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GEOTECHNICAL PROPERTIES OF SAND/LIGHTWEIGHT AGGREGATE MIXTURES

by

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B. Eng., Civil Engineering, Ryerson Polytechnic University

A thesis submitted

to the Faculty of Graduate Studies and Research

in partial fulfillment of the requirements for the degree of

Master of Engineering

in

Civil Engineering*

Carleton University

Ottawa, Canada

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submitted by

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in partial fulfillment of the requirements for the degree of

Master of Engineering

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A handwritten signature in black ink, appearing to read "G. E. Bauer", written over a horizontal line.

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ABSTRACT

Lightweight aggregates to be used as fill materials in embankments were investigated in triaxial and one-dimensional compression. Lightweight materials were considered as an alternative to reduce the unit weight of embankment fills. In this study expanded clay, high density polyethylene and shredded rubber tires were tested and added to sand in varying proportions. A common clean sand fill was chosen as the base soil. Specimens of natural and composite materials were tested in multi-stage triaxial compression. The stress, strain and volume change were monitored. Compression behaviour in one-dimensional compression was used for comparison with the axial strain behaviour in triaxial tests. Permeability was measured at various applied stresses during one-dimensional compression. It was concluded that the addition of sand to the compressible materials increased their strength and decreased their compressibility. This increase in strength was not directly related to the proportions of the constituents in a specimen. The change in compressibility was closely related to the proportions of the constituent materials in a specimen. The permeability decreased with the addition of sand to a mixed specimen, and decreased with increasing axial deformation. Theoretical models suggested by Shahabi, Das and Tarquin (1984) and Kenney et. al (1984) have not given an adequate description of the change in permeability for different specimens and vertical strains.

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To My Wife

Whose support over the course of this research has been instrumental to its
conclusion

1. Introduction

1.1 General

The design of any embankment must consider the native soil in two ways. The soil must have the strength to support the weight of the embankment safely, and it must not compress excessively, either immediately or over time.

If the embankment weight exceeds the bearing capacity of the soil in question, there are two common ways to overcome the problem. The overburden weight can be transferred to a soil stratum that is able to bear the load. This may result in relocation of the embankment, or improving the properties of the underlying soil, or use of a deep or piled foundation, transferring a portion of the weight to a stronger soil below. Alternatively, the weight of the embankment can be reduced in order not to exceed the bearing capacity of the soil in question.

The second requirement, that the soil must not undergo excessive settlement during or after completion of construction, is a judgement based on the definition of “excessive.” Most jurisdictions have now codified this judgement, specifying allowable total and differential settlements. Settlement due to consolidation of soils under new loading conditions is to be expected. When a soil does consolidate under a new load, the total settlement occurring is directly related to the load applied. Problems also arise when the settlement continues over long periods of time or has different magnitudes at different points under the embankment.

The options for avoiding these problems are to ensure that most of the settlement has occurred by the time the embankment is complete, or to ensure that the embankment weight is low enough not to cause excessive settlements.

In practice, rates of consolidation have been increased by adding a surcharge weight over and above the embankment weight for a period of time to bring about most of the total settlements before completion of the construction. Level of consolidation is determined by comparing the void ratio of the soil at the time in question to the void ratio of the soil in the far future. Adding extra weight speeds up the dissipation of pore water, causing the void ratio to decrease more quickly. The void ratio at the end of the construction will therefore be closer to the final void ratio for the embankment weight, after the surcharge is removed.

Increasing the permeability or the number of drainage paths of the soil being consolidated also increases the rate of consolidation before construction is completed. Rates of consolidation are directly related to the soil's ability to dissipate excess pore water pressure, allowing the particles to consolidate into a more compact arrangement. When drainage wells and wicks are introduced, the rate at which water can be expelled is increased, allowing the soil to reach its final level of consolidation more rapidly.

Embankment weight can be reduced, or pile foundations can transfer this weight to a stronger soil stratum below the surface. Soil strength is a function of the increase in stress relative to the existing stresses in the soil. With increasing depth, this stress increase is proportionally much lower than the same stress increase at the surface. As well, normally consolidated soils have higher shear strength at depth, due to higher confining

stresses. Structural and granular piles can be used to carry the load to below a weak soil, and thereby transfer the load to a stratum that is able to support the applied stress safely.

The construction of embankments commonly uses earth materials found near the construction site. The term “fill” relates to soils being brought in from other locations to be used in place of or in addition to the local soil. These soils used as embankment materials have unit weights that might induce settlement in weak or soft soils.

Embankments do not lend themselves easily to load transfer through the use of deep foundations.

Often embankments cannot be relocated to areas where the existing soil can support this surcharge load, as there are no suitable locations that can fulfill the design requirements. For example, a roadway alignment often cannot be changed to avoid a weak natural soil.

Load surcharging requires some excess material. This method is therefore limited to places where there is sufficient material to properly surcharge the embankment in question, and where the excess material can be used or disposed of after the underlying soil has sufficiently consolidated.

In the past two decades, there has been some progress in using materials that reduce the weight of embankments, as discussed in the following sections.

1.2 Use of Lightweight Fills

Lightweight fills are most commonly used over soft, compressible soils. These soils are generally found near large or slow moving bodies of water. Embankment designs in North America, the United Kingdom and Scandinavia have successfully used many

lightweight materials to avoid problems of overstressing such compressible soils. Expanded polystyrene (EPS) blocks have proven to be successful as lightweight fills in Europe, and the design of embankments using EPS has been codified in Britain and Scandinavia (Thompsett, 1995; Frydenlund, 1987).

In North America, the use of EPS as a lightweight fill has had moderate success, but the search for alternative and less expensive materials has led to the use of post-consumer waste plastics and used automobile tires. This thesis is aimed at evaluating some of these alternatives.

1.3 Problem Definition

Lightweight materials used in fills are traditionally manufactured specifically for use as embankment fills. This use of manufactured materials and manufacturing processes add significantly to the cost of embankment material. It therefore becomes important to evaluate other, less costly alternatives, such as waste materials.

In North America, the use of expanded clay and shredded tires in lightweight embankments has been studied over the last few decades, as discussed in the following chapter. Research into use for post-consumer waste has risen with the recent drive to reduce the amount of municipal solid waste that ends up in landfills. Materials suitable for lightweight embankment design are currently being studied.

Where the selected materials do not exhibit strength characteristics on their own as required in the design, they are mixed with various proportions of sand to increase the shear strength and reduce compressibility of the embankment material with a corresponding increase in unit weight.

1.4 Objectives of Research

Post-consumer wastes such as shredded HDPE and shredded automobile tires could prove useful in the construction of lightweight embankments but their mechanical behaviour must be determined. The objective of this thesis was to determine the shear strength, compressibility, stress-strain behaviour and permeability for the three lightweight materials, namely HDPE, rubber tire chips and expanded clay, in natural state and mixed with sand. The goal was to reduce the unit weight of a typical embankment fill while maintaining sufficient shear strength and sufficiently low compressibility.

1.5 Organization of Thesis

The following chapters will each examine an aspect of this study of materials for use in lightweight fills. Chapter 2 will review the current state of research into materials that have been used or are being considered for this application. Chapter 3 will discuss the laboratory tests that were carried out to investigate these properties. Chapter 4 presents the data acquired in these tests. Chapter 5 examines and analyses the data and discusses its implications and shortfalls. Finally, Chapter 6 presents a summary and conclusion regarding the investigated materials, veracity of results, and suitability of using the considered materials for lightweight embankment fills.

2. Literature Review

2.1 General

The use of lightweight materials in embankments and backfills is common for construction on problematic soils. Any foundation design must consider two characteristics of the underlying soil:

- a) the bearing capacity of the underlying soil, and
- b) the tolerable settlement of the embankment.

Settlement will occur when structural loads are applied to deposits of fine-grained soils or compressible soils at the areas to be developed. In some cases, the fine-grained soils may not exhibit much susceptibility to settlement, if they have in their history been subjected to loads, for example by glaciation, at or near the design loads in question.

Frydenlund (1987) listed six possible approaches towards addressing design problems in weak or compressible soils. His study was concerned with methods of reduction of settlement. The six methods are outlined in the following paragraphs.

2.1.1 Moving Structure to Area with Suitable Soil

The most direct solution to potential problems arising from a weak or compressible soil is to avoid loading that soil by relocating the structure. In the case of land development for buildings, the ownership of the land comes into question, and the developer may or may not have this option. For roadways, land is usually owned by the developer in question or it could be expropriated. Roadways are planned for traffic safety and continuity, though, so adjusting road alignment might not be a viable alternative.

Certainly in areas of large deposits of soft soils there may be little alternative for the alignment. In the case of small areas of unsuitable soil, then, it may be possible to avoid the affected area altogether, or adjust the position of the load to minimize the underlying soil's impact on settlement.

2.1.2 Soil Replacement

The second possible solution is to remove the unsuitable soil and replace it with soil having adequate strength and compression characteristics. Most roadway construction projects are already in need of good quality soil, so the process of bringing in suitable soil is only expanded. Its viability depends on the availability of suitable fill in amounts sufficient to replace the soil which must be removed from the construction site. This might be a time-consuming and expensive solution, and is rarely done to the exclusion of other methods. It is usually considered in tandem with other methods of site improvement.

2.1.3 Soil Improvement

For soils that have good drained shear strength but inadequate undrained strength, an option is to improve the soil by improving drainage of the site. Increased drainage increases the rate of consolidation, allowing the soil to approach maximum compression and bearing capacity at an earlier time than without improved drainage. The rate of drainage can be increased by adding wick and sand drains. Wick drains provide a path for water to travel out of the affected soil. Sand drains act as wells to shorten the flow path of water, and thereby accelerating the rate of water flow and thus the soil consolidation process.

As well, soils can be augmented by addition of other elements. Some clays have been stabilized by mixing in cementitious compounds. Others have been strengthened by adding geogrids, discrete fibres or geotextiles. These manufactured materials will increase the bearing capacity and shear strength of a soil by intersecting the shear plane and providing tensile strength to the parent soil.

