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Performance of Prototype Embankment Built with Tire Shreds and Nongranular Soil

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The mechanical response of a prototype embankment fill built with tire shreds and nongranular soil was evaluated. The test embankment consisted of three distinct sections, each 10 m (33 ft) long and 1.5 m (4.9 ft) high. Specifically, the embankment included a layered section composed of successive layers of soil and tire shreds, a soil–tire shred mixed section with 10% tire shreds by weight, and a pure soil section. The embankment was exposed to heavy-truck traffic immediately after construction. At 120 days after construction, the settlement rate in the two sections containing tire shreds converged to a rate similar to that observed in the section of pure soil. However, the section constructed with soil–tire shred mixture exhibited a better overall long-term behavior than the layered section, as it showed smaller differential settlements. The results collected in this study also provide insight into the in situ compression and compaction procedures and preparation characteristics of soil–tire shred mixtures and soil–tire shred layered systems.

Over the past few decades, the amount of scrap tires accumulated in disposal areas has reached alarming levels, especially in industrialized countries. In the United States, it is estimated that scrap tires are generated at an annual rate of about 1 unit per capita. The large size of stockpiles not only poses environmental hazards, such as being breeding grounds for mosquitoes and rodents, but it also constitutes a serious fire hazard. In several cases, tire stockpiles were documented to have experienced internal ignition that resulted in severely damaging fires (*1*). While incineration for tire-derived fuel generation accounts for >40% of the total reuse of tires (*2*), environmental impacts associated with this recycling practice may be as negative as the continued increase of tire stockpiles.

Even though civil engineering applications currently constitute the second largest sector in the reuse of scrap tires, they account for only 14% of the total annual production (*2*). The utilization of scrap tires in civil engineering applications is important because it represents an environmentally satisfactory means to give tires a second life, while contributing to reducing the use of nonrenewable virgin construction materials. The comparatively small unit weight of fills built with scrap tires can be particularly advantageous in projects involving construction over compressible soils and in retaining walls requiring lightweight backfill (*3*).

The practice of using whole tires and pure tire shreds (i.e., without mixing the tires with soil) declined in the 1990s because of reported cases of exothermic reactions within the tire mass of major embankment fills in the United States (*1, 4*). Conversely, no exothermic reactions have been reported to date in structures built with mixtures of tire and soil (*5*). Moreover, laboratory studies have indicated that soil–tire shred mixtures present better mechanical properties than pure tire shreds. This includes lower compressibility and higher shear strength (*5–8*).

The main purpose of this paper is to evaluate the mechanical behavior of a prototype embankment built with tire shreds and nongranular soil. The embankment consisted of three sections with distinct configurations. Specifically, the embankment included a section built with successive layers of soil and tire shreds, a section with a soil–tire shred mixture, and a section with pure soil. The embankment was subjected to heavy-truck traffic immediately after construction, and settlements were monitored for >2 years at different locations. This investigation also provides insight into the in situ compression and compaction procedures for soil–tire shred mixtures and pure tire shreds used as backfill materials. Although the mechanical characteristics of pure tire shreds and soil–tire shred composites have been investigated in laboratory studies, information about the field behavior of embankment fills constructed with these materials is still limited, particularly in structures using nongranular backfill soils.

MATERIALS

The tire shreds used in this research were produced with a hammer mill. The prototype embankment was constructed within the premises of Front Range Tire Recycle, Inc., a recycling and storage facility located in Sedalia, Colorado. The typical dimensions of the tire shreds ranged from 50.8 mm (2 in.) to 152.4 mm (6 in.) long and about 25.4 mm (1 in.) wide. These dimensions correspond to aspect ratios ranging from 2 to 6 (*5*). The specific gravity of the tire shreds is 1.26, which compares well with the values reported in the literature for tire shreds with steel belts (*6*). Water absorption of the rubber after soaking during five consecutive days equals 2.11%, which is in agreement with results reported elsewhere (*9*).

The backfill soil used for construction of the embankment is a silty sand with about 20% of particles passing the No. 200 sieve. The soil classifies as SM according to the Unified Soil Classification System. Results of standard Proctor tests conducted with this soil revealed a maximum dry unit weight of 18.6 kN/m^3 (118.4 pcf) and an optimum water content of 12.6%.

