volume).

25.3. Rinse electrodes with deionized water, blot off excess liquid.

25.4. Immerse electrodes into solution—stir gently.

25.5. Allow the electrodes to remain in the solution until the meter reading stabilizes. Refer to the instructions supplied with the meter and the electrode(s).

25.6. After stabilization, read and record the displayed reading (See manufacturer's operating instructions).

25.7. Before removing the electrode(s) from the solution refer to the proper operating instructions supplied with the meter and electrode(s) so as not to damage the meter.

25.8. Remove the electrodes from the solution, rinse and blot dry.

25.9. Consult the calibration curve to determine the concentration to which the displayed value in mV corresponds.

26. PRECAUTIONS

26.1. Periodically check the electrodes for any damage.

26.2. The temperature of the solution used to develop the calibration curve must be the same temperature as the sample being tested.

27. REPORT

27.1. Report the values (mg/kg) to the nearest whole number in accordance with R 11.

28. PRECISION AND BIAS

28.1. Data are not available at this time.

APPENDIX C

- MACTEC Report, April 29, 2003, "*Laboratory Testing of the Density of Expanded Shale, Clay and Slate Lightweight Aggregates*", pp. C1– C5
- Background on Determining the Compacted Density of ESCS Lightweight Aggregates, pp. C6 – C8
- ASTM D 698-00a, "*Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort*", pp. C9 – C19*
- ASTM D 4253-00, "*Standard Test Methods For Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*", pp. C20 – C33*
- ASTM D 4254-00, "*Standard Test Methods For Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*", pp. C34 – C42*

* Visit www.ASTM.org for document

April 29, 2003

Mr. John P. Ries, President
Expanded Shale, Clay & Slate Institute (ESCSI)
2225 E. Murray-Holladay Rd., Suite 102
Salt Lake City, UT 84117

Subject: **Report of Laboratory Testing of the Density of Expanded Shale, Clay and Slate Lightweight Aggregates**

Dear Mr. Ries:

As requested by Mr. T.A. Holm of ESCSI, MACTEC Engineering and Consulting of Georgia, Inc. (MACTEC) has completed the research and development testing program to determine methods for the evaluation on in-place density on four samples received from ESCSI. Out testing program consisted of determining the density of each of the four samples and determining the potential degradation of the aggregates after density testing using the following procedures:

- Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density. (ASTM D 4254)
- Standard Test Methods for Maximum Index Testing and Unit Weight of Soils Using a Vibratory Table. (ASTM D 4253)
- Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort. (ASTM D 698 – AASHTO T 99) – Modified Version. As requested by Mr. Holm, this method was modified and renamed "One Point Proctor Test". The aggregate was placed in 3 layers in a 0.5-cubic foot bucket, with each layer compacted by 25 blows of 5.5 lbf rammer dropped from a distance of 12 inches. The aggregate was compacted only once at the as received moisture content. This testing procedure was designed to replicate the in-place density of a granular lightweight aggregate after field compaction of at least two passes of pneumatic rolling equipment.
- Standard Test Method for Unit Weight and Voids in Aggregates (ASTM c 29 and AASHTO T 19) – Modified Version. As requested by Mr. Holm, this method was modified and renamed "Density Test by Rodding". The aggregate was placed in 3 layers in a 0.5-cubic foot bucket, with each layer rodded 25 times with a 5/8-inch diameter steel rod with one end rounded to a hemispherical tip. This procedure was performed on Sample Z only.

- Sieve Analysis of Fine and Coarse Aggregates (ASTM C 136 and AASHTO T 27) both before and after the Maximum Index Testing. Sieve analysis was also performed after the One Point Proctor Test and Density Test by Rodding for sample Z.

The results of the testing program are summarized in the tables below:

Table 1: Summary of Relative Density as obtained by One-Point Proctor Test

Sample ID	Maximum Index Density (pcf) (ASTM D 4253)	Minimum Index Density (pcf) (ASTM D 4254)	One-Point Proctor Density (pcf) (ASTM D 698 Rev)	Moisture Content as Tested (%)
W	55.5	51.6	54.3	2.0
X	51.6	49.7	54.0	3.0
Y	55.3	52.5	58.0	21.3
Z	41.6	38.2	41.0	0.4

Table 2: Sieve Analysis Before and After Maximum Index Test

Sample ID	W		X		Y	
Sieve Size	Before (%)	After (%)	Before (%)	After (%)	Before (%)	After (%)
1"	--	--	100	100.0	100.0	100.0
3/4"	--	--	99.4	99.7	98.1	98.9
1/2"	--	--	87.6	90.0	58.8	61.4
3/8"	100.0	100	63.1	64.8	32.3	35.5
#4	50.6	60.2	5.7	6.9	8.0	9.0
#8	4.0	4.2	--	--	--	--
#16	1.6	1.6	--	--	--	--

Table 3: Sieve Analysis Before and After Maximum Index Test, One-Point Proctor Test and Density Test by Rodding for Sample Z.

Sample ID	Z		Z		Z	
Test	Maximum Index Test		One-Point Proctor		Rodding Test	
Sieve Size	Before (%)	After (%)	Before (%)	After (%)	Before (%)	After (%)
3/4"	100	100	100	100.0	100	100.0
1/2"	92.1	93.6	92.1	92.0	92.1	91.3
3/8"	76.8	78.7	76.8	76.4	76.8	75.6
#4	8.3	8.2	8.3	8.6	8.3	8.0
#8	1.6	2.4	1.6	1.6	1.6	1.7
#16	1.1	1.5	1.1	1.0	1.1	1.3
#30	1.0	1.2	1.0	0.7	1.0	1.1
#50	0.8	1.0	0.8	0.6	0.8	0.9
#100	0.5	0.4	0.5	0.2	0.5	0.6
Compacted Density	Loose 38.21	41.63	Loose 38.21	41.05	Loose 38.21	40.78

It was observed that the density obtained from the One-Point Proctor Test for Samples X and Y were greater than the density obtained from the Maximum Index Test for the same samples. It was also observed from these test results that there was minimum degradation and development of fines while undergoing compaction by the three methodologies shown above.

We appreciate the opportunity to provide these services for you and would welcome the opportunity to provide any additional studies or services necessary to complete this or other project for ESCSI. If you have any questions regarding the laboratory testing results, or if you require any additional information, please do not hesitate to call.

Very truly your,

MACTEC ENGINEERING AND CONSULTING OF GEORGIA, INC.

Chan Tin, P.E.
Project Engineer

cc: File (1)
Tom Holm (ESCSI)

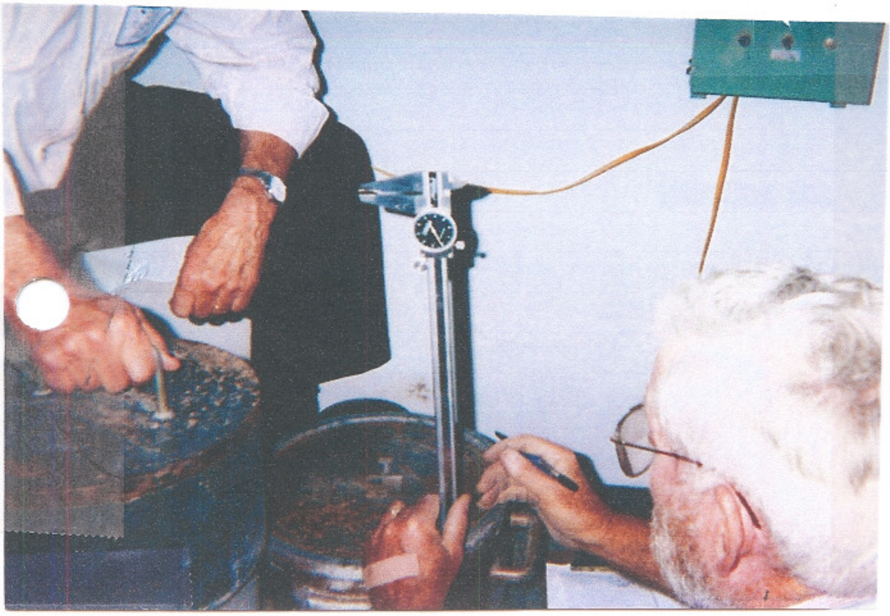
ONE POINT PROCTOR

Filling 0.5 cf bucket
Loosely place in 3 layers

5.5# Hammer, 12" Drop
25 Blows each layer
ASTM 698 Procedures

Screeded-off, weighed
One-Point Proctor Density
(Moist compact D 698 modified)

ASTM D 4253 "Standard Test Method For Maximum Index Density of Soils Using a Vibratory Table"



After vibration dial gauge measures depth of consolidation of sample

Lab technician attempts to pick up 190# surcharge weight

September 4, 2003

BACKGROUND ON DETERMINING THE COMPACTED DENSITY OF ESCS LIGHTWEIGHT AGGREGATES

The work of R.R. Proctor in the nineteen thirties directed attention to methods for determining the soil density which can be obtained during compaction. Experience on this subject has been accumulated to such a point that the testing procedure in ASTM and AASHTO are now standard practice on earthworks construction.

The density of natural soils is affected by three variables: Moisture content, soil type and grading and compactive effort. As the possible combinations of these variables are virtually unlimited, the direct comparison of various items of equipment is difficult. The practical problem on a given job is usually to arrange for utilization of the compaction equipment to the best advantage. For a field operation the term compaction effort can therefore be defined as a specified number of passes of a given machine of given weight at a given speed. Under a given compaction effort, the density of usual natural soils varies with moisture content. Furthermore, at a given moisture content, the density of a given soil varies with compactive effort, that is, with variation in number of passes or weight of the specified equipment. At a given water content and under a given compaction effort, a clay and a sand or even a clay and a silt will respond very differently.

Moisture-Density Relations

The interrelation of the three variables – density, moisture content, and compaction effort is graphically represented in diagrams often referred to as proctor curves. Proctor curves are in such common usage that a good understanding of their characteristics and significance is essential.

Each of the curves (1,2,3,4 See Fig.____) represents moisture-density data for the same soil under a particular compaction effort. Density is usually indicated in these diagrams by dry density weight, (Pounds per cubic foot) moisture content as moisture as a percentage of dry weight of solids. Proctor curves for cohesive soils are generally alike in respect to developing a more or less well-defined peak. This characteristic peak demonstrates that with a given compaction effort maximum density will be obtained at a particular water content. This water content is known as the optimum, that is, the best or most favorable moisture condition for obtaining the desired result. In practical terms these curves indicate that it is inefficient to undertake compaction when the soil is either drier or wetter than its optimum moisture content if attainment of maximum density is the objective. When the soil is too wet by this standard, the inefficiency of the operation can readily be observed in the field, as the roller tends to bog down and displace the soil rather than to compress it. Compaction on the "dry side" is not so obviously inefficient. Compaction and earthmoving equipment both operate with less difficulty on drier material, and contractors tend to prefer this condition.

Laboratory Compaction Techniques

In lab experiments the minus number 4 fraction of soil is compacted in a small (4 or 6") steel cylinder, not by any process of rolling similar to the field operation but by a standardized ASTM tamping process. The compaction effort in the laboratory test is a function of the weight of the hammer, the distance through which it is allowed to fall, and the number of times it is dropped on a given layer of soil in the cylinder. Laboratory tamping procedures only approximate the results produced by rolling or other field operations. This has led to the adoption of several compaction standards.

Standard Proctor Compaction (ASTM D 698), (AASHTO T 99)

The standard proctor test is conducted with 5.5 lb. tamper, which is allowed to drop within a sleeve 12 inches. The soil is compacted in three successive layers in the proctor cylinder, and the tamper is dropped 25 or 56 times in each layer. The densities obtained during this process are considered to be roughly the equivalent of densities obtained in the field under three passes of relatively light compaction equipment.

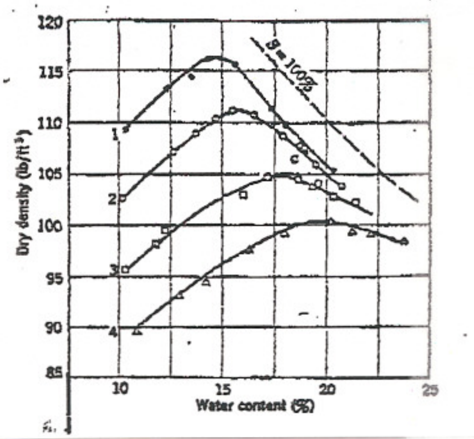
Modified Proctor (ASTM D 1557), (AASHTO T)

The modified compaction test is conducted with a 10 lb. hammer falling 18 inches. The soil is compacted in five rather than three successive layers, and each layer receives 25 or 56 blows of the tamper. The densities obtained are considered to be roughly the equivalent of field compaction under the relatively heavy post World War II earth moving and compaction equipment.

It is now fairly common practice to use the compaction test procedures to establish job compaction standards. It is also common to specify that during construction, densities at least equal to 95 percent of this maximum must be obtained in the field. The adoption of any such standard, however, is in part a matter of convenience and should be so recognized. For natural soils the objective of compaction is actually not an arbitrarily determined density, but rather, the attainment of a certain minimum soil strength or reduction in compressibility. Experience show that densities equivalent to 95 percent are adequate for normal requirements in respect to other soil properties. A high degree of compaction is unnecessary in the case of a granular lightweight aggregate which reaches a high level of stability with only two passes of rolling equipment adherence to an arbitrary standard is unjustifiable.

Typical Compacted Densities of Natural Soils

The compacted density of a natural soil is a function of the ingredients and their grading and can vary between limits of 90 pcf for organic clays to as more than 130 pcf for well graded sand and gravel. Comparisons with ESCS LA must be to the local natural soil in your area. These values for local materials are well known to soils and foundation engineers.



No.	Layers	Blows per Layer	Hammer Weight	Hammer Drop	
1	5	55	10 lb	18 in.	(mod. AASHO)
2	5	26	10	18	
3	5	12	10	18	(std. AASHO)
4	3	25	$5\frac{1}{2}$	12	

Note. 6 in. diameter mold used for all tests.

Dynamic Compaction of a Silty Clay

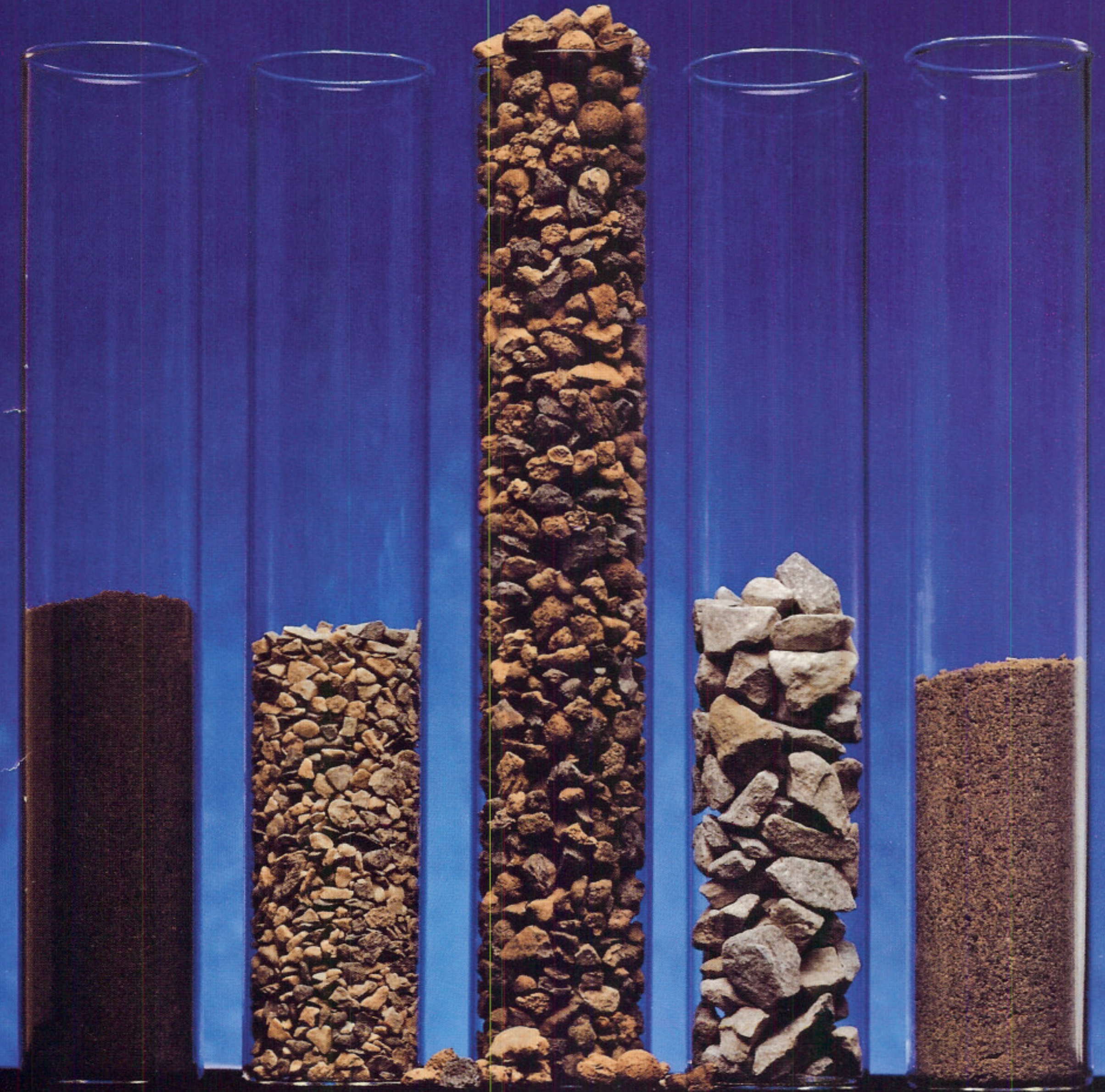
APPENDIX D

- ESCSI publication #6600 "*Compare the Difference*", pp. D1
- ESCSI Geotechnical information sheet 6001, pp. D2 – D5.
- ESCSI publication #6610, April 2001, "*ESCS Lightweight Aggregate Soil Mechanics Properties and Applications*", Holm and Valsangkar, pp. D6.
- *"Rotary Kiln Expanded shale, Clay or Slate Lightweight Aggregate for Sewer Bedding and Fill"*, ESCS No. 14, W.H. McCombs, June 1991, pp. D7 – D10
- *"Lightweight Fill Helps Albany Port Expand",* ASCE Childs, Porter, Holm, April 1983, pp. D11 – D14.
- *Determination of Density Factors for Lightweight Aggregate,* pp. D15 – D18

Rotary Kiln Produced

Lightweight Aggregate

For Geotechnical Applications



1 lb. Soil | 1 lb. Gravel | **1 lb. ESCS Lightweight Aggregate** | 1 lb. Limestone | 1 lb. Sand

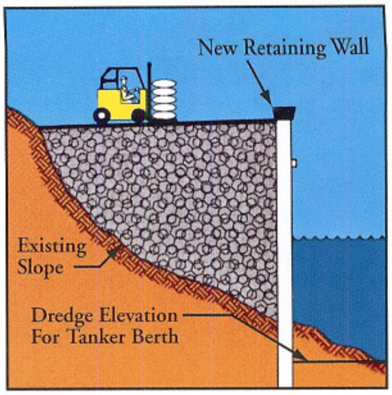
Compare The Difference

Port of Albany Marine Terminal Expansion ❖ *Albany, New York*
Engineer: Childs Engineering, Inc., Medfield, Mass.

Modifications to the Port Albany Marine Terminal reclaimed an area of approximately 1500' x 80' in an unstable slope area and provided increased dockside draft to permit service by large oil tankers. LWA backfill minimized lateral earth pressures, while also reducing overburden pressures on the sensitive silts. Transportation, placement and compaction of the LWA soil fill was readily accomplished in a minimum time frame and without logistic difficulties. Peak delivery rates were 1300 tons, approximately 55 truck-loads per day.

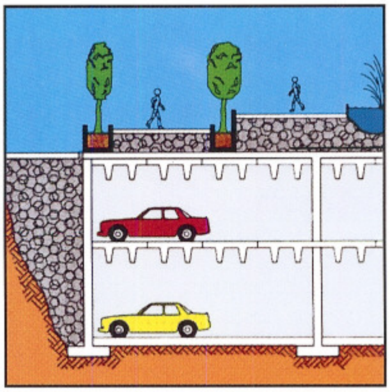
Retaining Wall Backfill ❖ *Providence Rhode Island*
Engineer: C.E. Maguire Engineers, Mansfield, Mass.

This project involved the construction of a retaining wall behind the Rhode Island State House at the Providence River. The weight of the entire project, including the wall, the backfill, and a future roadway at the top of the wall, was quite significant. With the area's soft clay strata, there were engineering concerns that too much weight might force the existing bulkhead toward the river. The use of approximately 3,500 cubic yards of LWA fill reduced the total project weight so dramatically that the probability of deep seated bulkhead failure was virtually eliminated.



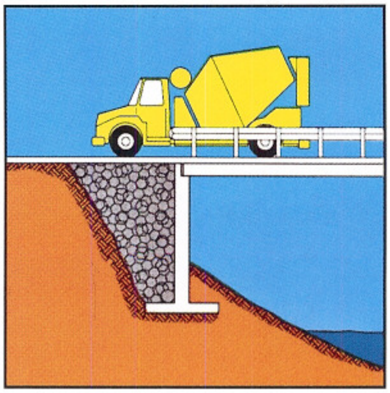
Waterfront Structures

- ❖ Allows economical modification to marine terminals
- ❖ Allows increased dockside draft
- ❖ Reduces lateral thrust/bending moments
- ❖ Allows free drainage and control of in-place density



Landscape & Plaza Fills

- ❖ Minimizes dead loads
- ❖ Free draining helps minimize hydrostatic potential
- ❖ More planters and levels can be added
- ❖ Easy to transport and install



Bulkheads & Retaining Walls

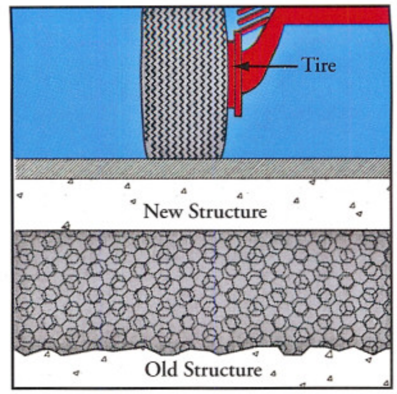
- ❖ Reduces soil thrust as well as bending moments
- ❖ Reduces forces against abutment and end slope
- ❖ Allows free drainage
- ❖ Improves embankment stability

Barney Allis Plaza ❖ *Kansas City, Missouri*
Architect/Engineer: Marshall & Brown Incorporated

6000 cubic yards of LWA (expanded shale) was used as loose granular fill on top of an existing underground parking garage. The material provided subsurface drainage, weight reduction and long term stability. In addition, the LWA material established the grade and contour for a plaza area which was built on top of the parking structure. The LWA material was graded ASTM C330 3/4" x No. 4.

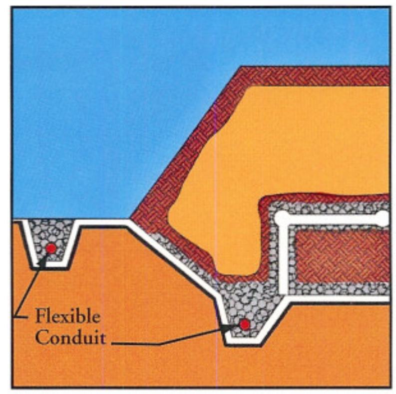
Calgary Pipeline ❖ *Calgary, Canada*
Engineers: City of Calgary / Pildysh & Associates Consultants, Ltd.

Watermains must be installed below the level of frost penetration. In Calgary this requires deep, wide trenches. Such trenches are expensive and often dangerous to workers. The insulating properties of LWA fill allowed engineers to reduce trench depth from 3.3 meters to 2.1 meters. This provided safer working conditions and reliable freeze protection with an environmentally "friendly" material. LWA backfill will also afford easier winter excavation for pipe repair, reduce disruption of water supply and street traffic by decreasing construction time, and eliminate the need for synthetic insulating board and wide trenches. With LWA backfill, present and future savings in capital cost alone are expected to be in the millions.



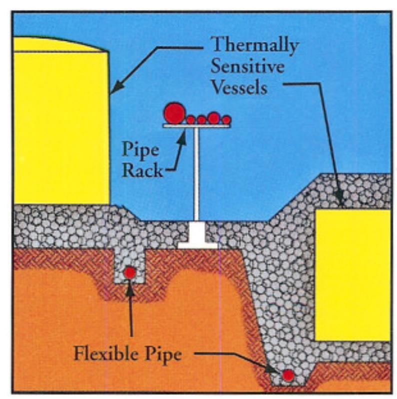
Structure Repair & Rehabilitation

- ❖ Reduces dead load on existing structures
- ❖ Easy transportation and installation increase productivity
- ❖ Precise gradations allow for a uniform and controlled in-place density



Landfill Drainage

- ❖ Inert; high chemical stability
- ❖ Reduces deadloads on pipes
- ❖ Allows free drainage of leachate/water
- ❖ Acid insoluable



Insulating Backfill

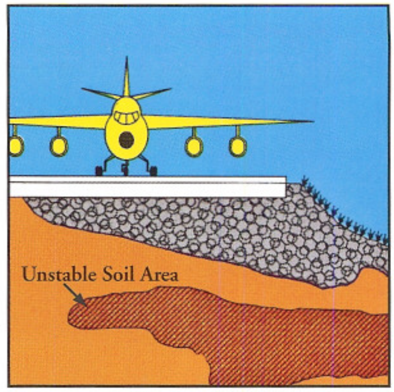
- ❖ Substantially reduces ground movement-induced stresses on buried pipes and structures
- ❖ Counteracts frost heaving, resists freeze/thaw cycles and highly insulative
- ❖ Inert, non-corrosive and stable

Runway Repair ❖ *Norfolk Naval Air Station* ❖ *Norfolk, Virginia*
Engineer: Patton, Harris, Rust & Associates

Much of this facility was built on marsh land. Poor soil conditions and intense traffic loads produced differential settlements and "alligator" cracking of the taxiway after only 3 years. High soil stability and relief from overburden pressures were provided by substituting compacted LWA for heavy, unstable soil to a depth of 4 feet. LWA material was placed at 6 inch lifts and hand compacted with a vibratory plate. Field compaction and projected yields were monitored using a nuclear densometer. The compacted base was then paved and air traffic restored in a timely manner. Differential settlement was economically solved.

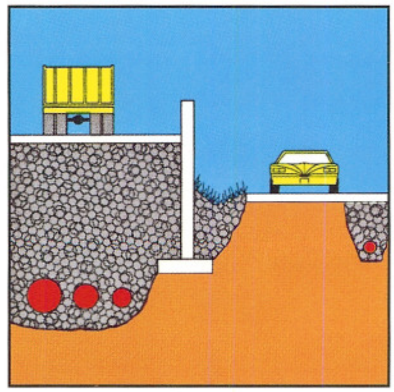
Embankment Fill ❖ *Louisiana DOT D Test Project* ❖ *Morgan City, Louisiana*

Highway embankment fills over unstable soils present particularly difficult problems. Uneven settlement can produce a "Roller Coaster" ride, as well as significant maintenance problems. The Louisiana Department of Transportation and Development constructed a series of roadway test sections with sand fill 9.5 ft. in depth. In one section, 2.5 ft. of sand was replaced with 2.5 ft. of LWA fill. The reduction in weight, coupled with the increase in long term stability provided by the LWA's high angle of internal friction, reduced settlement 40% to 60% as compared to the all-sand fill. Considerable savings in highway maintenance, repairs and replacement can be realized if differential settlement is reduced.



