

Effects of Fines Content on Hydraulic Conductivity and Shear Strength of Granular Structural Backfill

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For cast-in-place abutments and retaining walls, structural backfill is preferred to be free-draining, which generally implies less than 5% fines content. This fines content is expected to eliminate the need to design for hydrostatic pressures. The availability of high-quality structural backfill with naturally low fines content is declining. This situation warrants an evaluation of whether granular backfill materials with greater than 5% fines content could be successfully used in practice. Flexible wall, hydraulic conductivity tests on a granular structural backfill with 0%, 5%, 10%, 15%, 20%, and 25% nonplastic fines content were conducted at 41, 83, and 124 kPa (6, 12, and 18 psi) confining pressures followed by consolidated drained triaxial compression tests for obtaining associated drained shear strength parameters of these gradations. The 15.2-cm (6-in.) diameter specimens were prepared at optimum moisture content and 95% of maximum standard Proctor density. To enable a comparison with respect to modified Proctor maximum densities, modified Proctor tests were also performed for all base soil–fines content mixtures. The experimental results were compared with relevant studies found in the literature. This research indicates that a nonplastic fines content up to 10% may be justified in structural backfill specifications for retaining walls and abutments.

The design of retaining walls and bridge abutments commonly relies on the assumption that the soil material used for backfill is free-draining and will not produce hydrostatic pressure against the back of the wall. This is particularly important where retaining walls and bridge abutments are next to rivers and streams and where groundwater is above wall foundation level. The expectation is that reduced fines (particles finer than 0.075 mm) content in a granular structural backfill enables a free-draining condition, thereby eliminating the need to design the abutment or retaining wall to withstand hydrostatic pressure. AASHTO (1) advises that the soil material used as backfill must be drained either through the use of permeable material or by use of an effective drainage system, or both. If not, the wall must be designed for earth pressure loads plus hydrostatic pressure caused by water contained in the backfill.

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Various federal guidelines and recommendations for allowable fines content in structural backfills for abutments and retaining walls, including AASHTO (1), Naval Facilities Engineering Command DM7.02 (2), FHWA (3), U.S. Corps of Engineers (4), and unified facilities criteria (5), either recommend or imply, by specifying free-draining backfill, a maximum fines content of 5%. In many areas, the availability of such a high-quality structural backfill for the construction of transportation projects is declining (6). It is anticipated that as backfill supplies decrease, costs will most likely increase (6). For example, personal communications with contractors in Vermont indicate that up to 20% savings in the unit cost price of the retaining wall backfill item could be achieved by allowing a fines content by weight increase from the current Vermont standard specified maximum of 6% to 10%. The contractors reported that the savings would result from factors that include both local availability near the project site (haul distances) and overall availability as borrow sources are depleted. The Vermont Agency of Transportation (VTrans) evaluated borrow source availability in 1993 (7), finding that available sources were being depleted and that 94.5% of Vermont's remaining deposits are not available for extraction because of inaccessibility, conflicting land use, environmental sensitivity, and poor quality. This finding warrants an evaluation of whether granular backfill materials with greater than 5% fines could be used in practice.

VTrans surveyed state transportation agencies to assess if they allow greater than 5% fines content in granular structural backfill used behind bridge abutments and retaining walls. Table 1 summarizes survey responses from the states that provided a complete set of information, including typical details and specifications in addition to the allowable fines content. Fines content of greater than 5% is permitted in some states, but the survey respondents did not know the basis that led to the specific fines content specification.

This study investigated the effects of nonplastic fines on the hydraulic conductivity and drained shear strength parameters of a typical granular structural backfill by systematically varying the fines content from 0% to 25% in 5% increments, at three confinement pressures. The effects of fines on shear strength were investigated for their impact on computed lateral earth pressures. The basis of comparison was a relative compaction of 95% of the maximum density per standard Proctor test (ASTM D698, AASHTO T 99).