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PRELIMINARY FIELD TESTING PROGRAM

A testing pad was constructed to evaluate the compaction and compressibility characteristics of the different materials used in construction of the prototype embankment. Different procedures for mixing the nongranular soil with the tire shreds were also evaluated. The testing pad consisted of four distinct sections with base dimensions of 3 × 3 m (9.8 × 9.8 ft). Each section was constructed by placing two lifts with a thickness of 0.15 m (6 in.) each. The first two sections were built with mixtures containing 10% and 30% of tire shreds by weight, respectively. The amounts of 10% and 30% were selected based on the results of a previous laboratory testing program (*5*), which revealed that a tire shred content of 30% leads to the maximum shear strength. The third section was built with pure tire shreds, while the fourth section was constructed with pure soil. A sheepsfoot roller weighing 6.7 tons was used to conduct compaction operations. Following recommendations reported by Dickson et al. (*10*), the sections of the testing pad were compacted simultaneously to facilitate the construction process.

Field Compaction Evaluation

Unit weight (γ) and water content (w) were measured in the testing pad with a surface nuclear gauge, following ASTM standards D-2922 and D-3017, respectively. The accuracies of the measurements collected for pure soil were estimated to be ±1% for w and ±0.1 kN/m^3 (±0.6 pcf) for γ. Calibrations were performed in the laboratory with comparatively large samples in an attempt to reduce the influence of material heterogeneity and account for the broad range of sizes and shapes of tire shreds. The accuracy of field measurements of the unit weight of soil–tire shred mixtures was estimated to range from ±0.1 to ±0.8 kN/m^3 (±0.6 to ±5.1 pcf) for tire shred contents ranging from 0% to 100%, respectively.

Figure 1 indicates the changes in dry unit weight (γ_d) of the tested materials (i.e., soil, soil–tire shred mixtures, and pure tire shreds) with increasing number of passes of the compaction equipment. The dry unit weight results presented in the figure correspond to those obtained in the second lift of each section. Measurements were collected after the second pass of the compaction equipment and continued to be collected after subsequent passes (except for the fifth pass). The dry unit weight of the soil increased slightly from the second to the third pass of the roller but remained essentially constant beyond that compaction effort. After the third pass, γ_d in the pure soil section reached 17.9 kN/m^3 (113.9 pcf), which corresponds to a relative compaction of 96% in relation to the standard Proctor test. The soil–tire shred mixture with 10% tire shreds exhibited slightly lower values of γ_d compared with that of pure soil. Only minor changes in γ_d were observed after the second pass of the roller in the 10% tire–soil mixture, reaching a value of 17.1 kN/m^3 (108.8 pcf) after the third pass.

The test results obtained for the tire shred layer and for the 30% soil–tire shred composite revealed some scatter compared with those obtained for the previously tested materials. The higher scatter can be attributed to the presence of a larger amount of tire shreds. The average dry unit weight achieved in the 30% mixture equals 14.1 kN/m^3 (89.8 pcf). Despite the scatter, it may be concluded that changes in γ_d are negligible after the second pass of the roller. The results obtained with pure tire shreds show more scatter than the results obtained with 30% tire shred mixture. An average dry unit weight of 6.6 kN/m^3 (42 pcf) was obtained, which is within the range of typical values reported in the literature (*11*).

Consistent with results reported elsewhere (*12*), most of the compression observed in soil–tire shred mixtures and in pure tire shreds takes place with few passes of the compaction equipment, with a very small increase in compaction occurring afterward. Specifically, the increase in unit weight of pure soil was negligible after three passes of the roller. Based on the finding obtained in the testing pad, the number of passes was conservatively selected as four for construction of the prototype embankment.

Load-Displacement Behavior of Pure Tire Shreds

Figure 2 presents results of a plate load test conducted on the surface of a testing pad containing pure tire shreds. The test was carried out

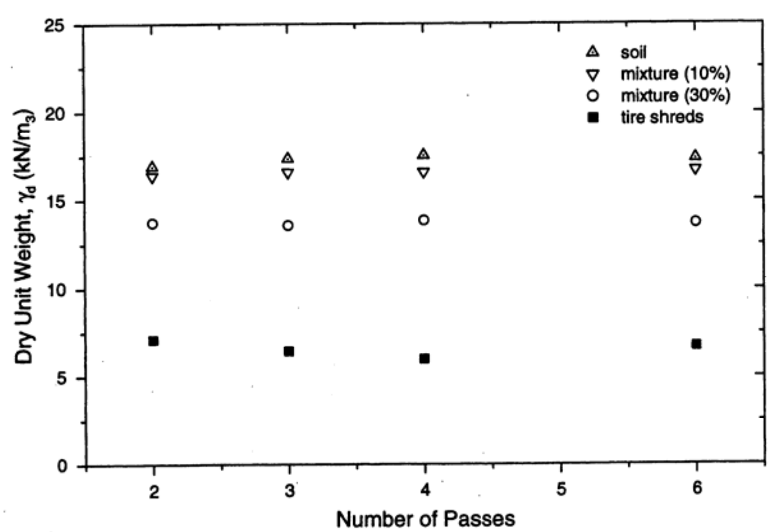

FIGURE 1 Dry unit weight versus number of passes for different tested materials.