2.1.4 Load Transfer

A weak soil overlaying a stronger stratum can be bypassed when designing a foundation by extending piles through the weaker soil and founding them in the competent deposit. The mechanism of transfer is shown in Figure 2-1. Piles are similar to columns in structural design, with two added concerns: downdrag and shear. Downdrag is the downwards-acting weight of the surrounding soil transferred to them by friction. Lateral motions of the soil must also be considered, as the shear stresses that can be induced in these conditions are not easily redistributed throughout the pile itself.

Piling can be an expensive solution, but there are many situations where there is no viable alternative.

2.1.5 Counter Weights and Berms

For embankments constructed over soft soils, quite often material is placed at the toe of the slopes for some distance. These counterweights are termed “berms” which hold the soil in place, minimizing lateral movement of the embankment. This method also ensures that the embankment load does not result in a “circular failure” of the underlying soil.

A circular failure will occur when the undrained or drained shear strength of the soil is insufficient to support the imposed surface load. This mode of failure is shown in Figure 2-2. Soil immediately below the load moves downwards and sideways in rotational shear. This tendency of a rotational shear failure can be minimised by placing berms adjacent to the toe of the slope.

2.1.6 Load Reduction

The most direct solution to minimize overloading a subsoil is to reduce the imposed load itself. In some cases this can be accomplished by reducing the size of the superstructure, whether it be a smaller building, fewer buildings, a lower embankment, or changing vertical alignment of the roadway. In the case of embankments, when these geometric solutions are not feasible, the soil used in the embankment construction can then be replaced by materials that have lower unit weights and densities. This thesis is directed at studying some potential lightweight materials in replacing common sand fill generally recommended for embankments. The materials considered are some manufactured materials, such as expanded polystyrene and expanded clay, and some post-consumer materials, such as tire chips and high density polyethylene as discussed in the following sections.

2.2 Expanded Polystyrene (EPS)

Expanded polystyrene has been used extensively in block form in Alaska, the United Kingdom, and Scandinavia. The benefits of using EPS are not limited to the load reduction only in embankments. Control of frost penetration and ground thawing beneath

roadways is also achieved when placing EPS sheets in predominantly horizontal layers. This type of application is of particular interest in permafrost areas, where road embankments can substantially change the freezing and thawing characteristics of the underlying ground.

Thompsett et al. (1995) presented the history of the use of EPS in road embankments in Europe, including a description of acceptable use of EPS blocks as specified in design requirements in the United Kingdom.

According to Thompsett et al., the first use of EPS in a fill occurred near Oslo in 1972. Prior to that, the embankment's rate of settlement had increased with each year, and at the time of reconstruction with EPS fill the rate of settlement was between 200 and 300mm/year. The reconstructed roadway had an initial rate of settlement of 80mm/yr for the first year after reconstruction, with no significant additional settlement in the following twelve years. This success led to extensive testing and use of EPS as a lightweight fill. The application of EPS blocks in embankments has now become standard practice in Norway and Sweden.

An additional benefit is the material's very low Poisson's ratio, simplifying the design of retaining walls or abutments, since minimal lateral stresses are transmitted when these blocks are placed adjacent to rigid walls.

Limitations of this method include the necessity of using labour to hand-place and cut to size the EPS blocks as shown in Figure 2-3. Since the material must be stockpiled on site, care must also be taken to protect it from excessive ultraviolet light, solvents or heat. EPS blocks cannot be used as fill below the water level in an embankment since its

density is significantly lower than that of water. Buoyant forces could cause heaving problems in the embankment.

Compressive stresses for typical EPS of varying densities range from 50 to 220 kPa. Most peak stresses were developed within 5% vertical strain.

Esch (1990) summarized the service history of an EPS-insulated road in Alaska over twenty years, discussing the beneficial effects of using EPS for insulation. Insulation under frost-susceptible soils reduces the penetration of frost, and reduces the resultant frost heave. It also reduces ground thawing in permafrost regions, as the summer heat is prevented from warming the soil underlying an exposed roadway. The EPS boards used in this application were only 51mm thick, and therefore had negligible impact on the weight reduction of the embankments.

2.3 Expanded Clay

Expanded shale and clay is a lightweight aggregate that has been used for many decades to produce lightweight structural concrete. It is made by heating shale or clay in a rotary kiln under controlled conditions at approximately 1150°C. The clay skeleton expands as the pore water evaporates, and the pores harden into airtight cells. The resulting particles are spherical to subangular in shape, durable and chemically inert. It can be used to replace regular granular material, and its only limitation is that it cannot withstand compressive stress to the same degree as regular granular material. Nevertheless, it has been used in construction where materials with lightweight properties were required (Baker, 1996; Stoll and Holm, 1985).

Stoll and Holm (1985) tested expanded shale aggregate from six different locations around the United States in triaxial compression and found the materials to have internal angles of friction between 44.5° and 48° in compact condition, with internal angles of friction varying to from 39.5° to 42° for loose material. The densities ranged from 700 to 908 kg/m^3 for loose samples, and 828.5 to 1042 kg/m^3 for dense samples. The average minimum and maximum densities were 793 and 899 kg/m^3 . They also found that the stress-strain curve for a consolidation test changed slope at about an axial stress of 100 kPa, corresponding to a degradation of the material at that stress.

Valsangkar and Holm (1990) confirmed these results for internal angles of shear in direct shear tests. Materials tested were all uniformly graded.

Expanded shale is currently mandated by the City of Calgary as standard insulating backfill material in trenches for city water lines based on the recommendations by the National Research Council of Canada (Baker, 1996).

2.4 Shredded Rubber Tires or Tire Chips

The US EPA (1991) estimates that there are over two billion old tires stockpiled across the United States, with an additional 189 million tires added annually. Tires have a significant impact on the amount of land used for waste disposal, and provide breeding grounds for pathogens. Tire dumps are also significant fire hazards, as witnessed in Hagersville, Ontario in 1990. Canadian data suggests that the amount of tires disposed in Canada are proportional to the population of the two countries (MOEE, 1993). Methods of disposal and reuse of tires are in need of alternatives to ensure that the amount disposed is reduced or even eliminated.

Clark et al. (1990) surveyed the recycling and disposal of whole and shredded waste tires. Shredded tires provided a material that was better suited to backfill, as it eliminates the buoyancy problems of EPS, and also reduced the bulk volume of individual tires by up to 75%. Many states have banned whole tires from landfills. The third option exercised in the US is creating managed landfills composed entirely of tire chips, eliminating the danger of creating breeding grounds and controlling the combustion hazard. Clark et al. also found recycled tires were used in lightweight fill, asphalt rubber and incineration and electricity generation.

Eldin and Piekarski (1993) analyzed the methods of disposal, legislation, hazards, and alternative uses for scrap tires in Wisconsin. Hazards found included the potential for fire and subsequent leaching of oils, and the potential for whole tires to serve as breeding grounds for disease-carrying mosquitoes. Wisconsin legislation disallows disposal of whole tires; other states have restrictions on disposal, storage and hauling of scrap tires. The only US federal legislation which promotes use of scrap tires (through preferential funding) is in the use of rubber-modified asphalt cement.

Hughes (1993) presented an analysis of possible uses of scrap tires. Energy can be extracted from the raw materials and could be used as fuel in Portland cement plants and other cogeneration facilities. Asphalt can be augmented with rubber from tires, with the potential to reduce wear and increase the service life of the road, at roughly break-even costs. Shredded tires as a substitute for granular soil can be used for drainage layers and embankments that are not saturated, especially in frost-susceptible areas, with little effect on ground water contamination. Case studies are mentioned in this regard. Whole tires can be used, filled with soil, to stabilize slopes or build retaining walls, and have been used

mainly in California. Examples include the Oregon Department of Energy Quality Administrative Rules regarding Solid Waste Management of tires and Lightweight Rubber Fill Specifications from Oregon, adapted from Minnesota's Regulations.

Edil and Bosscher (1992) found that waste tires have no propensity to leach harmful contaminants into the groundwater. Tests were conducted to detect base metals such as barium, cadmium, chromium, lead and mercury. Results showed that none exceeded the specified limit, and most couldn't even be detected. Other tests indicated that the metals did leach in minute quantities, but it was suggested that the methodology might produce higher concentrations than would be found in typical anaerobic field conditions. Further, the list of base-neutral organics for which Edil and Bosscher tested could not be detected.

In this study, Edil and Bosscher developed design criteria for the use of shredded tires in highway fills in Wisconsin. They concluded that, used under a thick cap of surface course, tire chips perform in a manner similar to regular gravel roads. Tire chips tend to compact only due to overburden pressure, and behave as an elastic material after initial construction.

Edil and Bosscher (1994) also tested tire chips mixed with soils to determine whether this would produce acceptable fills. Sands used were uniformly graded, with a maximum density of between 19.7 and 21.2 kN/m³. It was found that even with minimal addition of tires, the shear angle was greater than unreinforced sand. The tests were carried out in direct shear, with tire chips manually oriented to cross the shear plane, as it was thought that without care the chips would line up with the failure plane, not giving a true reflection of a randomly oriented mixture's shear strength. Hydraulic conductivity

tests were also carried out for varying hydraulic gradients and overburden stresses. It was found that with mixtures that had large voids, the flow tended to be turbulent, and had a very high conductivity. These phenomena occurred with high concentrations (>75% by weight) of tire chips.

Humphrey et al. (1993) tested three types of chipped tires for mechanical properties, revealing compacted densities of the order of 5.88 to 6.37 kN/m³. Shear strength parameters revealed cohesion was dependent on the exposure of steel fibres in the chip edges and internal angles of friction, as measured in direct shear, in the range of 19° to 26°. Compression tests revealed a constrained modulus of 1 to 5 MPa, compared to granular soils of 10 to 170 MPa.

