SPECIAL REPORT 97-13

Evaluation of Airport Subsurface Materials

Vincent C. Janoo, Robert Eaton, and Lynette Barna

May 1997

Abstract: Pavement structures located in regions with seasonal changes encounter regular cycles of freezing and thawing. Such environmental factors must be considered so that it can be certain that the pavement can accommodate continuous aircraft loading. Eleven subsurface materials specified by the Federal Aviation Administration (FAA) were examined to determine their susceptibility to frost heave and thaw-weakening. All but two of the materials were found to be frost-susceptible under the U.S. Army Corps of Engineers criterion that no more than 3% of fines be smaller than 0.02 mm (0.78 $\times 10^{-3}$ in.). The frost-susceptible materials were also evaluated using Asphalt Institute criteria, which also categorized them as frost-susceptible. The 11 materials were evaluated for susceptibility to thaw-weakening using the drainage model developed by Casagrande and Shannon (1951), which focuses on the permeability of the drainage layer. The final recommendations (which are based only on a literature review) are that, to reduce frost-susceptibility and thaw-weakening, the amount passing the no. 200 sieve should be kept lower than 2% and drainage layers should be installed below the pavement.

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PREFACE

This report was prepared by Dr. Vincent C. Janoo, Research Civil Engineer, Civil and Geotechnical Engineering Research Division; Robert Eaton, Research Civil Engineer, Research and Engineering Directorate; and Lynette Barna, Engineering Technician, Civil and Geotechnical Engineering Research Division, U.S. Army Cold Regions Research and Engineering Laboratory. Funding was provided by the Federal Aviation Administration.

The manuscript of this report was technically reviewed by Dr. Richard Berg (CRREL), William Quinn (CRREL) and Michel Hovan (FAA).

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CONVERSION FACTORS: U.S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

These conversion factors include all the significant digits given in the conversion tables in the ASTM *Metric Practice Guide* (E 380), which has been approved for use by the Department of Defense. Converted values should be rounded to have the same precision as the original (see E 380).

Multiply	*By*	*To obtain*
inch	25.4	millimeter
foot	0.3048	meter
$foot^3$	0.02831685	$meter^3$
pound	0.4535924	kilogram
pound-force	4.448222	newton
pound / $inch^2$	6894.757	pascal
foot / second	0.3048	meter / second
gallon / day	4.381×10^{-8}	$meter^3$ / second
atmosphere	1013.250	kilopascal
degree Fahrenheit	$t_{°C} = (t_{°F} - 32) / 1.8$	degree Celsius

EXECUTIVE SUMMARY

In areas of the world with seasonal frost, airport pavement structures are subjected to freezing and thawing every year. The effect of frost action on a pavement structure is seen as either uniform or differential frost heave during the winter and subsidence because of thaw-weakening in the spring or during intermittent winter thaws. Pavements constructed in frost areas must be able to accommodate the design aircraft load during the thaw-weakening periods and must also minimize pavement roughness from differential frost heave.

This study was conducted to evaluate the performance of several Federal Aviation Administration (FAA) specifications for subsurface materials with respect to frost heave and thaw-weakening. These specifications, including both unstabilized and stabilized materials, were for subbase (P-154), lime-treated subgrade (P-155), aggregate base (P-208), crushed aggregate base (P-209), caliche base (P-210), lime rock base (P-211), shell base (P-212), sand–clay base (P-213), soil–cement base (P-301), cement-treated base (P-304) and econocrete subbase (P-306). No actual tests were conducted in this study; the data presented were obtained from reviewing the literature. Most of the unstabilized materials allowed up to 15% fines passing the no. 200 sieve, with the exception of the crushed aggregate base, which allowed no more than 8% fines, and the sand–clay base, which allowed up to 25% fines.

The Corps of Engineers (COE) criterion for non-frost-susceptible soils is that there be no more than 3% fines smaller than 0.02 mm (0.78×10^{-3} in.). Only the P-154 and P-209 specifications met this criterion. After reviewing several other criteria, we decided to use a criterion that differentiated the frost-susceptibility of the material on the basis of the percentage passing the no. 200 sieve. This was considered acceptable, since several studies showed a correlation between frost heave and percentage passing the no. 200 sieve. However, this criterion has been found to be conservative, i.e., materials that would pass the COE criterion failed to pass the no. 200 criterion. Using the Asphalt Institute criteria, we evaluated other base and subbase materials, finding that, within the limits specified, the remaining unstabilized materials were frost-susceptible.

We also discovered that for oolitic limestone, the percentage passing the no. 200 sieve had no influence on frost heave. This was not the case for hard limestone. For the oolitic limestone, the amount of moisture present in the layer had a significant effect on frost heave. This finding also probably applies to the caliche and shell base materials. The addition of slag as substitute aggregate, up to 50% of the material, reduced frost heave.

Limited information was found in the literature concerning the frost-susceptibility of stabilized (lime and portland cement) soils. The data were mostly for fine-grained soils such as ML, CL and CH, and for one freeze application. We found no data concerning the effect of freeze–thaw cycling on frost heave. The data that we did find showed that a minimum of 3% lime or cement is required to reduce frost heave by about 50%. The addition of a pozzollith to lime or cement appeared to reduce frost heave significantly in ML and CL soils. In cohesionless soils, it was reported that about 3 to 8% cement is required to reduce frost heave. For frost-susceptible gravel soils, 2% cement is required to change it to a non-frost-susceptible material. It should be noted that if insufficient time has been allowed for the material to cure, frost heave up to 10 cm (4 in.) has been measured in lime–fly ash stabilizing bases.

Thaw-weakening of the base material was inferred from the time it took to drain an airport pavement by 20, 50, and 100%. Studies have shown that for roadway pavement, if the base course saturation level could be reduced from 100 to 80%, the base became stable under traffic loading. The COE drainage requirement is that 50% of the base be drained within 10 days. The drainage model developed by Casagrande and Shannon (1951) was used here. This model was calibrated with data obtained from several northern airfields during the spring thaw. The critical property for drainage is the permeability of the layer. After studying the literature, we decided to use the results from Barber and Sawyer (1952) to estimate the permeabilities of the bases. The permeabilities from Barber and Sawyer were a function of the percentage finer than the no. 200 sieve and the type of the fines. The final values chosen for this study were similar to those used by Casagrande and Shannon. We did our analysis with four fines levels and three thicknesses.

The results of the analysis indicated that none of the materials passed the COE criterion. Some

of the materials came close to it, suggesting that, if the fines contents were further reduced to 2 or 3%, they may meet the criterion. As for the P-213 material, the base appeared to remain saturated all year round. This material should *not* be used in seasonal frost areas.

There were not many data in the literature on the resilient modulus or strength of base and subbase materials subjected to freeze–thaw. Available data showed a hundred-fold reduction in the frozen and thawed modulus of materials that would meet the P-154 and P-208 specifications. Field test data showed a reduction in the bearing capacity of base and subbase materials that ranged from 13 to 60%.

For stabilized materials, we found that the results of the ASTM D560 durability test infer the frost-resistance (thaw-weakening) properties of the material. Data were found that clearly showed the reduction of strength as a function of freeze–thaw cycles for lime-treated soils. For cement-treated soils, the tensile strength of the material decreased with increasing freeze–thaw cycles. We also found that when 15% cement was added to the soil, there was no reduction in the tensile strength after 12 freeze–thaw cycles. The resilient modulus of cement-treated soils remained at the level found before freezing, when a cement content greater than 5% was added. Although the modulus was the same, the strength of the material decreased. It was reported that the permeability of soils treated with up to 6% lime increased. No information was found on the thaw-weakening characteristics of cement-treated bases and econocrete subbases.

In summary, the following recommendations are made. First, that the COE frost-susceptibility criterion should be included in all appropriate FAA specifications. Second, for reducing frost heave during the thaw-weakening period, the amount passing the no. 200 sieve should be limited to 2%. Third, an effort should be made to determine engineering properties for both unstabilized and stabilized base and subbase materials subjected to freeze–thaw cycling. This was lacking in the literature. These data will be critical to the development of the FAA mechanistic design procedure.

Evaluation of Airport Subsurface Materials

VINCENT C. JANOO, ROBERT EATON, AND LYNETTE BARNA

INTRODUCTION

In areas of the world having seasonal frost, airport pavements freeze and thaw annually. The effects that frost action have on a pavement structure are uniform or differential frost heave during the winter and subsidence in the spring and during intermittent thaws in the winter because of thaw-weakening. Pavements constructed in frost areas must accommodate the design aircraft load during thaw-weakening periods and minimize roughness caused by differential frost heave.

For frost heave to occur, three conditions must be present: 1) below-freezing temperatures, 2) frost-susceptible soil, and 3) water close to the freezing front. As the frost penetrates into the pavement structure, moisture is drawn to the freezing front by capillary action. Ice lenses are formed and the material is displaced, usually in the vertical direction. The displacement is translated to the surface and is called frost heave. Thaw-weakening takes place when the segregated ice lenses melt. Depending on permeability, the excess moisture from the melting ice can become trapped between the surface course and the frozen layer below. When the surface is loaded, the water is unable to dissipate, an undrained loading condition exists, and positive pore pressures develop in the saturated layer. This leads to a reduction of the effective stress that the layer can tolerate, thus reducing the bearing capacity of the pavement structure. So, an important soil property that affects thaw-weakening is the soil's hydraulic conductivity.

It is often assumed that frost heave and thaw-weakening occur primarily in the fine-grained subgrade soils. Base courses are usually thought of as layers that are resistant to frost heave and thaw-weakening. However, this is not necessarily the case. The presence of fines in a base or subbase layer can change the frost-heave-susceptibility of the material. In general, as the fine content increases, the frost-heave-susceptibility increases. Granular unbound bases containing frost-susceptible materials may also weaken significantly during thaw-weakening periods, owing to increased saturation and a decrease in moisture tension, combined with reduced density that comes from ice expansion when the base course was frozen. Granular material with excess fines can also weaken during the thaw-weakening period because its permeability is reduced. The presence of fines can also change the hydraulic conductivity of the material, i.e., as the fine content increases, the hydraulic conductivity of the layer decreases.

The objective of this study is to evaluate the current Federal Aviation Administration (FAA) specifications for subsurface materials used in airport pavement construction in cold regions. We assume that below-freezing temperatures are present in the winter and that the water table is close to the surface. The evaluation will focus on estimating the frost-heave and thaw-weakening susceptibility of the subsurface materials on the basis of the properties of the subsurface materials. The results and conclusions presented in this report are based on the limited materials data available in the FAA (1989) Advisory Circular AC150/5370-10A. The material specifications evaluated are presented in Table 1.

Table 1. FAA material specifications evaluated in this study.

FAA designation	*Material and layer type*
P-154	Subbase course
P-155	Lime-treated subgrade
P-208	Aggregate base course
P-209	Crushed aggregate base course
P-210	Caliche base course
P-211	Lime rock base course
P-212	Shell base course
P-213	Sand–clay base course
P-301	Soil–cement base course
P-304	Cement-treated base course
P-306	Econocrete subbase course

Table 2. Division of FAA materials into stabilized and unstabilized categories.

Stabilized	*Unstabilized*
Lime-treated subgrade	Subbase course
Soil–cement base course	Aggregate base course
Cement-treated base course	Crushed aggregate base course
Econocrete subbase course	Sand–clay base course
	Caliche base course
	Lime rock base course
	Shell base course

EVALUATION

FAA material specifications

The base course materials in Table 1 were divided into stabilized and unstabilized materials as shown in Table 2. A brief description of each of the materials is presented.

P-154 subbase course

The P-154 subbase course material is a blend of coarse aggregates and either fine sand, clay, stone dust, or other similar binding or filler materials. The bounds for the subbase course material are shown in Figure 1a. The material has a liquid limit of less than or equal to 25% and a Plasticity Index (PI) of no more than 6%. An additional specification was identified for use of this material in cold regions, i.e., the amount of material finer than 0.02 mm must be less than 3%.

P-155 lime-treated subgrade

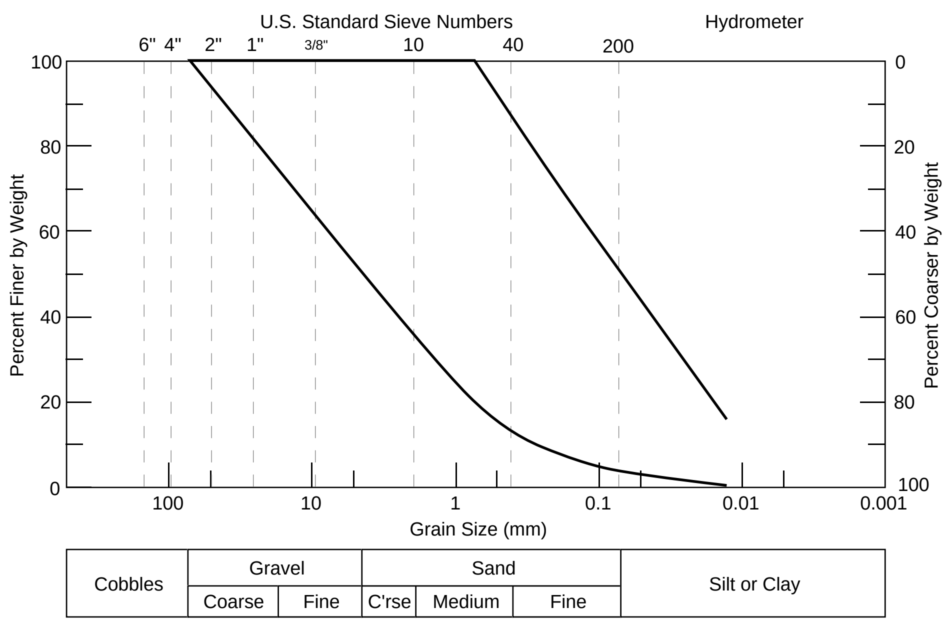
P-155 lime-treated subgrade is a mixture of soil, lime, and water. Hydrated lime is specified.