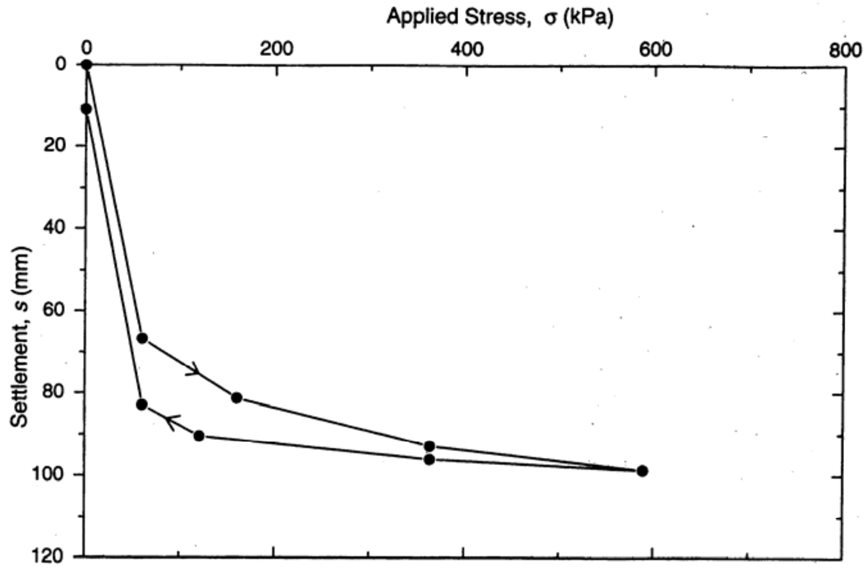

FIGURE 2 Stress–settlement relationship of pure tire shreds.

following ASTM Standard D-1196-93 for use in the evaluation and design of airport and highway pavements. The test was performed with a square steel bearing plate with a side length of 0.3 m (1 ft) and a thickness of 19 mm (¾ in.). The plate size and shape were chosen to simulate the loading area of a typical truck tire. The bearing plate was loaded in cumulative increments with a hydraulic jack and a hand pump assembly reacting against a wheeled loader with an approximate total weight of 15 tons. Load measurements were conducted with the hand pump manometer. Settlements were obtained with two dial gauges with a maximum stroke of 50 mm and a resolution of 0.01 mm. The dial gauges were mounted on two reference beams with articulated magnetic bases. The dial gauges were placed near the edges of opposite sides of the bearing plate, with their tips positioned directly on the surface of the bearing plate. The magnitude of the load used during the test was based on the maximum load of a standard AASHTO H-15 wheel load (53 kN or 12,000 lb).

The shape of the curve in Figure 2 is similar to that reported elsewhere (*6, 13*) for laterally confined tests on pure tire shreds. The settlements that take place for applied stresses of up to about 65 kPa (1,357 psf) are significant. However, the magnitude of the induced settlements decreases significantly for higher stress levels, and settlements reach a plateau at a stress of about 300 kPa (6,266 psf). The rebound upon unloading is significant, with plastic settlements accounting for only 11% of the total settlement. Because the tire shred layer was compacted, settlements were mostly due to bending and elastic deformation of individual shreds. This is because most settlements are caused by rearrangement and sliding of the tire shreds, which are expected to have taken place during the previous compaction process.

The stiffness of the layer of pure tire shreds was characterized by the modulus of subgrade reaction (K), which is estimated as follows:

$$K = \frac{\sigma}{s} \tag{1}$$

where σ is the stress level and s is the settlement. For the tested material, the computed K equals 6 MN/m^3 (38,193 pcf). The stress and settlement values used in Equation 1 correspond to the final loading stage. Typical values of K reported in the literature for 0.3-m-square footings range from 13.5 to 540 MN/m^3 (85,935 to 3,437,420 pcf) (*14*).

After characterization of the stress-displacement response of the system, the field Young's modulus (E) can be obtained by using elasticity concepts, as follows:

$$E = \frac{\sigma B(1 - \nu^2)}{s} I_s I_d I_h \tag{2}$$

where

B = plate diameter or width,
I_s = shape–stiffness factor,
I_d = embedment factor,
I_h = layer thickness factor, and
ν = Poisson's ratio.

Typical values of Poisson's ratio (ν), obtained with uniaxial tests on pure tire shred specimens, range from 0.2 to 0.3 (*6*). Considering the full loading cycle and assuming $\nu = 0.25$, $I_d = 1$, and $I_s I_h = 0.609$ (*15*), the Young's modulus E was estimated as 307 kPa (6,412 psf).