Fills Over Poor Soils & Marsh Lands

- ❖ Allows otherwise unuseable land to be reclaimed and developed
- ❖ Design elevations are achieved with low fill weight
- ❖ Low fill weight increases slope stability
- ❖ Controlled gradations assure uniform and consistent in-place density
- ❖ Long-term settlement is controlled and reduced
- ❖ Controlled fill allows uniform load distribution



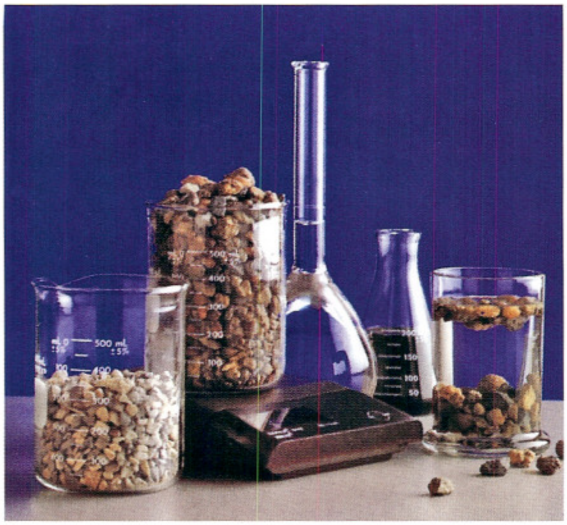
Underground Conduits & Pipelines

- ❖ Reduces dead loads on buried structures
- ❖ Allows construction of higher fills
- ❖ Minimizes hydrostatic potential
- ❖ Provides thermal insulation to underground facilities
- ❖ Economic alternative to flowable fills

Contact a Rotary Kiln Expanded Lightweight Aggregate producer listed on the back of this brochure for complete information and specifications.

Expanded Shale, Clay and Slate Lightweight Aggregate (LWA)

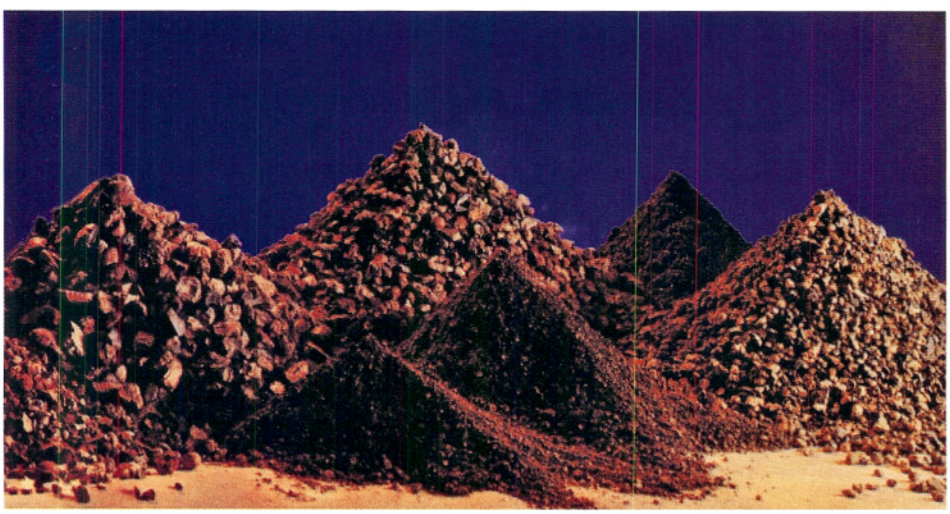
THE PROVEN SOLUTION



For almost 50 years Rotary Kiln produced Expanded Lightweight Aggregate (LWA) has been effectively used to solve geotechnical engineering problems and to convert unstable soil into usable land. Lightweight aggregate can reduce the weight of compacted geotechnical fills by up to one-half. Where thermal stability is required, LWA provides significantly greater thermal resistance when compared to soil, sand or gravel fill. It affords permanent economical insulation around water lines, steam lines and any other thermally sensitive vessel. This inert, durable, stable, free-draining and environmentally "friendly" lightweight aggregate is extremely easy to handle and provides economical long term solutions for geotechnical challenges.

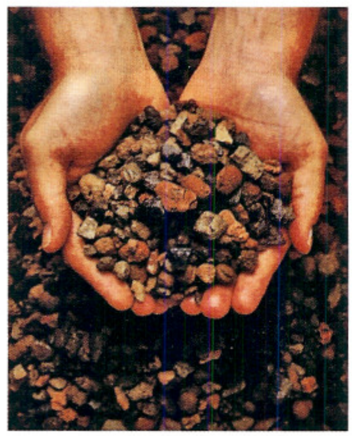
THE MATERIAL

Expanded shale, clay and slate lightweight aggregate (LWA) has a long track record of quality and performance. Since its development in the early nineteen hundreds, LWA produced by the rotary kiln process has been used extensively in asphalt road surfaces, concrete bridge decks, high-rise buildings, concrete precast/prestressed elements, concrete masonry and geotechnical applications. The quality of LWA results from a carefully controlled manufacturing process. In a rotary kiln, selectively mined shale, clay or slate is fired in excess of 2000°F. The LWA material is then processed to precise gradations. The result is a high quality, lightweight aggregate that is inert, durable, tough, stable, highly insulative, and free draining, ready to meet stringent structural specifications.



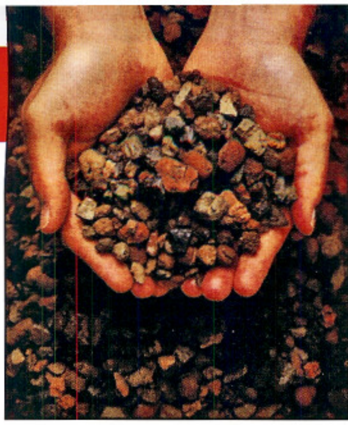
DESIGN ADVANTAGES

- Reduces Dead Loads
- Reduces Lateral Forces
- Reduces Over Turning Forces
- Provides High-Friction Angle
- Controlled Gradations
- Free Draining
- Water Insoluable
- Acid Insoluable
- High Insulation Value
- Chemically Inert
- High Strength & Durability
- Easy to Handle and Install
- Readily Available
- Environmentally "Friendly"



PHYSICAL PROPERTIES

The Physical properties for specific types of rotary kiln expanded lightweight aggregate may vary according to manufacturer. For precise information on unit weight, specific gravity, compacted density, friction angle, thermal conductivity and the other physical properties of a particular LWA material, consult the rotary kiln expanded shale, clay or slate producers listed on the back of this brochure.



Structural Lightweight Aggregate's Holistic Contribution and Commitment to Sustainable Development

The Expanded Shale, Clay and Slate Institute (ESCSI) and its member companies are committed to the long-term performance of our products and the sustainable development of the building and construction industry. Structural lightweight aggregate (LWA) contributes to sustainability when used in innovative, practical and responsible designs and construction practices, especially when included as part of life cycle cost assessment.

Structural lightweight aggregate has been successfully used in more than 50 different types of applications for well over two millennia. It has had widespread use in concrete masonry, high-rise buildings (concrete and steel frame), concrete bridge decks, marine structures, precast and prestressed concrete elements, asphalt chip seal road surfaces, and geotechnical lightweight fills. This track record of proven performance has demonstrated how LWA contributes to sustainable development by conserving energy, improving trucking efficiency of both raw materials and finished product, maximizing structural efficiency and increasing project service life. The use of LWA in site development assists designers in addressing the important environmental issue of storm water management with on-site treatment.

What is Rotary Kiln Produced Structural Lightweight Aggregate?

LWA is an environmentally friendly cellular ceramic aggregate produced in a rotary kiln from select shales, clays and slates. The process produces a high quality material that is structurally strong yet lightweight, durable, inert and highly insulative. LWA allows designers greater flexibility in creating practical, economical solutions to meet the challenges of excessive dead loads, poor soils, seismic conditions, construction schedules and energy budgets.

Expanded shale, clay and slate aggregate, as manufactured by the rotary kiln process (originally developed in 1908 and patented in 1918 as Haydite), is available throughout the world.

ESCSI

Expanded Shale, Clay and Slate Institute

2225 Murray-Holladay Road, Suite 102, Salt Lake City, Utah 84117 • (801) 272-7070 • Fax 801-272-3377

www.escsi.org

Publication # 6600

Updated 1-07

Lightweight Expanded Shale, Clay and Slate Aggregate for Geotechnical Applications

General Information

Compacted fills using Expanded Shale, Clay and Slate (ESCS) lightweight aggregates are approximately half the weight of fills using common materials. The load reduction, coupled with the high internal friction angle of the lightweight aggregate can reduce vertical and lateral forces by more than one-half. ESCS has been used to solve numerous geotechnical engineering problems and to convert soft and unstable soil into usable property. Since ESCS aggregate has high thermal resistivity, it provides durable, inorganic insulation around water and steam lines, and other thermally sensitive elements. ESCS aggregates provide a practical, reliable and economical geotechnical solution.

General Engineering Properties of ESCS 3/4" to No. 4 Aggregate Grading*

Aggregate Property	Measuring Method	Test Method	Commonly Used Specifications for ESCS	Typical Values For ESCS Aggregate	Typical Design Values For Ordinary Fills
Soundness Loss	Magnesium Sulfate	AASHTO T 104	<30%	<6 %	<6 %
Abrasion Resistance	Los Angeles Abrasion	ASTM C 131	<40 %	20 - 40%	10 - 45%
Chloride Content	Chloride Content of Soils	AASHTO T 291	<100 ppm	10 - 70 ppm	
Grading	Sieve Analysis	ASTM C 136	Comment No. 1	Comment No. 1	
Compacted In-Place Bulk Density (Unit Weight)	Density Test	Comment No. 2	<70 lb/ft³	40 - 65 lb/ft³ Moist	100-130 lbs/ft³
Stability (Phi Angle, ϕ)	Direct Shear Test Consolidated Drained Triaxial-Consolidated Drained	ASTM D 3080 Comment 3 Corps of Engineers EM 1110-2-1906 Appendix X Comment 3	Comment No. 3	35° - 45° +	30° - 38° (fine sand - sand & gravel)
Loose Bulk Density (Unit Weight)	Loose	ASTM C 29	Dry <50 lb/ft³ Saturated <65 lb/ft³	Dry 30 - 50 lb/ft³	89 -105 lb/ft³
pH	pH Meter	AASHTO T 289	5 - 10	7.0 - 10	5 - 10

*For Other Gradings See Comment No. 1

For Metric Conversion See Comment No. 5

Specifying ESCS Geotechnical Fill

Consult your expanded shale, clay or slate producer, preferably during the conceptual design phase of a project, for precise information on aggregate grading, bulk density (unit weight), in-place compacted density, friction angle, thermal conductivity, and placement method. The ESCS producer often has the ability to offer a variety of grading options. Use this versatility to specify the optimum material for any given application. As with ordinary aggregates, the engineering properties of ESCS vary depending on aggregate sources and grading.

Guide Specifications for Lightweight Geotechnical Applications

Aggregate

Lightweight aggregate shall be Expanded Shale, Clay or Slate (ESCS) produced by the rotary kiln process and meeting the requirements of ASTM C 330. Lightweight aggregate shall have a proven record of durability, and be non-corrosive, with the following properties:

Aggregate Physical Properties

A1 Soundness Loss: The maximum soundness loss shall be 30% when tested, with 4 cycles of Magnesium sulfate, in accordance with AASHTO T 104.

A2 Abrasion Resistance: The maximum abrasion loss shall be 40% when tested in accordance with ASTM C 131.

A3 Chloride Content: The maximum chloride content shall be 100 ppm when tested in accordance with AASHTO T 291.

A4 Grading: Aggregate grading comes in a wide variety of sizes and is specified based on performance needs. Grading shall be tested in accordance with ASTM 136. (*See Comment No. 1*)

Project Performance Specification

B1 In-place bulk density (unit weight): The maximum in-place compacted moist density shall be _______ lbs/ft^3 when tested in accordance with the method specified by the engineer. (*See Comment No. 2*)

B2 Stability (Phi angle, Φ): The minimum angle of internal friction Φ shall be _______ degrees when tested in accordance with the method specified by the engineer. (*See Comment No. 3*)

Construction

C1 Method of Construction: Lightweight fill shall be placed in uniform layers. The actual lift thickness, and exact number of passes by equipment used will be determined by the engineer, depending on the project requirements (i.e., stability, compaction, density).

In confined areas vibratory plate compaction equipment shall be used (5 hp to 20 hp) with a minimum of two passes in 6" lifts for a 5 hp plate and 12" lifts for a 20 hp plate.

The contractor shall take all necessary precautions when working adjacent to the lightweight fill to ensure that the material is not over compacted. Construction equipment, other than for placement and compaction, shall not operate on the exposed lightweight fill.

C2 Aggregate loose bulk density (unit weight): The maximum aggregate loose bulk density _______ shall be ______ lbs/ft^3 when tested in accordance with ASTM C 29. (*See Comment No. 4*)

Comments

1. Grading: ESCS aggregates are available in a wide variety of grading, therefore it is essential the specifier contact the ESCS supplier for the gradings that are available in a given location. Some common gradings are 3/4" to No 4, 1/2" to No. 4, 3/8" to No. 8, 3/8" to 0, 2" to 3/4", 2" to 0 or blends of these. ESCS aggregate suppliers can be found on ESCSI's website at **www.escsi.org.**

2. Several methods have been used to determine the in-place moist bulk density (unit weight) of a given aggregate, therefore contact the ESCS producer for recommendation on local practices. The following methods have proven performance:

> A. The lightweight aggregate producer shall submit verification of a compacted moist density of less than _______lb/ft³ when measured by a one point proctor test conducted in accordance with a modified version of ASTM D 698 "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort." Because of the cohesionless nature of coarse lightweight aggregate, the standard shall be modified as follows: The aggregate sample shall be placed in a 0.5 cubic foot bucket at the moisture content that the aggregate will be delivered to the jobsite. The sample shall be placed in three equal layers and compacted by dropping a 5.5 pound rammer from a distance of 12 inches 25 times on each layer (AASHTO T-99 modified as above).
>
> B. Material shall be compacted to a minimum 65% relative density as determined by ASTM 4253 and D 4254. Determine the maximum index density and unit weight by using a vibratory table when tested in accordance with ASTM D 4253. The minimum index density and unit weight is determined when aggregate is tested in accordance with ASTM D 4254.

3. ESCS Lightweight Aggregate has been tested by both Direct Shear and Triaxial test methods. With either method, the phi angle will vary in both ordinary and ESCS fill, depending on test procedure, aggregate grading, particle angularity, amount of compaction and amount of consolidating stress applied during the test. Design and specify the minimum phi angle appropriate for the project design and material(s) that are contemplated for use in the project. Contact the ESCS supplier(s) for specific properties of their materials.

> Direct Shear: The minimum angle of an internal friction shall be tested in accordance with ASTM D 3080 on a saturated representative sample (with particles larger than 0.75 inch removed) and tested in a round or square shear box that is a minimum of 12 inches across. Follow the procedure in D 3080 or shear the box at a rate of 0.01 inches per minute at normal loads of 250, 500 and 1,000 pounds per square foot.

4. For quality control and shipment quantities, the purchaser and supplier should agree on a maximum delivered loose bulk density (unit weight).

5. To convert bulk density (unit weight) in lb/ft^3 to metric kg/m^3, multiply by 16. To convert inches (in) to millimeters (mm) multiply by 25.4.

REFERENCE DOCUMENTS

ASTM Documents:

C 29 Standard Test Method for Unit Weight and Voids in Aggregate.

C 88 Standard Test Method for Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate.

C 131 Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregates by Abrasion and Impact in the Los Angeles Machine.

C 136 Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates

C 330 Standard Specification for Lightweight Aggregate for Structural Concrete.

D 698 Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft.-lbf/ft^3 (600 kN-m/m^3).

D 3080 Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

D 4253 Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table.

D 4254 Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density.

AASHTO Documents:

T 99-01 Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in) Drop

T 104 Standard Method of Test for Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate

T 260 Standard Method of Test for Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials.

T 288 Standard Method of Test for Determining Minimum Laboratory Soil Resistivity.

T 289 Standard Method of Test for Determining ph of Soil for Use in Corrosion Testing

T 291 Standard Method of Test for Determining Water Soluble Chloride Ion Content in Soil

T 290 Standard Method of Test for Determining Water Soluble Sulfate Ion Content in Soil

US Army Corps of Engineers Documents:

Engineer Manual, EM 1110-2-1906 Laboratory Soils Testing. Appendix X, Consolidated Drained Triaxial Test

Additional Information

Expanded Shale, Clay and Slate Institute

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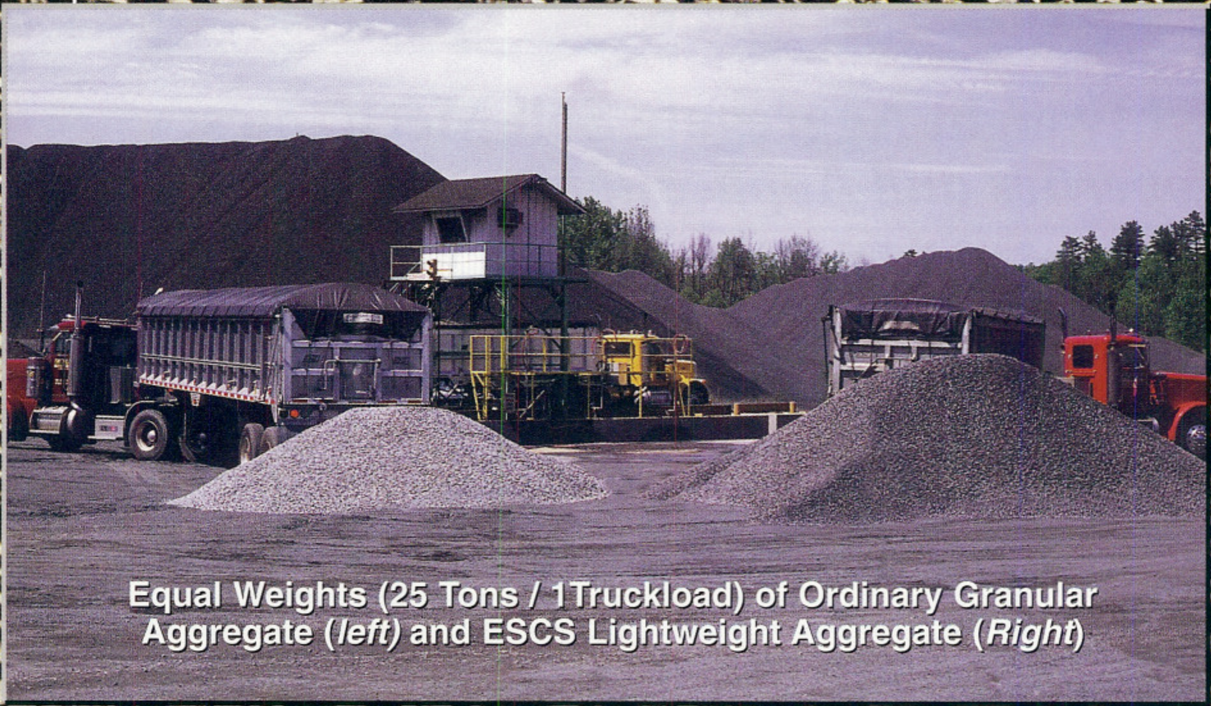
GEOTECH

Expanded Shale, Clay and Slate (ESCS) Lightweight Aggregate Soil Mechanics

Properties and Applications



Equal Weights (25 Tons / 1Truckload) of Ordinary Granular Aggregate (*left*) and ESCS Lightweight Aggregate (*Right*)

Expanded Shale, Clay and Slate (ESCS) compacted geotechnical fills are approximately half the weight of ordinary aggregate fills. This advantage, coupled with the high angle of internal friction of ESCS, can also reduce lateral forces by more than one-half. ESCS has been effectively used to solve numerous geotechnical engineering problems and to convert unstable soil into usable land. ESCS is a reliable, economical geotechnical solution.

ABSTRACT of the following paper: *Structural grade lightweight 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, and are now being widely used for geotechnical applications. Structural grade LWA, when used in backfills and over soft soils, provides geotechnical physical properties that include reduced density, high stability, high permeability, and high thermal resistance. These improved physical properties are found in aggregates with a reduced specific gravity and a predictable stability resulting from a consistently high angle of internal friction. The open texture available from a closely controlled manufactured aggregate gradation ensure high permeability. High thermal resistance results from porosity developed during the production process. In this publication, the physical properties of structural grade LWA and geotechnical engineering properties of LWA backfills are illustrated. Additionally, references to extensive testing programs that developed data on shear strength, compressibility, durability, and in-place density are given. Representative case studies are reported from [several hundred] projects that illustrate completed applications of structural grade LWA fills over soft soils and behind retaining walls and bridge abutments.*

Lightweight Aggregate Soil Mechanics: *Properties and Applications*

T.A. Holm and A.J. Valsangkar

For more than [80] years, shales, clays, and slates have been expanded in rotary kilns to produce structural grade LWA for use in concrete and masonry units. Millions of tons of structural grade LWA produced annually are used in structural concrete applications. Its availability is currently widespread throughout most of the industrially developed world. Consideration of structural grade LWA as a remedy to geotechnical problems stems primarily from the improved physical properties of reduced dead weight, high internal stability, high permeability, and high thermal resistance. These significant advantages arise from the reduction in particle specific gravity, stability that results from the inherent high angle of internal friction, the controlled open-textured gradation available from a manufactured aggregate which assures high permeability, and the high thermal resistance developed because of the high particle porosity.

PHYSICAL PROPERTIES OF STRUCTURAL LIGHTWEIGHT AGGREGATES

Particle Shape and Gradation

As with naturally occurring granular materials, manufactured LWA's have particle shapes that vary from round to angular with a characteristically high interstitial void content that results from a narrow range of particle sizes. Applications of LWA to geotechnical situations require recognition of two primary attributes: (a) the high interstitial void content typical of closely controlled manufactured granular coarse aggregate that closely resembles a clean, crushed stone, and (b) the high volume of pores enclosed within the cellular particle.

Structural grade LWA gradations commonly used in high-rise concrete buildings and long-span concrete bridge decks conform to the requirements of ASTM C330. The narrow range of particle sizes ensures a high interstitial void content that approaches 50% in the loose state. North American rotary kiln plants producing expanded shales, clays, and slates currently supply coarse [and fine] aggregates to ready-mix and precast concrete manufacturers with 20 to 5 mm (3/4 - #4), 13 to 5 mm (1/2 - #4), or 10 to 2 mm (3/8 - #8) gradations [and various fine aggregate gradings]. With [coarse] gradations there is a minimum percentage of fines smaller than 2 mm (#8 mesh) and insignificant amounts passing the 100 mesh screen.

Retaining wall backfill, Providence, Rhode Island

Particle Porosity and Bulk Density

When suitable shales, clays, and slates are heated in rotary kilns to temperatures in excess of 1100° C (2012°F), a cellular structure is formed of essentially noninterconnected spherical pores surrounded by a strong, durable ceramic matrix that has characteristics similar to those of vitrified clay brick. Oven-dry specific gravities of LWA vary but commonly range from 1.25 to 1.40. Combination of this low specific gravity with high interparticle void content results in LWA bulk dry densities commonly in the range of 720 kg/m^3 (45 pcf). Compaction of expanded aggregates in a manner similar to that used with crushed stone provides a highly stable interlocking network that will develop in-place moist densities of less than [960 kg/m^3 (60 pcf)].

Differences in porosity and bulk density between LWA's and ordinary soils may be illustrated by a series of schematic depictions. For comparative purposes, Figure 1 shows the interparticle voids in ordinary coarse aggregate. Although normal weight aggregates commonly have porosities of 1-2%, the schematic assumes ordinary aggregates to be 100% solid. For illustrative purposes, the bulk volume is shown to be broken into one entirely solid part with the remaining fraction being interparticle voids.

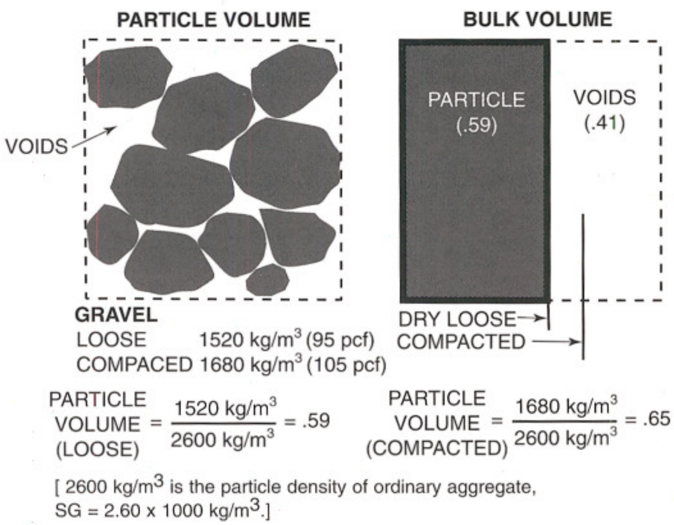


FIGURE 1 Voids in ordinary coarse aggregates

Figure 2 shows the cellular pore structure of a typical LWA. ASTM procedures prescribe measuring the "saturated" (*misnamed in the case of LWA's; partially saturated after a 1-day soak is more accurate*) specific gravity in a pycnometer and then determining the moisture content on the sample that had been immersed in water for 24 hours. After a 1-day immersion in water, the rate of moisture absorption into the lightweight aggregate will be so low that the partially saturated specific gravity will be essentially unchanged during the time necessary to take weight measurements in the pycnometer. When the moisture content is known, the oven-dry specific gravity may be directly computed. This representative coarse LWA with a measured dry loose bulk unit weight of 714 kg/m³ (44.6 pcf) and computed oven-dry specific gravity of 1.38 results in the aggregate particle occupying 52% of the total bulk volume, with the remaining 48% composed of interparticle voids.

The specific gravity of the pore-free ceramic solid fraction of a lightweight aggregate may be determined by standard procedures after porous particles have been thoroughly pulverized in a jaw mill. Pore-free ceramic solids specific gravities measured on several pulverized LWA samples developed a mean value of 2.55. The representative LWA with a dry specific gravity of 1.38 will develop a 54% fraction of enclosed aggregate particle ceramic solids and a remaining 46% pore volume (Figure 2).

This leads to the illustration of the overall porosity in a bulk loose LWA sample as shown in Figure 3. Interparticle voids of the overall bulk sample are shown within the enclosed dotted area, and the solid pore-free ceramic and the internal pores are shown within the solid particle lines. For this representative LWA, the dry loose bulk volume is shown to be composed of 48% voids, 28% solids, and 24% pores.