P-208 aggregate base

P-208 aggregate base course is a mixture of clean stones or gravel blended with sand, stone dust, or other binding or filler materials. The coarse portion of the material can be either crushed or uncrushed. Crushed coarse aggregate can be either crushed gravel, crushed stone, or crushed slag. Any fines obtained from the crushing of the coarse aggregate are used in the fine portion. Uncrushed fine material can be added. At least 30 to 65% (depending on the maximum particle size) of the material retained on the no. 4 sieve shall have at least one fractured face. Three gradation limits for this material are shown in Figure 1b. As with the subbase course material, the material has a liquid limit of less than or equal to 25% and a PI of no more than 6%.

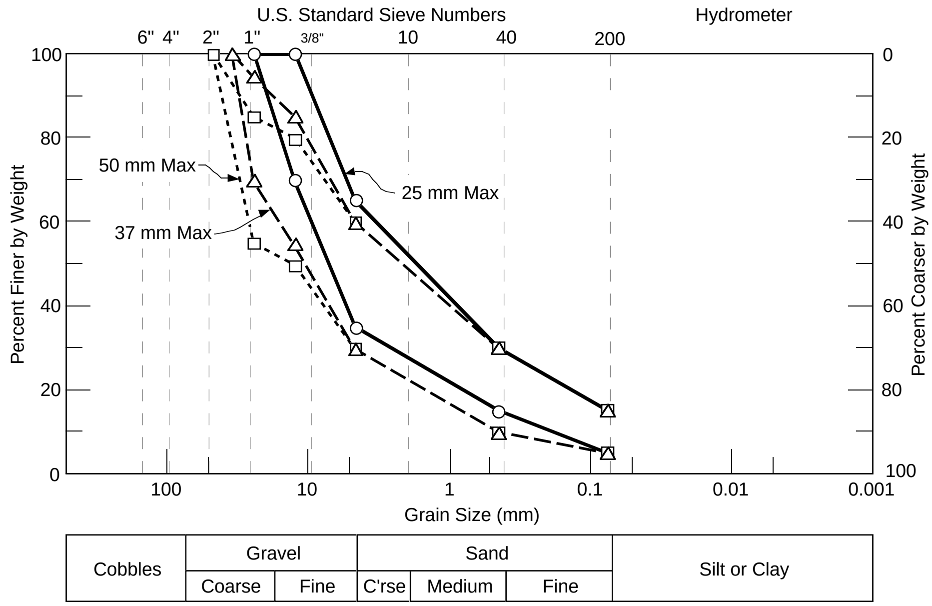
P-209 crushed aggregate base

The gradation of the P-209 crushed aggregate base course is shown in Figure 1c. The gradation for the P-208 aggregate base course is also shown in the figure for comparison. Although they are both could be considered crushed aggregate bases, the P-209 is different from the P-208 aggregate base material in the following manner. First, although the coarse portions of the gradations are alike, the fine portions are different. The maxi-

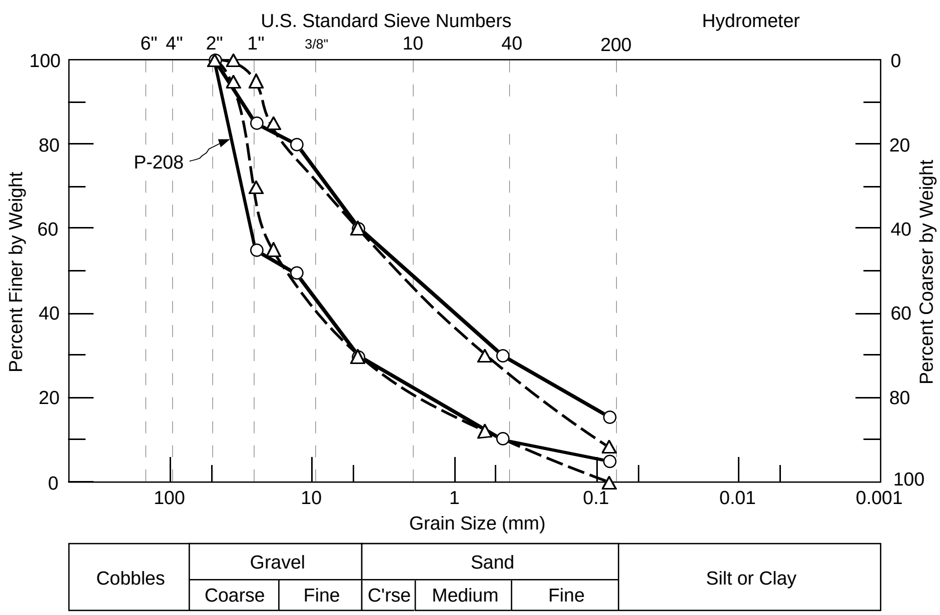


a. P-154 subbase course.

Figure 1. Grain size distribution limits.



b. P-208 aggregate base course.



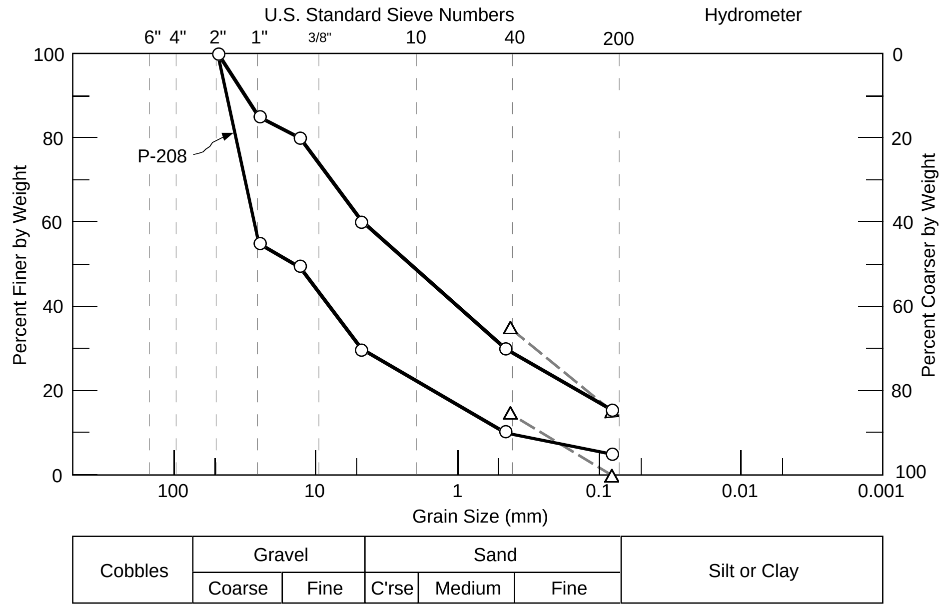
c. P-209 crushed aggregate base course.

Figure 1 (cont'd).

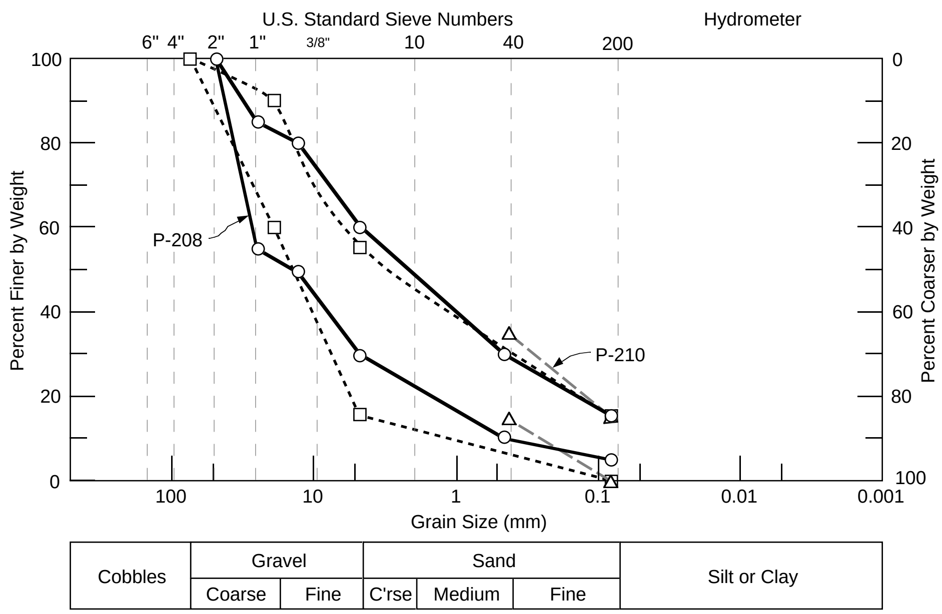
mum amount passing the no. 200 sieve for this material is almost half that of the P-208. Second, both the coarse and fine portions are crushed. Third, at least 90% of the particles by weight have two crushed faces and 100% of the particles have one crushed face. Fourth, for cold regions, the amount of material finer than 0.02 mm must be less than 3%. The aggregates can be either clean, crushed gravel, crushed stone, or crushed slag. This material has a lower PI (≤ 4%), with the same liquid limit of less than or equal to 25% as the P-208 material.

P-210 caliche base course

P-210 caliche base course is composed of caliche, caliche–gravel, or caliche–limestone. The material has a liquid limit of less than or equal to 35% and a PI of no more than 10%. The specified gradation limits for this material are shown in Figure 1d. The P-208 gradation limits are also



d. P-210 caliche base course.



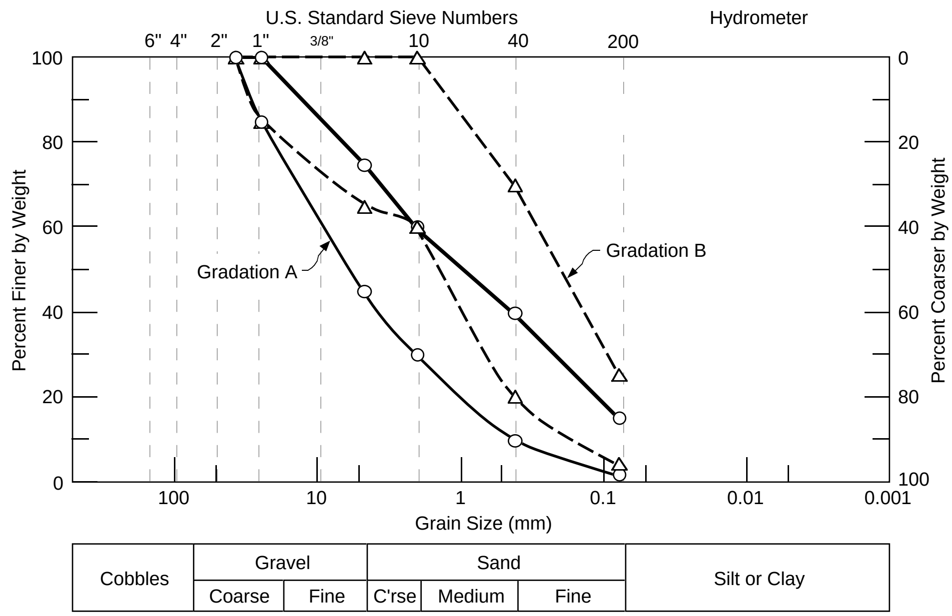
e. P-212 shell base course.

Figure 1 (cont'd). Grain size distribution limits.

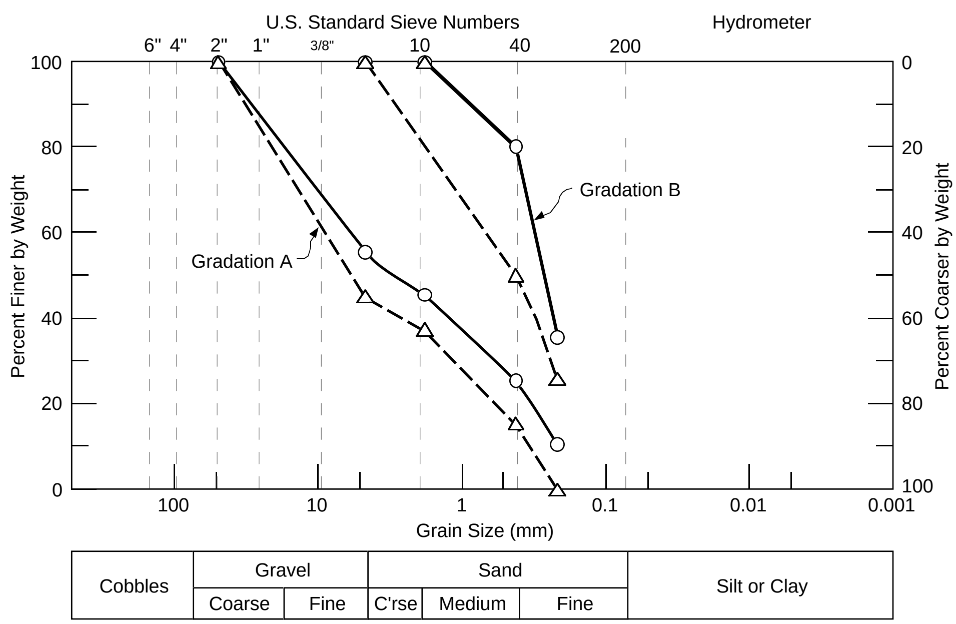
plotted in Figure 1d for comparison. Caliche ($CaCO_3$ or $MgCO_3$), also known as calcrete, is more common in the hot, semiarid regions of the U.S., such as Texas and Arizona (Hunt 1983). Caliche is considered a transitional limestone. This material may be hard or soft, depending on how it was deposited. Slight cementation of the base course may take place because of the carbonates (calcium or magnesium) present. Surface water entering into the base course can react with the caliche to form a weak acid, which then attacks the material. Also, since caliche may have minute pores, the amount of fines in the base course may not be significant enough to cause water to be drawn to the freezing front.

P-211 lime rock base course

P-211 lime rock base course is made with fossilliferous limestone (containing abundant seashells). The material is expected to show no



f. P-213 sand clay base course.