Humphrey and Eaton (1993) built and monitored a test section of a roadway that used tire chips as insulating fill, and found that frost penetration was reduced by 22% to 28% for a 152 mm thick layer of tire chips overlaid by a 305 mm layer of surface course gravel. Future data should shed some light on the resilience of such a roadway. Resilience is a measure of the durability of a compressible material under field loading conditions.

The major environmental concern when using tire chips in embankments seems to be that for high embankments the tires have shown to partially combust, releasing oils and noxious gases found in the tires. Each tire has approximately 3.5 litres of oil, so the amount of oil released in a typical embankment is a significant threat to the local environment. The two instances thus far reported were along the Interstate 70 in Glenwood Canyon, Colorado in October 1995 and in Garfield County, Washington State in April 1996 (Associated Press, April 3, 1996). In Colorado the embankment was twenty

metres high, and in Washington State the embankment was fifteen metres. The smouldering of the chips was halted in the Colorado case when the upper part of the fill was removed. The reasons why the reaction did not reoccur are not yet understood.

2.5 HDPE and other wastes

Over the last few decades there has been an increasing awareness of the need to reduce solid waste that is generated by residential communities and businesses. Recycling programs, reduction of waste at source, and the reuse of products have all become important to the reduction of the amount of waste that ends up in landfills.

Waste that can be diverted from the waste stream but cannot be recycled or reused in other consumer processes must still find a use or will have to be returned to the waste stream. Research has been carried out on finding other end uses for these materials, as documented in the study of use of tires above. Municipal solid waste (MSW) contains other materials that are not presently recycled in a cost-effective way. Plastics exhibit the ability to be separated out at source, especially in residential waste. Of the total MSW, 60% is industrial, commercial, or institutional waste. The remaining 40% of MSW is residentially generated.

According to the Ontario Ministry of Environment and Energy's 1993 survey (MOEE, 1993), plastic makes up 7% of the landfill composition by weight, but 14% by volume. Plastics made up 6.4% of the composition of residential waste by weight. This was further broken down into 5.4% HDPE and 1.0% other plastics (including low density polyethylene and polystyrene). Of this 5.4%, 0.1% was diverted from the landfill, meaning that 5.3% of the residential MSW was recyclable HDPE.

HDPE chips have been used as thermal insulation for municipal service lines under roads (Baker, 1996; Crawford et al. 1995). The results of their study on the thermal properties has led to the conclusion that HDPE chips would serve as adequate insulating fill in trenches, and test trenches are currently being monitored. To date there has been no damage to the pavement surface due to settlement. This HDPE exhibited a minimum unit weight of 3.28 kN/m^3 and a maximum unit weight of 4.96 kN/m^3 , and a compressive modulus of 5.83 MPa . Leachate tests were only able to detect trace amounts of boron and cyanide, both well below Ontario Ministry of Environment and Energy acceptable limits.

Waste wood fibres have also been used as a lightweight fill in various applications. One such use was in Idaho (Hardcastle and Howard, 1991), where an airport runway and aprons were excavated and reconstructed. Previously, the airstrip was settling considerably due to consolidation of the underlying soil. Up to 1 metre of rock and 2.4 metres of organic soil were removed and replaced by up to 2.4 metres of wood fibre.

The wood fibre was analyzed in the laboratory, and exhibited a compression coefficient of $2.32 \times 10^{-3} \text{ kPa}^{-1}$ and a internal angle of friction of 30° at 20% strain. Settlements were monitored, and found to be larger than predicted, but clearly reducing over time, suggesting that the reconstruction was a success.

Figure 2-1. Transfer of applied load from structure to lower stratum of soil by use of piles

Figure 2-2. Circular failure of embankment and underlying soil

Figure 2-3. Manual placement of expanded polystyrene blocks in a lightweight embankment, Flom, Norway, 1972 (Freydenlund, 1987)

3. Experimental Program

3.1 General

The purpose of this study is to test the permeability, shear strength and compression of various lightweight materials in their own natural states and mixed with a common sand fill. The sand was added to increase shear strength and decrease compression of the lightweight materials. The lightweight materials considered were high-density polyethylene (HDPE) chips, expanded polystyrene (EPS), shredded tire chips, and a lightweight expanded clay aggregate (Granulite™). These constituent materials were to be added in various proportions to the sand to form composite or mixed samples and to observe any changes in shear strength and compressibility as the unit weight of the composite material changed. The sand was chosen as a conventional fill commonly used in embankments.

EPS beads were eliminated as a potential material for two reasons. The first was that the material is highly compressible, and therefore not suitable for embankments or base courses. The second was that the material had a high tendency to segregate when mixed with sand, making it unsuitable to form a uniform lightweight composite.

The permeability and compression of the constituent materials and the mixtures were tested in a 254 mm Rowe cell doubling as a permeameter and as an oedometer apparatus. The shear strength was determined for the various materials and mixtures in a large-scale (225 mm diameter by 500 mm in height) triaxial apparatus. The purpose of this study, as stated before, was to determine whether these lightweight materials could

exhibit sufficient compressive and bearing characteristics to be used as fill and/or base course aggregates.

3.2 Soil and Lightweight Aggregates

The materials being tested were sand, Granulite™, which is heat-expanded clay, rubber tire chips, and HDPE chips. Each material was tested in its own natural state and mixed with sand in various proportions.

3.2.1 Sand

A clean sand conforming to the Ontario Ministry of Transport (MTO) specification Granular “B” Type 1 was chosen as the base material to which the lightweight materials would be added. This type of sand was considered to be representative of granular fill material commonly used in embankments. Classification data for this sand is presented in Table 3-1.

3.2.2 Expanded Clay (Granulite)

Granulite is a lightweight expanded clay aggregate, manufactured by Inland Concrete Ltd. in Calgary. The manufacturing process was briefly described in Section 3 of Chapter 2. Table 3-1 shows the basic physical properties of this material. Typical pellets are shown in Figure 3-1.

3.2.3 Rubber Tire Chips

The tires used in this investigation were cut from used automobile tires using a band saw. The chips were thus very rectilinear in shape, and had exposed metal wires in

the tire tread section. This exposed metal was sanded off in order to avoid puncturing the latex membranes during triaxial testing. The tire chips were approximately 15 mm wide and 30 mm in length, as shown in Figure 3-1. The size of the chips was governed by the diameter of the triaxial apparatus and the Rowe cell. Chips in practice would vary to a size of $\frac{1}{4}$ of a tire, with irregular shapes.

3.2.4 High Density Polyethylene (HDPE)

The HDPE chips were obtained from the National Research Council's Institute for Research in Construction (IRC). The same HDPE material was used by Baker et. al (1996) in a study to investigate the thermal properties of this aggregate for the thermal protection of buried municipal services.

The chips were oblong in shape, approximately 5 mm wide by 10 mm long, with a thickness of 1 to 2 mm.

3.2.5 Classification

3.2.5.1 Sieve Analysis

Each material was run through a stack of sieves with openings from 3 inches to a size 200 (75 μm) according to the test method defined by ASTM D 422-63. The various mixtures were also sieved. The grain size distributions are shown in Figures 3-2, 3-3 and 3-4.

3.2.5.2 Specific Gravity

The specific gravity of the particles of each material was determined. For sand the method specified by ASTM D 854-92, using a vacuum pump to remove the entrapped air

was used. For the other materials, a similar method was adapted to accommodate the larger particles. Specifically, a calibrated 1 litre flask was used instead of a pycnometer.

The container and a known volume of water, as marked on the container, was weighed at a specific temperature. A known mass of aggregate was placed in the container, and water added. The mixture was subsequently placed under a vacuum for a minimum of 30 minutes with gentle agitation of the aggregate-water mixture to encourage air removal. The container was then filled to the mark used previously, and weighed again. In this study, the water was kept at a uniform temperature of 27.0°C, eliminating the need to convert masses to volumes and vice versa.

HDPE had a specific gravity of less than unity; therefore the buoyant forces had to be overcome in order to determine the specific gravity. The material was anchored using a weight and a porous bag to submerge the particles. The procedure specified by ASTM D 854-92 was adapted to determine the specific gravity of the buoyant material.

The results of the specific gravity tests are presented in Table 3-1.

3.2.5.3 Density

Each material was tested for its minimum and maximum density using the dry method described in ASTM D 4253-93. The minimum density was measured by gently placing material into the Density Maximum Index (DMI) mold, and determining the mass that fit into the previously calibrated mold. The maximum density was determined on the same given mass of material by attaching the mold to a vibratory table, with a surcharge weight. The mold was vibrated at 60 Hz for 8 minutes. The volume of material was then

determined. This method was developed for cohesionless soils, and as such was applicable to the materials considered.

The results of the density tests are presented in Table 3-1.

3.2.5.3.1 Density Maximum Index Apparatus

Figures 3-5a and 3-5b show the DMI mold, vibratory table, surcharge weight and guiding sleeve, and dial gauge for measuring the change in height of the material after vibration.

3.3 Compression

The compression of a granular aggregate is directly related to particle rigidity and initial void ratio for a given material. The initial void ratio is known based on the volume of the soil and the mass of the soil portions, as well as the specific gravity of the particles. As the void ratio decreases, there will be a corresponding decrease in permeability of the material.

The most common method to test compressibility of a fine grained soil is in an oedometer (ASTM D2435-96). A typical oedometer test apparatus utilizes small sample sizes and dead loads to compress the material. Due to the large particles of the various aggregates considered in this study, the specimen size had to be large, necessitating large dead loads to give representative vertical loading stresses. In order to overcome these two requirements, a large diameter Rowe cell with pneumatic surcharge loading was used.

3.3.1 Rowe Cell

One-dimensional oedometer tests are usually carried out on saturated fine-grained soils. It is proposed that a large diameter consolidation apparatus is also suitable to determine the one-dimensional compression of free draining granular aggregates.