BACKGROUND LITERATURE

Terzaghi and Peck (8) discussed drainage properties of different soil types. For soil types with a hydraulic conductivity ranging between 10^2 to 10^{-4} cm/s (14×10^4 to 0.14 in./h), the drainage performance was

TABLE 1 Summary of Survey Responses from State Transportation Agencies

State	Fines Content (%)	In-Place Relative Compaction Required (%)	In-Place Moisture Content Required (%)	Standard or Modified Proctor Type ^a
Vermont	0–6	90, 95, 100	Optimum ± 2	Standard
Other northeastern states	0–5	95	Optimum	Modified
	0–10	95	Optimum	Standard
	0–12	95, 98	Optimum ± 2	Standard
	0–15	95	Optimum	Standard
Southeastern states	0	92, 97	Optimum ± 2	Modified
Midwestern states	0–2	90, 93, 95	Optimum	Standard
	0–7	95	Optimum ± 3	Standard
	0	98	Optimum	Standard
Western states	0	90, 95	Unavailable	Modified
	5–20	95	Optimum ± 2	Modified

^aStandard = ASTM D 698 and AASHTO T 99; Modified = ASTM D 1557 and AASHTO T 180.

considered good. They noted that soils with hydraulic conductivities as low as 10^{-3} cm/s include clean gravel, clean sands, and gravel mixtures.

Published data in the open literature on the effects of fines on the hydraulic conductivity and shear strength of compacted granular backfill soils by systematically varying the fines content appear to be sparse. Relevant laboratory data found in the literature on hydraulic conductivity measurements of granular soils with varying fines content (nonplastic or nearly nonplastic) are summarized in Table 2. Details of the study presented in this paper are also included in the table.

Merriman (9) conducted falling head permeability tests in a rigid permeameter on three types of compacted natural soils (a fine sand, a coarse sand, and a sand–gravel mixture). The soils were first washed to remove the natural fines and then incremented with natural

nonplastic 0% to 18% silt. Results showed that the addition of nonplastic fines between 0% and 18% reduced the hydraulic conductivity significantly, as summarized in Table 2. Thevanayagam et al. (10) reported results of flexible wall permeability tests on fine Foundry sand No. 55 with varying fines (ground silica Sil-Co-Sil No. 40 manufactured material) content. Hydraulic conductivity of the soil specimens ranged from 0.6 to 1.3×10^{-3} cm/s for the sand, 9×10^{-5} cm/s for 15% silt, 0.6 to 1.2×10^{-5} cm/s for 25% silt, and 3 to 5×10^{-6} cm/s for 60% and 100% silt soils. Tests performed by Siswosoebrotho et al. (11) on granular aggregate with fines contents of 0% to 16% showed that hydraulic conductivity decreased by about one order of magnitude (9.6×10^{-3} to 1.3×10^{-3} cm/s) as the nonplastic fines content increased from 0% to 16%. Mixtures of plastic fines, with a plasticity index range of 5 to 13, had a range of hydraulic conductivity between 1.2×10^{-3} and 2.6×10^{-5} cm/s. Tests were performed in a

TABLE 2 Comparison of Hydraulic Conductivity Testing with Other Investigations

Investigation	Base Soil Type ^a	Fines Content (%)	Moisture Content	Density	Permeameter Type and Sample Diameter	Confining Pressure kPa (psi)	Hydraulic Conductivity (cm/s)
Merriman (9)	Fine sand and coarse sand A-3 (SP); sand–gravel mixture A-1-b (SP) ^a	0–18	Optimum	95% of standard Proctor or 70% RD	Rigid wall, falling head 10.8 cm (4.23 in.)	138 (20) normal stress	Fine sand: 1×10^{-3} to 4×10^{-4} Coarse sand: 2×10^{-3} to 8×10^{-4} Sand–gravel mixture: 9×10^{-3} to 2×10^{-5}
Thevanayagam et al. (10)	Foundry sand #55	0–100	na	na	Flexible wall permeameter	100 (15)	0.6×10^{-3} to 3×10^{-6}
Siswosoebrotho et al. (11)	A-1-a (SP) ^a	0–16	Optimum	95% of modified Proctor	Rigid compaction, falling head 15.2 cm (6.0 in.)	na	9.6×10^{-3} to 1.3×10^{-3}
Bandini and Sathiskumar (12)	50:50 sand A-1-b (SP); ASTM sand A-3 (SP)	0–25	na	na	Flexible wall, constant and falling head 7.0 cm (2.75 in.)	50 (7), 100 (15), 200 (29), 300 (44)	2×10^{-3} to 2.2×10^{-5}
This study	A-1-b (SP)	0–25	Optimum	95% of standard Proctor	Flexible wall, constant flow 15.2 cm (6.0 in.)	41 (6), 83 (12), 124 (18)	2×10^{-3} to 3×10^{-5}