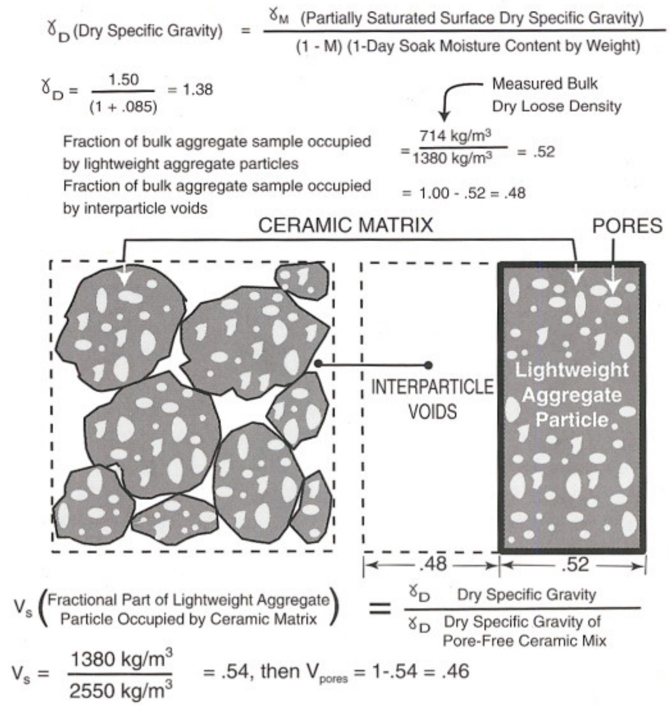
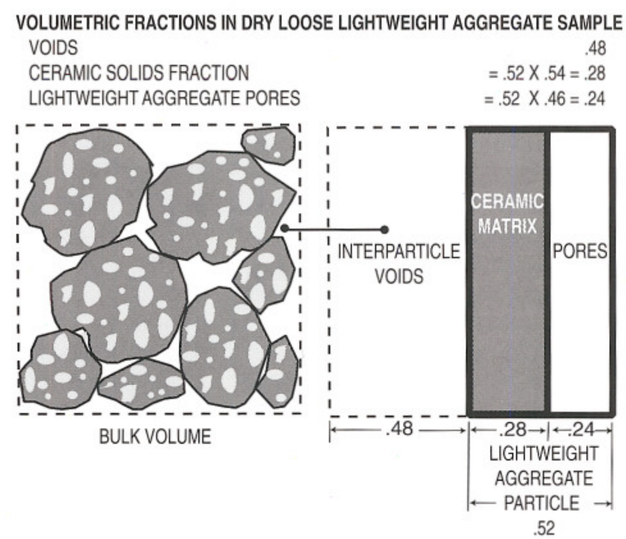


FIGURE 2 Interparticle voids and within-particle pores of lightweight aggregate (LWA)



LOOSE AGGREGATE CONDITION	INTERPARTICLE VOIDS	CERAMIC MATRIX	PORES	DENSITY kg/m³
DRY	–	714	–	714 (44.6 pcf)
PARTIALLY SATURATED ONE-DAY DRY SOAK	–	714	61	775 (48.4 pcf)
VACUUM SATURATION	–	714	240	954 (59.6 pcf)
LONG TIME SATURATION [Submerged]	480	714	240	1434 -1000 = *434 (27.1 pcf)

*Buoyant Unit Weight

FIGURE 3 Voids, pores, and ceramic matrix fraction in a lightweight aggregate (LWA) sample

Absorption Characteristics

LWA's stored in exposed stockpiles in a manner similar to crushed stone will have some internal pores partially filled and may also carry an adsorbed moisture film on the surface of the particles. The moisture content that is defined in ASTM procedures as "absorption" based on a 24-hour immersion and routinely associated in concrete technology with "saturated" surface-dry specific gravity is, in fact, a condition in which considerably less than 50% of the particle pore volume is filled.

The issue is further clarified by a schematic volumetric depiction (see Figure 4) of the degree of pore volume saturation of a LWA particle that shows that the sample had a measured damp loose bulk unit weight of 775 kg/m³ (48.4 pcf) with an 8.5% absorbed moisture and would, in fact, represent

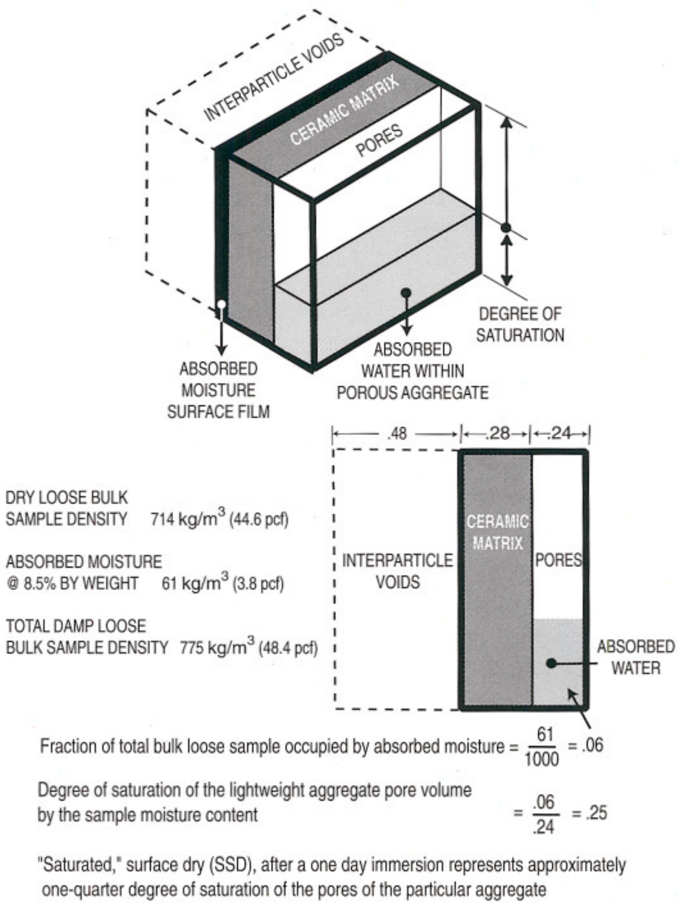


FIGURE 4 Degree of saturation of partially saturated lightweight aggregate (LWA)

a condition in which approximately 25% of the pore volume is water filled.

Structural grade LWA exposed to moisture in production plants and stored in open stockpiles will contain an equilibrium moisture content. LWA's that are continuously submerged will, however, continue to absorb water over time. In one investigation, the effective specific gravity of a submerged LWA sample was measured throughout a one-year period to demonstrate long-term weight gain. Long-term

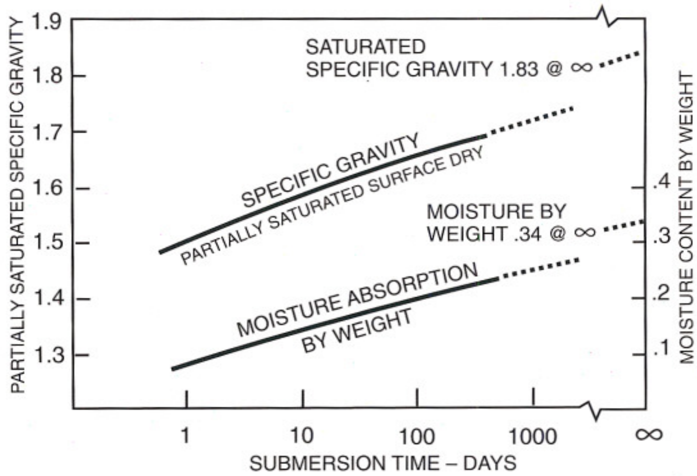


FIGURE 5 Moisture absorption (by weight) and partially saturated, surface dry specific gravity of lightweight aggregate (LWA) versus time of submersion

absorption characteristics are shown in Figure 5 for a LWA sample with a measured 1-day immersion moisture content of 8.5% associated with a partially saturated surface-dry specific gravity of 1.5. When moisture absorption-versus-time relationships are extrapolated or theoretical calculations used to estimate the total filling of all the LWA pores, it can be shown that for this particular LWA the absorbed moisture content at infinity will approach 34% by weight with a totally saturated specific gravity of 1.83. Complete filling of all pores in a structural grade LWA is unlikely because the non-interconnected pores are enveloped by a very dense ceramic matrix. However, these calculations do reveal a conservative upper limit for submerged design considerations.

Durability Characteristics

The durability of LWA's used in structural concrete applications is well known. More than 400 major U.S. bridges built using structural lightweight concrete (LWC) have demonstrated low maintenance and limited deterioration. Long-term durability characteristics of LWA's were demonstrated in 1991 by reclaiming and testing samples of the LWA fill supplied in 1968 to a Hudson River site. Magnesium soundness tests conducted on the reclaimed aggregate sample exposed to long-term weathering resulted in soundness loss values comparable to those measured and reported in routine quality control testing procedures 23 years earlier, indicating little long-term deterioration due to continuous submersion and freeze-thaw cycling at the waterline.

Although ASTM standard specifications C330 and C331 for lightweight aggregate make no mention of corrosive chemicals limitations, foreign specifications strictly limit SO_3 equivalents to 0.5% (*Japanese Industrial Standard J5002*) or 1.0% (*German Standard DIN 4226*). The American Concrete Institute Building Code (ACI 318) mandates chloride limitations in the overall concrete mass because of concern for reinforcing bar corrosion, but no limits are specified for individual constituents. Numerous geotechnical projects specifications calling for lightweight aggregates have limited water-soluble chloride content in the aggregate to be less than 100 ppm when measured by AASHTO T291.

GEOTECHNICAL PROPERTIES OF LIGHTWEIGHT FILL

In-Place Compacted Moist Density

Results of compacted LWA density tests conducted in accordance with laboratory procedures (Proctor tests) should be interpreted differently from those for natural soils. Two fundamental aspects of lightweight aggregate soil fill will modify the usual interpretation soils engineers place on Proctor test data. The first is that the absorption of LWA is greater than natural soils. Part of the water added during tests will be absorbed within the aggregate particle and will not affect interparticle physics (bulking, lubrication of the surfaces, etc.). Second, unlike cohesive natural soils, structural grade LWA contains limited fines, limiting the increase in density due to packing of the fines between large particles. The objective in compacting structural grade LWA fill is not to aim for maximum in-place density, but to strive for an optimum density that provides high stability without unduly increasing compacted density. Optimum field density is commonly achieved by two to four passes of rubber tire equipment. Excessive particle degradation developed by steel-tracked rolling equipment should be avoided. Field density may be approximated in the laboratory by conducting a one-point ASTM D698, AASHTO T99 Proctor test [using a 0.5 $ft.^3$ bucket] on a representative LWA sample that contains a moisture content typical of the field delivery. Many projects have been successfully supplied where specifications called for an in-place, compacted, moist density not to exceed 960 kg/m^3 (60 pcf).

Shear Strength

Structural grade LWA's provide an essentially cohesionless, granular fill that develops stability from inter-particle friction. Extensive testing on large 250 x 600 mm (10 x 24 in. high) specimens has confirmed angles of internal friction of more than 40 degrees (*1*). Triaxial compression tests completed on LWA from six production plants, which included variations in gradations, moisture content, and compaction levels, revealed consistently high angles of internal friction. With a commonly specified in-place moist compacted unit weight less than 960 kg/m^3 (60 pcf), it may be seen from a simplistic analysis that lateral pressures, overturning moments, and gravitational forces approach one-half of those generally associated with ordinary soils.

A summary of the extensive direct shear testing program conducted by Valsangkar and Holm (*2*), presented in the following table, confirm the high angle of internal friction measured on large-scale triaxial compression testing procedures as reported earlier by Stoll and Holm (*1*).

	Angle of Internal Friction (degree)	
Material	***Loose***	***Compacted***
Minto [LWA]	40.5	48.0
Solite [LWA]	40.0	45.5
Limestone	37.0	N/A
Solite (*1*) [LWA]	39.5	44.5

Compressibility

Large-scale compressibility tests completed on lightweight aggregate fills demonstrated that the curvature and slope of the LWA fill stress-strain curves in confined compression were similar to those developed for companion limestone samples (*2*). Cyclic plate-bearing tests on LWA fills indicated vertical subgrade reaction responses that were essentially similar for the lightweight and normal weight aggregate samples tested (*3*).

Attempts by concrete technologists to estimate aggregate strength characteristics by subjecting unbound LWA samples to piston ram pressures in a confined steel cylinder have provided inconsistent and essentially unusable data for determination of the strength making characteristics of concretes that incorporate structural grade LWA. By ASTM C330 specifications, all structural grade LWA's are required to develop concrete strengths above 17.2 MPa (2500 psi). Most structural grade LWA concrete will develop 34.4 MPa (5000 psi), and a small number can be used in concretes that develop compressive strengths greater than 69 MPa (10,000 psi).

Thermal Resistance

For more than [8] decades, design professionals have used lightweight concrete masonry and lightweight structural concrete on building facades to reduce energy losses through exterior walls. It is well demonstrated that the thermal resistance of LWC is considerably less than that of ordinary concrete, and this relationship extends to aggregates in the loose state (4).

Permeability

Attempts to measure permeability characteristics of unbound LWA have not been informative because of the inability to measure the essentially unrestricted high flow rate of water moving through open-graded structure. This characteristic has also been observed in the field, where large volumes of water have been shown to flow through LWA drainage systems. Exfiltration applications of LWA have demonstrated a proven capacity to effectively handle high volumes of storm water runoff. Subterranean exfiltration systems have provided competitive alternatives to infiltration ponds by not using valuable property areas as well as eliminating the long-term maintenance problems associated with open storage of water.

Interaction Between Lightweight Aggregate Fills and Geotextiles

Valsangkar and Holm (5) reported results of testing programs on the interaction between geotextiles and LWA fills that included the variables of differing aggregate types and densities, thickness of aggregate layer, and geotextile types. The results indicated that the overall roadbed stiffness is unaffected when LWA is used instead of normal weight aggregate for small deflections and initial load applications. These tests were followed by a large-scale test (2), which reported that the comparison of the friction angles between the LWA or the normal weight aggregate and the geotextiles indicate that interface friction characteristics are, in general, better for LWA than normal weight aggregates.

During the past decade several hundred diverse geotechnical applications have been successfully supplied with structural grade LWA. The applications primarily fit into the following major categories:

- **Backfill behind waterfront structures, retaining walls, and bridge abutments;**
- **Load compensation and buried pipe applications on soft soils;**
- **Improved slope stability situations; and**
- **High thermal resistance applications.**

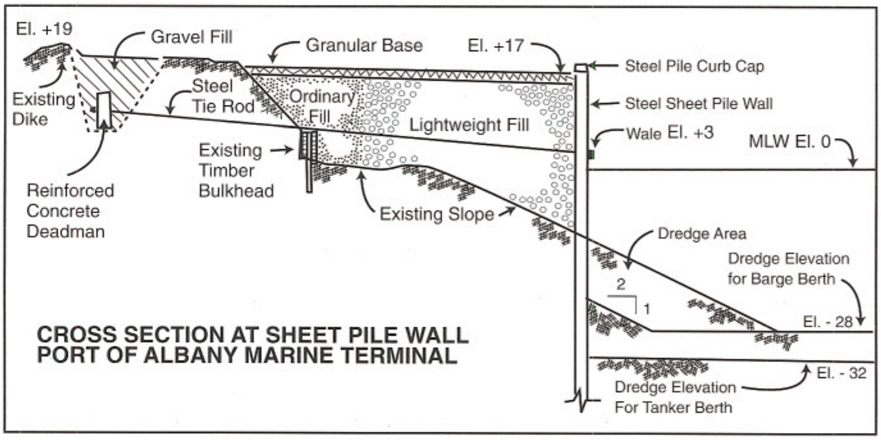


FIGURE 6 Rehabilitation of Port of Albany, New York [1981]

Backfill Behind Waterfront Structures, Retaining Walls and Bridge Abutments

A classic example of how unusable river front was reclaimed and large industrial site extended by the use of sheet piles and lightweight fill is demonstrated in Figure 6 (*6*). LWA fill specifications for this project required rotary kiln expanded shale to have a controlled coarse aggregate gradation of 20 to 5 mm (3/4 - #4) and laboratory test certification of an angle of internal friction greater than 40 degrees. No constructability problems were experienced by the contractor while transporting, placing, and compacting the LWA soil fill. Peak shipment were more than 1,000 tons per day without any logistical difficulties. The material was trucked to the point of deposit at the job site and distributed by front-end loaders. This project used approximately 20,000 m^3 (27,000 yd^3) of compacted LWA and resulted in overall savings by reducing sizes of sheet piling and lowering costs associated with the anchor system.

On the Charter Oak Bridge project, Hartford, Connecticut, constructed in 1989 to 1990, LWA fill was placed in the east abutment area to avoid placing a berm that would have been necessary to stabilize an earth fill embankment. According to the designer, construction of a berm would have required relocating a tributary river. LWA fill was also used in other areas to avoid increasing stresses and settlements in an old brick sewer (*7*). When all applications were totaled, this project incorporated more than 100,000 tons of structural grade LWA.

Load Compensation and Buried Pipe Applications on Soft Soils

In numerous locations throughout North America, design of pavements resting on soft soils has been facilitated by a "load compensation" replacement of heavy soils with a free-draining structural grade LWA with low density and high stability. Replacing existing heavy soil with LWA permits raising elevations to necessary levels without providing any further surcharge loads to the lower-level soft soils. Rehabilitation of Colonial Parkway near Williamsburg, Virginia, built alongside the James and York rivers, provides a representative example of the procedure. Soft marsh soil sections of this roadway had a low load-bearing capacity, and had experienced continuous settlement. The concrete roadway slabs were removed along with the soil beneath to a depth of more than 3 ft. The normal weight soil was then replaced with structural-grade LWA with a compacted moist density of less than 960 kg/m^3 (60 pcf), providing effective distribution of load to the soft soil layer, load compensation, and side slope stability. Reconstruction was completed in two stages by first completely rehabilitating in one direction, followed by excavation of the opposing lane with delivery, compaction, and slab construction routinely repeated.

Construction of pipelines in soft soil areas has frequently been facilitated by equalizing the new construction weight (pipe plus LWA backfill) to the weight of the excavated natural soil. Supporting substrates do not "see" any increased loading and settlement forces are minimized.

Improved Slope Stability

Improvement of slope stability has been facilitated by LWA in a number of projects prone to sliding. Waterside railroad tracks paralleling the Hudson River in the vicinity of West Point, New York, had on several occasions suffered serious misalignment due to major subsurface sliding because of soft clay seams close to grade level. After riverbank soil was excavated by a barge-mounted derrick, LWA was substituted and the railroad track bed reconstructed. Reduction of the gravitational force driving the slope failure combined with the predictable LWA fill frictional stability provided the remedy for this problem. Troublesome subsoil conditions in other area, including the harbors in Norfolk, VA, and Charleston, SC, have also been similarly remedied.

High Thermal Resistance Applications

Structural LWA has been effectively used to surround high-temperature pipelines to lower heat loss. Long-term, high-temperature stability characteristics can be maintained by aggregates that have already been exposed to temperatures of 1100° C (2012° F) during the production process. Other applications have included placing LWA beneath heated oil processing plants to reduce heat flow to the supporting soils.

ECONOMICS

An economic solution provided by a design that calls for an expensive aggregate requires brief elaboration. In many geographical areas, structural-grade LWA's are sold to ready-mix, precast, and concrete masonry producers on the basis of a price per ton, FOB the plant. On the other hand, the contractor responsible for the construction of the project bases costs on the compacted material necessary to fill a prescribed volume. Because of the significantly lower bulk density, a fixed weight of this material will obviously provide a greater volume. To illustrate that point, one may presume that if a LWA is available at $X/ton, FOB the production plant, and trucking costs to the project location call for additional $Y/ton, the delivered job site cost will be $(X+Y)/ton. As mentioned previously, many projects have been supplied with structural LWA aggregates delivered with a moist, loose density of about [770 kg/m^3 (48 pcf)] and compacted to a moist, in-place density [less than the typically specified 960 kg/m^3 (60 pcf)]. This would result in an in-place, compacted moist density material cost (*not including compaction cost*) of

$$\{\$(X + Y) \times 60 \times 27\}/2{,}000$$

for the compacted, moist lightweight aggregate.

[*Additional Economic Benefits* - April 2001]

- Approximately twice as much volume of LWA can be transported per load as compared to normal weight.
- In restricted or commercial areas, cutting the number of trucks by half is environmentally significant.
- Loader or crane bucket volume can be increased to allow faster placement and longer reaches.
- In tight spaces where hand placement and compaction is required, LWA is much easier to handle and offers considerable labor savings.

CONCLUSIONS

Structural grade LWA fills possessing reduced density, high internal stability, and high permeability have been extensively specified and used to replace gravel, crushed stone, and natural soils for geotechnical applications at soft soil sites and in backfills where the assured reduction in lateral and gravitational forces has provided economical solutions.

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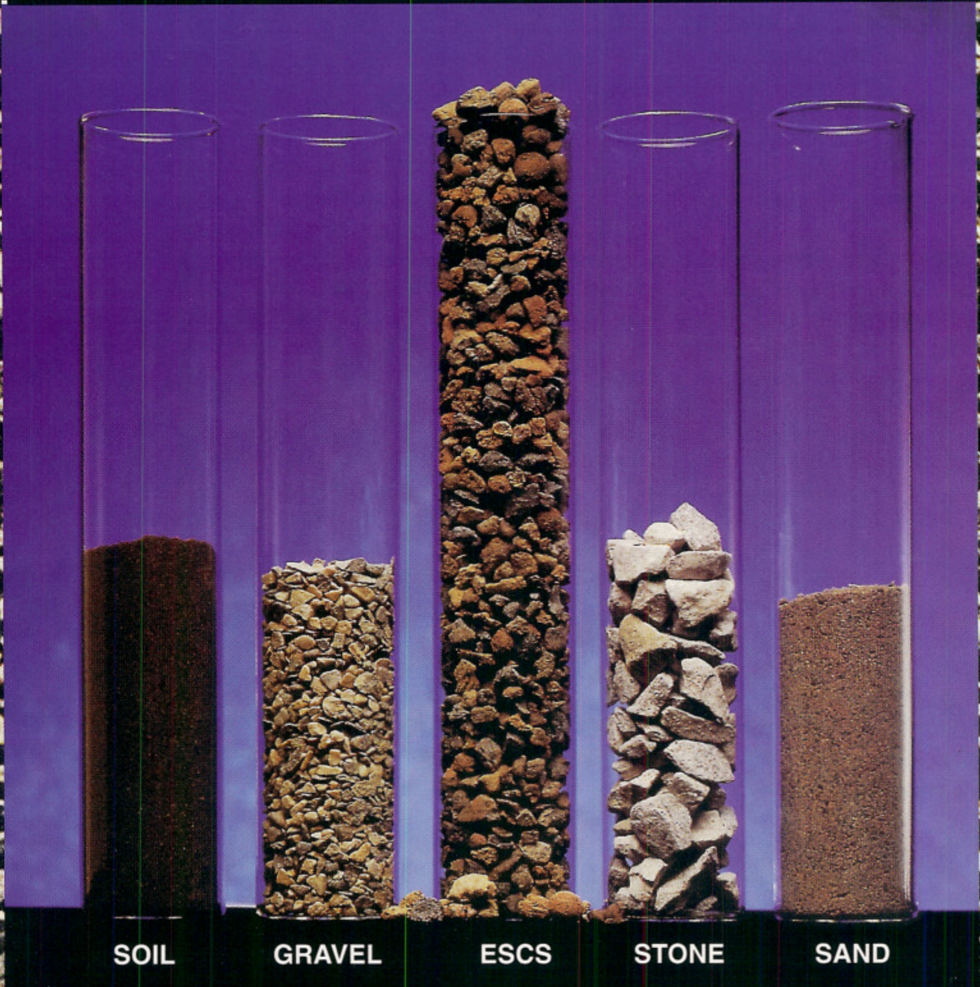
The principal author, Thomas Holm, has supervised this reprinting by the Expanded Shale, Clay and Slate Institute. Under his direction, Figures 1, 2, and 3 have been modified for clarity. Other updated information is indicated by brackets [ ].

ESCS Structural Lightweight Aggregate Is Approximately Half the Weight of Ordinary Aggregates



Equal Weights of Aggregates

(Note: ESCS is approximately twice the volume)

ESCSI

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Printed in U.S.A.

Lightweight Aggregate

No. 1A

INFORMATION SHEET

EXPANDED SHALE CLAY AND SLATE INSTITUTE, SALT LAKE CITY, UTAH, 84117

ROTARY KILN EXPANDED SHALE, CLAY OR SLATE LIGHTWEIGHT AGGREGATE FOR SEWER BEDDING AND FILL

RECOMMENDED SPECIFICATION

Lightweight aggregate to be used as fill or pipe bedding in unstable foundation areas shall be Rotary Kiln Expanded Shale, Clay or Slate, or approved equal. The loose volume unit weight shall not exceed 55 pounds per cubic foot. The aggregate shall be graded in accordance with ASTM Specification C-330 for 3/4 inch to No. 4.

LIGHTWEIGHT AGGREGATE
... for better and less costly sewer construction

By WILLIAM H. McCOMBS
McCombs-Knutson Associates, Inc.
Consulting Engineers
Plymouth, Minnesota

Revised and reprinted from the August 1972 issue of THE AMERICAN CITY Magazine.

Unfavorable soils make the construction of sewers, forcemain, and other underground utilities expensive and risky. If firm soil underlays unstable soil within five to six feet below the pipe, the unstable soil can be removed and replaced with washed rock in sizes from 3/4 to 2 inches. But this method becomes too costly and ineffective when the unstable soil extends beyond the five to six foot depth. Also, the use of rock can result in a "sloughing off" since it is appreciably heavier that the surrounding soil, permitting the pipe to settle and crack.

Pipe foundation may also be used, but increases the cost approximately four to six times that of pipe laid in good soils. Also the piling must be constructed and located where a subsequent load placed on the pipe by traffic or other changes in surface conditions will not result in the vertical failure of the pipe or the shifting of soils which could produce lateral movement of the pipe.

A third method employs lightweight pipe such as corrugated metal and plastic that floats on the unstable soil bedding. The theory here is that the lightweight pipe would stay in place making removal of the unstable soil unnecessary. However, construction of this type has generally resulted in failure of the pipe and subsequent replacement.

Minnesota is particularly plagued with areas of unfavorable soil conditions, especially adjacent to our numerous lakes and low swamp areas, where it is most often necessary to construct the sanitary and storm sewers. Our office has encountered several projects where the soils made the construction so expensive that it was not economically feasible to construct the facilities. Thus we had to find a better method of construction.

Our investigation centered around finding a material that could be placed under the pipe similar to the rock method of construction, and yet would be lighter than the existing soil. It would not experience the sloughing problems and in some cases could serve as a bridge across some marginal soils. On the job in question, located in the city of Medicine Lake, the major type of soil in the construction area was peat having a weight of 55 to 65 pounds per cubic foot. Thus we needed a material that would be less than this weight, yet would be strong, inert, insoluble, and noncorrosive to the pipe.

A lightweight aggregate similar to that used in the manufacture of lightweight concrete and concrete block seemed promising. Tests showed the lightweight aggregate to be light in weight, hard, durable, inert, and insoluble - all the properties needed for a good foundation material for underground piping.

To find out the bridging ability of the material we dug a trench and placed and compacted the lightweight aggregate. We then exposed the material by cutting away the side of the trench and finally dug a hole underneath the aggregate to observe its ability to carry a load.

Satisfied with test results, we used the material for the sewer foundations under PVC in the city of Medicine Lake. The unstable soils were removed (Figure 1) to a good foundation material and lightweight aggregate placed to the approximate centerline of the pipe. In some cases we placed the lightweight aggregate above the pipe to decrease the overall weight. Heavy compaction equipment was employed to thoroughly compact the material without adverse effect. Experience showed that the light material compacted well and provided a tighter, firmer, drier base on which to work than would have been obtained had a rock material been used.

FIGURE 1: Tested in the field under difficult conditions, the use of lightweight aggregate as bedding for a sewer line proved successful.

The material was also much lighter and easier to handle, resulting in savings in labor cost because of the ease and speed in constructing the pipe bed. Television inspection of the lines conducted four years after the original construction showed that the line and grade of the pipe was still to the true grade and no settling or damage had taken place.

As a result of this construction, the weight of the lightweight aggregate including the pipe construction was less than the weight of the soil removed. The successful experience on this first application led to a subsequent test area on a storm sewer. However, instead of removing all of the unstable subsoils we removed only sufficient material so that the combined weight of the pipe, the water inside the pipe, and the foundation material was less than that of the soil removed. Figure 2 gives an example of different bedding conditions utilized. Subsequent checks of this storm sewer showed that this pipe also stayed true to line and grade.

We have employed similar methods in other areas with equally satisfying results and found that they result in lower construction costs. The cost of the material is about the same as rock when considered on a volume basis, but it is much easier to handle. Also, the dollar savings by the elimination of the need for piling, and/or possible replacement where heavier materials settle and the pipe fails could be substantial.

We feel that lightweight aggregate when used in conjunction with unstable subsoils has a very wide application. However, the engineer must use caution in its application to insure that the underlying soil is stable enough to carry the lightweight material, and that it will not settle into the subsoil. Thus it should not be used in "soupy" soils.