The Rowe cell is not dependent on the application of dead loads to transfer the compressive stress to the material, and thus can be made in a large range of sizes. The cell used this study had a diameter of 254 mm and the surcharge loads were applied by compressed air pressure on a diaphragm placed over the soil specimen.

3.4 Permeability

Some oedometer apparatuses allow for permeability measurements based on the falling head test, which is most applicable for fine-grained soils. It is widely reported (Das, 1990; Fredlund and Rahardjo, 1993), that granular materials with high permeability are better tested using a constant head apparatus. The Rowe cell has valves that can easily be connected to an existing constant head apparatus. The inflow valve located at the top of the cell, however, caused some concern that the loading diaphragm might interfere with the laminar flow of the water. For this reason, the cell was extended using a 150 mm collar, thereby changing the sample height from 75 mm to 225 mm as shown in Figure 3-6. The inflow valve now was positioned in the middle of the sample, allowing uniform water flow through the apparatus regardless of the amount of soil compression.

Water flowing through granular materials at high velocities will be turbulent, and relations between the flow rate of the water and the hydraulic gradient are non-linear. A

summary of equations describing turbulent flow are given by Hansen et. al (1995), of two general forms:

$$i = av^N \quad (3-1)$$

or

$$i = sv + tv^2 \quad (3-2)$$

where

v is the water velocity,

i is the hydraulic gradient, and

a , s , t and N are constants.

For laminar flow, these constants reduce to give Darcy's law,

$$v = ki \quad (3-3)$$

where

k is the coefficient of permeability.

In order to ensure that the flow rate measured through the granular soils was uniform and laminar, the head was varied and the flow was measured to ensure that the flow rate varied linearly with the applied hydraulic head.

A second concern with the Rowe cell was that the diaphragm might experience wall friction during compression. This is more a concern when determining the time rate of consolidation than with the total compression of a specimen. Granular materials compress rapidly, with little secondary compression occurring due to rearrangement of grains under sustained loads.

A Teflon liner was added to the cell wall, and a silicon grease was applied in trial runs to ensure that wall friction effects, if any, were minimized. It was found that the

addition of grease had a negligible effect on the free movement of the diaphragm, and was therefore omitted for final tests.

Uniform pressure to the diaphragm was supplied by compressed air, up to a maximum of 690 kPa. The pressure was controlled by a regulator, and read by a pressure gauge that had been calibrated against electronic pressure transducers. The vertical deformation was measured by a linear variable-differential transducer (LVDT) which had a travel of 25mm.

Each material was placed in the mold at 100% of DMI maximum density to a standard height of 225mm. The sample was saturated and connected to the constant head permeability apparatus. The diaphragm was inflated until it was fully expanded but not exerting any significant pressure upon the soil. The LVDT was set to zero. The time for a specific volume of water to pass through the system was measured. The pressure was then increased to 20 kPa. The LVDT reading and water flow were recorded. The pressure was increased by a factor of two and the measurements were repeated, to a maximum pressure of 640 kPa or until a total compression of the specimen of 85 mm or 38% vertical strain was reached.

3.5 Shear Strength

Shear strength of soils can be tested by different methods. Two of the most commonly used are the direct shear and the triaxial compression tests. The shear box test is very simple to set up, but complex in modeling the actual behaviour of the soil in shear. Triaxial testing of soil specimens requires a more sophisticated apparatus, but the analysis of the results is simpler since the principle stresses and strains are known directly.

Tire chips have been tested for shear strength in direct shear by previous researchers, but have not been investigated for shear behaviour under triaxial conditions. Mixtures of tire chips and sand were investigated by Edil and Bosscher (1993) in a shear box, but the specimens were compacted manually and the chips were placed specifically to cross the shear plane. High density polyethylene has not been tested for compression or strength properties to the knowledge of the author.

HDPE chips or flakes are basically two-dimensional in character. When placing this material into a direct shear box, it will become predominantly oriented horizontally. This coincides with the plane of shear in a direct shear box. In triaxial specimens, shear will not necessarily occur along a horizontal plane. Therefore, triaxial compression is preferred for determining the shear strength of this material.

3.5.1 Triaxial Cell

The triaxial cell used in this study was able to accommodate specimens having a diameter of 230 mm (9 inches), and a height of up to 508 mm (20 inches). Drainage of the sample could be achieved at both the bottom and top as in a standard triaxial apparatus. Confining pressure was applied by compressed air. The cell and accessories are shown in Figures 3-7 through 3-12.

Specimens were compacted on a vibrating table in a split mold with a surcharge weight in a similar fashion to the Density Maximum Index apparatus. Each specimen was compacted to 100% DMI maximum density. A vacuum was applied to the interior of the specimen as it was transferred to the compression machine. As the cell pressure was

applied with compressed air, the vacuum was disconnected. Figures 3-13 and 3-14 show the mold, surcharge and vibrating table.

3.5.2 Types of Tests

In this study, the triaxial tests were carried out using compressed air as confining fluid and the specimens were tested in dry condition. The tests were performed as consolidated drained tests. The interior of the specimen was open to the atmosphere. A pressure transducer monitored the air pressure within the specimen. No excess air pressure occurred during testing.

All materials tested were very permeable, allowing consolidation to occur rapidly.

Each specimen was tested in multiple stages. The test load data was carefully monitored to identify as nearly as possible the peak deviator stress at a given confining pressure. Once this peak stress was reached, the specimen was unloaded and reloaded to peak. The confining pressure was then increased, and the sample loaded to the next peak stress. In some cases, five confining pressure increments were employed.

Sand, three lightweight materials and five mixtures of sand added in various portions to lightweight materials were tested. A total of ten triaxial tests were carried out with the large apparatus. The results from the sand specimens were used as control data in the analysis and as comparison to the performance of the other aggregates and mixtures.

3.5.3 Strain Rate

The tests were carried out at the rate of 0.05 mm/second or 3 mm/minute. In all cases excess pore air pressure did not exceed 1 kPa. The unloading rate, after a given

stage, was 0.02 mm/second or 1.2 mm/minute. This rate was slow enough that the various parameters could be monitored by the data acquisition system.

3.5.4 Data Acquisition

The experimental data were collected by a PC connected to a testing rack data output, LVDT and pressure transducers. The Tinius 400 Kip testing rack applied load and monitored the vertical deformation of the sample. In addition, an external LVDT measured the movement of the loading piston of the triaxial cell. Three load cells serving as the legs of the sample base monitored the applied load and also gave an indication of specimen tilt. The volume change was measured by use of two Pyrex graduated cylinders, one fitted inside the other. The change in volume of the air pocket inside the smaller one was a direct measurement of the volume change. To ensure that the specimen pore air pressure was equal to the ambient pressure, the cylinder was moved until the phreatic surfaces inside both cylinders were identical.

	Specific Gravity	DMI Minimum (kg/m ³)	DMI Maximum (kg/m ³)
Sand	2.75	1387	1640
Granulite	1.44	670	768
Tire chips	1.27	505	546
HDPE	0.90	283	345

Table 3-1. Classification data for sand and lightweight materials.

Figure 3-1. Expanded clay pellets and rubber tire chips

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Figure 3-5a. Density Maximum Index apparatus - vibrating table
surcharge and guiding sleeve

Figure 3-5b. Density Maximum Index apparatus - mold, sample to

Figure 3-6. 254 mm diameter Rowe cell with extended sample head connected to constant head apparatus

Figure 3-7. Triaxial cell (Felio, 1980)

Figure 3-8. Plan view of triaxial base

Figure 3-9.

Figure 3-10.

Figure 3-11. Removable triaxial cell

Figure 3-12.

Figure 3-13. Guiding sleeve, split compaction mold, and surcharge specimen creation

Figure 3-14. Vibrating table for triaxial specimen creation

4. Experimental Results

4.1 General

This chapter presents the results of the Rowe cell oedometer and permeability tests, and triaxial tests for the sand, lightweight aggregates and composite specimens. The classification results of the grain size distribution of the soils, the properties of specific gravity and density were given in the previous chapter. The results from the compressibility tests are given as the plots of void ratio versus compressive stress and coefficient of permeability versus void ratio. The results of the triaxial tests are plotted as deviator stress versus strain and volume change versus axial strain.

4.2 One-Dimensional Compression Tests

One-dimensional oedometer compression tests are generally used to determine compression and consolidation properties of fine-grained soils. These tests are performed to determine a relation for the time rate of consolidation (dependent on the dissipation of pore water pressures) and a relation between void ratio of the soil and the vertical effective stress. Coarse-grained soils are free-draining and thus considered to consolidate rapidly without generating excess pore pressures. Oedometer compression tests were used in this study to determine the relationship between void ratio and vertical stress for sand and lightweight aggregate specimens.

4.2.1 Materials

The results of the oedometer tests for sand, Granulite, tire chips and HDPE are presented in Figure 4-1. All data are given on semi-logarithmic plots. All materials exhibited a fairly linear relation of void ratio and $\Delta \log (p)$, which indicates constant compression indices for those materials over the stress range tested. These relations are further discussed in chapter 5.

The linear behaviour of the compression curve for Granulite became slightly steeper as the vertical stress was increased beyond 160 kPa, resulting in an increase of the compression index. Audible breaking of particles occurred after stress increases above 160 kPa. This change in grain size distribution was presented in Figure 4-12 of the previous chapter. This is consistent with the crushing strength reported elsewhere (WPL Engineering, 1987). The compression indices, C_c , for these materials are presented in Table 5-1.

4.2.2 Mixtures of Sand with Granulite or Tire Chips

Figure 4-2 presents the variation of void ratio with vertical stress for mixtures of sand and Granulite, while Figure 4-3 presents the variation of void ratio with vertical stress of mixtures of sand and tire chips compared to the compression curves of each component material. Both mixtures were made by adding sand to the respective aggregate in place, and vibrating the mixture until all voids were filled with sand. This would yield a void ratio of the composite smaller than the void ratio of the lightweight material. All mixed specimens are expressed as ratios of weight; for example, the HDPE-sand specimen with a ratio of 1:4 has one part by weight of HDPE chips and four parts by

weight of sand. These ratios were chosen such that the maximum density of the composite specimen was halfway between the maximum densities of the lightweight aggregate and sand.

