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Lightweight Artificial and Waste Materials for Embankments over Soft Soils

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Transportation Research Record 1422

Contents

Foreword

The use of fill materials other than soils for embankments, particularly over soft foundations, is becoming increasingly common. The 10 papers in this Record contribute significantly toward dissemination of information on the findings of engineers having experience with the use of such materials. The papers are grouped into two broad categories: lightweight artificial materials and lightweight waste materials fabricated to address special problems in design and construction. The latter category treats the techniques of using materials that normally end up in a landfill as materials of construction, thereby solving some of society's waste problems.

The paper by Monahan presents an overview of the use of artificial lightweight materials as well as a "weight-credit" concept that relates to the use of such material. The following three papers are on expanded shale. The paper by Holm and Valsangkar describes the general engineering properties of expanded shale that are of importance during design. In the next paper, Valsangkar and Holm report on the results of a series of tests that show the settlement and strength behavior of expanded shale under cyclic loads. Dugan's paper describes construction of the approach embankment to the new Charter Oak Bridge across the Connecticut River in Hartford, Connecticut. On this project, the lightweight expanded shale allowed the construction to be completed in a safe and timely manner. Harbuck summarizes the experiences of the New York State Department of Transportation with the use of lightweight foamed concrete fill. This paper describes typical applications, placement, and testing.

The remaining five papers in the Record are on the use of lightweight waste material. Humphrey et al. present the engineering properties, such as gradation, specific gravity, absorption, compacted density, and shear strength, of chips of waste rubber tires that require consideration during the design phase. Upton and Machan describe the use of tire chips for repairing an unstable embankment and results of the postconstruction monitoring program. The next two papers are on the use of wood fill in the state of Washington. Allen and Kilian report on a project that used a combination of geotextile reinforcement and wood fiber fill to span a soft deposit. Kilian and Ferry present results of a study that examined wood fiber fills that had been in service since 1972. They report that many of the fills examined showed little deterioration. Ahmed and Lovell examine the subject of rubber tires in transportation fills.

Weight-Credit Foundation Construction Using Artificial Fills

EDWARD J. MONAHAN

A description of the origin and evolution of a major type of artificial fill, foam plastics, is given. Case histories of weight-credit applications for a variety of materials, including solid (precast) foam plastics, cast-in-place and poured plastics, Elastizell, Solite, and waste materials (wood chips and shredded rubber tires), are described. Abstract information about pertinent mechanical and chemical properties is given and evaluated. Aspects of permanence and durability, including problems and caveats, are reviewed. An approach called "hybrid design" is suggested. A case is made for greater use of artificial fills. The literature that will enable a comprehensive study of weight-credit design and construction, with particular emphasis on case histories, is cited.

In the history of construction, the lightest material that would suffice, whether a wood pile in a prehistoric bog or a titanium alloy for today's jets, was chosen.

Perhaps the first known public reference to a conscious choice of a lightweight fill to achieve a weight credit was made by Benjamin Hough in a lecture before the Geotechnical Group of the New York Metropolitan Section, ASCE (about 1960). If a lightweight cinder fill with acceptable strength and stability was used, a significant "credit" in the form of a correspondingly higher structural load (payload) could be obtained.

It has been common practice among geotechnical engineers to specify the use of clean, well-graded granular materials for fills to replace unsuitable surface soils of relatively shallow depth (for example, peats). The compacted fill would, of course, be of significantly higher unit weight than the peat it replaced—perhaps 2000 kg/m^3 (125 pcf) for the compacted granular fill and 1600 kg/m^3 (100 pcf) for the peat. This would diminish the payload (structure weight) that would be permissible. Nevertheless, the procedure is routinely followed, because the readily available suitable lightweight fill, such as the cinder material suggested by Hough, is expensive.

In about 1965, the use of solid foam plastic for the hulls of small recreation sailboats was introduced at the New York Coliseum Boat Show. One of the selling points was its light weight. Indeed, potential customers were invited to lift the hull with an index finger to illustrate effectively this advantage. This material had to be strong as well as light to withstand the pressure of the foot of an adult male. The idea to use the material as a fill evolved. After preliminary investigation, it became evident that the weight credit that could be achieved was dramatic, far surpassing any so far possible. Accordingly, patents were applied for and awarded in 1971 and 1973 (*1,2*).

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One of the materials covered by the patents is known by the more technically explicit term, "expanded polystyrene." It is typically produced as "boards" that are tied together to form "bundles," or "blocks," commonly called EPS blocks.

The first known use of EPS blocks in highway construction was as insulation for highway subgrades in cold regions to protect against frost heave. This method of construction was patented in 1966 by Leonards. Applications for purposes of weight credit were not a part of the Leonards patent. By 1967, pavements insulated with EPS had been installed in 11 states and 3 Canadian provinces (*3*).

WEIGHT-CREDIT CONSTRUCTION WITH FOAM PLASTIC

Foam Materials

The only foam plastic for which specific properties are known, and has been used in weight-credit construction, is a solid precast extruded polystyrene foam made by Dow Chemical. The broad potential for the general use of foam plastics is illustrated by the following: "[F]oams may be produced which have densities ranging from less than one pcf to about 70 pcf, with an almost limitless range of chemical and mechanical properties" (*4*).

It appears feasible that for very large jobs, or very special circumstances, the expense of producing a special formulation to suit the particular use could be justified. In addition, combinations of existing products could be used to effect a design. Just as in the design of a pavement cross section, better materials could be placed where the stresses are highest, and lesser materials could be used where the stresses are lower.

Pickford Bridge

In the early 1970s, a representative of Dow Chemical was granted permission by the author to pursue a construction project covered by patents (*1,2*). The EPS blocks would be used, on a job for the Michigan Highway Department, to replace a badly deteriorating abutment and approach fill for a bridge at Pickford, Michigan (*5*, p. 83). The existing abutment and approach fill had settled to the point at which action was necessary (A. Maki, engineer, Dow Chemical, personal communication). The fill was replaced by a conventional compacted soil. However, because of the existence of deep deposits of soft clay below the fill, the new fill started to settle rapidly, with obvious potential damage to the abutment and bridge. To avoid complete failure, the approach fill was removed. The weight-credit construction that was then used is described by Coleman (*6*) but augmented here by commentary and results of follow-up inquiries.

Before the weight-credit construction began, the clay upon which the foam bundles were to be placed was reported to be extremely soft. The EPS bundles were placed by hand by two men in ½ day (A. Maki, personal communication).

The fill at the abutment was about 3m (10 ft) high. However, the thickness of plastic necessary to achieve the required weight credit was about 1.5m (5 ft), thus sensibly diminishing the amount and cost of the plastic fill. [The cost of the in-place foam at the Pickford Bridge was $52/m^3 ($40/yd^3), but subsequent costs fluctuated greatly because the plastic is an oil-based product.] The EPS bundles were covered with a polyethylene sheet to protect against oil or gasoline spills. Normal soil fill was placed to subgrade, serving to pin down the foam plastic to guard against floating during periods of high water table.

Because this project represents the earliest known use of large thicknesses of foam plastic for weight-credit construction, inquiries by the author about the performance record of the construction were recently made. The answer was that

> on the basis of my periodic on-site visual inspections . . . the longitudinal section of the bridge has remained stable. On the return-wall section, where the potential for movement was deemed greatest, there has been "some movement" . . . but it has been "manageable." All things considered, the construction has been satisfactory over its approximately 20-year history. (P. O'Rourke, Engineer, Materials and Technology Section, Michigan Department of Transportation, 1992.)

EXPANDED POLYSTYRENE: SELECTED TECHNICAL DATA

The following table gives properties of Styrofoam HI-35, the EPS that was used at the Pickford Bridge:

	Test Data	*Test Method*
Compressive strength at 5% deflection	241 kPa (2.5 tsf)	ASTM D1621-59T
Water absorption	0.25% (by volume)	ASTM C272-53
Density	40 kg/m^3 (2.5 pcf)	

Permanence and Durability

Although no direct inspection (by coring or excavation) has been made at the Pickford Bridge, periodic visual on-site inspections have shown that the material remains stable.

Between 1962 and 1966, EPS blocks have been used in at least 39 installations as insulation for highway pavement systems. In most cases, the amount of plastic used has been about 25 to 75 mm (1 to 3 in.), typically placed directly on the subgrade. Samples of foam taken from various highways after several years of service show very little water absorption. Accelerated laboratory tests, such as freeze-thaw cycling and soak tests, have shown very little moisture absorption (*7*).

Deformations caused by trafficking were measured in two insulated sections and one noninsulated (control) section. Deformations were of the order of 0.5 mm (0.02 in.) in the insulated sections and were well below those of the control section, reaching a maximum of about 0.8 mm (0.03 in.), during the spring thaw (*7*).

Problems and Caveats

Differential Icing

The purpose of the foam plastic on highway insulation installations is to minimize frost action in susceptible subgrade soils. However, there is danger of creating a much more serious, indeed dangerous, problem—differential icing. On days when the ambient temperature is at or slightly below freezing, an untreated section of the pavement receives heat from both the sun and the subgrade. Thus, if it rains, the water on the pavement will remain fluid. On an adjacent treated section, where the foam provides insulation from the heating effects of the subgrade, pavement water may freeze. This can be especially dangerous for a motorist driving fast on the untreated section who believes that the pavement is merely wet. Entering the iced section can be disastrous. In fact, in the early 1970s, a serious accident did occur and was judged to have been caused by the icing described. As a result, the manufacturer decided to stop using foam as an artificial fill. The company has done some studies of the icing problem and determined that icing would not be a problem in regions of less than 1000 degree-days. (Dow Chemical Co., early 1970s, personal communication) This is a partial explanation of why the techniques have not been used as extensively as they might have been, or as extensively as in Norway.

In most cases of weight-credit construction, however, the problem would not exist or would be manageable. At the Pickford Bridge, for example, the foam plastic is sufficiently buried not to create the problem of icing. In similar bridge approaches where the foam might be at shallower depths (for greater weight-credit), it would be advisable to install a hazard sign.

Ozone Depletion

Depletion of the ozone layer by the release of chlorofluorocarbons (CFCs) into the atmosphere is of major worldwide concern. Some scientists estimate that projected rates of depletion could cause major increases in skin cancers and eye cataracts and deplete humans' ability to fight infection. Crop damage and disruption of the ocean food chain could also result.

One of the sources of CFCs is reported to be "the propellants that are used in the production of foam plastics" (*8*). It is not known whether that includes the foams currently used for insulation of highways and weight-credit applications, but it is suspected to be the case. Recent United Nations talks have established a deadline for banning certain CFC-producing products by 1996 (*9*).

Flotation

Where soil overburden will not be sufficient to prevent flotation, provision would have to be incorporated in the design to pin down the foam. Soil "pins" analogous to anchor bolts in tunnel construction might be used. Studies need to be done to determine design specifics and to assess the effects on the integrity of the foam fill.

Chemical Resistance

Exposure of the in-ground foam to gasolines should be prevented. It is well known that his causes very rapid deterioration of the foam.

Sunlight

Direct exposure of foam plastic to sunlight for extended periods should be avoided. The manufacturer of Styrofoam HI-35 reports, "discoloration and degradation of properties may occur at the surfaces exposed to direct sunlight. Covering the product with a white plastic sheet is recommended if it will be exposed for more than three days" (A. Maki, Dow Chemical Company, personal communication).

Other Case Histories

Monahan (*10,11*) describes a variety of applications, actual projects, (route construction, highways, pipelines), and hypothetical applications. Cast-in-place applications that deal mainly with confined spaces (such as behind retaining walls, trenches) and a suggested plastic-filled weight-credit pile are described. A grade-separation case study for an intersection in a major eastern city is included. (An embankment slope stability problem is described that suggests the combined use of reinforced earth principles with those of weight-credit.) A poured plastic, called Poleset, has been used for installing utility poles. The method is quicker and neater than standard earth backfills. Pulling tests are reported that claim the material is stronger than poles backfilled with compacted soils (*12*). Although not a weight-credit application, the material could be used effectively as such.

An extensive amount of work using EPS blocks has been done in Norway. It has been reported that almost 100 road projects have been successfully completed in Norway since 1972 (*13,14*). EPS backfill has been used behind seven newly constructed bridge and overpass abutments in soft foundation areas near Vancouver, Canada (*15*, p.25).

Other Foam Plastics

Many foam plastics from which the designer may choose are available. One manufacturer makes about 12 varieties of solid foam materials, each with different properties, but all extremely lightweight. For example, Styrofoam HI-300 has a density of 53 kg/m^3 (3.3 pcf), compared with the Pickford foam density of 40 kg/m^3 (2.5 pcf), yet its compressive strength is approximately 3½ times greater (*16*). Thus, much stronger, but undoubtedly more expensive, materials are available with very little sacrifice of weight credit.

Another EPS product, Styropor, is made by the BASF company. A slope stabilization application was completed in Colorado (*17*).

WEIGHT-CREDIT CONSTRUCTION WITH NONFOAM MATERIALS

Materials other than foam plastics have been developed in recent years for weight-credit applications.

Elastizell

Elastizell is a pumpable lightweight "concrete," produced on site by adding a liquid concentrate of hydrolized protein to a cement and water slurry. There are six classes, I through VI, with cast densities ranging from about 300 to 1300 kg/m^3 (18 to 80 pcf). Corresponding compressive strengths range from about 280 to 4800 kPa (40 to 700 psi).

One of the larger jobs done with Elastizell (a proprietary product) was for a bridge abutment over weak soils on I-94 near Minneapolis, Minnesota. About 92 000 m^3 was poured (42,000 yd^3) (*18*). Typical designs using Elastizell incorporate more than one class, placing the stronger materials where performance requirements warrant.

An extension of this approach would be to use the much lighter foam plastics with the Elastizell where extremely weak soils require dramatic weight credit. Elastizell does not require compaction and, once set, does not apply lateral pressure to walls (*18*).

Solite

Depending on the locale of its manufacture, Solite, also a proprietary product, is produced from either shale, clay, or slate. It is expanded in a rotary kiln at high temperature to produce a lightweight, subangular granular material that is free-draining (*19*). The material is used either as a soil fill or as aggregate to produce lightweight concrete.

As a fill, it is normally compacted to densities less than 960 kg/m^3 (60 pcf), yielding a material with an angle of internal friction of about 40 degrees. The material is chemically inert.

As a concrete, its unit weight is about 1900 kg/m^3 (116 pcf), with a 28-day compressive strength of about 44 900 kPa (6,510 psi).

Hybrid Design

A combination of materials could be considered for overall weight-credit approaches. Because Solite may be used as a lightweight concrete in the main structural members of a bridge, it would be feasible to design an entire project using super-lightweight materials (foam plastics) and lightweight materials (for example, Elastizell) for all fills (fitting the material selections to the weight-credit needs) and to use lightweight concrete (Solite) for many of the structure components—a true hybrid design.

Waste Fills

Waste or recycled materials have been used successfully and often provide the secondary benefit of weight credit.

One such job, designed by a company in Minnesota, involved the use of geotextile, wood chips, and shredded rubber tires as fill that crossed unstable peat soils. Geotextile was placed at the bottom of a 1.5-m (5-ft) excavation, and wood chips were placed to a height of 0.3 m (1 ft) above the water table, as required by the Minnesota Pollution Control Agency. Shredded tires were then placed to a height one m (3 ft) above the original road surface. The tire layer was covered with geotextile, and the fabric was then sewn together with the lower fabric to form an enclosing bag. The shredded tires weigh about one-sixth what conventional soil fill weighs.

Much more extensive descriptions of the use of waste fills are contained in a work by Monahan (*11*).

CONCLUSIONS

The use of artificial fills will become more widespread for a number of reasons. Clean soil fills of suitable gradation are becoming scarcer, especially in more congested areas. In the 1970s, it had often become necessary to search for enough fill for a job, often using three or more borrow areas to obtain the necessary quantity of suitable fill. This situation developed before environmental regulations became widespread. Engineers looking for suitable soil must now be concerned with both suitable texture and the very complicated problems of possible contamination.

Another factor that would favor the increased use of artificial fills is the reported market lag of recycled materials, including plastics, glass, paper, and aluminum (*20*).

There are many benefits to be gained by the increased use of artificial fills: the avoidance of environmental entanglements (paperwork and possible law suits), economic benefits associated with conservation and recycling, perhaps indirectly major savings in energy consumption, and, very important, weight credit.

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DISCUSSION

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I am actively involved in research dealing with the application of rigid plastic foams to a wide variety of geotechnical problems, including lightweight fills (weight-credit construction). Such materials are now recognized as geosynthetics under the newly created product category of geofoams. An inventory of geofoam materials and functions identified to date is found in a work by Horvath (*1*).

I would like to clarify or correct several items in the paper by Monahan on the basis of current information relative to geofoams.

1. *Expanded polystyrene* and the corresponding acronym *EPS* as used by the author are not consistent with current plastics-industry terminology throughout the world (the only known exception is Japan). There are two types of rigid polystyrene foams manufactured by different processes: molded bead and extrusion. Differentiation between these two polystyrene foams is not trivial. There are significant differences in cost, environmental effects related to manufacture, finished product size, and material properties. The term expanded polystyrene (EPS) is used only when referring to the molded-bead product. The extruded product is called extruded polystyrene (XPS). Consequently, readers of the paper should be aware that the author appears to mean XPS in most instances in which he uses EPS (Dow Chemical, referenced extensively in the paper, produces only XPS and not "true" EPS). To complicate matters, there are cases in which the author uses the acronym EPS correctly. These exceptions are toward the end of the paper where projects in Norway, Canada, and Colorado are noted. On these projects, the molded-bead product (EPS) is meant.

Other terminology issues are that the word Styrofoam is not a generic name for all plastic foams (as many believe) or even a generic name for XPS. It is the brand name of the particular XPS product manufactured by the Dow Chemical Company (there are at least three manufacturers in the United States besides Dow that produce XPS, each with its own product brand name). Also, Styropor is the BASF Corporation's brand name of the basic polystyrene beads (called expandable polystyrene) from which EPS is produced, not the finished EPS product. (BASF does not make the finished product.) The remainder of this discussion will use the correct terminology as defined.

2. EPS was invented more than 40 years ago and has been used in geotechnical applications for more than 30 years. Ini-

tial use was for thermal insulation below roads to prevent frost heave (XPS was used for this also, as noted by the author), followed by use as lightweight fill for a highway embankment in Norway in 1972. Thus there have been decades of experience in which the geotechnical marketplace has compared EPS with other plastic foams (primarily XPS). The overwhelming choice has been and still is EPS. The primary reason is cost. EPS is typically half, or less than, the cost per unit volume of XPS. In addition, EPS blocks are larger (in the United States, typically 610 mm thick versus 102 mm maximum for XPS); therefore, placement is faster because fewer pieces must be handled.

Another issue that is increasingly more important (it is already a significant concern in Europe) is that EPS is the only rigid plastic foam that does not use gases such as CFC or HCFC (which deplete the upper-atmosphere ozone layer) in its manufacture. Thus the statement by the author that "the only foam plastic for which specific properties are known, and which has been used in weight-credit construction, is a solid precast extruded polystyrene foam . . ." (i.e., XPS) is incorrect and, unfortunately, gives a very misleading impression as to past and current geofoam use. Examples and references of the extensive use of EPS for lightweight fill and other functions are found elsewhere (*2*,*3*). Material behavior of EPS for engineering analysis purposes is well defined in literature readily available from the EPS industry (*4–9*). A synthesis of basic EPS properties for geotechnical application has recently been prepared (*10*).

3. The statement that XPS can be produced with a density of 70 pcf does not seem plausible. Solid polystyrene has a specific gravity of approximately 1.1. Thus a solid block of polystyrene would have a density less than 70 pcf, making it physically impossible that a polystyrene-based foam with voids could be denser.

In summary, potential users of geofoams should be aware that EPS, not XPS, is the material of choice for lightweight fill, as well as other geotechnical applications in which a rigid plastic foam is being considered. This has been true for more than 20 years. In addition, technical data are available that allow EPS to be "engineered" probably better than any other plastic foam because of its extensive geotechnical use.

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AUTHOR'S CLOSURE

I thank the discussant for pointing out the important distinctions between expanded polystyrene (EPS) and extruded polystyrene (XPS). The standardization and codification of these terms and their acronyms appear to be a very recent development, as evidenced by the references cited by the discussant.

In my early writings and presentations, the broader terms "rigid foam plastics" and "cast-in-place" (or "poured") were used exclusively. Indeed, my patents (*1*,*2*) are couched in terms that include all forms of such foam plastics. Early writings by others used such terms as "polystyrene foam" or simply "plastic foam" (*3*,*6*). The first use of an acronym, as far as I am aware, was in September 1987 (*14*). I erroneously assumed that this was simply a shorthand notation to designate all solid foam plastics and thus assumed this usage for the first time for the preparation of my paper. The term XPS was seen for the first time in the discussion submitted in response to this paper.

The discussant points out that two important distinctions between EPS and XPS are cost and potential environmental hazard and says that Dow Chemical "produces only XPS" and that "EPS is the only rigid plastic foam that does not use gases such as CFC or HCFC in its manufacture." However, I spoke with the research director of Dow Chemical a few days before the presentation and was told that Dow's product has been made without the generation of such gases for some 2 years or so (Dow Chemical Company, personal communication). Because there appears to be a discrepancy between the discussant's statement and that made by the Dow research director, future users should be aware of the apparently conflicting claims.

Relative to other distinctions between EPS and XPS, potential users are advised to evaluate other aspects of physical, mechanical, and chemical properties (particularly water absorption and compressive strength), and of course relative cost, before making a design choice.

The discussant asserts that the "material behavior of EPS is readily available from the EPS industry" and cites five references of the BASF Corporation (*4–9*). However, I wrote to the BASF Corporation in 1989 but was not told of the availability of their technical literature. Other references cited by the discussant are either in press or of very recent vintage in journals not widely circulated, so there is some question about the discussant's assertion that the information is "readily available."

On a technical matter, the discussant states that it is "physically impossible" that a polystyrene-based foam could have a density as high as 70 pcf, inasmuch as the "solid polystyrene has a specific gravity of approximately 1.1." This matter is not pertinent to the principal focus of my paper, because no one would consider plastics of such high density for weight-credit applications.

However, because the point was raised, a response is warranted. I do not agree. First, specific gravity is not an absolute quantity (such as e or π). The discussant implicitly acknowledges this by using the correct term: "approximately." Second, there is documentation that the experimental determination of specific gravity (and other properties) can vary surprisingly widely from laboratory to laboratory. Finally, the discussant assumes that the voids in a block of polystyrene are all air and thus "weightless." In my experience, the air-dried moisture content of even granular soils, which have mineralogy with the least affinity for water, assumes values of 3 or 4 percent in a temperate region. Because the plastics are presumably produced in "normal" ambient settings, it is reasonable to assume that moisture (and other impurities) will be trapped in the voids. A moisture content of less than 2 percent would yield a density of about 70 pcf, and this seems reasonable. Thus an upper limit of 70 pcf is not only possible, it is a virtual certainty. Moreover, the 70 pcf figure was a quotation from the *Modern Plastics Encyclopaedia* (*4*) and was probably based on these factors.

I would like to close with additional comments about the reticence of some to respond to inquiries. During the approximately 23 years since I first conceived of and patented the use of foam plastics for weight-credit foundation construction, there have been a number of incidents of unanswered inquiries.

I am at a loss to understand how the discussant, after reading my paper, prepared a discussion that contained the history of EPS usage as perhaps its principal focus and yet made no reference to the fact that I was issued a patent (*1*) on the method in the year before Flaate's first use. Indeed, the discussant even continues to imply that the weight-credit idea originated with the Norwegians, by saying, "Initial use was for thermal insulation . . . , followed by use as lightweight fill for a highway embankment in Norway in 1972." Not mentioned, of course, is the intervening issuance of the patent to me. Most curious.

I can think of a number of reasons for the lack of response over the years, some having to do with legal (patent) rights and some relating to personal and corporate ethics, but I recognize that this closure is not the forum in which to expound on these matters at any length. It is hoped that the publication of this paper, especially the discussion and closure, will result in better communication between members of the profession toward the betterment of the profession and the public we serve.

Lightweight Aggregate Soil Mechanics: Properties and Applications

T. A. HOLM AND A. J. VALSANGKAR

Structural grade lightweight aggregates have been extensively used throughout North America for more than 70 years in cast-in-place structural lightweight concretes for high-rise buildings and bridges and are now being widely used for geotechnical applications. Structural grade lightweight aggregates, when used in backfills and over soft soils, provide geotechnical physical properties that include reduced density, high stability, high permeability, and high thermal resistance. These improved physical properties result from aggregates with a reduced specific gravity and a predictable stability that results from a consistently high angle of internal friction. The open texture available from a closely controlled manufactured aggregate gradation ensures high permeability. High thermal resistance results from porosity developed during the production process. Physical properties of structural grade lightweight aggregate and geotechnical engineering properties of lightweight aggregate backfills are illustrated, along with references to extensive testing programs that developed data on shear strength, compressibility, durability, and in-place density. Representative case studies are reported from the almost 100 projects that illustrate completed applications of structural grade lightweight aggregate fills over soft soils and behind retaining walls and bridge abutments.

For more than 70 years, shales, clays, and slates have been expanded in rotary kilns to produce structural grade lightweight aggregates for use in concrete and masonry units. Millions of tons of structural grade lightweight aggregate produced annually are used in structural concrete applications, with current availability widespread throughout North America and most of the industrially developed world. Consideration of structural grade lightweight aggregate as a remedy to geotechnical problems stems primarily from the improved physical properties of reduced dead weight, high internal stability, high permeability, and high thermal resistance. These significant advantages arise from the reduction in particle specific gravity, stability that results from the inherently high angle of internal friction, controlled open-textured gradation available from a manufactured aggregate that assures high permeability, and high thermal resistance developed because of the high particle porosity.

PHYSICAL PROPERTIES OF STRUCTURAL LIGHTWEIGHT AGGREGATES

Particle Shape and Gradation

As with naturally occurring granular materials, manufactured lightweight aggregates have particle shapes that vary from round to angular with a characteristically high interstitial void content that results from a narrow range of particle sizes. Applications of lightweight aggregate to geotechnical situations require recognition of two primary attributes: (a) the high interstitial void content typical of a closely controlled manufactured granular coarse aggregate that closely resembles a clean, crushed stone and (b) the high volume of pores enclosed within the cellular particle.

Structural grade lightweight aggregate gradations commonly used in high-rise concrete buildings and long-span concrete bridge decks conform to the requirements of ASTM C330. The narrow range of particle sizes ensures a high interstitial void content that approaches 50 percent in the loose state. North American rotary kiln plants producing expanded shales, clays, and slates currently supply coarse aggregates to ready-mix and precast concrete manufacturers with 20 to 5 mm (¾-#4), 13 to 5 mm (½-#4), or 10 to 2 mm (⅜-#8) gradations. With these gradations there is a minimum percentage of fines smaller than 2 mm (#8 mesh) and insignificant amounts passing the 100-mesh screen.

Particle Porosity and Bulk Density

When suitable shales, clays, and slates are heated in rotary kilns to temperatures in excess of 1100°C (2012°F), a cellular structure is formed of essentially noninterconnected spherical pores surrounded by a strong, durable ceramic matrix that has characteristics similar to those of vitrified clay brick. Oven-dry specific gravities of lightweight aggregate vary but commonly range from 1.25 to 1.40. Combination of this low specific gravity with high interparticle void content results in lightweight aggregate bulk dry densities commonly in the range of 720 kg/m^3 (45 pcf). Compaction of expanded aggregates in a manner similar to that used with crushed stone provides a highly stable interlocking network that will develop in-place moist densities of less than 1040 kg/m^3 (65 pcf).

Differences in porosity and bulk density between lightweight aggregates and ordinary soils may be illustrated by a series of schematic depictions. For comparative purposes, Figure 1 shows the interparticle voids in ordinary coarse aggregates. Although normal weight aggregates commonly have porosities of 1 to 2 percent, the schematic assumes ordinary aggregates to be 100 percent solid. For illustrative purposes, the bulk volume is shown to be broken into one entirely solid part with the remaining fraction being interparticle voids.

Figure 2 shows the cellular pore structure of lightweight aggregates. ASTM procedures prescribe measuring the "saturated" (misnamed in the case of lightweight aggregates; "par-

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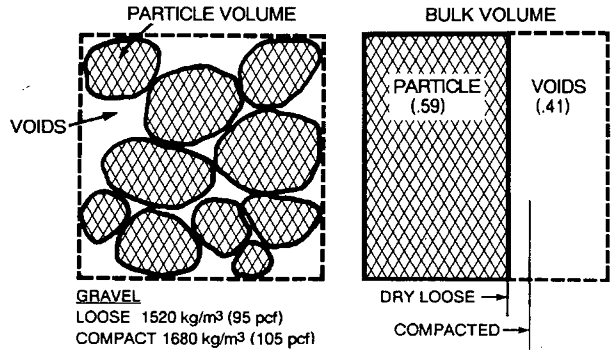


$$\text{PARTICLE VOLUME (LOOSE)} = \frac{1520\ \text{kg/m}^3}{2600\ \text{kg/m}^3} = .59 \qquad \text{PARTICLE VOLUME (COMPACTED)} = \frac{1680}{2600} = .65$$

FIGURE 1 Voids in ordinary coarse aggregates.

tially saturated after a 1-day soak" is more accurate) specific gravity in a pycnometer and then determining the moisture content on the sample that had been immersed in water for 24 hr. After a 1-day immersion in water, the rate of moisture absorption into the lightweight aggregate will be so low that the partially saturated specific gravity will be essentially unchanged during the time necessary to take weight measurements in the pycnometer. When the moisture content is known, the oven-dry specific gravity may be directly computed. This representative coarse lightweight aggregate with a measured dry loose bulk unit weight of 714 kg/m³ (44.6 pcf) and computed oven-dry specific gravity of 1.38 results in the aggregate particle occupying 52 percent of the total bulk volume, with the remaining 48 percent composed of interparticle voids.

The specific gravity of the pore-free ceramic solid fraction of a lightweight aggregate may be determined by standard procedures after porous particles have been thoroughly pulverized in a jaw mill. Pore-free ceramic solid specific gravities measured on several pulverized lightweight aggregate samples developed a mean value of 2.55. The representative lightweight aggregate with a dry specific gravity of 1.38 will develop a 54 percent fraction of enclosed aggregate particle ceramic solids and a remaining 46 percent pore volume (Figure 2).

This leads to the illustration of the overall porosity in a bulk loose lightweight aggregate sample as shown in Figure 3. Interparticle voids of the overall bulk sample are shown within the enclosed dotted area, and the solid pore-free ceramic and the internal pores are shown within the solid particle lines. For this representative lightweight aggregate, the dry loose bulk volume is shown to be composed of 48 percent voids, 28 percent solids, and 24 percent pores. Vacuum-saturated and submerged particle densities are also shown.

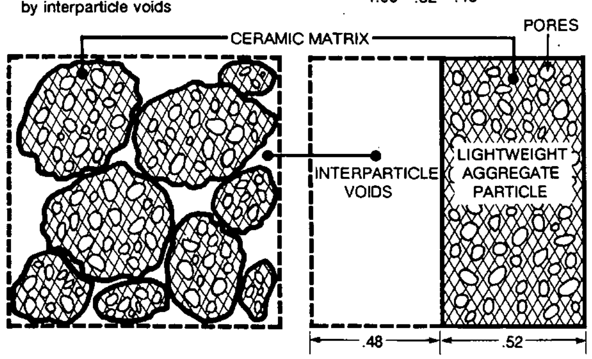
$$\gamma_D\ \text{(Dry Specific Gravity)} = \frac{\gamma_M\ \text{(Partially Saturated Surface Dry Specific Gravity)}}{(1+M)\ \text{(One Day Soak Moisture Content by Weight)}}$$

$$\gamma_D = \frac{1.50}{(1+.085)} = 1.38$$

Measured Bulk Dry Loose Density

Fraction of bulk aggregate sample occupied by lightweight aggregate particles $= \dfrac{714\ \text{kg/m}^3}{1380\ \text{kg/m}^3} = .52$

Fraction of bulk aggregate sample occupied by interparticle voids $= 1.00 - .52 = .48$



$$V_S \left(\begin{array}{l}\text{Fractional Part of Lightweight Aggregate}\\ \text{Particle Occupied by Ceramic Matrix}\end{array}\right) = \frac{\gamma_D\ \text{(Dry Specific Gravity)}}{\gamma_C \left(\begin{array}{l}\text{Dry Specific Gravity of}\\ \text{Pore Free Ceramic Matrix}\end{array}\right)}$$

$$V_S = \frac{1380\ \text{kg/m}^3}{2550\ \text{kg/m}^3} = .54,\ \text{then } V_{pores} = 1 - .54 = .46$$

FIGURE 2 Interparticle voids and within-particle pores of lightweight aggregate.

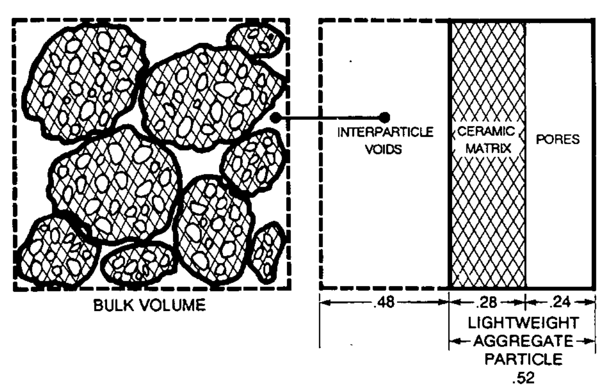
Absorption Characteristics

Lightweight aggregates stored in exposed stockpiles in a manner similar to crushed stone will have some internal pores partially filled and may also carry an adsorbed moisture film on the surface of the particles. The moisture content that is defined in ASTM procedures as "absorption" based on a 24-hr immersion and routinely associated in concrete technology with "saturated" surface-dry specific gravity is, in fact, a condition in which considerably less than 50 percent of the particle pore volume is filled.

This issue is further clarified by a schematic volumetric depiction (see Figure 4) of the degree of pore volume satu-

VOLUMETRIC FRACTIONS IN DRY LOOSE LIGHTWEIGHT AGGREGATE SAMPLE

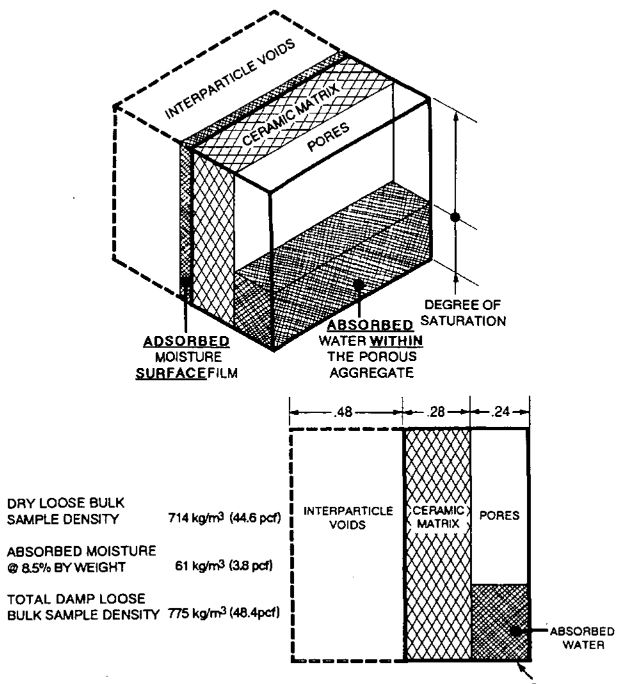
VOIDS		= .48
CERAMIC SOLIDS FRACTION	= .52 X .54	= .28
LIGHTWEIGHT AGGREGATE PORES	= .52 X .46	= .24



LOOSE AGGREGATE CONDITION	INTERPARTICLE VOIDS	CERAMIC MATRIX	PORES	DENSITY kg/m3
DRY	—	714	—	714 (44.6 pcf)
PARTIALLY SATURATED ONE DAY SOAK	—	714	61	775 (48.4 pcf)
VACUUM SATURATION	—	714	240	954 (59.6 pcf)
LONGTIME SUBMERSION	480	714	240	1434 -1000 * 434 (27.1 pcf)

* Buoyant Unit Weight

FIGURE 3 Voids, pores, and ceramic matrix fraction in a lightweight aggregate sample.



Fraction of total bulk loose sample occupied by absorbed moisture $= \frac{61}{1000} = .06$

Degree of saturation of the lightweight aggregate pore volume by the sampled moisture content $= \frac{.06}{.24} = .25$

"Saturated", surface dry (SSD), after a one day immersion represents approximately one-quarter degree of saturation of the pores of the particular aggregate.

FIGURE 4 Degree of saturation of partially saturated lightweight aggregate.

ration of a lightweight aggregate particle that shows that the sample had a measured damp loose bulk unit weight of 785 kg/m^3 (49.1 pcf) with an 8.5 percent absorbed moisture and would, in fact, represent a condition in which approximately 25 percent of the pore volume is water filled.

Structural grade lightweight aggregate exposed to moisture in production plants and stored in open stockpiles will contain an equilibrium moisture content. Lightweight aggregates that are continuously submerged will, however, continue to absorb water with time. In one investigation, the effective specific gravity of a submerged lightweight aggregate sample was measured throughout a 1-year period to demonstrate long-term weight gain. Long-term absorption characteristics are shown in Figure 5 for a lightweight aggregate sample with a measured 1-day immersion moisture content of 8.5 percent associated with a partially saturated surface-dry specific gravity of 1.5. When moisture absorption–versus–time relationships are extrapolated or theoretical calculations used to estimate the total filling of all the lightweight aggregate pores, it can be shown that for this particular lightweight aggregate the absorbed moisture content at infinity will approach 34 percent by weight with a totally saturated specific gravity of 1.83. Complete filling of all pores in a structural grade lightweight aggregate is unlikely because the noninterconnected pores are enveloped by a very dense ceramic matrix. However, these calculations do reveal a conservative upper limit for submerged design considerations.