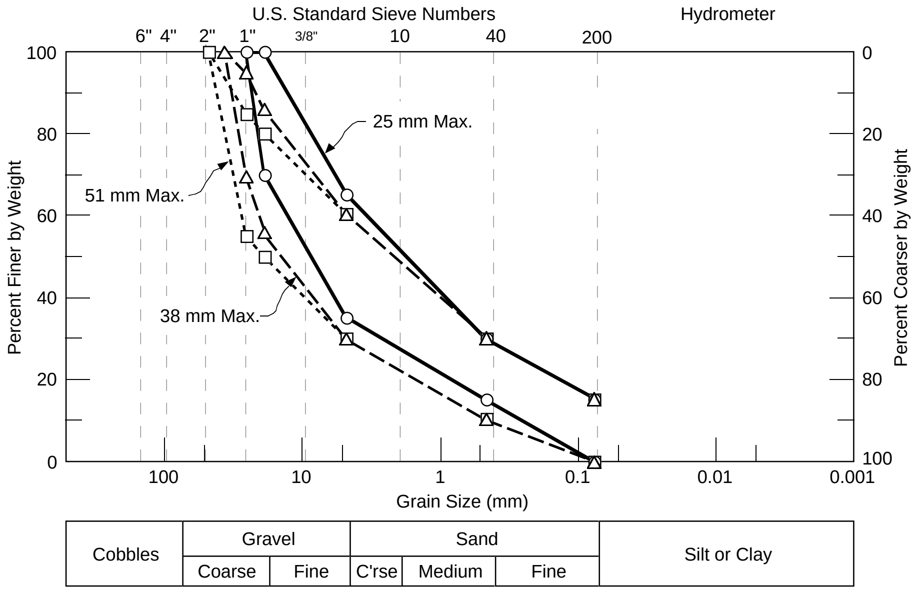


g. P-304 cement-treated base course.

Figure 1 (cont'd).

tendency to "air slack" or undergo chemical changes when exposed to the environment. If oolitic (softer variety of limestone) lime rock is used, then the minimum amount of either calcium or magnesium carbonate present is 70%. The amount of iron and aluminum oxide has to be less than 2%. The minimum combined amount of carbonates, oxides, and silica is 97%. This material also has to be nonplastic. If other types of lime rock are used, the minimum amount of calcium and magnesium carbonates is 95%. The liquid limit is less than or equal to 35% and the PI ≤ 6%. All fine materials used are obtained from the crushing operation. The allowable gradation is that 100% passes the 90-mm (3.5-in.) sieve and 50 to 100% passes the 19-mm (0.75-in.) sieve.

Limestone is a sedimentary rock made from predominantly calcite ($CaCO_3$). So, surface water entering into the base course can react with the limestone to form a weak acid that then attacks it.



h. P-306 Econocrete subbase course.

Figure 1 (cont'd). Grain size distribution limits.

Also, as with caliche, limestone rock may have minute pores and the amount of fines in the base course may not be significant enough to cause water to be drawn to the freezing front.

P-212 shell base course

P-212 shell base course is made from reasonably clean oyster or clam shells. Chemically, these shells form from calcium carbonate ($CaCO_3$) precipitate. Shell base courses were used under several military airfields in the Gulf coast area in the mid- to late-1950s. The strength of this material is probably lower than that of caliche or lime rock. The specified gradation limits are shown in Figure 1e.

P-213 sand–clay base course

P-213 sand–clay base course is a mixture of clay and mineral aggregate. The mix can be a sand–clay, sand–clay–gravel, disintegrated granite or topsoil; however, the coarse portion of this mix has to be gravel or stone. Two gradations are specified (Fig. 1f) based on the PI. Gradation A has a PI ≤ 4% and gradation B a PI ≤ 6%. The liquid limit for both gradations is less than or equal to 25%. The PI limits in this specification appear to be low for the allowable amount of fines (clay) passing the no. 200 sieve.

P-301 soil–cement base course

P-301 soil–cement base course is made from a mixture of soil, portland cement, and water. The cement specified can be ASTM (1992a) C150 Types I, II, III, IV, or V. The soil must have no more than 45% retained on the no. 4 sieve. Any gravel in the soil must be smaller than 1 in. (2.5 cm). Test specimens should be subjected to the ASTM (1992c, 1996) D559 and D560 durability tests for wet–dry and freeze–thaw strength losses. At the end of 12 wet–dry or freeze–thaw cycles, the weight loss has to be no more than 14% for granular materials, 10% for granular soils with some plasticity, and 7% for clay soils. The compressive strength of the stabilized material has to increase with age and cement content.

P-304 cement-treated base course

P-304 cement-treated base course is a mixture of cement (portland cement or bitumen), water, and mineral aggregates. The aggregates can be crushed or uncrushed and meet the gradation limits in Figure 1g. The portland cement specified can be ASTM (1992 a, b) C150 Types I, II, III, IV, or V or ASTM C595 Types IS, IS-A, IP, IP-A, P, or PA. Aggregates containing any amount of sulfates that would cause expansion of the cement-treated base course by reacting with the alkalis in the cement should not be used, or sulfate-resistant cements should be specified. If bitumen is used, either cutback (RC-70 and RC-250) or emulsified (RS-1, SS-1 or CRS-1) asphalts are specified. The liquid limit and PI of the blended material passing the no. 40 sieve has to be less than or equal to 25% and less than 6% respectively. The 7-day compressive strength must be at least 170 kPa (3550 lb / ft^2). In freeze–thaw areas, the weight loss has to be less than 14% (ASTM D560).

P-306 econocrete subbase course

P-306 econocrete subbase course is a mixture of subbase quality aggregate, cement, and water. The aggregate can be crushed or not. The limits of the subbase material gradation is shown in Figure 1h. The cement specified is ASTM C150 Type I. Admixtures that are pozzolanic (fly ash), air-entraining (not less than 4% and not more than 9%), or water reducing can be used. The compressive strength after 7 and 28 days must at least be 3445 and 5167 kPa (500 and 750 lb / in.^{2}), respectively. An upper limit of 8268 kPa (1200 lb / in.^{2}) is recommended since compressive strengths greater than that may induce cracking in the overlying pavement. A minimum amount of cement of 119 kg / m^3 (7.43 lb / ft^3) is also specified. In freeze–thaw areas, the maximum weight loss should not exceed 14% and the air content should be between 6 and 10% (± 2%).

The above materials were first evaluated for frost-heave susceptibility, then for thaw-weakening. Since *laboratory tests were not conducted* to determine properties, we assumed some of the material properties from information in the literature.

Frost heave

Chamberlain (1981), after a thorough review of existing frost-susceptibility index tests used in the U.S. and abroad, decided that the criterion developed by Casagrande in the early 1930s was the best indicator of frost-susceptibility. Based on field tests, Casagrande (1931) found that the amount of material in the soil finer than 0.02 mm (0.78×10^{-3} in.) was a fairly good indicator of frost-susceptibility in terms of rate of heaving. Based on this, the Corps of Engineers (COE) has developed a frost-susceptibility classification as shown in Table 3. In most cases for granular materials, if the amount of material finer than 0.02 mm is kept at less than 3%, the material could be clas-

Table 3. Corps of Engineers (COE) frost-susceptibility classification.

Frost group	*Soil*	*Percentage finer than 0.02 mm by weight*	*Typical soil types under Unified Soil Classification System*
NFS*	(a) Gravel Crushed stone Crushed rock	0–1.5	GW, GP
	(b) Sands	0–3	SW, SP
PFS†	(a) Gravel Crushed stone Crushed rock	1.5–3	GW, GP
	(b) Sands	3–10	SW, SP
S1	Gravely soils	3–6	GW, GP, GW-GM, GP-GM
S2	Sandy soils	3–6	SW, SP, SW-SM, SP-SM
F1	Gravely soils	6–10	GM, GW-GM, GP-GM
F2	(a) Gravely soils	10–20	GM, GW-GM, GP-GM
	(b) Sands	6–15	SM, SW-SM, SP-SM
F3	(a) Gravely soils	over 20	GM, GC
	(b) Sands, except very fine silty sands	over 15	SM, SC
	(c) Clays, PI > 12	—	CL, CH
F4	(a) Silts	—	ML, MH
	(b) Very fine silty sands	Over 15	SM
	(c) Clays, PI < 12	—	CL, CL–ML
	(d) Varved clays and other fine-grained, banded sediments	—	CL, ML and SM, CL, CH and ML, CL, CH, ML and SM

* Non-frost-susceptible.

† Possibly frost-susceptible, requires lab test to determine frost design soil classification.

Table 4. Frost-susceptibility in current FAA specifications.

FAA designation	*Material and layer type*	*Frost-susceptibility specifications*
P-154	Subbase course	Yes– < 3% finer than 0.02 mm
P-155	Lime-treated subgrade	Not available
P-208	Aggregate base course	No
P-209	Crushed aggregate base course	Yes– < 33% finer than 0.02 mm
P-210	Caliche base course	No
P-211	Lime rock base course	No
P-212	Shell base course	No
P-213	Sand–clay base course	No
P-301	Soil–cement base course	Not available
P-304	Cement-treated base course	Not available
P-306	Econocrete subbase course	Not available

sified as non-frost-susceptible. There were only two materials in the reviewed FAA specifications where a limit was established for minimizing frost heave (Table 4). We also found that there were no data for the remaining base course materials on the amount finer than 0.02 mm in the FAA-specifications. The gradation stopped at the no. 200 sieve. Therefore, an alternative was needed to classify the frost-susceptibility of the FAA specified materials.

In his review, Chamberlain reported that other agencies have used the amount passing no. 200 sieve for estimating the frost-susceptibility of the soil. Chamberlain cited a study by Townsend and Csathy (1963), in which they found that the grain size criterion (passing no. 200 sieve) was conservative, i.e., although the criterion was good for finding frost-susceptible soils, it sometimes classified non-frost-susceptible material as frost-susceptible. Reviewing some of the laboratory results of frost heave tests, Kaplar (1974) found that for gravels the amount finer than 0.02 mm was approximately half the amount passing the no. 200 sieve as long as the amount passing the no. 200 sieve was less than 18% (Fig. 2). This relationship was used to estimate the amount finer than 0.02 mm in the FAA specifications.

From laboratory frost heave tests on noncohesive soils (4-in.-diameter × 6-in.-long samples [10 × 15 cm]), Croney and Jacobs (1967) found a relationship between the percentage passing the no. 200 sieve and frost heave (Fig. 3). The criterion used by Croney and Jacobs for differentiating levels of frost-susceptibility is shown in Table 5. The table shows the amount of frost heave at the end of 10 days. The results (Fig. 3) clearly indicate that as the amount passing the no. 200 sieve increases, so does the amount of frost heave. Croney and Jacobs also found a relationship between the amount of frost heave and the PI of cohesive soils

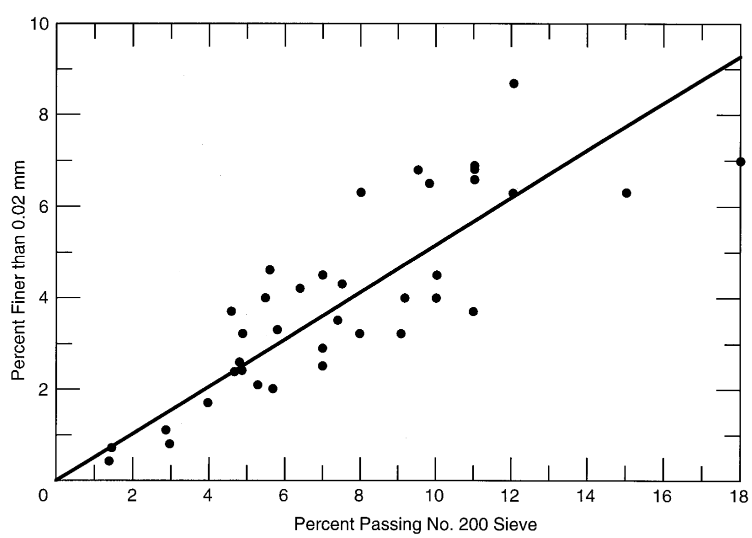


Figure 2. Relationship between percentage passing the no. 200 sieve and the percentage finer than 0.02 mm for coarse-grained gravel soils.

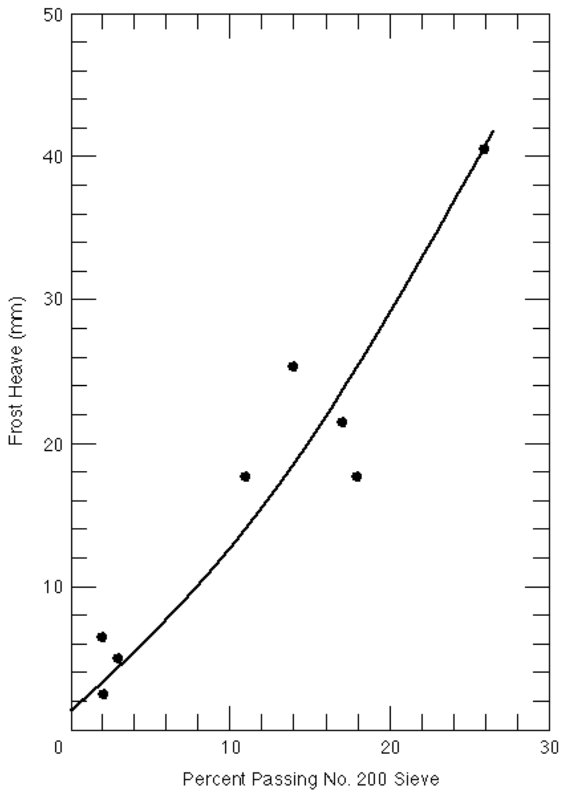


Figure 3. Relationship between frost heave and percentage passing the no. 200 sieve—noncohesive soils other than limestone gravels (1 in. = 25.4 mm).

Table 5. Criterion for determining frost-susceptibility of soils.

*Frost heave of soil (in.)**	*Frost-susceptibility*
≤ 0.5	Non-frost-susceptible
0.5 to 0.7	Marginal
≥ 0.7	Very

* 1 in. = 25.4 mm.

Table 6. Frost-susceptibility as a function of the Plasticity Index (PI).

FAA designation	*Material and layer type*	*PI*	*Relative frost-susceptibility*
P-154	Subbase course	6	medium
P-208	Aggregate base course	6	medium
P-209	Crushed aggregate base course	4	low
P-211	Lime rock base course	6	medium
P-213	Sand–clay base course		
	Gradation A	4	low
	Gradation B	6	medium

(Fig. 4). They reported that as the PI increased, the amount of frost heave decreased. Since the FAA specifications limit the PI of the subsurface materials used in airport construction, the PI could be used as an indicator of frost heave.