NOTE: Although not specifically reported, particles greater than 19 mm (¾ in.) were probably removed from the base soil before testing. SP = poorly graded sands; na = not applicable; RD = relative density.

^aAASHTO (13) and ASTM D2487.

150-mm compaction permeameter using constant head and falling head tests with samples prepared with the modified Proctor method. Bandini and Sathiskumar (12) reported results of constant and falling head flexible wall permeability tests on two types of uniform, fine sands (Ottawa sand and ASTM 20-30) with fines (ground silica Sil-Co-Sil manufactured material) content of 0% to 25% performed at several confining pressures. Hydraulic conductivity decreased with increasing fines content and confining pressures. The study by Bandini and Sathiskumar (12) is probably the most comprehensive; however, comparison of the effects of fines on hydraulic conductivity at specified relative compaction (which could be related to field compaction) was not done in this study. Shear strength parameters of the sand-silt mixtures were also not determined. Thevanayagam (14), Salgado et al. (15), and Carraro et al. (16) investigated the effects of fines on the stress-strain response of fine sands; however, the basis of comparison was not relative compaction.

EFFECTS OF FINES ON HYDRAULIC CONDUCTIVITY AND SHEAR STRENGTH

Soils, Sample Preparation, and Testing Methods

The grain size distribution of the base granular soil used in this study is shown in Figure 1. For reference, the VTrans current standard specification (17) of structural backfill for abutments and retaining walls is also included in Figure 1. In addition, grain size distributions of the base soils used in other investigations (Table 2) are also

included. All of the base soil used in this study was obtained from a bank run borrow pit comprised of postglacial meltwater sediments deposited in a terrace flanking the shore of early postglacial period Lake Hitchcock, described by Stewart and MacClintock (18). Cobbles in the coarser portions of the sampled deposit ranged to about 4 in. with approximately 2-in. minus size material being sampled for this study, and that material was subsequently screened in the laboratory to ¾-in. minus for compaction, hydraulic conductivity, and shear strength testing.

An abundant source of fines was identified within the same quarry pit where the base material was obtained, which allowed systematic variation of the fines content from 0% to 25%. Hydrometer analysis (ASTM D422) and Atterberg limit (ASTM D4318) results indicated that the fines had similar properties to the natural fines in the base material and were nonplastic or had very low plasticity. Before testing, the base material was first sieved and washed to remove the natural fines and then incremented with the selected fines.

The target specimen densities were 95% of maximum dry densities per standard Proctor test at optimum moisture content. The standard Proctor density is most commonly specified, as per the survey (Table 2). Standard Proctor tests were performed on six combinations of the base soil and fines contents (0%, 5%, 10%, 15%, 20%, and 25% by mass). For comparison purposes, modified Proctor tests were also performed. Specimen densities of 95% of maximum dry density per standard Proctor tests related to about 91% of maximum dry density per modified Proctor tests. All Proctor tests were performed in a 15.2-cm (6-in.) mold per standards ASTM D698 and D1557 because the soil contained particle sizes up to 19 mm