4.2.3 Mixtures of Sand with HDPE

Figure 4-4 shows the compression behaviour of composite specimens of sand and HDPE chips, compared to the corresponding compression curves of each component material. As with previous composite specimens, the mixtures of ratios 1:1.4 and 1:2.7 were made by adding sand to the mould containing HDPE chips and vibrating it to the specified density. The mixture with a ratio of 1:4 was made by adding the two materials in a container and premixing them before compaction. The 1:4 mixture had a lower void ratio than sand alone.

The ratios were chosen to reflect the range of densities between the maximum density of sand and the maximum density of HDPE. The following ratios were chosen:

1. HDPE with some sand in pores (1:1.4 mixture),
2. HDPE with pores filled with sand (1:2.7 mixture), and
3. Sand with HDPE interspersed (1:4 mixture).

Compression indices for the mixtures are given in Table 5-1.

4.3 Permeability

Coefficients of permeability at various void ratios were determined for all materials investigated in this study. As will be discussed in Chapter 5, the void ratios correlate to the compression of the materials under compressive stress. Figure 4-5 shows the change

in permeability with change in void ratio for the sand specimens. Figures 4-6 through 4-8 show the same relationship for Granulite, tire chips and HDPE. The coefficients are all plotted according to the relation given by Das (1990)

$$k \propto \frac{e}{1 + e^3} \quad (4-1)$$

where k is coefficient of permeability, expressed as a velocity, and e is void ratio, expressed as a dimensionless number.

This relation was calibrated such that the k value was taken to be equal to the measured k under a compressive stress of 20 kPa for comparison purposes. The difference between the function and the measured values is discussed in the following chapter.

Figures 4-9 through 4-11 show the change in permeability of the composite specimens with change in void ratio. All figures show the relationship between k and void ratio with the corresponding values for sand for comparison.

4.4 Triaxial Tests

Presented in this section are the graphical relations of the deviator stress and volume change as related to the vertical strain in consolidated-drained triaxial compression of the sand, aggregates and mixtures. For all specimens, except for the rubber tire chips, multi-stage triaxial testing was employed (Fredlund and Rahardjo, 1993). This method is used to maximize the data collected from a single specimen, which reduces the effect of specimen variability on the test results. Each stage is defined by its unique confining

pressure. Confining pressures ranged from 20 to 200 kPa and are indicated on the figures cited below.

4.4.1 Sand

The triaxial stress-strain curve for sand is presented in Figure 4-12a. The four stages of the test are related to the confining pressure. For the sand specimens, the confining pressures were 50, 100, 150, and 200 kPa. The curve exhibits a well-defined change in slope, indicating incipient failure in the first stage of testing. For each subsequent stage, the sample was returned to zero stress and loaded back to the stress of incipient failure, before the next confining pressure was applied. The specimen was consolidated under the new increment of confining stress before the axial stress was increased. The axial stress was increased until the next point of incipient failure was reached in the stress-strain relation. The change in volume with vertical strain for the four stages is shown in Figure 4-12b. Only sand and Granulite specimens exhibited a dilative behaviour. All other aggregates and composites exhibited compression only regardless of amount of vertical strain.

The initial 0.6% of vertical strain, with no significant deviator stress, can be attributed to a seating error of the sample. Analysis of the results reflects the change in strain.

4.4.2 Expanded Clay (Granulite)

Granulite was tested on its own and mixed with sand; the stress - strain curves are presented in Figures 4-13a and 4-14a respectively. The Granulite specimens were subjected to confining pressures of 50, 100, 150 and 200 kPa. The internal angle of

friction exhibited in the first test, assuming zero cohesion after membrane correction, would have required that the deviator stress be much greater than the stress at which material was found to crush in one-dimensional compression tests. The second load cycle did not reach the level of the initial incipient failure. Particle crushing was considered to be the problem as was borne out by a subsequent grain size analysis.

Granulite mixed with sand at a weight ratio of 1:0.65 was tested at confining pressures of 20, 30, 50, 75, and 100 kPa to ensure that the deviator stress did not exceed the crushing strength of the Granulite particles. The mixture showed greater shear strength overall, and the strength was developed at a smaller strain than with the natural Granulite specimen, albeit at a lower confining pressure.

Both types of specimens, Granulite and sand-Granulite composite, exhibited volumetric increase with vertical strain, a behaviour consistent with dense granular materials (Figures 4-13b, 4-14b).

4.4.3 Rubber Tire Chips

Rubber tire chips were subjected to triaxial compression on their own and mixed with sand. Figures 4-15a and 4-16a present the stress - strain curves.

Tire chips on their own did not exhibit any tendency to fail in shear. There was no pronounced decrease of stress over the course of the test. The rate of increase of axial stress did not change substantially even at 30% of vertical strain. The tests on tire chips were conducted at 30 and 50 kPa confining pressure. The confining pressures were chosen to minimize the amount of compression. A definite shear failure did not occur. The test at 50 kPa, Figure 4-16a, shows a sudden decrease of the confining pressure. At

that point in the test the air line broke and had to be replaced. The stress versus strain can be seen to be quite linear in particular over the first 10% of vertical strain.

This same near-linearity was exhibited in the test carried out at 30 kPa confining pressure. Since the tire chips did not exhibit any deviator stress decrease over the course of the test at a confining pressure of 30 kPa, it was decided not to attempt further triaxial tests on them. Volume change with vertical strain is presented in Figure 4-15b. It can be seen that the decrease in volume was quite substantial within the first 10% of strain, i.e. 10% at $\sigma_3 = 30$ kPa and 18% at $\sigma_3 = 50$ kPa.

The composite specimen of tire chips and sand with a weight ratio of 1:0.9 was tested at confining pressures of 20, 28 and 40 kPa. These low confining pressures were chosen to limit the magnitude of the deviator stress since the natural tire chip specimens under went a large amount of compression without shear failure. The test was halted at 30% of vertical strain.

Figure 4-16a shows that tire-sand mixtures did fail in shear as indicated by the peaks of deviator stresses for each stage. Incipient failure is reached for each confining pressure, and stage reloading returned to deviator stress of the previous level. The strain required to reach the deviator stress in each stage was quite large, which was mainly due to the compressibility of the tire chips. There was significant recovery of sample height in each unloading cycle. This compressibility during triaxial testing will be discussed in chapter 5 as it relates to rebound and the reloading modulus. Volume change data is presented in Figure 4-16b.

4.4.4 HDPE

Stress - strain curves for the triaxial tests of HDPE and HDPE chips mixed with sand are presented in Figures 4-17a through 4-20a. The tests were carried out at confining pressures of 50, 100, 150 and 200 kPa, except for the test of the 1:1.4 mixture, where only the first two confining pressures were used.

The test of natural HDPE was successfully unloaded and reloaded at the end of the first stage, but the reloading cycle in the subsequent stages did not always attain the previous peak stress, suggesting that some slippage of the material had occurred. The unevenness of the stress-strain curve (Figure 4-17a) also indicates continuous local slippage occurring between the individual chips. Another potential explanation is that the measurement apparatus was at its lower limit, and the unevenness is noise.

The composite specimen with the weight ratio 1:1.4 was tested once. After two stages were completed, the specimen had undergone 30% of axial strain. Similar to the behaviour of natural tire chips, it was apparent that the governing mode of failure of the composite specimen would be by excessive deformation. Therefore, it was decided not to test the specimen further for shear strength.

All stress-strain curves indicated incipient failure, and were reloaded to the previous deviator stress prior to the start of the subsequent stage. All materials experienced a decrease in volume with vertical strain, as shown in Figures 4-17b through 4-20b.

Each sample was disassembled after testing to examine whether any segregation had occurred during compaction or testing. No significant migration of sand particles was seen for any of the mixed specimens.

Figure 4-1. Void ratio vs. vertical effective stress for natural materials

Figure 4-2. Void ratio vs. vertical effective stress for mixtures of Granulite and sand

Figure 4-3. Void ratio vs. vertical effective stress for mixtures of tire chips and sand

Figure 4-4. Void ratio vs. vertical effective stress for mixtures of HDPE and sand

Figure 4-5. Coefficient of permeability vs. void ratio for sand

Figure 4-6. Coefficient of permeability vs. void ratio for Granulite

Figure 4-7. Coefficient of permeability vs. void ratio for tire chips

Figure 4-8. Coefficient of permeability vs. void ratio for HDPE

Figure 4-9. Coefficient of permeability vs. void ratio for mixtures of sand and Granulite

Figure 4-10. Coefficient of permeability vs. void ratio for mixtures of sand and tire chips

Figure 4-11. Coefficient of permeability vs. void ratio for mixtures of sand and HDPE

Figure 4-12a. Deviator stress vs. vertical strain for triaxial test of sand

Figure 4-12b. Volume change vs. vertical strain for triaxial test of sand

Figure 4-13a. Deviator stress vs. vertical strain for triaxial test of Granulite

Figure 4-13b. Volume change vs. vertical strain for triaxial test of Granulite

Figure 4-14a. Deviator stress vs. vertical strain for triaxial test of mixture of Granulite and sand (1:0.65)

Figure 4-14b. Volume change vs. vertical strain for triaxial test of mixture of Granulite and sand (1:0.65)

Figure 4-15a. Deviator stress vs. vertical strain for triaxial tests of tire chips

Figure 4-15b. Volume change vs. vertical strain for triaxial tests of tire chips

Figure 4-16a. Deviator stress vs. vertical strain for triaxial test of mixture of tire chips and sand (1:0.9)

Figure 4-16b. Volume change vs. vertical strain for triaxial test of mixture of tire chips and sand (1:0.9)

Figure 4-17a. Deviator stress vs. vertical strain for triaxial test of HDPE chips

Figure 4-17b. Volume change vs. vertical strain for triaxial test of HDPE chips

Figure 4-18a. Deviator stress vs. vertical strain for triaxial test of mixture of HDPE and sand (1:1.4)

Figure 4-18b. Volume change vs. vertical strain for triaxial test of mixture of HDPE and sand (1:1.4)

Figure 4-19a. Deviator stress vs. vertical strain for triaxial test of mixture of HDPE and sand (1:2.7)

Figure 4-19b. Volume change vs. vertical strain for triaxial test of mixture of HDPE and sand (1:2.7)

Figure 4-20a. Deviator stress vs. vertical strain for triaxial test of mixture of HDPE and sand (1:4)

Figure 4-20b. Volume change vs. vertical strain for triaxial test of mixture of HDPE and sand (1:4)

5. Analysis and Discussion of Results

5.1 General

This chapter will present the comparison and discussion of results obtained in the experimental program. Analysis of compressibility, permeability, and Mohr-Coulomb failure criteria are presented and discussed relative to the constituent materials. Comparisons of the secant modulus, reload modulus, initial tangent modulus and the constrained modulus are also presented.