We reviewed of some of the laboratory test results of Kaplar (1974) for coarse-grained gravelly base course materials (GW, GP, GW-GM, GP-GM, GM, GW-GC, GP-GC, GM-GC, GC) and found an increasing trend between PI, up to 7%, and average rate of heave (Fig. 5). The rate of heave decreased at PI levels greater than 8%, which agreed with the results of Croney and Jacobs (1967). Table 6 shows the PI limits in the FAA specifications and the relative frost-susceptibility determined from Figure 5. The materials in Table 6 were classified as having either low or medium frost-susceptibility. It should be noted that the PI

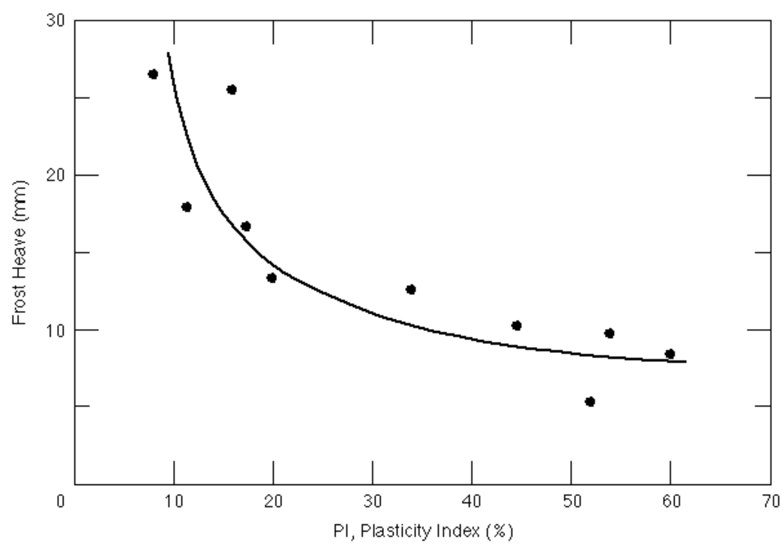


Figure 4. Relationship between frost heave and PI for cohesive soils (1 in. = 25.4 mm).

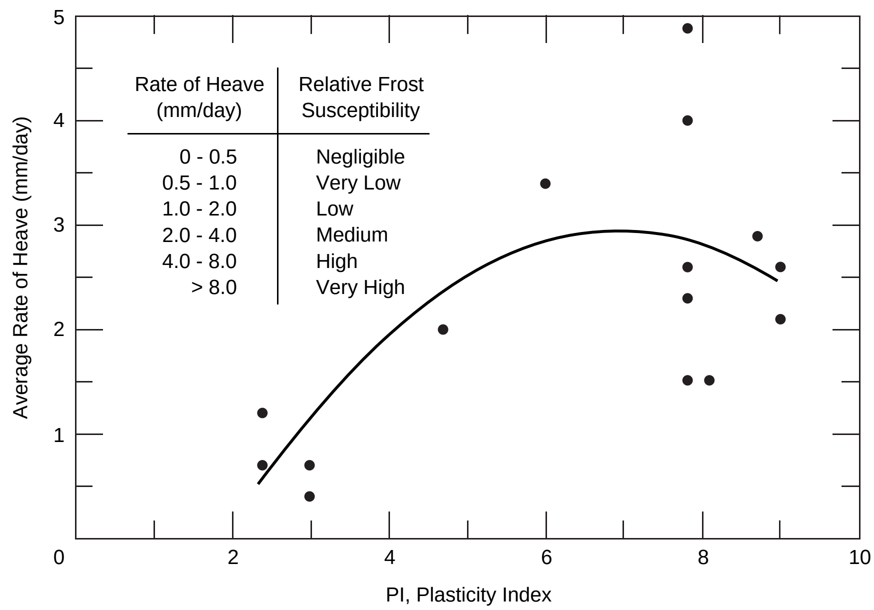


Figure 5. Relationship between PI and average rate of heave (1 in. = 25.4 mm).

does not identify the amount of the fine material but the plasticity of the fines. What Table 6 suggests is that, in addition to the amount of fines, the plasticity of the fines can either increase or decrease the frost-susceptibility of the material. Therefore, although the PI provides some idea of the frost-susceptibility of the material, the amount of fines passing the no. 200 sieve is the more critical parameter.

Several criteria for percentage passing the no. 200 sieve were evaluated and are listed in Table 7. These criteria were obtained from Chamberlain (1981) and are specifically for unstabilized base and subbase materials. A material is considered frost-susceptible if the amount passing the no. 200 sieve exceeds the value presented in the second column of Table 7. The reliability indicates how well each criterion was able to distinguish between non-frost- and frost-susceptible materials; we used data generated by Kaplar (1974). For reference, note that the reliability of the COE frost-susceptibility criterion was 0.67 and 1.00 for classifying non-frost- and frost-susceptible soils.

An analysis of Table 7 tells us that most states or provinces in cold regions limit the amount passing the no. 200 sieve to 10%. Also, the criterion of limiting the amount of fines passing the no. 200 sieve to less than 7% had a reliability of 80% for frost-susceptibility prediction. The reliability for predicting non-frost-susceptibility was in the range of 40 to 80%.

Table 7. Criteria developed by others for predicting the frost-susceptibility of soils.

Agencies	*Frost-susceptibility criteria (% passing no. 200 sieve)*	*Reliability*	
		Non-frost-susceptibility	*Frost-susceptibility*
Asphalt Institute	7	0.82	0.81
Newfoundland, Canada	6	0.60	0.82
Japan	6	0.60	0.82
Alaska	6	0.60	0.82
Colorado	5–10	0.67	0.75
Kansas	15	0.80	0.36
Maryland	12	0.80	0.45
Massachusetts	10	0.80	0.55
Minnesota	10	0.80	0.55
New Hampshire	8	0.80	0.45
Ohio	15	0.80	0.36
Vermont	10	0.80	0.73
Washington	10	0.80	0.64
Wisconsin	5	0.40	0.82

Table 8. Frost-susceptibility classification of FAA-specified base–subbase materials.

FAA designation	*Allowable % passing no. 200 sieve*	*Allowable % finer than than 0.02 mm*	*Asphalt Institute criterion*	*COE criterion*
P-154 subbase	< 15	< 3	No	Yes
P-208 aggregate base	5–15		Yes, ≤ 7% No, > 7%	Yes, ≤ 6% No, > 6%
P-209 crushed aggregate base	0–8	< 3	Yes, ≤ 7% No, > 7%	Yes
P-213 sand–clay base (A)	2–5		Yes, ≤ 7% No, > 7%	Yes, ≤ 6% No, > 6%
P-213 sand–clay base (B)	4–25		Yes, ≤ 7% No, > 7%	Yes, ≤ 6% No, > 6%

For evaluating the FAA's base course materials, two specifications were chosen. One was the Asphalt Institute criterion that defines a frost-susceptible soil as one that contains more than 7% passing the no. 200 sieve. The second was the COE criterion that the amount finer than 0.02 mm should be one-half of the amount passing the no. 200 sieve. The frost-susceptibility ratings are shown in Table 8.

Croney and Jacobs (1967) found that the amount passing the no. 200 sieve for limestone (oolitic and magnesian) materials had no effect on frost heave. However, the saturation moisture content did have a significant effect on frost heave. They found that as the saturation moisture content increased, so did the amount of heave (Fig. 6 and 7). The reason suggested is the porous nature of limestone. Suction of water to the freezing front is maintained by the fine capillaries in the rock. This finding probably also applies to the caliche and shell base course materials. With hard limestones, this was not the case and the amount passing the no. 200 sieve had an effect on frost heave (Fig. 8).

Based on the chosen criteria, Table 7 tells us that, depending on the gradation used, all the unstabilized base materials, with the exception of the P-209 crushed aggregate base and P-154 subbase, are frost-susceptible. To make the frost-susceptible material (with the exception of caliche, shell, and lime rock) non-frost-susceptible, the amount passing the no. 200 sieve

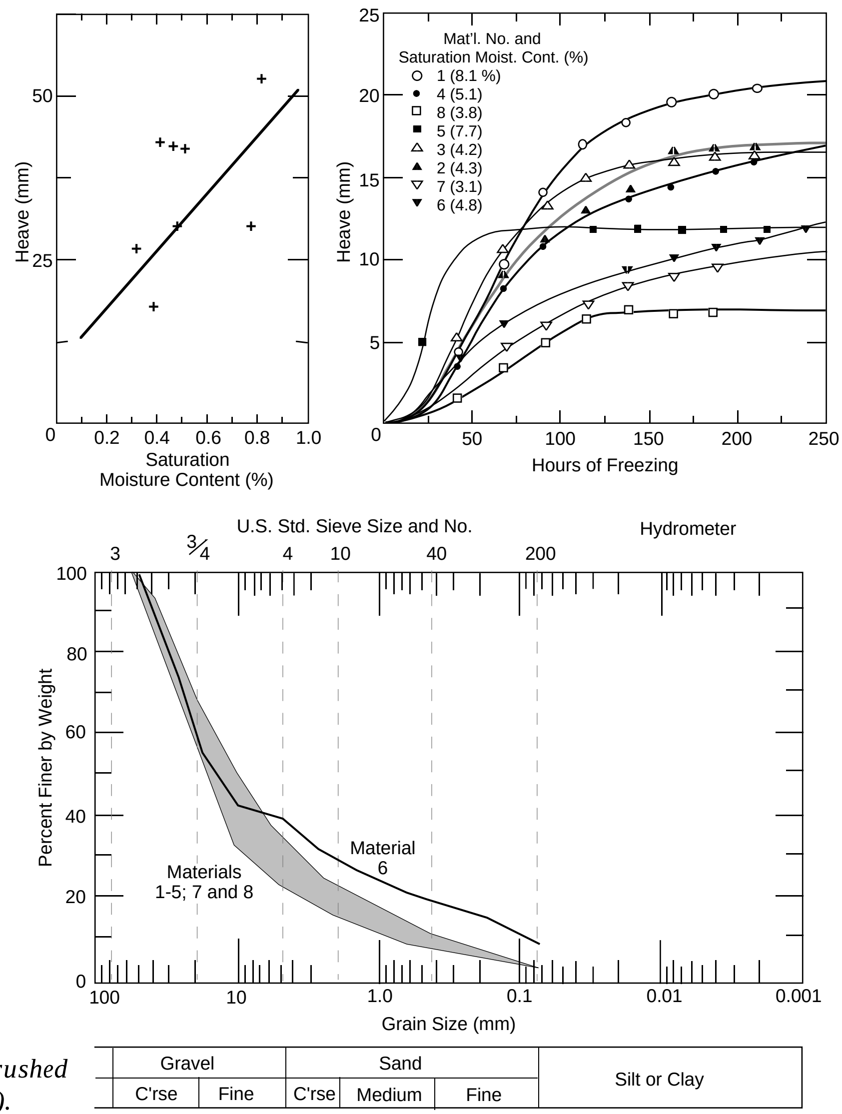


Figure 6. Frost heave of eight crushed oolitic limestones (1 in. = 25.4 mm).

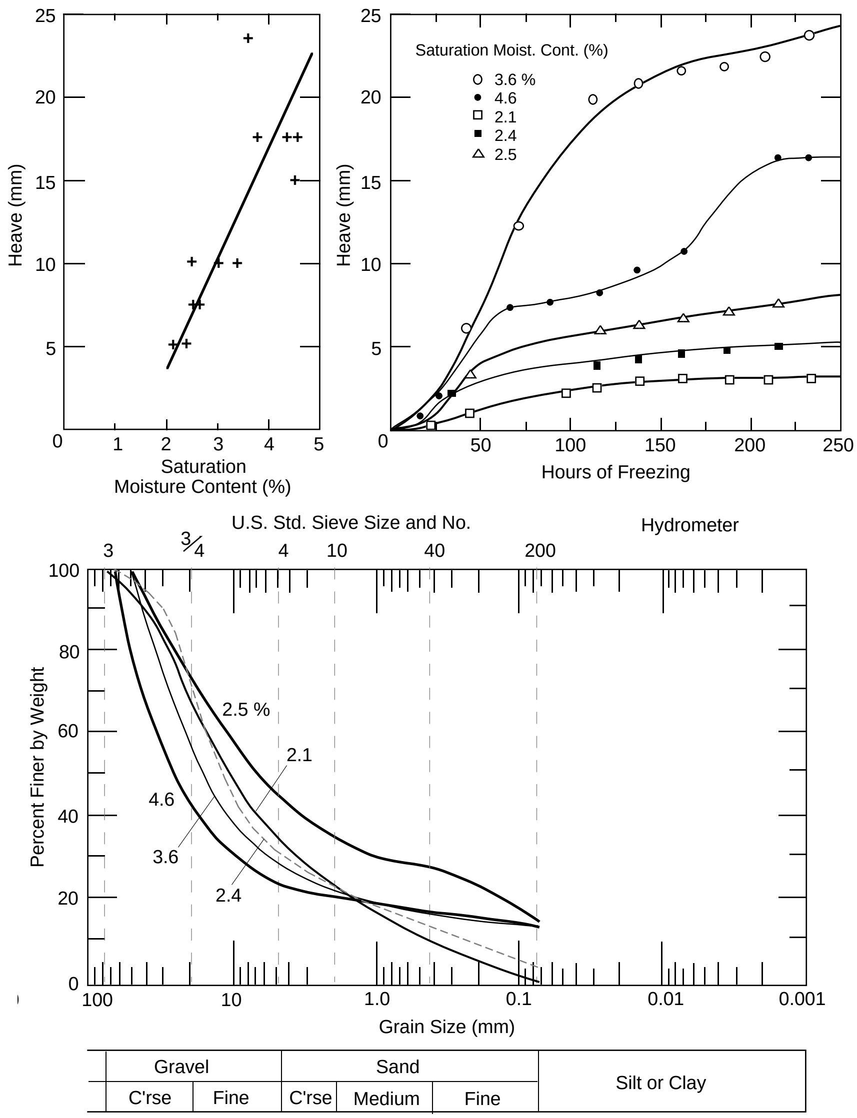


Figure 7. Frost heave of various magnesian limestones (1 in. = 25.4 mm).

should be reduced to 6% or at least to 8%, similar to that currently specified for the P-209 crushed aggregate base. A second specification that should be added is that, in cold regions, the material finer than 0.02 mm has to be less than 3%.