Field Mixing Procedures

Five different procedures were evaluated before construction of the prototype embankment to establish efficient in situ procedures for placing thoroughly mixed layers of soil–tire shred mixtures with equipment typically available in highway construction. The characteristics of each method, including their advantages and shortcomings, are presented in Table 1.

Mixing was very difficult to control by Procedure P1, as this procedure led to significant contamination of the backfill soil and the tire shreds with the subgrade soil. Procedures P3 and P4 allowed a relatively fast mixing but required the use of several pieces of construction equipment. Procedure P5 was considered inadequate

TABLE 1 Methods Evaluated to Produce Homogeneous Soil–Tire Shred Mixture

Procedure	Used Equipment	Procedure	Comments
P1	One wheeled front-end loader with a bucket equipped with a straight blade edge (capacity 1.6 m^3).	a) The target volume of tire shreds was loaded and dumped on the ground; b) the target volume of soil was loaded and dumped over the tire shreds; c) both materials were mixed together with the loader.	Collecting the materials from the ground was difficult to control using this equipment. A significant amount of subgrade soil contaminated the backfill soil–tire shred mixture.
P2	One wheeled front-end loader, with bucket equipped with a tooth-set edge (capacity 1.6 m^3).	Same as Procedure P1.	The bucket equipped with a tooth set allowed better mixing than a bucket with straight edge. Contamination of the backfill soil–tire shred mixture was less than in Procedure P1.
P3	Two wheeled front-end loaders similar to that used in Procedure P1.	a) The target volume of tire shreds was dumped from bucket of loader 1 into bucket of loader 2; b) the target volume of soil was dumped from bucket of loader 1 into bucket of loader 2; c) the total volume was dumped into the bucket of loader 1, and then back to loader 2.	This method generated a better mixture and was faster than methods 1 and 2; however, this procedure required two pieces of equipment. Also, a considerable amount of mixture fell from the buckets during the mixing process.
P4	Two wheeled front-end loaders similar to those used in method 3 and a dump truck.	a) The target volume of soil was loaded into the truck with loader 1; b) the target volume of tire shreds was loaded into truck with loader 2; c) the loaders continued this process until all of the mixture had been placed into the dump truck; d) the mixture was dumped on the ground	This method was faster than method 3, and also generated cleaner mixtures without losses. However, it mobilized three pieces of equipment.
P5	Wheeled front-end loaders used in Procedure P1 and a scraper.	a) The loaders dumped a layer of tire shreds next to a layer of soil; b) the scraper ran across these layers in order to mix them.	Materials were not mixed thoroughly using this procedure.

because it did not lead to a homogeneous mixture. Although somewhat slower than Procedures P3 and P4, Procedure P2 was selected for the final embankment project because it required only one piece of equipment and led to satisfactory mixtures. The mixing procedures took about 10 minutes to prepare 3 m^3 (3.9 yd^3) of soil–tire shred mixtures.

The preliminary field mixing program was useful for assessing the suitability of field mixing of large tire shred contents. Based on laboratory testing, the tire shred content that led to the highest composite shear strength was 30%. However, large-scale production of mixtures containing 30% of tire shreds by weight was not feasible, as it was particularly difficult to prepare homogeneous field mixtures with such high tire shred content. Problems in mixing tire shreds and nongranular soils in the field were also reported by Edil and Bosscher (*6*).

PROTOTYPE EMBANKMENT CONSTRUCTION

The prototype embankment was constructed on an access road of the recycling facility. The embankment includes three distinct sections with a length of 10 m (33 ft) and a height of 1.5 m (4.9 ft). Each section was constructed with a base width of 17.5 m (57.4 ft) and a crest width of 9 m (29.5 ft). The west and east side slopes of the embankment were constructed with slope inclinations of 3H:1V and 2.5H:1V, respectively. The embankment was covered by using a 0.3-m (1-ft) thick layer of soil. Figure 3 presents a plan view of the embankment geometry, showing the location of the instrumentation used to monitor its performance and the location where plate load tests were performed.

Figure 4 indicates the geometry of the constructed cross sections. The first section of the embankment (Section A) has two 0.15-m (6-in.) thick layers of pure tire shreds, and two 0.6-m (2-ft) thick layers of pure soil. Section B involves a soil–tire shred mixture with a tire shred content of 10% by weight prepared following mixing Procedure P2. A 0.3-m (1-ft) thick soil cover was constructed over the mixed section. To allow comparison of their performance, the layered and mixed sections were constructed with the same total amount of tire shreds. Section C was built with pure soil and served as a reference for comparison of the behavior of the other two sections.