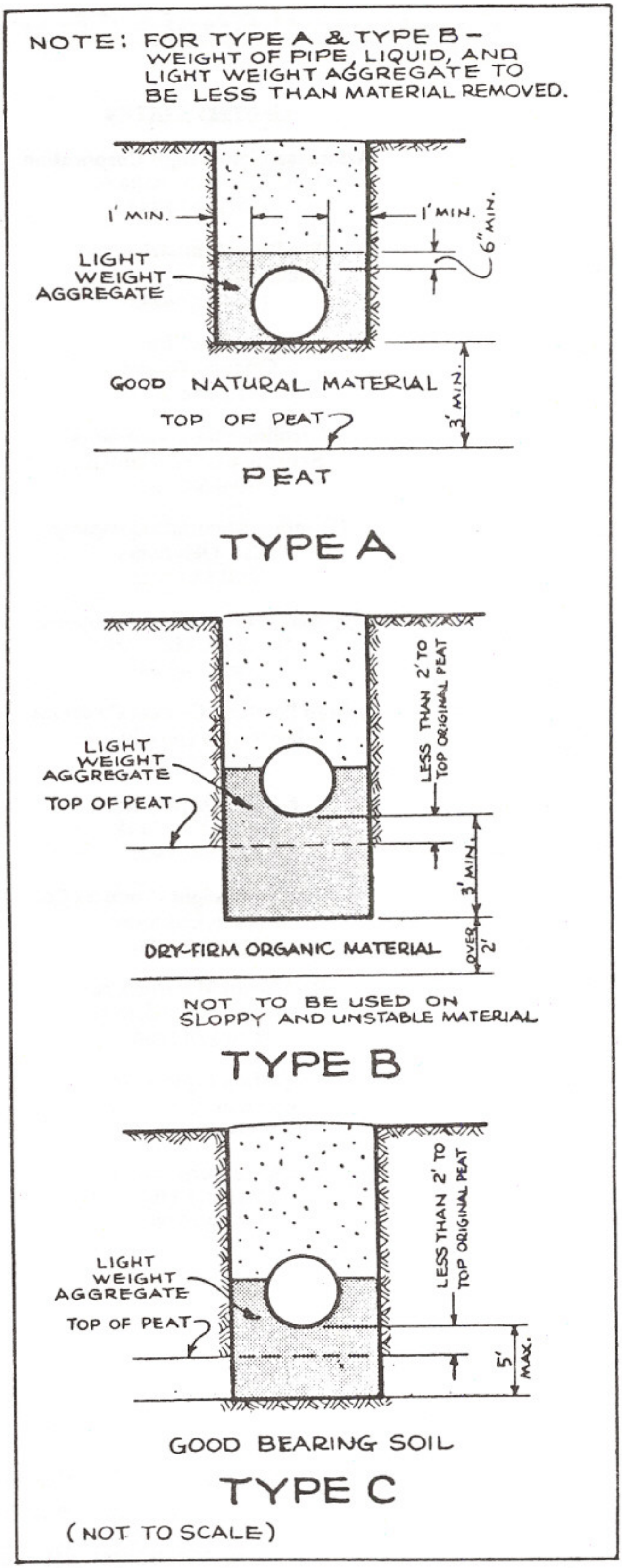


FIGURE 2: Typical application lightweight aggregate bedding.

Expanded Shale Clay & Slate Institute Members

UNITED STATES

Arkansas Lightweight Corporation
West Memphis, Arkansas
(501) 735-7932

Big River Industries, Inc.
Baton Rouge, Louisiana
(504) 627-4242

Buildex, Inc.
Ottawa, Kansas
(913) 242-2177

Carolina Stalite Company
Salisbury, North Carolina
(704) 637-1515

Chandler Materials Company
Tulsa, Oklahoma
(918) 836-9151

Hydraulic Press Brick Company
St. Louis, Missouri
(314) 621-9306

Lehigh Portland Cement Company
Allentown, Pennsylvania
(215) 776-2600

Norlite Corporation
Cohoes, New York
(508) 582-9931

Pacific Lightweight Products Co.
Glendale, California
(818) 240-5160

Port Costa Materials, Inc.
Port Costa, California
(415) 228-7266

Solite Corporation
Richmond, Virginia
(804) 321-6761

Utelite Corporation
Coalville, Utah
(801) 467-2800

CANADA

Cindercrete Products Limited
Regina, Saskatchewan
(306) 789-2636

Consolidated Concrete Limited
Calgary, Alberta
(403) 259-3559

Consolidated Concrete Limited
Edmonton, Alberta
(403) 459-8757

Kildonan Concrete Limited
Winnipeg, Manitoba
(204) 233-5666

EUROPE

Norcem Bygg A/S (Norsk-Leca)
Aker ExClay A.S.
Oslo, Norway

Fratelli Buzzi S.p.A.
Corso Giovana, Italy

Gralex S.A.
Brussels, Belgium

Lias Franken-LIAPOR
Pautzfeld & Tuningen, West Germany

S.M.A.E. Societa Meridionale Argille Espanse S.p.A.
Roma, Italy

JAPAN

Nihon Cement Company, Ltd.
Asano-Lite Div.
Tokyo, Japan

Nihon Measlite Industry Company, Ltd.
Chiba-Ken, Japan

For more information about expanded shale, clay or slate lightweight aggregate, contact any member listed above or:

Expanded Shale Clay & Slate Institute
2225 East Murray Holladay Road, Suite 102, Salt Lake City, Utah 84117
(801) 272-7070 / FAX (801) 272-3377

601-G *Reprinted 6/91*

K. CHILDS
D. L. PORTER
T. A. HOLM

A marine terminal expansion project used lightweight aggregate as a soil fill to stabilize the dockside area.

LIGHTWEIGHT FILL HELPS ALBANY PORT EXPAND

The modification of the Marine Terminal in the Port of Albany was designed to reclaim a functional area of approximately 1500 : 80 ft (460 x 25 m) and provide ncreased dockside draft to allow arge oil tankers to service the CIBRO Petroleum Products Disribution Terminal. In addition to ncreasing the draft from 26 to 32 t (8 to 10 m) the reclamation pro;ram stabilized the area and pro'ided for increased operating safey. Continuous operation of the acility during the expansion contruction was an additional design onstraint.

Fig. 1 shows how the unusable xisting slope into the Hudson River was reclaimed and the site ehabilitated by the sheet pile vall, lightweight fill and the use of ie backs to reinforced concrete lead men. The extreme southern ortion of the site was developed hrough the use of cellular cofferams that reached a shallow bedock stratum. At the northern secion, tied back piles and lightveight fill provided the solution. ig. 2 shows a plan overview of the ite.

During the design of the modifications it was determined that no one type of bulkhead structure would be cost effective over the entire length of the project. Relatively high bedrock (elevation −33.0 {−10 m}) at the south end of the site and low bedrock (elevation −83.0 {−25 m}) at the north end required a variation in the type of structure used to minimize cost.

Gravel filled steel sheet pile cells provided the best alternative at the south end of the site. The cell structures require little or no penetration into the overburden or bedrock to maintain stability. In addition the cells provided an environmentally acceptable containment area for dredge spoil disposal. The gravel fill provided a cleansing filter for dredge spoil leachates which might find their way back to the river.

In the area between the tanker and barge berths where the bedrock varies from elevation −35.0 to −83.0 ft (−11 to −25 m) and the overburden consists of layers of loose sand, gravel, clay and loose silt, the tiedback HZ sheet pile wall systems with lightweight backfill proved most economical.

The loose silt and clay layers were a major concern from the standpoint of overall slope stability and sheet pile wall kickout resistance. The combination of the lightweight backfill which minimized lateral earth pressures while also minimizing the overburden pressures on the sensitive silts, together with the H-pile penetration to rock, caused the least disturbance to the existing soils. The HZ wall system provided a cost effective solution since the Z sheets could be terminated at elevations well above the bedrock (−42 ft or −13 m) while the H-piles could be driven to the rock where wall kickout resistance was established. In addition the HZ wall provided the required strength necessary for an exposed wall height of 49 ft (15 m), between elevation −32.0 ft to +17.0 ft (−10 to +5 m) without modification. A standard Z sheet pile wall would have required welding on steel plates to increase strength.

At the north end of the project where dredge depths were less (elevation −26.0 {−8 m} versus elevation −32.0 {−10 m}) a high-strength Z type sheet pile wall proved sufficient since a gravel layer above the bedrock was available to provide kickout resistance. Again lightweight fill was specified to reduce the lateral earth pressures.

Reprinted from Civil Engineering Magazine-ASCE April 1983

View of completed rehabilitation of the Port of Albany's marine terminal, now in operation.

FIGURE 1
CROSS-SECTION AT SHEET PILE WALL

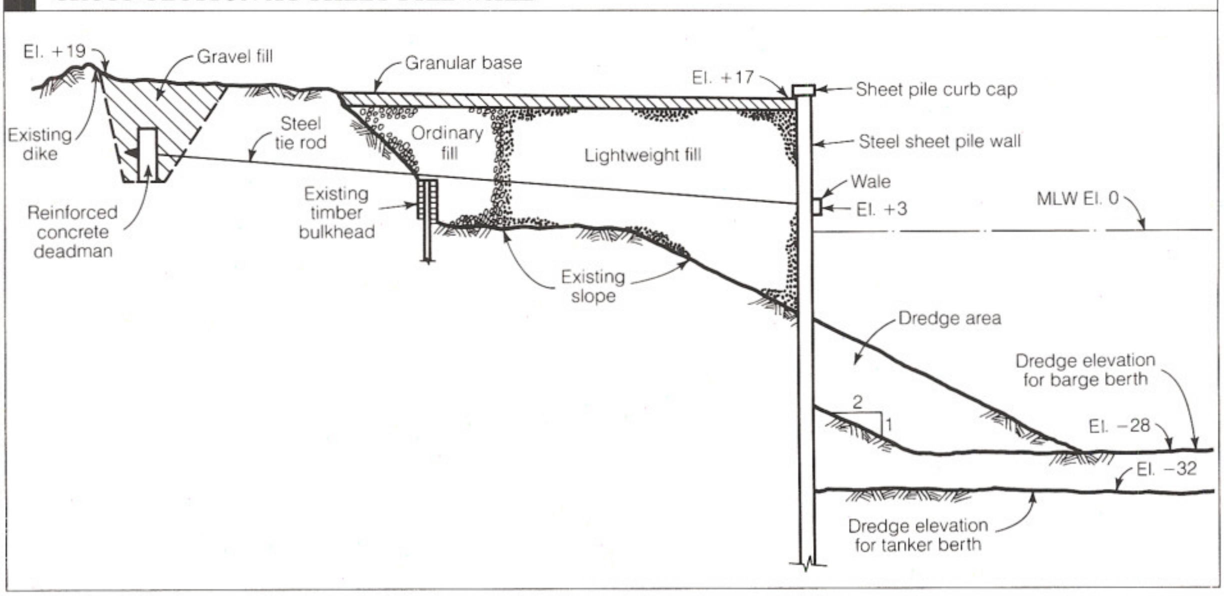
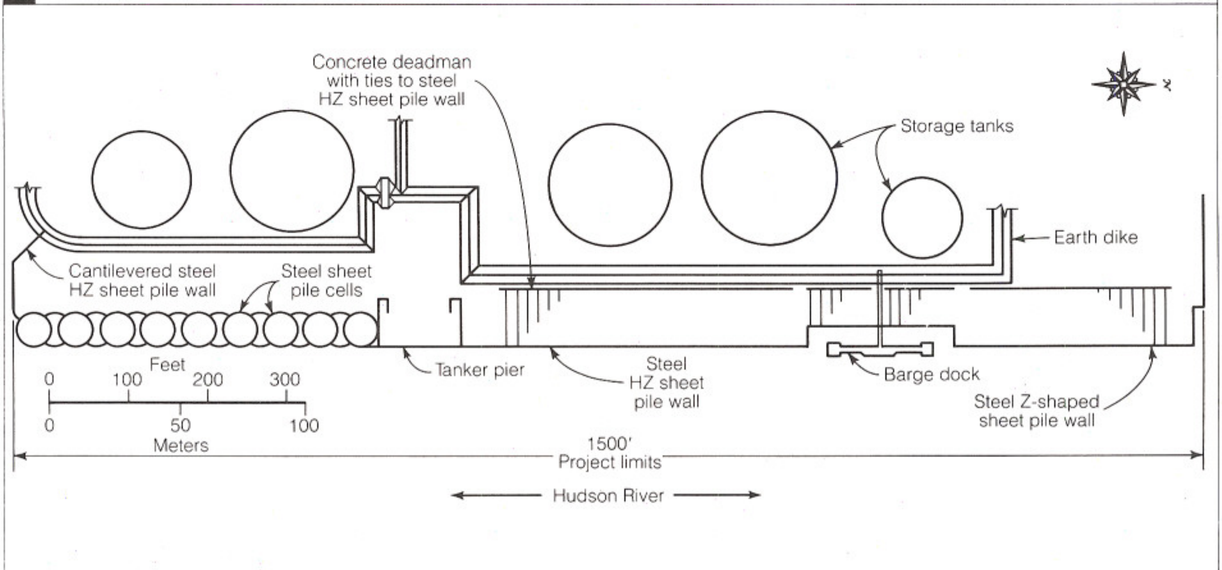


FIGURE 2

OVERVIEW OF MARINE TERMINAL MODIFICATIONS



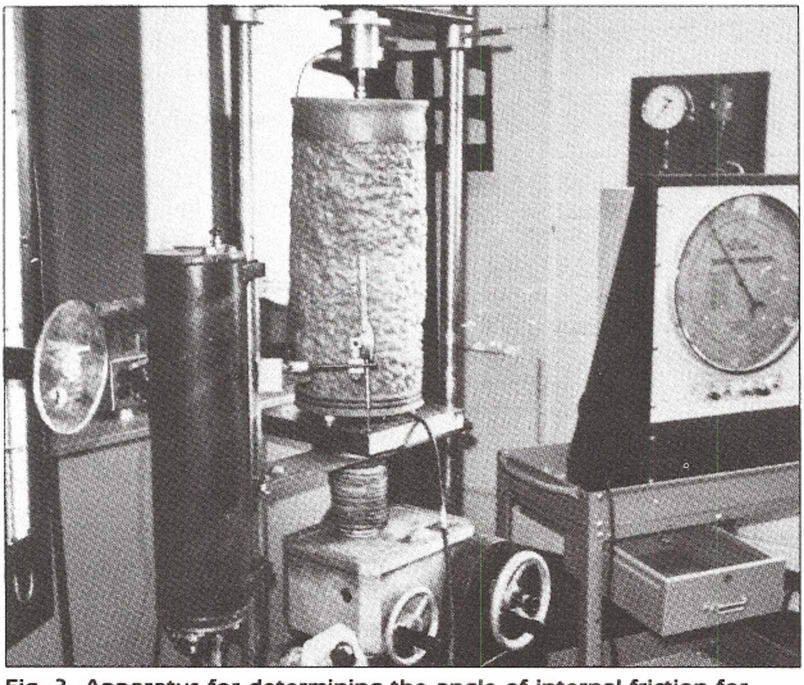


Fig. 3. Apparatus for determining the angle of internal friction for coarse lightweight aggregate.

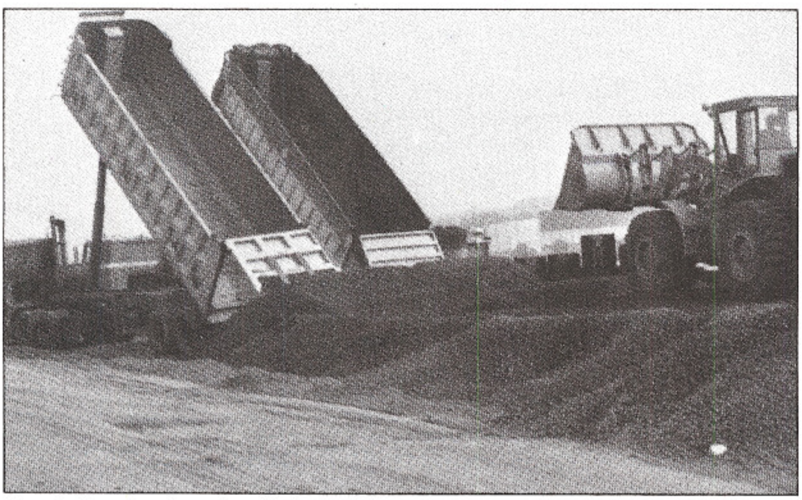


Fig. 4. Trucks dumping lightweight aggregate behind sheet piling wall.

LIGHTWEIGHT AGGREGATE

The specifications for the lightweight fill called for a rotary kiln-produced expanded slate or shale composed of inert, granular, inorganic material with a continuous, coarse aggregate gradation and exhibiting a minimum angle of internal friction (ϕ) of 40 degrees.

The lightweight aggregate used has been successfully used in numerous other soil fill applications including a Boston waterfront project designed by Childs Engineering in 1972. The expanded aggregate is produced in several gradations available for masonry and structural concrete applications.

While the technology of structural lightweight concrete and lightweight concrete masonry is well documented, the physical properties appropriate to soil mechanics applications are less developed and under continuing research. Rotary kiln-produced lightweight aggregate is capable of producing concrete strength in excess of 5000 psi (35 kPa); from the standpoint of individual particle strength and toughness its mechanical and weathering performance in structures, including exposed bridge decks, is well known and fully documented. For this project a coarse aggregate (¾-in. to #4) (20 to 4 mm) was selected for optimum combinations of low density and high stability, coupled with free draining characteristics. A typical delivered gradation was:

Sieve size	Percent passing
1 in. (25 mm)	100
¾ in. (20 mm)	92
½ in. (12 mm)	46
⅜ in. (10 mm)	16
#4 (4 mm)	1

Laboratory tests have shown that granular materials do not have the well developed peak compacted density typical of cohesive materials. Practical variations of standard ASTM Proctor tests may, however, be conducted to evaluate in-place densities as a function of the compactive energy while also determining the degree of aggregate breakdown under compaction. A compacted moist, bulk density of 70 lb/ft^3 (1120 kg/m^3) was determined by Childs Engineering to be the appropriate design requirement that would reduce lateral pressures, provide a compacted substrate and develop in-place stability without excessive degradation of the aggregate.

In order to determine the resistance to lateral forces developed by the compacted aggregate, large-scale triaxial compression tests were conducted at Columbia University's Geotechnical Laboratory under the direction of Professor Robert D. Stoll (Fig. 3). A testing arrangement designed and fabricated by Dr. Stoll incorporated a representative test specimen 10 in. in diameter by 24 in. high (250 x 600 mm) encapsulated by an elastic membrane, which provided a sample size that minimized restraint of the platens. The failure surface developed during the course of the test was always easily visible.

The usual small scale triaxial compression test samples are appropriate for sand size particles but not for coarse aggregate specimens. A comprehensive triaxial compression testing program conducted on a number of stockpile samples gave an assurance of repeatability in testing. Further tests evaluated the influence of the aggregate moisture conditions on the angle of internal friction (Ø). Finally a two-year program was conducted on five lightweight aggregates from other rotary kiln plants in other geographic areas to determine the effects of differing aggregate properties (particle strength, shape and gradation) on the angle of internal friction. Based on this extensive series of tests, the angle of internal friction was determined to be in excess of 40 degrees in a loose condition and slightly higher in a compacted condition. Stress-strain curves developed in the triaxial compression testing program on various samples of lightweight aggregate, under differing conditions of compaction, were similar to test results on other granular materials.

CONSTRUCTION

The contractor had little difficulty in the transportation, placing and compacting of the lightweight aggregate soil fill. Peak shipment reached 1300 tons (1170 mt) per day (55 truck deliveries) without logistical difficulties or overtime requirements on the part of the constructor's personnel. At first the material was moved with a Michigan 75B loader, while later a Caterpiller 966C front end loader was used because of its larger bucket and the room to move a greater volume. The project was organized with telescoping belts for aggregate handling in some areas, while in other areas two trucks were arranged for tandem dumping followed by movement of the material by front end loader (Fig. 4). Lightweight aggregate shipments started in July 1981 and were virtually completed by October.

During the course of the project, 215 test samples were taken on the lightweight aggregate with detailed information reported on gradation, bulk and particle density, moisture content, and in-place density. Test samples were taken by the aggregate producer's quality control and field service personnel as well as by an independent testing lab hired by the owner.

Dry loose delivered bulk density was specified not to exceed 55 lb/ft^3 (880 kg/m^3) with in-place, compacted bulk density not to exceed 70 lb/ft^3 (1120 kg/m^3). Field tests indicated the delivered moist bulk density was less than 55 lb/ft^3 (880 kg/m^3) and in-place compacted bulk density averaged approximately 65 lb/ft^3 (1040 kg/m^3).

While laboratory tests on small samples are necessary and instructive, the best measure of accuracy in predicting in-place density is when the contractor's estimates of quantities, based upon cross-sectional volume calculations, are realized by actual shipments to the project. This was the case on this job as the estimated quantity of 23,700 tons (21,500 mt)—approximately 27,000 yd^3 (20,600 m^3) in-place—was realized within 1%. The project has performed as expected and is currently off-loading large tankers serving the upstate area.

Acknowledgments

The marine terminal modifications were conducted at the Port of Albany, Albany, New York, owned by the Albany Port District Commission, Frank W. Dunham, Jr., General Manager. Engineering design, drawings, specifications and inspection were provided by Childs Engineering, Inc., Medfield, Mass. The project was partially funded by Cibro Petroleum Products, Inc., J.S. Plunkett, plant manager, and the State of New York. Administrative aspects of New York State's funding were monitored by the New York State Dept. of Transportation. The general contractor was Edward B. Fitzpatrick Construction Corporation, Contractors & Engineers, Williston Park, N.Y., G.J. Galvin, project engineer. Lightweight aggregate was supplied by Northeast Solite Corp., Mt. Marion, N.Y.

The project is noteworthy for the cooperative efforts whereby private sector capital was matched by local government funding for the overall rehabilitation of the Port area, which is considered a cornerstone of economic recovery for the entire Capital District region.

Kenneth M. Childs is president of Childs Engineering Corporation in Medfield, Mass. He has been involved in the design, construction, and inspection of all types of marine structures including floating dry docks, piers, retaining walls, and slope stabilization projects. Mr. Childs has performed some or all of the diving on many of these jobs.

David L. Porter is a vice president of Childs Engineering. As principal-in-charge of field operations and a diver, he has participated in and directed waterfront engineering projects above and below the water throughout North America, Europe and West Africa, including several marine railways and a 1400-ft prestressed concrete pier in Provincetown, Mass.

Thomas A. Holm is director of engineering for Solite Corporation in Mount Marion, New York. In addition to his many professional activities with the American Concrete Institute and other societies, he has written frequently on the properties and uses of structural lightweight concrete and is a member of the delegation in the USA-USSR exchange on the technology of concrete housing systems.

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APPENDIX A
TO ASTM C 330
DETERMINATION OF SPECIFIC GRAVITY FACTORS OF STURCTURAL LIGHTWEIGHT AGGREGATE

Methods presented herein describe procedures for determining the density factors of lightweight aggregates containing absorbed water.

Pycnometer method for fine and coarse lightweight aggregates:

a. A pycnometer consisting of a narrow-mouthed 2-qt mason jar with a pycnometer top (Soil test G-335, Humboldt H-3380, or equivalent).

b. A balance or scale having a capacity of at lest 5 kg and a sensitivity of 1 g.

c. A water storage jar (about 5-gal. capacity) for maintaining water at room temperature.

d. Isopropyl (rubbing) alcohol and a medicine dropper.

Calibration of pycnomter

The pycnometer is filled with water and agitated to remove any entrapped air. Water is added to "top off" the jar. The filled pycnometer is dried and weighed and the weight (weight B in grams) is recorded. (Regarding this method, a review of ASTM C 128 may be helpful)

Sampling procedure

Representative samples of about 2 to 3 ft^3 (.06 to .09 m^3)of each size of aggregate should be obtained from the stockpile and put through a sample splitter or quartered until the correct size of the sample desired has been obtained. During this operation with damp aggregates, extreme care is necessary to prevent the aggregates from drying. The aggregate sample should occupy one-half to two-thirds the volume of the 2-quart pycnometer.

Test procedure

Two representative samples should be obtained of each size of lightweight aggregate to be tested. The first is towel dried to remove surface (absorbed) water, weighed, placed in an oven at 105°C and dried to constant weight. "Frying pan drying" to constant mass is an acceptable field expedient. The dry aggregate weight is recorded, and the aggregate moisture content (percentage of aggregate dry mass) is calculated.

The second aggregate sample is also towel dried, weighed (mass *C* in grams). The sample is then placed in the empty pycnometer and water is added until the jar is three-quarters full. The time of water addition should be noted.

The air entrapped between the aggregate particles is removed by rolling and shaking the jar. During agitation, the hole in the pycnometer top is covered with the operator's finger. The jar is then filled and agitated again to eliminate any additional entrapped air. If foam appears during the agitation and prevents the complete filling of the pycnometer with water at this stage, a minimum amount of isopropyl alcohol should be added with a medicine dropper to eliminate the foam. The water level in the pycnometer must be adjusted to full capacity and the exterior surfaces of the jar must be dry before weighing.

The pycnometer, thus filled with the sample and water, is weighed (mass *A* in grams) after 5, 10, and 30 minutes of sample immersion to obtain complete data, and the weights at these times are recorded after each "topping-off". Fig. A shows a typical plot of such determinations. Extrapolation should be avoided.

Calculation

The pycnometer density factor S, after any particular immersion time, is calculated by the following formula.

$$S = \frac{C}{C + B - A}$$

Where

A = mass of pycnometer charged with aggregate and then filled with water, g

B = mass of pycnomter filled with water, g

C = mass of moist aggregate tested, g

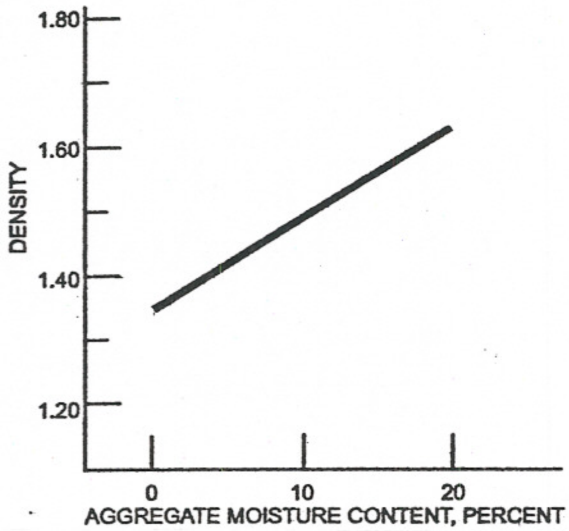
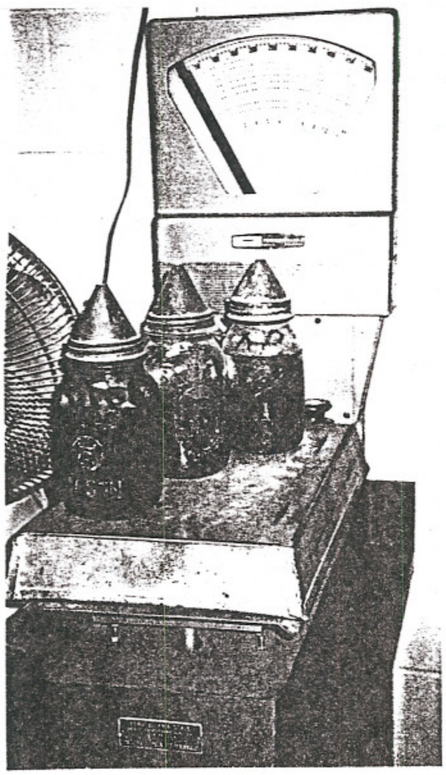


Figure A. Example of relationship between pycnometer specific Gravity factor and moisture content for lightweight aggregate



C 150	Standard Specification for Portland Cement
C 173	Standard Test Method for Air Content of Freshly Mixed Concrete by Volumetric Method
C 192	Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory
C 330	Standard Specification for Lightweight Aggregates for Structural Concrete
C 494	Standard Specification for Chemical Admixtures for Concrete
C 566	Standard Test Method for Total Moisture Content of Aggregate by Drying
C 567	Standard Test Method for Unit Weight of Structural Lightweight Concrete

5.2—Cited references

1. Wills, Milton H., Jr., "Lightweight Aggregate Particle Shape Effect on Structural Concrete," ACI JOURNAL, *Proceedings* V. 71, No. 3, Mar. 1974, pp. 134-142.