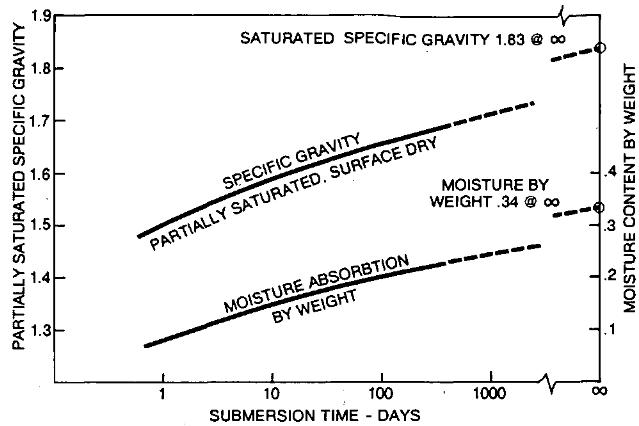


FIGURE 5 Moisture absorption (by weight) and partially saturated, surface dry specific gravity of lightweight aggregate versus time of submersion.

Durability Characteristics

The durability of lightweight aggregates used in structural concrete applications is well known. More than 400 major U.S. bridges built using structural lightweight concrete have demonstrated low maintenance and limited deterioration. Long-term durability characteristics of lightweight aggregates were demonstrated in 1991 by reclaiming and testing samples of the lightweight aggregate fill supplied in 1968 to a Hudson River site. Magnesium soundness tests conducted on the reclaimed aggregate sample exposed to long-term weathering resulted in soundness loss values comparable to those measured and reported in routine quality control testing procedures 23 years earlier, indicating little long-term deterioration due to continuous submersion and freeze-thaw cycling at the waterline.

Although ASTM standard specifications C330 and C331 for lightweight aggregate make no mention of corrosive chemicals limitations, foreign specifications strictly limit SO_3 equivalents to 0.5 percent (Japanese Industrial Standard J5002) or 1.0 percent (German Standard DIN 4226). The American Concrete Institute Building Code (ACI 318) mandates chloride limitation in the overall concrete mass because of concern for reinforcing bar corrosion, but no limits are specified for individual constituents. Numerous geotechnical project specifications calling for lightweight aggregates have limited water-soluble chloride content in the aggregate to be less than 100 ppm when measured by AASHTO T260.

GEOTECHNICAL PROPERTIES OF LIGHTWEIGHT AGGREGATE FILL

In-Place Compacted Moist Density

Results of compacted lightweight aggregate density tests conducted in accordance with laboratory procedures (Proctor tests) should be interpreted differently from those for natural soils. Two fundamental aspects of lightweight aggregate soil fill will modify the usual interpretation soils engineers place on Proctor test data. The first is that the absorption of lightweight aggregate is greater than natural soils. Part of the water added during the test will be absorbed within the aggregate particle and will not affect interparticle physics (bulking, lubrication of the surfaces, etc.). Second, unlike cohesive natural soils, structural grade lightweight aggregates contain limited fines, limiting the increase in density due to packing of the fines between large particles. The objective in compacting structural grade lightweight aggregate fill is not to aim for maximum in-place density, but to strive for an optimum density that provides high stability without unduly increasing compacted density. Optimum field density is commonly achieved by two to four passes of rubber tire equipment. Excessive particle degradation developed by steel-tracked rolling equipment should be avoided. Field density may be approximated in the laboratory by conducting a one-point ASTM D698, AASHTO T99 Proctor test on a representative lightweight aggregate sample that contains a moisture content typical of field delivery. Many projects have been successfully supplied where specifications called for an in-place, compacted, moist density not to exceed 960 kg/m^3 (60 pcf).

Shear Strength

Structural grade lightweight aggregates provide an essentially cohesionless, granular fill that develops stability from interparticle friction. Extensive testing on large size 250 × 600 mm (10 × 24 in. high) specimens has confirmed angles of internal friction of more than 40 degrees (*1*). Triaxial compression tests completed on lightweight aggregates from six production plants, which included variations in gradations, moisture content, and compaction levels, revealed consistently high angles of internal friction. With a commonly specified in-place moist compacted unit weight less than 960 kg/m^3 (60 pcf), it may be seen from a simplistic analysis that lateral pressures, overturning moments, and gravitational forces approach one-half of those generally associated with ordinary soils.

A summary of the extensive direct shear testing program conducted by Valsangkar and Holm (*2*), presented in the following table, confirmed the high angle of internal friction measured on large-scale triaxial compression testing procedures as reported earlier by Stoll and Holm (*1*).

	Angle of Internal Friction (degrees)	
Material	*Loose*	*Compact*
Minto	40.5	48.0
Solite	40.0	45.5
Limestone	37.0	N/A
Solite (*1*)	39.5	44.5

Compressibility

Large-scale compressibility tests completed on lightweight aggregate fills demonstrated that the curvature and slope of the lightweight aggregate fill stress-strain curves in confined compression were similar to those developed for companion limestone samples (*2*). Cyclic plate-bearing tests on lightweight fills indicated vertical subgrade reaction responses that were essentially similar for the lightweight and normal weight aggregate samples tested (*3*).

Attempts by concrete technologists to estimate aggregate strength characteristics by subjecting unbound lightweight aggregate samples to piston ram pressures in a confined steel cylinder have provided inconsistent and essentially unusable data for determination of the strength making characteristics of concretes that incorporate structural grade lightweight aggregates. By ASTM C330 specification, all structural grade lightweight aggregates are required to develop concrete strengths above 17.2 MPa (2500 psi). Most structural grade lightweight aggregate concrete will develop 34.4 MPa (5000 psi), and a small number can be used in concretes that develop compressive strength levels greater than 69 MPa (10,000 psi).

Thermal Resistance

For more than 7 decades, design professionals have used lightweight concrete masonry and lightweight structural concrete on building facades to reduce energy losses through exterior walls. It is well demonstrated that the thermal resistance of lightweight concrete is considerably less than ordinary concrete, and this relationship extends to aggregates in the loose state (*4*).

Permeability

Attempts to measure permeability characteristics of unbound lightweight aggregates have not been informative because of the inability to measure the essentially unrestricted high flow rate of water moving through the open-graded structure. This characteristic has also been observed in the field, where large volumes of water have been shown to flow through lightweight aggregate drainage systems. Exfiltration applications of lightweight aggregate have demonstrated a proven capacity to effectively handle high volumes of storm water runoff. Subterranean exfiltration systems have provided competitive alternatives to infiltration ponds by not using valuable property areas as well as eliminating the long-term maintenance problems associated with open storage of water.

Interaction Between Lightweight Aggregate Fills and Geotextiles

Valsangkar and Holm (*5*) reported results of testing programs on the interaction between geotextiles and lightweight aggregate fills that included the variables of differing aggregate types and densities, thickness of aggregate layer, and geotextile types. The results indicated that the overall roadbed stiffness is unaffected when lightweight aggregate is used instead of normal-weight aggregate for small deflections and initial load applications. These tests were followed by a large-scale test (*2*), which reported that the comparison of the friction angles between the lightweight aggregate or the normal weight aggregate and the geotextiles indicate that interface friction characteristics are, in general, better for lightweight than normal weight aggregates.

APPLICATIONS

During the past decade, almost 100 diverse geotechnical applications have been successfully supplied with structural grade lightweight aggregate. These applications primarily fit into the following major categories:

- Backfill behind waterfront structures, retaining walls, and bridge abutments;
- Load compensation and buried pipe applications on soft soils;
- Improved slope stability situations; and
- High thermal resistance applications.

Backfill Behind Waterfront Structures, Retaining Walls, and Bridge Abutments

A classic example of how an unusable riverfront was reclaimed and a large industrial site extended by the use of sheet piles and lightweight fill is demonstrated in Figure 6 (*6*). Lightweight aggregate fill specifications for this project required rotary kiln expanded shale to have a controlled coarse aggregate gradation of 20 to 5 mm (¾-#4) and laboratory test certification of an angle of internal friction greater than 40 degrees. No constructability problems were experienced by the contractor while transporting, placing, and compacting the lightweight aggregate soil fill. Peak shipments were more

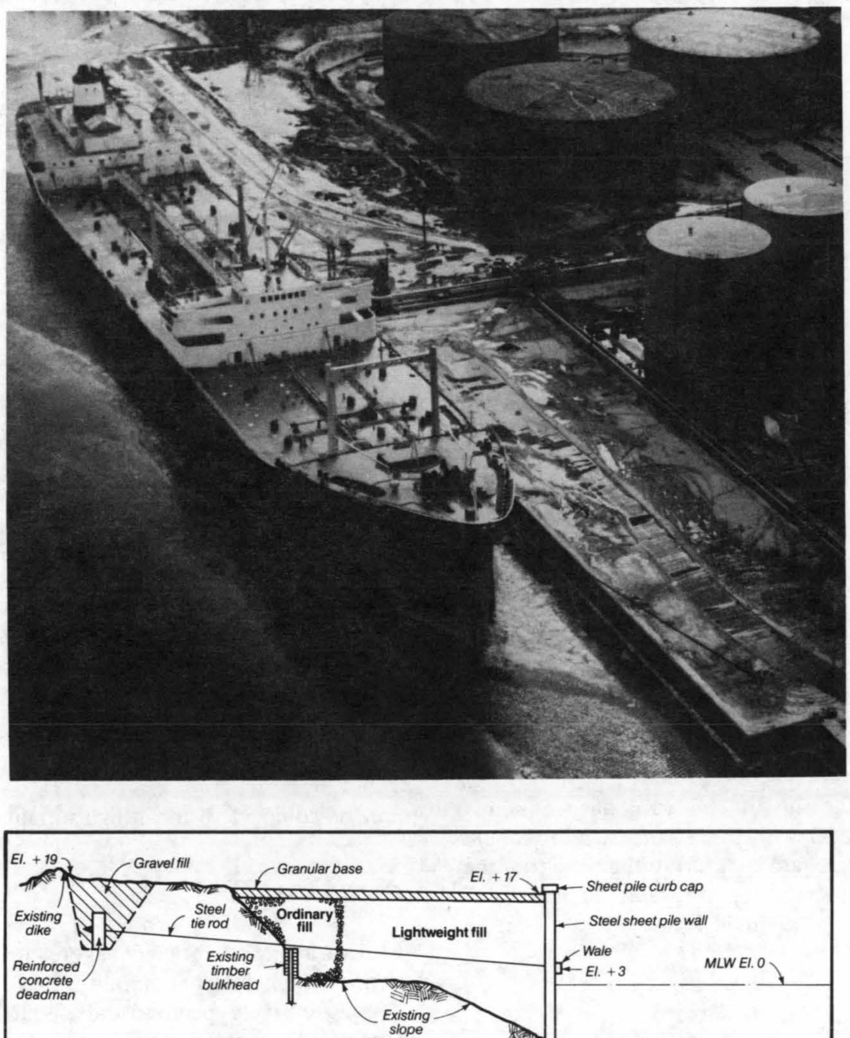


FIGURE 6 Rehabilitation of port of Albany, N.Y., marine terminal.

than 1,000 tons per day without any logistical difficulties. The material was trucked to the point of deposit at the job site and distributed by front-end loaders. This project used approximately 20 000 m^3 (27,000 yd^3) of compacted lightweight aggregate and resulted in overall savings by reducing sizes of sheet piling and lowering costs associated with the anchor system.

On the Charter Oak Bridge project, Hartford, Connecticut, constructed in 1989 to 1990, lightweight aggregate fill was placed in the east abutment area to avoid placing a berm that would have been necessary to stabilize an earth fill embankment. According to the designer, construction of a berm would have required relocating a tributary river. Lightweight fill was also used in other areas to avoid increasing stresses and settlements in an old brick sewer (7). When all applications were totaled, this project incorporated more than 100,000 tons of structural grade lightweight aggregate.

Load Compensation and Buried Pipe Applications on Soft Soils

In numerous locations throughout North America, design of pavements resting on soft soils has been facilitated by a "load compensation" replacement of heavy soils with a free-draining structural grade lightweight aggregate with low density and high stability. Replacing existing heavy soil with lightweight aggregate permits raising elevations to necessary levels without providing any further surcharge loads to the lower-level soft soils. Rehabilitation of Colonial Parkway near Williamsburg, Virginia, built alongside the James and York rivers, provides a representative example of this procedure. Soft marsh soil sections of this roadway having a low load-bearing capacity had experienced continuous settlement. The concrete roadway slabs were removed along with the soil beneath it to a depth of more than 3 ft. The normal weight soil was then replaced with structural-grade lightweight aggregate with a compacted moist density of less than 960 kg/m^3 (60 pcf), providing effective distribution of load to the soft soil layer, load compensation, and side slope stability. Reconstruction was completed in two stages by first completely rehabilitating in one direction, followed by excavation of the opposing lane with delivery, compaction, and slab construction routinely repeated.

Construction of pipelines in soft soil areas has frequently been facilitated by equalizing the new construction weight (pipe plus lightweight aggregate backfill) to the weight of the excavated natural soil. Supporting substrates do not "see" any increased loading and settlement forces are minimized.

Improved Slope Stability

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

High Thermal Resistance Application

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

ECONOMICS

An economic solution provided by a design that calls for an expensive aggregate requires brief elaboration. In many geographical areas, structural-grade lightweight aggregates are sold to ready-mix, precast, and concrete masonry producers on the basis of the price per ton, FOB the plant. On the other hand, the contractor responsible for the construction of the project bases costs on the compacted material necessary to fill a prescribed volume. Because of the significantly lower bulk density, a fixed weight of this material will obviously provide a greater volume. To illustrate that point, one may presume that if a lightweight aggregate is, for example, available at \$$X$/ton, FOB the production plant, and trucking costs to the project location call for additional \$$Y$/ton, the delivered job site cost will be a total of \$$(X + Y)$/ton. As mentioned previously, many projects have been supplied by structural lightweight aggregates delivered with a moist, loose density of about 720 kg/m^3 (45 pcf) and compacted to a moist, in-place density of approximately 960 kg/m^3 (60 pcf). This would result in an in-place, compacted moist density material cost (not including compaction costs) of

$$[\$(X + Y) \times 60 \times 27]/2{,}000$$

for the compacted, moist lightweight aggregate.

CONCLUSIONS

Structural grade lightweight aggregate fills possessing reduced density, high internal stability, and high permeability have been extensively specified and used to replace gravel, crushed stone, and natural soils for geotechnical applications at soft soil sites and in backfills where the assured reduction in lateral and gravitational forces has provided economical solutions.

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Cyclic Plate Load Tests on Lightweight Aggregate Beds

A. J. VALSANGKAR AND T. A. HOLM

In recent years lightweight aggregates have been used increasingly with or without polymeric reinforcement in geotechnical applications. Results of a series of plate load tests performed on beds of expanded shale lightweight aggregate with or without geogrid reinforcement are presented. All tests were performed in a large test facility so that lightweight aggregate beds could be prepared using light compaction equipment. The relative density of the aggregate and locations of the polymeric reinforcement with respect to the base of the plate were varied in the experimental program.

The present testing program is part of an ongoing research project to determine the geotechnical properties of expanded shale lightweight aggregate at the University of New Brunswick, Canada. The research program began in 1985, and initially large-size one-dimensional compression and direct shear tests were carried out on lightweight aggregate specimens (*1*). The large direct shear apparatus was also used for determining angle of friction between geotextiles and expanded shale lightweight aggregate (*1*). Model footing tests on peat-geotextile-lightweight aggregate systems were undertaken following the direct shear and compression testing. Some of the results of this model testing have been reported by Valsangkar and Holm (*2*).

The scope of the testing program reported in this paper was to carry out preliminary laboratory plate load tests on beds of lightweight aggregate with or without geogrid reinforcement. The variables studied were relative density of the aggregate and location of the geogrid with respect to the base of the plate.

MATERIALS

Expanded shale aggregate manufactured by Solite Corporation was used in this study. This aggregate is manufactured by heating shale in a rotary kiln at a temperature of about 1150°C. At this temperature the shale particles reach a pyroplastic condition and expand through formation of gases that result from the decomposition of some of the compounds. The expanded, vitrified particles are screened to produce the desired gradation for a particular application. In the geotechnical applications, coarse aggregates with particle sizes between 5 and 25 mm are commonly used.

The lightweight aggregate used in the present study has a grain size distribution from between 19 and 4.7 mm with a uniformity coefficient of 1.4. Table 1 gives the shear strength data for the lightweight aggregates from two sources, along with the data for limestone aggregate.

The polymeric reinforcement used in the testing was a low-strength HDPE geogrid (Tensar SR-1). The properties of this geogrid as reported in Koerner (*3*) are shown in Table 2. The critical properties of the geogrid for its use as a soil reinforcement are aperture size in relation to particle size of the soil, long-term design load, tensile modulus at low strain levels, and service life of the grid (*3*).

EQUIPMENT AND PROCEDURE

Plate load tests were performed in a test pit 3.2 × 3.2 × 1.6 m deep. The facility is equipped with loading frames, and the reaction beam can be adjusted in the vertical position depending on the thickness of the soil in the test pit. The schematic details of the test setup are shown in Figure 1. A standard steel plate 300 mm in diameter was used in all the tests. The loads were applied by a hydraulic ram, and the settlements were monitored using two dial gauges. The data from the dial gauges and the level vial mounted on the plate were used to ensure that plate tilting did not occur during testing.

In all the tests performed, the thickness of the lightweight aggregate was at least 900 mm. Loose relative density was achieved by end dumping the aggregate in the test pit. An average dry density of 800 kg/m^3 was achieved when the aggregate bed was prepared by end dumping.

After completion of testing of the loose lightweight aggregate, the aggregate was removed from the test pit. A small vibratory plate compactor (530- × 610-mm plate) was then used to compact 150-mm-thick lifts of lightweight aggregate. Density measurements made after compaction indicated that an average dry density of 950 kg/m^3 was achieved.

Polymeric reinforcement was used in combination with compacted aggregate. In one series the geogrid was located 150 mm below the bottom of the plate, and in the second series, at a depth of 200 mm. The location of geogrid below plate was selected on the basis of previous research, which concluded that for one layer of soil reinforcement to be effective, it has to be placed within a depth equal to or less than the width of the footing (*4*).

When the plate was properly seated, load was applied with the hydraulic ram. For loose aggregate beds, the loads were monotonically applied in increments of 1 kN until a settlement of 12 mm was achieved. For the compacted aggregate bed, monotonically increasing loads were applied in increments of about 2 to 3 kN until the plate settlement reached 12 mm.

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TABLE 1 Angle of Internal Friction for Coarse Aggregates (*1*)

	Dry Density kg/m³		Angle of friction, degrees	
Material	Loose	Compact	Loose	Compact
Solite	840	934	40.0	45.5
Minto[a]	929	1,062	40.5	48.0
Limestone	1,706	1,887	37.0	---

[a] Minto expanded shale lightweight aggregate has the same gradation as Solite.
--- Unavailable

TABLE 2 Properties of SR-1 Uniaxial Geogrid (UX1400) (*3*)

Property		Value	
Structure		Punched-sheet drawn	
Polymer composition		Polyethylene	
Mass/unit area		512 g/m² ASTM D3776-84	
Aperture size:			
	Machine direction	145 mm	
	Cross machine direction	15 mm	
Thickness:			
	at rib	0.8 mm	ASTM D1777-64
	at junction	2.8 mm	ASTM d1777-64
Wide width strip tensile:			
	2% strain	14.6 kN/m	
	5% strain	24.8 kN/m	
	ultimate	54.0 kN/m	

Load increments for reinforced aggregate varied from 4 to 6 kN during the monotonic application of loads. Irrespective of the magnitude of the load increment, each load increment was maintained until the rate of settlement was less than 0.02 mm/min for a minimum of three successive minutes.

The choice of 12-mm settlement as the maximum settlement was adopted on the basis of the ASTM standard for plate load testing (ASTM D1195-64). However, load cycling before reaching 12-mm settlement was not carried out as recommended in ASTM D1195-65, because the primary objective of the study was to determine the coefficient of subgrade reaction for monotonic loading. The other reason for adopting the 12-mm settlement criterion and not cycling the load before this much settlement occurred is found in the work by DeBeer (*5*), which concluded that the settlement at the onset of bearing capacity failure of granular soils with high relative density is on the order of 5 percent of the width of the loaded area.

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

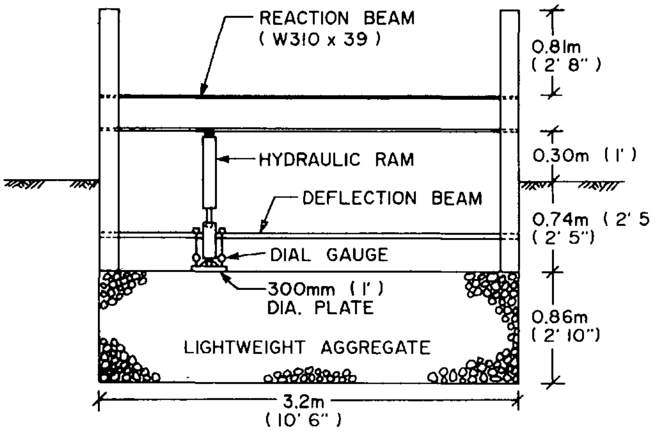

FIGURE 1 Test setup.

RESULTS

Plate load test results for unreinforced lightweight aggregate are presented in Figure 2 for compact and loose beds. The bearing stress for 12-mm settlement increased from 116 kPa to 456 kPa because of moderate compaction. The values of coefficient of vertical subgrade reaction were determined from the slope of the bearing stress–versus–settlement data obtained during the monotonic loading. The results are given in Table 3. Typically, values of coefficient of vertical subgrade reaction of 8 MN/m³ (loose) and 38 MN/m³ (compact) are used for normal-weight coarse-grained soils (*6*). Thus, the plate loading tests confirm that the behavior of tested coarse

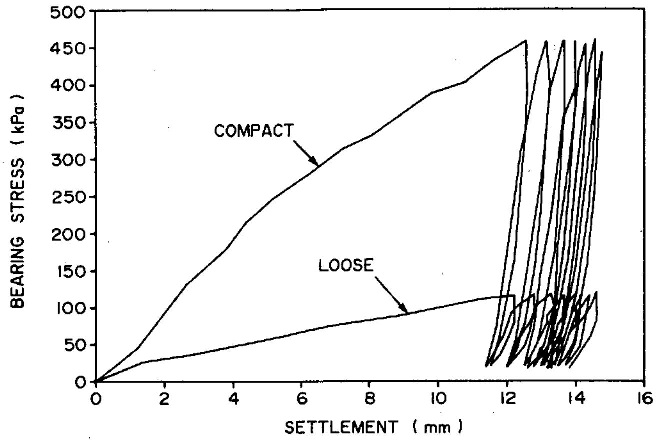

FIGURE 2 Effect of relative density on plate settlements.

lightweight aggregate is similar to that of normal-weight aggregates.

The effect of cyclic loading on plate settlements is given in Figures 2 and 3. From Figure 2 it is seen that the slopes of the unloading and reloading curves are very steep when compared with the slope of the bearing stress–versus–settlement data during initial monotonic loading. The reloading coefficient of subgrade reaction for loose and compact aggregate beds is evaluated to be 190 and 1500 MN/m^3, respectively.

Figure 3 shows the effect of repetition of loading on the cumulative settlements for both loose and compact lightweight aggregate beds. Note that the linear trend observed between number of load cycles plotted on the logarithmic scale and cumulative settlement on natural scale, which is common for coarse-grained normal-weight soils (*7*), is also applicable to lightweight soils.

The beneficial effect of including geogrid reinforcement in compacted lightweight aggregate is seen from the data given in Figure 4. The bearing stress to cause 12-mm plate settlement increased from 456 to 1000 kPa, irrespective of whether the geogrid was located 150 or 200 mm below the base of the plate. The coefficient of vertical subgrade reaction due to the inclusion of geogrid reinforcement increased from 42 to 130 MN/m^3.

Figure 5 gives the effect of cyclic loading on the cumulative settlements. Again a linear trend is observed between the magnitude of settlement and number of cycles plotted on the logarithmic scale. Also, it is seen that the cumulative settlements observed for aggregate with geogrid reinforcement of 150 mm deep were somewhat lower than when the geogrid was at a depth of 200 mm (Figure 5). However, more testing is required to delineate this trend.

TABLE 3 Coefficient of Vertical Subgrade Reaction for Coarse Lightweight Aggregate

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

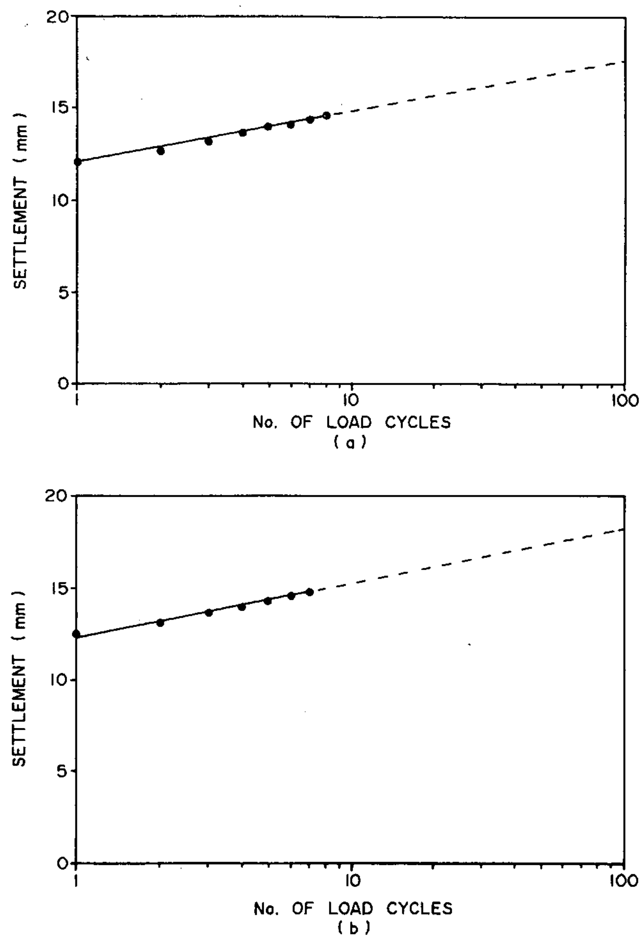

FIGURE 3 Cumulative settlements due to cyclic loading: *top*, loose, bearing stress = 116 kPa; *bottom*, compact, bearing stress = 456 kPa.

CONCLUSIONS

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

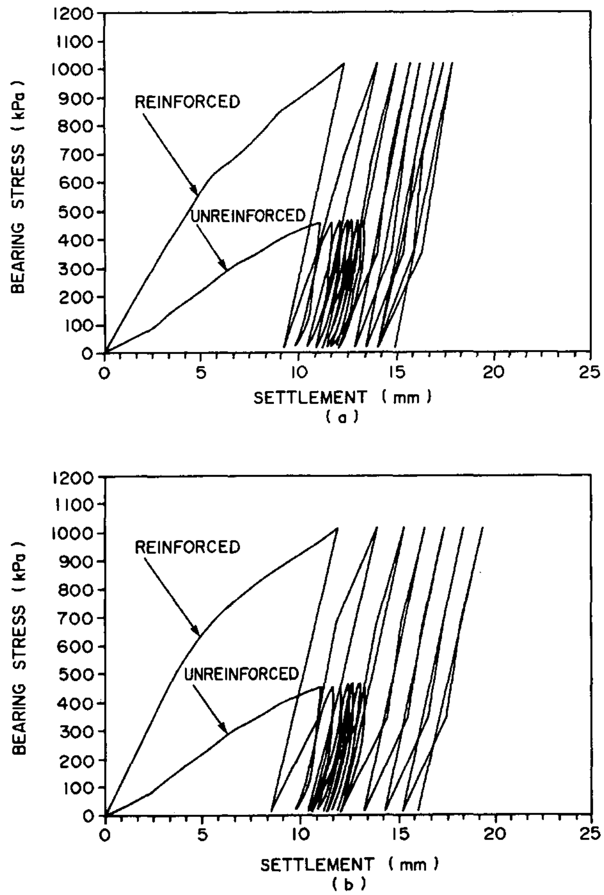

FIGURE 4 Effect of geogrid reinforcement on plate settlement response: *top*, geogrid at 150-mm depth; *bottom*, geogrid at 200-mm depth.

ACKNOWLEDGMENTS

The experimental work reported in this paper was done by undergraduate students R. S. Gallagher, I. Page, A. MacKenzie, and P. Mawhiney. Their efforts, and the assistance of the authors' technical staff, are greatly appreciated.

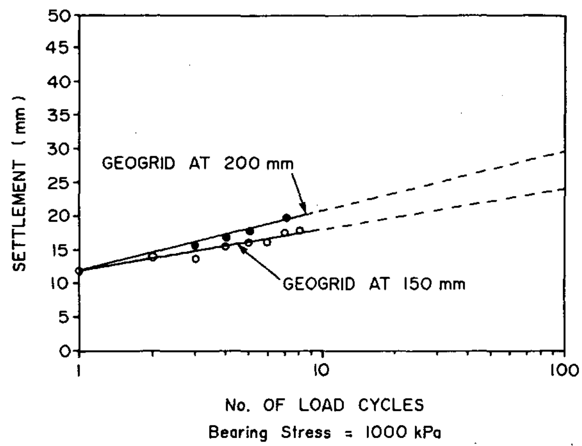

FIGURE 5 Cumulative settlements due to cyclic loading for geogrid-reinforced aggregate.

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Lightweight Fill Solutions to Settlement and Stability Problems on Charter Oak Bridge Project, Hartford, Connecticut

JOHN P. DUGAN, JR.

Design and construction of the Charter Oak Bridge and approaches over soft soils were complex and challenging. To solve settlement and stability problems arising from highway and bridge construction over deep deposits of soft varved clay in the Connecticut River valley the following applications of lightweight fill were made. Lightweight fill was placed for the high approach fill for the east abutment. The reduced stresses imposed in the clay layer, combined with the lightweight fill's higher shear strength compared with that of an earth fill, solved this embankment stability problem. Lightweight fill was placed in approach embankments for a replacement bridge to reduce settlements of the adjacent existing bridge. To avoid minor settlements to an aging sanitary sewer that crossed the west approach, soil above the sewer was replaced with lightweight fill. The resulting stress reduction balanced effects of additional stresses imposed by nearby fills and pile driving. The overall slope stability of a wharf, with an anchored sheet pile bulkhead, was improved by replacing existing soil with a 1.5-m (5-ft) layer of lightweight fill.

This paper summarizes applications of lightweight fill (expanded shale) to solve settlement and stability problems arising from highway and bridge construction over deep deposits of soft varved clay in the Connecticut River valley.

More than 61 200 m³ (80,000 yd³) of lightweight fill was placed for the 14.0-m (46-ft)-high east approach fill. The reduced stresses imposed in the clay layer, combined with the lightweight fill's higher shear strength compared with that of an earth fill, solved the embankment stability problem. Lightweight fill was placed in approach embankments for a replacement bridge to reduce settlements of the adjacent existing bridge.

To avoid even minor settlements to an aging, 2.0-m (6.5-ft)-diameter sanitary sewer that crossed the west approach, soil above the sewer was excavated and replaced with lightweight fill. The resulting stress reduction balanced effects of additional stresses imposed by nearby fills and pile driving.

The overall slope stability of a wharf, with an anchored sheet pile bulkhead, was improved by replacing existing soil with a 1.5-m (5-ft) layer of lightweight fill.

PROJECT DESCRIPTION

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

Haley & Aldrich, Inc., 110 National Drive, Glastonbury, Conn. 06033.

LIGHTWEIGHT FILL

Lightweight fill was expanded shale aggregate produced by expanding shale, clay, or slate by heating in a rotary kiln to approximately 1149°C (2,100°F). The expanded, vitrified mass was then screened to produce the desired gradation. The pores formed during expansion are generally noninterconnecting. The particles are subgranular, durable, chemically inert, and insensitive to moisture.

For this project, the following gradation was specified

Square Mesh Sieve Size	*Percent Passing by Weight*
25.4 mm (1 in.)	100
19.0 mm (¾ in.)	80–100
9.5 mm (⅜ in.)	10–50
No. 4	0–15

For design, a unit weight of 961 kg/m³ (60 lb/ft³) and an angle of internal friction of 40 degrees were used.

The lightweight fill was placed in 0.61-m (2-ft)-thick lifts and compacted with four passes of a relatively light 4.5-Mg (5-ton) vibratory roller operating in vibratory mode. The compaction effort was designed to prevent overcompaction, which could result in breakdown of particles leading to a more well-graded material with higher-than-desirable unit weight.

SUBSURFACE CONDITIONS

The site is in the floodplain of the Connecticut River. Subsurface conditions, in the order of increased depth, are

- Existing fill, (a) random fill [1.5 m (5 ft) to more than 4.6 m (15 ft) thick] containing man-made and discarded organic material and (b) roadway fill that is relatively free of nonmineral material.
- Alluvial sand and silt stratum consisting of floodplain and channel deposits 9.1 to 12.2 m (30 to 40 ft) thick.
- Very soft to soft, varved clay and silty clay, in regular layers 6.3 to 12.7 mm (¼ to ½ in) thick, [more than 25.4 mm

(1 in) thick at some locations], deposited in glacial Lake Hitchcock during the Pleistocene epoch. These deposits are approximately 10.7 m (35 ft) thick on the west side and from about 27.5 to 45.8 m (90 to 150 ft) thick on the east side of the river. Compressibility, stress history, and undrained shear strength data are given in Table 1. For other engineering properties, see work by Smith (*1*).

- Glacial till stratum consisting of dense to very dense sandy silt with subordinate coarse to fine gravel, clay, and occasional cobbles.
- Groundwater levels within the alluvial sand and silt and approximately 1.5 m (5 ft) above normal level in the Connecticut River.

EMBANKMENT STABILIZATION

If constructed of earthen material 2,002 kg/m^3 (125 lb/ft^3), the maximum 14.0-m (46-ft)-high embankment for the Charter Oak Bridge's east approach would not have an acceptable safety factor against slope instability. The safety factor against slope failure toward the adjacent Hockanum River, using earth fill, was estimated to be only 1.0 to 1.1 (Figure 1).

Many stabilization alternatives were considered. A toe berm placed in the river was the most economical but rejected to avoid delays that would occur because of time required to obtain environmental permits. Therefore, it was decided to construct the embankment of lightweight fill. The 62 730 m^3 (82,000 yd^3) of lightweight fill is one of the largest quantities of lightweight fill placed for one project in the United States.

Lightweight fill significantly reduced stresses in the weak varved clay. Even so, it was necessary to excavate a portion of the approach fill to the existing bridge to provide the design safety factor of 1.25. The lightweight fill's 40 degree angle of internal friction was higher than provided by earth fill, which increased resisting forces along the potential failure plane.

TABLE 1 Compressibility and Strength Parameters for Varved Clay at East Abutment

The clay is overconsolidated by at least 3.5 KPa (3.5 kips/ft^2) at all depths.		
Compression Ratio		
Virgin compression	0.31 to 0.37	
Recompression	0.03	
Coefficient of Consolidation		
Normally consolidated	0.0004 cm^2/sec (0.04 ft.^{2}/day)	
Overconsolidated	0.0037 cm^2/sec (0.37 ft.^{2}/day)	
Coefficient of Secondary Compression		
El. 0 to -30	1.06% per log cycle time	
El. -31 to -60	0.87% per log cycle time	
Below El. -60	0.98% per log cycle time	
Coefficient of Horizontal Permeability / Coefficient of Vertical Permeability =5		
Shear Strength, $s_u=S(OCR)^m\sigma_v$		
	S	m
Undrained	0.19	0.7
Plane Strain Compression	0.21	0.8
Plane Strain Extension	0.20	0.75
Direct Simple Shear	0.14	0.7

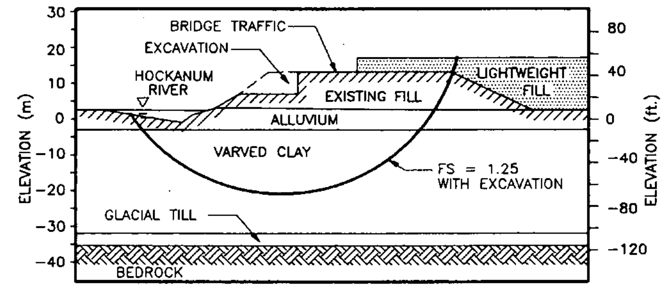

FIGURE 1 Slope stability for east abutment. Final conditions with lightweight fill.

Another benefit of the lightweight fill was the significantly reduced settlement, compared with an earth fill. The total settlement, over the first 15 years, of a lightweight fill embankment was predicted to range from 0.43 to 0.64 m (1.4 to 2.1 ft), compared with estimates of up to 1.98 m (6.5 ft) for earth fill. Observed settlement at the east abutment over a year is in line with the predicted values. Hence, the surcharge fill and vertical drains that were planned to speed consolidation of an earth fill were unnecessary. Nevertheless, the lightweight fill technique cost an additional $2 million in construction compared with the more conventional earth fill/berm/surcharge design.