The first task in the construction of the embankment involved excavating the access road and leveling the surface of the foundation soil. The same equipment used in the testing pad (a front-end loader and a sheepsfoot roller weighing 6.7 tons) was used to build the prototype embankment. Compaction of the prototype embankment was conducted following the same procedure used in the testing pad. Based on the experience collected during construction of the testing pad, three passes of the roller were deemed sufficient for all materials to achieve the target degree of compaction. However, each lift of the embankment was conservatively compacted with four passes of the compaction roller. Compaction quality control involved nuclear gauge measurements, which were conducted after compaction of each individual lift. The data collected during the quality control program showed good agreement with the results obtained in the testing pad. As in the testing pad, a relative compaction of 96% for the soil was achieved.

After construction of the embankment was completed, 15 steel stakes with a length of 0.45 m (18 in.) were installed on the surface of the embankment to survey the settlement of the various sections. The locations in the embankment of the survey points are presented in Figures 3 and 4. The elevations of the points, measured with a theodolite with an accuracy of ±1 mm (±0.04 in.), used a 1.5-m (5-ft)-high plastic pole cemented as reference with the surrounding soil. Immediately after installation of the survey points, the embankment was subjected to heavy-truck traffic in two directions. Settlements were monitored during 824 days. An average of 20 trucks per day passed through the embankment. The maximum registered weight of individual trucks passing through the embankment was 12.4 tons.

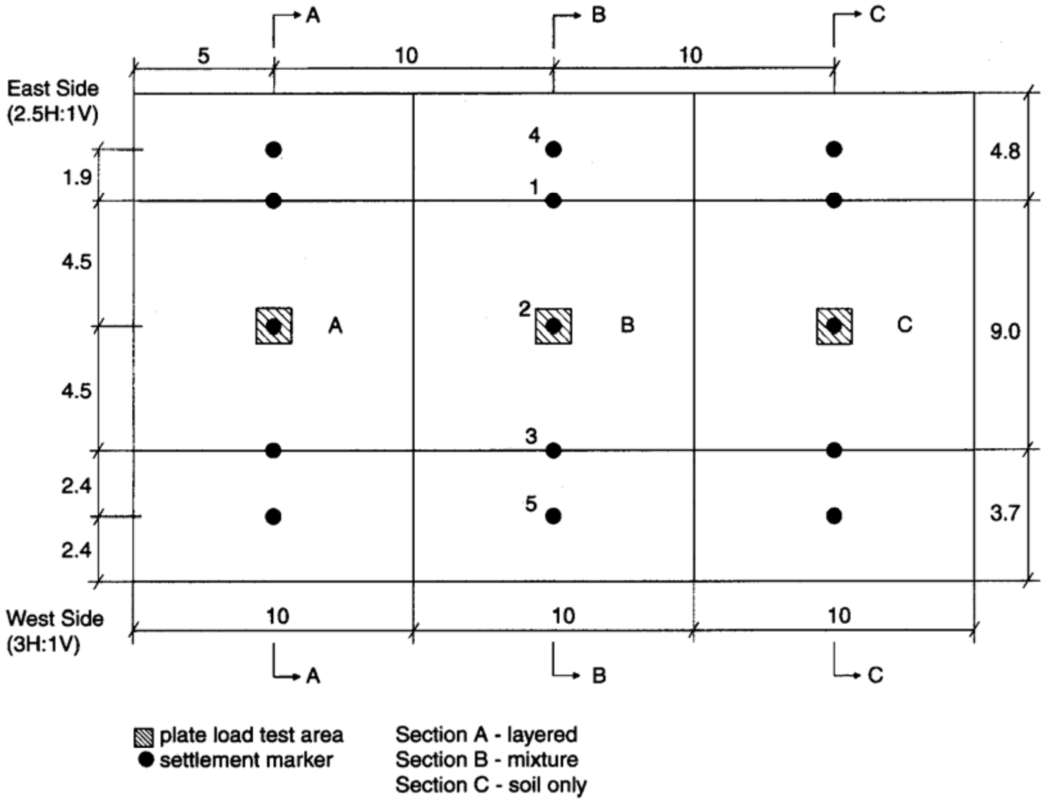

FIGURE 3 Plan view of prototype embankment (dimensions in meters) (H = horizontal, V = vertical).

PERFORMANCE OF PROTOTYPE EMBANKMENT SUBJECTED TO TRAFFIC

As in pure soil embankments, settlements in fills constructed with soil–tire shred mixtures include initial, primary consolidation, and secondary compression settlements. These components may occur simultaneously. Because the embankment is unsaturated, the consolidation settlement component is negligible. However, distortion, bending, and reorientation of the tire shreds embedded in the soil matrix as well as elastic deformation of individual tire shreds upon loading are significant. The interaction between tire shreds and soil particles results in interface friction, which affects the settlement processes. Based on the results of triaxial tests conducted with granular soil–tire shred mixtures with 10% tire shred content and shreds with an aspect ratio of 4, the shear strength of the composite can be characterized by a friction angle of about 39°.