The FAA allows slags as aggregates in some of its base course specifications. Results from Croney and Jacobs (1967) found that slags are non-frost-susceptible even though they have excessive fines (up to 26%). Kettle and McCabe (1985) found that addition of slag (up to 50%) reduced frost heave by 60% in a frost-susceptible granular mix.

Stabilized materials

Pavement structures have been stabilized to either improve unsatisfactory subgrade soils or to decrease their thickness, or both. Types of admixtures commonly used for increasing the bearing capacity of the layer are cementing agents such as lime, lime–fly ash, portland cement, or asphalt. Another kind of stabilization is mechanical, where gravel is added to soil to improve the gradation and mechanical properties.

Stabilization admixtures used for reducing frost heave include chemicals to prevent freezing of the soil water (such as calcium chloride or sodium chloride), chemicals to aggregate the soil particles into larger units, or chemicals that waterproof the soil particles and cements (lime, cement, and asphalt). This report will concentrate on cementing agents, since the FAA specifications being evaluated use this type of admixture for stabilization.

Soils generally suited for lime stabilization are granular materials and lean clay (CL) subgrades.

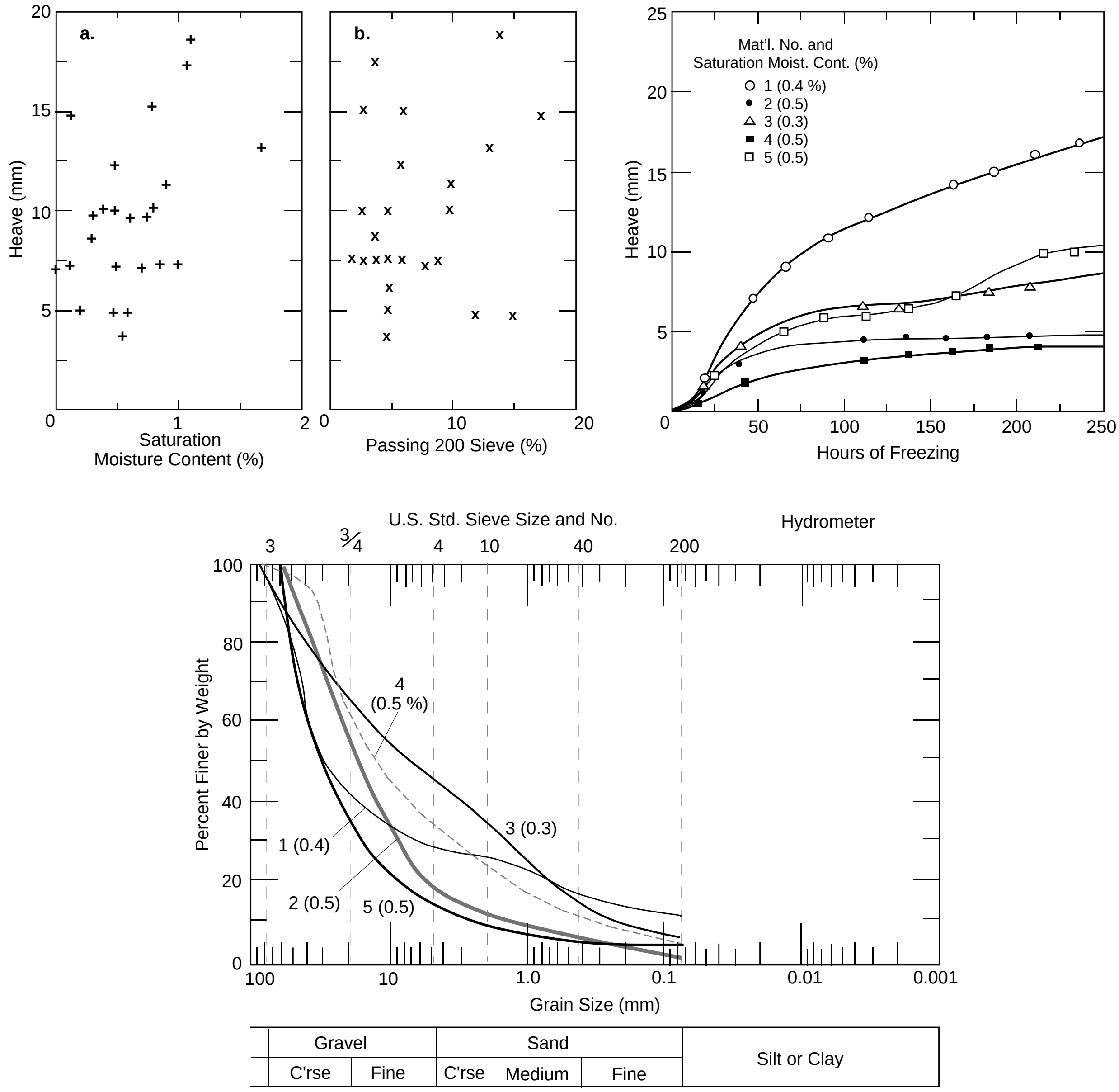


Figure 8. Frost heave of hard limestones (1 in. = 25.4 mm).

Adding lime to clay soils causes the structure of the clay to flocculate. In the lime-stabilized mixture, after a chemical reaction with water (pozzolanic action), cementation takes place, perhaps through calcium silicate or aluminate bonding. The addition of a pozzolan, such as fly ash to lime, speeds up the cementation process and also produces extra strength. However, the quantities of fly ash required are generally high, 10–20% (Yoder and Witzcak 1975). The amount of lime added is about 2 to 5% by weight.

Portland cement, when added to soils, hardens the mixture through hydration of the cement. Factors that affect the portland cement–soil mixture are soil type, quantity of cement, degree of mixing and time of curing (Yoder and Witzcak 1975). Of all the factors mentioned above, the cure time is the critical factor. For example, after 20 days, a 5% soil (CL) and cement mixture acquired an unconfined compressive strength of approximately 4100 kPa (600 lb/in.^{2}), and a 10% soil–cement mixture reached an approximate unconfined compressive strength of 5500 kPa (800 lb/in.^{2}). The amount of portland cement used for strength improvement varies around 9–15% for sandy soils, base, and subbase and around 5–9% for clean clays. Highly plastic soils (CH) may require 15–20% cement by weight to bring about hardening.

A primary difference between soil–lime and soil–cement mixtures is the rate of strength gain during curing. Soil–cement mixes gain strength rapidly, whereas soil–lime mixtures take longer. For example, a soil (CL) and lime (5–10%) mixture attains an unconfined compressive strength of about 2760 kPa (400 lb/in.^{2}) after 36 weeks, whereas soil (CL) and cement mixtures attain the same strength after 4 days.

There is very little information in the literature on the behavior of stabilized materials that are subjected to frost action. The COE conducted several studies between 1943 to 1949 on how to reduce the frost-susceptibility of base materials. However, most of the studies concentrated on reducing the freezing point of water. We did obtain some information from Lambe and Kaplar (1971) and Lambe et al. (1971). They used portland cement and lime as the stabilizing agents. They considered the two to be either void filling or cementatious and thought them uneconomical. Void filling in subgrades or base courses would alter the materials, making them similar to asphalt concrete or portland cement concrete. Since all the voids are filled, there would be no capillary action, thus making the materials non-frost-susceptible. Cementing could increase the tensile strength of the materials and thus their capability to resist frost heave.

In the Lambe and Kaplar (1971) and Lambe et al. (1971) work, lime or portland cement was added dry and blended mechanically. This mix was then allowed to cure (7 days) before the freezing test was done. The samples were also saturated prior to testing. The frost-susceptibility of the samples was evaluated from a portion of the heave versus time curve, where the slope was relatively constant. The result was reported as the average of the rate of heave (over 5 days) of all samples tested. The rate of penetration was also relatively linear at about 6.4 mm (0.25 in.) to 19 mm (0.75 in.) per day. Since the heave rate is a function of molding moisture content, compacted density, overburden pressure, and rate of freezing, the authors suggested that heave rates reported were probably within ±15%.

The gradations of the unstabilized soils used by Lambe and Kaplar in 1971 are shown in Figure 9. These were fine-grained materials, called: 1) Boston blue clay (CH), having a liquid limit and PI of 53 and 26, respectively; 2) New Hampshire silt (ML-CL), having a liquid limit and PI of 24 and 6, respectively; and 3) Fort Belvoir sandy clay (CL), having a liquid limit and PI of 41 and 19 respectively. For the sandy clay, all material above the no. 10 sieve was removed. No tests were conducted with base course type soils or gradations.

The rate of heave ratio presented in Table 9 is the ratio of the rate of heave of the stabilized soil to the rate of heave of the unstabilized soil. A ratio of less than 1 indicates improvement. A value

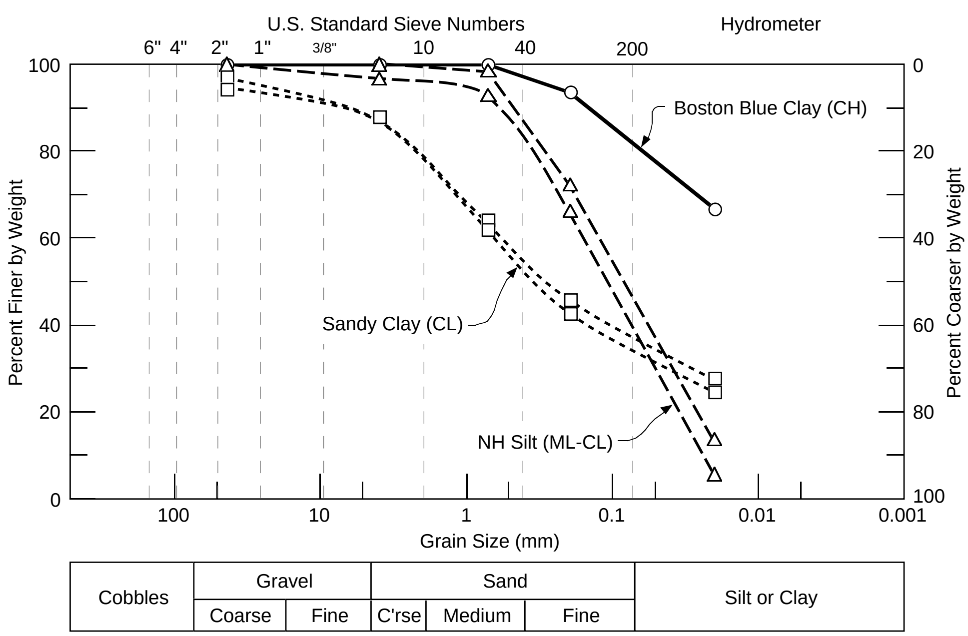


Figure 9. Gradation curves for soils used in stabilization study of Lambe and Kaplar (1971) and Lambe et al. (1971).

Table 9. Effect of portland cement on frost heave (after Lambe and Kaplar 1971).

Additive		*Heave ratio*		
Type	*Percentage*	*Boston blue clay (CH)*	*New Hampshire silt (ML-CL)*	*Sandy clay (CL)*
Portland cement	1	1.35	1.74	1.04
	2	2.15	0.63	0.81
	3	0.46	0.45	1.08
	3*		0.16	0.49
	5*		0.17	0
Portland cement and pozzolith	1 + 0.1	1.35	0.59	0.67
	3 + 0.2	0.56		0.74

* Results of tests run at a later date.

greater than 1 means that the stabilized soil was more frost-susceptible than the unstabilized soil. The freezing tests were conducted on 32-mm-diameter and 79-mm-tall samples (1.25×3 in.). The results presented in Table 9 are for portland cement soil mixtures and are for one freeze sequence, i.e., the samples were not subjected to freeze–thaw cycling. The authors suggest that if their samples were subjected to freeze–thaw cycles, the results would be different from those presented in Table 9. A CH material with up to 2% portland cement added actually experienced an increased heave rate, which translated into more heave. A similar result was found with 1% portland cement added to the ML-CL soil. With 3% portland cement, the rate of heave was reduced to 46 and 45% for the CH and ML-Cl soils. No improvement was seen with the sandy clay. In another study Lambe et al. (1971) found that adding 3% cement to the ML-CL material reduced the rate of heave to 16%, and for the sandy clay heave was reduced 49%. Although the sandy clay had the same gradation, it was different as shown in Table 10.

As seen in Table 10, the major difference between the first and second sets of tests was the density of both materials, which were higher in the second set. If the rate of frost heave is that sensitive to density (2% change), then the results for the first set of tests are suspect. The densities of the test specimens varied by 5%. Croney and Jacobs (1967) found that by increasing the molding dry density by 10%, they were able to reduce frost heave in silty clays by 30%. Upon increasing the dry density by 15%, frost heave was reduced by approximately 60%. It is unlikely that a 2% change in density would produce a 23% change in frost heave. Croney and Jacobs (1967) also reported that, for noncohesive soils, density had no effect on frost heave.

However, Lambe et al. (1971) suggest that, based on their second set of tests, reducing frost heave to any significant level will require about 3 and 5% portland cement in silts (ML-CL) and lean clays (CL). However, as shown above, this result is not conclusive.

Croney and Jacobs (1997) found that the addition of cement to cohesionless soils reduced frost heave to the non-frost-susceptible category (Fig. 10). The amount varied from 2.5 to 8%. With frost-susceptible gravels, they reported that 2% cement was sufficient to significantly reduce frost heave.

Lambe et al. (1971) reported frost heave results on soils stabilized with lime. The same soils were used in the portland cement tests discussed above, with the exception of the Boston blue clay (CH).

Table 10. Properties of New Hampshire silt (ML-CL) and Fort Belvoir sandy clay (CL).

Soil	*Percent cement*	*γ_d (kN/m^3)*	*Void ratio*	*Percent sat.*	*Average water content: Before freezing*	*Average water content: After freezing*	*% heave*	*Avg. rate** of heave (mm/day)*	*Heave ratio*
ML-CL*	3	15.7	0.74	91.5	24.4	35.3	21.2	1.08	0.46
ML-CL†	3	16.0	0.68	90.9	22.5	28.3	16.3	1.39	0.16
CL*	3	16.8	0.58	88.5	18.9	31.6	27.4	1.60	1.08
CL†	3	17.5	0.52	98.8	18.9	21.0	13.3	1.21	0.49

* Lambe and Kaplar (1971).
† Lambe et al. (1971).
** 1 in. = 25.4 mm.