SETTLEMENT REDUCTION AT EXISTING BRIDGE

A part of the overall project was replacement of Route 15 over Main Street in East Hartford, Connecticut, with a new bridge—a single-span structure 55.8 m (183 ft) wide, at the existing bridge, but extending 21.4 m (70 ft) north and 7.6 m (25 ft) south. Plans called for stage construction, with traffic maintained on the existing bridge while the north section of the new bridge was built. Then traffic was carried entirely on the north half of the new bridge while the existing bridge was being demolished and the south half of the bridge being built. Lightweight fill made it possible to keep the existing bridge in service while the north portion of the new bridge was being built and to avoid more expensive alternatives to prevent settlement.

The existing bridge is supported on spread footings bearing on a sand layer over approximately 42.7 m (140 ft) of soft varved clay. A recent inspection had reported 7.6 cm (3 in.) settlement of the west abutment and rotation and horizontal movements of both abutments of the single-span bridge. Temporary corrective repairs were planned; however, there was little tolerance for additional deflections.

Although the new bridge was designed to be supported on deep end-bearing piles, the 7.6-m (25-ft)-high approach fills would increase stresses and lead to settlements in the clay beneath the existing bridge. If an earthen embankment was used, predicted bridge settlements ranged from 1.3 to 5.1 cm (½ to 2 in.), which were considered intolerable. The project was therefore designed using lightweight fill for portions of the approach embankments within 22.9 m (75 ft) of the existing bridge. The lightweight fill reduced stress increases in the clay, lowering predicted settlements of the existing bridge to tolerable limits, to approximately half the magnitudes for earth fill. Measured settlements of the two bridge abutments, during the 1½-year period between embankment placement and demolition of the bridge, were 0.16 cm (¾ in.) and 0.22 cm (1 in.), which are within the range expected for the lightweight fill.

The lightweight fill option was significantly less expensive than underpinning the existing bridge and lengthening the new bridge to provide greater distance between the approach fills and the existing structure.

SETTLEMENT PREVENTION AT EXISTING SEWER

A 2.0-m (6.5-ft)-diameter sewer crosses the existing and new bridge alignments between the west abutment and Pier 1. This 60-year-old cast-in-place concrete pipe founded in the loose silty alluvium is underlain by varved clay (Figure 2). Preload fill for construction of the bridge, adjacent pile driving, and new alignment of I-91 northbound required up to 6.1 m (20 ft) of fill over the sewer and would cause settlements in the varved clay and unacceptable movements in this old pipe.

The most severe settlement problem was solved by designing a pile-supported bridge to carry I-91 over the sewer pipe. Nevertheless, stress increases in the clay from the adjacent approach fills and the effects of pile driving were estimated to cause 2.5 to 5.1 cm (1 to 2 in.) of settlement beneath the pipe. To prevent pipe settlement, 1.5 m (5 ft) of alluvium from above the pipe was replaced with lightweight fill. This decreased the effective stress in the clay below the pipe by approximately 300 P (300 lb/ft^2) and counteracted settlement

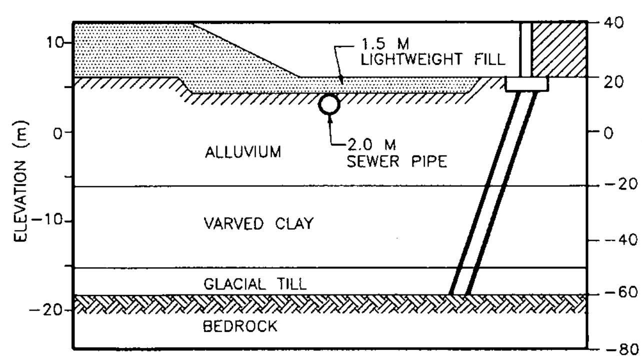

FIGURE 2 Lightweight fill above MDC sewer pipe.

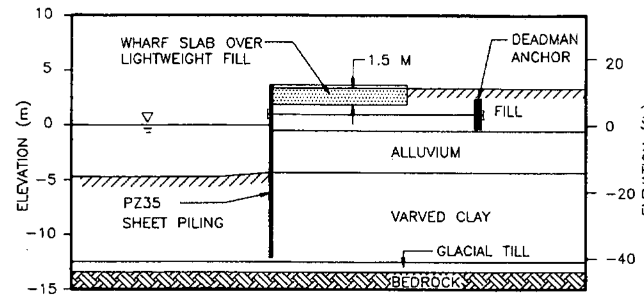

FIGURE 3 Lightweight fill placed to improve stability for wharf's sheet pile bulkhead.

effects from the other sources. No significant pipe settlement was measured.

WHARF STABILIZATION

The project included construction of a wharf and boat launch ramp along the west shore of the Connecticut River south of the Charter Oak Bridge. Lightweight fill was designed to provide stability for the wharf's anchored sheet pile bulkhead.

The bulkhead retains 7.6 m (25 ft) of soil above dredge level in the river (Figure 3). Stability analyses of circular failure surfaces indicated an unacceptably low factor of safety. As an alternative to anchoring a stiffer wall into underlying bedrock, a layer of lightweight fill was designed to reduce stresses in the weak varved clay and alluvium deposits and increase the factor of safety for overall slope stability to 1.25. The design called for replacing existing soil with a 1.5-m (5-ft) thickness of lightweight fill. The 0.2-m (8-in.)-thick reinforced concrete wharf slab was placed on a 0.3-m (12-in.)-thick layer of compacted gravel fill over the lightweight fill.

CLOSING

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

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Lightweight Foamed Concrete Fill

DEBRA I. HARBUCK

Since 1981 the New York State Department of Transportation (NYSDOT) has used lightweight foamed concrete fill (LFCF) to reduce loads on clayey and organic soils that are weak and highly compressible. LFCF is also used to reduce lateral loads on abutments and retaining walls. To date, it has been used successfully in place of conventional fill on seven projects involving 12 placement areas: 10 permanent bridges, 1 temporary detour bridge, and 1 set of existing retaining walls. LFCF, typical applications, and placement, as well as quality assurance and testing, are described. NYSDOT specifications, design considerations, and a comparison with another lightweight fill are discussed, and NYSDOT's experiences with LFCF and its performance are summarized.

New York State soils are complex and variable. Of greatest concern are clayey and organic soils, which are weak and highly compressible and may result in differential settlement or embankment foundation instability, or both. Weakness and compressibility of embankment foundation soils can also induce drag on pile foundations and intolerable lateral loads on abutments and retaining walls. New York uses lightweight foamed concrete fill (LFCF) to minimize or eliminate these geotechnical issues.

LFCF is a low-density cellular concrete consisting of a portland cement matrix containing uniformly distributed, noninterconnected air voids (Figure 1). These are introduced into the matrix by a foaming agent, facilitating development of wet-cast densities ranging from 288 to 1280 kg/m³ (18 to 80 pcf) and corresponding 28-day compressive strengths from 69 to 2067 kPa (10 to 300 psi).

At present, two suppliers have submitted product information and samples containing their foaming agent to the New York State Department of Transportation (NYSDOT) Materials Bureau for evaluation and approval. (These foaming agents are Elastizell Concentrate, supplied by Elastizell Corporation of America, Ann Arbor, Michigan, and Mearl Geofoam Liquid, supplied by Mearl Corporation of Roselle Park, New Jersey.) New York's experiences are limited to use of these two products.

TYPICAL APPLICATIONS

NYSDOT uses this fill to prevent increased loads on embankment foundations. This is based on the concept of "balanced" excavations (*1*). By removing a quantity of existing fill or natural material and replacing it with no more than an equal weight of lighter fill to the required grade line, no additional load is applied to the foundation soil. For example, if 0.3 m (1 ft) of existing material with a density of 1920 kg/m³ (120 pcf) is excavated, 0.9 m (3 ft) of lightweight fill with a density of 640 kg/m³ (40 pcf) can be placed without inducing any additional loads on the foundation soils.

LFCF is also used as a backfill to prevent increased lateral loads on existing abutments and retaining walls. In some placements, a denser LFCF layer is used as a footing base. Some placements also involve a dense top lift on which a reinforced concrete pavement is directly placed.

PLACEMENT

In preparation for placing LFCF, forms are positioned as needed around the perimeter of the placement area. The type of form used depends on the contractor's experience with the product and the job site restrictions. The formwork often consists of nothing more than sheets of plywood leaning against stakes that have been tapped into the ground or a previously placed lift of LFCF fill. In many instances, the placement perimeter is bounded by a structure, such as an abutment or retaining wall, or by the excavation. Consequently, the only form required would be at the open end of the excavated area. If the placement area is large, the contractor will occasionally separate it into smaller areas by using temporary interior forms (Figure 2).

Preparation of the fill requires the following equipment (Figure 3): a unit to dilute and mix the foaming agent, a mixing/calibrating unit, a cement truck with a hopper to measure the cement, and a water tanker (if a local source is not available).

The process begins by measuring the foaming agent (usually based on experience), placing it in a dilution chamber, adding water, and mixing. The resulting foam is then routed to a mixing/calibrating unit, where a measured amount of cement is added. The fill is then pumped through a hose to the placement area. At this stage, the fill is sampled at the point of placement by the on-site supplier's representative and a NYSDOT inspector to ensure conformity to the required maximum wet-cast density. If necessary, proportions are adjusted.

Fill placement is limited to lifts of no more than 0.6 m (2 ft) for two reasons

1. Typically, the worker places the fill by laying the hose on the ground and slowly shuffling through the puddling fill to minimize voids next to structures or formwork (Figure 4). Limiting placement depth to 0.6 m (2 ft) makes this easier.
2. With depths greater than 0.6 m (2 ft), excessive heat of hydration may develop, negatively affecting LFCF air void content.

New York State Department of Transportation, Soil Mechanics Bureau, W. Averell Harriman State Office Campus, 1220 Washington Ave., Albany, N.Y. 12232.

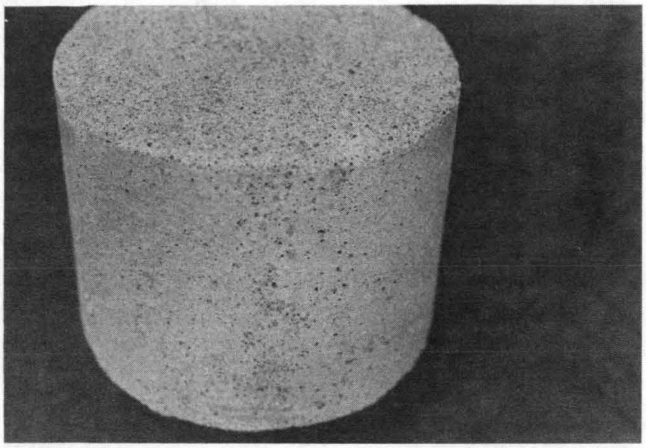

FIGURE 1 Sample of LFCF.

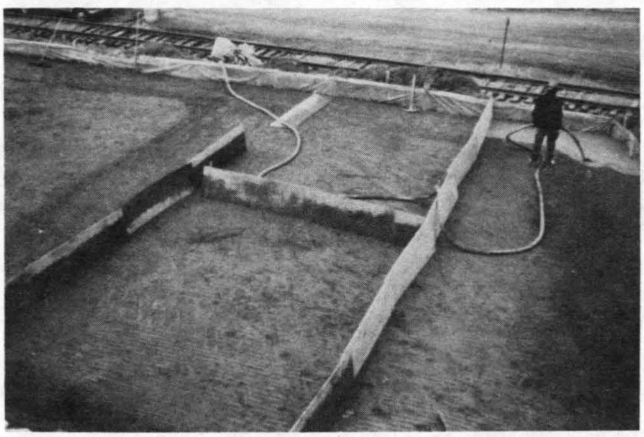

FIGURE 2 Temporary forms used in placement area (*2*).

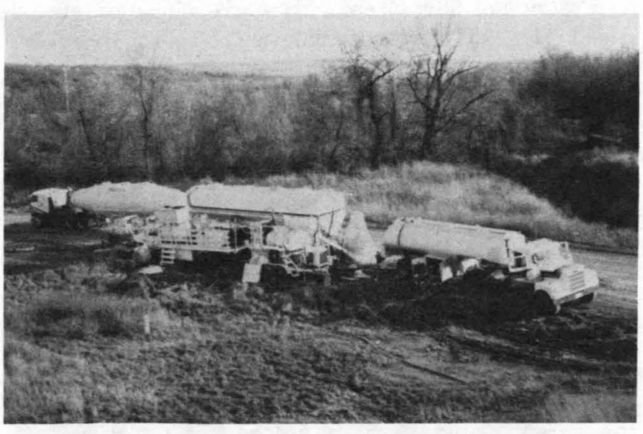

FIGURE 3 Equipment (*2*).

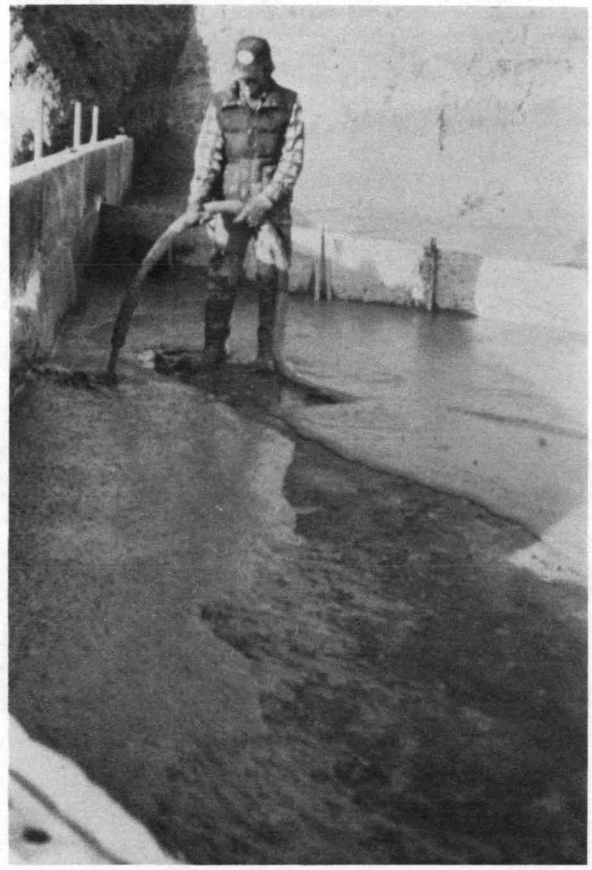

FIGURE 4 Placement of LFCF (*2*).

Before each lift sets up, the surface is sacrificed with a broom or rake (Figure 5), providing a roughened surface on which to place the next lift. Each subsequent lift is placed after a minimum 12-hr waiting period.

QUALITY ASSURANCE AND TESTING

To ensure that the maximum wet-cast-density requirement is being met, a density test is run on fill samples gathered at the point of placement. These are taken from the initial mix and every 30 min thereafter. To check density, a cylinder of known weight and volume is filled with the LFCF. The filled cylinder is then weighed (Figure 6) and the density calculated. On the basis of the test results, the process is adjusted as necessary.

Several factors can affect the mix. For example, as noted by Douglas (*1*), the amount of foaming agent added governs the number of air voids in the fill, but mix temperature governs their size. In addition, if the placement hose and the distance pumped exceeds about 244 m (800 ft), the air voids break down.

Compressive strength is evaluated by both the supplier and NYSDOT Materials Bureau of samples gathered at the point of placement. The supplier takes four 8- × 15-cm (3- × 6-in.) cylinders for each day's placement or each 61 m^3 (80 yd^3) of fill placed. NYSDOT takes four 15- × 30-cm (6- × 12-in.) cylinders for each day's placement or each 77 m^3 (100 yd^3) of fill placed. Although both the supplier and NYSDOT

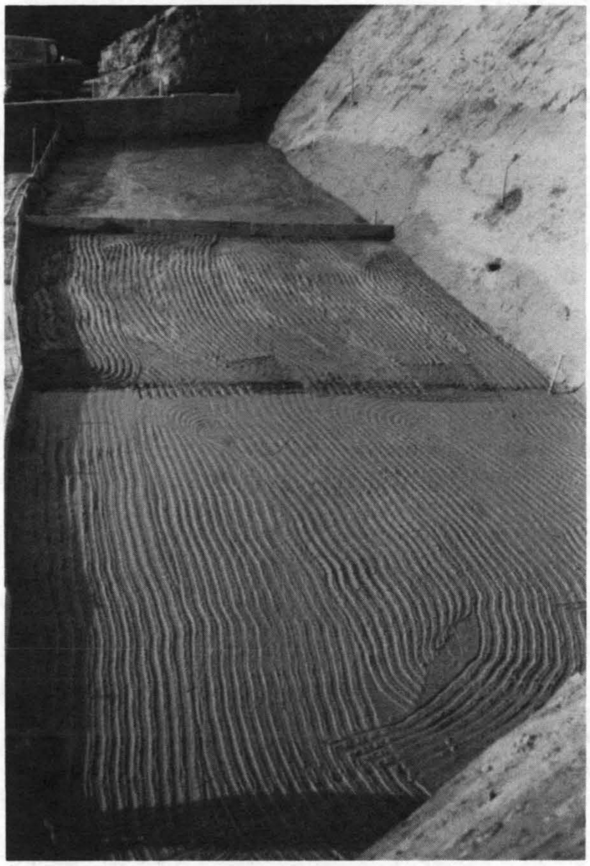

FIGURE 5 Scarified surface of lift (*2*).

run 28-day compressive tests, NYSDOT results govern. Additional samples are often gathered for compressive tests at 7- and 14-day intervals.

On at least two projects, some larger NYSDOT samples have failed to meet minimum 28-day compressive strength. In each case, the supplier's smaller samples have exceeded the minimum requirement. NYSDOT is currently gathering data to correlate sample size and compressive strength. Data are also being collected on the 7- and 14-day breaks to correlate compressive strength results with the 28-day breaks.

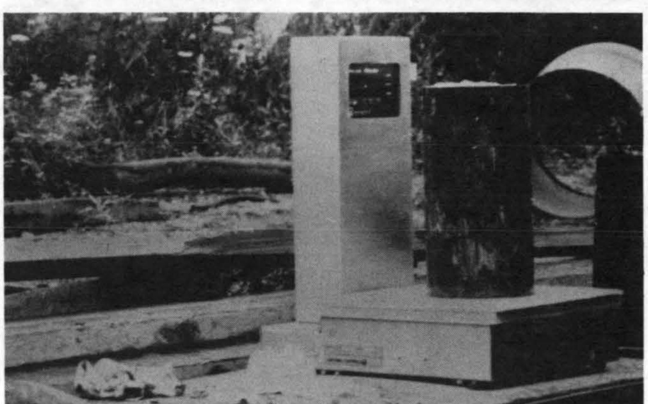

FIGURE 6 Field density testing (*1*).

NYSDOT SPECIFICATIONS

Although LFCF is available in a wide range of densities, NYSDOT specifications restrict its use to one of two densities, identified as Types A and B. These densities produce adequate strengths, and meet the requirement for reduced loads. Current specifications are for a maximum wet-cast density of 480 kg/m^3 (30 pcf) for Type A and 672 kg/m^3 (42 pcf) for Type B. Contract plans indicate which, if not both, is to be used for the project and where it will be placed.

COMPARISON WITH OTHER LIGHTWEIGHT FILLS

To reduce loads, NYSDOT also considers using expanded shale or slag with an in-place density from 880 to 1280 kg/m^3 (55 to 80 pcf), which is two to three times greater than that of the LFCF. Consequently, the excavation requirements for using expanded shale can be as much as 50 percent greater and frequently involve excavating below the groundwater or tide level. This also adds additional costs of dewatering and cofferdams.

The cost of expanded shale or slag ranges from \$30 to \$40/m^3 (\$40 to \$50/yd^3). LFCF costs typically range from \$50 to \$70/m^3 (\$67 to \$94/yd^3). Overall, lightweight costs vary with the quantity required for the project, contractor's experience with the product, and hauling distance.

Ultimately, the decision of which lightweight fill to use is based on economics, project site constraints, and availability.

DESIGN CONSIDERATIONS

When the use of LFCF was first considered, several questions about placement arose. Was LFCF feasible in an urban project with extremely high traffic volumes or in a project with limited space for staging? Could utilities be installed in the placement area? Could roadway grades, slopes, and profiles be met with this fill? With respect to fill performance, once placed how would it be affected by water? Could it be placed below the groundwater table? Would it float? Would it become saturated and increase in density if exposed to groundwater or infiltration through the pavement surface, or both? Would it be susceptible to freeze-thaw cycles? How would the fill be affected by traffic loading, especially in high-volume areas? Could pavement be placed directly on the fill?

Resolution of these questions—explained in detail by Douglas (*1*)—and subsequent experience with LFCF produced a list of design considerations now used by NYSDOT. Placement of LFCF in areas with high-traffic volumes, where offsite detours are impractical or where staging areas are limited, poses no difficulty. Preparation and placement of the fill require only four pieces of equipment (as previously listed) or less if some of the units are self-contained or combined. If there are many placement areas, it is frequently possible to cover them from one staging area.

Utility installation in the placement area is easily accommodated by setting utility pipes on temporary supports (Figure 7). Or (if allowed by the sequence of operation), when the fill has risen to just below the utility elevation, temporary

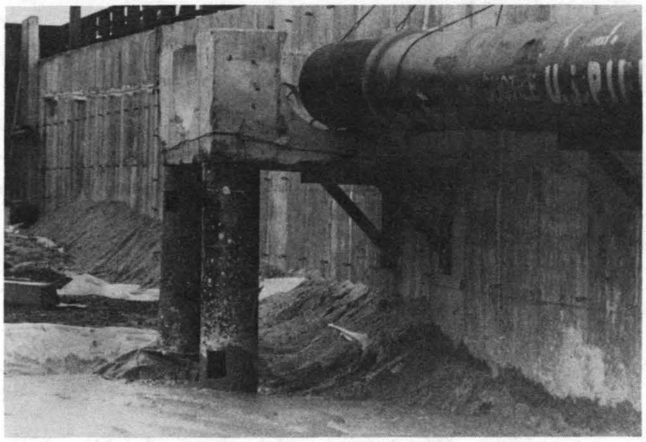

FIGURE 7 Utility placement (*1*).

blocking or bracing can hold the pipe in place as the fill is placed around it. Postconstruction utility installation, to meet future needs, can be accomplished by excavating the fill with a backhoe, jackhammer, or even hand tools. Pipe jacking or boring operations are other possibilities.

Grades and profiles can be established by placing the fill in stepped 15-cm or 0.3-m (6-in. or 1-ft) lifts (Figure 8) that are then trimmed and overlain with an asphalt truing-and-leveling course. Another method includes slightly overpouring the top lift and then removing the excess with hand tools. To establish a side or end slope, the fill can be placed in stepped lifts and topped with conventional fill, topsoil, or slope protection (Figure 8). In yet another method (for profiles or grades up to about 5 percent), a thickening agent can be added to the fill mix design. Because LFCF has the characteristics of a solid sponge and low density is a specific requirement in most projects, water absorption potential can be a concern. It was suggested (*1*), however, that ratios of exposed surface area to total volume for the laboratory samples and larger construction applications were not comparable; absorption of water in placement above high tide and groundwater level would not significantly increase loading on the foundation soils. This also reduces any potential for buoyancy. It was also concluded that overlying subbase or pavement, or both, is sufficient to keep the fill in place.

To prevent water absorption or buoyancy, however, NYSDOT places LFCF above normal groundwater and high-tide elevations. To compensate for occasional extremes of these elevations and prevent absorption of infiltration through the roadway surface, several techniques were identified to limit exposure of the LFCF surface area. The bottom of the placement area can be lined with a sheet of polyethylene. If the fill is not placed directly against the backs of wingwalls, concrete curtain walls can be built to protect the sides of the placement. Water flow between the interface of the fill and the curtain wall can be prevented by casting a waterstop into both. The top of the fill can be sealed with an asphalt emulsion. Asphalt hot mix also works but is considerably more tedious to apply. Drainage can be enhanced by placing underdrains at the base of the curtain walls, wingwalls, abutments, and at the pavement edge. Geotextile, however, should not be used with drainage—the fill will seal the fabric.

Freeze-thaw concerns were also addressed. By using any or all of the techniques described, very little water is likely to find its way to the fill. Furthermore, subbase or in some placements a lift of denser LFCF placed on top of the less dense LFCF acts as insulation from freezing temperatures.

Although it was believed that LFCF would respond at least as well as compacted subbase in areas of high traffic volumes, a top lift of denser LFCF was recommended to provide some performance insurance. As for placing the concrete pavement on the fill, there was speculation that the asphalt emulsion would allow the pavement to move over the fill if subjected to heavy traffic. For such a situation, it was recommended that the pavement slab be keyed into the underlying fill (Figure 9).

In some placement areas, the fill must have sufficient compressive strength to support footing or construction loads (Figures 8 and 10). In others, the primary consideration is frequently the low density. In this type of placement, when it is in place, the fill needs only to be as strong as compacted embankment material.

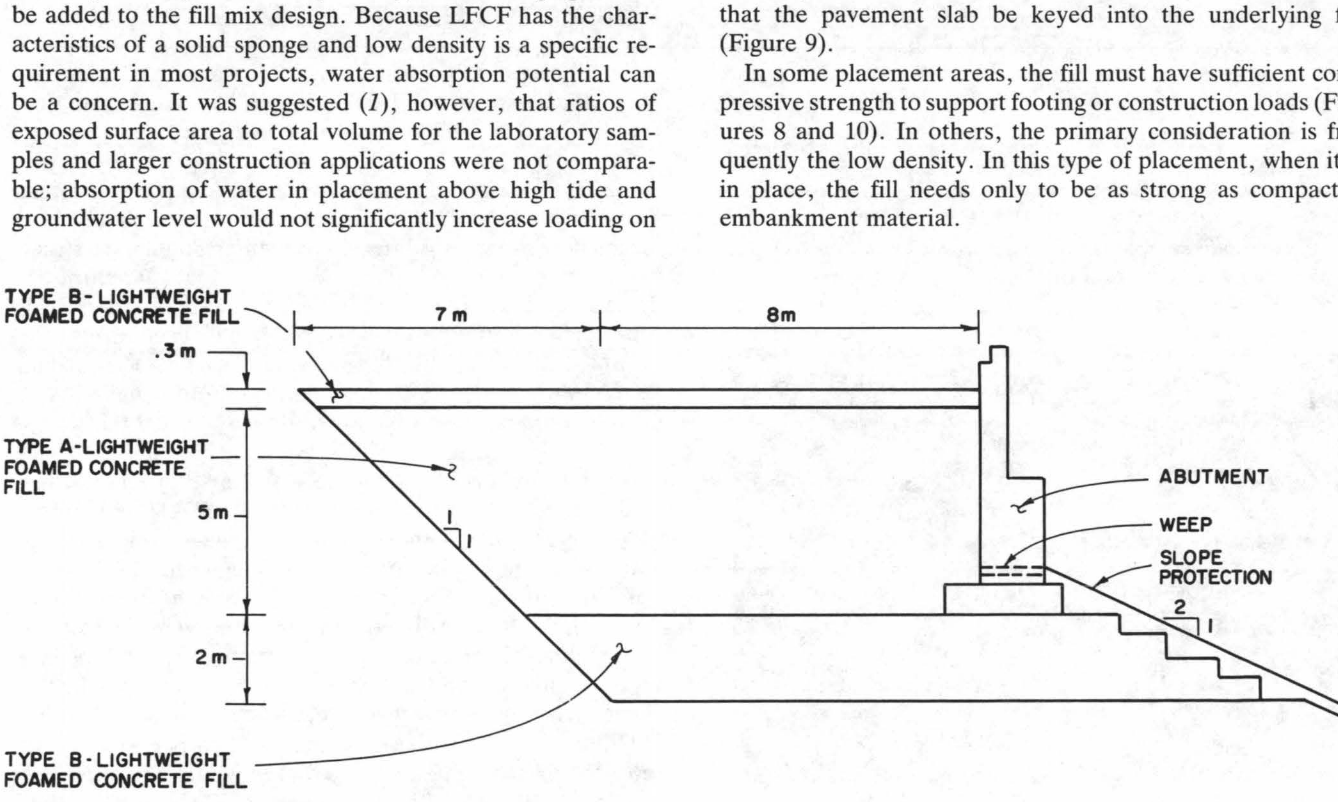

FIGURE 8 LFCF layered, stepped, and under a footing.

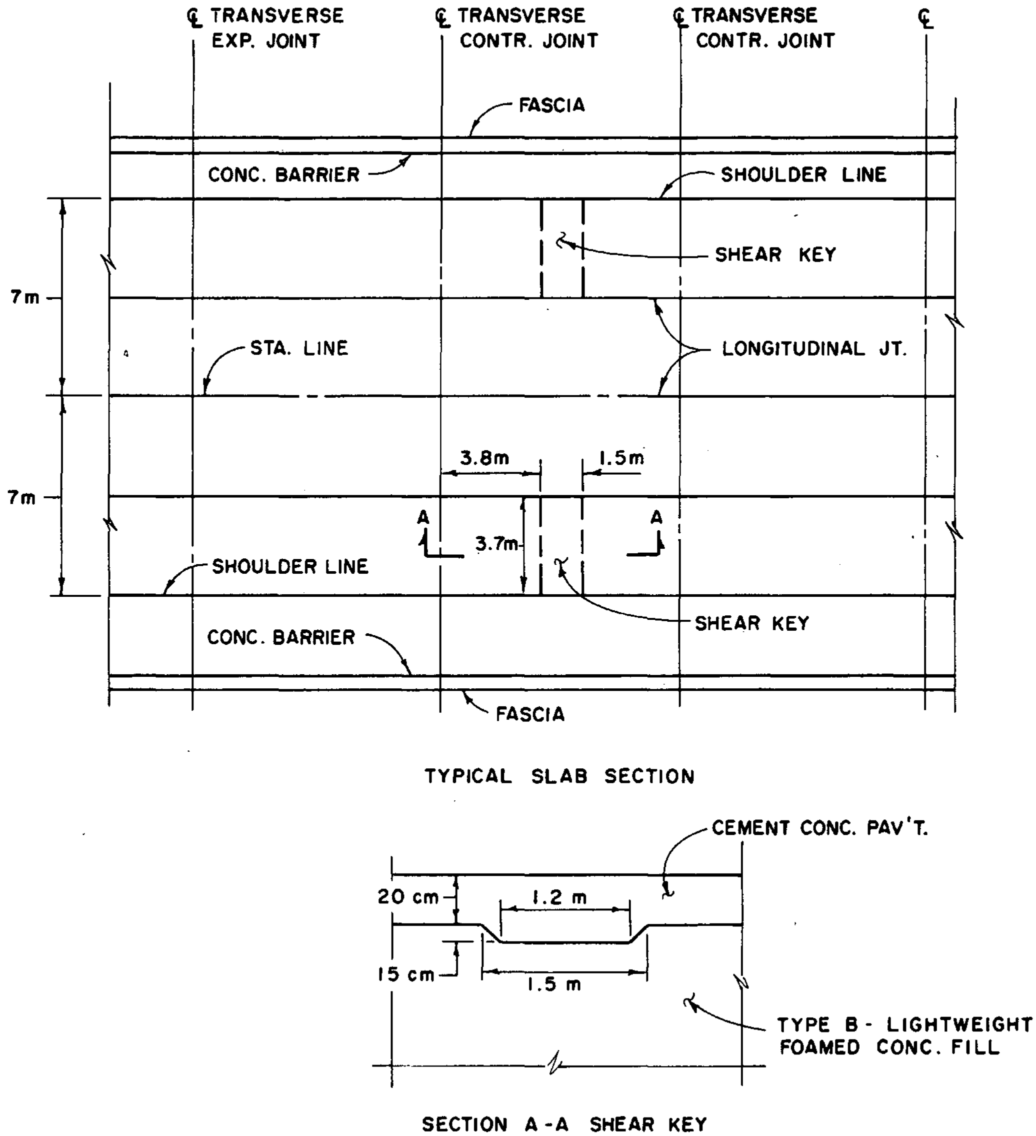

FIGURE 9 Shear key of reinforced concrete slab into LFCF (*1*).

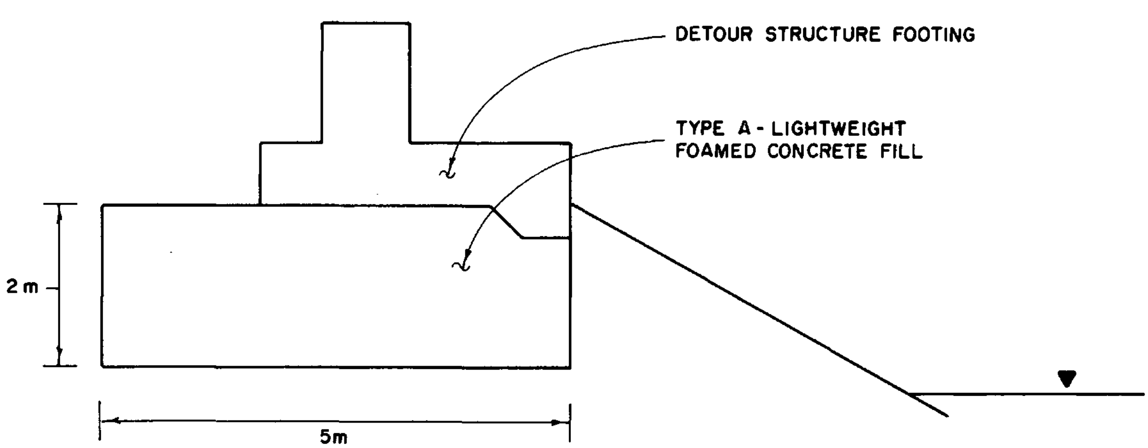

FIGURE 10 LFCF under detour structure footing.

EXPERIENCES WITH LFCF

Since 1981 NYSDOT has used the fill on seven projects, involving 12 placement areas. Although the areas varied somewhat in soil profile, bearing capacity, and embankment height, they were similar in the need to minimize loading on foundation soils or existing structures. Douglas (*1*) and McGrath (*2*) documented two of the earliest projects. Two other placement areas are described. Typical placement details are shown in Figures 8, 9, 10, 11, 12, and 13.

Pine Island Turnpike

To replace a structure carrying the Pine Island Turnpike over Pochuck Creek in the Town of Warren, Orange County, New York, an on-site detour embankment and structure were planned, to be placed beside the existing embankment and structure.

NYSDOT geotechnical engineers familiar with the area anticipated settlement difficulties. Subsequent subsurface explorations verified their concerns—the foundation soils consisted of 2 m (7 ft) of peat over 1 to 3 m (3 to 10 ft) of silty sand and 8 m (25 ft) of silty clay.

Settlement analyses for the 3-m (9-ft) approach embankments to the detour structure estimated 0.6 m (2 ft) of settlement and potential for failure of the approach embankment endslopes into the creek. Estimated settlement and failure potential jeopardized the detour structure.

As a lump-sum bid item, the contractor was responsible for design of the detour embankment and structure. To alert the contractor, a note was placed in the contract plans calling attention to the very low bearing capacity of the foundation soils.

On the basis of this information, the contractor's design consultant recommended that natural soil be replaced with LFCF [maximum wet-cast density of 672 kg/m^3 (42 pcf) and minimum 28-day compressive strength of 689 kPa (100 psi)] in the area under the detour structure footing (Figure 10). This replacement area was the width of the footing, 2 m (5 ft) deep, and 5 m (15 ft) from the front of the footing, which was 3 m (10 ft) wide, to 2 m (5 ft) behind the back of the footing.

No special provisions were made for the detour approach embankments. The contractor chose to maintain the roadway profile using additional shimming with asphalt rather than attempt to minimize the settlement. No provisions were made to prevent absorption of groundwater or infiltrating surface water. Because this was a temporary detour, potential short-term absoption was not considered a problem.

Route 150

An existing two-span structure carrying Route 150 over the Amtrak Railroad and Brookview Station Road in the Town of Schodack, Rensselaer County, New York, was replaced

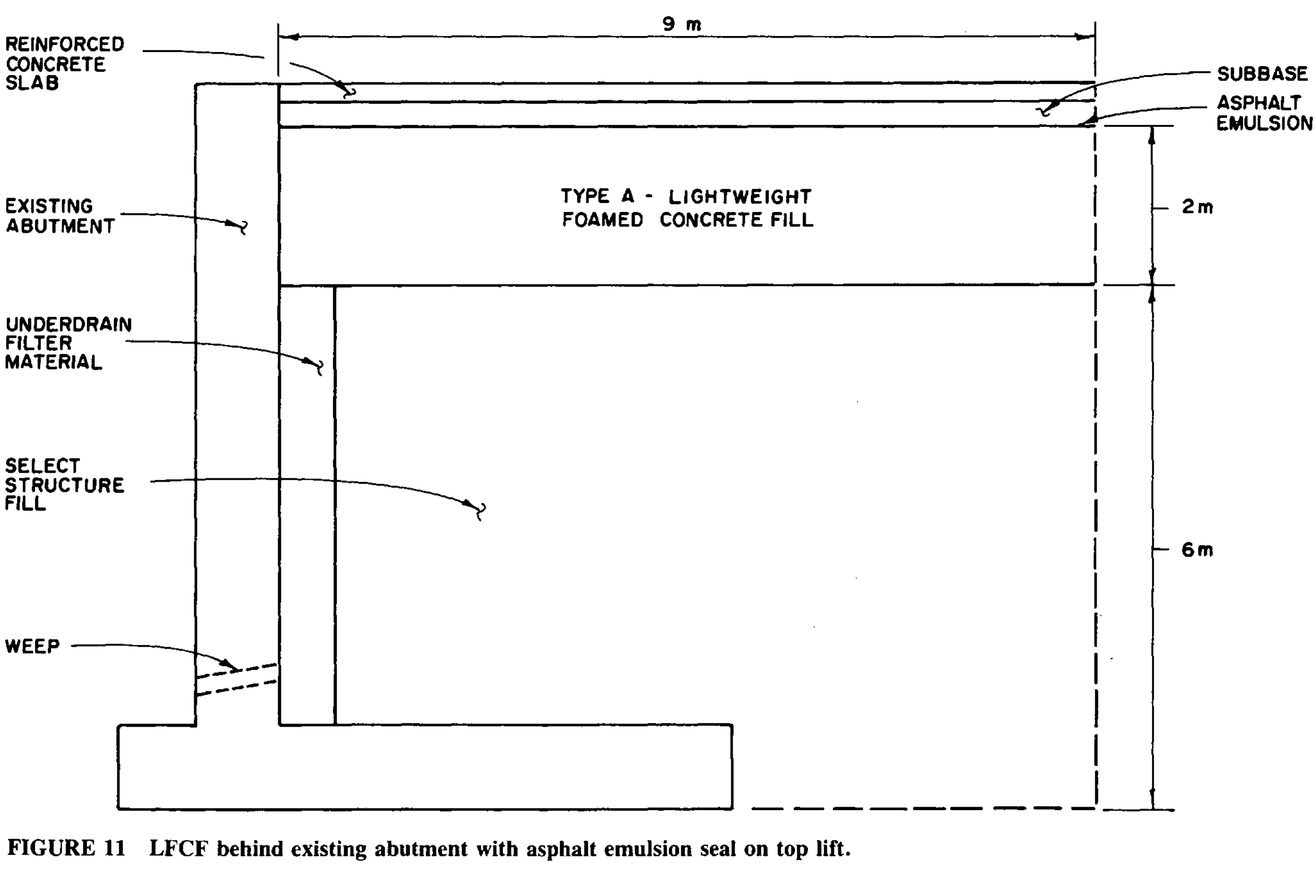

FIGURE 11 LFCF behind existing abutment with asphalt emulsion seal on top lift.