Settlement of pure tire shred fills is basically due to distortion, bending, reorientation, and elastic deformation of tire shreds. The large voids in pure tire shred fills dominate the overall behavior of the material. As a result, settlement of pure tire shred fills is generally larger than that of fills of soil–tire shred composites. Moreover, settlement in pure tire shred fills can also be caused by the migration of soil particles from the cover layer (or from upper layers, in the case of layered systems) into the voids of the tire shred mass. This is analogous to raveling processes that take place in landfills (*16*).

Raveling is usually accompanied by the formation of sinkholes on the embankment surface. Geotextiles have often been used as separation elements between the soil and tire shred layers to minimize raveling. In this study, the embankment was built without separation elements to evaluate the extent of raveling when nongranular soils are used. After more than 2 years of continued traffic loads, the layered section (Section A) developed a few small sinkholes, comparable to those observed in the 10% mixed section (Section B).

Figure 5 presents the settlement as a function of time obtained after subjecting the prototype embankment to traffic loading. The data presented in this figure were collected at the center of each section (Point 2 in Figures 3 and 4). In general, Sections A and B indicated a satisfactory long-term performance. Settlements measured in the layered section of the embankment (Section A) were larger than those measured in the mixed section of the embankment (Section B). However, settlements in both sections involving tire shreds were larger than those measured in Section C (soil only). The maximum settlements in the soil-only section were about 65 mm (2.6 in.). At the end of the survey period (824 days), the maximum settlements in Sections A and B were 87% and 62% larger, respectively, than the maximum settlement in Section C.

The results presented in Figure 5 also indicate that settlement rates in all sections were more pronounced during the initial 120 days but decreased significantly after this initial period. The settlement rates (r) for each section (from the beginning of the survey period to Day 120, Stage I) and the long-term settlement rate (from Day 120 to the end of the survey period, Stage II) are presented in Table 2. During Stage I, the settlement rate in Section A is twice as high as that observed in Section C. Also, the settlement rate in Section B is 63% larger than that obtained in Section C during the same period. Throughout Stage II, Sections A and B indicate comparable settlement rate values, about 0.04 mm/day (0.0016 in./day). This settlement rate is an order of magnitude smaller than the rates measured during Stage I. The settlement rate obtained during Stage II in the sections involving tire shreds is similar to that obtained in Section C (soil only).

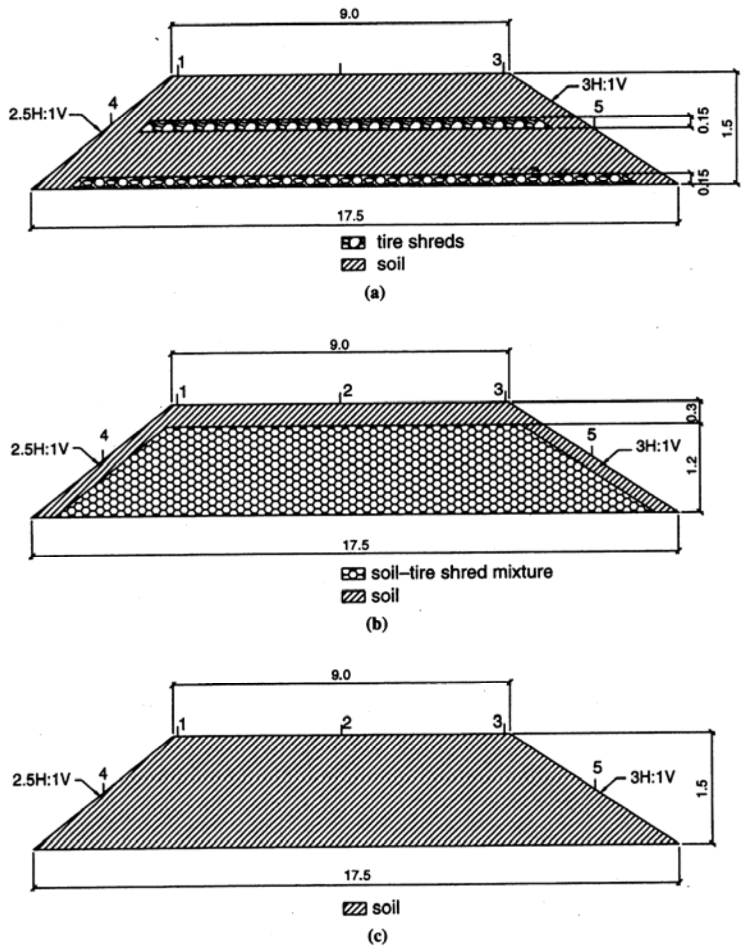

FIGURE 4 Prototype embankment cross-section characteristics (dimensions in meters): *(a)* Section A, *(b)* Section B, *(c)* Section C (H = horizontal, V = vertical).