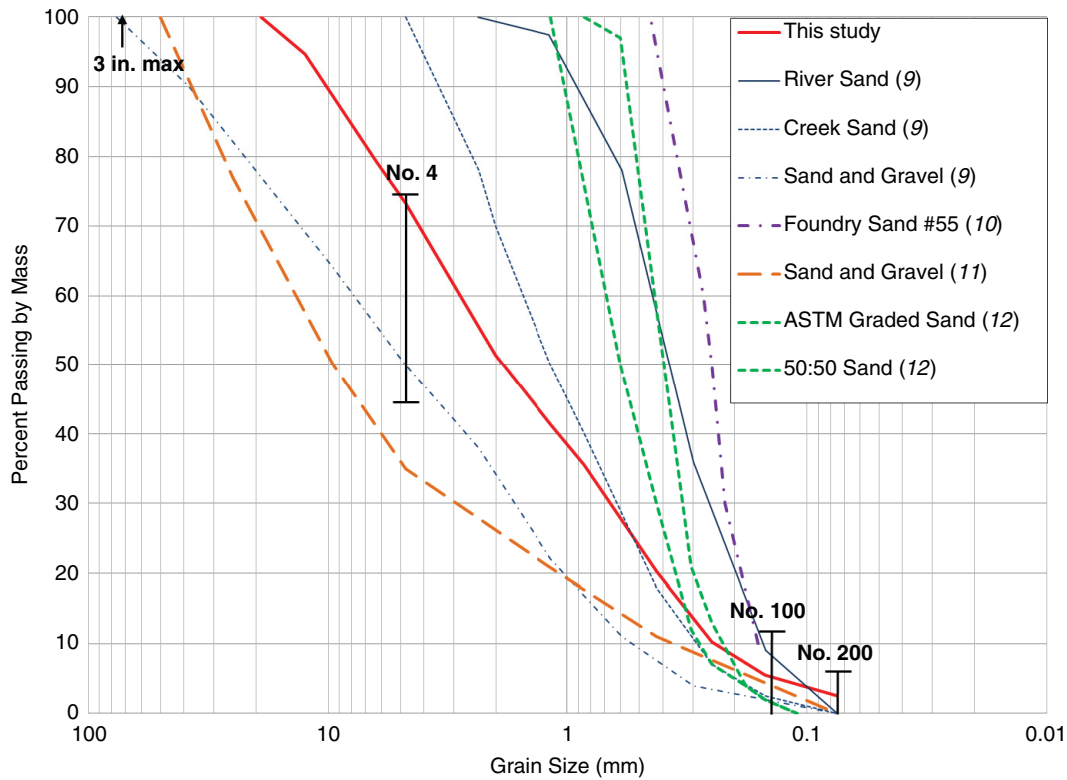


FIGURE 1 Grain size distribution of base material of this study compared with VTrans specifications (vertical bars), with base soils used in other studies included (max = maximum).

TABLE 3 Summary of Standard and Modified Proctor Test Results

Fines Content (% by mass)	AASHTO Soil Classification	USCS Soil Classification	Standard Proctor		Modified Proctor	
			Maximum Dry Density [g/cm ³ (pcf)]	Optimum Moisture Content (%)	Maximum Dry Density [g/cm ³ (pcf)]	Optimum Moisture Content (%)
0	A-1-b	SP	2.01 (126)	10.5	2.11 (132)	7.7
5	A-1-b	SP-SM	2.09 (130)	8.5	2.19 (137)	6.0
10	A-1-b	SP-SM	2.12 (132)	8.0	2.23 (139)	5.3
15	A-1-b	SM	2.18 (136)	7.3	2.26 (141)	5.3
20	A-1-b	SM	2.22 (139)	7.4	2.24 (140)	7.0
25	A-1-b	SM	2.17 (135)	7.9	2.21 (138)	6.0

NOTE: USCS = Unified Soil Classification System; pcf = pounds per cubic foot; SM = silty sand, sand-silt mixtures.

($\frac{3}{4}$ in.). An automated hammer mechanism was used. Table 3 summarizes maximum dry densities and optimum moisture contents obtained from the Proctor tests. In general, the maximum dry density increased and optimum moisture content decreased as the fines content increased. As expected, maximum dry densities and optimum moisture contents from modified Proctor tests were greater and smaller, respectively, than those from standard Proctor tests for a given fines content.