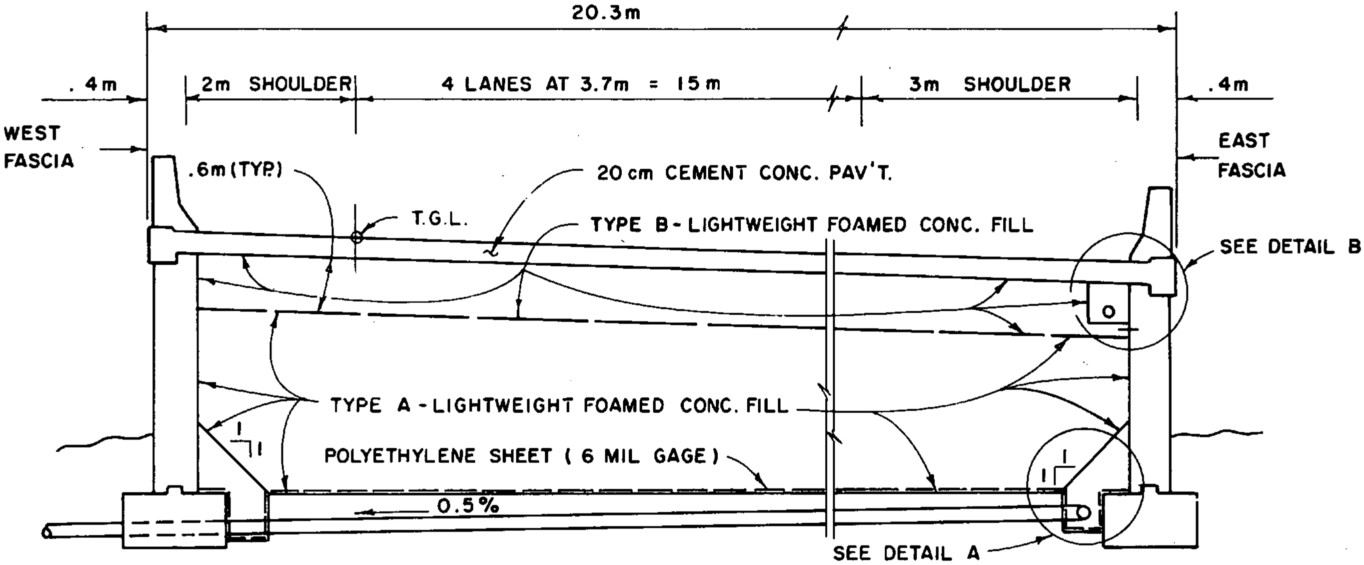

FIGURE 12 LFCF, underdrains, polyethylene sheeting, and waterstop (*1*). Details A and B are enlarged in Figure 13.

with a single-span structure. Because the existing laid-up stone abutments built in 1899 were still structurally sound, they were modified to support the new superstructure.

To accommodate the increased height of the new superstructure, it was necessary to increase the grade of the approach embankments by 1 m (3½ ft)—8- to 9-m (26- to 28-ft) high approach embankments on 9 to 10 m (28 to 33 ft) of very soft to soft clay and silty clay underlain by loose to very compact silt. Analyses of foundation soils under the existing abutments, however, indicated the soil was not capable of supporting the increased design loads.

To reduce the proposed loading, LFCF, with a maximum wet-cast density of 480 kg/m^3 (30 pcf) and a minimum 28-day compressive strength of 276 kPa (40 psi), was chosen to replace the conventional fill for 9 m (30 ft) behind each abutment (Figure 11). Replacement depth on the west side was a little less than 2 m (6 ft) and a little more than 2 m (8 ft) on the east.

To minimize any effect by water, a column of underdrain filter material 0.6 m (2 ft) thick was placed under the LFCF and against the back of each abutment. Weeps outletted through the abutments. The LFCF top surface was sealed with an asphalt emulsion. The overlying pavement section consisted of 30 cm (12 in.) of subbase topped by a reinforced concrete approach slab.

Contract plans specified a crown of 6 mm (¼ in.) to 0.3 m (1 ft) of roadway profile. To accomplish this, the last lift of LFCF was slightly overpoured and smoothed. After setting, excess LFCF was easily removed with hand tools.

To facilitate timely placement of the new superstructure, additional cylinders were taken during the LFCF placement to evaluate 7-day compressive strength. The 7-day break results, from 241 to 531 kPa (35 to 77 psi), were deemed close enough to the required 276 kPa (40 psi) to allow the contractor to proceed.

To minimize damage to the 2-week-old LFCF top lift, 2 × 3 m (8 × 10 ft) pads constructed of three crisscrossed layers of 5 × 19s (2 × 6s) were placed 2 m (8 ft) from each abutment backwall. On these, a 127-Mg (140-ton) crane was placed on the east end and a 91-Mg (100-ton) crane was placed on the west end. Planking was also placed under each crane outrigger. Beams for the new superstructure were then lifted into place. This technique worked well—no visible damage occurred to the LFCF.

PERFORMANCE

Monitoring of each placement area varies after completion of a project. Some areas are heavily monitored with slope indicators, settlement platforms, and survey hubs. Other areas have only a few survey hubs. Still others are given only a visual check for cracks or undergo a rideability reading. Monitoring depends on site movement history and amount of movement anticipated.

To date, although one placement area has undergone anticipated long-term settlement of as much as a meter (several feet), the LFCF has performed well. Settlements are minimal and no movement of original structures has been noted. There is no indication of water absorption or failure of the fill caused by traffic loading. Furthermore, as noted by McGrath (*2*), at least one placement area was left open and exposed during winter and had no sign of deterioration.

CONCLUSIONS AND RECOMMENDATIONS

Douglas (*1*) and McGrath (*2*) present conclusions and recommendations that have resulted in current NYSDOT specifications and design considerations. LFCF has proved to be an effective lightweight fill for areas with underlying weak and compressible clayey and organic soils. It has also been effective as backfill for existing abutments and retaining walls that are unable to withstand additional loads.

It is recommended that the specialty contractor have documented experience with the product. Successful LFCF mix-

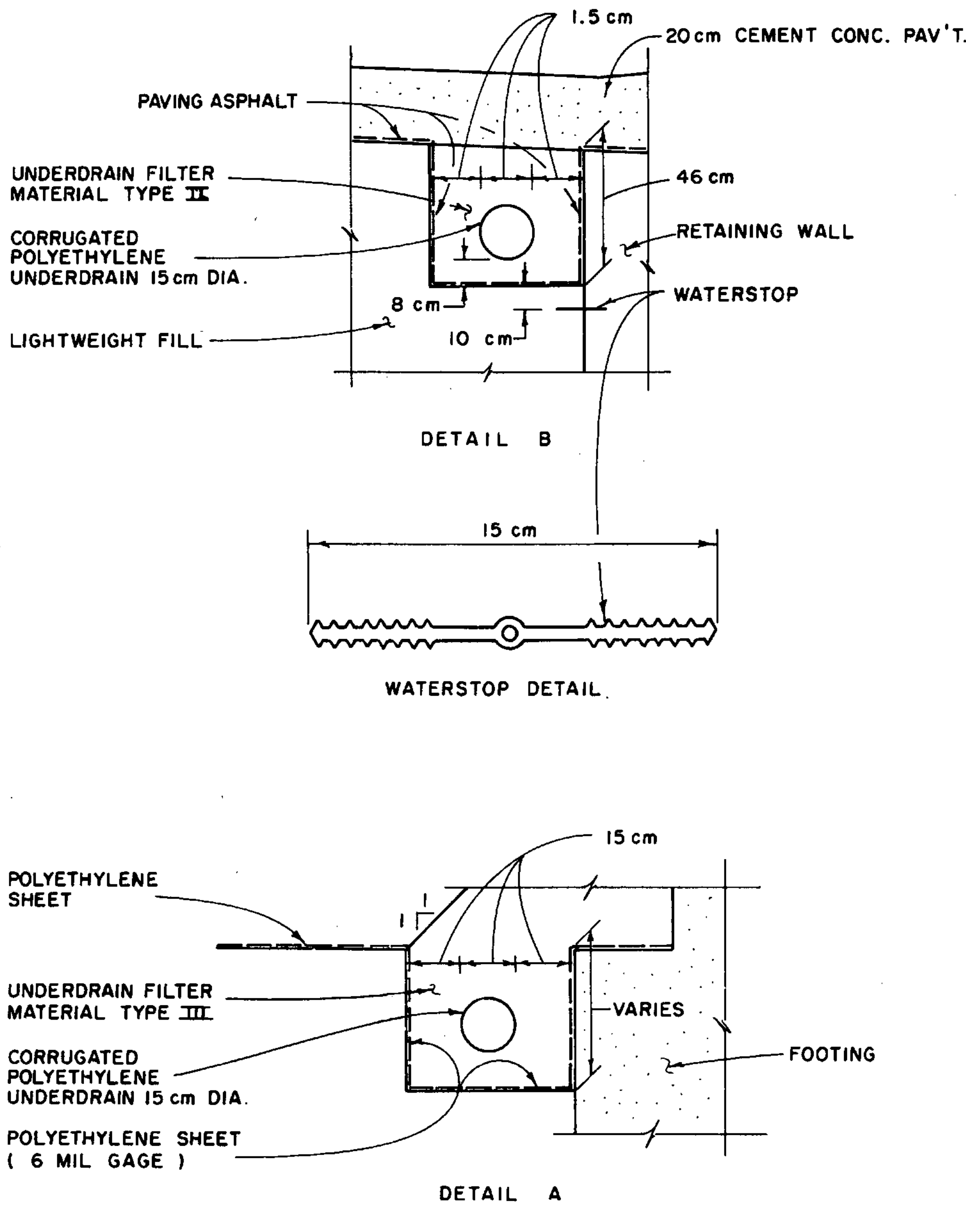

FIGURE 13 Details of underdrains and waterstops (*1*).

ing and placing depends very much on experience. It is also recommended that the supplier's representative be on site during initial placement to ensure proper mix design and answer questions throughout construction.

Finally, a correlation needs to be established between the different sizes of samples taken by the supplier and owner. A correlation also must be established between compressive strengths of any 7- and 14-day tests and the required 28-day tests.

ACKNOWLEDGMENTS

The author thanks those in the NYSDOT Soil Mechanics Bureau, experienced with LFCF, who offered advice and provided data, graphics, and photographs and eagerly critiqued this paper.

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Shear Strength and Compressibility of Tire Chips for Use as Retaining Wall Backfill

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Scrap tires that have been cut into chips are coarse grained, free draining, and have a low compacted density, thus offering significant advantages for use as lightweight fill and retaining wall backfill. The engineering properties needed to put tire chips into use are presented. The properties determined for tire chips, from three suppliers, are gradation, specific gravity, compacted density, shear strength, compressibility, and coefficient of lateral earth pressure at rest. The 76-mm (3-in.) maximum size and high compressibility of the tire chips necessitated design and fabrication of custom-made testing equipment. The tests showed that the tire chips are composed of uniformly graded, gravel-sized particles that absorb only a small amount of water. Their compacted density is 0.618 to 0.642 Mg/m^3 (38.6 to 40.1 pcf), which is about one-third that of compacted soils. The shear strength was measured in a large-scale direct shear apparatus. The friction angle and cohesion intercept ranged from 19 to 25 degrees and 8 to 11 kPa (160 to 240 psf), respectively. The compressibility tests showed that tire chips are highly compressible on initial loading, but that the compressibility on subsequent unloading and reloading cycles is less. The horizontal stress was measured during these tests and showed that the coefficient of lateral earth pressure at rest varied from 0.26 for tire chips with a large amount of steel belt exposed at the cut edges to 0.47 for tire chips composed entirely of glass-belted tires.

Disposal of the estimated 2 billion scrap tires that have been discarded in huge open piles across the United States is a monumental problem. Furthermore, an additional 189 million are added to these piles each year (*1*). These piles are a serious fire hazard, prolific breeding ground for mosquitoes, and ugly scar on our landscape. Society is increasingly looking to the transportation industry to help solve the scrap tire disposal problem, as evidenced by the requirement of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) that, by 1997, one-fifth of all road projects must include 10 kg of recycled rubber per megagram (20 lb/ton) of hot mix and 150 kg of recycled rubber per megagram (300 lb/ton) of sprayed binder.

Another use for scrap tires is fill. In this application the tires are cut into durable, coarse grained, and free draining chips that have a low compacted density. Because each cubic meter of tire chip fill contains about 100 waste tires (75 tires per cubic yard), there is potential for using a large number of tires especially when compared with the 1.5 tires that ISTEA requires be used per megagram (1.4 tires per ton) of hot mix. Furthermore, it is much easier to cut tires into large chips than to produce the crumb rubber or liquefied rubber needed for use in hot mix.

Waste tire chips are already used as lightweight fill for highway embankments (*2–4*) and an insulating layer beneath an unpaved road in a northern climate (*5*). Another use is as backfill behind retaining walls and bridge abutments. The low compacted density would potentially result in low horizontal pressures on the wall. Thus, a lighter wall could be used to retain them. Furthermore, their low compacted density will reduce the settlement of underlying compressible soils and increase the global stability of the wall. In some cases, this will allow the wall to be placed on a spread footing rather than on a pile foundation, which would significantly reduce construction costs. Because tire chips are free draining, there is no need for clean granular backfill.

A necessary first step is to determine the engineering properties of tire chips. The gradation, specific gravity, compacted density, and compressibility of tire chips from a supplier in Hampden, Maine, were determined (*6,7*). In the present study these properties and the shear strength were determined for tire chips from three additional suppliers. This will provide the basis for future field trials using tire chips as retaining wall backfill.

The three tire chip suppliers are F&B Enterprises, New Bedford, Massachusetts; Palmer Shredding, North Ferrisburg, Vermont; and Pine State Recycling, Nobleboro, Maine. The F&B chips were composed entirely of glass-belted tires and were less than 38 mm (1½ in.) in size. The Palmer and Pine State chips were composed of a mixture of glass- and steel-belted tires. The Palmer chips had a large amount of steel belt exposed at the cut edges of the tire chips. The Palmer chips were 76-mm (3-in.) maximum size and the Pine State chips 51-mm (2-in.) maximum size. The Palmer and Pine State chips tended to be long in relation to their thickness, and the F&B chips tended to be more equidimensional.

The large size and high compressibility of the tire chips necessitated modification of conventional test procedures and design and fabrication of custom-made testing apparatus. The test procedures, apparatus, and results for each property will be discussed. Further details are given in a work by Humphrey et al. (*8*).

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GRADATION, SPECIFIC GRAVITY, AND COMPACTED DENSITY

The gradation of the tire chips from the three suppliers was determined using AASHTO T27-87 (*9*). The tire chips are uniformly graded and composed of gravel sized particles (Figure 1). The Palmer chips were the coarsest and the F&B chips the finest.

The specific gravity of the tire chips was determined using AASHTO T85-85 (*9*), except that the samples were air dried rather than oven dried at the start of the tests. The apparent specific gravities based on the average of two tests were 1.14 for F&B chips, 1.27 for Palmer chips, and 1.24 for Pine State chips. These specific gravities are less than half of those typical of soils. The specific gravity of the F&B chips is lower than the other two because the F&B chips are entirely glass belted.

The test procedure used to determine the compacted density of air dried tire chips was adapted from AASHTO T180-86 (*9*). A mold 254 mm (10 in.) in diameter and 254 mm (10 in.) high with a volume of 0.012 m^3 (0.44 ft^3) was used. The tire chips were compacted in three layers with a 4.536-kg (10-lb) hammer falling 0.457 m (18 in.). Previous research showed that decreasing the compaction energy from modified Proctor to 60 percent of standard Proctor reduced the density by only 0.03 Mg/m^3 (2 pcf) and that compaction of wet versus air dried tire chips made only a 0.016 Mg/m^3 (1 pcf) difference in the density (*6,7*). Because the compaction energy and wet versus air dried tire chips had only a small effect, 60 percent of standard Proctor energy and air dried tire chips were used for this study. The compacted density of air dried tire chips from the three suppliers fell within a fairly narrow range. The compacted density based on the average of three tests was 0.618 Mg/m^3 (38.6 pcf) for F&B chips, 0.619 Mg/m^3 (38.7 pcf) for Palmer chips, and 0.642 Mg/m^3 (40.1 pcf) for Pine State chips. These values are about one-third of those typical for compacted soils showing the potential for tire chips to be used as lightweight fill.

SHEAR STRENGTH

Testing Apparatus

The shear strength of tire chips was determined using a direct shear apparatus custom designed to accommodate the large size and high compressibility of the tire chips. In addition, special provisions were made to eliminate friction between the two halves of the shear box.

A 305-mm (12-in.) square shear box (nominal dimension) made from steel 9.5 mm (3/8 in.) thick was chosen for the initial design. This was adequate because the largest-size tire chips to be tested were minus 76 mm (3 in.). Thus, the shear box would be four times larger than the largest tire chip. The lower half of the shear box was 76 mm (3 in.) high and bolted to a supporting bench. The top half of the shear box was 152 mm (6 in.) high. This height was needed to accommodate the large compressibility of the tire chips. To determine whether the area of the shear box influenced the test results, a 406-mm (16-in.) square shear box (nominal dimension) was also fabricated.

It was essential to maintain a gap between the two halves of the box to prevent introduction of additional horizontal stresses due to friction. During sample preparation a 6-mm (1/4 in.) gap was opened by placing spacers at each corner between the halves of the box. Then to maintain the gap during testing, two steel wheels 51 mm (2 in.) in diameter

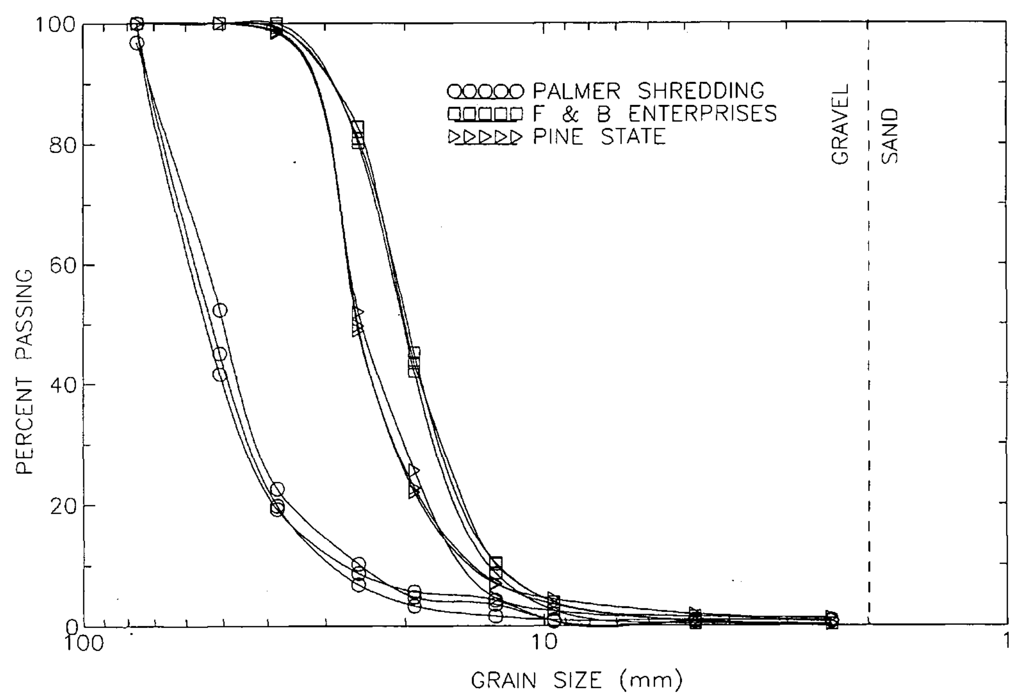

FIGURE 1 Gradation of tire chips from three suppliers.

with low-friction ball bearing hubs (similar to old fashioned roller skate wheels) mounted in a steel frame were clamped to each end of the box. The spacers were then removed. During testing the wheels rode along the top of the supporting bench, carrying the top of the box to maintain the gap between the box halves.

The normal stress was applied using dead weights hung from a hanger suspended under the sample. A maximum dead load of 5570 kN (1,250 lb) could be applied to the sample. This resulted in a maximum normal stress of 68 kPa (9.9 psi) for the 305-mm box, which is equivalent to approximately 3 m (10 ft) of soil fill.

The horizontal shearing force was provided by a ⅛-hp electric motor acting through a gear box, which allowed the rate of horizontal deformation to be adjusted. A rate of approximately 7.6 mm/min (0.3 in./min) was used. The horizontal shearing force was measured with a 4450-kN (1,000-lb) capacity load cell. Two linear variable differential transformers (LVDTs) were used to measure horizontal and vertical displacements.

Sample Preparation

The inside of the upper half of the shear box was greased to minimize the portion of the applied vertical load transmitted to the sides of the box by friction. Then, the samples were compared with 60 percent of standard Proctor energy. The box was filled in three 64- to 76-mm (2.5- to 3-in.) layers to approximately 25 mm (1 in.) from the top. To ensure that there was no effect on the shearing plane from a smooth surface between the first two layers, care was taken that the top of the first layer did not coincide with the gap between the halves of the box.

Results

Direct shear tests were run using the 305-mm box at three normal stresses. Three tests were done for each normal stress for each of the three suppliers. A total of 27 tests was done with the 305-mm box. In addition, tests were done using the 406-mm box with Pine State tire chips with three normal stresses.

In direct shear tests, failure is considered to be the peak shear stress or, if no peak is reached, failure is generally taken as the shear stress at a horizontal displacement equal to 10 percent of the length of box (*9*). The latter criterion controlled for tire chips. Thus, for the 305-mm box, which had an inside dimension of 286 mm (11.25 in.), failure was taken as the shear stress at a deformation of 28.6 mm (1.1 in.). For the 406-mm box, which had an inside dimension of 387 mm (15.25 in.), failure would be at a deformation of 38.7 mm (1.5 in.). However, the travel of the LVDT used to measure horizontal displacement was limited to 35.6 mm (1.4 in.), so failure for tests with this box were taken to be the stress at this displacement.

The shear stress versus horizontal deformation for Pine State tire chips with the 305-mm box is given in Figure 2. This shows that the shear stress continues to increase past a horizontal deformation equivalent to 10 percent of the length of the box. The curves for the F&B and Palmer chips in the 305-mm box and the Pine State chips in the 406-mm box were similar (*8*).

The average shear stress versus average normal stress at each of the three loading increments for each of the samples is given in Figure 3. Each point is the average of two or three trials at a given normal stress. All these lines plot slightly concave down. For the Pine State tire chips, the 305- and 406-mm boxes give nearly identical results (Figure 3). Thus, the 305-mm box is large enough for the size tire chips investigated.

Comparison of the failure envelopes shows that the F&B chips are stronger than the others (Figure 3). This may be because these tire chips were smaller and more equidimensional. During shearing the tire chips would tend to lock together more instead of sliding past one another on the shearing plane as did the larger, flatter pieces. This is particularly true because the large flat pieces tended to be oriented parallel to the horizontal shear plane.

The friction angles ϕ and cohesion intercepts c were determined using best fit straight lines through the data and are

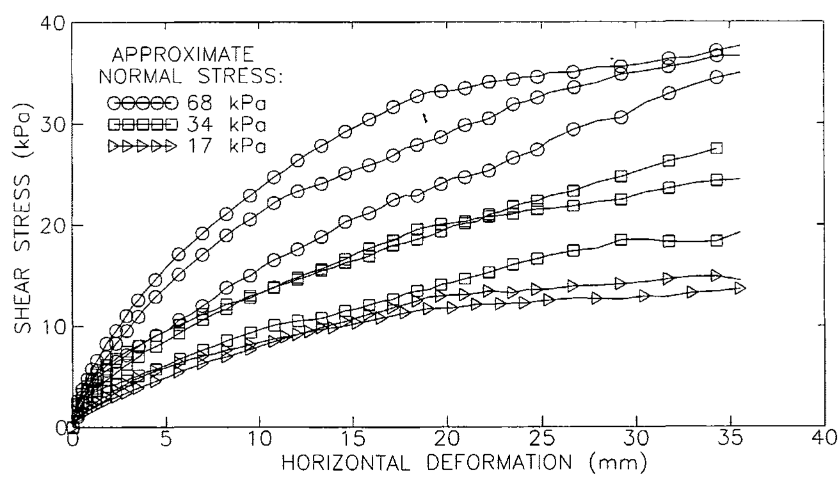

FIGURE 2 Shear stress versus horizontal displacement for Pine State tire chips.

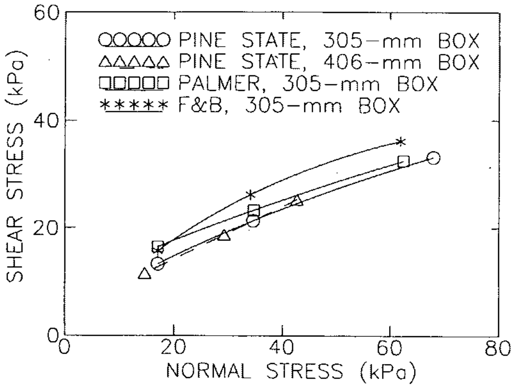

FIGURE 3 Failure envelopes for tire chips from three suppliers.

given in Table 1. This shows that the Palmer chips had the highest cohesion although their friction angle was low. This may be because they have a large amount of exposed steel belts, which interlock and do not rely on normal stress to develop their strength.

The choice of failure as the shear stress at a horizontal deformation equal to 10 percent of the length of the box (28.6 mm for the 305-mm box) is rather arbitrary. To investigate what effect this could have, the ϕ and c were determined for shear stresses at 15.2 mm (0.6 in.) and 35.6 mm (1.4 in.) of horizontal deformation. In general, they showed that the cohesion intercept decreased as the horizontal deformation chosen as failure decreased but that there was only a small effect on the friction angle. This suggests that a low or zero cohesion intercept should be used for design because it appears that significant deformation is needed to develop the cohesion.

Vertical deformation was also measured during the tests. All tests exhibited a decrease in height. The samples with the highest vertical stress tended to have the largest decrease in height.

COMPRESSIBILITY

Testing Apparatus

Sample Container

The container used for the compression tests consisted of a piece of schedule 40 PVC pipe 305 mm (12 in.) in diameter (nominal) and 318 mm (12.5 in.) long with a wall thickness of 8.1 mm (0.32 in.). Four strain gauges were placed with a horizontal orientation 89 mm (3.5 in.) above the base. They were calibrated to give the horizontal stress exerted on the inside of the container by the tire chips. Two additional strain gauges were placed vertically. They were calibrated to measure the portion of the applied load transferred from the tire chips to the container by friction (*6,7*).

TABLE 1 Shear Strength of Tire Chips from Three Suppliers

Supplier	ϕ	c (kPa)
Pine State (305-mm box)	21°	7.7
Pine State (406-mm box)	26°	4.3
Palmer Shredding	19°	11.5
F&B Enterprises	25°	8.6

1 kPa = 20.89 psf

Loading and Data Acquisition System

An Instron 4204 universal testing machine controlled by an IBM-compatible 80286 computer was used to apply the vertical load. The computer controlled the rate of deformation and obtained measurements of the vertical load and vertical displacement. A wheatstone bridge took readings from the strain gauges. The output voltages from the bridge were read by an analog to digital converter with an accuracy of 16 bits. The readings were taken at 10-sec intervals. To help offset electronic noise and imbalance at the time of a reading, the computer would take 10 readings from each strain gauge, which were averaged for the final reading (*6,7*).

Testing Methodology

Sample Preparation

Compacted samples were prepared by clamping the container to the steel base plate. Grease was brushed on the inside of the container to reduce the friction between the tire chips and the wall of the container. The tire chips were compacted in five layers with 60 percent of standard Proctor energy (*6,7*). The sample was then placed in the Instron and the clamps were removed.

Data Acquisition and Stress Computations

The load was applied to the sample at a constant rate of deformation of 13 mm/min (0.5 in./min). Readings from the strain gauges, vertical load, and vertical deformation were taken every 10 sec. From these readings the average vertical stress in the sample (σ_{avg}), the vertical strain (ε_v), vertical stress in the sample at the strain gauge height (σ_{gauge}), and the horizontal stress at the gauge height (σ_h) were calculated. The relationship between σ_{avg} and the known stresses at the top of the sample (σ_{top}) and σ_{gauge} is given in Figure 4. The vertical stress at gauge height (σ_{gauge}) is found by subtracting the load transmitted by friction to the container as measured at the gauge height (P_{frict}) from the load applied at the top of the sample ($P_{applied}$) and then dividing by the area of the sample. The average vertical stress (σ_{avg}) is the vertical stress at mid-height of the sample. It was computed by assuming that the load transmitted by friction to the container varies linearly from zero at the top of the sample to a maximum at the bottom. Because the strain gauges are 89 mm from the bottom, the load carried at mid-height (P_{avg}) is given by

$$P_{avg} = P_{applied} - [(H/2) * P_{frict}/(H - 89)]$$

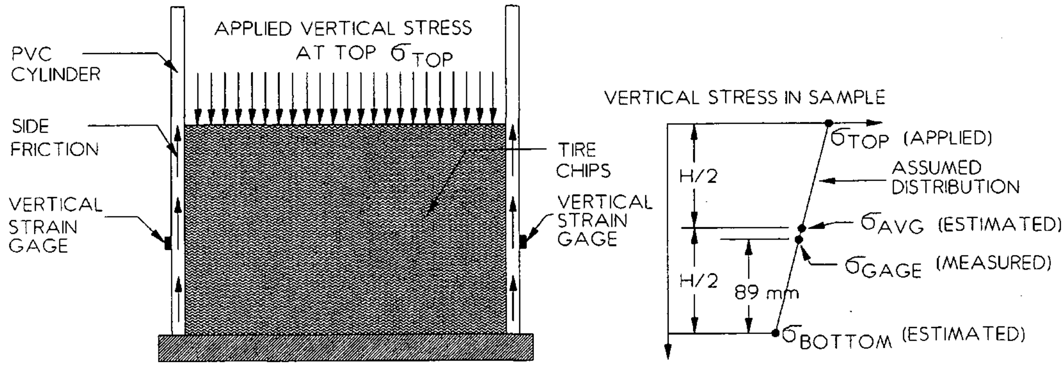

FIGURE 4 Effect of friction on vertical stress (*6,7*).

where H is the current height of the sample in mm. The average vertical stress (σ_{avg}) is found by dividing P_{avg} by the area of the sample.

Loading and Unloading Cycles

Most samples were subjected to three cycles of loading and unloading. The loading and unloading cycles are of particular importance for highway applications because they indicate the deformation behavior that would occur under repetitive vehicle loading. To apply the first loading cycle, the vertical load ($P_{applied}$) was increased until it reached 4.1 Mg (9,000 lb). This was chosen as the upper limit of loading because it is near the maximum capacity of the Instron. The clamps that held the container to the base were put in place, and the sample was then unloaded until the average vertical load in the middle of the sample (P_{avg}) was reduced to 2980 kN (670 lb) or about 41 kPa (6 psi). This process continued until three cycles of loading and unloading had been performed. The clamps were left in place for the second and third loading and unloading cycles.

Results

For each supplier, three tests were performed on samples compacted with 60 percent of standard Proctor energy. Most of these tests consisted of three loading and unloading cycles. Vertical compressibility and horizontal stresses are discussed in separate sections. Selected results are presented to illustrate the general compressibility behavior. Summaries are made to permit a comparison of the compressibility of tire chips from the three suppliers. Then elastic parameters computed from the combined measurements of vertical compressibility and horizontal stresses are presented. Complete compressibility results are given in a work by Humphrey et al. (*8*).

Vertical Compressibility

Results from one test on Palmer chips are given in Figure 5 to illustrate a typical graph of vertical strain (ε_v) versus average vertical stress (σ_{avg}). The initial portion of the first loading curve is very steep, indicating high compressibility. The first loading curve then flattens out at higher stresses. The slopes of subsequent unloading and reloading curves are similar to the flatter part of the first loading curve. The reloading curves lie slightly above the unloading curves. Tests on tire chips from the other suppliers showed similar behavior (*8*).

To permit a comparison of the initial compressibility, the vertical strain for the first loading cycle at average vertical stresses of 69 and 276 kPa (10 and 40 psi) is given in Table 2. Ordering the results from least to most compressible (F&B, Pine State, Palmer) shows that there is a general trend of increasing compressibility with increasing amounts of exposed steel belts. However, from a practical viewpoint, the difference in compressibility between tire chips from the three suppliers is small.

Horizontal Stress

The horizontal strain gauges were used to measure the increase in horizontal stress as the sample was loaded. A typical graph for stresses at gauge height of horizontal stress (σ_h) versus vertical stress (σ_{gauge}) for compacted Pine State tire chips is given in Figure 6. For the initial loading the graphs

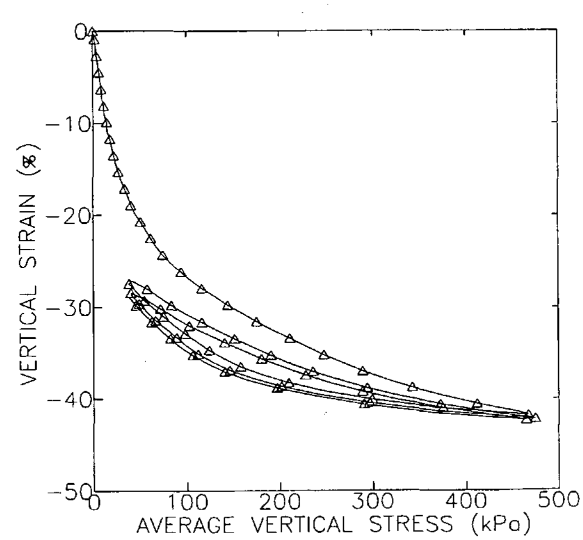

FIGURE 5 Deformation behavior of Pine State tire chips.

TABLE 2 Vertical Strains at Average Vertical Stresses 69 kPa and 276 kPa

		Vertical Strain At Average Vertical Stress =	
Supplier	Test No.	69 kPa	276 kPa
Pine State Recycling	1	23.6	36.4
	2	28.7	39.4
	3	29.5	39.4
	Average	27.3	38.4
Palmer Shredding	1	26.0	40.9
	2	30.6	42.9
	3	30.3	43.8
	Average	29.0	42.5
F&B Enterprises	1	24.8	38.4
	2	21.6	38.4
	3	22.9	35.9
	Average	23.1	37.6

1 kPa = 0.1450 psi

show a flatter slope up to a horizontal stress of approximately 69 kPa (10 psi). After this point the line is steeper. This change in slope coincides with the point at which the calibration curve for the horizontal strain gauges changes from a straight line for stresses less than 69 kPa to a second-order polynomial for higher stresses. This causes the distinct transition at 69 kPa. Nonetheless, the initial portion of the curve has a flatter slope. It has been theorized that the flatter initial slope is due to the compression of the voids and the steeper upper portion is due mainly to deformation of the rubber particles (*6,7*). Tests on tire chips from the other suppliers showed a similar behavior (*8*).

Elastic Parameters

Elastic parameters were calculated using the measurements of vertical compressibility and horizontal stress. These parameters were then used to make another comparison of the compressibility of the tire chips from the three suppliers and will be used for a future numerical analysis of retaining wall behavior when tire chips are used as backfill.