The analyses are made for the four component materials, namely sand, Granulite, tire chips and HDPE, and the mixtures of sand with the other three components in varied ratios.

A summary of the results of these tests is presented in Tables 5-1 and 5-2.

The vertical strains measured in these tests should in no way be taken to represent the vertical strains found in the field. Construction methods and deformation under self-weight of the embankment would eliminate the initial strains found during these tests. The deformations found in the field would more closely correspond to values found during reloading in the triaxial tests, as discussed in Section 2.2 of this chapter.

5.2 Compression

5.2.1 One-Dimensional Compression

Confined compression is a common state for soil in nature. Soil strain behaviour can be modelled by the one-dimensional compression test. This test has been used

extensively for testing of fine-grained materials, and has been applied to sands and granular materials as well (Lambe and Whitman, 1979). These soils are tested in compression under effective stress conditions. For fine-grained materials, time is required for the dissipation of pore water pressures to occur, in order that the applied stress can be carried exclusively by the soil skeleton. Free-draining materials, like those used in this study, allow for rapid dissipation of pore pressure due to their high permeability.

A common way of presenting results from a one-dimensional compression test is to plot the change in void ratio against the logarithm of the applied load. This will generally result in a linear relationship for normally consolidated soils. The compression index, C_c , for any given soil is defined as the slope of this linear plot. This is expressed as

$$C_c = -\frac{e_2 - e_1}{\log p_2 - \log p_1} \quad (5-1)$$

where e is the void ratio, i.e. the ratio of volume of voids to volume of solids in a specimen, and p is the effective stress on the soil. The values of the compression index are presented in Table 5-1 for the materials tested. Figure 5-1 presents graphically the change of C_c with maximum density index for the mixed constituent specimens.

From the results of a one-dimensional compression test one can also obtain a constrained modulus (E_c). The modulus will increase with increasing strain for most granular soils. This modulus is defined in this study by the slope of the secant between a point on the stress-strain curve and the origin. It is given as

$$E_c = \frac{p'}{\Delta h / h_0} \quad (5-2)$$

where p' is the applied effective stress, Δh is the change in specimen height, and h_0 is the initial specimen height. Ranges of the constrained modulus for the tested materials are

presented in Table 5-1. The relationship between E_c and vertical strain are presented in Figures 5-3 through 5-11.

5.2.2 Triaxial Compression

The behaviour of a soil in triaxial compression can be described by three separate moduli. The initial tangent modulus (E_i) describes the relationship between axial stress and strain at the beginning of the test, and is graphically represented as the tangent to the stress-strain curve passing through the origin. E_i is defined as

$$E_i = \left[\frac{d\sigma_d}{d\varepsilon} \right]_{\varepsilon=0} \quad (5-3)$$

where σ_d is the deviator stress, $\sigma_1 - \sigma_3$, and ε is the axial strain, dh/h_0 ; h is the sample height, and h_0 is the initial sample height.

At higher stress levels, the stress is not linearly related to the strain for a soil, and a secant modulus (E_s) may be used to describe the relation between the stress and strain. Graphically this modulus is represented by the secant between the point of known stress and strain and the origin. It is given as

$$E_s = \frac{\sigma_d}{\varepsilon} \quad (5-4)$$

for a given ε . This secant modulus varies with strain. The range of values is presented in Table 5-1, and the relationship between E_s and vertical strain are presented in Figures 5-3 through 5-11.

For cyclic triaxial tests, the reload modulus (E_r) may be used to describe the stress-strain relationship for reloading a soil during cyclic testing. Graphically it is represented by the slope of the secant through the point of zero stress and strain at the

beginning of the reloading phase and through the point at which the stress-strain curves for the reloading and unloading phases intersect. It is defined as

$$E_r = \frac{\sigma_{d(intersect)}}{\epsilon_{intersect} - \epsilon_{0(reload)}} \quad (5-5)$$

where $\sigma_{d(intersect)}$ is the deviator stress at which the two stress-strain curves intersect, $\epsilon_{intersect}$ is the strain at which the two curves intersect, and $\epsilon_{0(reload)}$ is the initial strain for the reloading phase. The values are given in Table 5-1. The values of E_r did not significantly change for a given material during subsequent reloading phases.

As stated above, E_s and E_c are not necessarily constant but are dependent on the strain level. It then becomes important to understand the way the modulus changes.

The secant modulus for the mixed constituent specimens was computed over the loading phase of the first stage of each triaxial test and compared to the secant constrained modulus for constituent materials. The results are presented in Figures 5-12 through 5-17. The secant modulus for the tire chips-sand and Granulite-sand mixtures were determined at a confining pressure of 20 kPa. The secant modulus for the remaining materials was determined at a confining pressure of 50 kPa.

It is a common phenomenon that as the soil specimen is strained axially the constrained modulus increases in the one-dimensional compression test. According to Rankine's theory of earth pressures, a soil under increasing vertical stress will increase its horizontal stress as well. The increase in constrained modulus with vertical strain is due to this increase in confining pressure under at-rest (K_0) conditions.

It is also common that the secant modulus decreases over the course of a triaxial test, as the specimen is strained axially. All natural and composite specimens exhibited this behaviour.

Sand and Granulite exhibited the highest values for both moduli. Granulite had a higher range of values than sand for secant modulus, and both had a very similar range of values for constrained modulus. The composite specimen of sand and Granulite exhibited a higher secant modulus at the beginning of the triaxial test than either constituent, but had a similar value towards the end of the test. Its constrained modulus was greater than either constituent over the course of the one-dimensional compression test. This could be explained by the reduction of crushing, as discussed in Section 4 of this chapter.

Tire chip and HDPE chip specimens showed dramatically lower values for all moduli than corresponding values for sand. The mixture of tire chips and sand had ranges of values for the moduli similar to those for natural tire chips. It seems that the addition of sand did not appreciably increase the moduli. HDPE chip and sand mixed with a high HDPE content also exhibited little change of moduli. The secant moduli only increased noticeably for a ratio of 1:2.7, increasing slightly again as the amount of sand was increased to 4 parts (by weight) to 1 part HDPE. The constrained modulus did not significantly change until the sand accounted for 80% by weight of the mixture.

Tire chips have had a reported range of constrained modulus of 1.27 to 1.74 MPa (Humphrey et. al 1993) at an applied stress of 500 kPa, and a constrained modulus of 1.62 MPa at a vertical stress of 690 kPa (Edil and Bosscher, 1994). These values are somewhat higher than the range of modulus found in this study, but the maximum applied stress in this study was only 280 kPa. As stated before, the constrained modulus is

dependent on the applied stress. Typical data presented by Humphrey and Manion (1992) for a stress level of 300 kPa reveal a range for constrained moduli from 0.75 to 0.97 MPa, which corresponds closely with the findings in this study.

Solid HDPE has an elastic modulus ranging between 0.42 and 1.4 GPa (John, 1983). For comparison, ceramic materials have elastic moduli of between 150 and 450 GPa (ibid.). Sand particles can be characterized as ceramic in character. Constrained moduli for dense Ottawa sands under initial loading conditions have been reported to range from 200 to 500 MPa (Lambe and Whitman, 1979) for applied stresses between 100 and 700 kPa. If a relation exists between the elastic modulus of a solid and the constrained modulus for a granular material made of these solid particles, it could be expected that the constrained modulus for HDPE would be between 0.42 and 1.4 MPa, i.e. three orders of magnitude less than that of the solid material. As seen in Table 5-1, the constrained modulus was 0.93 MPa at an applied stress of 320 kPa which seems in agreement with previous studies.

The initial tangent modulus for all materials conformed to values of secant modulus for low strain, by definition. The addition of sand to the lightweight materials resulted in a small increase in E_t in all cases. This increase was small for the mixtures of sand and tire chips. E_t became much larger than either constituent material for the mixture of sand and Granulite. This is likely due to the reduced particle crushing, as discussed in Section 4 of this chapter. The value for E_t for the composite material of sand and HDPE was closely related to the densities of those mixtures.

The reload modulus for sand, HDPE, and mixtures of these two constituents was within a range of 28.5 to 44.8 MPa. This suggests that reloading in compression will yield

similar values of reload modulus regardless of the composition. The main difference for these specimens lies then in the various strength properties.

Granulite and a mixture of Granulite and sand exhibited notably higher reload moduli than sand alone. The high value of reload modulus also corresponded to a high peak stress at the beginning of the unload (and the end of the reload) cycle. This is also reflected in the Mohr-Coulomb failure envelope for these two specimens as will be discussed in the next section.

Tire chips were not unloaded and reloaded. No reload modulus was determined for them. The mixture of tire chips and sand displayed a reload modulus of 2.3 MPa, indicating that there was significant strain recovery occurring during the unloading cycle.