Hydraulic conductivity and consolidated drained triaxial compression (CD) tests were performed using automated Geocomp Flowtrac II flow pumps, 4.4-kN (10-kip) load frame, and a triaxial cell that accommodated 15.2-cm (6-in.) diameter and about 30.5-cm (12-in.) high specimens. Flexible wall permeability tests were chosen to allow for back pressure saturation and to reduce potential side leakage, which is common in rigid permeameters. The specimens were prepared using a split mold to allow the soil mixture to be compacted to the desired initial density. The dry mass of base material and silt was added together, mixed, and then water was added to optimum moisture. This mixture was then compacted in equal layers in the split mold by hand to a fixed height, to the target density. A membrane thickness of 0.635 mm (0.025 in.) was used to reduce the chance of puncture during compaction, and membrane correction was applied in data reduction. After sample preparation, the tubing and pumps were deaired while the sample was allowed to saturate with deaired water under a low gradient before being connected to flow pumps. Back pressure saturation was performed to verify the B parameter of 0.95 or greater using the automated flow pumps. Hydraulic conductivity tests were performed using constant flow. No loss of fines was observed during or after testing and the porous stones were inspected and cleaned before each test.

A head loss check was performed on the system before testing as suggested by ASTM D5084. This test was performed using a hollow Plexiglas cylinder, in place of the soil sample, with a membrane to observe the effect of the tubing and porous stones without any soil. This check showed that the porous stones and losses in other components of the apparatus had little effect on the measured hydraulic conductivities of the soil specimens.

For each combination of the base soil and fines content, three triaxial specimens were prepared. Tests were performed using ASTM D7181 for CD tests. Hydraulic conductivity was measured at 41 kPa (6 psi) confining pressure followed by a CD test for the first specimen. For the second specimen, hydraulic conductivity was measured at a confining pressure of 83 kPa (12 psi) after the 41-kPa (6-psi) measurement and then a CD test was performed. Hydraulic conductivities were measured at 41, 83, and 124 kPa (6, 12, and 18 psi)

confining pressures followed by a CD test for the third specimen. Occasionally, an additional specimen was prepared for repeating a test. All CD tests were conducted at a shearing rate of 0.01%/min.

Experimental Results and Analysis

The hydraulic conductivity measurements are plotted in Figure 2, *a* and *b*. Hydraulic conductivities were measured on three specimens at 41 kPa (6 psi), on two specimens at 83 kPa (12 psi), and once at 124 kPa (18 psi) and were fairly repeatable. As expected, hydraulic conductivity decreased with increasing confining pressure as seen in Figure 2*a*. As seen in Figure 2*b*, hydraulic conductivities of 0% to 10% fines contents were close to each other, and greater than 10^{-4} cm/s (0.14 in./h) for up to 15% fines content. There is a distinct drop in hydraulic conductivity between 10% and 15% fines content, and the hydraulic conductivity continued to decrease with greater fines content.

This investigation included only one base soil. To evaluate if the above conclusions could be generalized to other granular soils, the results obtained here are combined in Figure 3 with other results found in the literature (9–12). The gradations of the soils used by these investigators are included in Figure 1. Other specifics of their test conditions are summarized in Table 2. The base soil types and test conditions in the investigations varied significantly. For example, the base soils included fine sand to mostly gravelly soils. Both rigid and flexible wall permeameters were used and the techniques of measuring hydraulic conductivity also varied (constant head, falling head, and constant flow). The confining pressures also differed. Despite these differences, the estimated best fit of the hydraulic conductivities summarized in Figure 3 is consistent with this study in that the hydraulic conductivity of the granular base soils did not change appreciably for nonplastic fines content of up to 10%.

As described previously, a consolidated drained strength test was conducted on each specimen after the hydraulic conductivities were measured. The effective internal friction angle and effective cohesion values for peak and ultimate failure conditions are summarized in Table 4. The internal friction angle decreased with an increase in fines content. The small effective cohesions are the result of a slight decrease that occurred in peak friction angle with increased confinement and applying a straight-line fit to each of the corresponding failure circles. The peak internal friction angle decreased from about 39 degrees for 0% fines to about 33 degrees for 25% fines. For the same fines content range, the ultimate friction angle decreased from about 35 degrees to 32 degrees.