The coefficient of earth pressure at rest K_o was determined from the slope of the vertical stress at gauge height versus

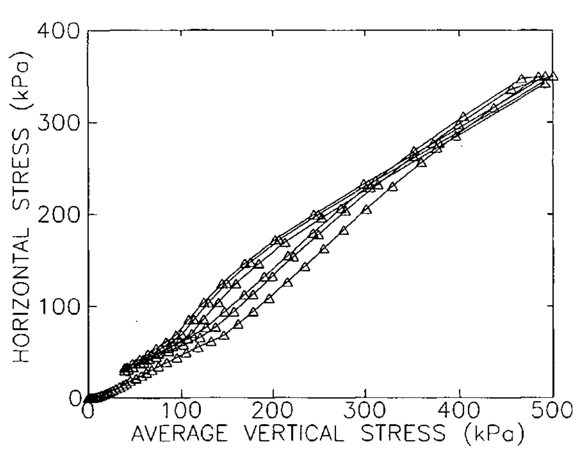

FIGURE 6 Horizontal stress versus vertical stress for Pine State tire chips.

horizontal stress at gauge height curves

$$K_o = \Delta\sigma_h/\Delta\sigma_{gauge}$$

The values of K_o were determined at a horizontal stress less than 69 kPa because this corresponds to vertical stresses most likely to be encountered in highway applications. Using K_o, Poisson's ratio μ can be determined using the following relationship (*10*)

$$\mu = K_o/(1 + K_o)$$

The constrained modulus, D, is found from the slope of the average vertical stress versus vertical strain graphs

$$D = \Delta\sigma_{avg}/\Delta\varepsilon_v$$

D was determined using the slope of the unloading and reloading portions of the curve between the transition from the unloading to the loading phase and an average vertical stress of 110 kPa (16 psi). This stress was used because it is the smallest vertical stress from all the tests that was observed at a horizontal stress of 69 kPa. Stresses in this range are typical of those encountered in highway applications. The unloading and reloading portion of the curve was used because this is closest to the deformation behavior that will be encountered under repetitive vehicle loading.

Young's modulus, E, can be determined with the following relationship from a work by Lambe and Whitman (*11*) using the constrained modulus and Poisson's ratio

$$E = (1 + \mu)(1 - 2\mu)D/(1 - \mu)$$

The elastic parameters are given in Table 3. By examining K_o and μ and recalling the amount of exposed steel belt in the tire chips from the different suppliers, it is seen that these parameters decrease with increasing exposed steel belt. The implication is that tire chips with a significant amount of exposed steel belt would produce lower horizontal stresses on retaining walls.

It is instructive to compare the K_o and μ for tire chips with values typical for granular soils. The average K_o values for tire chips were from 0.26 to 0.47 as compared with typical K_o of normally consolidated granular soils of 0.35 to 0.50 (*12*). Thus, only the K_o for the Palmer sample falls below the typical range for granular soils. Typical μ for granular soils were from 0.15 to 0.45 (*13*). The average values for tire chips (0.20 to 0.32) fall in the lower half of this range. For comparison the μ of solid tire rubber is 0.5 (*14*).

The constrained modulus of the tire chips was from 1270 kPa (184 psi) for the F&B tire chips to 1680 kPa (244 psi) for the Palmer tire chips. Young's modulus was from 770 kPa (112 psi) for the F&B chips to 1130 kPa (165 psi) for the Pine State chips. This suggests that small glass-belted tire chips have lower unloading and reloading modulus than mixtures of larger glass- and steel-belted tire chips. For comparison, the Young's modulus of the tire rubber itself is from 1240 to 5170 (180 to 750 psi) (*14*) and for granular soils typically from 10,000 to 170,000 kPa (1,500 to 25,000 psi) (*13*). Thus, the Young's modulus of tire chips is two to three orders of magnitude less than the modulus of granular soils typically used

TABLE 3 Elastic Parameters of Tire Chips

Supplier	Test No.	K_0	μ	D (kPa)	E (kPa)
Pine State Recycling	1	0.55	0.35	1340	830
	2	0.33	0.25	1690	1390
	3	0.34	0.25	1390	1160
	Average	0.41	0.28	1470	1130
Palmer Shredding	1	0.29	0.22	790	700
	2	-----	-----	2510	-----
	3	0.22	0.18	1740	1530
	Average	0.26	0.20	1680	1120
F&B Enterprises	1	0.40	0.29	1040	480
	2	0.55	0.36	1240	740
	3	0.45	0.31	1520	1100
	Average	0.47	0.32	1270	770

1 kPa = 0.1450 psi

as a base beneath paved roads. The implication of this is that 0.6 to 1.8 m (2 to 6 ft) of conventional soil fill is needed on top of the tire chip layer to prevent excessive deflections of the overlying pavement. Additional discussion of this statement is in works by Manion and Humphrey (*6*) and Humphrey and Manion (*7*).

CONCLUSIONS

Several conclusions can be drawn from this research.

1. Gradations of the tire chips from the three suppliers show that the chips were uniformly graded from 13 to 76 mm (0.5 to 3 in.) in size.
2. The specific gravity of the tire chips was slightly greater than that of water and ranged from 1.14 to 1.27. Tire chips composed entirely of glass-belted tires have a lower specific gravity than those composed of a mixture of glass- and steel-belted tires.
3. The compacted dry densities of the tire chips were in a narrow range of 0.618 to 0.642 Mg/m^3 (38.6 to 40.1 pcf), which clearly shows the potential of tire chip use as lightweight fill.
4. Compression tests indicate that the tire chips are highly compressible during the initial portion of the first loading cycle but that the compressibility is significantly less during subsequent unloading and reloading cycles.
5. The friction angle of the tire chips was between 19 and 25 degrees and the cohesion between 8 and 11 kPa (160 and 240 psf).
6. The amount of exposed steel belt appears to have a systematic effect on some of the engineering properties of tire chips. Large amounts of exposed steel belts tend to cause higher compressibility during the first loading cycle, higher Young's modulus during unloading and reloading cycles, lower coefficient of earth pressure at rest K_o, and lower shear strength.
7. These laboratory results suggest that there may be some advantage to using tire chips with large amounts of exposed steel belt as retaining wall backfill because they have a lower K_o.

ACKNOWLEDGMENTS

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Use of Shredded Tires for Lightweight Fill

RICHARD J. UPTON AND GEORGE MACHAN

Shredded waste tires were used as lightweight fill to repair a landslide in a highway improvement project in southwest Oregon. Approximately 580,000 shredded waste tires were trucked to the site from four different sources 240 to 440 km (150 to 275 mi) away. The tires were placed and compacted with a dozer, then capped with 0.9 m (3 ft) of soil and a pavement section with 20 cm (8 in.) of asphalt pavement over 58 cm (23 in.) of aggregate base course. The in-place shredded tire fill cost $\$16.82/m^3$ ($\$12.87/yd^3$), which included a significant rebate from the Department of Environmental Quality. Without the rebate, the cost would have been $\$35.16/m^3$ ($\$26.91/yd^3$). The shredded tire fill was instrumented and monitored for 1 year following installation. Instrumentation included inclinometers, piezometers, settlement plates, and survey hubs. Falling weight deflectometer tests were also performed. The shredded tire fill compressed linearly in relation to surcharge load as the soil cap and pavement section were placed. Compression appears to be related to shredded tire fill thickness. Creep or compression under traffic loading occurred during the monitoring period. The compacted density of the shredded tires varied from 730 to 845 kg/m^3 (45 to 53 pcf) at various stages of compaction and surcharging. A standard asphalt pavement with aggregate base was adequate over the shredded tire fill. The shredded tire embankment represents a softer subgrade condition than do surrounding soil embankments. However, pavement deflections were considered within acceptable limits after 20.3 cm (8 in.) of asphalt pavement was in place.

A landslide associated with highway embankment construction was repaired with lightweight fill constructed of shredded tires. This use of waste tires was experimental, and a program was established to monitor installation and performance of the shredded tire fill.

This paper presents the results of the monitoring program and discussions of background information, remedial design, construction, monitoring, and performance of the shredded tire fill.

BACKGROUND

In the United States each year 240 million tires are discarded. Federal regulations limit disposal, and waste tires accumulate throughout the country, with the current stockpile estimated at 2 billion. Beneficial uses for the stockpile are continually being sought.

In 1986 the Minnesota Department of Forestry demonstrated the feasibility of using shredded waste tires as lightweight fill in roadway embankment construction (*1*). The application at the Minnesota installation was intended to limit embankment settlement over soft foundation soils.

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Reducing embankment loads by using lightweight materials is also an accepted landslide repair technique. Lightweight embankments constructed of shredded tires represent a beneficial use for waste tires.

LANDSLIDE REPAIR

Design

As part of an improvement project on U.S. Highway 42 in southern Oregon (Figure 1) an existing highway embankment 3.3 m (11 ft) deep was widened 6.1 m (20 ft) and raised 1.2 m (4 ft). The additional embankment load remobilized an old landslide that moved progressively downslope perpendicular to the highway. The approximate extent of the slide is given in a plan view of the site in Figure 2.

A geotechnical investigation showed that slide movement could be arrested by reducing embankment load and adding a downslope counterbalance (*2*). The specific design was to replace embankment soils with lightweight fill and use the excess soils to construct the counterbalance. Sawdust and shredded tires were considered for the lightweight fill. Shredded tires were selected because there was concern about deterioration of sawdust. Shredded tire material costs were favorable because there is a state rebate for beneficial use.

The repair design also included a rock blanket and french drain system to maintain the groundwater level below the shredded tires. A cross section of the proposed repair design is given in Figure 3.

Environmental Considerations

The shredded tire fill was considered a solid waste disposal site by the Oregon Department of Environmental Quality (DEQ). A formal authorization process was required.

DEQ referred to a Minnesota Pollution Control Agency study (*2*) to review disposal plans. DEQ cited a potential for contamination of the groundwater in contact with the tire chips. DEQ approved the disposal plan, which included the rock blanket and french drain system to isolate the shredded tires from groundwater.

For several years, DEQ has had a program in place to collect a \$1 per tire fee upon disposal. This fee was used for programs to encourage beneficial use of waste tires. At the time of shredded tire fill construction, DEQ had in place a program to reimburse beneficial users \$22/Mg (\$20/ton). This reimbursement program made the cost of shredded tires competitive and resulted in the use of shredded tires in the landslide repair.

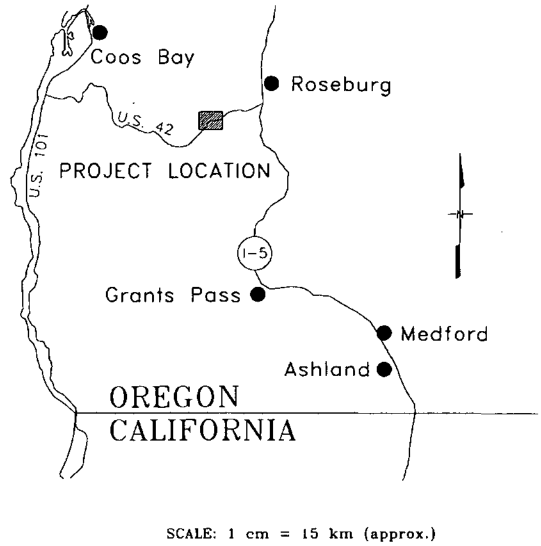

FIGURE 1 Vicinity map.

SHREDDED TIRE CONSTRUCTION

Scheduling

The landslide repair was part of the ongoing highway improvement project. Completion of the repair by fall 1990 was required to support overall project scheduling. Shredded tire fill construction took place in two phases to allow continuous highway traffic. Earthwork began on the landslide repair in June 1990. Shredded tire fill construction milestones are given on the time line in Figure 4.

Acquisition

The proposed design required approximately 580,000 waste tires to generate the 6400 Mg (5,800 tons) of lightweight fill material. No single source in the region had this quantity, so shredded tires were bought from four different vendors 240 to 440 km (150 to 275 mi) from the site. The tires were shredded at the vendor locations and trucked to the site in 76 m^3 (100 yd^3) trucks. Transport of the shredded tires to the site from the remote vendors was a critical scheduling item. Shredded tire transport began at the same time as site preparation. The tire chips were stockpiled near the landslide repair.

Manufacturing

The waste tire shredding process involves feeding tires through a series of rotating blades that compress and slice the tires into smaller pieces or chips. The chips are screened and sorted by size. Chips are fed through the shredder several times and screened to approximately 5 cm (2 in.) to provide a uniform product. Wire can be removed from smaller chips frequently used as low-grade fuel.

The shredded tire chip specification for the project was taken from work done in Minnesota (*1*). The specification dealt primarily with chip size and wire encasement. The size specification required 80 percent to be smaller than 20 cm (8 in.) and 50 percent to be larger than 10 cm (4 in.). The maximum size was 61 cm (24 in.).

To meet the chip specification, tires were passed through the shredder once. This process produced chips that substantially met the specification, but excursions in maximum size and exposed wire were common. Dull shredder blades of one

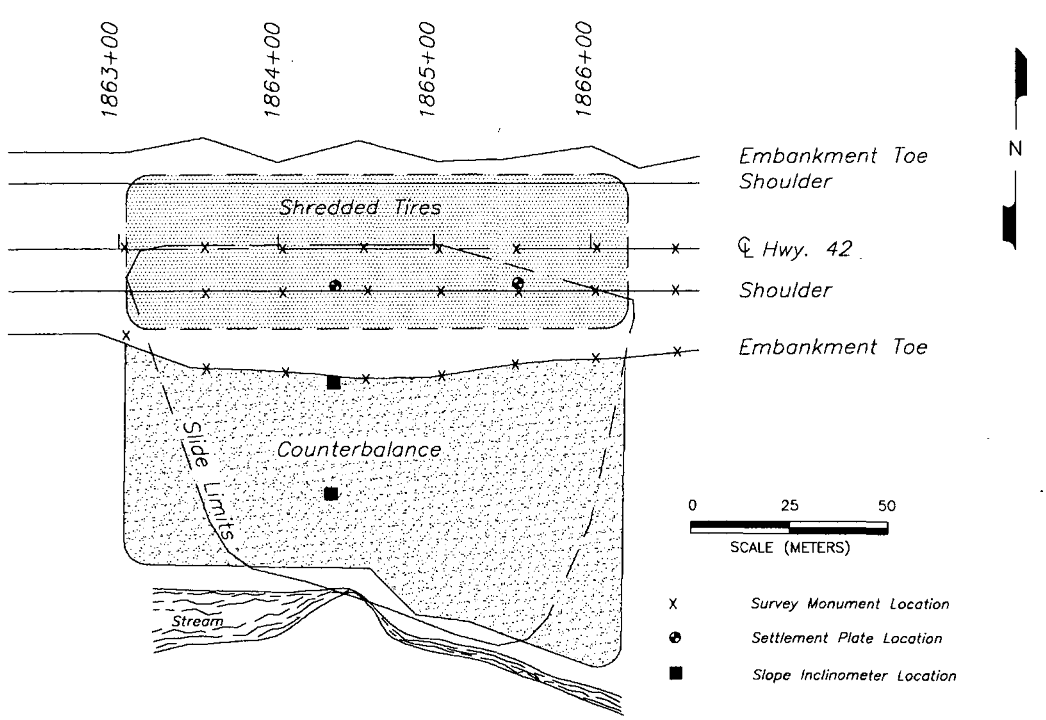

FIGURE 2 Site plan view.

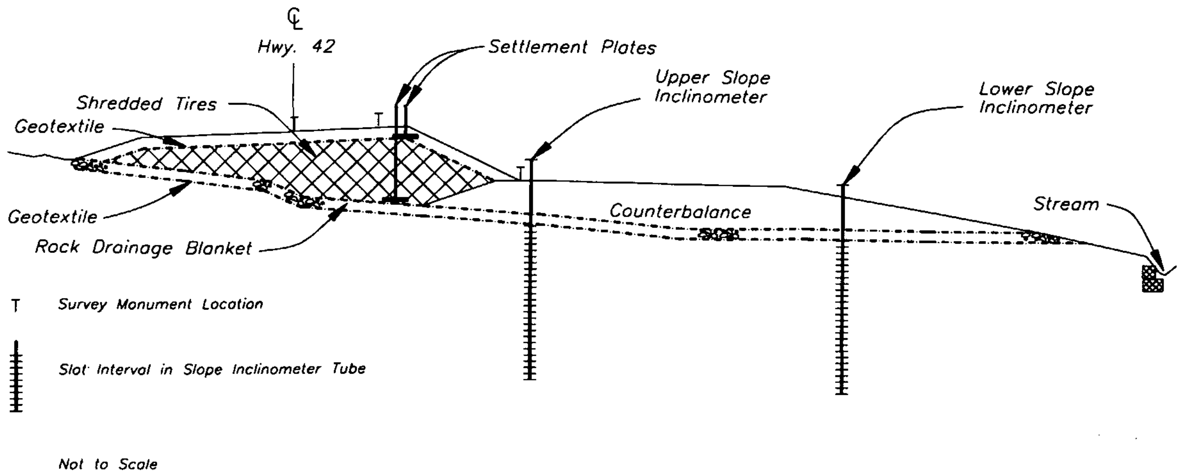

FIGURE 3 Typical cross section.

of the vendors increased excursion rate. Further processing of the chips could have resulted in tighter adherence to the specification. This was considered but not pursued because it would have added to the cost. The excursions were not considered detrimental to shredded tire fill performance.

Placement

The embankment foundation area was prepared by dozing and placing the rock blanket. Tire chips were moved from the stockpile to the fill in 7.6-m^3 (10-yd^3) dump trucks and dropped at one end of the prepared area. Dump trucks were not routed over the in-place shredded tires to avoid tire puncture from exposed wires in the shredded tire chips.

Compaction

The tire chips were spread in 0.9-m (3-ft) lifts and compacted with a D-8 dozer. The dozer was routed back and forth longitudinally on the shredded tires until at least one track pass had been accomplished everywhere. The dozer was then routed back and forth transversely until one track pass was again

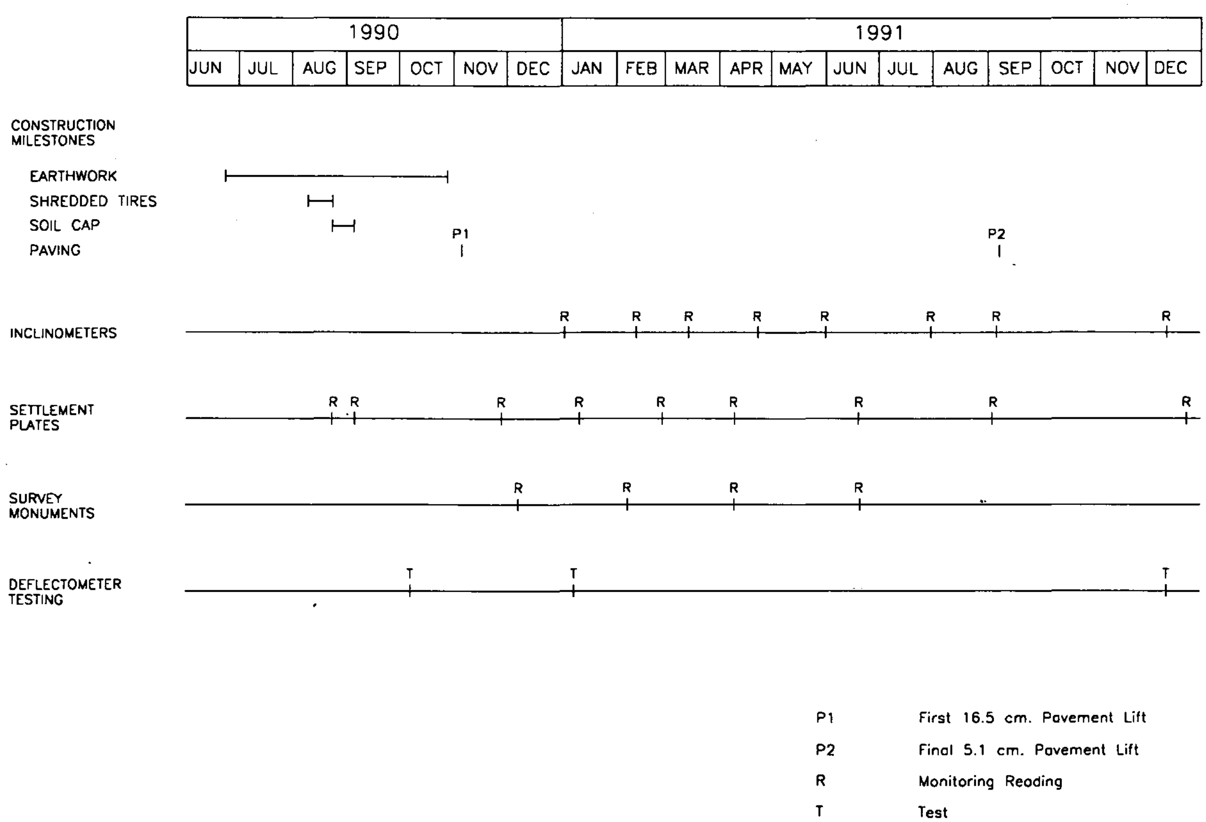

FIGURE 4 Time line.

accomplished everywhere. Full coverage in either direction was considered to be one compaction pass. At least three compaction passes were completed for each lift.

A test lift was compacted with a D-6 dozer in a similar routine. The lift compacted with the lighter dozer was visibly looser and could be compacted further with the heavier D-8 dozer. The lighter D-6 dozer was not used for compaction.

Compaction was also attempted by a series of in-place turning maneuvers or squirming with the D-8 dozer. This tended to loosen already compacted shredded tires and was discontinued.

Slope Trimming

Final trimming of the shredded tire side slopes was attempted with a dozer, but this resulted in a rough, uncompacted surface. Final trimming was successfully achieved by overbuilding the slope approximately 0.3 m (1 ft) and trimming with a hoe-type excavator situated at the top of the tire embankment. The excavator was equipped with a "thumb" bucket used in a grabbing motion. The resulting surface was relatively smooth and compact.

Geotextile Placement

The geotextile (Figure 3) was placed on top of the tires to separate the chips and soil cap. Field joints were attempted by lapping the geotextile 0.9 m (3 ft), but the joints tended to separate as soil was placed and compacted. To overcome this tendency, brass "hog-ring" clips were used to pin the joints together. The panel edges were overlapped 0.3 m (1 ft), and the clips were placed at 1.8-m (6-ft) intervals along the joints. The clips successfully prevented field joint separation.

Soil Cap Placement

The 0.9-m (3-ft) soil cap over the shredded tire fill was placed using standard 20.3-cm (8-in.) maximum lift thickness. Compaction requirements were 95 percent of maximum density as determined by standard Proctor except for the first lift, which was 90 percent. Compaction was achieved with an Ingersoll-Rand LD 150 compactor. During compaction of the first lift the earth cap deflected significantly, but 90 percent compaction was achieved with normal compactive effort. With each additional lift of capping soil, aggregate base, or asphalt, the deflections became progressively smaller.

The south slope of the shredded tires (right side, Figure 3) had a rise of approximately 3 m (10 ft). On this slope, the soil cap was placed and compacted in standard lifts approximately 3 m (10 ft) wide. The resulting vertical soil cover thickness was approximately 1.5 m (5 ft).

The north slope (left side, Figure 3) had a rise of less than 0.9 m (3 ft). The slope angle was designed flat enough to allow the soil cap to be constructed as part of the cap on the top of the tire fill.

Vertical Cut Performance

The shredded tire fill was constructed in two phases to accommodate highway traffic during construction. When the first phase of the fill was complete, the soil cap and aggregate base were placed to facilitate highway traffic. A 2.4-m (8-ft) vertical face was cut along the first phase of in-place tire chips to prepare for the second phase of tire chip placement. Highway traffic was routed over the first phase fill for 28 days. The pavement and shredded tires visibly deflected under truck traffic; however, no permanent deflection or distress was observed.

Costs

The shredded tires delivered to the site cost \$33/Mg (\$30/ton). The \$22/Mg (\$20/ton) DEQ reimbursement resulted in a net cost of \$11/Mg (\$10/ton) for shredded tires. The cost of placing and compacting the shredded tires was \$9.18/Mg (\$8.33/ton). Consequently, the net cost of the in-place shredded tire fill was \$20.18/Mg (\$18.33/ton), which equals \$16.82/m^3 (\$12.87/yd^3).

The DEQ reimbursement program was a significant factor in selecting shredded tires for the lightweight fill in this installation. The cost of the installation without the reimbursement would have been \$42.18/Mg (\$38.33/ton) or \$35.16/m^3 (\$26.91/yd^3). At this higher cost, other lightweight materials, such as sawdust, had a cost advantage. With the reimbursement program, the cost of the in-place shredded tire embankment was competitive with rock-fill embankment construction.

Construction Challenges

A major construction challenge was the impact of wire strands exposed on the shredded tire chips. These frequently punctured tires on construction equipment and prevented haul trucks from being routed over the fill. The placement sequence required an additional step to spread the chips, resulting in lost efficiency and extra cost. Shredded tire chips were also scattered throughout the stockpile area and dropped along the haul route, creating a continual puncture hazard.

Tighter adherence to the encasement requirement at the shredding plant might have reduced, or eliminated, this problem. This might have been achieved with sharper shredder blades. Communication about shredded tire chip quality was difficult without an inspector at the vendor plants. If the excursions from the specifications had been more severe, an inspector at the plant would have been necessary.

PAVEMENT SECTION CONSTRUCTION

Pavement design was based on the structural requirements of the natural subgrade materials surrounding the shredded tire embankment. The design section was 20.3 cm (8 in.) of asphalt pavement over 53.3 cm (21 in.) of aggregate base. The pavement section was constructed in phases, allowing several opportunities for observations and testing.

The full depth of aggregate base course was placed at the same time as the soil cap to facilitate highway traffic staging. Fill compression resulted in placement of approximately 58.4 cm (23 in.) of aggregate base to achieve design grade. An asphalt surface coat was sprayed on the aggregate base and performed well during 2 months of highway traffic.

The first lift of asphalt pavement was placed in January 1991. Lift thickness was 15.2 cm (6 in.). Shortly after placement, cracks were seen in a wheelpath over the shredded tire fill. The cracks propagated along the wheelpath and rutting also began. Drainage improvement failed to stem the deterioration. By late summer the rutting and cracking had affected the entire width of the lane for approximately 15.2 m (50 ft) over the shredded tire fill. The area was excavated to the top of the shredded tires. The soil cap in this area was only 0.5 to 0.6 m (1.5 to 2 ft) thick. The shredded tires were then excavated down to accommodate the full 0.9-m (3-ft) soil cover, and the pavement section was restored. After the repair, the final 5.1 cm (2 in.) of asphalt pavement was placed.

SHREDDED TIRE FILL MONITORING AND TESTING

During and after construction, the shredded tire fill was instrumented to aid in assessing performance (*3*). Monitoring devices included two inclinometers, two settlement plate installations, and three rows of survey monuments. The inclinometer casings were slotted and used for piezometers. Monitoring device locations are given in Figure 2. A typical cross-section view of monitoring devices is given in Figure 3. Pavement testing was performed with falling weight deflectometers. The various instruments were installed and monitored on different schedules. A summary of the timing of monitoring activities is given in Figure 4.

Inclinometer Installations

Inclinometer installations consisted of commercially prepared casing, which was also slotted to act as a piezometer. The distance from ground surface to the top of the groundwater table was measured and plotted versus time (Figure 5).

Settlement Plate Installations

Settlement plate installations consisted of one plate at the bottom of the shredded tire layer and one at the top. Settle-

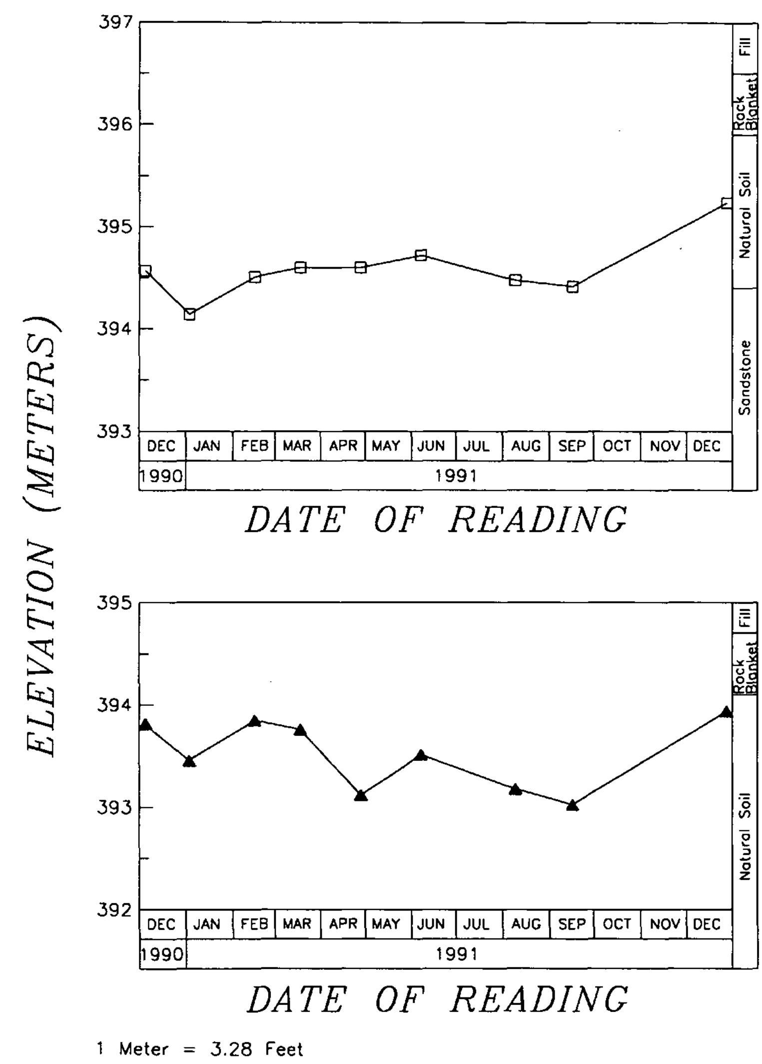

FIGURE 5 Water level data: *top*, upper slope inclinometer; *bottom*, lower slope inclinometer.

ment plate data were reduced to yield plots of shredded tire fill thickness versus time (Figure 6). The data were further analyzed to yield shredded tire compression versus surcharge load (Figure 7).

Settlement plate data and measurements taken during construction facilitated computation of shredded tire densities in several different conditions. The conditions were loose in haul vehicles before and after hauling, compacted by dozer, surcharged by soil cap and pavement section, and final after 1 year of traffic. The computed densities are given in Table 1.

Loose Density

Average loose densities were calculated using weights and dimensions taken from one long-haul truck load from three different vendors. Two trucks were measured immediately after loading, and one truck was measured after a 64-km (40-mi) haul.

The loose density of the shredded tire material depended on chip size. Larger chips resulted in lower densities. The material weighing 485 kg/m³ (30 pcf) at loading is most repre-

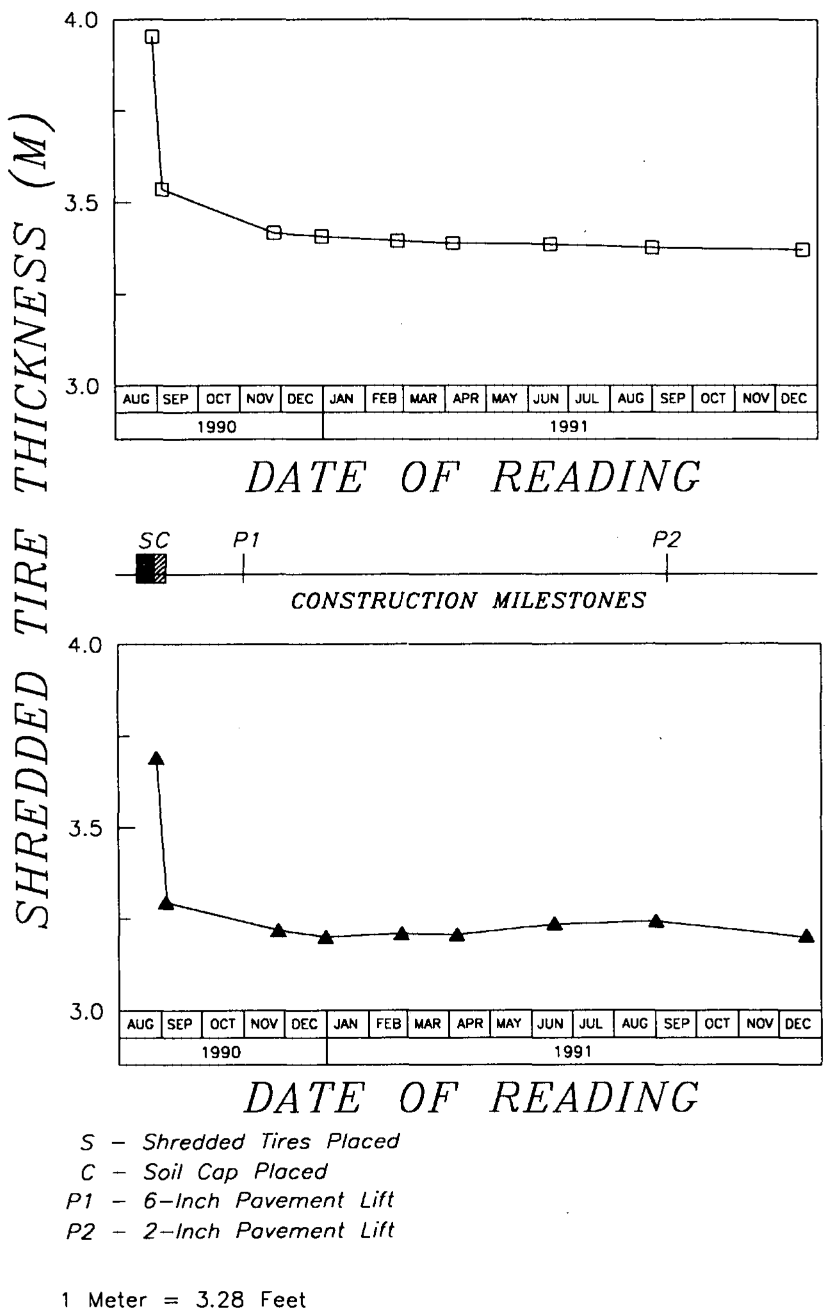

FIGURE 6 Settlement plate data: *top*, Station 1864 + 33.5; *bottom*, Station 1865 + 50.

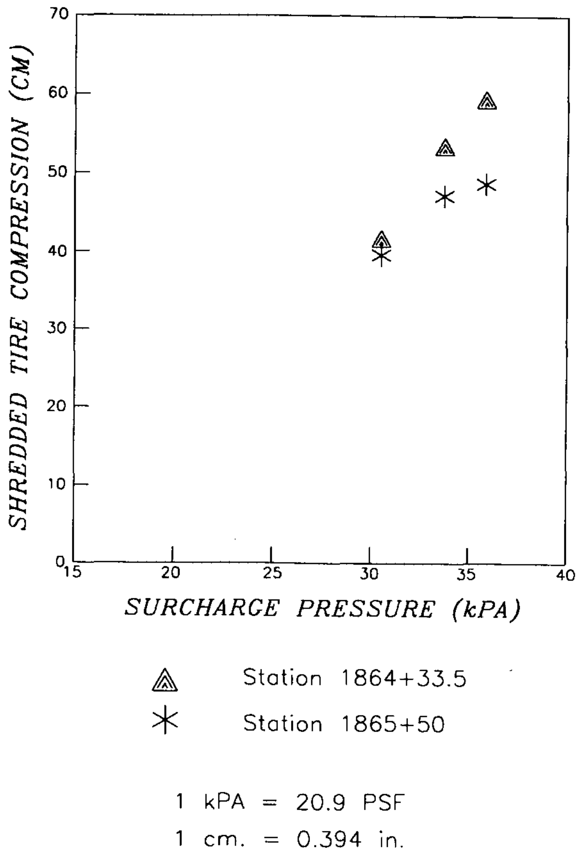

FIGURE 7 Shredded tire compression versus surcharge.

sentative of that intended by the specifications. This density is consistent with 470 kg/m³ (29 pcf) loose density estimated from the data for the Minnesota project (*1*). The computed 390 kg/m³ (24 pcf) density appeared to be the result of oversize material and should not be considered typical.

Density increased approximately 10 percent after hauling 64 km (40 mi). Density was not estimated after the full haul distances of 240 to 400 km (150 to 250 mi). Density appeared to increase with greater haul distances.

Compacted Density

The average compacted density was estimated by dividing the weight of shredded tires incorporated in the embankment by the volume occupied by the tires at the end of compaction. Weight was estimated from quantities delivered minus the reject and excess. Volume was estimated from cross sections on 15.2-m (50-ft) centers before placement and after compaction.

Surcharged Density

The average surcharged density was estimated following loading with 0.9 m (3 ft) of soil, 58.4 cm (23 in.) of aggregate base, 15.2 cm (6 in.) of asphalt pavement, and 3 months of highway traffic. This density was determined by adjusting the volume of compacted tires to compensate for the compression of the tire fill. Compression was measured at settlement plate locations and extrapolated to the rest of the fill by assuming that settlement was directly proportional to shredded tire thickness.

Final Density

Compression associated with the final 5.1-cm (2-in.) pavement lift and 1 year of creep settlement was also estimated by adjusting previous estimates for settlement.

Survey Monuments

Survey monuments consisted of driven pins spaced at 15.2-m (50-ft) intervals long three lines (Figure 2). Survey monument data were reduced to yield plots of deflection versus station for each of the three lines (Figure 8). The deflection shown in the plots is the total deflection measured from December 1990 to June 1991. After June, no additional measurements were taken before the hubs were paved over in September 1991.