2. Troxell, George E., and Davis, Harmer E., *Composition and Properties of Concrete*, McGraw-Hill, 1956, p. 45.

APPENDIX A—DETERMINATION OF SPECIFIC GRAVITY FACTORS OF STRUCTURAL LIGHTWEIGHT AGGREGATE

Methods presented herein describe procedures for determining the specific gravity factors of lightweight aggregates, either dry or moist.

Pycnometer method for fine and coarse lightweight aggregates:

a. A pycnometer consisting of a narrow-mouthed 2-qt mason jar with a pycnometer top (Soiltest G-335, Humboldt H-3380, or equivalent).

b. A balance or scale having a capacity of at least 5 kg and a sensitivity of 1 g.

c. A water storage jar (about 5-gal. capacity) for maintaining water at room temperature.

d. Isopropyl (rubbing) alcohol and a medicine dropper.

Calibration of pycnometer

The pycnometer is filled with water and agitated to remove any entrapped air and adding water to "top off" the jar. The filled pycnometer is dried and weighed and the weight (weight B in grams) is recorded. (A review of ASTM C 128 may be helpful regarding this method.)

Sampling procedure

Representative samples of about 2 to 3 ft^3 of each size of aggregate should be obtained from the stockpile and put through a sample splitter or quartered until the correct size of the sample desired has been obtained. During this operation with damp aggregates, extreme care is necessary to prevent the aggregates from drying. The aggregate sample should occupy one-half to two-thirds the volume of the 2-qt pycnometer.

Test procedure

Two representative samples should be obtained of each size of lightweight aggregate to be tested.

The first is weighed, placed in an oven at 105 to 110 C and dried to constant weight. "Frying pan drying" to constant weight is an acceptable field expedient. The dry aggregate weight is recorded, and the aggregate moisture content (percentage of aggregate dry weight) is calculated.

The second aggregate sample is weighted (weight C in grams). The sample is then placed in the empty pycnometer and water is added until the jar is three-quarters full. The time of water addition should be noted.

The air entrapped between the aggregate particles is removed by rolling and shaking the jar. During agitation, the hole in the pycnometer top is covered with the operator's finger. The jar is then filled and agitated again to eliminate any additional entrapped air. If foam appears during the agitation and prevents the complete filling of the pycnometer with water at this stage, a minimum amount of isopropyl alcohol should be added with the medicine dropper to eliminate the foam. The water level in the pycnometer must be adjusted to full capacity and the exterior surfaces of the jar must be dried before weighing.

The pycnometer, thus filled with the sample and water, is weighed (weight A in grams) after 5, 10, and 30 min of sample immersion to obtain complete data, and the weights at these times are recorded after each "topping-off." Fig. A shows a typical plot of such determinations. The variation is usually approximately linear in the lower range of moisture contents, but may digress from linearity at higher moisture contents. The full curve, therefore, should be established and extrapolation should be avoided.

Calculation

The pycnometer specific gravity factor S, after any particular immersion time, is calculated by the following formula.

$$S = \frac{C}{C + B - A}$$

where

A = weight of pycnometer charged with aggregate and then filled with water, g
B = weight of pycnometer filled with water, g
C = weight of aggregate tested, moist or dry, g

Buoyancy methods for coarse aggregates

If larger test samples of coarse aggregate than can be evaluated in the pycnometer are desired, coarse aggregate gravity factors may be determined by the wholly equivalent weight-in-air-and-water procedures described in ASTM C 127. The top of the container used for weighing the aggregates under water must be closed with a screen to prevent light particles from floating away from the sample.

Specific gravity factors by this method are calculated by the equation

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$$\text{Specific gravity factor } S = \frac{C}{C - E}$$

where

C = save as above (the weight in air)

E = weight of coarse aggregate sample under water, g

S = specific gravity factor, equal (by the theory of the method) to the pycnometer specific gravity factor

APPENDIX B—DETERMINATION OF STRUCTURAL LIGHTWEIGHT COARSE AGGREGATE ABSORPTION

The method presented hereafter describes a procedure for determining the absorption of lightweight coarse aggregate by spin-drying in a centrifuge to produce a saturated surface dry condition following 24 hr of immersion in water.

Apparatus

a. A bench-top centrifuge with a speed control capable of spinning a 300 to 400 g sample of graded coarse aggregate at 500 rpm. A centrifuge similar to an International Model HN or a centrifugal extraction apparatus similar to a Soiltest Model AP 179-B are satisfactory.

b. A bowl or colander approximately $8^1/_2$ in. in diameter and 3-in.-deep mounted on the axis of the centrifuge and fitted with a lid to prevent loss of the aggregate when spun. Centrifugal extractors are manufactured with such bowls; therefore, this requirement does not apply to them.

c. A balance having a capacity of at least 1000 g and a sensitivity of 0.1 g.

Sampling procedure

Representative samples of about 20 to 30 kg of graded aggregate should be taken from the stockpile and reduced with a sample splitter or quartered until a 300 to 400 g sample is obtained. During this operation, definite precautions should be taken to prevent segregation of the coarser particles from those smaller in size. Two or more representative samples should be taken.

Test procedure

Immerse the samples of graded, lightweight coarse aggregate for approximately 24 hr in tap water at room temperature. After that period, decant the excess water and transfer the sample into the bowl or colander and secure the lid. Activate the centrifuge and spin the sample at 500 rpm for 20 min. Remove the sample and measure its saturated surface dry weight. Dry the sample to constant weight by any of the procedures described in ASTM C 566—electric or gas hot plate, electric heat lamps, or a ventilated oven capable of maintaining the temperature surrounding the sample at 105 to 115 C. Fig. B shows a typical plot of determining lightweight coarse aggregate absorption.

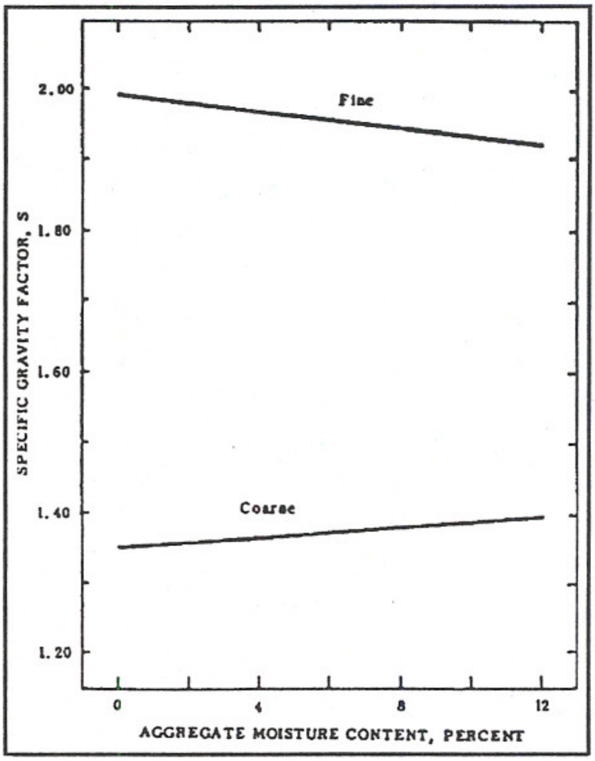

Fig. A—Example of relationship between pycnometer specific gravity factor and moisture content for lightweight aggregate.

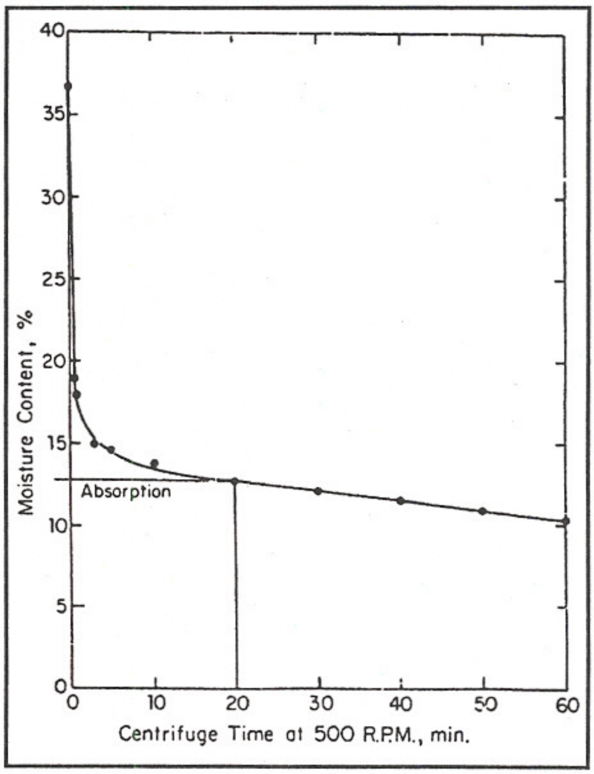

Fig. B—Typical relationship illustrating measurement of lightweight aggregate absorption.

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APPENDIX E

- *"Expanded shale Lightweight Fill: Geotechnical Properties"*, Stoll and Holm, pp. E1 – E4.

- "*Geotechnical Properties of Expanded Shale Lightweight Aggregate*", Valsangkar and Holm, March 1990, pp. E5 – E12.

- "*Cyclic Plate Load Tests on Lightweight Aggregate Beds",* Valsangkar and Holm, Transportation Research Record, January 1993, pp. E13 – E16.

- "*Lightweight Fill Solutions to Settlement and Stability Problems on Charter Oak Bridge Project, Hartford, Connecticut*", Dugan, Transportation Research Record, 1993, pp. E17 – E20.

- *"Model Tests on Peat – Geotextile – Lightweight Aggregate System",* Valsangkar and Holm, August 1987, pp. E21 – E30.

EXPANDED SHALE LIGHTWEIGHT FILL: GEOTECHNICAL PROPERTIES

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Director of Engineering, Solite Corporation, Richmond, Virginia, Member ASCE

SOLITE®

INTRODUCTION

Expanded shale lightweight aggregate has been used by the construction industry for many decades to produce lightweight structural concrete and lightweight concrete masonry units. The aggregate is expanded by heating shale (shale is used throughout this article to represent shale, clay or slate) in a rotary kiln under carefully controlled conditions at temperatures of approximately 2,100°F (1,150°C). The expanded, vitrified mass that results from this process is then screened to produce the desired gradation for a particular application. The pores formed during expansion are generally noninterconnecting and the particles are subangular in shape and very light in weight because of the vesicular nature of expanded shale. They are durable, chemically inert and insensitive to moisture.

Recently, lightweight aggregates have been found to be cost-effective alternatives in certain applications in the field of geotechnical engineering (2). Typical examples of where lightweight aggregate may offer significant advantages are the construction of controlled fills over very soft, compressible soil and backfill next to structural elements where there is the potential for excessive earth pressure or a stability problem when using ordinary fill materials. In order to evaluate the potential advantage of using lightweight aggregate, it is necessary to know both the unit weight and the mechanical properties of the aggregate under various kinds of loading. Most aggregate manufacturers can furnish data on the physical and engineering properties of aggregate produced at a particular plant. However, information on the stress-strain-strength response of unbonded lightweight aggregate is virtually non-existent. Information on such properties as the angle of internal friction and the compressibility of a fill under various levels of overburden stress is essential for any rational evaluation of the potential use of these materials in a geotechnical application.

The purpose of this paper is to present data from tests on lightweight aggregate from several different locations in the eastern United States. Large scale triaxial compression tests were performed on specimens from five different locations and uniaxial strain tests (consolidation tests) were run on aggregate from one of the sites. The results may be compared with data for ordinary fills when the lightweight aggregate is being considered as a design alternative.

TRIAXIAL TESTS

All of the triaxial tests were run on specimens approximately 10 in. (25.4 cm) in diameter and 24 in. (61.0 cm) long. Specimens were confined in a rubber sleeve with a wall thickness of approximately 1.5 mm. Isotropic confining stress was applied to specimens by connecting a controlled vacuum through a port in either the top or bottom platen. All tests were run at a constant rate of axial displacement which was equivalent to an average strain rate of 0.7% per min.

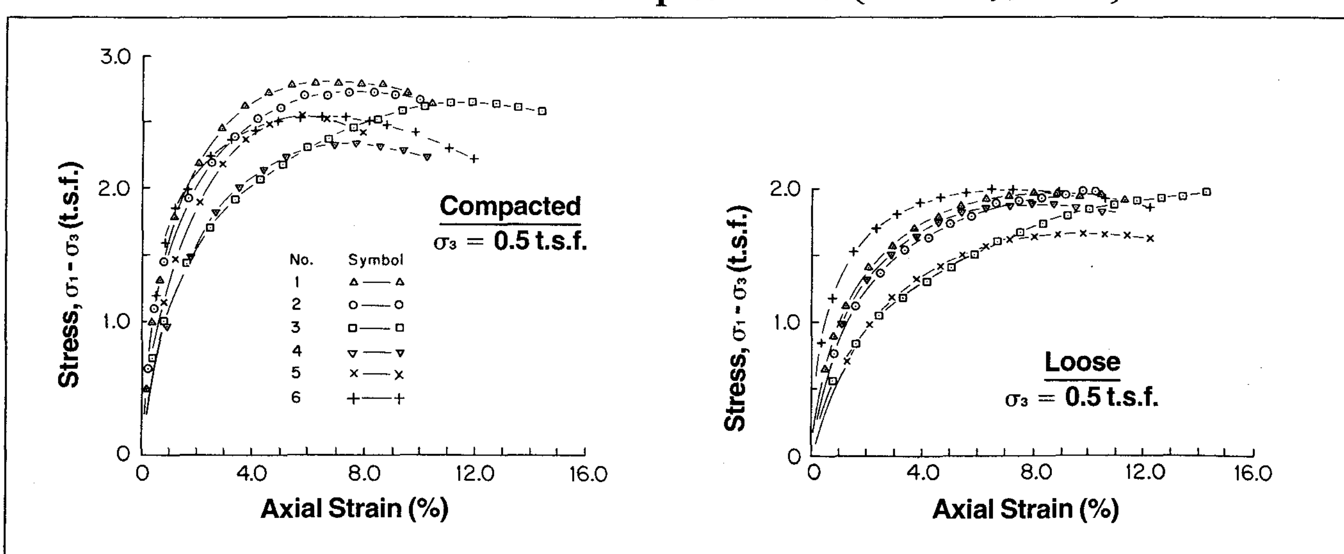
Tests were run on "loose" and "compacted" specimens for each different material. The loose specimens were prepared by gently placing the aggregate into the forming mold one scoop at a time, with an effort made to avoid vibration or other disturbance. Once in place the aggregate was not leveled or rearranged. In the tests on "compacted" aggregate, each specimen was compacted in five layers with 25 blows of a 5.5 lb. (24.5 N) hammer falling 12 in. (30.5 cm) on each layer. The densities produced by these procedures as well as other information about the source of the samples are given in Table 1. The difference in density between the loose and compacted specimens is about the same as the difference between the maximum and minimum dry densities that resulted when the standard ASTM tests for the relative density of cohesionless soils (ASTM D2049) were performed.

Fig. 1 shows the stress-strain curves obtained for six sets of tests. Most of the tests were run at the moisture content "as received" in the lab. Four of the tests (1 through 4) were run on a coarse fraction (passing the ¾ in. sieve and retained in the No. 4 sieve) which is commonly available from stock at many of the plants. From the figure it is obvious that there is a difference in response between aggregates 1 and 2 and that of aggregate 3. A physical inspection revealed a difference in the general shape and hardness of the particles. While there is some variation in the angle of friction determined at the peak stress, a more significant difference may be the amount of strain that is required to develop the full shearing strength.

TABLE 1.—Source and Other Information for Aggregates Used in Tests

Number in figure (1)	Source of aggregate (2)	Gradation passing/ retained (3)	Water Content at Test Time (%)		Dry Unit Weight (pcf)	
			Compact (4)	Loose (5)	Compact (6)	Loose (7)
1	Saugerties, New York	¾ in./No. 4	5.3	7.1	~52	~46
2	Aquadale, N. Carolina	¾ in./No. 4	7.2	6.7	53.4	47.6
3	Bremo, Virginia	¾ in./No. 4	4.0	6.0	49.2	41.7
4	Green Cove, Florida	¾ in./No. 4	8.1	8.4	50.6	46.4
5	Green Cove, Florida	⅜ to Pan	8.2	8.4	61.9	53.9
6	Hubers, Kentucky	⅜ in./No. 8	0.1	1.4	53.0	47.1

FIG. 1.—Stress-Strain Curves for Triaxial Compression Test (1 t.s.f. = 95.8 kPa)



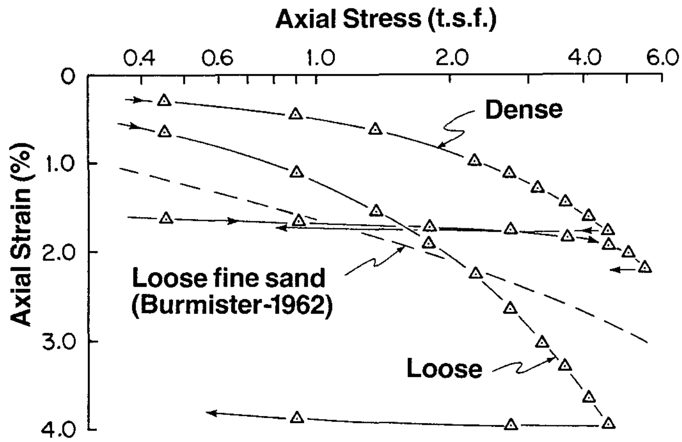
Aggregates 5 and 6 in Fig. 1 contain intermediate and fine fractions which are also commonly available at many processing plants. In these materials the coarsest particles are those passing the 3/8 in. sieve and there is a more noticeable stress drop-off after the peak, as is typical in many well graded granular soils. In general, the curves shown in Fig. 1 are quite similar to what is obtained for many common gradations of ordinary fill. For the compacted aggregates, the angle of internal friction corresponding to the peak stress difference varies from 44.5 to 48°, whereas for the loose material the range of 0 is from 39.5 to 42°.

In the case of the Saugerties, New York aggregate (No. 1), tests were performed at several different confining pressures to insure that the Mohr envelope was essentially a straight line passing through the origin. In addition, tests were run on this material after it had been soaked in water for a period of five weeks. In the tests on water-soaked aggregate, the angle of internal friction was 1 to 2° lower than for the tests on the air dry or slightly moist materials.

CONSOLIDATION TESTS

Consolidation tests were performed on aggregate No. 1 using a 10 in. diameter floating ring with 1 in. wall thickness for lateral confinement and a specimen thickness of 4.5 in. Test results for a compacted and a loose specimen are shown in Fig. 2. In addition to the test curves, a consolidation curve for "loose fine sand" given by Burmister (1) has been included in Fig. 2 for reference. In the case of the lightweight aggregates, the curvature and the slope of the stress-strain curves corresponding to the first monotonic loading increases rather rapidly after a stress level of about 1 t.s.f. (98.8 kPa) is exceeded. On the other hand, when the specimen is unloaded and then reloaded, the reload curve is much flatter than one would expect, even on the first cycle of unloading and reloading in a typical cohesionless soil. This suggests that some degradation may be occurring during the first monotonic loading, and that once the interparticle contacts have been stabilized, the material reacts in a much stiffer manner to subsequent load applications. Further testing is planned to verify this conjecture.

FIG. 2.— Consolidation Stress-Strain Curves for Coarse New York Aggregate (1 t.s.f. = 95.8 kPa)



SUMMARY AND CONCLUSIONS

In summary, our tests on expanded shale lightweight aggregates from several different sites showed that the response under triaxial loading was similar to that of many ordinary coarse fill materials; the principal difference is that the lightweight aggregates weigh roughly half as much as their naturally-occurring counterparts. Thus the lightweight aggregates may prove to be useful substitutes for ordinary fill materials when the combination of low weight and substantial shear strength warrant the increased cost. The mechanical properties of the aggregate tend to vary somewhat from source to source so that they should be verified in each instance. However, once the properties have been established for a given plant, the variation will be much less than is normally encountered by a designer utilizing ordinary fill from a borrow area.

APPENDIX. — REFERENCES

1. Burmister, D.M., "Physical, Stress-Strain, and Strength Response of Granular Soil," *Special Technical Publication No. 322,* ASTM, 1962, pp. 87-93.
2. Childs, K., Porter, D. L., and Holm, T. A., "Lightweight Fill Helps Albany Port Expand," *Civil Engineering,* ASCE, Apr., 1983, pp. 54-57.

Suggested SOLITE® Soil Fill Specifications

The information listed below is a suggested specification for SOLITE® Lightweight Aggregate soil fill. It is best to consult with SOLITE® engineering and sales representatives during a project's conceptual design phase in order to call for the most appropriate geotechnical and material physical properties.

Materials

Lightweight Aggregate fill shall be SOLITE® Lightweight Aggregate or approved rotary kiln substitute meeting all the requirements of a recently completed (2 years max.) ASTM C-330 certification. No by-product slags or cinders are permitted. Lightweight aggregate shall have a proven record of durability and be non-corrosive (less than 100 ppm chloride when measured by FHWA-RD-77-85) with the following physical properties:

A. Delivered Gradation

Sieve Size	% Retained
1″ (25 mm)	_____
½″ (13 mm)	_____
#4 (5 mm)	_____

B. The dry loose unit weight shall be less than 55 pcf (880 kg/m^3). The lightweight aggregate producer shall submit verification of a compacted density of less than 60 pcf (960 kg/m^3) when measured by a one point test conducted in accordance with ASTM D-698 "The Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 5.5 lb. Hammer and 12 inch Drop" (AASHTO T-99).

C. The lightweight aggregate producer shall submit verification that the angle of internal friction shall be greater than 40° when measured in a triaxial compression test on a laboratory sample with a minimum diameter of 10 inches.

D. The maximum Los Angeles Abrasion loss when tested in accordance with ASTM C-131 (B grading) shall be 50%.

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GEOTECHNICAL PROPERTIES OF EXPANDED SHALE LIGHTWEIGHT AGGREGATE

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Geotechnical Properties of Expanded Shale Lightweight Aggregate

REFERENCE: Valsangkar, A. J. and Holm, T. A., **"Geotechnical Properties of Expanded Shale Lightweight Aggregate,"** *Geotechnical Testing Journal*, GTJODJ, Vol. 13, No. 1, March 1990, pp. 10–15.

ABSTRACT: In recent years lightweight aggregates are being used increasingly in geotechnical applications. This paper presents results of large-size one-dimensional compression and direct shear tests performed on lightweight aggregate. The compressibility and shear strength characteristics of the lightweight aggregate are compared with those of normal-weight aggregate using the same experimental setup. Results of the direct shear tests performed to determine the angle of friction between the geotextile and lightweight aggregate are also presented. In addition, static shear modulus values as determined from model pile tests are presented and compared with those reported for normal weight aggregates.

KEYWORDS: angle of internal friction, angle of friction, coarse aggregates, compressibility, geotextiles.

Extensive research data are reported in literature dealing with lightweight structural concrete. In recent years, lightweight aggregates are being increasingly used in geotechnical applications such as embankments on soft ground and backfill behind retaining structures. In the former case, use of lightweight aggregate leads to reduced settlement and increased stability and in the later case results in reduced lateral pressures. The use of lightweight fill behind pile-supported bridge abutments leads to reduced lateral pressures and drag loads on piles. In view of the increased use of these materials in geotechnical applications, it is necessary to determine engineering properties of lightweight aggregates.

Recently, data were presented [*1*] on expanded shale lightweight aggregate by performing one-dimensional compressibility and triaxial tests. In the present paper results of large-size one-dimensional compressibility tests, direct shear tests, and model pile torsion tests for determining static shear modulus are presented. In addition, results of direct shear tests performed to determine the angle of friction between geotextile and the lightweight aggregate are also presented.

The present testing program was undertaken as part of overall research pertaining to the use of geotextiles and lightweight aggregate in paved and unpaved road structures on peat subgrade. The experimentally determined values of shear moduli of aggregates and the angle of friction mobilized at the interfaces between geotextile, peat, and aggregate were used in a simple mathematical model to predict the load response of road structures. The basic compressibility and shear strength characteristic determined in the present investigation are general in character and can be used in other geotechnical applications.

Materials

Expanded shale aggregates from two sources were studied. Both Minto and Solite aggregates are manufactured by heating shale in a rotary kiln at temperatures of about 1,150°C. In the kiln the shale particles reach a pyroplastic condition and expand through the formation of gases, primarily carbon dioxide, that result from the decomposition of some of the compounds. The particles produced are subangular in shape, durable and chemically inert. The expanded, virtified particles are screened to produce the desired gradation for a particular usage. In the geotechnical applications, coarse aggregates with particle sizes between 5 mm to 25 mm are commonly used and materials within this gradation were used in the present testing program.

The Minto and Solite aggregates studied have a grain size distribution varying between 19 and 4.7 mm. The uniformity coefficient of Solite material is 1.4, whereas for the Minto material the coefficient is 1.5.

One-dimensional compressibility, direct shear, and model rigid piles subjected to torsion tests using normal weight crushed limestone aggregate were performed in 1986 [*2*], and these data are used in the present work for comparison purposes. This material has a uniformity coefficient of 1.4 and a grain size distribution varying between 19 and 4.7 mm.

Two types of geotextiles, woven and nonwoven, were used in a direct shear test apparatus to determine the angle of friction between the aggregate materials and the geotextiles. The properties of the geotextiles used are presented in Table 1. In this series, tests were performed using a normal weight limestone aggregate and lightweight aggregate from Minto. In all the tests carried out in this series, the aggregates were placed to achieve a loose relative density.

Model piles for torsion tests consisted of 60 mm O.D. steel pipe with 10-mm wall thickness. One of the model piles had a knurled surface, whereas the other pile had a relatively smooth surface. Torsion tests were performed using the crushed limestone aggregate and the Solite lightweight aggregate.

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TABLE 1—*Properties of the geotextiles studied (after Mirafi).*

Property	Mirafi P350	Mirafi P500
Structure	Nonwoven	Woven
Polymer	100% polyester	100% Polypropylene
Mass	400 g/m^2	160 g/m^2
Specific gravity	1.38	0.91
Thickness	3.5 mm	0.50 mm
Tensile strength	800 N	890 N
Break elongation	100%	22%
Burst strength	2400 kPa	2500 kPa
Opening size	0.140 mm	0.088 mm
Permeability	0.190 cm/s	0.005 cm/s

Equipment and Procedures

Compressibility Tests

One-dimensional compressibility tests were performed in a 550-mm-diameter, 305-mm-deep floating steel ring. The vertical loads were applied by a 100-ton capacity hydraulic jack, and settlements were measured by three dial gages. One of the special features of the large-size consolidometer is the provision of three strainsert bolts attached to the confining steel ring. These bolts are 19 mm in diameter and instrumented with strain gages. The bottom end of the bolt is connected to the outside wall of the floating ring. The top of the bolt is inserted into a slot in the cylindrical housing, which is attached to the bottom plate of the consolidometer (Fig. 1). With this arrangement, the frictional forces mobilized on the walls of the consolidometer exert tensile force on the strainsert bolts, which are monitored during the loading of soil specimens. For each load increment, the applied load at the top of the soil specimen is known, and the load at the bottom is calculated from the strainsert bolt data. The average axial stress on the soil specimen is calculated by taking the algebraic mean of the load at the top and bottom.