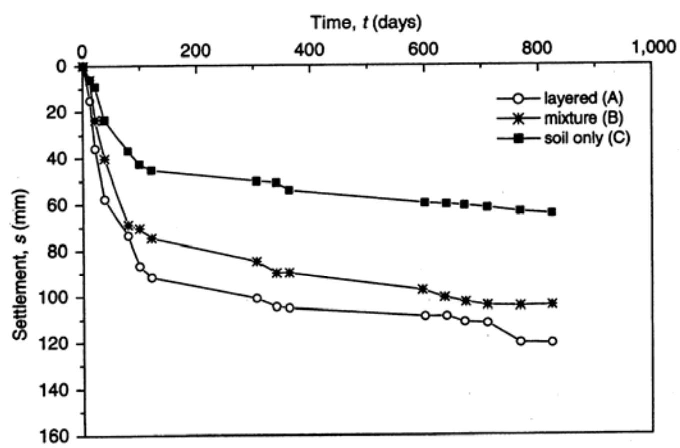

FIGURE 5 Settlement of each section after construction and submission to traffic.

TABLE 2 Settlement Rate of Sections

Period	Settlement Rate, r (mm/day) Section A	Section B	Section C
Until Day 120 (Stage I)	0.760	0.620	0.380
Day 120 to 824 (Stage II)	0.041	0.042	0.027

The settlement component in Section A due to the compression of the two layers of pure tire shreds caused by the permanent weight of the soil layers can be estimated by using the stress–settlement behavior presented in Figure 2. A settlement of 35 mm (1.4 in.) can be estimated for the tire shred layers. This corresponds to about 40% of the total settlement experienced in Section A during the first 120 days after construction.

Figure 6 presents the profiles defined using settlement measurements in the five survey locations across the surface of each section (see Figures 3 and 4). The data in the figure correspond to Days 38, 120, and 824 after construction. In general, Section C exhibits the smallest settlements among all measured locations, followed by Sections B and A, respectively. The settlements measured in Day 38 (Figure 6*a*) in Sections B and C are similar along the lateral slopes. On the other hand, Section A exhibits larger displacements both along the slopes and at the center of the embankment. The settlement pattern at the shoulders is unclear. During Day 120 (Figure 6*b*), settlements in Section A were significantly larger than those measured in the other sections, particularly along the slopes. Although settlements of Section B are more pronounced at the center and shoulders than in Section C, settlements of both sections remained similar along the slopes. Figure 6*c* indicates that, at the end of the survey period (824 days), settlements along the slopes and shoulders of Section A became markedly larger than those at the center. The self-weight of the embankment fill and the presence of the transient loads caused more compression of the two tire shred layers near the edges of the embankment due to the smaller confinement in those regions.

Angular distortions (β) were computed from the differential settlements observed between the shoulders and the center of the sections (i.e., between Points 1, 2, and 3 in Figures 3, 4, and 6), as follows:

$$\beta_{ij} = \frac{\Delta s_{ij}}{L_{ij}} \tag{3}$$

where Δs_{ij} is the differential settlement between Survey Points i and j and L_{ij} is the distance between Survey Points i and j. The calculated distortions are presented in Table 3. The settlement data presented in this table correspond to the final stage of the survey (824 days).

Section C showed comparatively small angular distortions, comparable to the limit value recommended for flexible brick walls (i.e., 1/150) (*17*). Except for the distortion between Points 2 and 3,