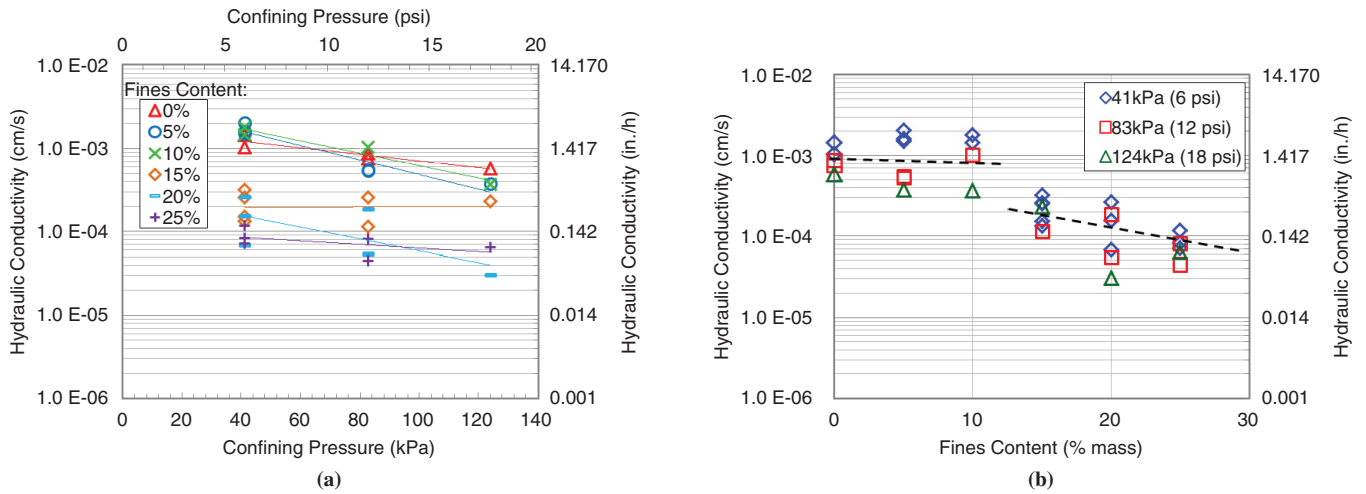


FIGURE 2 Hydraulic conductivity measurements: (a) measured hydraulic conductivity versus confining pressure and (b) measured hydraulic conductivity versus fines content (dashed lines are based on judgment and not statistically determined).

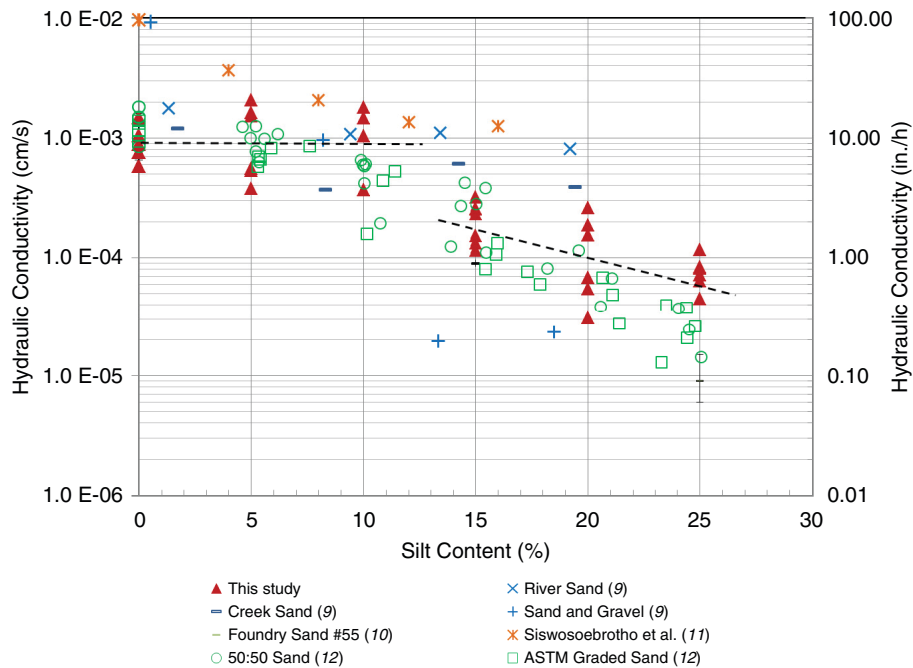


FIGURE 3 Hydraulic conductivities from this study compared with other relevant studies.