The friction mobilized along the perimeter of the floating ring increased with the applied axial load and the relative density of the soil specimen. At a maximum load of 150 kN, approximately 30% of the applied load was transmitted in side friction for loose specimens for all aggregates tested. These data indicate the importance of measurement and accounting for the side friction in large-scale one-dimensional compression testing.

Loose specimens were prepared by pouring soils through a hopper and hose arrangement [3]. Compact soil specimens were prepared by a raining method in which soil is deposited at a fixed rate from an appropriate sieve from a predetermined height to achieve the required relative density [4]. These procedures were also used for specimen preparation for the direct shear and the model pile tests.

All the compressibility tests were performed using a load increment ratio of about 1 except for the last load increment. During each load increment time-dependent settlements were monitored, and the next load increment was applied when the settlement under the previous load was complete. All the tests were done on dry specimens only.

Direct Shear Tests

The size of the shear box is 450 by 305 by 600 mm deep. The lower and upper boxes are each 300 mm deep. This equipment was developed for testing coarse materials and has been used to test peat, landfill samples, and coarse aggregates. The upper box is fixed in its position, and the lower box is pushed on specially designed roller bearings using a hydraulic jack (Fig. 2). A unique feature of the apparatus is the provision of two jacks for application of normal loads. The pressures in the jacks are manipulated during shearing of the soil specimen to prevent lifting and tilting of the shear box and to counteract moments generated by the nonaligned nature of horizontal forces on the lower and upper boxes of the shear device. The soil specimen is sheared at a fairly constant rate using the hydraulic jacks, and the horizontal loads are measured using either a proving ring or a load cell.

In the series of tests where geotextiles were used, a specially designed clamp was used and attached to the walls of the lower box to hold the geotextile in place. The clamp consisted of a turnbuckle and a wooden rod. In all the tests performed in this series, the geotextile was always located at the interface between the two halves of the shear box.

Model Pile Torsion Tests

Figure 3 shows the details of the setup for performing torsion tests on model piles. The soil container is 900 by 900 by 1200 mm deep and is made with stiffened laminated plywood. A flap

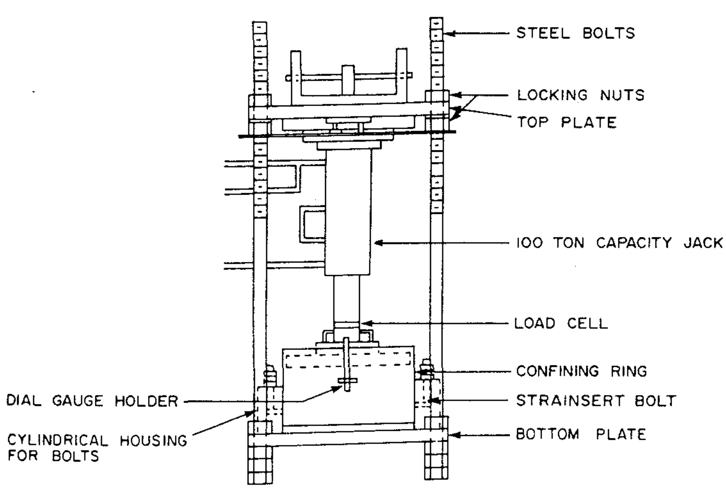

FIG. 1—*100-ton capacity consolidometer.*

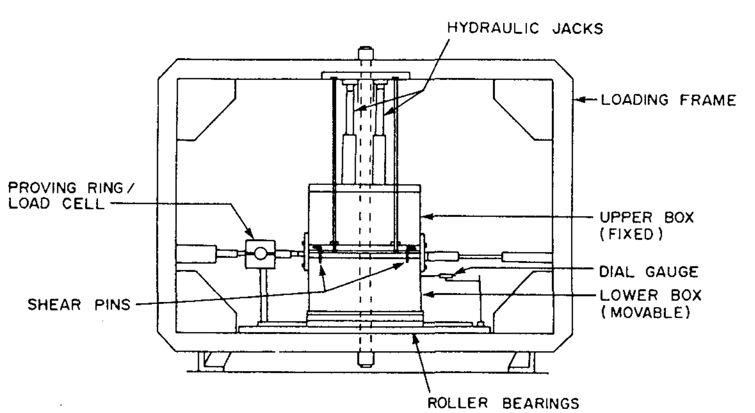

FIG. 2—*Large-size direct shear test apparatus.*

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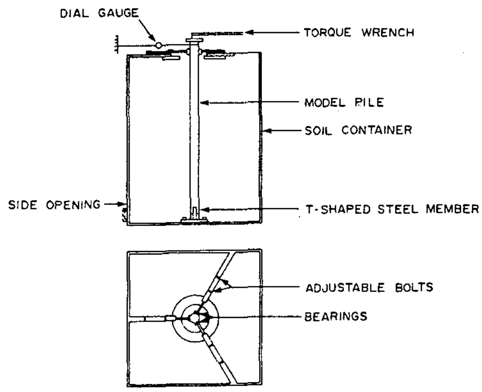

FIG. 3—*Model pile test set up.*

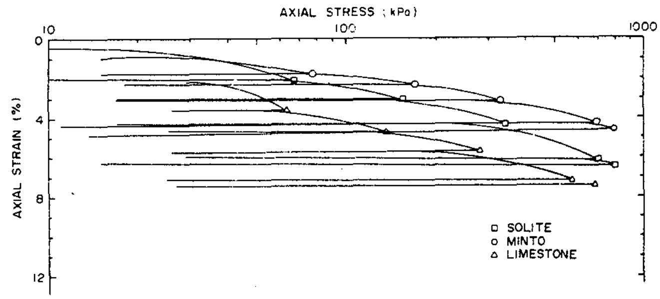

FIG. 4—*One-dimensional compression stress/strain curves for compact course aggregates.*

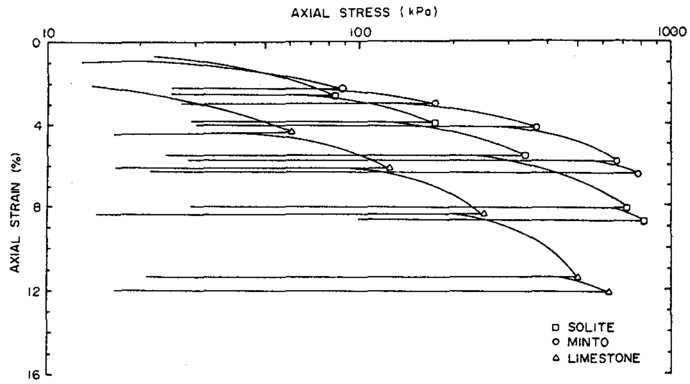

FIG. 5—*One-dimensional compression stress/strain curves for loose course aggregates.*

is provided at the bottom of the box for emptying the soil subsequent to performing torsion tests.

The model pile is 60 mm (O.D.) in diameter and 1300 mm long. It is held in a vertical position in the center of the box by three adjustable bolts located at the top of the box. Roller bearings are provided at the tip of the bolts to minimize friction between the pile and the bolts during specimen preparation. The model pile is hollow, and the bottom end rests on a smooth T-shaped steel member as shown in Fig. 3. Grease is applied to the bottom of the pile and the T-member to eliminate any contribution to the torsional resistance from the base of the pile.

Soil is placed with the model pile in a vertical position by ~ther raining or by hopper and hose arrangement, depending ..n the relative density required.

The torsional load is applied by a torque wrench, and the rotation of the pile is measured by an arrangement consisting of a wire attached to the outer surface of the pile and the other end connected to a dial gage (Fig. 3). The pile is rotated at a constant rate of about 0.5 degrees per minute. Torque and rotation readings are monitored continuously for 7 to 9 min to obtain complete torque versus rotation response.

Similar to compressibility and direct shear testing, all the tests with model piles were carried out on dry soils only.

Results

Compressibility Tests

Table 2 summarizes the materials and densities employed in the compressibility test series. Compressibility test results are presented in Figs. 4 and 5 for compact and loose samples, respectively. In addition to the test data for lightweight aggregates, compressibility curves for normal weight crushed limestone aggregate are also presented for comparison purposes. From Fig. 4 it is seen that the curvature and the slope of the stress-strain curves in confined compression for the first monotonic and subsequent cyclic loadings are similar for crushed limestone and Solite lightweight aggregate. The Minto lightweight aggregate with a similar grain size and relative density appears to be relatively less compressible. The trend reported in Ref *1* of the increased slope of stress-strain curve subsequent to first monotonic loading is observed both for the normal-weight and lightweight aggregates. Also, the unloading and reloading curves are very flat for all the aggregates tested in the present series.

At the end of five cycles of loading, grain-size analysis tests were performed to investigate the extent of particle breakage due to cyclic loading. Comparison of the grain-size distribution curves before and after cyclic loading indicated that no noticeable degradation had occurred. However, minor degradation occurring during the first few cycles of loading appears to lead to more stabilized interparticle contacts, and the material reacts in a much stiffer manner to subsequent load application.

TABLE 2—*Compressibility test series.*

Material	Dry Unit Weight, kN/m³ Loose	Compact
..nto	9.11	10.41
Solite	8.24	9.16
Limestone	16.73	18.50

Direct Shear Tests

The first series of direct shear tests was performed on loose and compact specimens of normal weight and lightweight aggregates to investigate the effect of the relative density on the angle of internal friction. Results are presented in Table 3 for all the aggregates tested. Typical shear stress/displacement curves are

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TABLE 3—*Angle of internal friction for coarse aggregates.*

Material	Angle of Internal Friction, degrees	
	Loose	Compact
Minto	40.5	48.0
Solite	40.0	45.5
Limestone	37.0	N/A
Solite[a]	39.5	44.5

NOTE: Unit weights for loose and compact specimens are the same as for consolidation tests.
[a]Data from Stoll and Holm, 1985, triaxial tests [1].

presented in Figs. 6 through 8 for lightweight and normal-weight aggregate specimens. The horizontal displacements at the time of failure were 25 to 30 mm for loose lightweight aggregate specimens. These displacements decreased to 17 to 20 mm when the relative density of the lightweight aggregate was compact. The loose normal-weight aggregate exhibited the most ductile behavior with the failure displacements of 40 mm. The normal stresses in all the tests on lightweight aggregate varied from 15 to 75 kPa, and the angles of internal friction presented in Table 3 are valid for this pressure range.

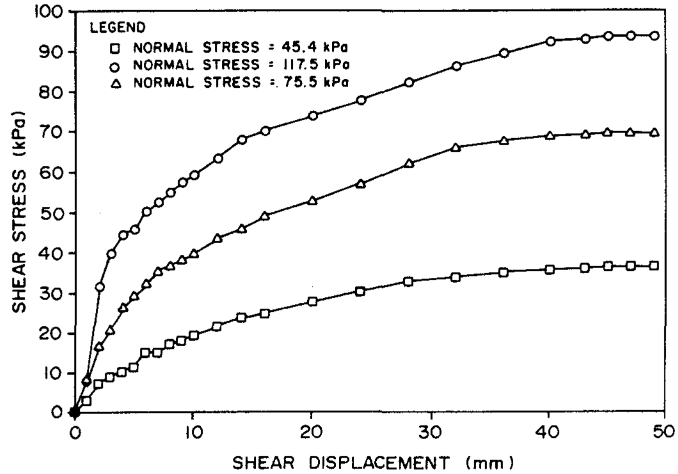

FIG. 8—*Shear stress/displacement curves for loose crushed limestone aggregate.*

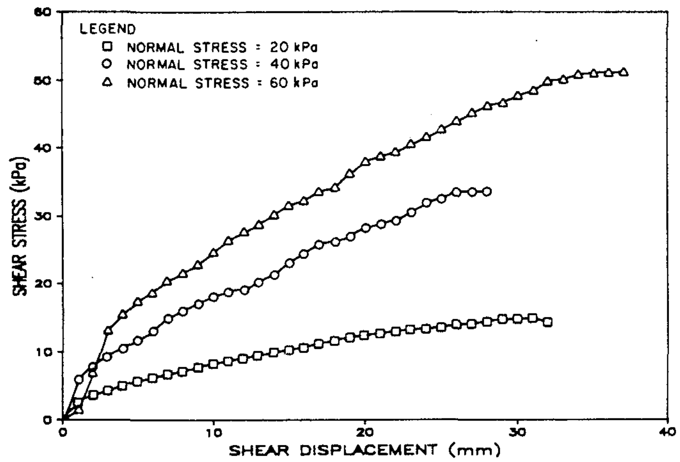

FIG. 6—*Shear stress/displacement curves for loose Minto lightweight aggregate.*

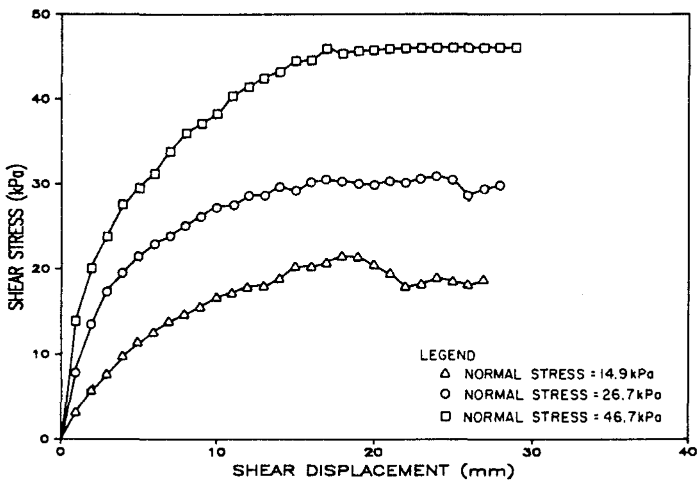

FIG. 7—*Shear stress/displacement curves for compact Solite lightweight aggregate.*

A comparison of the data from the present series with results of triaxial tests (on one material) reported in Ref *1* is presented in Table 3. It is seen that results of the direct shear tests are in good agreement with the triaxial testing data. From Table 3 it is seen that there is a difference in response between the Minto and Solite aggregate for compact relative density. A detailed visual examination indicated that the Minto lightweight aggregate is relatively more angular, which explains high values of angles of internal friction and less compressible behavior. The results of the direct shear testing indicate that the shear strength characteristics of lightweight aggregate are similar to commonly used normal weight aggregates.

Results of the direct shear tests performed with geotextiles incorporated at the interface between the upper and lower shear box are presented in Table 4. It should be noted that all the tests in this series were performed on loose aggregate specimens only. For the combination of aggregate/geotextile/peat subgrade, the road structure was inverted in the shear box due to high com-

TABLE 4—*Friction angle between geotextiles and course aggregates.*

Material in Lower Box	Material in Upper Box	Fabric	Friction Angle, Degrees
Limestone	Limestone	Woven	41.0
Limestone	Limestone	Nonwoven	42.0
Minto	Minto	Woven	47.0
Minto	Minto	Nonwoven	47.0
Peat	Peat	. . .	31.0
Limestone	Peat	Woven	32.0
Limestone	Peat	Nonwoven	32.0
Minto	Peat	Woven	32.0
Minto	Peat	Nonwoven	32.0
Peat	Peat	Woven	31.0
Peat	Peat	Nonwoven	30.0

NOTE: Water content of peat = 600%; Unit weight of limestone aggregate = 13.5 kN/m^3; Unit weight of Minto aggregate = 8.5 kN/m^3.

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pressibility of peat. The aggregate was placed in the lower box and peat in the upper box with geotextile located at the interface. With this arrangement, the geotextile remained at the interface between the two halves of shear box in spite of considerable compression of the peat under normal stress. Typical shear stress/displacement diagrams for the geotextile/aggregate interface are presented in Figs. 9 through 12.

From Table 4 it is seen that the friction angle between the peat and either the woven or nonwoven geotextile was equal to the angle of internal friction of peat. However, the friction angle between the normal weight aggregate and either geotextile was greater than the internal friction angle of the aggregate with higher unit weight. A similar trend was also observed for the Minto lightweight aggregate. This anomaly appears to be due to slight misalignment of the fabric at the interface. A comparison of the friction angles between the lightweight aggregate or the normal weight aggregate and the geotextile indicates that the interface friction characteristics are in general better for lightweight aggregates than the normal-weight aggregates.

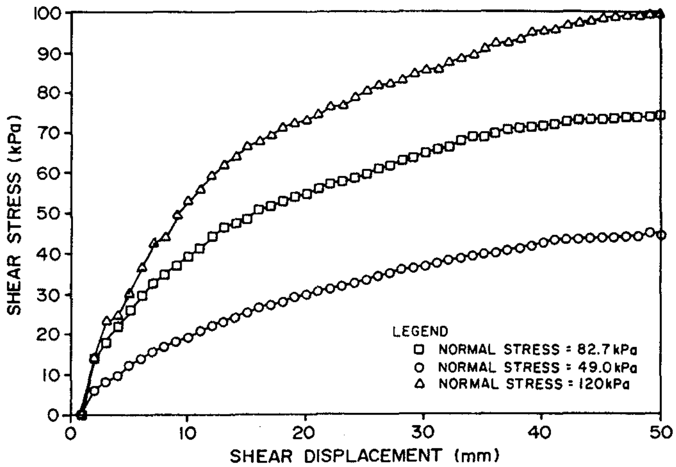

FIG. 11—*Shear stress/displacement curves for woven geotextile-incorporated loose limestone aggregate.*

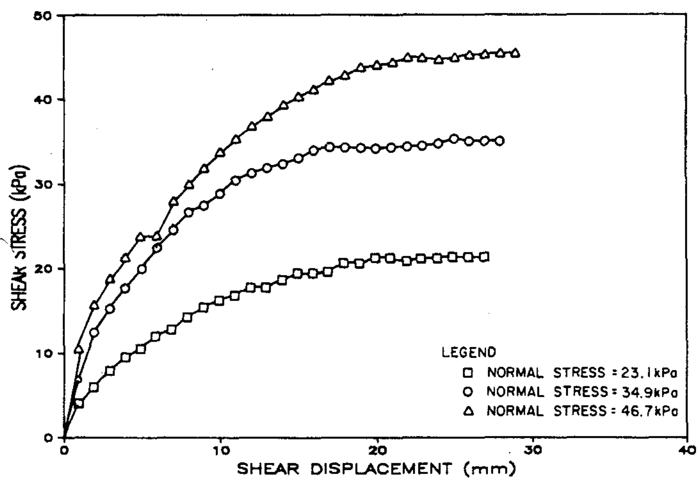

FIG. 9—*Shear stress/displacement curves for woven geotextile-incorporated loose Minto lightweight aggregate.*

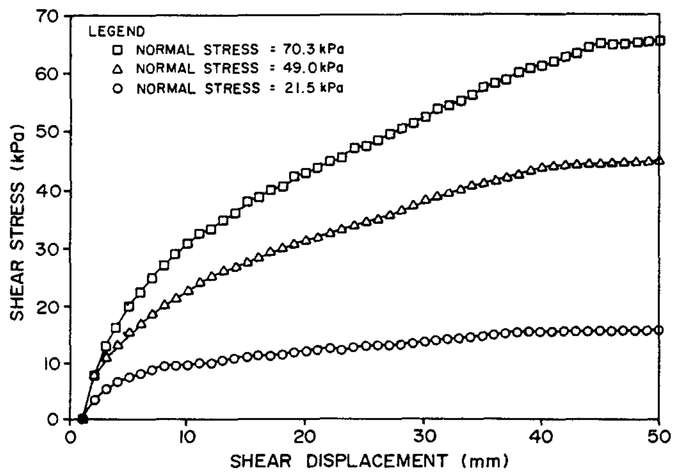

FIG. 12—*Shear stress/displacement curves for nonwoven geotextile-incorporated loose limestone aggregate.*

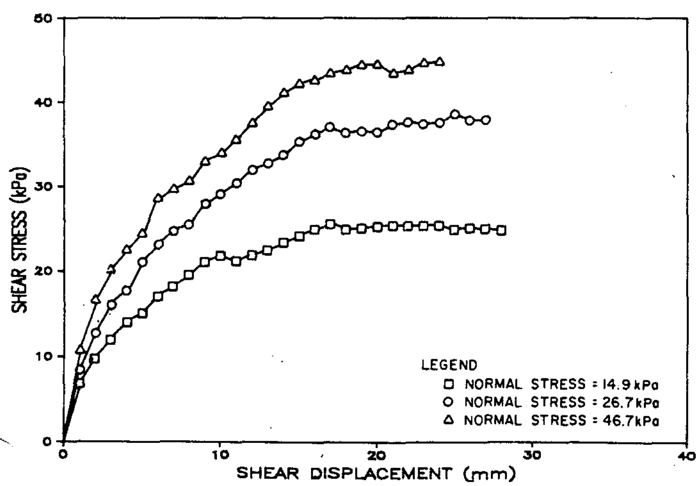

FIG. 10—*Shear stress/displacement curves for nonwoven geotextile-incorporated loose Solite lightweight aggregate.*

The unit weights of both the Minto and normal weight aggregates in the series of tests with fabric incorporated were lower than the earlier test series. This was due to the increased rate of mass flow during specimen preparation using hopper and hose arrangement.

Model Pile Torsion Tests

Typical torque versus rotation curves for model piles embedded in lightweight and normal-weight aggregates are presented in Figs. 13 and 14. The data presented in these figures are for the model pile with the smooth surface. In all the tests the rotation increased linearly with the applied torque up to a limiting value, and increased rotation was observed at a constant limiting torque. Similar results have been reported in Ref *5*.

The initial linear portion of the torque versus rotation curve was used to obtain static shear modulus based on the elastic solutions [*6*]. Assuming the soil to be elastic and the shear mod-

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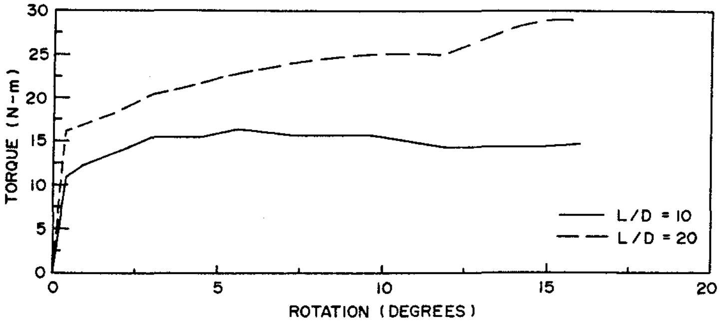

FIG. 13—*Torque/rotation curves for model pile embedded in Solite aggregate.*

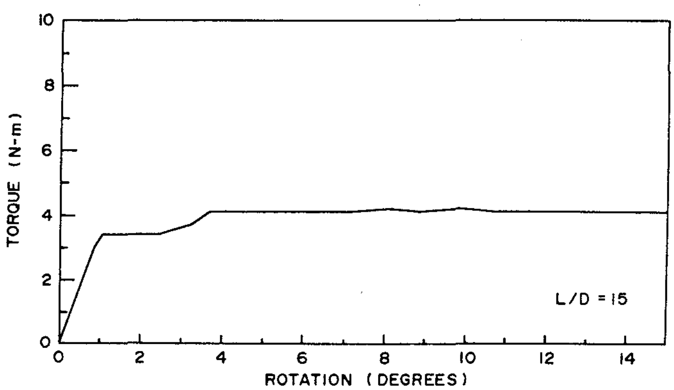

FIG. 14—*Torque/rotation curves for model pile embedded in limestone aggregate.*

ulus to be constant with depth, the expression for the shear modulus can be written as [*6*]

$$G_s = (TR)/(12.12R^3L\theta) \quad (1)$$

where

G_s = shear modulus,
T = torque,
R = outside radius of the pile,
L = length of pile embedment in soil, and
θ = angular rotation of the pile.

Equation 1 is valid for rigid piles and assumes no contribution to the torsional resistance from the base of the pile.

Results of the shear modulus values are presented in Table 5 for smooth and knurled piles for two depths of embedment in Solite and limestone aggregates. In addition to tests with coarse aggregates, model pile tests were also performed using silica sand. The primary objective of these tests was to compare the results with those published in Ref *5*. These results are also presented in Table 5 and indicate that the testing procedure used in the present study gave comparable results reported earlier in the literature.

TABLE 5—*Shear modulus for coarse aggregates, kN/m²*.

Material	Smooth Surface			Knurled Surface		
L/D	10	15	20	10	15	20
Sand	10.7	. . .	12.8	10.0	. . .	19.9
Solite	39.0	. . .	31.3	43.1	. . .	38.0
Limestone[a]	. . .	18.5	. . .	. . .	34.9	. . .
Sand[b]	. . .	11.6[c]	. . .	. . .	. . .	. . .

[a]From Addo 1986 [*2*].
[b]From Smith and Slyth, 1985 [*5*].
[c]*L/D* = 13, where *L* = depth of embedment of the pile, and *D* = diameter of the pile.

Conclusions

Results of the present testing program indicate that the compressibility and shear strength behavior of lightweight aggregates is similar to that of normal-weight aggregates used in roadway and engineered fill construction. In view of comparable geotechnical properties, lightweight aggregates can be used in place of normal-weight aggregates to reduce settlements and increase the stability of embankments on soft ground and paved or unpaved roadways. The results of model pile tests also indicate that the lateral load capacity of pile supported structures would not be significantly different if lightweight aggregates were used in place of normal-weight aggregate.

Acknowledgments

Most of the experimental work was performed by the following students: M. Young, K. Horgan, M. Murray, and K. Addo. Authors are thankful to them for their contributions. Some of the experimental facilities used were developed by Professor A. O. Landva of the Department of Civil Engineering, University of New Brunswick, and the authors are thankful to him for his many valuable contributions to the present research.

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Suggested SOLITE® Soil Fill Specifications

The information listed below is a suggested specification for SOLITE® Lightweight Aggregate soil fill. It is best to consult with SOLITE® engineering and sales representatives during a project's conceptual design phase in order to call for the most appropriate geotechnical and material physical properties.

Materials

Lightweight Aggregate fill shall be SOLITE® Lightweight Aggregate or approved rotary kiln substitute meeting all the requirements of a recently completed (2 years max.) ASTM C-330 certification. No by-product slags or cinders are permitted. Lightweight aggregate shall have a proven record of durability and be non-corrosive (less than 100 ppm chloride when measured by FHWA-RD-77-85) with the following physical properties:

A. Delivered Gradation

Sieve Size	% Retained
1″ (25 mm)	____
½″ (13 mm)	____
#4 (5 mm)	____

B. The dry loose unit weight shall be less than 55 pcf (880 kg/m^3). The lightweight aggregate producer shall submit verification of a compacted density of less than 60 pcf (960 kg/m^3) when measured by a one point test conducted in accordance with ASTM D-698 "The Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 5.5 lb. Hammer and 12 inch Drop" (AASHTO T-99).

C. The lightweight aggregate producer shall submit verification that the angle of internal friction shall be greater than 40° when measured in a triaxial compression test on a laboratory sample with a minimum diameter of 10 inches.

D. The maximum Los Angeles Abrasion loss when tested in accordance with ASTM C-131 (B grading) shall be 50%.

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Cyclic Plate Load Tests on Lightweight Aggregate Beds

A. J. VALSANGKAR AND T. A. HOLM

In recent years lightweight aggregates have been used increasingly with or without polymeric reinforcement in geotechnical applications. Results of a series of plate load tests performed on beds of expanded shale lightweight aggregate with or without geogrid reinforcement are presented. All tests were performed in a large test facility so that lightweight aggregate beds could be prepared using light compaction equipment. The relative density of the aggregate and locations of the polymeric reinforcement with respect to the base of the plate were varied in the experimental program.

The present testing program is part of an ongoing research project to determine the geotechnical properties of expanded shale lightweight aggregate at the University of New Brunswick, Canada. The research program began in 1985, and initially large-size one-dimensional compression and direct shear tests were carried out on lightweight aggregate specimens (*1*). The large direct shear apparatus was also used for determining angle of friction between geotextiles and expanded shale lightweight aggregate (*1*). Model footing tests on peat-geotextile-lightweight aggregate systems were undertaken following the direct shear and compression testing. Some of the results of this model testing have been reported by Valsangkar and Holm (*2*).