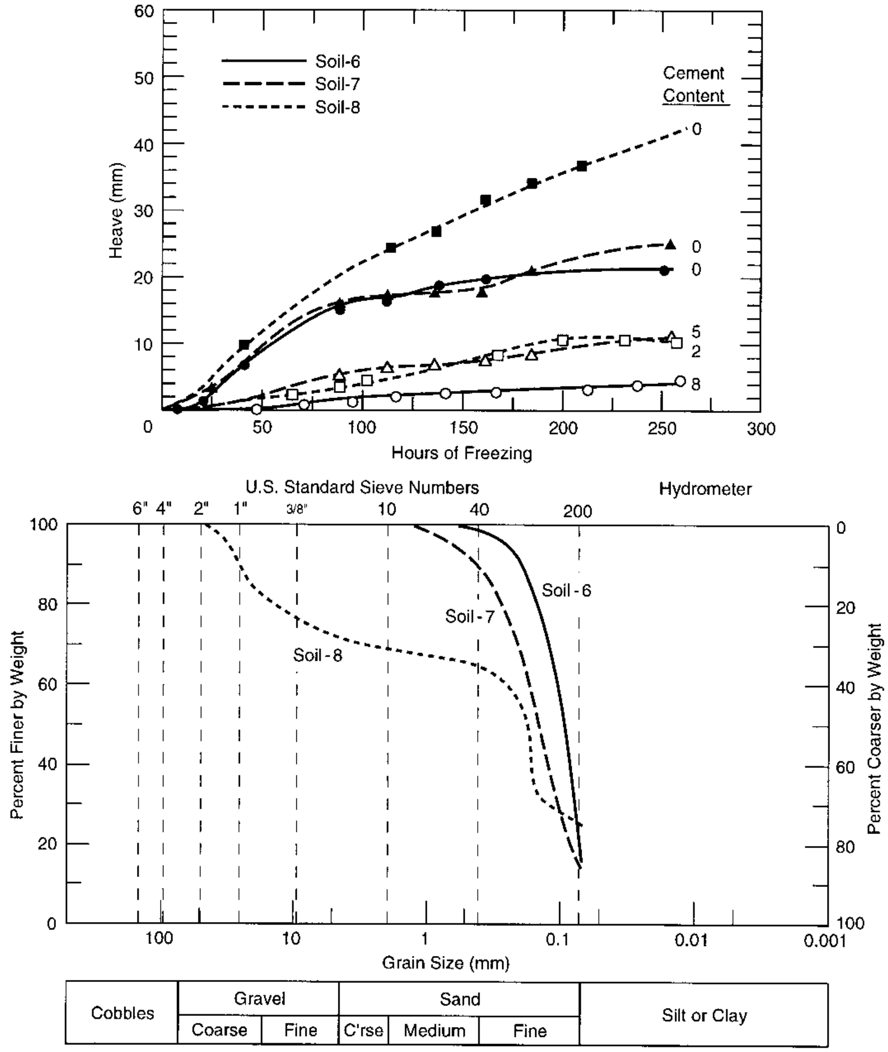


Figure 10. Effect of cement stabilization on frost heave of three noncohesive soils (1 in. = 25.4 mm).

They added lime, quicklime, gypsum, and lime–fly ash. The lime was hydrated lime (calcite) and the quicklime was chemically pure calcium oxide (CaO). The lime with the fly ash was hydrated lime. The lime and quicklime were added to the soil dry (percent by weight), and all samples were cured for 7 days prior to testing. The same sample size and calculation of the frost heave ratio used for portland cement were used for the lime. The results are presented in Table 11. They found that 3% lime reduced frost heave in the silt significantly, as did the addition of fly ash to the lime–soil (1:4) mixture. Increasing the fly ash above the 1:4 ratio increased frost heave in the sandy clay and produced no additional improvement in the silty soil.

It must be remembered that the results presented in Table 11 are for one freeze sequence. Lambe et al. advise that the response may be different when the same lime-stabilized soil is subjected to freeze–thaw cycling. Yoder and Witczak (1974) also state that freeze–thaw cycles may actually be destructive to lime-treated soils.

Lambe et al. (1971) reported results on frost heave in soils stabilized with asphalt emulsions. They found that, to reduce frost heave to any meaningful level, they had to use a minimum of 3% of asphalt and that it had to cure.

Thaw-weakening

Base and subgrade materials are thaw-weakened when the water from surface infiltration (from melting snow) or melting ice lenses is unable to drain. This undrained condition will accelerate pavement damage when it is loaded. To provide drainage, a base or subbase should contain few or no fines.

The allowable fines content (percentage pass-

Table 11. Effect of lime on frost heave (after Lambe et al. 1971).

Additive Type	Additive Percentage	Heave ratio: New Hampshire silt (ML-CL)	Heave ratio: Sandy clay (CL)	Heave ratio: Silty gravel sand (SW-SM)
Lime	1	1.06	0.70	
	3	0.27	0.16	0.56
Quicklime	1	0.93	1.74	
	3	0.43	0.13	
1:1 Lime–fly ash	25	0.18	0.21	
1:4 Lime–fly ash	25	0.09	0.14	
1:9 Lime–fly ash	25	0.08	0.66	

ing the no. 200 sieve) in the current FAA specifications for base courses is presented in Table 12. Examining COE field tests in the 1940s on base courses in the northern tier of states, Johnson (1974) reported that bases with 10% fines passing the no. 200 sieve showed serious thaw-weakening and significantly reduced (up to 70%) bearing capacity (in terms of CBR). This reduction in bearing capacity has to be related to the drainability of the base course, which in turn is affected by the amount of fines it contains. Therefore, the critical property of the base–subbase material during thaw-weakening is its permeability (vertical and horizontal). The horizontal permeability does not have to be the same as the vertical permeability. If a criterion of 10% fines is set, then all of the FAA unstabilized base courses, with the exception of P-209 crushed aggregate base, are prone to thaw-weakening.

One way to assess the thaw-weakening potential of the base course is to estimate how long it will take the base course to drain to at least a saturation level of 80%. Haynes and Yoder (1963) found that granular materials subjected to repeated loading became unstable when the degree of saturation was greater than 80%. This was substantiated by Thompson (1969a), who used results from the Illinois pavement test track. Thompson also reported that the failure (rutting) of the test section could not totally be attributed to the subgrade. From measurements he found that, with the exception of the top 25 mm (1 in.), the subgrade moisture content did not change when the test sections were soaked. However, the base course was in the range of 86 to 90% saturation. He concluded that the failure of the test sections was attributable to the reduction in the bearing capacity of the base course.

Table 12. Maximum allowable fines in FAA base–subbase specifications.

FAA designation	Max. allowable fines (%)
P-154 Subbase	15
P-208 Aggregate base	15
P-209 Crushed aggregate base	8
P-210 Caliche base course	15
P-212 Shell base course	15
P-213 Sand–clay base (A)	15
P-213 Sand–clay base (B)	25

We analyzed base drainage using the method developed by Casagrande and Shannon (1951). This model was based on field observations of six airfields in Maine, Michigan, Wisconsin, and North and South Dakota by the COE. Most of their observations of saturation of the base courses were made during spring thaw. The model assumes symmetry along the centerline of the pavement; the equations represent drainage for one-half of the base course, ABCD (Fig. 11). The base drains in two parts. First, it is saturated and the free water surface changes from AD to AC (Fig. 11). This assumption is based on the premise that water is allowed to drain freely through the open face CD. Second, the water surface changes from AC to BC. Other assumptions are that the centerline and the bottom of the base course are impervious and that the phreatic surface is a straight line. This assumption of an impermeable subgrade is valid during spring thaw when the subgrade is still frozen.

Liu et al. (1983) reported on base course drainage characteristics and showed that there was no significant improvement in the results when a parabolic instead of a straight line free surface was used. An additional assumption made for this study is that the base is sloping. The calculations are done in two stages.

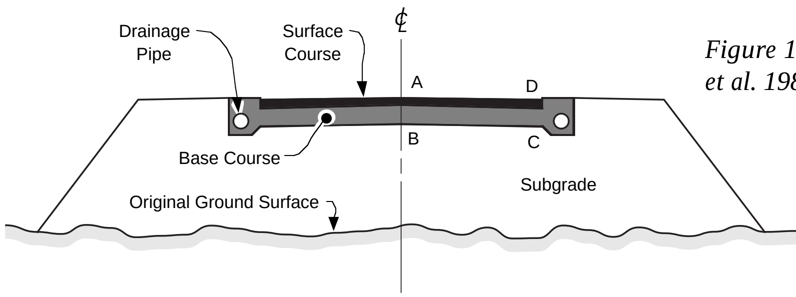


Figure 11. Cross section of pavement (after Lui et al. 1983).

Stage 1:

$0 < U \leq 50\%$

$$T = 2US - S^2 \ln \frac{S + 2U}{S}.$$

Stage 2:

$50\% < U \leq 100\%$

$$T = S + S \ln \frac{2S - 2US + 1}{(2 - 2U)(S + 1)} - S^2 \ln \frac{S + 1}{S}$$

where U = drained area / total area

S = slope factor = $\dfrac{H}{L \tan \alpha}$ (Fig. 12)

T = time factor = $\dfrac{2tkH}{cnL^2}$

$c = 2.45 - \dfrac{0.8}{\sqrt[3]{S}}$

L = one-half width of pavement

H = thickness of base course

α = angle of slope of base layer

t = time

k = permeability of base course

n = porosity of base course.

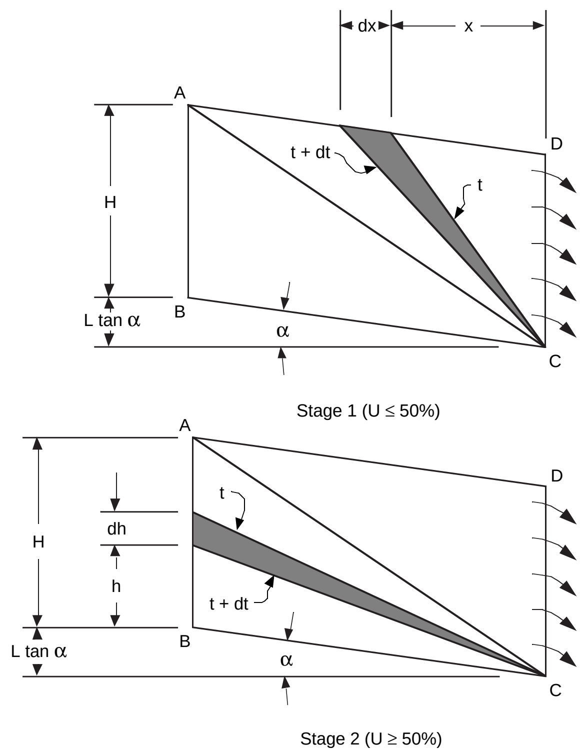
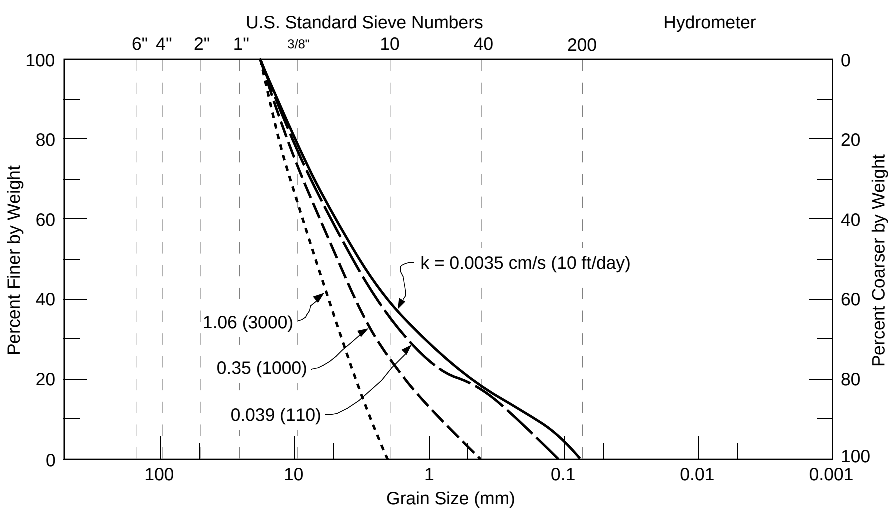


Figure 12. Casagrande and Shannon model for base course drainage.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	C'rse	Medium	Fine	

Figure 13. Typical k values for base course gradations.

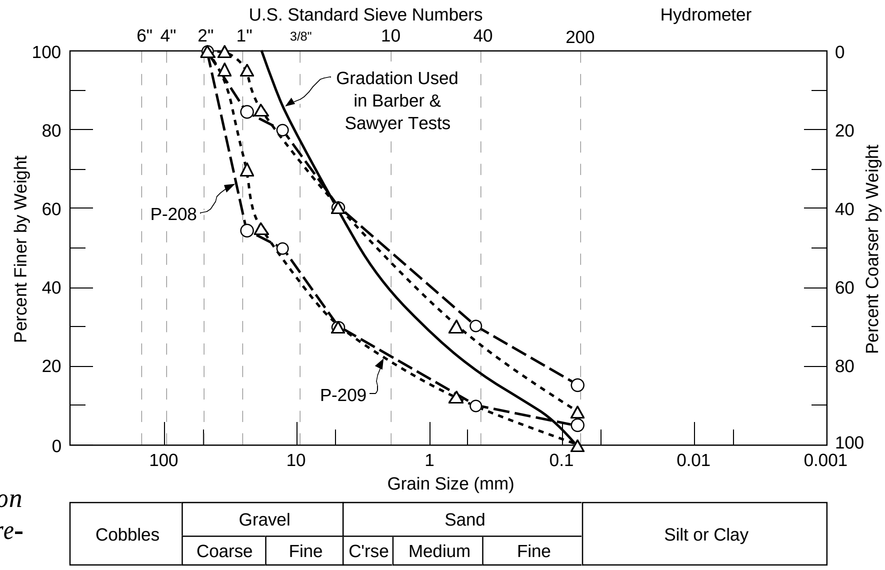


Figure 14. Grain size distribution limits for P-209 crushed aggregate base course.