TABLE 4 Summary of Drained Shear Strength Parameters

Fines Content (%)	Peak Strength		Ultimate Strength	
	Cohesion [kPa (psi)]	Friction Angle (degrees)	Cohesion [kPa (psi)]	Friction Angle (degrees)
0	12.4 (1.78)	38.9	2.3 (0.34)	34.5
5	11.9 (1.72)	39.7	0.0 (0.00)	35.3
10	8.2 (1.19)	36.4	0.8 (0.12)	34.2
15	19.1 (2.77)	34.1	0.4 (0.06)	35.5
20	6.2 (0.90)	33.4	4.1 (0.60)	32.2
25	5.2 (0.75)	33.1	4.3 (0.62)	31.7

CONCLUSIONS AND DISCUSSION

For the soils investigated, the measured hydraulic conductivities for 0%, 5%, and 10% fines contents were quite close to each other. Hydraulic conductivities were significantly lower for fines content in excess of 15%. These results compared well with other relevant studies found in the literature that included varying granular soil types (fine sand to mostly gravel) and test conditions. A nonplastic fines content of up to about 10% for free-draining structural backfill is well supported by this study and data reported in published work by others. On an empirical basis, the survey of state transportation agencies indicates that some states are using fines contents from 5% to 15% (with one state at 20%) without reports of adverse effects.

How much, if any, of a fines content above 10% can be justified is less clear. Beyond about 10%, there is both greater variability and decreasing permeability such that the free-draining designation is perhaps not justifiable for a broadly applied specification for structural wall backfill. In specific situations that warrant investing in the combination of additional testing, design effort, and attention to controlling material variability in construction, the permeability data reported here indicate that the fines content could potentially be increased a small amount if the specific soils have favorable hydraulic conductivity. However, attention would need to be paid to the potential for greater frost susceptibility and material variability, which would result in added quality control costs for the contractor and quality assurance expenses for the owner.

The effect of fines on drained shear strength showed a decrease in the effective internal friction angle with increased fines. Values of peak and ultimate friction angles varied between 39 degrees and 33 degrees and between about 34.5 degrees and 31.5 degrees, respectively, for fines content between 0% and 25%. If the fines content of up to 10% was allowed, the peak internal friction angle may decrease from about 39 degrees for 0% fines to about 34.5 degrees for 10% fines. For the same fines content range (0% to 10%), the ultimate friction angle may decrease from about 35 degrees to 34 degrees. To generalize this conclusion, however, shear strength tests should be conducted on additional soils.

It is important to note that the objective of considering a backfill soil specification with higher fines content than presently used was for cost savings. The abutment and retaining walls under consideration are considered to be primarily in river crossing settings, and wall performance with the current backfill and bottom of wall weep hole configuration in Vermont has been satisfactory.

Design recommendations for mechanically stabilized earth walls [e.g., National Concrete Masonry Association (19)] with limits of 35% passing the No. 200 sieve size potentially with plastic fines are predicated on employing aggressive drainage measures in the wall backfill configuration to limit groundwater and surface water infiltration from entering those backfill soils. Because introducing the amplified drainage poses a risk of causing undesirable wall performance for abutments and walls subjected to periods of high water, it is anticipated that the current backfill drainage configurations for the cast-in-place walls and abutments probably do not need to be altered.

The effects of initially unsaturated conditions or aging and other effects of compacted soils on permeability and strength and how they change with fines content were not specifically evaluated in the testing for this and the referenced studies, which could be a topic for future investigations.

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