The scope of the testing program reported in this paper was to carry out preliminary laboratory plate load tests on beds of lightweight aggregate with or without geogrid reinforcement. The variables studied were relative density of the aggregate and location of the geogrid with respect to the base of the plate.

MATERIALS

Expanded shale aggregate manufactured by Solite Corporation was used in this study. This aggregate is manufactured by heating shale in a rotary kiln at a temperature of about 1150°C. At this temperature the shale particles reach a pyroplastic condition and expand through formation of gases that result from the decomposition of some of the compounds. The expanded, vitrified particles are screened to produce the desired gradation for a particular application. In the geotechnical applications, coarse aggregates with particle sizes between 5 and 25 mm are commonly used.

The lightweight aggregate used in the present study has a grain size distribution from between 19 and 4.7 mm with a uniformity coefficient of 1.4. Table 1 gives the shear strength data for the lightweight aggregates from two sources, along with the data for limestone aggregate.

The polymeric reinforcement used in the testing was a low-strength HDPE geogrid (Tensar SR-1). The properties of this geogrid as reported in Koerner (*3*) are shown in Table 2. The critical properties of the geogrid for its use as a soil reinforcement are aperture size in relation to particle size of the soil, long-term design load, tensile modulus at low strain levels, and service life of the grid (*3*).

EQUIPMENT AND PROCEDURE

Plate load tests were performed in a test pit 3.2 × 3.2 × 1.6 m deep. The facility is equipped with loading frames, and the reaction beam can be adjusted in the vertical position depending on the thickness of the soil in the test pit. The schematic details of the test setup are shown in Figure 1. A standard steel plate 300 mm in diameter was used in all the tests. The loads were applied by a hydraulic ram, and the settlements were monitored using two dial gauges. The data from the dial gauges and the level vial mounted on the plate were used to ensure that plate tilting did not occur during testing.

In all the tests performed, the thickness of the lightweight aggregate was at least 900 mm. Loose relative density was achieved by end dumping the aggregate in the test pit. An average dry density of 800 kg/m³ was achieved when the aggregate bed was prepared by end dumping.

After completion of testing of the loose lightweight aggregate, the aggregate was removed from the test pit. A small vibratory plate compactor (530- × 610-mm plate) was then used to compact 150-mm-thick lifts of lightweight aggregate. Density measurements made after compaction indicated that an average dry density of 950 kg/m³ was achieved.

Polymeric reinforcement was used in combination with compacted aggregate. In one series the geogrid was located 150 mm below the bottom of the plate, and in the second series, at a depth of 200 mm. The location of geogrid below plate was selected on the basis of previous research, which concluded that for one layer of soil reinforcement to be effective, it has to be placed within a depth equal to or less than the width of the footing (*4*).

When the plate was properly seated, load was applied with the hydraulic ram. For loose aggregate beds, the loads were monotonically applied in increments of 1 kN until a settlement of 12 mm was achieved. For the compacted aggregate bed, monotonically increasing loads were applied in increments of about 2 to 3 kN until the plate settlement reached 12 mm.

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TABLE 1 Angle of Internal Friction for Coarse Aggregates (1)

Material	Dry Density kg/m³		Angle of friction, degrees	
	Loose	Compact	Loose	Compact
Solite	840	934	40.0	45.5
Minto[a]	929	1,062	40.5	48.0
Limestone	1,706	1,887	37.0	---

[a] Minto expanded shale lightweight aggregate has the same gradation as Solite.
--- Unavailable

TABLE 2 Properties of SR-1 Uniaxial Geogrid (UX1400) (3)

Property		Value	
Structure		Punched-sheet drawn	
Polymer composition		Polyethylene	
Mass/unit area		512 g/m² ASTM D3776-84	
Aperture size:			
	Machine direction	145 mm	
	Cross machine direction	15 mm	
Thickness:			
	at rib	0.8 mm	ASTM D1777-64
	at junction	2.8 mm	ASTM d1777-64
Wide width strip tensile:			
	2% strain	14.6 kN/m	
	5% strain	24.8 kN/m	
	ultimate	54.0 kN/m	

Load increments for reinforced aggregate varied from 4 to 6 kN during the monotonic application of loads. Irrespective of the magnitude of the load increment, each load increment was maintained until the rate of settlement was less than 0.02 mm/min for a minimum of three successive minutes.

The choice of 12-mm settlement as the maximum settlement was adopted on the basis of the ASTM standard for plate load testing (ASTM D1195-64). However, load cycling before reaching 12-mm settlement was not carried out as recommended in ASTM D1195-65, because the primary objective of the study was to determine the coefficient of subgrade reaction for monotonic loading. The other reason for adopting the 12-mm settlement criterion and not cycling the load before this much settlement occurred is found in the work by DeBeer (5), which concluded that the settlement at the onset of bearing capacity failure of granular soils with high relative density is on the order of 5 percent of the width of the loaded area.

In all the tests performed, cyclic loads were applied after the monotonic load was applied to achieve a 12-mm settlement. In each case the maximum load corresponding to 12-mm settlement was applied six to eight times to study the behavior under cyclic loading. Each test was done at least twice to ensure that data and trends were reproducible.

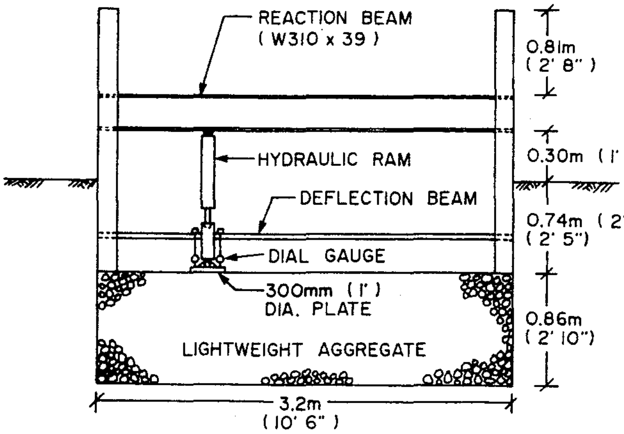

FIGURE 1 Test setup.

RESULTS

Plate load test results for unreinforced lightweight aggregate are presented in Figure 2 for compact and loose beds. The bearing stress for 12-mm settlement increased from 116 kPa to 456 kPa because of moderate compaction. The values of coefficient of vertical subgrade reaction were determined from the slope of the bearing stress–versus–settlement data obtained during the monotonic loading. The results are given in Table 3. Typically, values of coefficient of vertical subgrade reaction of 8 MN/m³ (loose) and 38 MN/m³ (compact) are used for normal-weight coarse-grained soils (6). Thus, the plate loading tests confirm that the behavior of tested coarse

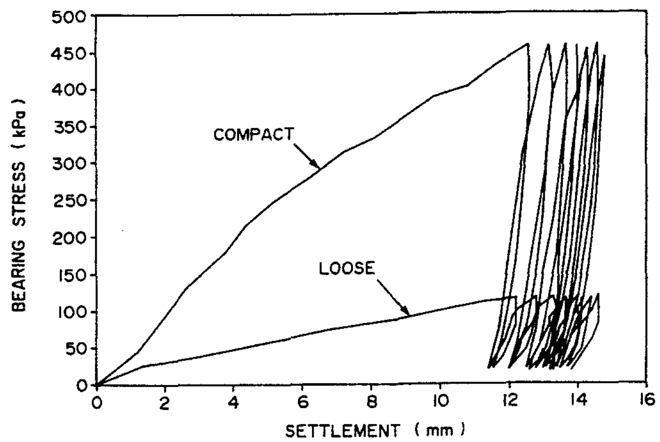

FIGURE 2 Effect of relative density on plate settlements.

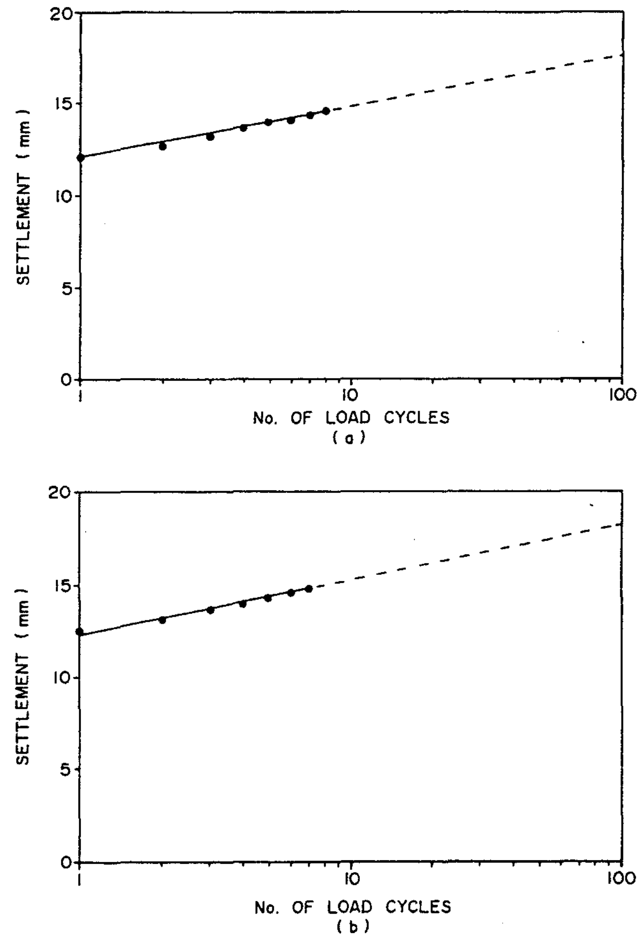

FIGURE 3 Cumulative settlements due to cyclic loading: *top*, loose, bearing stress = 116 kPa; *bottom*, compact, bearing stress = 456 kPa.

lightweight aggregate is similar to that of normal-weight aggregates.

The effect of cyclic loading on plate settlements is given in Figures 2 and 3. From Figure 2 it is seen that the slopes of the unloading and reloading curves are very steep when compared with the slope of the bearing stress–versus–settlement data during initial monotonic loading. The reloading coefficient of subgrade reaction for loose and compact aggregate beds is evaluated to be 190 and 1500 MN/m^3, respectively.

Figure 3 shows the effect of repetition of loading on the cumulative settlements for both loose and compact lightweight aggregate beds. Note that the linear trend observed between number of load cycles plotted on the logarithmic scale and cumulative settlement on natural scale, which is common for coarse-grained normal-weight soils (*7*), is also applicable to lightweight soils.

The beneficial effect of including geogrid reinforcement in compacted lightweight aggregate is seen from the data given in Figure 4. The bearing stress to cause 12-mm plate settlement increased from 456 to 1000 kPa, irrespective of whether the geogrid was located 150 or 200 mm below the base of the plate. The coefficient of vertical subgrade reaction due to the inclusion of geogrid reinforcement increased from 42 to 130 MN/m^3.

Figure 5 gives the effect of cyclic loading on the cumulative settlements. Again a linear trend is observed between the magnitude of settlement and number of cycles plotted on the logarithmic scale. Also, it is seen that the cumulative settlements observed for aggregate with geogrid reinforcement of 150 mm deep were somewhat lower than when the geogrid was at a depth of 200 mm (Figure 5). However, more testing is required to delineate this trend.

TABLE 3 Coefficient of Vertical Subgrade Reaction for Coarse Lightweight Aggregate

Test No.	Plate Diameter mm	Relative Density	Coefficient of Subgrade Reaction, MN/m^3
1	300	Loose	9
2	300	Loose	10
2	300	Compact	42
4	300	Compact	38

CONCLUSIONS

Results of the preliminary plate load testing program reported in this paper indicate that the coefficient of vertical subgrade reaction values of lightweight aggregates is similar to that of normal-weight aggregates used in roadway and engineered fill applications. The inclusion of geogrid as a soil reinforcement enhances the compressibility characteristics of the lightweight aggregate similar to the normal-weight aggregate. Even though relatively few tests have been done in this program, the extensive testing done previously at the University of New Brunswick, with the results of the present investigation, indicates that geotechnical behavior of coarse lightweight aggregate is similar to that of normal-weight aggregate.

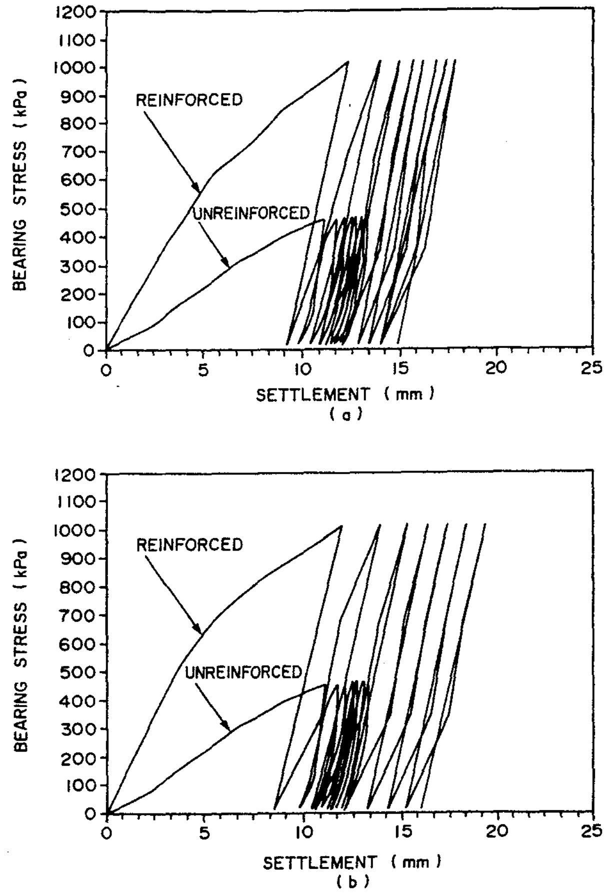

FIGURE 4 Effect of geogrid reinforcement on plate settlement response: *top*, geogrid at 150-mm depth; *bottom*, geogrid at 200-mm depth.

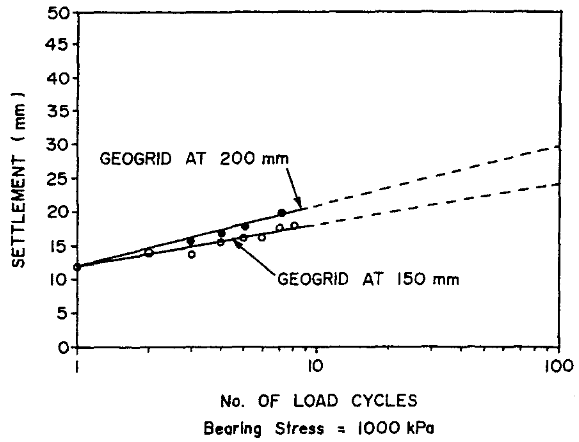

FIGURE 5 Cumulative settlements due to cyclic loading for geogrid-reinforced aggregate.

ACKNOWLEDGMENTS

The experimental work reported in this paper was done by undergraduate students R. S. Gallagher, I. Page, A. MacKenzie, and P. Mawhiney. Their efforts, and the assistance of the authors' technical staff, are greatly appreciated.

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ghtweight Fill Solutions to Settlement nd Stability Problems on Charter Oak Bridge Project, Hartford, Connecticut

JOHN P. DUGAN, JR.

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Lightweight Fill Solutions to Settlement and Stability Problems on Charter Oak Bridge Project, Hartford, Connecticut

JOHN P. DUGAN, JR.

Design and construction of the Charter Oak Bridge and approaches over soft soils were complex and challenging. To solve settlement and stability problems arising from highway and bridge construction over deep deposits of soft varved clay in the Connecticut River valley the following applications of lightweight fill were made. Lightweight fill was placed for the high approach fill for the east abutment. The reduced stresses imposed in the clay layer, combined with the lightweight fill's higher shear strength compared with that of an earth fill, solved this embankment stability problem. Lightweight fill was placed in approach embankments for a replacement bridge to reduce settlements of the adjacent existing bridge. To avoid minor settlements to an aging sanitary sewer that crossed the west approach, soil above the sewer was replaced with lightweight fill. The resulting stress reduction balanced effects of additional stresses imposed by nearby fills and pile driving. The overall slope stability of a wharf, with an anchored sheet pile bulkhead, was improved by replacing existing soil with a 1.5-m (5-ft) layer of lightweight fill.

This paper summarizes applications of lightweight fill (expanded shale) to solve settlement and stability problems arising from highway and bridge construction over deep deposits of soft varved clay in the Connecticut River valley.

More than 61 200 m^3 (80,000 yd^3) of lightweight fill was placed for the 14.0-m (46-ft)-high east approach fill. The reduced stresses imposed in the clay layer, combined with the lightweight fill's higher shear strength compared with that of an earth fill, solved the embankment stability problem. Lightweight fill was placed in approach embankments for a replacement bridge to reduce settlements of the adjacent existing bridge.

To avoid even minor settlements to an aging, 2.0-m (6.5-ft)-diameter sanitary sewer that crossed the west approach, soil above the sewer was excavated and replaced with lightweight fill. The resulting stress reduction balanced effects of additional stresses imposed by nearby fills and pile driving.

The overall slope stability of a wharf, with an anchored sheet pile bulkhead, was improved by replacing existing soil with a 1.5-m (5-ft) layer of lightweight fill.

PROJECT DESCRIPTION

The new Charter Oak Bridge, which links Hartford and East Hartford, Connecticut, was opened to traffic in August 1991, 72 months from the start of design and 40 months from the start of construction. The 6-lane, 1,037-m (3,400-ft)-long, $90 million multigirder steel structure built 61 m (200 ft) south of the old bridge carries U.S. Route 5 and State Route 15 over the Connecticut River and its flood plain. The project included extensive construction of approach roads and bridges, valued at $110 million.

Haley & Aldrich, Inc., 110 National Drive, Glastonbury, Conn. 06033.

LIGHTWEIGHT FILL

Lightweight fill was expanded shale aggregate produced by expanding shale, clay, or slate by heating in a rotary kiln to approximately 1149°C (2,100°F). The expanded, vitrified mass was then screened to produce the desired gradation. The pores formed during expansion are generally noninterconnecting. The particles are subgranular, durable, chemically inert, and insensitive to moisture.

For this project, the following gradation was specified

Square Mesh Sieve Size	*Percent Passing by Weight*
25.4 mm (1 in.)	100
19.0 mm (¾ in.)	80–100
9.5 mm (⅜ in.)	10–50
No. 4	0–15

For design, a unit weight of 961 kg/m^3 (60 lb/ft^3) and an angle of internal friction of 40 degrees were used.

The lightweight fill was placed in 0.61-m (2-ft)-thick lifts and compacted with four passes of a relatively light 4.5-Mg (5-ton) vibratory roller operating in vibratory mode. The compaction effort was designed to prevent overcompaction, which could result in breakdown of particles leading to a more well-graded material with higher-than-desirable unit weight.

SUBSURFACE CONDITIONS

The site is in the floodplain of the Connecticut River. Subsurface conditions, in the order of increased depth, are

- Existing fill, (a) random fill [1.5 m (5 ft) to more than 4.6 m (15 ft) thick] containing man-made and discarded organic material and (b) roadway fill that is relatively free of nonmineral material.
- Alluvial sand and silt stratum consisting of floodplain and channel deposits 9.1 to 12.2 m (30 to 40 ft) thick.
- Very soft to soft, varved clay and silty clay, in regular layers 6.3 to 12.7 mm (¼ to ½ in) thick, [more than 25.4 mm

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(1 in) thick at some locations], deposited in glacial Lake Hitchcock during the Pleistocene epoch. These deposits are approximately 10.7 m (35 ft) thick on the west side and from about 27.5 to 45.8 m (90 to 150 ft) thick on the east side of the river. Compressibility, stress history, and undrained shear strength data are given in Table 1. For other engineering properties, see work by Smith (*1*).

- Glacial till stratum consisting of dense to very dense sandy silt with subordinate coarse to fine gravel, clay, and occasional cobbles.
- Groundwater levels within the alluvial sand and silt and approximately 1.5 m (5 ft) above normal level in the Connecticut River.

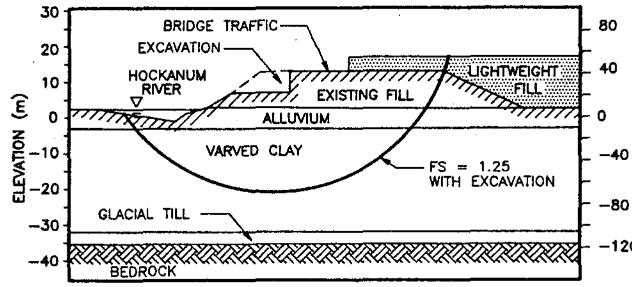

FIGURE 1 Slope stability for east abutment. Final conditions with lightweight fill.

EMBANKMENT STABILIZATION

If constructed of earthen material 2,002 kg/m³ (125 lb/ft³), the maximum 14.0-m (46-ft)-high embankment for the Charter Oak Bridge's east approach would not have an acceptable safety factor against slope instability. The safety factor against slope failure toward the adjacent Hockanum River, using earth fill, was estimated to be only 1.0 to 1.1 (Figure 1).

Many stabilization alternatives were considered. A toe berm placed in the river was the most economical but rejected to avoid delays that would occur because of time required to obtain environmental permits. Therefore, it was decided to construct the embankment of lightweight fill. The 62 730 m³ (82,000 yd³) of lightweight fill is one of the largest quantities of lightweight fill placed for one project in the United States.

Lightweight fill significantly reduced stresses in the weak varved clay. Even so, it was necessary to excavate a portion of the approach fill to the existing bridge to provide the design safety factor of 1.25. The lightweight fill's 40 degree angle of internal friction was higher than provided by earth fill, which increased resisting forces along the potential failure plane.

TABLE 1 Compressibility and Strength Parameters for Varved Clay at East Abutment

The clay is overconsolidated by at least 3.5 KPa (3.5 kips/ft²) at all depths.

<u>Compression Ratio</u>		
Virgin compression	0.31 to 0.37	
Recompression	0.03	
<u>Coefficient of Consolidation</u>		
Normally consolidated	0.0004 cm²/sec (0.04 ft.²/day)	
Overconsolidated	0.0037 cm²/sec (0.37 ft.²/day)	
<u>Coefficient of Secondary Compression</u>		
El. 0 to -30	1.06% per log cycle time	
El. -31 to -60	0.87% per log cycle time	
Below El. -60	0.98% per log cycle time	
$\frac{\text{Coefficient of Horizontal Permeability}}{\text{Coefficient of Vertical Permeability}} = 5$		
<u>Shear Strength, $s_u=S(OCR)^m\sigma_v$</u>		
	<u>S</u>	<u>m</u>
Undrained	0.19	0.7
Plane Strain Compression	0.21	0.8
Plane Strain Extension	0.20	0.75
Direct Simple Shear	0.14	0.7

Another benefit of the lightweight fill was the significantly reduced settlement, compared with an earth fill. The total settlement, over the first 15 years, of a lightweight fill embankment was predicted to range from 0.43 to 0.64 m (1.4 to 2.1 ft), compared with estimates of up to 1.98 m (6.5 ft) for earth fill. Observed settlement at the east abutment over a year is in line with the predicted values. Hence, the surcharge fill and vertical drains that were planned to speed consolidation of an earth fill were unnecessary. Nevertheless, the lightweight fill technique cost an additional $2 million in construction compared with the more conventional earth fill/berm/surcharge design.

SETTLEMENT REDUCTION AT EXISTING BRIDGE

A part of the overall project was replacement of Route 15 over Main Street in East Hartford, Connecticut, with a new bridge—a single-span structure 55.8 m (183 ft) wide, at the existing bridge, but extending 21.4 m (70 ft) north and 7.6 m (25 ft) south. Plans called for stage construction, with traffic maintained on the existing bridge while the north section of the new bridge was built. Then traffic was carried entirely on the north half of the new bridge while the existing bridge was being demolished and the south half of the bridge being built. Lightweight fill made it possible to keep the existing bridge in service while the north portion of the new bridge was being built and to avoid more expensive alternatives to prevent settlement.

The existing bridge is supported on spread footings bearing on a sand layer over approximately 42.7 m (140 ft) of soft varved clay. A recent inspection had reported 7.6 cm (3 in.) settlement of the west abutment and rotation and horizontal movements of both abutments of the single-span bridge. Temporary corrective repairs were planned; however, there was little tolerance for additional deflections.

Although the new bridge was designed to be supported on deep end-bearing piles, the 7.6-m (25-ft)-high approach fills would increase stresses and lead to settlements in the clay beneath the existing bridge. If an earthen embankment was used, predicted bridge settlements ranged from 1.3 to 5.1 cm (½ to 2 in.), which were considered intolerable. The project was therefore designed using lightweight fill for portions of the approach embankments within 22.9 m (75 ft) of the existing bridge. The lightweight fill reduced stress increases in the clay, lowering predicted settlements of the existing bridge to tolerable limits, to approximately half the magnitudes for earth fill. Measured settlements of the two bridge abutments, during the 1½-year period between embankment placement and demolition of the bridge, were 0.16 cm (¾ in.) and 0.22 cm (1 in.), which are within the range expected for the lightweight fill.

The lightweight fill option was significantly less expensive than underpinning the existing bridge and lengthening the new bridge to provide greater distance between the approach fills and the existing structure.

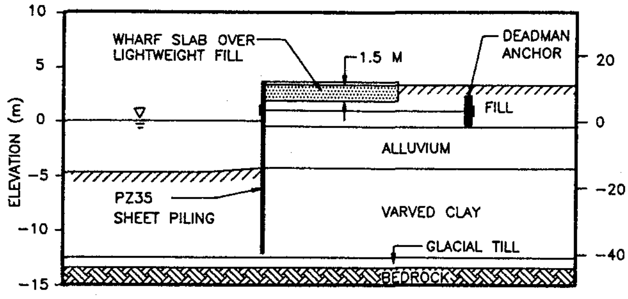

FIGURE 3 Lightweight fill placed to improve stability for wharf's sheet pile bulkhead.

SETTLEMENT PREVENTION AT EXISTING SEWER

A 2.0-m (6.5-ft)-diameter sewer crosses the existing and new bridge alignments between the west abutment and Pier 1. This 60-year-old cast-in-place concrete pipe founded in the loose silty alluvium is underlain by varved clay (Figure 2). Preload fill for construction of the bridge, adjacent pile driving, and new alignment of I-91 northbound required up to 6.1 m (20 ft) of fill over the sewer and would cause settlements in the varved clay and unacceptable movements in this old pipe.

The most severe settlement problem was solved by designing a pile-supported bridge to carry I-91 over the sewer pipe. Nevertheless, stress increases in the clay from the adjacent approach fills and the effects of pile driving were estimated to cause 2.5 to 5.1 cm (1 to 2 in.) of settlement beneath the pipe. To prevent pipe settlement, 1.5 m (5 ft) of alluvium from above the pipe was replaced with lightweight fill. This decreased the effective stress in the clay below the pipe by approximately 300 *P* (300 lb/ft²) and counteracted settlement effects from the other sources. No significant pipe settlement was measured.

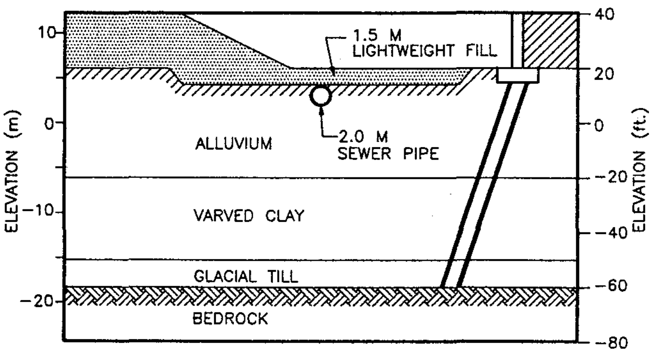

FIGURE 2 Lightweight fill above MDC sewer pipe.