How quickly a base course will drain essentially depends on its coefficient of permeability *k*, effective void ratio (or porosity), in-situ density, and soil structure. The permeability of soil is usually presented as a function of its fine contents. However, the type of fines also affect *k*. Barber and Sawyer (1952) reported *k* values of graded aggregate materials as a function of gradation, fine contents, and type of fines.

The effect of gradation on the permeability of graded aggregates is shown in Figure 13. As the amount of material passing the finer sieves is reduced, the permeability of the material increases significantly. The gradation with a $k = 3.5 \times 10^{-3}$ cm / s (1.148×10^{-4} ft / s) (0% passing no. 200 sieve), which was considered to be the gradation of typical highway base course material, is similar to the P-208 and P-209 FAA aggregate material (Fig. 14). Figure 15 shows the effect of fine types on the permeability of the soil. The *k* values given in Table 13 were estimated using Figure 15. We assumed that all the fine materials in the P-208 and P-209 base materials were silica based and, in the P-213, they were predominantly clay. Since this criterion is based on the amount of material passing the no. 200 sieve, the *k* value for the P-154 subbase material was the same as that of the P-208 base material (Table 13).

Typical base course *k* values reported by Casagrande and Shannon (1951) ranged between 27×10^{-4} and 1×10^{-6} cm / s (8.86×10^{-5} and 3.3×10^{-8} ft / s) for base course materials with fines in the range of 3 to 5%. The values in Table 13 are considered to be reasonable.

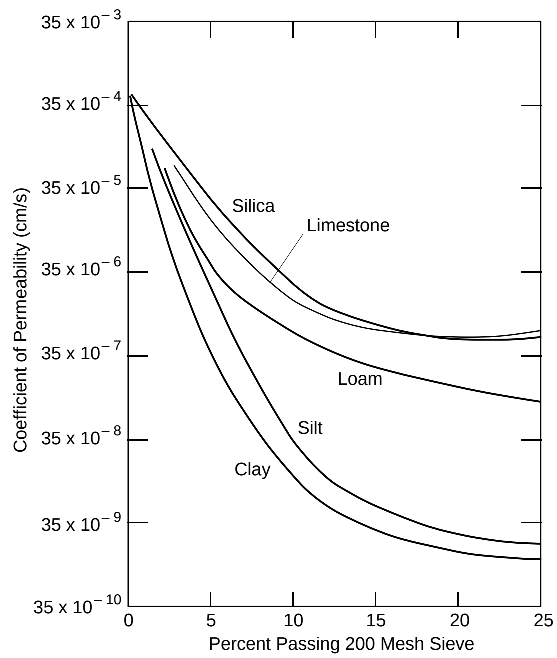


Figure 15. Effect of fines on permeability of graded aggregate (1 cm/s = 0.033 ft/s) (after Barber and Sawyer 1952).

Using *k* and the Casagrande and Shannon model for drainage, we determined the number of days for the base–subbase to drain by 20, 50, and 99% (Table 14). On the basis of limited test data, for a well-graded base course material, we used an effective porosity of 0.15 (Allen 1991) in the analysis. In addition three base or subbase thicknesses were considered—15, 61, and 152 cm (6, 24, and 60 in.). Three levels of drainage (*U*) were considered. The 20% drainage level describes the material at a sat-

Table 13. Estimated *k* values used in analysis.

Material type	Percent passing no. 200 sieve	Permeability* (cm/s)
P-154, P-208, P-209	3	2.12×10^{-3}
	5	3.00×10^{-4}
	10	2.82×10^{-5}
	15	1.41×10^{-5}
P-213	3	8.82×10^{-5}
	5	3.53×10^{-6}
	10	1.76×10^{-7}
	15	3.00×10^{-8}

* 1 cm / s = 0.033 ft / s.

uration level of 80%. As mentioned earlier, once the degree of saturation is below 85%, base courses tend to become stable again. The COE criterion for drainage layers is that 50% drainage should take place within 10 days, thus the choice of 50%. The 99% level describes the base course as being completely drained.

The effect of fine content on the drainability of a 61-cm-thick (24-in.-thick) base or subbase layer is shown in Figure 16. Clearly, the results show that as the fines content increases so does the number of days to drain to any drainage level. None of the FAA specified base or subbase materials meet the COE criteria. However, for thicker bases (> 60 cm), this criterion could be met (or nearly met) if the fines levels were below 3%. According to the results of this analysis, P-213 is fairly impermeable, so this type of base material should not be used in seasonal frost areas.

As one would expect, the drainability of the base and subbase affects the resilient modulus (M_r) of the layers. As the moisture content decreases, the resilient modulus of the material will increase. Other factors that directly or indirectly affect the resilient modulus are temperature, permeability, angularity, and percent fines in the mixture. Our limited data show that the change in M_r of bases or subbases during thaw-weakening is dramatic. A typical resilient modulus–temperature relationship for base and subbase material is shown in Figure 17 (Cole et al. 1987). As the temperature increases (as during spring thaw), the resilient modulus of the subbase and base course materials at Albany County Airport, New York, changes by a factor of 100.

The pavement structure under taxiway A at

Table 14. Base–subbase drainage.

Material type	Percent passing no. 200 sieve	Thickness* (cm)	Number of days to drain U = 20%	U = 50%	U = 99%
P-154, P-208, P-209	3	15	10	42	353
		61	4	23	350
		152	2	12	295
	5	15	71	295	2,492
		61	31	163	2,473
		152	15	85	2,079
	10	15	754	3,129	26,480
		61	329	1,733	26,271
		152	156	902	22,092
	15	15	1,508	6,258	52,960
		61	658	3,466	52,542
		152	313	1,804	44,183
P-213	3	15	320	1,327	11,226
		61	139	735	11,137
		152	66	382	9,365
	5	15	7,991	33,164	280,641
		61	3,486	18,367	278,429
		152	1,657	9,562	234,133
	10	15	159,821	663,281	5,612,815
		61	69,727	367,340	5,568,590
		152	33,137	191,238	4,682,654
	15	15	940,124	3,901,656	33,016,560
		61	410,160	2,160,821	32,756,409
		152	194,922	1,124,929	27,545,025

* 1 in. = 25.4 mm.

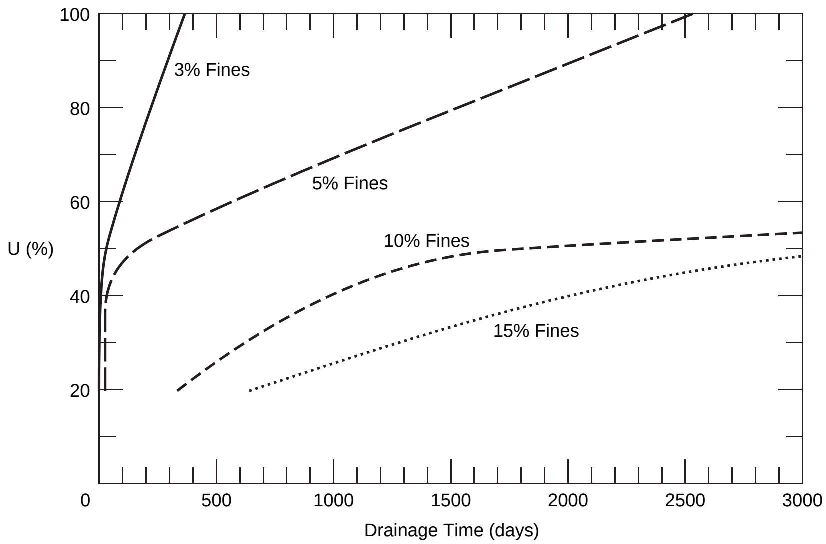


Figure 16. Effect of fines content on drainage.

the airport consisted of 330 mm (13 in.) of AC, 584 mm (23 in.) of base and 914 mm (36 in.) of subbase over a silty, fine sand subgrade. The base and subbase gradations are shown in Figure 18. The subbase falls within the FAA P-154 specifications, while the base falls within the P-208 aggregate base course specifications. The percentage passing the no. 200 sieve for either material was approximately 12%. The results in Figure 17 were determined from laboratory tests and clearly show that, as the base and subbase material thawed, there is a significant decrease in the material modulus.

Other indications of the reduction of the bearing capacity of base and subbase were obtained from field CBR tests by the COE in the mid-1940s. In-place CBR tests were conducted on the top of the base or on top of the subgrades at several airfields (flexible) during the fall and in the thaw-weakening period in the spring. Table 15 shows the change in average CBR during the normal and thaw-weakening periods on top of the base and subbase layers. The base and subbase materials in the table are classified using the United Soil Classification System. When possible, the amount of fines in the material is shown in the table. There is a reduction of 13 to 62% in CBR during the thaw-weakening period. Factors that would affect the amount of reduction are the depth of frost penetration, fines content, and permeability of the base or subbase layers.

Stabilized materials such as the lime-treated subgrade (P-155), soil–cement base (P-301), cement-treated base (P-304) and econocrete subbase (P-306) are classified as cementatious materials. The thaw-weakening resistance of these stabil-

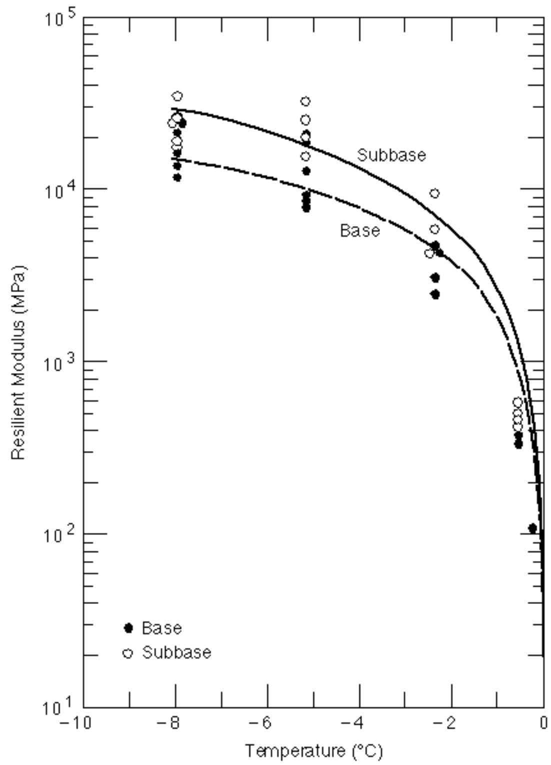


Figure 17. Change in resilient modulus of base and subbase during thaw (1 kip/in.^{2} = 6.89 MPa).

ized materials are inferred from the freeze–thaw durability tests (D559 and D560, ASTM 1992c, 1996). The results of these tests simply show whether or not the material would retain a specified percentage of weight at the end of 12 cycles of freeze–thaw cycling. If it does, then it is considered to be frost resistant. The results from the durability test cannot be used as engineering proper-

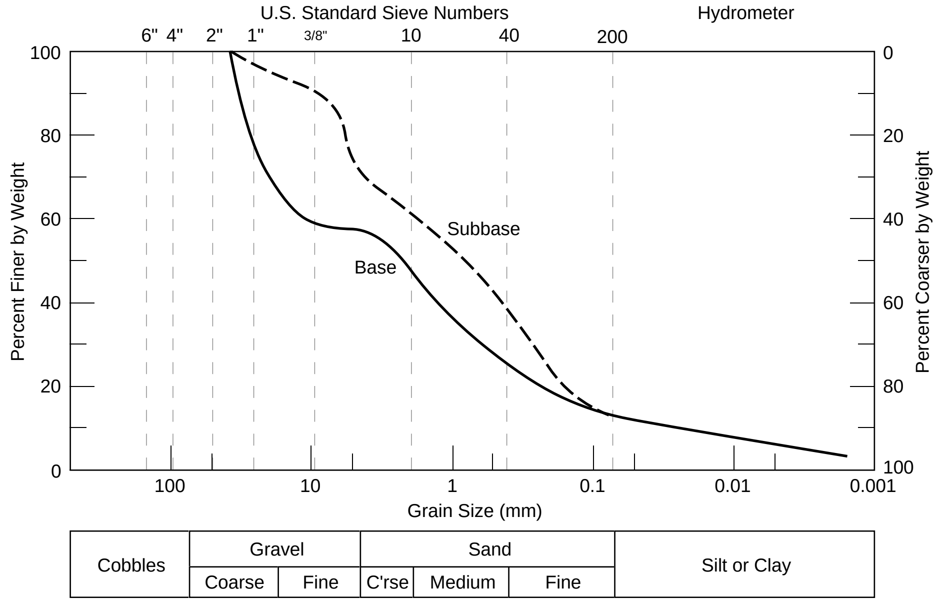


Figure 18. Grain size distribution of base and subbase under Taxiway A, Albany County Airport, New York.

Table 15. Change in CBR during thaw-weakening period.

Layer	*USCS*	*Percent passing no. 200 sieve*	*CBR Normal*	*CBR Thaw-weakening period*	*Percent reduction*
Subbase	GM	10–20	64	33	48
Base	GM-SM		37	27	27
Base	GC	6–12	37	14	62
Base	GW		58	28	52
Base	GW		24	21	13

ties for mechanistic design. There are very limited data on the engineering properties (required for mechanistic design) of stabilized soils subjected to freeze–thaw cycling. Laughlin (1984) in his review reported that these soils undergo large volume changes when subjected to changes in temperature and moisture, with moisture being the more critical factor. Volume changes can lead to cracking of stabilized layers. Others, like Yoder and Witczak (1975), suggest that freeze–thaw cycling may actually be destructive to lime-treated soils.

Thompson (1969b) reported that the strength of lime-treated soils is reduced when subjected to freeze–thaw cycles (Fig. 19). Strength is usually obtained from unconfined compression tests. There is about a 69-kPa (10-lb/in.^{2}) loss per freeze–thaw cycle. However, Thompson believes that, although there is a strength loss, the residual strength of the lime-treated soil after freeze–thaw cycling is high enough for roadway pavements. For performance analysis, the unconfined strength, the tensile strength, shear strength and, if elastic theory is used, the resilient properties of stabilized soils are needed.