5.3 Mohr-Coulomb Failure Envelope

5.3.1 Mohr-Coulomb Failure Envelope for Lightweight Aggregates

Mohr circles were plotted for each stage of each test, and the failure envelopes were drawn as a best fit tangent to these circles, shown in Figures 5-18 through Figure 5-26. The test results were corrected to account for the extra confinement provided by the rubber membrane, by assuming that the sand had zero cohesion. This gave the proper additional confining pressure for each stage of the tests.

In all tests, except for the HDPE and Granulite test specimens, linear Mohr-Coulomb failure envelopes were found with no significant cohesion intercept. This was common for the low confining pressures used in this investigation for a free-draining granular material.

The test of Granulite did not exhibit a linear failure envelope because the test progressed past the vertical effective stress at which the particles were found to crush. WPL Engineering (1987) found that an axial compressive stress of 300 kPa caused significant particle crushing. In this study, there was audible crushing occurring during the second, third and fourth stages of triaxial testing. The failure envelope was taken to be tangent to the first Mohr circle, intercepting with zero shear and normal stress. The resulting angle of internal friction was found to be 42° , which is in agreement with the reported range of 40 to 45° for Granulite (WPL, 1987) and slightly higher than the 37 to 40° found for other tested expanded shales (Valsangkar and Holm, 1990).

In the triaxial tests for HDPE, the specimen failed at the end of the second stage (Figure 5-11). This was indicated by the lower peak deviator stress during the reloading phase compared to the peak stress during the initial loading phase.

The lower peak stress from the second stage was plotted as a deviator stress for a Mohr circle along with the peak stress for the initial loading of the third stage. The line tangent to these two circles has a non-zero intercept with the shear stress axis, indicating that the envelope is not linear. From this it is apparent that staged tests conducted on HDPE might be applicable only for low confining stresses. Residual strength might eventually become apparent, but this test did not reveal any consistent residual strength for this material. The predominantly horizontal layers of the HDPE specimens and slippage, rather than shear, might also be responsible for the non-linear strength envelope. Once incipient failure has occurred the specimen cannot be reconsolidated under subsequent confining stresses.

For the triaxial test of tire chips, the failure was taken to occur at a vertical strain of 20%, at which point the volumetric strain rate had approached zero change. In triaxial testing, this condition is sometimes called the “ultimate” or “constant volume” condition (Lambe and Whitman, 1979). This condition corresponded to an internal angle of friction of 28° , assuming zero cohesion after membrane correction. The stress carried by the tire chips continued to increase with strain, suggesting that the material never actively sheared. As compression behaviour will probably be the controlling factor in the geotechnical use of tire chips in an embankment, it is felt that the compression index or moduli will better define the mechanical characteristics of this lightweight material.

5.3.2 Mohr-Coulomb Failure Envelope for Sand/Lightweight Aggregate Specimens

Edil and Bosscher (1992) conducted direct shear tests on mixtures of sand and tire chips. The mixtures were 25% tire chips by weight, 35% by volume. Their analysis compared the internal angle of friction of the composite material to that of the sand in loose and dense states. It was found that in this mixture, the direct shear tests produced a Mohr-Coulomb failure envelope that was between those of the sand in loose and dense states. Further tests explored the effect of placing chips specifically across the shear plane and they found that the Mohr-Coulomb failure envelope of the natural sand in its dense state fell below that of the sand and tire chip composite.

The results of the triaxial tests in this study confirm the findings of Edil and Bosscher's study. The internal angle of friction of a randomly mixed specimen of sand and HDPE chips yielded a small range of values (between 41° and 43°) for various mixing ratios. The composite material was made of one part of tire chips to 0.9 parts of sand by

weight. It was found to have a shear angle of 41° . In both cases the composite materials had internal angles of friction that exceeded that of sand in its natural dense state.

Membrane compliance was not considered for this study, as the materials of interest (mixtures) would have similar compliance to the sand alone.

5.4 Impact of Grain Size Distribution and Particle Properties

All lightweight aggregates when mixed with sand had the effect of increasing the internal angle of friction of the composite when compared to sand specimens. In the case of Granulite, this result was obvious as the material itself exhibited shear strength greater than sand. The mixtures of HDPE and sand all exhibited larger internal angles of friction than the natural constituent materials, as did the mixture of sand and tire chips.

All mixtures were significantly gap graded. The compressible lightweight aggregates mitigated this somewhat by their ability to change shape to conform to the pore spaces available. This action of particle deformation had a significant impact on the moduli discussed in previous sections of this chapter.

It was observed that sand particles would partially penetrate the HDPE particles under loading, increasing the surface roughness of the HDPE chips. These indentations would disappear and the chips would return to their smooth shape after a period of approximately 24 hours of load removal. This penetration of HDPE chips by sand particles, coupled with the change in shape of the HDPE leading to an interlock, could explain the large amount of increase of the internal angle of friction with the addition of only a small proportion of sand to the HDPE chips.

Skermer and Hillis (1970) found that coarse-grained soils with particles likely to be crushed in triaxial testing would tend to change its gradation to conform to Fuller's optimum gradation, as expressed by:

$$\text{Percent Passed} = \sqrt{\frac{\text{sieve aperture}}{\text{max size of soil particle}}} \quad (5-6)$$

A material that starts with a gradation curve differing from the "Fuller Curve" would experience particle crushing until the gradation of the soil at failure would approximate this gradation.

In the case of the HDPE and tire chips, the gradation did not change as no particle breakage occurred. Granulite, on the other hand, was prone to crushing, and the gradation did indeed change over the course of the test, as demonstrated by the different grain size distribution curves in Figure 5-27.

It was found that adding Granulite to sand had the effect of increasing the shear strength to roughly the same strength as Granulite tested in its natural state. Sand did not increase the internal angle of shear of the Granulite, but did reduce the crushing of Granulite.

5.5 Permeability

It has been suggested that the coefficient of permeability of a soil is related to its uniformity coefficient (C_u), its effective size (D_x) with x% by weight having smaller particle size, and its void ratio, e . Shahabi, Das and Tarquin (1984) proposed the relation in Equation 4-1. The equation was further refined to

$$k = 1.2 C_u^{0.735} D_{10}^{0.89} \frac{e^3}{1+e} \quad (5-7)$$

relating the coefficient of permeability to the uniformity coefficient, C_u , and the nominal size D_{10} .

Kenney et. al (1984) found that the coefficient of permeability of variously graded soils was related to D_α , in the relation

$$k = \beta_\alpha D_\alpha^2 \quad (5-8)$$

with α and β as constants. α was chosen based on laboratory analysis. For a given value of α it was suggested that there will be a unique value of β . Kenney et. al found that a value of $\alpha = 5$ would give the best correspondence to the data. D_α^2 was chosen as the proper dimensional relation to k through dimensional analysis. Their study found that reliance of β on C_u was minimal at best. It was suggested by Kenney et. al that a nominal size of D_5 or smaller would control, owing to the effects of small pore spaces and effective surface area on fluid flow, regardless of the size of larger particles.

Plots of constituent materials' coefficient of permeability versus void ratio were presented in the previous chapter (Figures 4-5 through 4-8). Lines representing a relationship parallel to equation 5-2 were plotted on the same graph, with the $1.2C_u^{0.735}D_{10}^{0.89}$ expression replaced by a constant. This constant was chosen such that the equation and the experimental value of k were equal at a vertical effective stress of 20 kPa. Except for the test of Granulite which involved the crushing of some particles, the C_u and D_{10} of each specimen remained constant for the duration of each compression test. This permitted the substitution of a constant for this expression. The values of C_u and D_{10} both reduced over the course of the test of Granulite, resulting in a lower calculated value for k . D_{10} in particular would have reduced the calculated value of k by a factor of 5. As can be

seen in Figure 4-6, the calculated k is twice the experimental value. Reducing the calculated k by a factor of 5 would make it 0.4 of the experimental value.

It can be seen that the calculated and measured values of k for these materials did not vary with e as suggested by Shahabi et. al (1984). Further, the slope for the compression test of sand was less than the corresponding calculated curves, meaning that k declined more rapidly with reduction in void ratio than predicted by Equation (5-1). The slopes for the tire chips and HDPE were higher, revealing a smaller reduction of k with a reduction of e . The coefficient of permeability of Granulite changed less than predicted by Equation (5-7)) as well, when the change in C_u and D_{10} were taken into account.

The D_5 for sand is between 20 and 200 times smaller than that for the other constituent materials. Thus the coefficient of permeability of these materials would have to be between 20^2 and 200^2 times larger than that of sand, which was not found to be the case. The difference between coefficients is closer to an order of magnitude between the most free-draining material and the least.

A relation similar to Equation 4-1 was suggested for normally consolidated clays (Das, 1990), with the numerator changed from e^3 to e^n . This suggests that there is a natural void ratio that applies to the soil being considered. In light of this, the application of this relation to a single soil specimen at varied void ratios is questionable.

5.6 Permeability of Sand/Lightweight Aggregate Specimens

The gap-graded character of the mixtures precluded a strong influence on the change in coefficient of permeability by the change in D_5 . By examining the grain size distributions, as was shown in Chapter 3, it can be seen that D_5 consisted of a very small

portion of all mixtures, with the largest D_5 being equal that of the smallest D_5 , as seen in the grain size distributions (Figures 3-2 and 3-3). The great variability of the coefficient of permeability between each mixture, as shown in Figures 5-28 through 5-30, cannot be explained by this nominal size alone.

It must be pointed out that there are no other physical properties of a geotechnical material that can change over many orders of magnitude as the coefficient of permeability does. Therefore any material's suitability for a certain geotechnical application must be evaluated once the range of coefficients of permeability has been established from laboratory or field tests for a that material.