WHARF STABILIZATION

The project included construction of a wharf and boat launch ramp along the west shore of the Connecticut River south of the Charter Oak Bridge. Lightweight fill was designed to provide stability for the wharf's anchored sheet pile bulkhead.

The bulkhead retains 7.6 m (25 ft) of soil above dredge level in the river (Figure 3). Stability analyses of circular failure surfaces indicated an unacceptably low factor of safety. As an alternative to anchoring a stiffer wall into underlying bedrock, a layer of lightweight fill was designed to reduce stresses in the weak varved clay and alluvium deposits and increase the factor of safety for overall slope stability to 1.25. The design called for replacing existing soil with a 1.5-m (5-ft) thickness of lightweight fill. The 0.2-m (8-in.)-thick reinforced concrete wharf slab was placed on a 0.3-m (12-in.)-thick layer of compacted gravel fill over the lightweight fill.

CLOSING

Design and construction of the Charter Oak Bridge and approaches over soft soils proved to be complex and challenging. Lightweight fill was an invaluable tool to increase slope stability and reduce settlements, both for facilitating the new construction and protecting sensitive existing structures.

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Model Tests on Peat – Geotextile – Lightweight Aggregate System

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ABSTRACT

Considerable research has been done in recent years dealing with the interaction of normal weight aggregate and geotextiles overlying soft compressible soils. In some instances, lightweight aggregate is used instead of normal weight aggregate to reduce settlements, and in the bridge abutment areas, to minimize lateral forces and to reduce drag loads on piles. However when used with geotextiles, it is not known whether the overall roadbed stiffness is affected when lightweight aggregate is used in place of normal weight fill. This paper reports the results of experimental research dealing with interaction of lightweight aggregate and geotextiles overlying peat subgrades. Variables investigated in the present study are: differing aggregate types and densities, thickness of the aggregate layer and geotextile types. The results indicate that the overall roadbed stiffness is unaffected when lightweight aggregate is used instead of normal weight aggregate, for small deflections and initial load application.

1 INTRODUCTION

Many of the present design methods of geotextile reinforced roadways are based on qualitative and quantitative data obtained from model tests. In

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recent years, field investigations have also been undertaken to validate findings of the model tests and analytical methods.

All the available data indicate that incorporation of geotextiles in a road structure leads to an improved performance in one way or another. However, the mechanism by which this improvement takes place is not fully understood and has attracted the attention of research workers. Heinjen and Lubking[1], and Grossmann and Mindner[2], indicate that the reinforcement function of geotextiles is insignificant and that the primary reason for improved performance is separation. In contrast, Barenberg *et al.*[3] and LeFlaive[4] report a significant reinforcement effect by a geotextile in a roadbed structure. In view of such contradictory evidence, an extensive laboratory testing program has been undertaken at the University of New Brunswick with the results reported to date by Douglas *et al.*,[5] and Douglas and Kelly.[6] These studies have confirmed the earlier findings that the reinforcement function of geotextiles is insignificant, at least for small deflections and during initial load cycles.

The present work is a continuation of the studies initiated by Douglas *et al.*[5] where normal weight fill is under study. The primary object of this investigation is to investigate the lightweight aggregate–geotextile–soft soil subgrade interaction. To achieve this goal, a series of model tests were performed by placing lightweight aggregate on a geotextile which in turn rested on a prepared peat subgrade. Crushed limestone and two rotary kiln produced structural grade expanded shale lightweight aggregates, and woven and nonwoven geotextiles were used in the experimental program. The loads were cycled and the thickness of the aggregate layer was varied.

2 MODEL TEST APPARATUS

Model road structures were constructed and tested in a steel box $1220 \times 300 \times 610$ mm developed by Douglas *et al.*[5] and modified by Addo.[7] The schematic diagram of the apparatus is shown in Fig. 1. Deformations of the entire roadbed structure could be monitored through the plexiglass sides which were stiffened by steel angles. Due to the highly compressible nature of the peat subgrade, an arrangement of pulleys and balancing weights was used to counterbalance the dead weight of the loading apparatus. In this way, small loads could be applied to the roadbed structure, starting from a true zero load.

The peat subgrade was prepared adopting the procedures developed by Jarrett.[8] The procedure consisted of wetting a fixed mass of horticultural sphagnum peat in the test box to achieve a water content of approximately 1200%. The peat and water mixture was left undisturbed for 12 hours to

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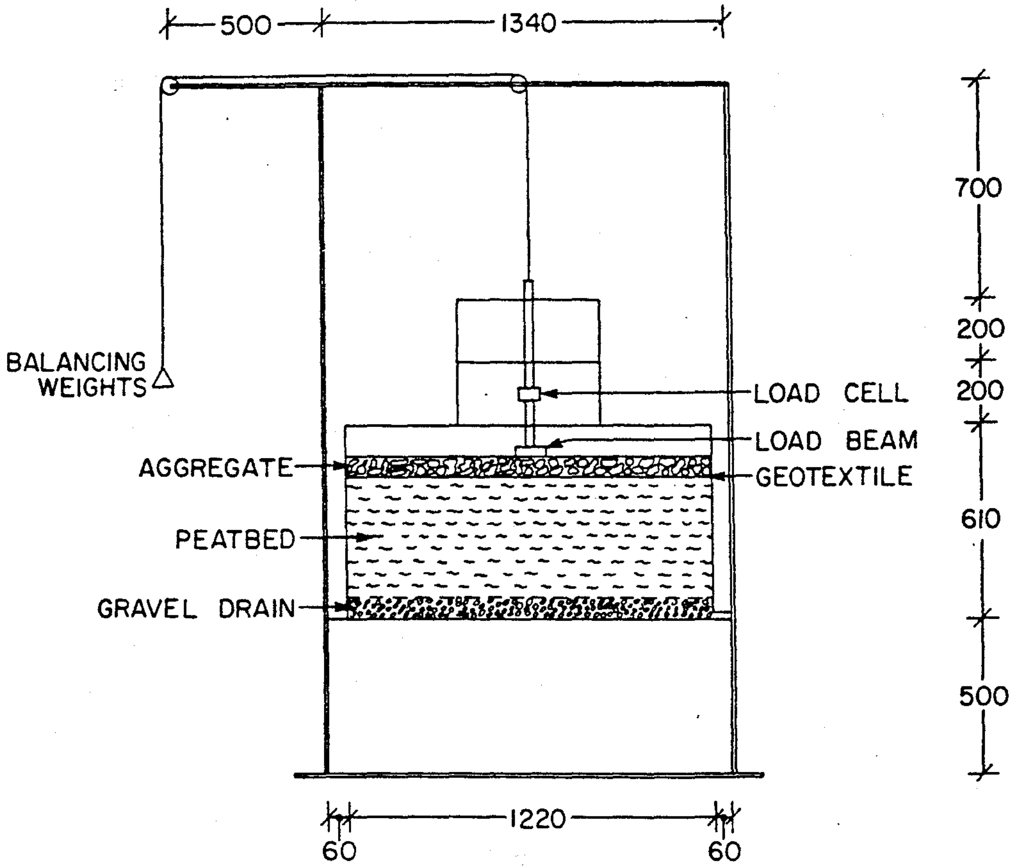

Fig. 1. Small scale model test apparatus.

ensure that a uniform moisture content was established. Subsequent to moisture content equalization, the peat layer was drained through the gravel drain at the base of the box. A sheet of geotextile was then placed on the top of the prepared peat bed and a layer of aggregate was placed on the top of the geotextile using 'raining' method to achieve low relative density.

The loads were applied to the gravel surface using a 75 mm wide strip footing in increments, and the settlement of the footing and of the roadbed surface outside the loaded area were monitored. During each load increment, settlements were recorded. Subsequent load increments were applied when the rate of settlement was less than 0·1 mm/min. In most of the tests, at least two load cycles were performed to investigate the effect of initial load cycling.

In addition to performing a large number of small model tests, one large scale model test was performed in a steel bin 2·4 m square by 1·5 m deep. The strip footing width used for testing in the large bin was 250 mm. The details of this facility are given by Douglas and Kelly.[6]

For each of the fabric and aggregate types used, the thickness of the aggregate was varied from 25 mm to 110 mm in small model tests. This

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thickness, h, expressed in terms of width of the footing, B, gave ratios of h/B varying from 0·33 to 1·5. The test in the large bin was performed for a h/B ratio of 0·33 and non-woven geotextile.

3 MODEL MATERIALS

Tests using normal weight crushed limestone aggregate were performed by Addo[7] and these data are used in the present work for comparison purposes. This material had a uniformity coefficient of 1·43, and a grain size distribution varying between 19 mm and 4·7 mm. The unit weight of this material in the small model tests varied from 13 to 15 kN/m^3.

Two structural grade, rotary kiln produced lightweight aggregates were studied in the present investigation. Both Minto and Solite aggregates had a grain size distribution varying between 19 mm and 4·7 mm. The uniformity coefficient of Solite material was 1·4, whereas for Minto lightweight aggregate the coefficient was 1·5. The unit weight of both the aggregates was about 6 kN/m^3 for the loose relative densities achieved in the model tests.

Two types of geotextiles, namely Mirafi P350 (nonwoven polyester) and P500 (woven polypropylene) were used in the present study. Wide strip tensile testing of these geotextiles was performed by Addo.[7] The pertinent properties in a summary form are presented in Tables 1 and 2 based on the manufacture's data as well as wide strip testing.

A series of direct shear tests were performed in a large direct shear box of dimensions 430 × 285 × 460 mm to determine angle of shearing resistance of normal weight and lightweight aggregates. For the low relative densities achieved in model tests, the angle of internal friction of both the lightweight

TABLE 1
Fabric Properties (manufacturers' literature) (after Mirafi[10])

Properties	*Mirafi P350*	*Mirafi P500*
Structure	nonwoven	woven
Polymer	100% polyester	100% polypropylene
Mass	400 g/m^2	160 g/m^2
Specific gravity	1·38	0·91
Thickness	3·5 mm	0·5 mm
Tensile strength	800 N/5 cm	890 N/5 cm
Break elongation	100%	22%
Burst strength	2 400 kPa	2 500 kPa
Opening size	140 μm	88 μm
Permeability	$1{\cdot}9 \times 10^{-3}$ m/s	5×10^{-5} m/s

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TABLE 2
Ratios of Woven to Nonwoven. Mechanical Properties:[7] Wide Strip Tests

Direction for woven	*Tensile strength ratio*	*Failure strain ratio*	*Tensile modulus ratio*
Machine	1·3	0·7	1·6
Cross machine	1·9	0·5	4·2

aggregates was found to be 40°. These data confirm the results of triaxial compression tests of lightweight aggregate reported by Stoll and Holm.[9] The corresponding value for normal weight aggregate was 37°.

4 TEST RESULTS

Typical load settlement curves for small model tests are presented in Figs 2 to 4. Results from large bin model tests are presented in Fig. 5. It should be noted that the water content of the peat subgrade varied from 600% to 800% during the testing program. However, this variation did not affect the overall stiffness values as found earlier by Douglas and Kelly.[6]

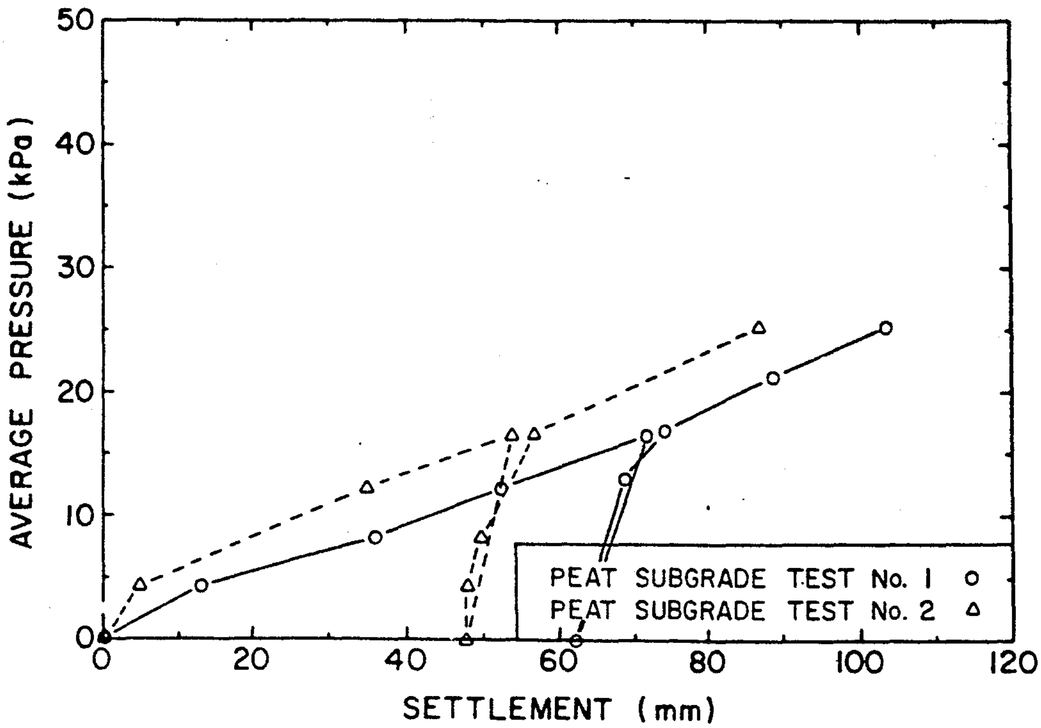

Fig. 2. Response of peat subgrade to average footing pressure.

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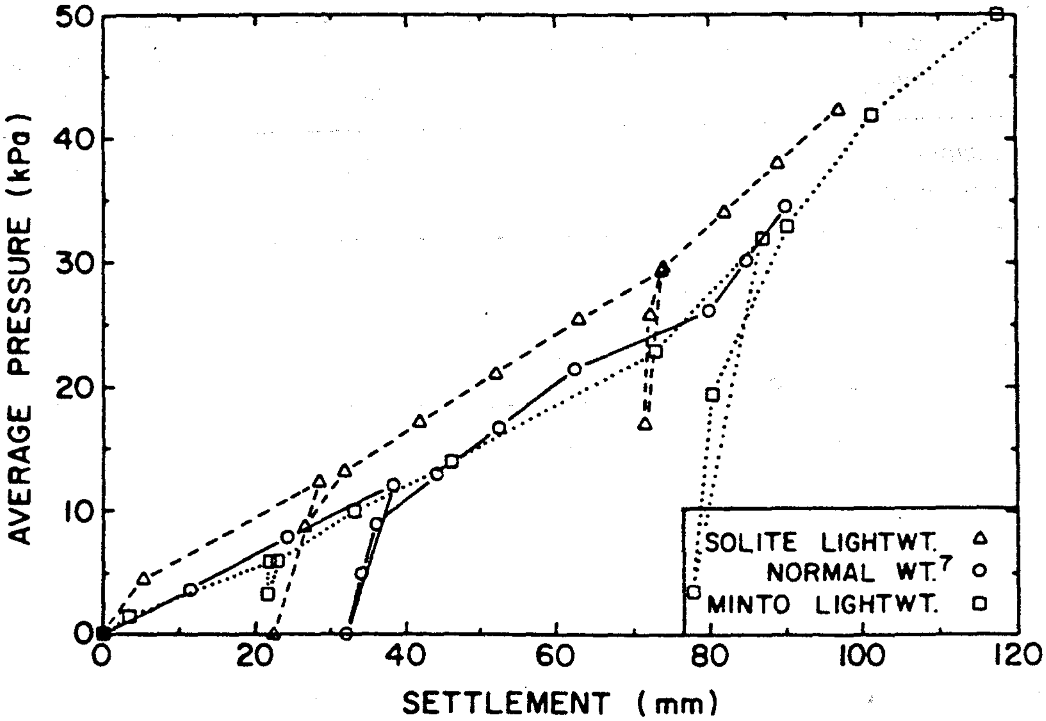
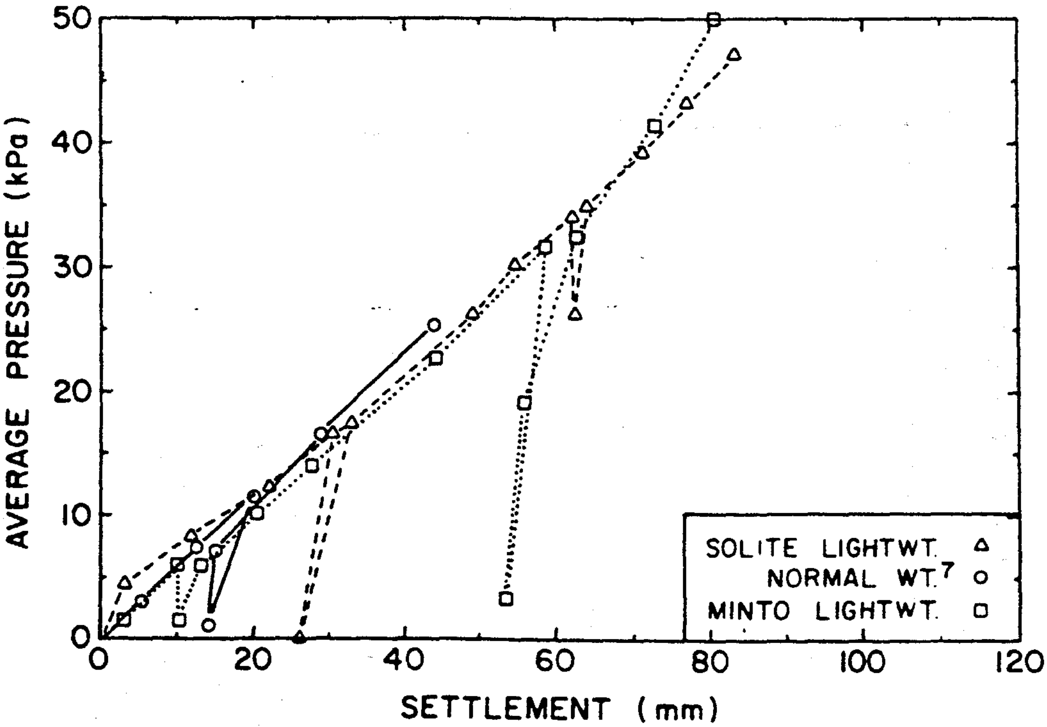


(a)



(b)

Fig. 3. Response of a roadbed structure reinforced with woven geotextile. (a) $h/B = 0{\cdot}33$; (b) $h/B = 1{\cdot}5$.

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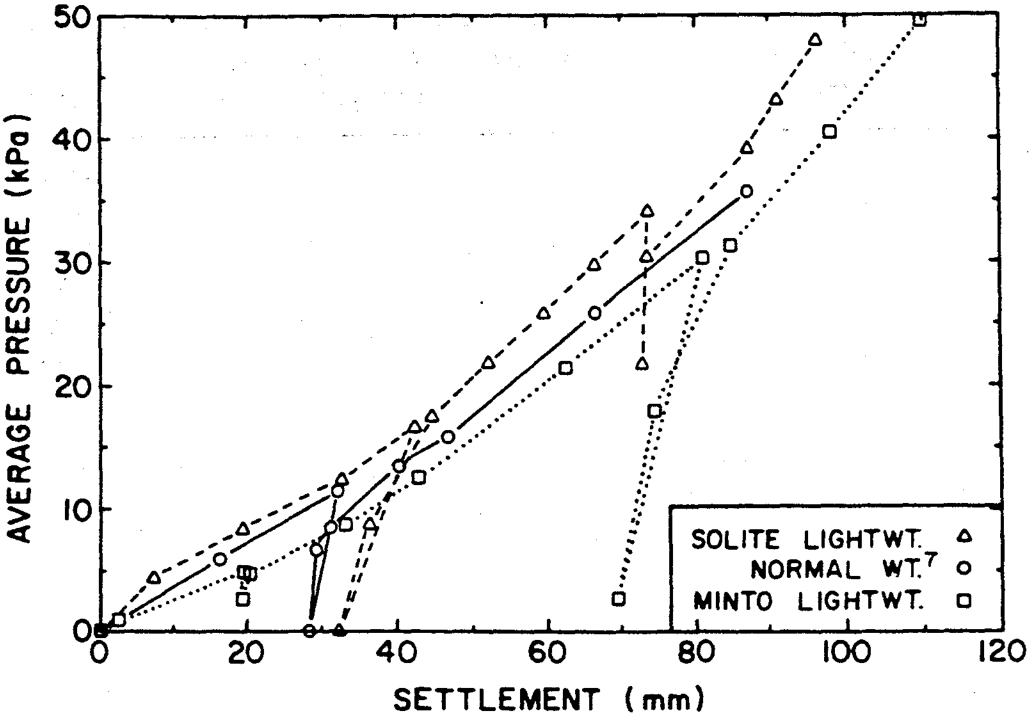
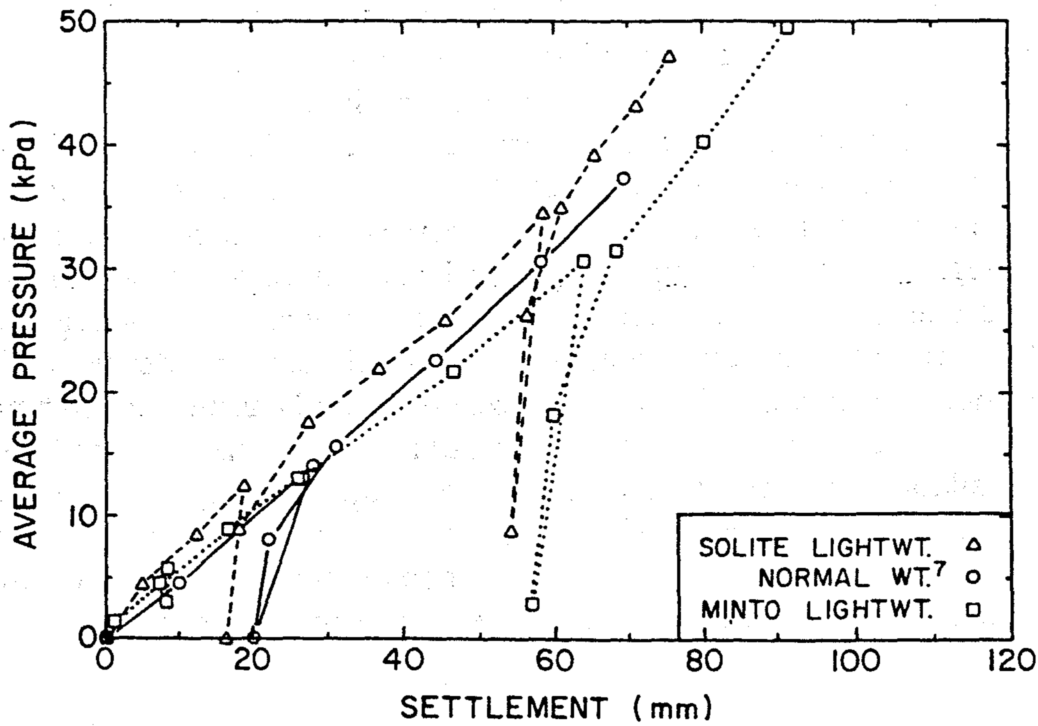


(a)



(b)

Fig. 4. Response of a roadbed structure reinforced with nonwoven geotextile. (a) $h/B = 0{\cdot}33$; (b) $h/B = 1{\cdot}5$.

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TABLE 3
Small Model Tests: Average[a] Initial Stiffness Values (kN/m^3)

h/B	*Normal weight roadbed*	*Solite lightweight aggregate roadbed*	*Minto lightweight aggregate roadbed*	*Type of geotextile*
0·33	346	350	378	Woven
0·67	412	306	446	Woven
1·0	453	441	456	Woven
1·5	558	530	533	Woven
0·33	344	364	434	Nonwoven
0·67	377	428	444	Nonwoven
1·0	443	461	502	Nonwoven
1·5	518	591	573	Nonwoven

[a] Average initial stiffness values as quoted are based on the average slope of the curves up to 50 mm settlement.

The results presented in Figs 3 and 4 indicate that the average initial tangent modulus or stiffness of the roadbed structure up to a settlement of 50 mm was independent of the type of aggregate or geotextile used. Thus, all the combinations investigated exhibited more or less equivalent initial performance. To provide base data, the load settlement response of the model footing resting on the peat subgrade is given in Fig. 2.

Table 3 presents, in a summary form, average initial stiffness values of model roadbed structures investigated in the small scale apparatus. The results indicate that the roadbed stiffness increases with increasing thickness of the aggregate layer but appears to be independent of type of geotextile or aggregate used. It can be noted from Fig. 5 that the response of large scale model was softer in comparison with the small scale model tests. This difference in behavior is likely due to size effects and has been noted before by many researchers working in the area of model testings. Due to limited data available, no attempt can be made in this paper to propose correlations to account for size effects. However, the results of preliminary testing in the large bin facility confirm the findings of the small model tests that the initial roadbed stiffness is not dependent on the type of aggregate used.

5 CONCLUSIONS

This paper presents results of an experimental research dealing with lightweight aggregate–geotextile–peat subgrade interaction. The primary objective of the study has been to investigate whether replacement of

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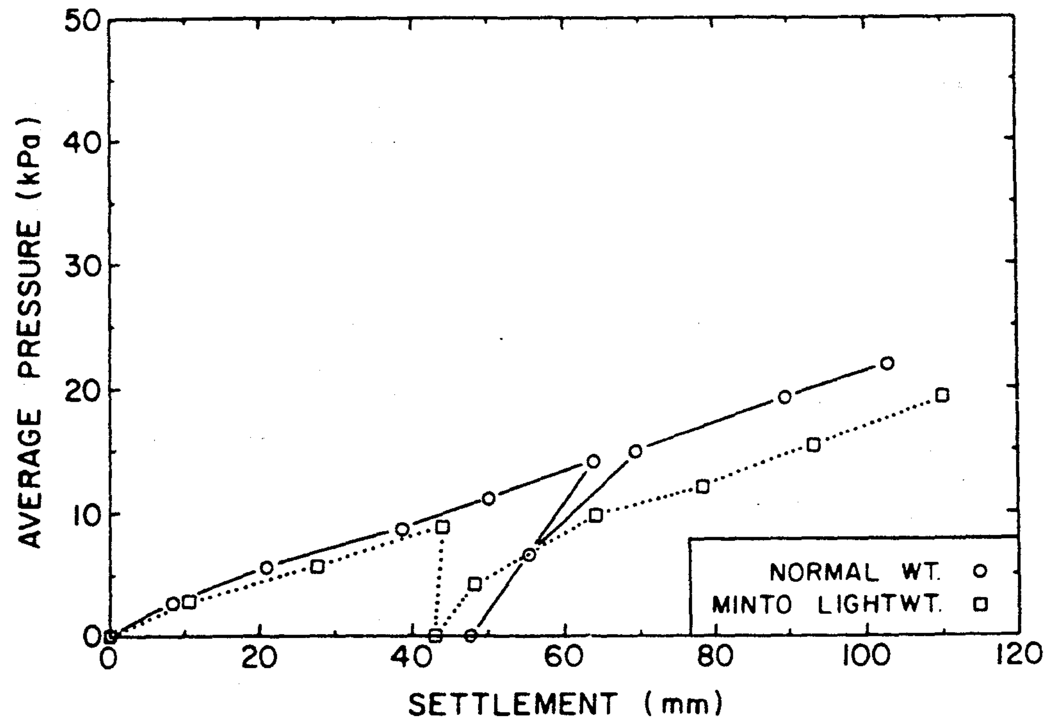

Fig. 5. Roadbed response for large scale model test.

normal weight aggregate by lightweight aggregate would affect the overall road stiffness. The results of the model testing indicate that the roadbed stiffness is unaffected by the type of aggregate used. These results also confirm earlier findings that the reinforcing role of a geotextile is insignificant during the initial stages of load application. Thus, the rut depth for given loading conditions appear to be mainly dependent on the thickness of the aggregate layer.

ACKNOWLEDGMENTS

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