TABLE 1 Shredded Tire Densities

Condition	Density
Loose Density (as loaded in trucks)	390 - 485 kg/m³ (24 - 30 pcf)
Loose Density (after 64 km haul in trucks)	535 kg/m³ (33 pcf)
Compacted Density (after three dozer passes)	730 kg/m³ (45 pcf)
Surcharged Density (after final pavement lift)	845 kg/m³ (52 pcf)
Final Density (after 1 year of compression)	860 kg/m³ (53 pcf)

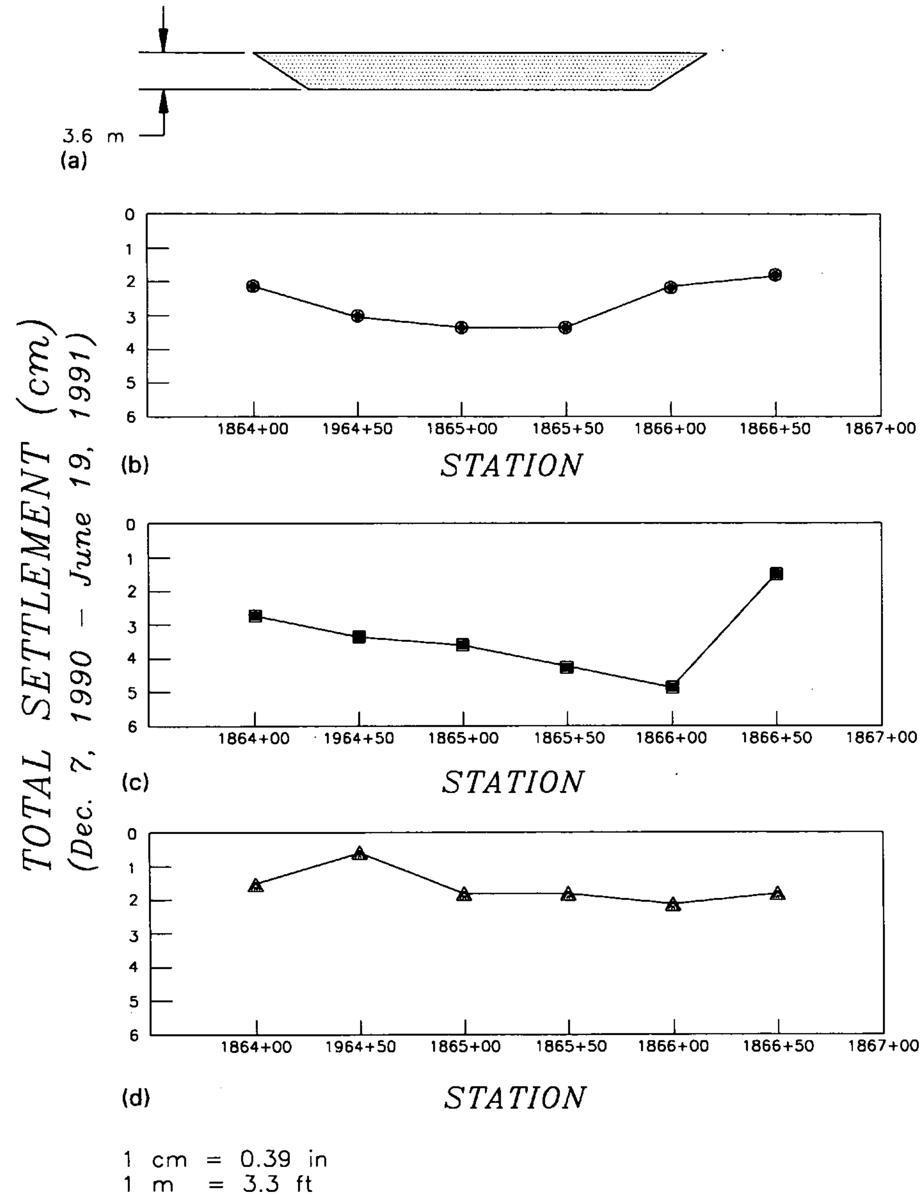

FIGURE 8 Survey monument data: (*a*) approximate compacted shredded tire thickness at shoulder, (*b*) centerline, (*c*) shoulder, (*d*) embankment toe.

Deflectometer Tests

Falling weight deflectometer tests, commonly used to estimate overlay thickness in road surface rehabilitation projects, consist of dropping a weight equivalent to 40 kN (9,000 lb) onto the highway surface and measuring deflection near the impact point. The tests were conducted at the landslide repair site at approximately 15.2-m (50-ft) intervals along the eastbound lane over the shredded tire fill and beyond each end. Deflection test results for each of three test dates were plotted versus highway station (Figure 9).

OBSERVATIONS

The project engineer made periodic site visits to inspect the installation. The embankment over the shredded tire fill section consistently appeared to be performing well, with no signs of settlement, sloughing, or erosion. Embankment slopes retained their shape. No ground surface movement or cracks were observed that would suggest landslide activity. Inclinometer data also confirm that the landslide at the shredded tire fill is no longer moving.

During site visits at the various stages of pavement section construction, vibrations associated with truck traffic could also be felt. When the aggregate base course was in place, heavy commercial trucks would cause vibrations of the fill felt while each truck was over the shredded tires. After placement of the first lift of asphalt pavement the vibrations were less perceptible but could still be felt. After placement of the final asphalt pavement lift, heavy trucks could only be felt as their wheel passed within 6 m (20 ft) of the observer.

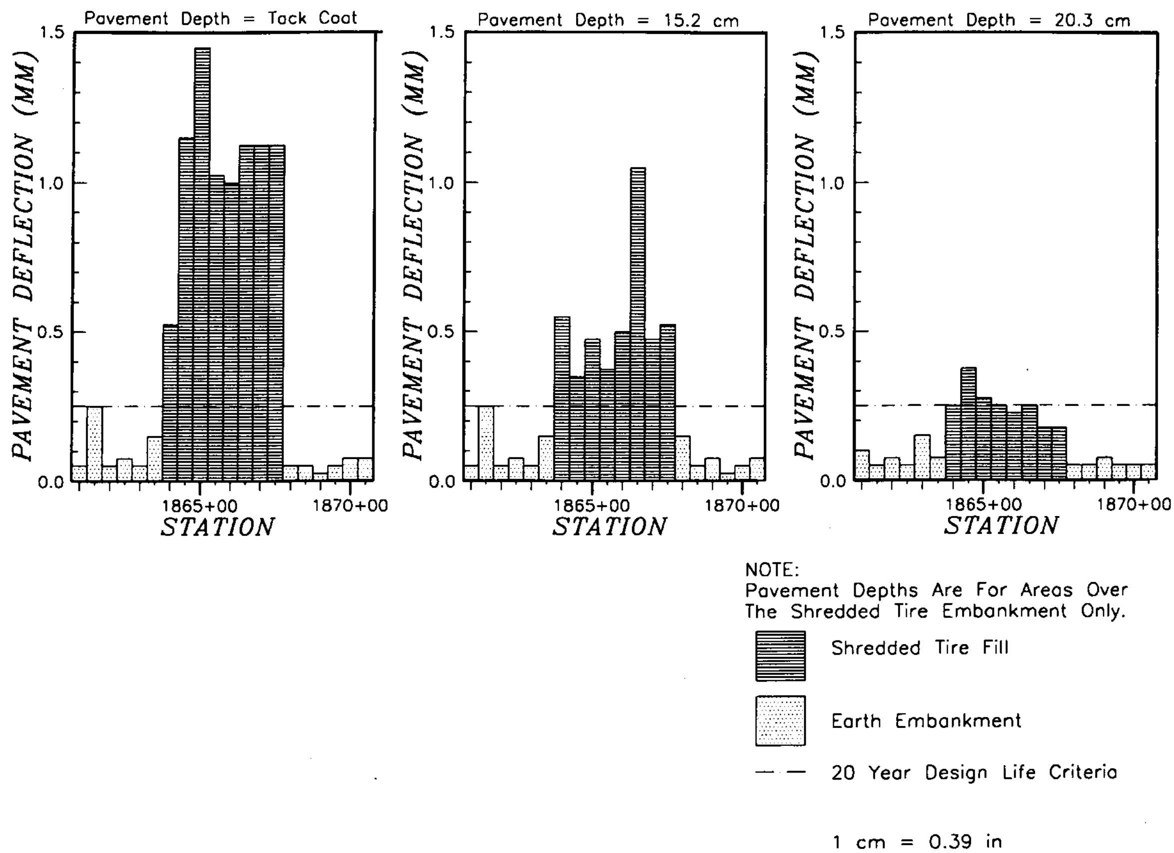

FIGURE 9 **Deflectometer test data:** ***left*****, October 30, 1990;** ***middle*****, January 7, 1991;** ***right*****, December 12, 1991.**

DISCUSSION OF RESULTS

Shredded Tire Fill Compression

Shredded tire fill compression under surcharge loading was anticipated from experience at the Minnesota installation (*4*) where 2.7 m (9 ft) of shredded tires compressed approximately 10 percent under a 1.2-m (4-ft) soil cap.

At the landslide repair site, settlement plate data indicate that the 3.6-m (12-ft) shredded tire fill compressed 15 percent under the soil and pavement surcharge. A plot of shredded tire fill thickness versus time (Figure 6) shows compression in response to various stages of surcharge loading. A plot of shredded tire fill compression versus surcharge load is given in Figure 7.

The settlement plate data reflected compression of 1.3 and 3.1 (0.5 and 1.2 in.) under constant surcharge between January and September 1991. This indicates some type of creep compression or compression associated with traffic loading occurred in the shredded tires.

The survey monument data (Figure 8) indicate greater settlement near the center of the shredded tire fill and less toward the ends. The shredded tire fill is thicker near the center and thinner toward the ends, indicating a correlation between shredded tire thickness and compression.

Density Comparison

Table 2 gives a comparison of compacted and surcharged densities measured in this investigation and reported in other investigations.

The Minnesota installation (*1*) consisted of 2.8 m (9 ft) of shredded tires capped with 1.2 m (4 ft) of soil and aggregate. Chips size and compactive effort were similar to this investigation.

The laboratory test samples were 25.4 cm (10 in.) in diameter and 25.4 cm (10 in.) high. The chips used in the laboratory testing were 5 cm (2 in.) and smaller. Compactive effort simulated standard and modified Proctor tests (ASTM D698 and D1557).

Pavement Deflection

The first falling weight deflectometer test sequence was performed over the aggregate base course. It confirmed that an asphalt pavement section could be constructed over the shredded tire fill. Deflections were on the order of 1 m (0.04 in.).

The second test sequence was performed on the first lift of asphalt pavement and used as a design aid in selecting the

TABLE 2 Comparison of Compacted and Surcharged Densities

Installation	Compacted Density	Surcharged Density
This Investigation	730 kg/m^3 (45 pcf)	845 kg/m^3 (52 pcf)
Minnesota Installation (1)	550 kg/m^3 (34 pcf)	615 kg/m^3 (38 pcf)
Laboratory Testing (4)	650 - 665 kg/m^3 (40 - 41 pcf)	-[a]

[a] No data

final pavement lift thickness. It was decided to add 5.1 cm (2 in.) more pavement in the final lift. The third test sequence was used to confirm performance of the completed pavement section.

Results from the second and third test sequences displayed a greater deflection over the shredded tire fill relative to the surrounding earth fill. On the basis of these deflection measurements, the shredded tire fill appears to represent a softer subgrade than does the surrounding earth embankment. The deflection magnitudes measured in the third test sequence meet Oregon Department of Transportation (ODOT) criteria for a 20-year pavement design life.

Groundwater Levels

The groundwater levels were monitored in the slotted inclinometer casing. A plot of groundwater levels for the monitoring period is given in Figure 5. Groundwater levels were consistently 0.6 to 1.8 m (2 to 6 ft) below the bottom of the rock blanket at the inclinometer installations. This indicates that the rock blanket and french drain system were successful in maintaining the groundwater level below the shredded tires.

SUMMARY

As part of a landslide repair project on a highway in southern Oregon, a lightweight fill was constructed of shredded tires. The embankment was instrumented and monitored for 1 year after construction.

Deflectometer tests indicate that the pavement section over the shredded tire fill meets 20-year design life criteria; however, it deflects more than a similar pavement section over earth embankment.

ACKNOWLEDGMENTS

The authors thank FHWA for funding this project as part of the Experimental Projects program. The authors also thank Tim Dodson of ODOT for designing the shredded tire fill and initiating the monitoring program. Joe Thomas (ODOT) also contributed to the field work as construction inspector.

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Use of Wood Fiber and Geotextile Reinforcement To Build Embankment Across Soft Ground

TONY M. ALLEN AND ALAN P. KILIAN

A case history describing the use, by the Washington State Department of Transportation, of wood fiber and geotextile reinforcement in combination to build a lightweight fill across very soft ground is presented. The fill was completed in 1988 as part of a newly constructed two-lane highway, State Route 109 Spur, west of Hoquiam, Washington. The fill, 180 m long and 13.4 m high, was built over organic sandy and clayey silt up to 15.2 m thick having an undrained shear strength of 7.2 kPa and SPT values of 1/61 cm. A conventional earth fill was not feasible for stability reasons. Wood fiber was used to reduce the driving forces, to enhance stability, and to reduce fill settlement to an acceptable magnitude. Five layers of geotextile were used to prevent lateral spreading and enhance stability. The geotextile layers were in the bottom 2.1 m of the fill. The rate of fill placement was controlled to take advantage of strength gain in the foundation soil to enhance stability. Total settlement of the fill over 2 years, before paving, was 1.2 m. The fill was allowed to settle approximately 1.1 year before paving. The use of the wood fiber with geotextile reinforcement was $730,000 less costly than a bridge.

This paper describes the design and construction of a portion of new highway, State Route 109 Spur, by the Washington State Department of Transportation (WSDOT) in a coastal area of western Washington. The new 2.86-km segment of two-lane highway begins just outside the western city limits of Hoquiam, Washington, and extends northeasterly connecting with SR-101 (Figure 1). Grading construction at the site began in late 1986 and was complete in fall 1987. The roadway was paved in late 1988.

The southern end of the highway begins near sea level and traverses undeveloped timber and swampland. Initially the roadway makes a 56-m cut through a bluff. Then for about 180 m it crosses swampland, which is the subject site area, and continues through a cut, across the Little Hoquiam River, and then roughly parallels the Little Hoquiam River with sidehill cuts and fills to its terminus.

The design and construction of a lightweight wood fiber fill, reinforced with geotextile layers, built across the very soft valley soils are discussed. In addition, controlled rate of construction and instrumentation control were used to maintain stability and a delay period to mitigate settlement.

SITE DESCRIPTION

At the site area the roadway grade crosses about 13.4 m above the valley floor. The valley floor is at an approximate elevation of 2 m. Foundation soils are very soft and compressible, posing stability and settlement problems for any fills and necessitating deep foundation support systems for any bridge. An unnamed creek flows year round through the site. The water level in the creek is influenced by tidal action in Grays Harbor and with winter rainfall results in frequent flooding of the valley.

SITE GEOLOGY

During the Eocene to middle Miocene epochs, thousands of feet of sedimentary rocks were deposited on the much older volcanic rocks already present in the Olympic Peninsula area of northwest Washington State. Subsequent deformation and uplift of these rocks during the middle to late Miocene epoch formed the Olympic Mountains.

Consequent erosion and deposition of the eroded material from the Olympic Mountains and the Willapa Hills occurred during the late Miocene to the Pliocene epochs. The lowlands bordering what is now Grays Harbor received the eroded sedimentary material. These sediments range from fluvial sands and gravels to fine-grained lacustrine silts and clays.

The hillsides on both sides of the site area are composed of fairly well indurated river-laid sand, gravel, and silt, as shown in Figure 2. Recent alluvium filled the valley bottom at the fill site area, overlying the sands and gravels.

SOIL CHARACTERIZATION

Soil conditions at the fill area consist of very dense silty gravelly sand overlain by about 5 m of dense silty sand, 3 m of loose sandy silt, and 12 m of very soft organic sandy silt. A cross section of the soil stratigraphy beneath the fill is shown in Figure 3. The surficial layer of organic sandy silt controlled design of the highway fill.

The average standard penetration test blow count for the organic sandy silt was less than one. The effective unit weight was from 1.38 to 4.52 kN/m^3, with an average of 2.99 kN/m^3. In-place moisture content varied from 94.3 to 363.9 percent, with an average of 171.9 percent. The liquid limit varied from 61 to 90 percent and the plastic limit from 53 to 65 percent.

The average unconsolidated undrained strength as determined from triaxial shear testing was 7.2 kPa. Field vane shear testing was unsuccessful because of the existence of fibrous peat, twigs, and roots. During construction a temporary earth

Washington State Department of Transportation, Headquarters Materials Laboratory, P.O. Box 47365, Olympia, Wash. 98504.

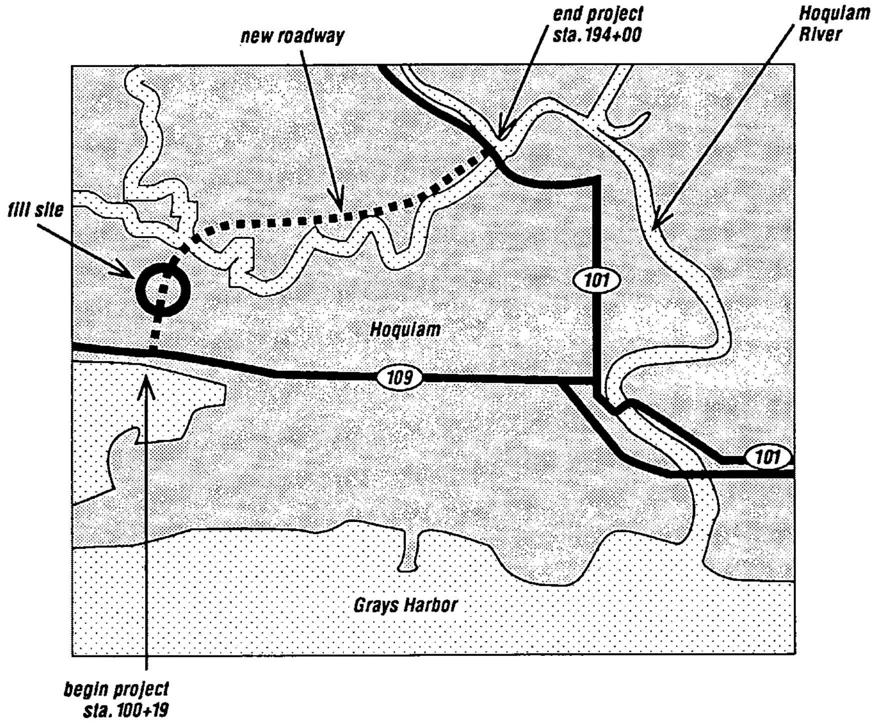

FIGURE 1 Project vicinity map.

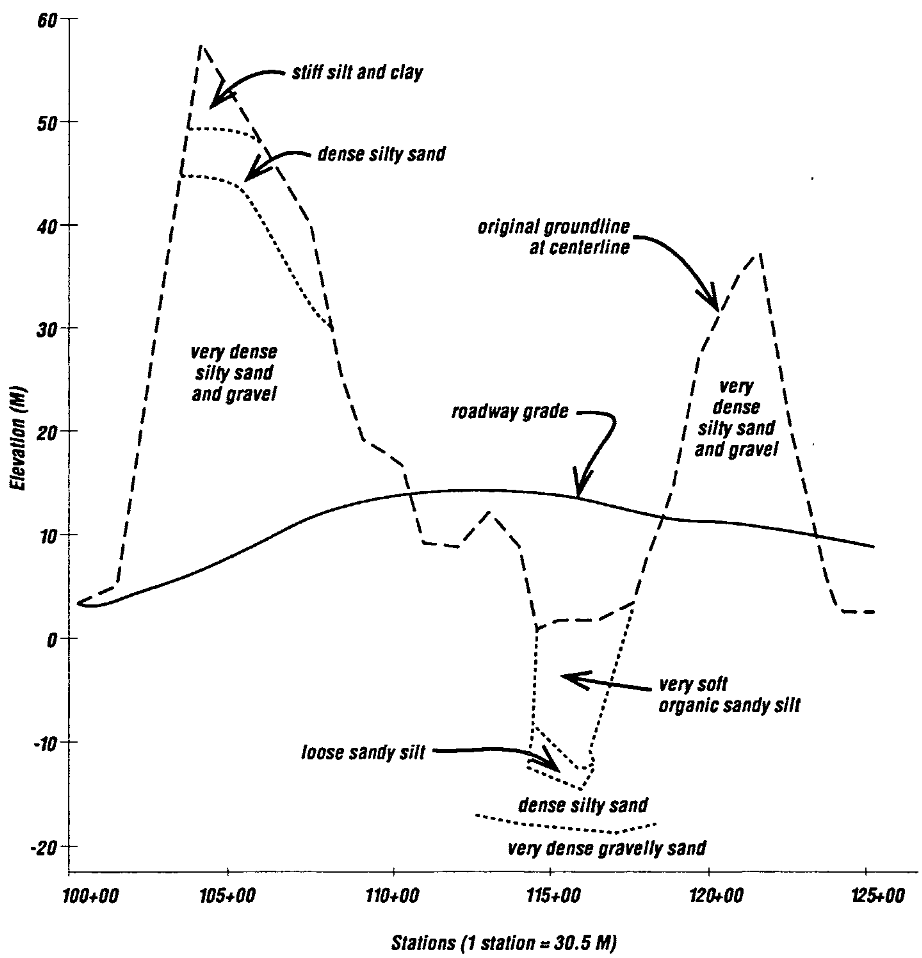

FIGURE 2 Generalized soil profile at site.

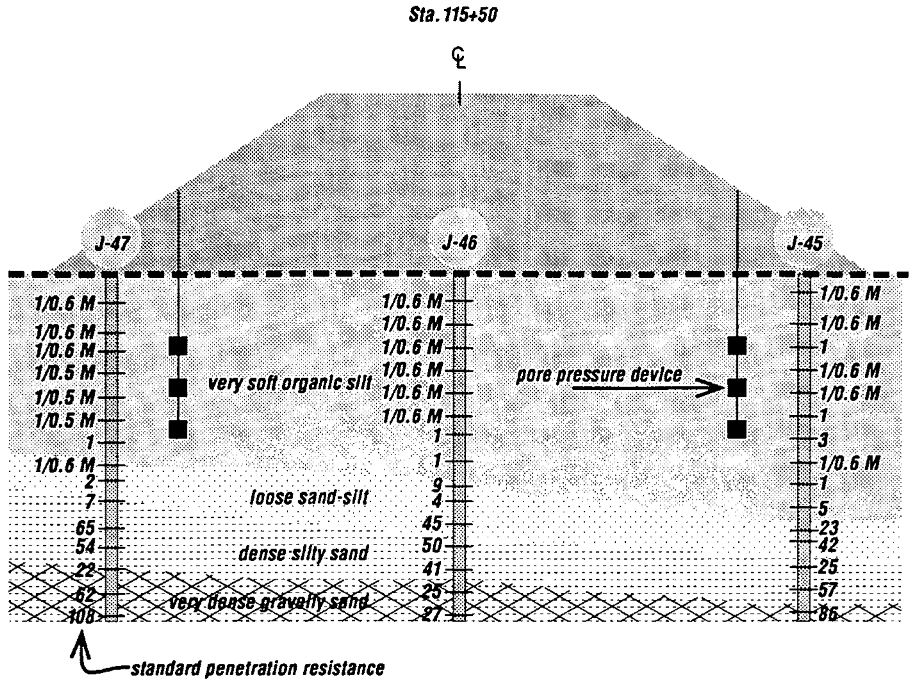

FIGURE 3 Soil cross section at wood fiber fill location.

stockpile was rapidly built 4.6 m high over the same geologic deposit about 1.6 km away, and failed. Back analysis of that fill failure yielded an undrained shear strength 10.6 kPa. The average undrained sheer strength used for the design was 7.2 kPa. On the basis of consolidated undrained triaxial tests, with pore pressure measurement, an effective angle of internal friction of 15 degrees was used.

Laboratory consolidation testing was also performed for the site. On the basis of test results, a compression index of 0.13, a coefficient of consolidation of 33.8 m^2/yr, and a coefficient of secondary compression of 0.008 were used for design.

DESIGN OF GEOTEXTILE-REINFORCED WOOD FIBER FILL

The stability of the fill was evaluated initially for two potential modes of failure—rotational slope stability and bearing capacity. Fill settlement was also considered. The total height of fill required, including surfacing, was to be 13.4 m to meet roadway grade requirements. Proposed side slopes were limited to 2H:1V to limit the right-of-way and fill volume required and to minimize the wetlands taken.

Through inspection and engineering judgment it became obvious that the use of granular soil for the entire fill would be impractical because of stability and settlement problems resulting from the soft, weak nature of the foundation soil. The fill construction rate needed to ensure stability would require the fill to be built slowly over 20 months to provide the necessary soil strength gain, which was considered to be impractical. Without strength gain of the foundation soil, the maximum height of granular fill that could safely be constructed was determined to be approximately 3.7 m, which correlated well with observations of the performance of previously built fills near the site 3.0 to 3.7 m high. Settlement of a granular fill 13.4 m high was estimated to be 2.4 m, with primary consolidation taking approximately 3 years to occur once fill construction began. This amount of settlement was considered to be excessive, especially considering that a culvert 1.5 m in diameter would be required at the base of the fill.

On the basis of this initial analysis, it was determined that alternative methods of fill construction would be required. Options considered included the use of lightweight wood fiber fill, controlled fill construction rates, geotextile reinforcement, or a combination of two or more of these. The final design selected for the embankment was a combination of all three. The design was optimized to minimize settlement, construction time, and the right-of-way and fill volume required, yet still provide a stable embankment with a minimum factor of safety of 1.25 for slope stability and 1.5 for bearing capacity.

Wood fiber could not be used to construct the entire fill. Environmental constraints dictated that the wood fiber must not extend below the mean high-water level. Because of the potential for a large settlement and this environmental constraint, the bottom 1.5 m of fill was constructed using a silty gravelly sand. A significant thickness of surfacing was also required, because of the compressibility of the wood fiber fill and heavy truck traffic, to provide acceptable roadway performance. A 1.2-m surfacing thickness was used for design. The remainder of the fill was constructed using lightweight wood fiber fill. The lightweight fill material was assumed to have a unit weight of 6.3 kN/m^3 and an angle of internal friction of 40 degrees. The granular soils used in the fill were assumed to have a unit weight of 19.6 kN/m^3 and an angle of internal friction of 37 degrees.

The number of geotextile reinforcement layers required was determined such that the amount of soil shear strength gain required for stability would be approximately the same for both the bearing capacity and slope stability mode of failure. Therefore, the soil shear strength required to meet factor of safety requirements for bearing capacity was first determined. On the basis of that shear strength, the number of geotextile layers required to meet factor of safety requirements for slope stability was determined. It was necessary to keep the required geotextile tensile strength low enough that commonly available geotextiles could be used if possible.

The reinforced fill was designed using the methodology in the FHWA *Geotextile Engineering Manual* (*1*). The geotextile reinforced portion of the embankment was assumed to act as a mat, distributing the vertical load due to the weight of the fill evenly over the width of the fill. This assumption was considered valid if the reinforced embankment is designed to resist lateral spreading (*1*). The bearing capacity of the soil, determined using two-layer theory (*2*), was compared with the average vertical stress at the base of the embankment to determine the bearing capacity safety factor. Slope stability was determined using the Bishop method.

The final embankment configuration resulting from this design is given in Figure 4. Six geotextile reinforcement layers at an allowable tensile strength of 23 kN/m were required because of slope stability and lateral spreading considerations. The geotextile reinforcement is needed only until the soil gains enough strength to support the fill without reinforcement, which would result in the geotextile reinforcement layers being fully loaded for up to 8 months, on the basis of the calculations. The geotextile layers were therefore considered to be temporary, allowing a relatively high creep limit of 60 percent of ultimate to be used. This resulted in an ultimate wide width tensile strength of 38 kN/m to be required. The reinforcement was also designed to resist lateral deformation of the embankment by requiring a secant modulus of 230 kN/m at 10 percent strain. The geotextile selected by the contractor to meet these property requirements was a polypropylene slit film woven geotextile with a unit mass of 190 gm/m^2.

An organic root mat was at the ground surface below the proposed fill. The root mat was considered to possess tensile strength not accounted for in the soil shear strength design values used. The root mat tensile strength was assumed to be equivalent to the tensile strength of one geotextile reinforcement layer, or 23 kN/m. Therefore, the number of geotextile layers required at the maximum embankment height could be reduced to five (see Figure 4). The strength of the root mat was preserved by requiring a working platform to be constructed and the surface root mat not to be disturbed.

The calculations indicated that the soil shear strength had to increase to 16.8 kPa to provide the needed embankment stability, assuming five layers of reinforcement were used.

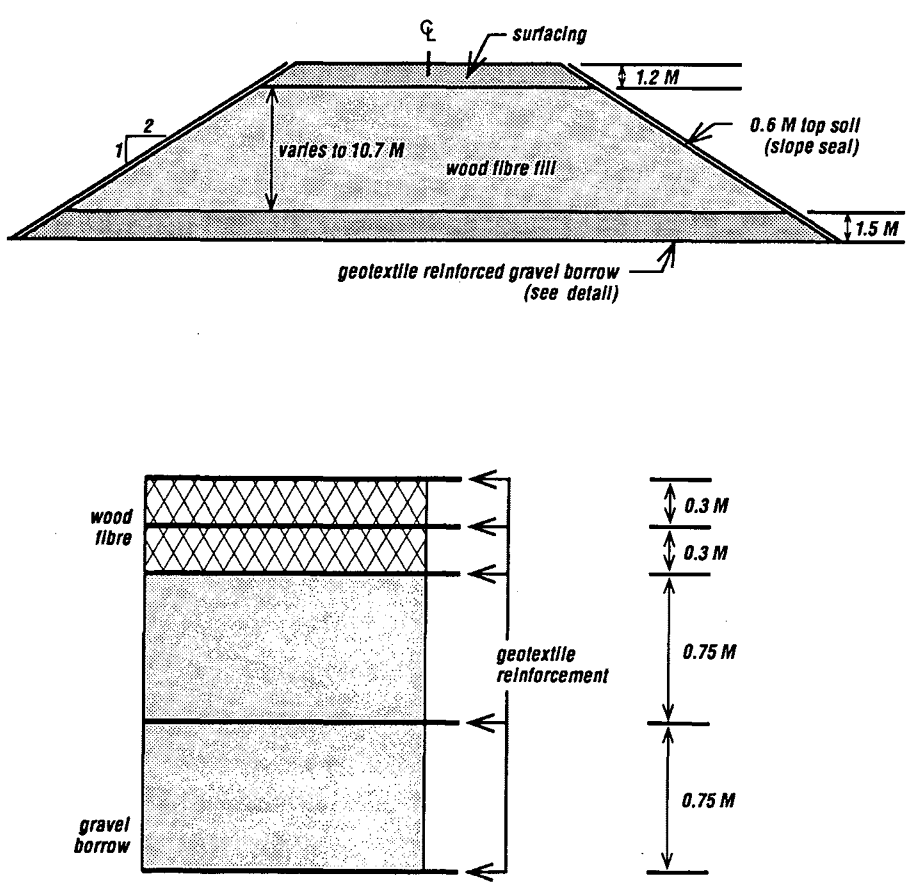

FIGURE 4 Design cross section for reinforced wood fiber fill: *top*, fill cross section; *bottom*, geotextile reinforcement detail.

The amount and rate of strength gain expected were established using the triaxial drained strength and consolidation parameters obtained from the laboratory test program provided and using the methodology of Su et al. (*3*). It was estimated that fill construction would take a minimum of 4 months, or 0.7 m/week, to ensure that the embankment would remain stable during construction. The pore pressure response of the foundation soil to embankment loading was actually used to control the embankment construction rate. On the basis of the laboratory test data and subsequent analysis, the ratio of pore pressure increase to the maximum embankment vertical load was required to be 0.33 or less to ensure embankment stability during construction.

Embankment settlement was determined using the laboratory consolidation data obtained at the site and conventional settlement estimating techniques. The two issues regarding settlement that had to be addressed were (a) the effect of the settlement magnitude on the culvert at the base of the fill and on the amount of embankment overbuild required and (b) the time required for settlement to be complete so that the time to begin paving could be determined. Primary settlement for the wood fiber fill was estimated to be approximately 1.5 m. Accounting for a fill construction time of 4 months, this settlement was estimated to take up to 21 months to be completed once fill construction began. Secondary consolidation was estimated to be approximately 0.13 m over the following 20 years.

The settlement magnitude estimated was used to determine the amount of overbuild required. It was necessary for wood fiber to be used for the additional fill, instead of granular surfacing above the wood fiber, as much as possible to minimize the load added to the embankment. The length of time required for settlement was longer than normally desirable for a highway fill. In this case, however, the long settlement period could be tolerated in the highway completion schedule.

Because the culvert had to be installed before embankment construction because of the presence of a small creek, the culvert had to be designed to tolerate the large settlement expected. First, the culvert and creek were moved to the edge of the fill area where the depth of compressible soil was less and settlement was less severe. The 1.5-m culvert was also sized 0.3 m in diameter larger than needed so that if a sag in the culvert developed, the culvert would still have adequate flow capacity. Finally, a minimum camber of 0.3 m was placed in the culvert to account for differential settlement along the length of the culvert.

FILL SPECIFICATION AND CONSTRUCTION

Specifications for installing the geotextile reinforcement in the fill were developed on the basis of the FHWA *Geotextile Engineering Manual* (*1*). A working platform using granular soil with enough thickness to cover all stumps, logs, or other protrusions with 15 cm of material was required to preserve the root mat and minimize damage to the first geosynthetic layer. Stumps were cut flush with the ground as much as possible.

The contractor placed an unauthorized haul road in the area where the reinforced wood fiber fill was to be placed. The haul road was constructed of 2.4 to 4.6 m of silt fill. The contractor placed some trees and branches below the fill to help it float over the soft foundation soil. Test holes drilled through the fill showed that it did not break through the root mat and branches at the ground surface, although a considerable amount of displacement and settlement occurred. The contractor was required to completely remove the haul road fill from the site because of concern about creating an area under the final fill that would be partly consolidated and could cause the wood fiber fill to settle differentially.

Materials used for the fill consisted of gravel borrow and wood fiber fill, as mentioned. The gravel borrow, a silty 3.2 cm minus gravelly sand, was used for both the working platform and the bottom 1.5 m of fill. The remainder of the fill, with the exception of the top 1.2 m of surfacing, was well-graded wood fiber fill, consisting of fibrous, irregularly shaped particles that varied from 0.6 to 15 cm, predominantly 1.3 to 5 cm. The wood fiber as placed in the fill was fresh (not degraded), as required by the specifications. The as-compacted unit weight of the wood fiber fill was 6.0 kN/m^3, just under the 6.3-kN/m^3 unit weight assumed for design. A 0.6-m-thick top soil slope seal was placed on the outer surface of the wood fiber fill to protect it from oxygen and fire. The gravel borrow was compacted in maximum 20-cm lifts to 90 percent of maximum density using vibratory and static compaction rollers. The wood fiber fill was compacted by routing hauling equipment a minimum of two times with complete coverage over each lift. The maximum lift thickness allowed was 0.3 m. The minimum weight of the hauling equipment used to compact the fill was 15 T.

The geotextile was to be laid in the fill so that the machine direction would be perpendicular to the embankment centerline to ensure that maximum geotextile strength would be available in the direction of maximum stress. The strips of geotextile were to be joined together with sewn seams, using a double-sewn "J" seam, Type SSn-1, with parallel stitching placed 1.3 cm apart. The geotextile was actually shipped to the site with two rolls of geotextile sewn together using factory seams, forming 7.6-m-wide panels. In general, the factory-sewn seams were of a higher quality than the field-sewn seams. The poorer quality of the field-sewn seams was the result of worker inexperience with sewing, difficulty in getting the geotextile panels properly lined up, attempting to work with too much geotextile at one time, and wind moving the geotextile panels around. The field sewing operation was labor intensive at times.

The specifications for fill placement over the geotextile were designed to (a) keep the weight and amount of fill uniformly distributed over the width of the fill, (b) minimize potential for damage to the geotextile during fill placement, and (c) use the weight of the fill to pretension the geotextile to limit deformation. A minimum thickness of 0.3 m of fill between the geotextile and the spreading equipment was required to prevent damage to the geotextile. One of two methods for fill placement was to be used to pretension the geotextile layers and keep the fill evenly distributed across the fill width, depending on whether a small, controlled mudwave formed as the fill was placed. If a mudwave formed, the fill was to be placed using a concave advancement pattern, as shown in Figure 5. If a mudwave did not form, the fill was to be placed in a convex advancement pattern (Figure 6). Generally for fills over soft soils, the mudwave will form only during place-

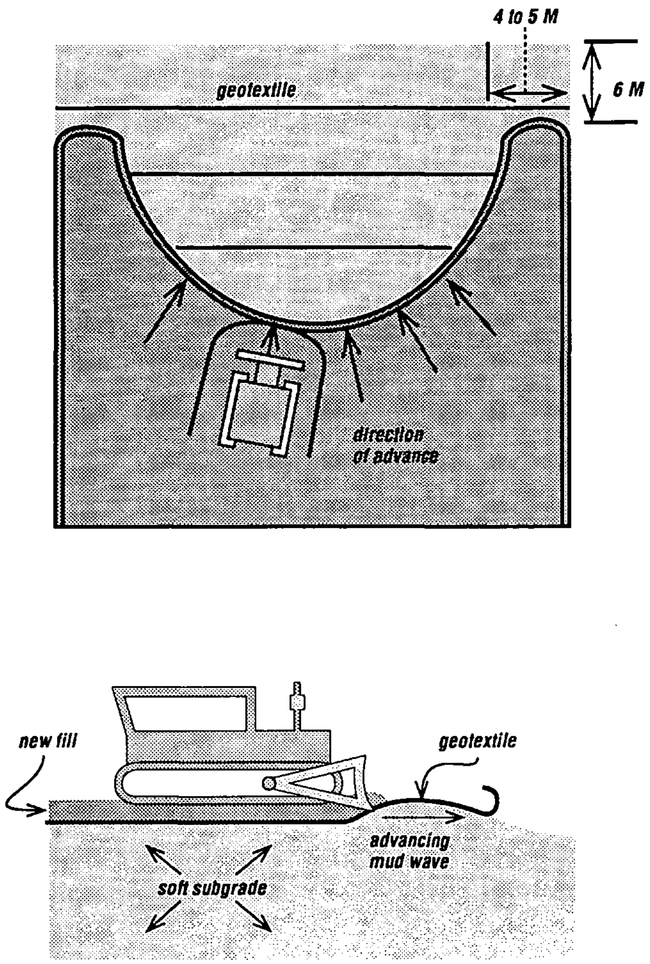

FIGURE 5 Reinforced fill construction method if mudwave forms (*1*): *top*, plan; *bottom*, mudwave formation during fill placement.

ment of the first 0.3 to 0.6 m of soil, if it forms at all. In the case of the subject fill, a mudwave never formed, possibly because of the working platform in place before the first layer of geotextile was placed and the presence of the root mat. Therefore, the convex advancement pattern was actually used for fill construction. Photographs of the actual fill construction are given in Figure 7.