	C_c (log kPa) ⁻¹	E_c MPa (@ vertical strain)	E_i MPa	E_s MPa (@ vertical strain)	E_r MPa
Sand	0.034521	1.66 @ 1.2% 12.2 @ 2.6%	13.4	10.8 @ 0.4% 6.45 @ 2.0%	41.9
Granulite	0.064391	5.44 @ 0.3% 12.4 @ 2.6%	28.2	27.5 @ 0.4% 14.8 @ 2.0%	50.9
Tire chips	0.57193	0.3 @ 4.0% 0.58 @ 27.6%	0.73	0.8 @ 4.0% 0.56 @ 16.0%	
HDPE	0.591591	0.24 @ 8.2% 0.93 @ 34%	1.42	1.0 @ 1.0% 0.44 @ 5.0%	35
Sand + Granulite	0.050941	1.8 @ 0.4% 0.65 @ 2.0%	35.7	1.8 @ 0.4% 0.65 @ 2.0%	82.1
Sand + Tire Chips	0.280404	0.14 @ 14.0% 1.14 @ 28.0%	1.20	2.75 @ 2.0% 1.35 @ 13.0%	2.32
Sand + HDPE (1:1.4)	0.513125	0.3 @ 6.6% 1.21 @ 26.5%	2.75	2.67 @ 3.0% 2.25 @ 18.0%	28.5
Sand + HDPE (1:2.7)	0.310135	0.3 @ 6.6% 2.5 @ 25.8%	6.35	6.25 @ 2.0% 4.31 @ 8.0%	29.9
Sand + HDPE (1:4.0)	0.132098	1.9 @ 1.1% 5.9 @ 10.9%	9.43	7.0 @ 1.0% 5.5 @ 6.0%	44.8

Table 5-1. Summary of Results of Compression Tests

	Unit weight, DMI (kN/m ³)	Internal Angle of Friction (°)	k (mm/s) (@ applied vertical stress, kPa)
Sand	16070	28	4.75x10 ⁻³ @ 20 4.05x10 ⁻³ @ 160
Granulite	7520	42	14.8x10 ⁻³ @ 20 12.3x10 ⁻³ @ 160
Tire chips	5350	28	6.85x10 ⁻³ @ 30 5.84x10 ⁻³ @ 160
HDPE	3385	8	11.9x10 ⁻³ @ 20 9.37x10 ⁻³ @ 160
Sand + Granulite	11650	43	3.70x10 ⁻³ @ 20 3.40x10 ⁻³ @ 160
Sand + Tire Chips	10820	40	5.48x10 ⁻³ @ 20 4.40x10 ⁻³ @ 160
Sand + HDPE (1:1.4)	6765	42	9.37x10 ⁻³ @ 20 5.65x10 ⁻³ @ 160
Sand + HDPE (1:2.7)	9810	43	7.92x10 ⁻³ @ 20 4.45x10 ⁻³ @ 160
Sand + HDPE (1:4.0)	12890	42	5.94x10 ⁻³ @ 20 3.96x10 ⁻³ @ 160

Table 5-2. Summary of Results of Triaxial and Constant Head Permeability Tests

Figure 5-1. Internal angle of friction vs. unit weight of mixtures

Figure 5-2. Compression Index vs. unit weight of mixtures

Figure 5-3. Moduli vs. strain for sand

Figure 5-4. Moduli vs. strain for HDPE

Figure 5-5. Moduli vs. strain for tire chips

Figure 5-6. Moduli vs. strain for Granulite

Figure 5-7. Moduli vs. strain for mixture of Granulite and sand

Figure 5-8. Moduli vs. strain for mixture of tire chips and sand

Figure 5-9. Moduli vs. strain for mixture of HDPE and sand (1:1.4)

Figure 5-10. Moduli vs. strain for mixture of HDPE and sand (1:2.7)

Figure 5-11. Moduli vs. strain for mixture of HDPE and sand (1:4.0)

Figure 5-12. Secant moduli vs. strain for mixtures of HDPE and sand

Figure 5-13. Constrained moduli vs. strain for mixtures of HDPE and sand

Figure 5-14. Secant moduli vs. vertical strain for mixtures of tire chips and sand

Figure 5-15. Constrained moduli vs. vertical strain for mixtures of tire chips and sand

Figure 5-16. Secant moduli vs. vertical strain for mixtures of Granulite and sand

Figure 5-17. Constrained moduli vs. vertical strain for mixtures of Granulite and sand

Figure 5-18. Mohr-Coulomb failure envelope for sand

Figure 5-19. Mohr-Coulomb failure envelope for Granulite

Figure 5-20. Mohr-Coulomb failure envelope for mixture of Granulite and sand (1:0.65)

Figure 5-21. Mohr-Coulomb failure envelope for tire chips

Figure 5-22. Mohr-Coulomb failure envelope for mixture of tire chips and sand (1:0.9)

Figure 5-23. Mohr-Coulomb failure envelope for HDPE

Figure 5-24. Mohr-Coulomb failure envelope for mixture of HDPE and sand (1:1.4)

Figure 5-25. Mohr-Coulomb failure envelope for mixture of HDPE and sand (1:2.7)

Figure 5-26. Mohr-Coulomb failure envelope for mixture of HDPE and sand (1:4)

Figure 5-27. Grain Size Distribution of Granulite before and after compression testing

Figure 5-28. Coefficient of permeability vs. unit weight of mixtures (at applied stress 40 kPa)

Figure 5-29. Coefficient of permeability vs. unit weight of mixtures (at applied stress 840 kPa)

Figure 5-30. Coefficient of permeability vs. unit weight of mixtures (at applied stress 160 kPa)

6. Summary and Conclusions

6.1 Summary

Expanded clay, rubber tire chips and high density polyethylene chips were considered for potential use as lightweight embankment fill. The study consisted of carrying out a series of triaxial compression and oedometer tests on lightweight materials and mixtures of lightweight materials and a common sand fill. The triaxial tests were carried out in multiple stages, with a single reload cycle for each stage.

The triaxial specimens had a diameter of 230 mm and a height of 460 mm. The triaxial specimens were tested under confining pressures ranging from 20 to 200 kPa. The stress-strain relations were analyzed to compare the behaviour of the lightweight materials to each other, the sand, and mixtures of the sand and lightweight materials.

Addition of sand to the lightweight materials resulted in a gain in shear strength, as exhibited by an increase in the internal angle of friction for the lightweight materials.

The reloading modulus for each specimen was compared for each cycle within the test, as well as to other materials. The secant moduli and initial tangent moduli of the specimens were compared for the initial stage.

Oedometer test samples had a diameter of 254 mm and an initial height of 230 mm. The samples were vertically strained to a maximum of 30%, or to an applied vertical stress of 640 kPa. Constant head permeability tests were carried out over a range of vertical strains during the oedometer test. Coefficients of permeability were compared for specimens at varying void ratios and against specimens made of the natural materials.

Constrained moduli for each specimen were compared to the secant moduli of the specimens of the same constitution as found in the triaxial compression tests.

6.2 Conclusions

The purpose of this thesis was to evaluate the suitability of potential lightweight aggregates for use in embankments based on geotechnical properties. The mixed specimens all exhibited the desired increase in strength and decrease in compressibility while retaining their low unit weight.

6.2.1 Strength

Addition of sand to lightweight materials increased the strength of the weaker lightweight materials, i.e. HDPE and rubber tire chips. It did not significantly alter the strength of the expanded clay. Strength was determined by the Mohr-Coulomb failure law; as such, the increase in strength was due to an increase in the internal angle of friction.

All mixed constituent samples exhibited a similar failure envelope, with an internal angle of friction between 40° and 42° . Variation of the ratios of sand to lightweight material seemed to have little effect on the overall shear strength of the samples over the range considered. The internal angle of friction must vary significantly at the initial addition of one material to the other, whether considering the addition of the lightweight material to the sand or vice versa.

The internal angle of friction of the mixture of expanded clay and sand was similar to the internal angle of friction of the expanded clay alone. The addition of sand to Granulite does not affect significantly the Mohr-Coulomb failure envelope of Granulite.

6.2.2 Deformation

The strain to failure increased with higher proportions of HDPE or tire chips. This was reflected in both the triaxial tests and the oedometer compression tests. Tire chips exhibited the highest vertical strain under a given applied stress. The addition of sand to fill the pore spaces in the tire chips did not significantly reduce the amount of vertical deformation for an applied stress in triaxial compression. The compression index C_c decreased linearly with the change in density maximum index unit weight for all materials. This shows that the compression index of a mixed constituent material is a function of the compression indices of the constituent materials and the mixture ratio, as described by the unit weight of the materials.

6.2.3 Soil Moduli

The initial tangent moduli for lightweight materials was very low when compared to sand. The exception to this was the Granulite, which exhibited an initial tangent modulus much higher than sand. The initial tangent modulus for tire chips did not significantly change with the addition of sand; it did increase linearly with density maximum index unit weight for the mixtures of sand and HDPE.

Rebound moduli for most materials were in a small range, varying from 28.5 to 50.9 MPa. The exceptions were the mixtures of Granulite and sand and rubber tire chips and sand. These latter two were significantly outside this range. The rebound modulus

for the mixture of Granulite and sand was 82.1 MPa, and 2.32 MPa for the mixture of tire chips and sand. Tire chips' reload modulus was not determined.

Secant moduli for all lightweight materials increased with the addition of sand. All materials exhibited a decrease with continued vertical strain, as shown by the decrease of the secant moduli with increasing vertical strain.

Constrained moduli for tire chips and HDPE increased with the addition of sand, but this increase was not in direct proportion to the sand added. The most significant increase in modulus occurred for ratios of 1:2.7 and 1:4 of HDPE and sand. The constrained modulus of Granulite increased upon the addition of sand; the resulting was greater than the moduli of either constituent material.

6.2.4 Coefficient of Permeability

The coefficient of permeability decreased with the addition of sand. The addition of sand to Granulite resulted in a coefficient of permeability lower than that of either of the two constituent materials. The other coefficients of the lightweight materials varied more linearly with the change in unit weight of the mixture. The materials are characterized as free-draining. The coefficients were determined assuming laminar flow of water through the soil matrix.

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TEST TARGET (QA-3)

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