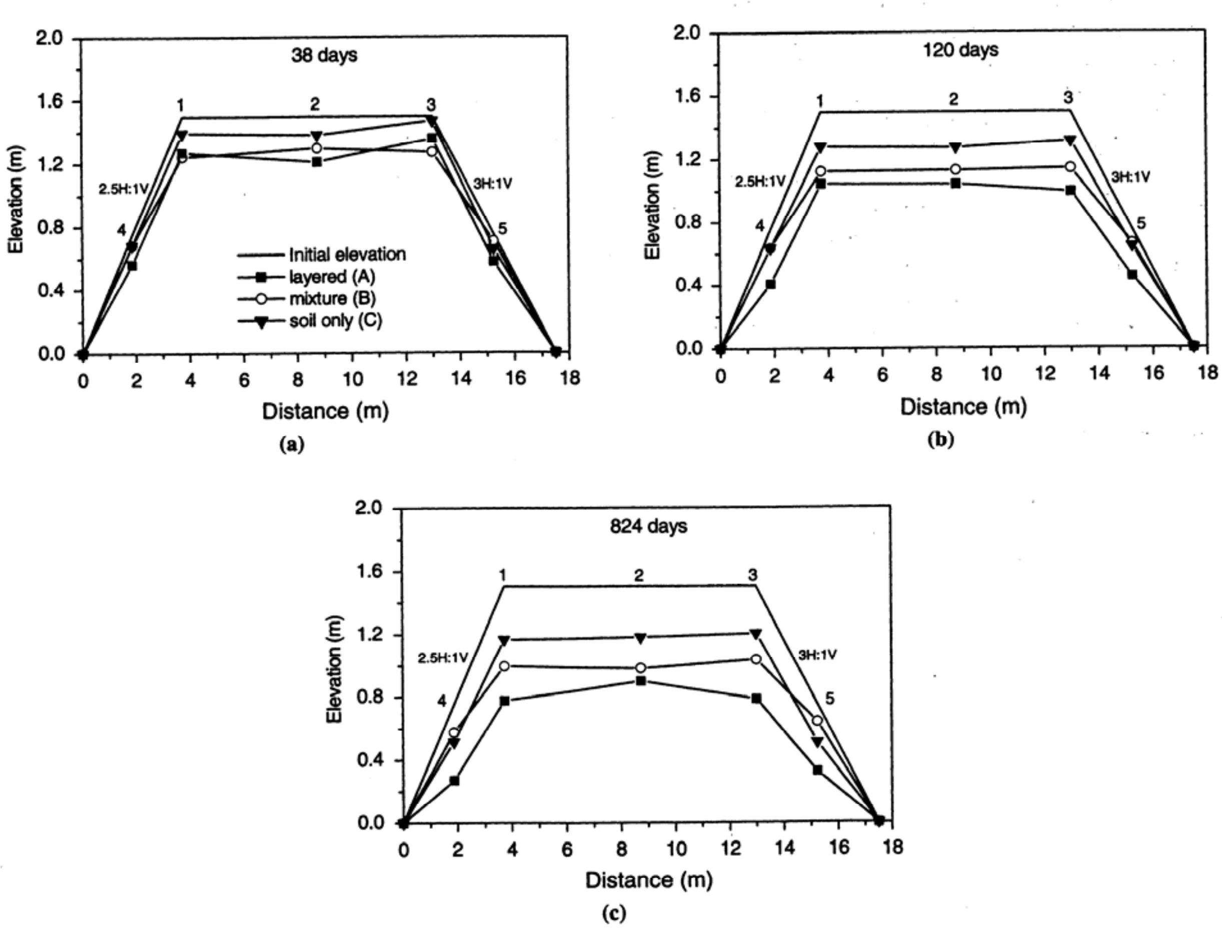

FIGURE 6 Settlement profiles of embankment cross sections (*a*) 38 days, (*b*) 120 days, and (*c*) 824 days after construction (displacements magnified by scale factor of 5).

TABLE 3 Angular Distortions on Top Surface of Sections A, B, and C

Section	Angular Distortion, β		
	β_{13}	β_{12}	β_{23}
Layered (A)	1/592	1/18	1/19
Mixture (B)	1/113	1/169	1/42
Soil only (C)	1/118	1/167	1/92

which was comparatively larger in Section C, the angular distortions in Section B are similar to those obtained in Section C. As previously mentioned, Section A exhibited large differential settlements, with the shoulders settling more than the center.

SUMMARY AND CONCLUSIONS

A field investigation was conducted to assess the mechanical behavior of a prototype embankment built with tire shreds and nongranular soil. The embankment included sections with successive layers of soil and tire shreds, soil–tire shred mixture, and pure soil. The embankment was subjected to heavy-truck traffic immediately after construction, and settlements were monitored for >2 years at different locations across the surface of each section.

Field mixing procedures may not be able to achieve tire shred contents that are achievable in the laboratory. In particular, a tire shred content of 30% could not be achieved in the field. A single-wheeled loader with a bucket equipped with a tooth edge set proved to be efficient for preparing large quantities of soil–tire shred mixtures with 10% tire shreds by weight.

Field monitoring results indicate that embankment sections built with tire shreds and nongranular soil showed satisfactory long-term performance under traffic load. Most of the settlements took place during the initial 120 days after construction. Total settlements in the two sections containing tire shreds were higher than those in the section with pure soil. However, after 120 days, the settlement rate in the sections with tire shreds became similar to that observed in the pure soil section. The section containing the soil–tire shred mixture showed a better overall long-term performance than the layered section.

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