Whether this statement applies to airport pavements is a question. Thompson (1969b) also concluded that the material was weakest during the first spring thaw after construction. He expected the material to continue to cure and increase in strength during the following summer and fall. Brandl (1981) reported that after 1 to 2 years, the strength became fairly constant and that, after 7 years, there was no change.

Kettle and McCabe (1985) presented some data on the freeze–thaw resilient modulus and tensile strength of cement-treated soils. The soils tested ranged from a dense graded stone to silty clays. The gradations of the soils tested are shown in Figure 20. Unfortunately, none of these gradations fall within the P-304 or P-306 gradation limits. However, the results are useful for P-301 soil–cement base course. The soils were blended with portland

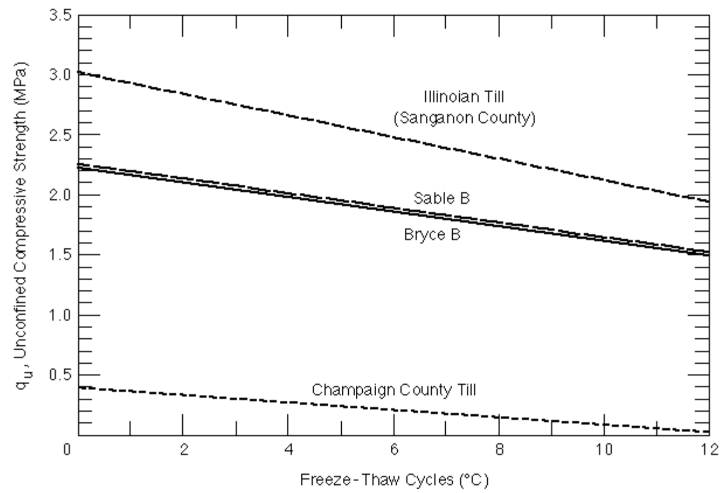


Figure 19. Influence of freeze–thaw cycles on unconfined compressive strength of lime–soil mixtures (48-hour curing at 120°F) (1 kip/in.^{2} = 6.89 MPa) (after Thompson 1969b).

cement (5, 10, 15%), compacted at the optimum moisture content and cured for 7 days. The CL-ML and ML soils were compacted at 14% moisture content. The tests were conducted using Marshall-type cylindrical samples. Kettle and McCabe attempted to relate the results from the durability test (D-560, ASTM 1992c) to the resilient modulus and tensile strength of the soil subjected to a number of freeze–thaw cycles. They found that the resilient modulus after three thaw cycles was a good indicator of materials that failed the durability test.

A summary of the tensile strength and M_r is presented in Tables 16 and 17. The before-freeze data in Table 16 refer to the average strength of the material prior to freezing. The residual refers to strength of the thawed specimens after the ASTM freeze–thaw test (D-560), i.e., after 12 freeze–thaw cycles. The results indicate that the strengths of most of the soils at 5% cement treatment are drastically reduced after 12 freeze–thaw cycles. At 15% cement level, most of the soils actually gained some strength.

Brandl (1981) also reported results where low lime contents actually reduced the tensile strength of the mixture. He attributed it to poor cementing between the soil particles. The results after freeze–thaw cycling (≤10% cement) indicate either poor cementation or breaking of the soil–cement bonds. Poor cementation may not be the

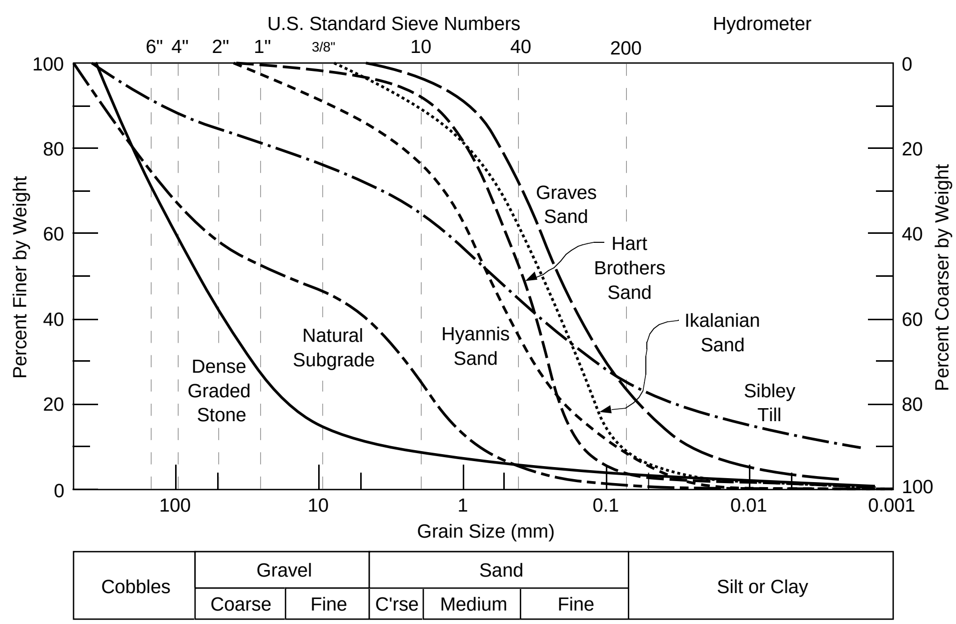


Figure 20. Grain size distributions of test soils and natural subgrade.

Table 16. Summary of tensile strength data.

Material	*Percent fines passing no. 200 sieve*	*Cement content (%)*	*Tensile strength (kPa)** *Before freeze*	*Residual*
Hart (SM)	19	5	250	20
		15	540	720
Graves (SM)	33	5	76	35
		10	221	165
		15	460	475
Sibley (SC-SM)	42	5	610	550
		10	710	720
		15	1010	1180
DGS (SP-SM)	9	5	400	40
		10	790	420
Hyannis (SM)	23	5	69	21
		10	280	120
		15	410	490
Manchester silt (CL-ML)	93	5	7	0
		10	14	1
		15	25	24
Ikalanian (SM)	35	5	9	1
		10	27	20

* 1 kip / in.^{2} = 6.89 MPa.

reason, because the before-freeze tensile strengths actually increased with increased lime content.

The resilient modulus tests were done at 5% of the tensile strength load level. A Poisson's ratio of

Table 17. Summary of resilient modulus data.

Material	*Cement content (%)*	*Before-test* modulus (MPa)*	*Thaw modular ratios (%)* *1-cycle*	*3-cycle*	*12-cycle*
Hart (SM)	5	1931	0.64	0.30	0.04
	10	17582	0.98	1.00	1.01
	15	22629	0.91	0.98	1.03
Graves (SM)	5	1820	—	0.23	0.10
	10	4599	—	1.17	1.32
	15	9494	—	1.17	1.18
Sibley (SC-SM)	5	11784		1.00	1.08
	10	10915	1.04	1.06	1.20
	15	14824	0.96	1.13	1.21
DGS (SP-SM)	5	21892	0.95	0.02	0.00
	10	28056	0.88	0.85	0.69
Hyannis (SM)	5	3234	0.88	0.35	0.25
	10	7315	0.91	0.45	0.41
	15	10977	0.99	1.022	1.04
Ikalanian (SM)	5	3592	—	0.50	0.35
	10	7460	—	1.06	1.15
Fairbanks (ML)	5	7522	0.96	1.16	1.17
	10	14976	1.00	1.11	1.15

* 1 kip / in.^{2} = 6.89 MPa.

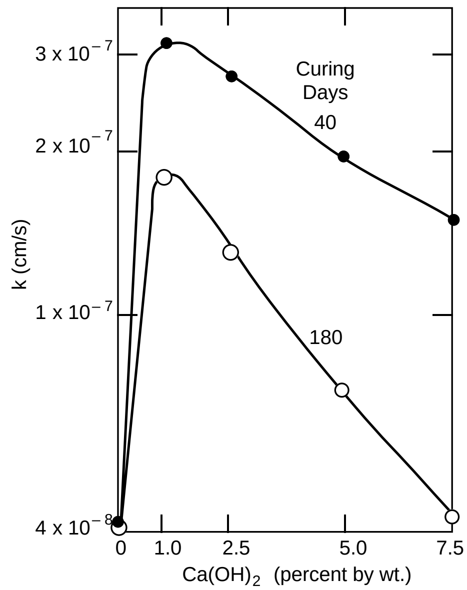


Figure 21. Effect of lime content on the permeability of the soil–cement mix (1 cm/s =3.3 × 10^{-2} ft/s).

0.25 was assumed. Again, as with the tensile strength data, most of the soils had reduced strength when only 5% cement was used. Increasing the cement content to 10% and above actually increased the stiffness of the material by about 10 to 30% after freeze–thaw cycling. Although the material has a high elastic modulus, the tensile strength data show a reduction of strength at the same lime content. However, the results were at optimum moisture content. Changing the moisture content to saturation levels (as occurs during the thaw periods) may produce lower modulus values. At low lime content (≤6%), Brandl (1981) reported that the permeability of lime treated ML soils increased (Fig. 21).

CONCLUSION

We evaluated 11 FAA subsurface material specifications for freeze–thaw resistance. Most of the unstabilized materials allowed up to 15% fines passing the no. 200 sieve, with the exception of the crushed aggregate base, which allowed no more than 8% fines, and the sand–clay base, which allowed up to 25% fines. We came to the following conclusions on the basis of our review of the literature.

- With the exception of the P-154 (subbase) and P-209 (crushed aggregate) materials, the remaining unstabilized soils did not meet the COE frost-susceptibility requirements of no more than 3% finer than 0.02 mm

(0.78×10^{-3} in.). Since the FAA specifications for the remaining materials gave no information on the percentage finer than 0.02 mm, the percentage passing the no. 200 sieve was used to determine frost-susceptibility. However, this has been found to be conservative, i.e., materials that would pass the COE criterion failed the percentage passing the no. 200 criterion. Using the Asphalt Institute criterion, we evaluated the remaining base and subbase materials and found that, within the limits specified, the remaining unstabilized materials were frost-susceptible.

- We also found that for oolitic limestone, the percentage passing the no. 200 sieve had no influence on frost heave, although this was not the case for hard limestone. For the oolitic limestone, the amount of moisture present in the layer had a significant effect on frost heave. This finding also probably applies to the caliche and shell base materials. Addition of slag as substitute aggregate, up to 50%, was found to reduce frost heave.
- For the frost-susceptibility of stabilized (lime and portland cement) soils, the data showed that a minimum of 3% lime or cement is required to reduce frost heave by about 50%. The addition of a pozollith to lime or cement appeared to reduce frost heave significantly in ML and CL soils. In cohesionless soils, it was reported that about 3 to 8% cement is required to reduce frost heave. For frost-susceptible gravel soils, 2% cement is required to change it to a non-frost-susceptible material.
- Thaw-weakening of the base material was inferred from the time it took to drain an airport pavement by 20, 50, and 100%. The COE drainage requirement is that 50% of the base be drained within 10 days. We used the drainage model developed by Casagrande and Shannon in this study, finding that none of the materials passed the COE criterion. Some of the materials came close to it, suggesting that if the fines contents were further reduced to 2 or 3%, they may come close to meeting the criterion. As for the P-213 material, the base appeared to remain saturated all year round. This material should not be used in seasonal frost areas.
- Available data showed a hundred-fold reduction in the frozen and thawed modulus of materials that meet the P-154 and P-208 specifications. Field test data showed a reduction in the bearing capacity of base and subbase materials that ranged from 13 to 60%.
- For stabilized materials, we found that the results of the ASTM (1992c) D-560 durability test can be used to infer the frost-resistance (thaw-weakening) properties of the material. Data were found that clearly showed the reduction of strength as a function of freeze–thaw cycles for lime-treated soils. For cement-treated soils, we found that the tensile strength of the material decreased with increasing freeze–thaw cycles. We also found that when 15% cement was added to the soil, there was no reduction in the tensile strength after 12 freeze–thaw cycles. The resilient modulus of cement-treated soils remained at its before-freeze level when cement contents greater than 5% were added to the soil. Although the modulus was the same, the strength of the material decreased. The permeability increased in soils treated with up to 6% lime. No information was found on the thaw-weakening characteristics of cement-treated bases and econocrete subbases.

RECOMMENDATIONS

The results presented in this report are based on limited information found in the literature, so actual laboratory and field tests should be conducted to validate some of the findings. In any case, the following recommendations are made.

- The COE frost-susceptibility criterion should be included in all appropriate FAA specifications.
- For reducing frost heave and the thaw-weakening period, the amount passing the no. 200 sieve should be kept below 2%. We highly recommend that drainage layers, meeting the COE criteria, be included in future airport pavement design and construction in cold regions. Drainage layers should be placed below the pavement surface and at the top of the subgrade.
- An effort should be made to develop engineering properties of both unstabilized and stabilized base and subbase materials subjected to freeze–thaw cycling. This was found lacking in the literature. These data will be critical to the development of the FAA mechanistic design procedure.

- A laboratory or field study must be conducted to validate the drainage time calculated in this study. A laboratory study using a flume can be conducted to determine the horizontal permeability of base and subbase materials.

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13. ABSTRACT (*Maximum 200 words*)

Pavement structures located in regions with seasonal changes encounter regular cycles of freezing and thawing. Such environmental factors must be considered so that it can be certain that the pavement can accommodate continuous aircraft loading. Eleven subsurface materials specified by the Federal Aviation Administration (FAA) were examined to determine their susceptibility to frost heave and thaw-weakening. All but two of the materials were found to be frost-susceptible under the U.S. Army Corps of Engineers criterion that no more than 3% of fines be smaller than 0.02 mm (0.78×10^{-3} in.). The frost-susceptible materials were also evaluated using Asphalt Institute criteria, which also categorized them as frost-susceptible. The 11 materials were evaluated for susceptibility to thaw-weakening using the drainage model developed by Casagrande and Shannon (1951), which focuses on the permeability of the drainage layer. The final recommendations (which are based only on a literature review) are that, to reduce frost-susceptibility and thaw-weakening, the amount passing the no. 200 sieve should be kept lower than 2% and drainage layers should be installed below the pavement.

14. SUBJECT TERMS

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Drainage	Stabilized bases
Freeze–thaw	Thaw-weakening

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