Measurements from six pneumatic pore pressure devices installed below the fill (Figure 3) were used to control the rate of fill construction using a maximum pore pressure ratio of 0.33 as discussed. Settlement data from pneumatic settlement indicating devices at the base of the fill were also used to interpret the stability of the fill. Because of problems with the pore pressure devices, which were associated with installation and possibly the result of gas pressure caused by organic matter decay, six new pore pressure devices were installed in approximately the same locations after the first 1.5 m of fill was placed. Even some of the new devices did not appear to work properly, and eventually four of the six new devices failed.

The maximum allowable pore pressure ratio was equalled or exceeded twice during construction, on the basis of measurements from the few pore pressure devices that appeared to work, when the fill height was at 6.6 m and when it was at 9.5 m. Fill construction was stopped in both cases to allow the pore pressure to dissipate and the soft soil to consolidate and gain strength. In the first case, fill construction was stopped for 52 days, in the second case, for 130 days. In the second case, part of the delay was the result of construction scheduling and inclement weather. In both cases there was no visible evidence of embankment failure or mudwave formation. The total time required to construct the fill was just under 11 months, considerably longer than the 4-month fill construction period estimated from the laboratory test data during design.

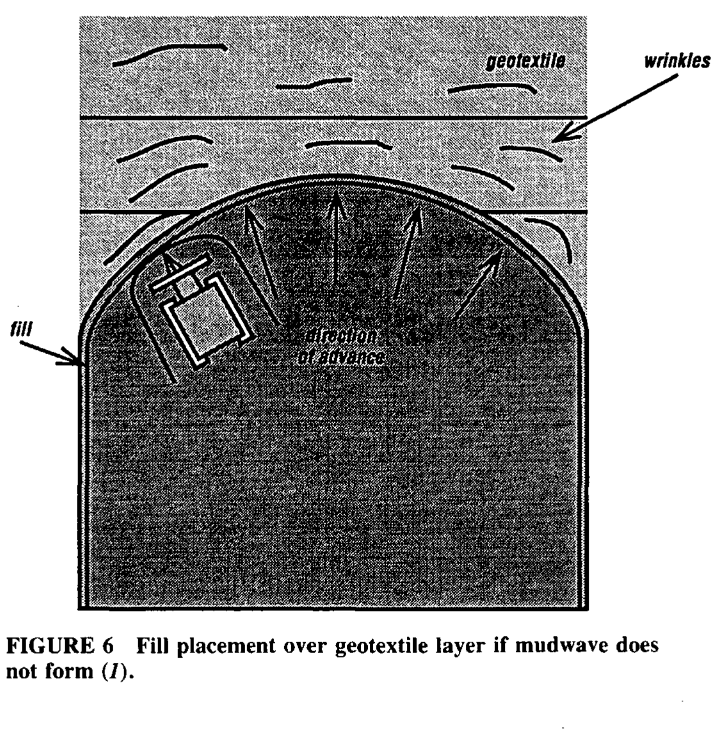

FIGURE 6 Fill placement over geotextile layer if mudwave does not form (*1*).

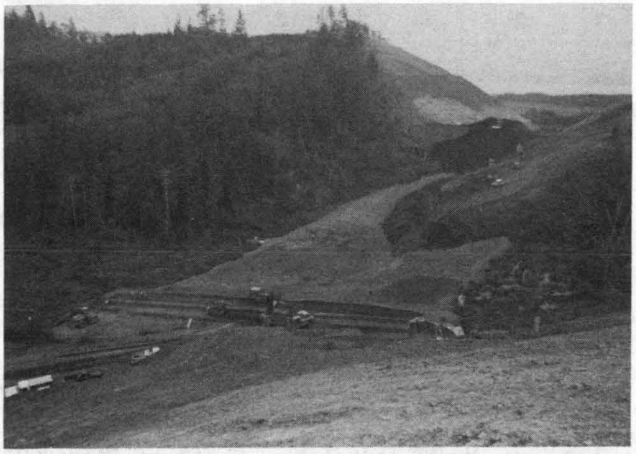

FIGURE 7 Wood fiber fill construction: *top*, granular fill construction and culvert installation; *bottom*, placement of geotextile and wood fiber fill.

PERFORMANCE

The wood fiber fill was completed to subgrade level in September 1987. Paving began more than 1 year later, October 1988. Figure 8 gives settlement data for the time since initial construction. Fill settlement, when subgrade level was reached, was 0.97 m. Before paving, fill settlement had increased to 1.2 m. As of September 1992, 1.4 m of settlement had occurred. Total settlement is projected to reach 1.5 m.

The contractor haul road mentioned caused the fill to settle differentially across its width as anticipated. The differential settlement across the roadway width at the top of the fill was approximately 0.1 to 0.2 m. The greatest settlement occurred on the side of the fill opposite the haul road.

The performance of the wood fiber has been excellent. Samples of 5-year-old wood fiber were exhumed from below the 0.6-m topsoil cover and found to be nearly fresh, with a classification of 2 by WSDOT (see Table 1).

The pavement to date has shown no distress despite settlement and predominately logging truck traffic, with the exception of a small crack where the pavement transitions from cut to fill. Water samples taken upstream and downstream of the site indicated no difference in water quality (no negative impacts from any leachate).

The culvert at the base of the fill was located, as noted, as close to the hillside as possible to mitigate the effects of settlement. The culvert has suffered significant differential settlement and now has a sag in the middle but is still functioning. The culvert has settled approximately 0.4 m more than anticipated at its center, primarily because the culvert was not as close to the edge of the very soft soil deposit as desired during design because of channel flow requirements.

PROJECT COST SAVINGS

It was initially planned to cross the site area with an earth fill and divert the stream flow through a 1.2-m culvert. Because

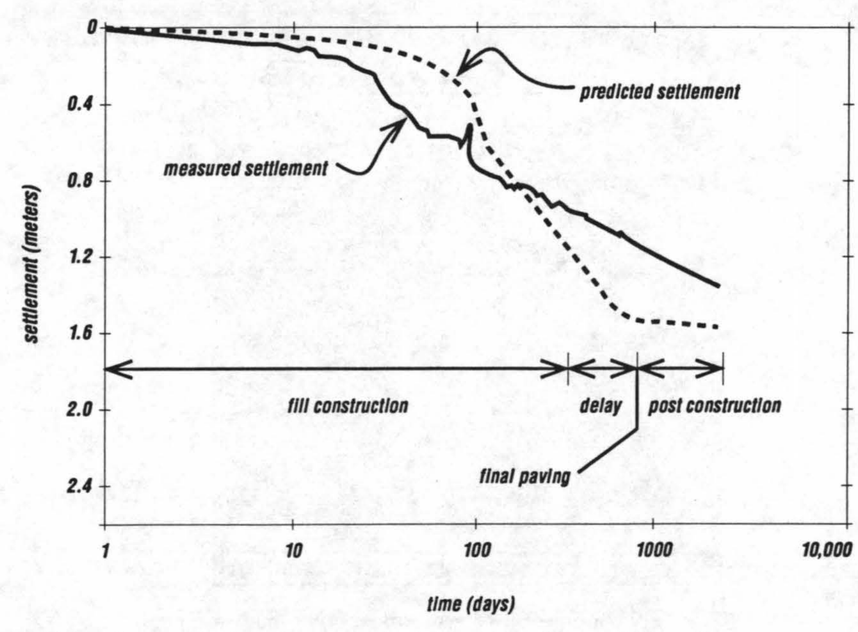

FIGURE 8 Predicted and measured settlement for the wood fiber fill.

TABLE 1 WSDOT Wood Fiber Classification Criteria

Class	General Appearance	Appearance of Decay[a,b]	Particle Strength (Breaking)[c,d]	Particle Stiffness (Bending Capacity)[d]
1	Woodlike, sharply defined graininess	Fresh: Sharp color, fresh woody smell, no disintegration	Cannot be broken with fingers	Retains its shape with force
2	75% of material is woodlike, well defined graininess	Initial signs of decomposition: Distinct color, definite wood smell, very little disintegration of wood fibers	Very difficult to break with fingers	Easily returns to original shape with release of force
3	50% of material is woodlike, complete but poorly defined graininess	Middle stage of decomposition: Fading color, weak wood smell, some disintegration of wood fibres	Breaks with firm finger force	Shape is permanently, but slightly, distorted with force
4	25% of material is woodlike, only partial graininess remains	Advanced stage of decomposition: Fading color, organic smell, mostly disintegrated	Breaks easily with fingers	Shape is permanently distorted with force
5	No longer woodlike, no graininess	Completely decomposed: Dull color, foul smell, completely disintegrated	Squeezes between fingers	No longer returns to original shape; spongy

[a]Primary emphasis is on disintegration
[b]All discriptors may not apply
[c]Standard testing size is 2" x 1/2" x 3/8"
[d]Moisture content for tested sample is "wet to touch"

of the soil conditions at the site, the initial option for crossing the site was not possible and a bridge was chosen. The estimated cost for a bridge was $1,700,000.

Because of the high cost of the bridge, alternatives were considered, including reconsideration of the earth fill. One option was to place an earth fill, force a bearing capacity failure, and thus displace the very soft foundation soils. This option was environmentally unacceptable. Unsuitable removal was also environmentally unacceptable, and impractical. The earth-fill option using berms was unacceptable because it required additional wetlands, which was undesirable, would at best be marginally stable, and resulted in unacceptable settlement.

An acceptable alternative was ground improvement using stone columns, at an estimated cost of $1,500,000. Environmental, stability, and settlement constraints could all be met with this alternative. A lightweight fill using wood fiber was considered feasible on the basis of previous successful use of wood fiber by WSDOT for permanent roadway applications (*4*). The combination of large fill height and very soft ground required the addition of geotextile reinforcement. The actual cost of the lightweight fill was $972,221. This was a more than

$500,000 cost savings compared with the next lowest cost alternative, and more than $700,000 less than a bridge.

SUMMARY

The use of wood fiber as a lightweight fill, with geotextile reinforcement, on this project proved to be a cost-effective solution for constructing a 13.4-m-high fill over 15.2 m of very soft organic silt soils. No stability problems were encountered during or after construction, and total settlement was within project requirements. Continued secondary consolidation is taking place in the foundation soils. There is no evidence of postconstruction settlement within the wood fiber.

The condition of the wood fiber, as of August 1992, is excellent. The wood fiber below the 0.6-m topsoil cover was graded as nearly fresh in a recent study. On the basis of the performance of the wood fiber at this and other WSDOT sites, it appears that wood fiber can be used for permanent applications with design lives of more than 50 years (*5*).

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Long-Term Performance of Wood Fiber Fills

ALAN P. KILIAN AND CHRISTINE D. FERRY

The results of a research project to determine the long-term performance of 21 wood fiber embankments, constructed by the Washington State Department of Transportation (WSDOT) beginning in 1972, are presented. The wood fiber was placed above and below the water table and in fluctuating groundwater conditions. At the time the embankments were constructed, there was concern that they would only provide a 15- to 20-year service life. Performance of existing wood fiber fills was evaluated on the basis of the quality of the wood fiber material, quality of the effluent, and condition of the pavement. A visual classification system rating the wood fiber from fresh to completely decomposed was developed and used to establish a criterion from which all wood fiber material could be rated. Visual examination and laboratory tests were used as determining aspects for the effluent quality. The WSDOT pavement management system was used to evaluate relative pavement performance. Site descriptions giving specific characteristics and properties of inventoried fills are presented. An analysis of this information was done to determine the effectiveness of the fills. Over half the wood fiber samples were found to be nearly fresh or fresh after 15 years, and none after 5 to more than 19 years was found to be completely decomposed. In all but one case the pavement quality over the wood fiber fills surpassed the entire comparative milepost section rating, indicating that the wood fill performance exceeded that of the adjoining area. Generally, the surface water in the vicinity of the wood fiber was found to be clean and pure, indicating no adverse impact of effluent. Given the above findings, embankments constructed of wood fiber were found to perform well over an almost 20-year period. Service life of more than 50 years can be expected of wood fiber fills.

In 1972, the first wood fiber fill was constructed on the Washington State highway system as an emergency repair on SR-101 to repair a landslide that had destroyed a section of roadway. Wood fiber was selected for two primary reasons. The first had to do with constructability as an all-weather material. Rain does not affect the placement and compaction of the embankment. Second, a lightweight fill was used to lessen the driving forces of the unstable ground that was causing the instability. In addition, wood fiber material was readily available at low cost and could be obtained on short notice from local sawmills. Historically, timber areas such as the cities of Raymond, Aberdeen, and Hoquiam along the Washington State coast have used wood fiber material to construct non-engineered fills over very weak marine sediments.

The primary disadvantages of wood fiber are leachate and short-term settlement of the wood fiber. Methods used to control leachate are (a) reduce the water flow into the wood, control the type of wood fiber, or (b) entrap the leachate. Controlling the effects of wood fiber compressibility involves surcharging or delay periods or both, to allow settlement to take place before the pavement section is placed.

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Another concern is the possibility of spontaneous combustion. Known conditions such as warm, rainy weather and large-size piles of sawdust have been cited as possible reasons behind spontaneous ignition (*1*). It is believed that spontaneous combustion occurs because of a combination of mechanisms. Biological reactions initially raise the temperature to a level at which physiochemical processes take over. From this point the physiochemical reactions continue until ignition occurs (*2*). Suggested precautions include initially restricting height of wood fiber to 5 m and reducing ventilation or air to the wood fiber.

A long-standing concern has been the durability and life of wood fiber as an engineering material for use in highway embankments. Therefore a research project was undertaken to examine existing wood fiber fills to determine their long-term performance.

GENERAL

The Washington State Department of Transportation (WSDOT) had two critical concerns about the use of wood fiber as embankment material for permanent roadways. First, Would wood fiber resist decay and rotting sufficiently to provide an embankment life of more than 75 years? Some estimates gave wood fiber a design life of only 15 to 30 years. Old investigations by WDOT, reportedly 70 years old, of sawdust piles found a decomposed outer zone 0.61 to 0.91 m thick with an inner core in which no decomposition had taken place.

The second critical concern was the risk of spontaneous combustion, which could cause the wood fiber fill to catch fire. Biological oxidation increases the fiber temperature to approximately 75°C, followed by a chemical reaction that increases the temperature to ignition. Controlling the fill temperature and reducing the availability of oxygen are methods of preventing a fire (*1*).

A number of terms are used to describe the wood fill material. In this paper, wood fiber is used as a generic term. The actual fill material may be hog fuel, sawdust, or woodwaste. Hog fuel is defined as the ground wood and bark that is burned in a steam boiler. The small particles of wood generated when logs are cut into lumber are classified as sawdust. Woodwaste is made of sawdust, hog fuel, bark chips, or a combination of the three. This material is generated from handling the logs at the saw mill.

Since the first WSDOT fill, 20 additional fills were constructed between 1973 and 1986. They were all built in areas of western Washington with high to moderate rainfall. The average age of these fills is 15 years.

The goal of this research was to evaluate the general long-term performance of existing wood fiber fills. To evaluate the performance of the fills, the investigation focused on the quality of the wood fiber material and the leachate and the performance of the pavement section.

An evaluation of the quality of the wood fiber fills was done by classifying the fill material primarily by observation, because no testing method is presently available to measure the degree of rotting. No existing classification system was identified in the literature. A system was developed, as part of this project, using five classifications to rate the amount of decomposition. The classification criterion developed to rate the wood fiber's current condition was based on four categories. These categories followed a progressive description from fresh to completely decomposed. "General Appearance" was used to describe the wood fiber on a macroscale, compared with the "appearance of decay" that considered the microscale.

Visual observations of the wood fiber's grain quality were the basis for determining the general appearance rating. Appearance of decay used descriptors indicating the odor of the wood, the amount of decomposition ranging from fresh to completely decomposed, and in-depth visual observations such as the color and amount of disintegration. Of these four descriptors, the primary emphasis was on the amount of disintegration. However, not all of the descriptors may apply to a specific sample of wood fiber. If this situation arose, the remaining descriptors determined the quality of the wood fiber.

Wood fiber's material properties were determined by its strength and stiffness. Selected pieces of wood fiber that were not saturated or dry, but were wet to touch, and approximately fit the dimensions 5.1 × 1.3 × 1.0 cm were used to test material properties by applying finger force. Some samples may not have pieces fulfilling the moisture content and dimension requirements. If this situation occurred, the remaining criteria were used to determine the wood fiber's quality.

Because no previous classification system existed before this research, there is no information stating the freshness of the wood fiber at time of original construction. The given ratings can only indicate the quality of wood fiber at the time of this field work.

The second performance measure is a determination of the quality of leachate. When water interacts with the wood fiber an extraction of the wood occurs and a solution is formed. This solution is known as leachate or effluent and is an environmental concern when near streams or surface water. Characteristics of this aqueous solution composed of extractions of cellulose and lignin include high oxygen demand, low pH, dark color, and foul odor. Methods of controlling leachate are reducing water flow through the fill, treating the effluent that exits the fill, and controlling the type of material placed in fill (*3*).

Pavement performance was evaluated by comparing the existing condition of the pavement with the previous WSDOT pavement management system rating. Adjacent areas without wood fiber fills were compared to evaluate relative performance. The pavement rating system considered the following four major features when developing a rating: (a) alligator, longitudinal, and transverse cracking; (b) patching; (c) raveling; and (d) flushing. Alligator cracking was further defined by hairline, spalling, and pumping. Depending on the width of a crack or the length of a patch, or both, points were subtracted from 100 to give an overall score of the roadway section.

The pavement rating provided in this research is a representation of the pavement directly above the lightweight fill. Generally, this is a small percentage of the milepost section that receives a WSDOT Pavement Management system rating. Table 1 gives the research rating for the fill section followed by the comparative WSDOT rating for the highway segment in parentheses. It should be noted that maintenance patch areas can cause local variances from the average roadway rating.

FIELD STUDIES

Each of the 21 existing sites having wood fiber fill was reviewed in the field to evaluate the long-term performance of the wood fiber. Table 2 gives a listing of the sites, year built, application, and size of fill. The wood classification criteria shown in Table 3 were used in the field to evaluate the overall quality of the wood fiber. Pavement ratings were made following the established criteria of the WSDOT pavement management system.

Temperature readings were taken to determine whether they were an indication of decomposition. Generally it was found that temperature indicated that biological processes were still active but not necessarily indicative of the quality of the wood fiber.

BOD levels were measured at 11 of 21 sites. They were useful as a site-specific reference to adjacent water not affected. Because it was summer when samples were taken, there was difficulty locating water for the samples. Therefore some sites were not tested. In addition, there are some limitations on use of BOD test results because a national standard does not exist for the condition of a given site, because each body of water must be evaluated individually to determine its allowable BOD level. Therefore the actual BOD values for the sites tested are not used for analysis but are useful for further reference and comparative site-specific studies.

The results of the field work are given in Table 1. Generally the wood fiber was found to be in good to excellent condition with most ratings from 1 to 3. Four sites had samples that exhibited Class 4 ratings. At the Kelso site it was only the wood fiber immediately under the topsoil cover that had the low rating. Below that, wood fiber was fresh with a Class 1 rating. Two of the Cosmopolis Hill sites built in 1976, MP 71.9 and 77.27, showed signs of decomposition. It is not documented but is well known within WSDOT that during that time it was preferred to have the wood fiber fills built of aged wood fiber. This was to reduce the potential for spontaneous combustion possibly by reducing the active biological processes. Records of which fills were built of the aged wood fiber were not kept. The fourth site, at Victor on SR-302, was built with fresh wood fiber. The advanced aging at this

TABLE 1 Wood Fiber Project Summary

SR	PROJECT	DEPTH (m)	CLASS	TEMP C	DENSITY * kN/m3	PAVEMENT RATING SECTION(SEGMENT)
2	Skykomish Wye	0.52	2	14.8	8.1	80 (68)
		soil		23.4		
5	Kelso Frontage Rd	0.82	4	21.9		99 (NA)
		2.0	1	17.4		
		soil		18.9		
12	Washout at Aberdeen	0.67	2	21.1		100 (100)
		soil		22.0		
12	Baila Dip	0.82	2	17.2		none
	site 1	new	1	21.7		
		0.46	3			
	site 2	0.92	2			
		air		16.7		
16	Burley Olalla Rd.	0.76	1	21.7	4.6	94 (89)
		1.3	1	--		
		soil		15.6		
101	Port Dock to Fowler	0.82	2	16.7	5.5	RT 80 (46)
		1.2	2	17.4		LT 80 (65)
		soil		16.6		
101	Rock Crusher Hill	0.73	3	17.8		92 (65)
		0.98	2	16.7		
		soil		16.1		
101	Cosmopolis Hill					
	MP 71.77	1.5	3	28.9	9.7	100 (46)
		rocks		25.5		
	MP 71.9	0.55	4	23.5	5.2	84 (46)
		1.2	2	23.2		
		soil		18.3		
	MP 77.27	0.40	4	24.1	6.0	84 (46)
		1.0	4	24.7		
		soil		22.3		
	MP 77.35	#1 0.15	3	none		88 (46)
		0.76	2			
		#2 0.15	3			
		0.98	2			
	MP 77.61	0.92	3	none		88 (46)
	MP 77.97	0.98	1	19.3		90 (46)
		soil		20.4		
101	Emergency Repair	0.64	2	28.4		92 (60)
		0.92	2	29.6		
		soil		22.4		
101	Cosmopolis to SR 107	0.76	3	19.4		90 (60)
		soil		18.7		
101	SR 107 to Cosmopolis	0.92	1	none		92 (44)
		1.2	1			
109	West Hoquiam	0.82	2	22.9	6.0	100 (100)
		1.1	2	22.5		
		1.4	2	21.1		
		soil		18.8		
109	Bob Wain Hill					99 (100)

(continued on next page)

TABLE 1 *(continued)*

SR	PROJECT	DEPTH (m)	CLASS	TEMP C	DENSITY * kN/m3	PAVEMENT RATING SECTION(SEGMENT)
109	Pt. Grenville	0.4	1	15.8	3.2	none
		soil		15.9		
302	Victor Cutoff Rd.	0.46	4	none		none
		0.92	3			
505	Cedar Creek Slide	0.46	2	22.2		none
		1.07	2	26.7		
		soil		17.8		

* In-place, wet density
1m = 3.28ft, C = (F-32)/1.8, 1N = 0.225lb

site may be due to exposure of the wood fiber. Most of the wood fiber was placed in an excavated cavity created by removing landslide debris. The upper portion was sealed from the air with an asphalt emulsion. Subsequently the landslide continued to move. This exposed the upper portion of the wood fiber allowing access to air. This may account for the advanced aging, but this is not certain.

Temperature variations did not prove to be particularly significant in that temperature could not be correlated to wood aging. It was, however, interesting to find wood fiber fills of 16 years having elevated temperatures in the wood fiber, indicating a relatively high level of biological activity. The implication was that the decay process is still active.

The variance of in-place unit weights was significant. The moist unit weights varied from 5.2 to 8.3 kN/m^3. The compactive effort specifications for all the fills was similar. Required compaction was specified as two passes with a D8-Caterpillar tractor or equivalent on a maximum 1-ft lift.

Pavement ratings were generally excellent in the wood fiber areas. All pavements are flexible asphalt concrete. It appears that the combination of a 0.6-m flexible pavement section with the elastic wood fiber fill worked well as a system.

CONCLUSION

The research conducted during this study verified the generally excellent performance of wood fiber used in engineered fills for up to 20 years. It was found that some sites had significant degradation of the wood fiber. Questions about the quality of the wood fiber initially placed arose. Although not formally documented it is known that particularly during the 1970s the use of aged wood fiber was encouraged to lessen the danger of fire.

To evaluate the long-term performance of wood fiber, the site locations were separated into three categories, ranging from more than 15, 10 to 15, and 5 to 10 years old. The first range, or oldest, included three areas that remained nearly fresh. Although the wood fiber at Baila Dip was replaced with fresh wood fiber material, the old wood fiber remained intact with only some degradation. Although the remaining two sites had wood fiber exposed on the side slope face, the material performed well with minimal amount of decomposition in one area and a little more in the other.

The middle category ranged from fresh to almost decomposed. The classification scores varied between site locations

TABLE 2 Wood Fiber Project Inventory

SR	PROJECT	DATE	APPLICATION	WOOD DEPTH(m)	FIBRE LENGTH (m)
002	SKYKOMISH WYE	1980	Soft soils	3.35	244
005	KELSO FRONTAGE ROAD	1977	Soft soils	9.15	392
012	WASHOUT AT ABERDEEN	1976	Landslide correction	6.10	30.5
012	BAILA DIP	1974	Landslide correction	4.57	94.6
016	BURLEY OLALLA ROAD	1976	Soft soils	6.10	275
101	PORT DOCK TO FOWLER	1976	Soft soils	6.10	290
101	COSMOPOLIS HILL MP 71.77	1976	Landslide correction	2.55	27
101	COSMOPOLIS HILL MP 71.90	1976	Landslide correction	2.55	40.3
101	ROCK CRUSHER HILL	1978	Landslide correction	7.62	110
101	COSMOPOLIS HILL MP 77.27	1976	Landslide correction	5.60	36.6
101	COSMOPOLIS HILL MP 77.35	1976	Landslide correction	4.07	83.0
101	COSMOPOLIS HILL MP 77.61	1976	Landslide correction	2.55	54.3
191	COSMOPOLIS HILL MP 77.97	1976	Landslide correction	2.55	79.3
101	COSMOPOLIS EMERGENCY REPAIR	1972	Landslide correction		
101	COSMOPOLIS TO SR 107	1973	Landslide correction	3.05	76.3
101	SR 107 TO COSMOPOLIS	1982	Landslide correction	3.05	21.4
109	WEST HOQUIAM	1986	Soft soils	10.7	247
109	BOB WAIN HILL	1979	Soft soils	3.05	191
109	PT. GRENVILLE	1976	Landslide correction	1.22	275
302	VICTOR CUTOFF RD.	1978	Landslide correction	3.05	137
505	CEDAR CREEK SLIDE	1982	Landslide correction	4.57	153

TABLE 3 WSDOT Wood Fiber Classification Criteria

CLASS	GENERAL APPEARANCE	APPEARANCE OF DECAY[a,b]	PARTICLE STRENGTH (BREAKING)[c,d]	PARTICLE STIFFNESS (BENDING CAPACITY)[d]
1	WOODLIKE, SHARPLY DEFINED GRAININESS	FRESH: SHARP COLOR, FRESH WOODY SMELL, NO DISINTEGRATION	CANNOT BE BROKEN WITH FINGERS	RETAINS ITS SHAPE WITH FORCE
2	3/4 MATERIAL IS WOODLIKE, WELL DEFINED GRAININESS	INITIAL SIGNS OF DECOMPOSITION: DISTINCT COLOR, DEFINITE WOOD SMELL, VERY LITTLE DISINTEGRATION OF WOOD FIBRES	VERY DIFFICULT TO BREAK WITH FINGERS	EASILY RETURNS TO ORIGINAL SHAPE WITH RELEASE OF FORCE
3	1/2 MATERIAL IS WOODLIKE, COMPLETE, BUT POORLY, DEFINED GRAININESS	MIDDLE STAGE OF DECOMPOSITION: FADING COLOR, WEAK WOOD SMELL, SOME DISINTEGRATION OF WOOD FIBRES	BREAKS WITH FIRM FINGER FORCE	SHAPE IS PERMANENTLY, BUT SLIGHTLY DISTORTED WITH FORCE
4	1/4 MATERIAL IS WOODLIKE, ONLY PARTIAL GRAININESS REMAINS	ADVANCED STAGE OF DECOMPOSITION: FADED COLOR, ORGANIC SMELL, MOSTLY DISINTEGRATED	BREAKS EASILY WITH FINGERS	SHAPE IS PERMANENTLY DISTORTED WITH FORCE
5	NO LONGER WOODLIKE, NO GRAININESS	COMPLETELY DECOMPOSED: DULL COLOR, FOUL SMELL, COMPLETELY DISINTEGRATED	SQUEEZES BETWEEN FINGERS	NO LONGER RETURNS TO ORIGINAL SHAPE, SPONGY

a primary emphasis is on disintegration
b all discriptors may not apply
c standard testing size is 5.1cm x 1.3cm x 1.0cm
d moisture content for tested sample is "wet to touch"

and the depth of the fill from where samples were taken. Most samples taken deeper in the fill were fresher than those from the surface. Soil and earth, asphalt, and asphalt emulsion seals placed on the wood fiber for protection varied in performance. Asphalt seals generally began breaking apart after 10 years, exposing the wood fiber. In general, sites with soil covers had better ratings indicating that quasi-isolation protected the wood fiber and enhanced durability. The 0.3- to 0.6-m topsoil coverings worked well at all locations, removing the danger of fire from cigarettes and glass and reducing water infiltration with resultant reduction in leachate.

Recent fills, in the past 5 to 10 years, have remained intact and nearly fresh. One location was completely fresh, and the remaining two locations were almost fresh. At Cedar Creek difficulties arose when interlock between the wood chips was not achieved. To avoid similar problems in the future, specifications required the wood fiber material to be in various sizes with a maximum dimension of 0.15 m and interlocking.

Although no comparison was available for leachate test results, the values for flowing water were very low and samples looked clean and pure. Of the 21 sites tested for water quality, 10 could not be tested for BOD levels due because there was no water. Two sites appeared to have water flowing nearby. Two of the 21 sites had relatively higher BOD counts. At the SR-109 West Hoquiam site the adjacent downstream water tested better than did the comparative reference sample taken upstream. There was an oily film in some stagnant pools, yet the BOD counts indicated no adverse effect. The other site that had comparatively higher BOD levels was the SR-101 Rock Crusher Hill site. Standing or flowing water was not present. A pit was dug, allowed to fill with groundwater, sampled, and tested. BOD counts were higher but their effect was inconclusive. A definite conclusion about the water quality of these areas was not made because the data were limited.

Pavement condition is one indication of the wood fiber's performance. Overall, a stable roadway was established by the wood fiber. Of the five fills over soft soils, four have pavement scores of 99 and above. Roadways over the landslide fills have some patching or resurfacing, but most scores are in the high eighties and above.

During field sampling, five in situ densities were taken. These values were from approximately 5.2 to 8.3 kN/m^3 for moist wet unit weights. Classification of these samples varied from 2 to 4 but no relationship could be made to relate these scores to the density. Generally, the drier samples had lower densities compared with a higher density for the wetter material. This generalization applied to all but one sample that was set. Its in-place density was only 6.0 kN/m^3.

Research should be done in another 15 to 20 years to evaluate the rate of degradation of the wood. At that time the wood fiber may resemble the old sawdust piles previously studied (J. Hart, unpublished data) that have an outer seal of decomposed material protecting a fresh one. If the fills follow this pattern, possibly significant decomposition will not occur. If more decomposition is occurring, research could be done on the quality and types of seals being used.

Settlement is a factor taken into account when designing a lightweight fill. Installing settlement devices in controlled fills in both the soil and fill during construction is needed to better define settlement characteristics of the wood and provide data for future designs. Data for the soil and fill enable a more precise value of the wood fiber fill's contribution to the overall settlement.

Further research should be done on the effects of water on wood strength. Because some of the fills had a large amount of water, it would be valuable to know whether strength is lost when the wood soaks in the enclosed water.

RECOMMENDATIONS

The 20-year history of successful use of wood fiber on WSDOT projects as engineered fills indicates that its use should continue for lightweight fills. Degradataion of some fills documented in this research indicates wood fiber may have a finite life as an engineered material in some situations. Use on high volume lifeline roadways, where repairs are prohibitive, should be considered carefully. Use on major roadways of moderate volume, or less critical roadways, should be considered as acceptable.

The technical guidelines suggested for continued use of engineered wood fiber fills are

1. For areas with climates similar to western Washington State only fresh wood fiber should be used to build the fill. This will prolong the life of the fill.
2. To mitigate the effects of wood leachate, the volume of water entering the wood should be controlled so that a minimum of water flow occurs.
3. Fill slopes of 1.5H:1V or flatter with a 0.6-m soil cover are recommended to reduce the decay of the wood and lessen danger of fire.
4. A minimum 2-ft pavement section should be used.

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Rubber Soils as Lightweight Geomaterials

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The literature review and laboratory testing results from an ongoing research study, which investigates the feasibility of using rubber soils as lightweight geomaterial in highway construction, are presented. An overview of conventional lightweight materials; generation and disposal options for scrap tires; a summary of the various field and laboratory studies on the use of shredded tires as lightweight fill; results from compaction, compressibility, and permeability testing of compacted rubber soils samples; and the salient conclusions of this study are also presented. The use of shredded tires in highway construction offers technical, economic, and environmental benefits under certain conditions. The benefits are reduced weight of fill and backfill pressures. Shredded tires serve as a good drainage medium and have longer life. Tire chips are practically indestructible and available in abundance at practically no cost. Recycling large quantities of discarded tires has a positive impact on the environment. Potential problems are leachate of metals and hydrocarbons, fire risk, and large compressibility of tire chips. Information about stress-strain-strength behavior of tire chips for design and performance prediction of tire embankments and long-term environmental impacts of shredded tires is lacking.

Both the stability and settlement of embankments on soft foundations can be improved by using lightweight embankment fill (*1,2*). Lightweight materials that have been used successfully in highway embankments are bark, sawdust, dried peat, fly ash, slags, cinders, cellular concrete, expanded clay or shale, expanded polystyrene, and oyster and clam shells (*3*). Engineers and researchers are constantly trying to develop civil engineering materials that are more durable, more economical, and lighter to replace conventional materials to enhance the stability of slopes and foundations and reduce settlements in problem areas. Field and laboratory studies (*4*) have indicated that these apparently contradictory requirements can be potentially reconciled by the use of rubber soil.

Millions of scrap tires are discarded annually in the United States and other developed countries of the world. Most of them are currently landfilled or stockpiled. This uses valuable landfill space, creates a fire hazard, and provides a breeding ground for mosquitos. Efforts to sharply reduce the environmentally and economically costly practice of landfilling have stimulated the pursuit of nonlandfill disposal or reuse of scrap tires. Tires have useful engineering properties and have been used in a variety of engineering applications. Various highway agencies in the United States (Colorado, Minnesota, Oregon, Vermont, and Wisconsin) and abroad have experimented with and evaluated the use of shredded tires as a lightweight fill material. The experiences of these agencies show that the use of shredded tires in embankments is feasible and quite beneficial (*4–6*).

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This paper is based on an ongoing laboratory study that investigates the feasibility of using shredded tires in highway construction.

CONVENTIONAL LIGHTWEIGHT MATERIALS

Various types of lightweight materials and their salient properties are given in Table 1. All have been used in the past, although some materials are more popular than others and some have only been used experimentally or for structures other than highway embankments. The performance and cost differences between the various materials are significant. However, all have compacted densities significantly less than the unit weights of soils commonly used in embankment construction. Their use can therefore substantially reduce the effective weight of embankment. A questionnaire survey by Holtz (*2*) showed that lightweight fill has been used to some extent by 40 percent of the U.S. highway agencies that responded to the questionnaire.

Lightweight materials are usually expensive, especially if they are manufactured (e.g., expanded shales and clays, foamed plastics, lightweight concrete, etc.). Typically, costs range from \$50 to \$100/yd^3 and includes the cost of transportation (*2*). Some waste materials (i.e., sawdust, bark, shells, cinders, slags, and ashes) are almost free at the source and need only to be transported to the site. Their cost will depend on the distance between the source of waste material and the site.

GENERATION AND DISPOSAL OPTIONS FOR SCRAP TIRES

The waste tire problem in the United States is great and has far reaching environmental and economic implications. Current estimates by the Environmental Protection Agency (*7*) indicate that more than 242 million scrap tires are generated each year in the United States. The current waste tire disposal practice is that of the 242 million tires discarded annually in the United States, 5 percent are exported, 6 percent recycled, 11 percent incinerated, and 78 percent are landfilled, stockpiled, or illegally dumped. In addition, about 2 billion waste tires have accumulated in stockpiles or uncontrolled tire dumps across the country. The various tire disposal options are given in Figure 1.

Of the available options, no single one can significantly minimize the tire disposal problem, economically and environmentally. Many options must be simultaneously tried and developed to solve the problem (*8*). Three nonhighway applications that can potentially use large quantities of waste tires are breakwaters, artificial reefs, and reclaiming of rubber

TABLE 1 Lightweight Embankment Fill Materials (*2,22–25*)

Material	Unit Weight (pcf)	Comments
Bark (Pine & Fir)	35-64	Waste material used relatively rarely as it is difficult to compact and requires pre-treatment to prevent groundwater pollution. Long-term settlement of bark fill may amount to 10% of compacted thickness.
Sawdust (Pine & Fir)	50-64	Usually used below permanent groundwater level. May be used in embankments, if properly encapsuled.
Peat	19-64	Long term large settlement is a major concern.
Fuel ash, slag, cinders, etc.	64-100	Such materials may: possess cementing properties; absorb water with time, which may increase density; and leach substance which may adversely effect adjacent structures and groundwater quality.
Scrap cellular concrete	64	Significant volume decrease results when the material is compacted. Excessive compaction reduces the material to a powder.
Expanded Clay or shale	20-64	Possesses good engineering properties for use as lightweight fill; is relatively expensive; and should be encased in minimum of 20 in. soil cover.
Shell (oyster, clam, etc.	70	Commercially mined or dredged shells available mainly off Gulf and Atlantic coasts. Sizes 0.5 to 13 in. (12 to 75 mm). When loosely dumped, shells have a low density and high bearing capacity because of interlock.
Expanded polystyrene	1.3-6	A super light material. The material is very expensive, but the very low density may make it economical in certain circumstances.
Low-density cellular concrete, Elastizell: Class I Class II Class III Class IV Class V Class VI	 24 30 36 42 50 80	This is a lightweight fill material manufactured from portland cement, water, and a foaming agent with the trade name "Elastizell EF" and is produced by Elastizell Corporation of America, Ann Arbor, Michigan. Six different categories of engineered fill are produced. The material is cast in situ and has been used as lightweight fills in a variety of geotechnical applications, such as highway embankments, bridge approaches, foundations, etc.

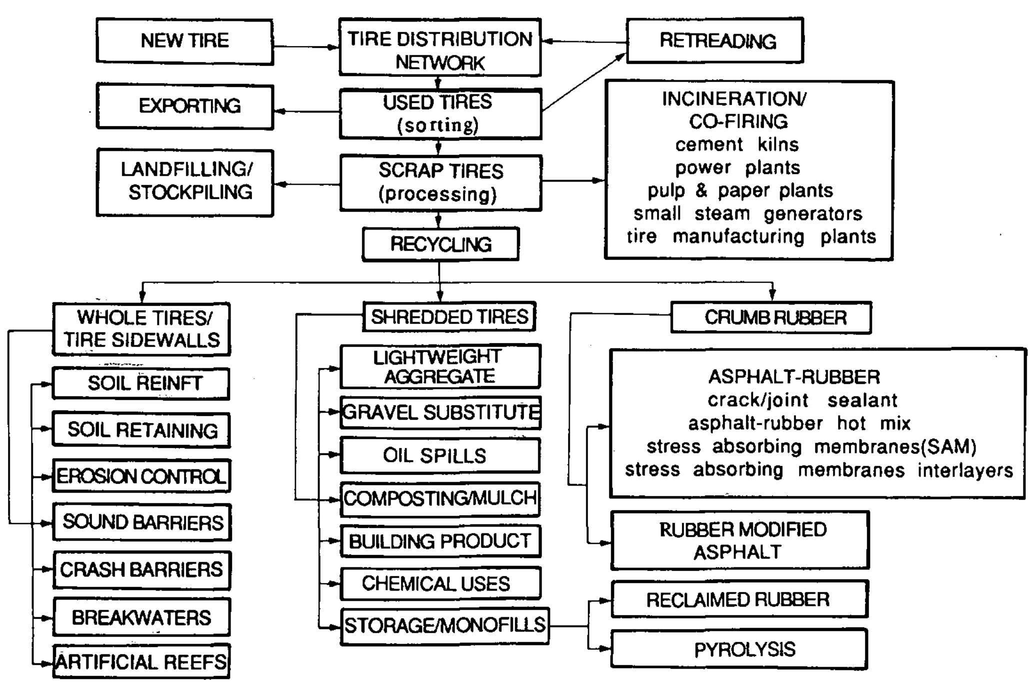

FIGURE 1 Summary of recycling and disposal options for scrap tires.

and other ingredients. A review of available technologies and markets suggests that these applications are not commercially beneficial now. Three possible uses of tires, which hold significant potential for future projection in highway construction, are use of crumb rubber additive in asphalt pavements, use of tires and their products for soil reinforcement, and use of shredded tires as a lightweight material. This paper addresses the use of shredded tires as lightweight fill material in highway construction.

FIELD EXPERIENCE

Various agencies, in the United States and abroad, have evaluated the use of shredded tires as a lightweight material in embankment construction and also for enhancing the stability of slopes in slide areas. The experience of some of the state highway agencies is described in detail.

Minnesota Projects

The Minnesota Pollution Control Agency (MPCA) documented over 23 sites in February 1992 that have used more than 80,000 yd^3 of shredded tires (about 2.2 million tires). More than half of these projects are on privately owned driveways and roads, four on city and township roads, three on county roads, and two on DNR forest roads. A few of the projects used shredded tires for purposes other than in road fills. One project in Minneapolis used the lightweight tire shreds as a fill material to support a park and landscaping above an underground parking garage. At another site, tire chips were used as lightweight fill over an existing water main (*9*).

Its experience indicates that the use of shredded tires as lightweight fill material is technically feasible and cost effective. In Minnesota the tire shreds cost from \$1.25 to \$3.25/yd^3 (\$5 to \$12/ton) delivered to the job site. This cost is further reduced when subsidized by the state to clean up tire dump sites. Economic analysis indicates that tire chips are cost-effective compared with other conventional lightweight materials, such as foamed or cellular concrete and polystyrene. However, there is concern about lack of information on long-term environmental impacts and mechanical behavior of chips (*9*).

Oregon Slide Correction Project

The Oregon Department of Transportation (DOT) used shredded tires in a slide area on U.S. Highway 42 (Oregon State Route 35, Coos Bay–Roseburg), approximately 25 mi west of Roseburg, Oregon (*10*). The construction involved replacement of 12,800 yd^3 of existing soil with 5,800 tons of shredded tires (an estimated 580,000 tires). The tire chips were spread and compacted by a D-8 bulldozer. At least three compaction passes were specified for each 3-ft lift of tires. A 10 percent compression was anticipated on the basis of in situ performance of a tire chips embankment constructed in Minnesota (*11*). It was observed that the thickest portion of the shredded-tire fill (approximately 12.5 ft) compressed 13.4 percent during construction in the following manner (*10*).

- Sixteen in. during placement of 3 ft of soil cap,
- Two in. during placement of 23 in. of aggregate base, and
- Two in. during 3 months of traffic and placement of 6 in. of asphalt concrete.

Read et al. (*10*) concluded that embankment construction using waste shredded tires is a viable technology and can use large quantities of discarded tires with significant engineering benefits. The cost of the tire chips delivererd to the site, by vendors of the shredded tire materials from a distance of 150 to 250 mi, has been reported as \$30/ton. The cost of placing and compacting the tires was \$8.33/ton. The total cost of the fill at final in-place density of 52 pcf, after \$20/ton reimbursement from Oregon Department of Environmental Quality, was \$18.33/ton.

Wisconsin Test Embankment Containing Shredded Tires

The University of Wisconsin–Madison, in cooperation with the Wisconsin DOT, conducted a field experiment to determine the feasibility of incorporating shredded tires in highway embankments (*12,13*). A 16-ft-wide and 6-ft-high test embankment consisting of 10 different sections, each 20 ft long, was constructed. Locally available soil and shredded tires were used in a number of different ways—pure tire chips, tire chips mixed with soil, and tire chips layered with soil. The embankment configuration for different sections of embankment was varied to determine the optimum side slope. A geotextile fabric was placed on all sides of tire chips to serve as a separator between materials of the embankment and the surrounding materials. The embankment was constructed parallel to the access road of a solid waste landfill and exposed to the heavy incoming truck traffic.

Edil et al. (*12*), on the basis of construction and early post-construction evaluations, reported that construction of embankment with tire chips does not present unusual problems. Leachate characteristics indicated little or no likelihood that shredded tires would affect groundwater. The main problem is reportedly related to control of compressibility. Monitoring and evaluating the test embankment for 2 years support the use of properly confined tire chips as a lightweight fill in highway applications (*13*).

Tire Chips Use on New Interstate in Colorado

The Colorado DOT recently experimented with the use of shredded tires as a lightweight fill material (*13*). Shredded tires have been used on a 200-ft portion of Colorado's new Interstate 76. More than 400,000 tires chips of about 4 in. have been used in a 5-ft fill. The tire embankment was instrumented for monitoring the long-term performance of the fill. The shredded tires for this project were donated by the local vendors. The cost of transportation for a 20-mi distance, placement, and compaction was initially estimated to be \$8.00

to $8.50/yd^3. The actual cost of the project has not yet been published.

LABORATORY STUDIES

Wisconsin Study

A limited experimental program was carried out at the University of Wisconsin–Madison to develop quantitative information about the compaction and compression behavior of tire chips and analysis of leachates from a test embankment made of rubber soil (*12*). The experiment involved placement of rubber chips of different sizes, alone and mixed with sand in a 6-in. Proctor mold, followed by load application using a disk placed on the tire chips. The load-deformation response of rubber chips indicated that the major compression occurs in the first cycle of loading. A portion of this compression is irrecoverable, but there is significant rebound on unloading. The subsequent cycles tend to be similar with less rebound; however, the rebound is nearly the same from one cycle to another. It is observed that the slope of the recompression-rebound curve is markedly lower beyond a certain vertical pressure of about 35 psi.

Edil et al. (*12*) also conducted compression tests on rubber-sand mixes, varying sand and chip ratios. Their tests on rubber-sand mixes yielded compression curves similar to rubber chips alone. However, the maximum compression increased as more and more cycles of loading took place, and the magnitude of the maximum compression was less than 0.1 in. as compared with about 2 in. for the tire chips alone. The test results, on specimens of sand and chip ratios varying from 100 percent sand to 100 percent chips, indicated that the compression increases significantly when tire chips content was increased more than 30 percent by weight of sand. Edil et al. performed experiments in a compaction mold that was probably too small in diameter for the size of chips tested (chip sizes of 1.5 in. and even larger were tested in a 6-in. Proctor mold).

Edil et al. (*12*) have also reported duplicate EP toxicity and AFS leaching tests performed on tire chip samples by the Wisconsin State Laboratory of Hygiene. The EP toxicity test was run for barium, cadmium, chromium, lead, and mercury but not for arsenic, selenium, or silver. The AFS test procedures were followed for evaluating the leaching behavior of metals, anions, and organic and inorganic indicator parameters. The test results indicate that the shredded automobile tire samples show no likelihood of being a hazardous waste. The shredded tires appear to release no base-neutral regulated organic materials. The tire samples showed detectable, but very low release patterns for all substances and declining concentrations with continued leaching for most substances. Bosscher et al. (*13*) reported that an overall review of the available leach data and results of the recent leach tests on samples collected from two lysimeters, installed during construction of the test embankment in December 1989, confirm that shredded automobile tires show no likelihood of having adverse effects on groundwater quality.

Minnesota Study on Tire Leachates

The MPCA sponsored a study on the feasibility of using waste tires in subgrade road beds (*14*). Twin City Testing Corporation (TCTC) of St. Paul, Minnesota, performed the laboratory study to evaluate the compounds produced by the exposure of tires to different leachate environments. As a result of elaborate testing and analysis, TCTC reached the following conclusions (*14*):

- Metals are leached from tire materials in the highest concentrations under acid conditions; constituents of concern are barium, cadmium, chromium, lead, selenium, and zinc.
- Polynuclear aromatic hydrocarbons and total petroleum hydrocarbons are leached from tire materials in the highest concentrations under basic conditions.
- Asphalt may leach higher concentrations of contaminants of concern than tire materials under same conditions.
- Drinking water recommended allowable limits (RALs) may be exceeded under worst-case conditions for certain parameters.
- Codisposal limits, EP toxicity limits, and TCLP criteria are generally not exceeded for the parameters of concern.
- Potential environmental impacts from the use of waste tires can be minimized by placing tire materials only in the unsaturated zone of the subgrade.

Permeability of Tire Chips

A laboratory study was conducted by Bressette (*15*) to determine the feasibility of using tire chips as an alternative to conventional aggregate in drainage layers or channels. Bressette performed constant head permeability tests on compacted and uncompacted specimens of chopped scrap tire material (approximately 2-in. squares), shredded tires (100 percent passing 2-in. sieve), and coarse aggregate (open-graded, percent passing sieves 2-, 1½-, 1-, ¾-, and ½-in. was 100, 99, 43, 39, and 1 percent, respectively). The permeability values for the three materials were within the same order of magnitude—10^4 ft/day (3.53 cm/sec), with only 3 exceptions in 42 tests. All values were in the upper range of permeability values required for subdrainage material.

Blumenthal and Zelibor (*16*) reported the study, performed by Shive-Hattery Engineers & Architects, Inc. (1990) for the Iowa Department of Natural Resources, that investigated the hydraulic properties of shredded scrap tires as a drainage soil substitute. They found that the average coefficients of permeability of 1.5-in. and 0.75-in. scrap tire chips were 2.07 and 1.93 cm/sec, respectively.

LABORATORY TESTING OF RUBBER SOILS

Testing Materials

The first phase of this study consisted of determining the compaction and compression behavior of rubber soils. The testing program was formulated to develop quantitative information about the compaction and compression characteristics of the tire chips alone and when mixed with different soils. The tire chips used for this study were supplied by ASK Shredders Corporation, East Chicago, Indiana; Baker Rubber, South Bend, Indiana; Rubber Materials Handling, East Chicago, Indiana; and Carthage Machine Company, New York. The samples of tire chips vary in size from sieve No. 4 to 2 in. plus. The tire chips have generally clean cuts, and only a small percentage of steel wires is exposed at the edges. A mechanical analysis was performed on tire chip samples collected from the various shredding agencies, the results of which are given in Figure 2. The grading curves of various chip samples generally indicate a uniform gradation of tire chip samples.

Two types of soils, fine and coarse grained, were used for this study. Crosby till, which is a natural fine-grained soil, has been routinely used in many research studies at Purdue University (*17*). The test soil was thoroughly mixed in the laboratory to eliminate the possibility of spatial variabilitiy in the properties of this natural soil and to correctly understand the effects of adding rubber chips on the compaction and compression behavior of soil. The soil has been classified as CL-ML (sandy silty clay) according to the Unified Soil Classification System (USCS) and A-4(0) according to the AASHTO classification system. The coarse-grained soil used in this study is white medium to fine Ottawa sand. The desired gradation was achieved by mixing three different types of Ottawa sand in equal proportions—Flintshot (AFS Range 26-30), #17 Silica (AFS Range 46-50), and F-125 (AFS Range 115-130). The sand is classified as SP (poorly graded sand) according to USCS and A-3(0) according to AASHTO classification system. The grain size distribution curves of the test soils are given in Figure 2.

Compaction Testing

The compaction tests conducted for this research were performed using manual compaction, a mechanical compactor, and an electromagnetic, vertically vibrating table. The compaction tests on Crosby till were performed following procedures described in ASTM D698 (AASHTO T99-61) and ASTM D1557 (AASHTO T180-61). A mechanical rammer and 6-in. diameter mold were used to perform the compaction tests on rubber soil with tire chips of sizes up to 1 in. A steel mold 12 in. in diameter and 12.5 in. high was used for testing chip sizes up to 2 in. The compaction tests on Ottawa sand were performed using procedures described in ASTM D4253. An electromagnet, vertically vibrating table was used for providing the desired level of vibration. The variables considered included compactive effort, size of chips, and the ratio of soil and chips. Three different compactive efforts were used—modified Proctor, standard Proctor, and 50 percent of standard Proctor. Tire chips of seven different sizes ranging from sieve No. 4 to 2 in. plus are investigated in this study. The soil and chip ratios were varied from pure soil to pure chips—quantity of chips in mix varied from 0 to 100 percent of dry weight of soil.

The following conclusions are drawn, on the basis of a critical analysis of the results obtained from the compaction testing of rubber soils and rubber chips alone (*4*).

- It is found that vibratory methods of compaction are suitable for rubber sands. Nonvibratory methods (e.g., Proctor-type compaction) are more appropriate for compacting chips alone and mixes of chips and fine-grained soils.
- The effect of compactive effort on the resulting unit weight of rubber soils decreases with increasing chip/soil ratios. Only a small effect is observed for the amount of chips greater than 20 percent of dry weight of soil (see Figure 3). Figure 4 also shows that the unit weight of chips alone is not much affected by the compactive effort. Only a modest compactive effort is required to achieve the maximum unit weight of chips. This unit weight is about one-third that of the conventional soil fills.
- The chip unit weight is not very sensitive to the size of chips. However, a trend of increasing unit weight with increasing chip size is found, except in the case of vibratory compaction. In this case the maximum unit weight decreases with increasing chip sizes (Figure 4).

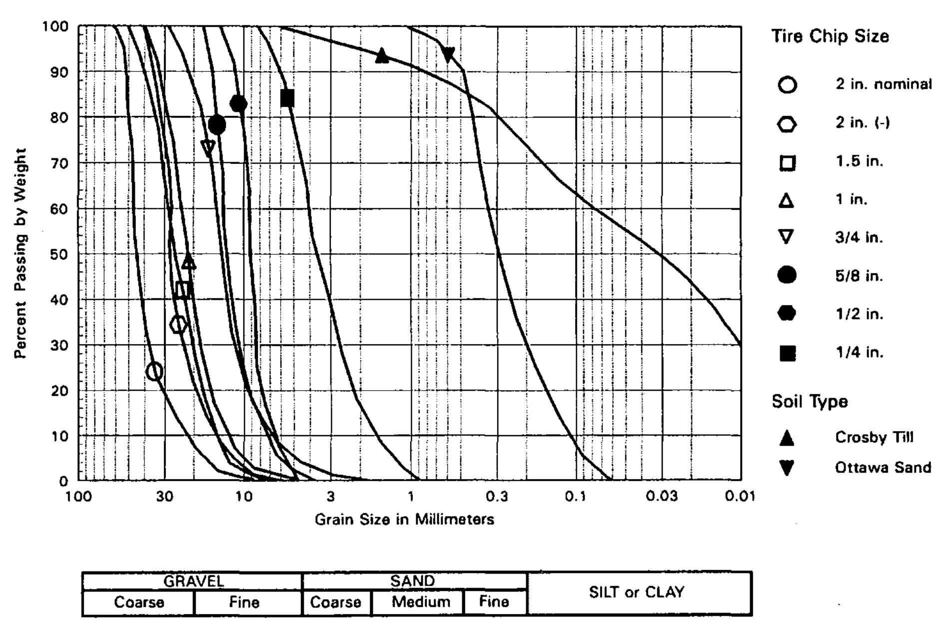

FIGURE 2 Gradations of rubber chips and test soils.

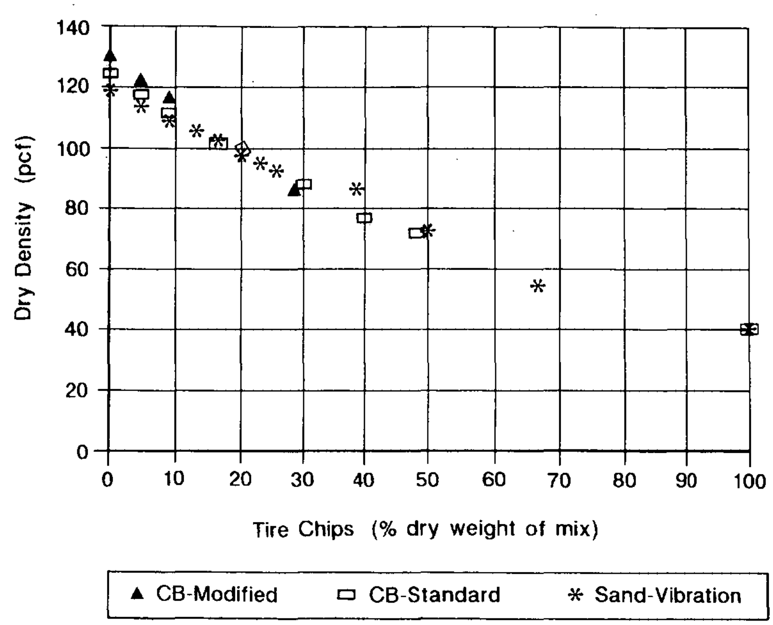

FIGURE 3 Comparison of compacted densities of rubber sand and rubber soil samples.

Compressibility Testing

A stainless steel compression mold 12 in. in diameter, 12.5 in. high, and having a wall 0.4 in. thick was used to perform compression tests on tire chips of sizes varying from 0.5 in. to less than 2 in. (See Figure 2 for the gradation curves of various chip sizes.) The samples were compacted in eight layers using a 10-lb hammer with 18-in. drop. Three different compactive efforts were used—modified Proctor, standard Proctor, and 50 percent of standard Proctor. Tests were also performed on uncompacted tire chip samples. All the samples were subjected to four cycles of loading and unloading using an MTS soil testing system. The samples were loaded and unloaded incrementally using a loan increment ratio of one. For the first two cycles, the samples were loaded to a maximum stress of about 25 psi, which is equivalent to approximately 25 ft of soil fill, and then unloaded to a seating load of 0.12 psi. For the third cycle, the samples were loaded to about 15 psi and then unloaded to 1 psi. Finally, in the fourth cycle, the samples were reloaded to the maximum stress and

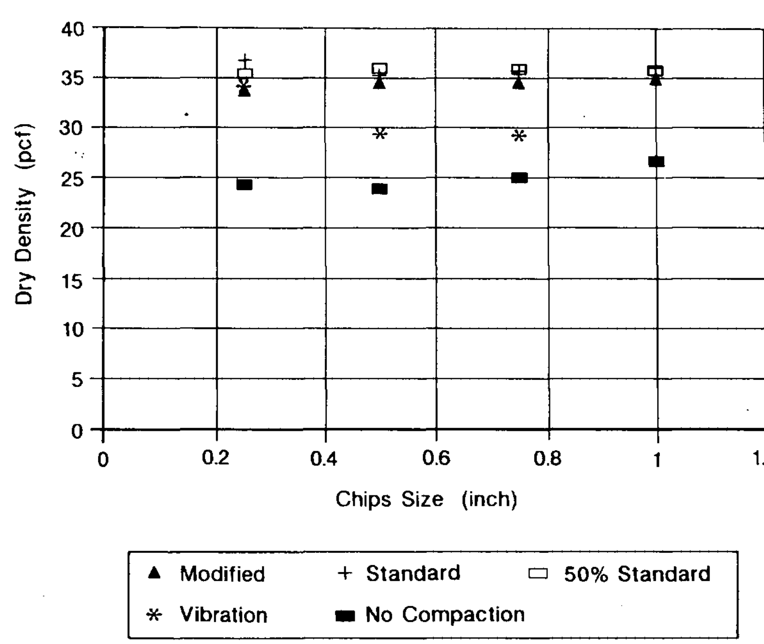

FIGURE 4 Unit weight versus chip size for different methods of compaction and compaction effort levels.

then completely unloaded. Similarly, a blend of rubber-sand mix with tire chips varying from 0 to 100 percent were also tested.

The data obtained were plotted as vertical strain versus log stress. On the basis of a critical analysis of the test results, the following observations are made. The load-deformation response of tire chips (see Figure 5 for typical compression curves) indicates that the three mechanisms mainly responsible for total compression of tire chip samples are (a) compression due to rearrangement/sliding of chips—a small compression occurs, mainly during first loading cycle, and is mostly irrecoverable, (b) compression due to bending/flattening of chips—responsible for the major portion of total compression and mostly recoverable on unloading, and (c) compression due to elastic deformation of tire chips—a small compression occurs because of this mechanism and all of it is recoverable. This indicates that compression of rubber chips can be reduced by increasing confining and overburden pressures or filling air voids with material less compressible than tire chips.

The vertical strain decreased with increasing chip size in the case of samples compacting using 50 percent of standard effort. A maximum difference of about 4 percent was observed for chip sizes varying from 0.5 in. to 2 in. However, variation in chip sizes had little effect on load-deformation response for higher compactive efforts. The higher compression of large-size chips observed in the case of lower compactive effort is mainly due to rearrangement/sliding of particles, because the large-size chips could not be tightly packed by a very small compactive effort.

The increase in compactive effort from standard to modified had no effect on the compression curves for various chip sizes. However, samples compacted using 50 percent of standard effort yielded vertical strains 2 to 4 percent higher during the first loading cycle than those compacted with standard or modified effort. The uncompacted samples also produced higher strains during the first loading cycle. However, compactive effort had little effect on the load-deformation response of chips for subsequent loading and unloading cycles.

Figure 6 shows a plot of vertical strain versus log vertical stress for various ratios of rubber-sand mixes. The curves show that the total compression of samples increases with increasing percent of tire chips, the highest value of compression being for 100 percent tire chips. This demonstrates that a blend of rubber soil provides a mix with lower void ratio, which compresses less than one of pure chips, and will also cause lesser settlement of foundation soil due to reduced weight of fill. About 40 percent chips by weight of soil is an optimum value for the quantity of chips in a rubber-soil mix, where large settlements are a matter of concern. This chip/soil ratio will yield a compacted dry unit weight of rubber-soil mix that is about two-thirds that of soil alone (see Figure 7).

Permeability Testing

A stainless steel mold, 8 in. in diameter, is used to determine the hydraulic properties of compacted samples of tire chips under constant head conditions. The samples are 9 in. high and compacted using three different compactive efforts—modified Proctor, standard Proctor, and 50 percent of standard Proctor. The results indicate that the coefficient of permeability for 1-in. size tire chips varies from 0.54 to 0.65 cm/sec with compactive effort decreasing from modified Proctor to 50 percent of standard Proctor.

DISCUSSION OF RESULTS

A review of commonly used lightweight materials (see Table 1) indicates significant diversity in their engineering charac-

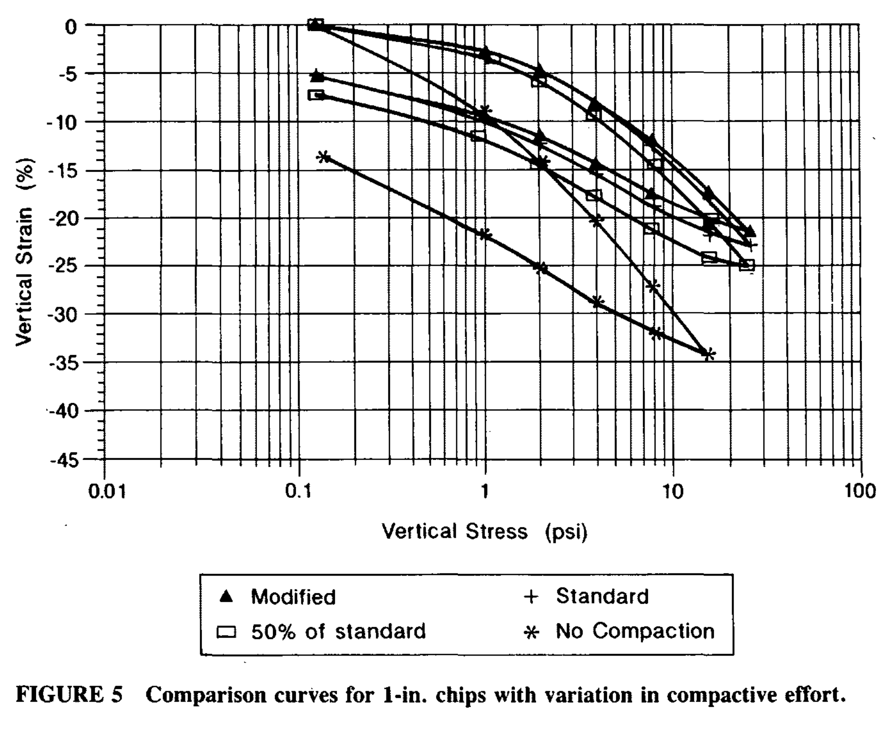

FIGURE 5 Comparison curves for 1-in. chips with variation in compactive effort.

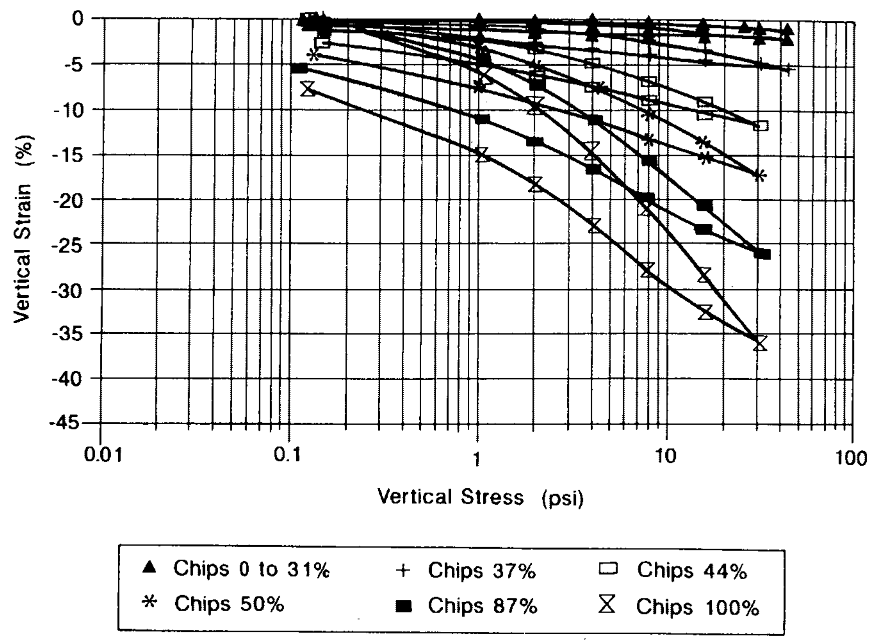

FIGURE 6 Compression behavior of rubber sand with variation in chip/soil ratios—first cycle.

teristics. They also differ widely in their relative cost and impact on the environment. Hence, dry unit weight or any other single characteristics cannot be used as the sole basis for material selection. Some materials, especially manufactured ones, have very attractive engineering properties, but they also cost more. In certain cases some manufactured materials are not available in the large quantities required for highway construction purposes.

Lightweight waste materials, such as sawdust, bark, slags, cinders, and ashes, are generally available in abundance and mostly for no cost at the source. These materials have traditionally been used as lightweight fills by highway agencies in the United States and may be rationally compared with another discard, such as tire chips. Sawdust and bark have unit weights ranging from 35 to 64 pcf (see Table 1), are biodegradable, difficult to compact, require treatment to prevent groundwater pollution, need to be encapsuled by a soil cover, and undergo significant long-term settlement. Salient properties of slags, cinders, and ashes are dry unit weights ranging from 64 to 100 pcf. They may absorb water, resulting

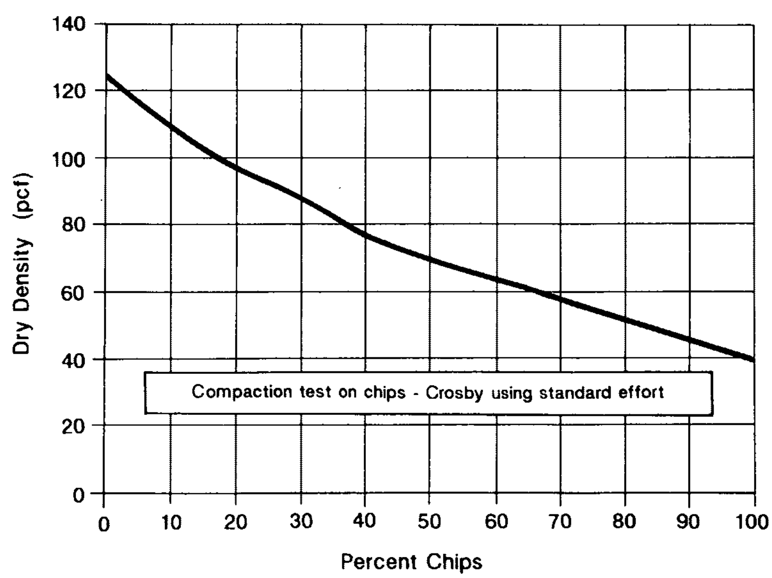

FIGURE 7 Variation in compacted density of rubber chip/Crosby soil mixes with change in percentage chips—standard Proctor effort.

in an increase in unit weight, and have high variability. Leachates may adversely affect groundwater quality or the structures in the vicinity of waste material (see Table 1; *5,18*).

Millions of rubber tires are discarded annually in the United States and tire chips are available in abundance (*7*). Tire rubber has high tensile strength, is chemically stable, and practically indestructible (*19*). Field density of shredded tires varies from 20 to 52 pcf, depending on the size of chips, method of compaction, and thickness of compacted layers (*4*). No unusual problems have been encountered during field compaction of tire chips. A backhoe is suitable for spreading the chips and a D-8 crawler tractor is appropriate for compaction (*10,12*). The environmental impact studies indicate that shredded tires are not a hazardous material, because the parameters of concern do not generally exceed the EP toxicity and TCLP criteria (*12–14*).

To minimize the potential adverse effects of leachates from tire chips, MPCA (*14*) recommended the use of tire chips in unsaturated zones only. The various leaching parameters of concern depend on the environmental conditions prevalent in embankment fill—pH of permeant and soil. Hence, the conditions upon which the conclusions have been based (low pH values) may not exist in a shredded tire embankment.

A major concern in using tire chips in embankments is the large settlements (about 10 to 15 percent) observed in various field and laboratory studies (Figures 5 and 6 and *4,9–12*). Holtz (*2*) emphasizes that little information is available on tolerable settlements of highway embankments. It has been reported (*20*) that postconstruction settlements during the economic life of a roadway of as much as 1 to 2 ft are generally considered tolerable provided they are reasonably uniform, do not occur next to a pile-supported structure, and occur slowly over a long period of time. Postconstruction settlements of shredded tire embankment can be reduced by placing a thick soil cap over tire fills—increasing confining pressure and using a rubber-soil mix instead of tire chips alone. The detrimental effects of settlements can also be reduced by using flexible pavement over such fills and perhaps using stage construction.

Another concern in using tires in embankment may be the potentially combustible nature of tires. To reduce the possibility of fire, a protective earth cover may be placed on top and side slopes of tire embankments. A similar soil cover is recommended for some other lightweight materials, such as wood chips, sawdust, slags, ashes, expanded clay, or shale, for protection against fire or to prevent leaching of undesirable materials into groundwater. During construction, normal caution is required to avoid any fire in tires stockpiled on the site or tires placed in the embankment and not yet capped with soil.

Compacted tire chips (2.0 to 0.75 nominal size) have permeability values equivalent to typical values for coarse gravel (*5,16,21*). This property of chips renders them suitable for use in subdrainage as an alternative aggregate, if feasible environmentally. Pore pressure developments are minimized in tire fills and backfills, because they are a highly permeable material. Use of tire chips in alternating layers with fine grained soils, such as clays and silty clays, will provide a shorter drainage path and thus help to accelerate consolidation of the soil layers.

The use of shredded tires in embankments offers the potential benefit of disposing of large volumes of tires in short sections of highway. For example, the use of an asphalt-rubber pavement overlay uses only about 3,600 tires/mi of a two-lane road. On the other hand 1 mi of two-lane embankment 20-ft high would use about 5 million tires (one tire equals approximately 1 ft^3 loose bulk unit weight before compaction (*15*).

The cost of using shredded tires in embankments depends on a number of factors that vary with the local conditions—cost of chips (primary shreds are generally available now free at the source in most of the states, distance of shredding facilities from the site and the cost of transportation, cost of placement and compaction, subsidies or rebates offered by the state, and the cost of conventional mineral/lightweight aggregates. In Indiana, the major vendors of shredded tire materials are in East Chicago. Currently, they are willing to offer the primary tire shreds without cost. The transportation costs in Indiana vary from $5 to $10/ton for a distance of 100 mi.

CONCLUSIONS

A solution to enhance the stability and reduce the settlement of highway structures on slopes and highly compressible soils is to replace the existing material with a material of lower unit weight or use lighter weight fills. On the basis of an analysis of limited data on rubber soils from this study and those reported in the literature, it is concluded that the use of shredded tires in highway construction offers technocal, environmental, and economic benefits under certain conditions. The salient benefits of using tire chips are reduced weight of fill, which helps increase stability, reduce settlements, and correct or prevent slides on slopes, and reduced backfill pressure on retaining structures. Tire chips serve as a good drainage medium, preventing development of pore pressures during loading of fills. They can be substituted for conventional premeable materials for subdrainage, provide separation to prevent the underlying weak or problem soils from mixing with subgrade and base material, allow conservation of energy and natural resources, and use large quantities of local scrap tires—a positive impact on the environment.

Potential problems associated with the use of shredded tires in highway embankments are leachate of metals and hydrocarbons, fire risk, and large compressibility of tire chips. RALs for Minnesota are found to be exceeded under worst-case conditions (*14*). However, a recent field study reports that shredded automobile tires show no likelihood of having adverse effects on groundwater quality (*13*). However, concerns for long-term effects still persist. Proper soil cover is required on the top and side slopes of shredded tire embankments for safety against fire. During construction, precautions are required to prevent fire in stockpiles or in tires placed in the embankment but not yet capped with soil.

A major concern in using tire chips in embankments is the large settlements (about 10 to 15 percent) observed in various field and laboratory studies. However, potentially large settlements can be reduced by providing a thicker soil cap and using a rubber-soil mix instead of chips alone. It is found that about 40 percent chips by weight of soil is an optimum value for the quantity of chips in a rubber-soil mix, where large settlements are a concern. This chip/soil ratio will yield a compacted dry unit weight of rubber-soil mix that is about two-thirds that of soil alone. Detrimental effects of postconstruction settlements can be reduced by using tires under flexible pavements only and allowing the chips to compress gradually under traffic for some time.

Information on the use of shredded tires in highway structures is severely lacking. Areas of concern are lack of requisite data on stress-strain and strength behavior of chips and chip-soil mix for design and prediction of performance of highway structures, and long-term impact on the environment.

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