travel speeds do not, however, yet exist.

In real situations it is necessary to consider practical factors other than energy efficiency in the selection of the best equipment and procedures for a given job. Also, in compactor design, the mechanical efficiency in applying the energy to the soil may be more important than the compaction energy required by the soil, but the results of this study suggest that there are opportunities for significant improvements in design.

## CONCLUSIONS

Much more information than is available from the literature or from this limited investigation is necessary to provide a thorough understanding of the energy requirements of soil compaction. Nevertheless, some preliminary conclusions about the compaction of cohesive soils appear justified.

1. For all moisture conditions, static compaction is the most efficient.
2. Either impact or kneading compaction may be next most efficient, depending on the details of the procedures used.
3. The optimum moisture content is apparently the lowest at which excessive shear of the soil occurs during compaction.
4. The most important factors controlling the compactive effort required to obtain a specified density with a given cohesive soil at a given moisture content are (a) the magnitude of the compactor-soil contact pressures (the highest contact pressure that does not cause excessive shear is most efficient) and (b) the rate at which the load is applied and the length of time the load is held on the soil (the slower the rate and the longer the load is applied, the higher the efficiency).
5. The data for energy efficiency with respect to the strengths of compacted soils are contradictory. This question needs more study.
6. Much more information is needed, but significant improvements in compaction equipment design could be made that would increase efficiency of operation and yield a compacted soil with better engineering properties. The problem is certainly worthy of additional study.

## ACKNOWLEDGMENTS

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## REFERENCES

1. T. W. Lambe. The Engineering Behavior of Compacted Clay. Journal of Soil Mechanics and Foundations Division, Proc., ASCE, Paper No. 1655, May 1958.
2. A. W. Johnson and J. R. Sallberg. Factors Influencing Compaction Test Results. HRB, Bulletin 319, 1962.
3. C. L. Dhawan and J. C. Bahri. Stable Density. Proc., 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 1, 1957, p. 149.
4. G. G. Sowers and C. M. Kennedy. Effects of Repeated Load Applications on Soil Compaction Efficiency. HRB, Bulletin 93, 1954, pp. 61-64.
5. R. R. Proctor. Relationship Between the Foot Pounds per Cubic Foot of Compactive Effort to Soil Density and Subsequent Consolidation Under Various Loadings. Proc., 2nd International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 5, 1948, p. 223.
6. D. J. Maclean and F. H. P. Williams. Soil Mechanics for Road Engineers. Her Majesty's Stationery Office, London, 1952.
7. H. B. Seed and C. L. Monismith. Relationship Between Density and Stability of Subgrade Soils. HRB, Bulletin 93, 1954, pp. 16-32.
8. J. L. McRae and P. C. Rutledge. Laboratory Kneading of Soil to Simulate Field Compaction. Proc., HRB, Vol. 31, 1952, pp. 593-600.
9. K. S. Sankaran and K. Muthukrishnaiah. Efficiency of Compaction of Soil Structure. Journal of Materials, Vol. 5, No. 4, Dec. 1970, p. 739.
10. H. B. Seed, J. K. Mitchell, and C. K. Chan. The Strength of Compacted Cohesive Soils. In Shear Strength of Cohesive Soils, ASCE, June 1960, p. 877.
11. A. W. Johnson and J. R. Sallberg. Factors that Influence Field Compaction of Soils. HRB, Bulletin 272, 1960.

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# Mix Design, Durability, and Strength Requirements for Lime-Stabilized Layers in Airfield Pavements

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**Laboratory and field evaluation programs leading to the development of a design process for lime-stabilized airfield pavement layers are described. The entire process can be completed in 3 to 7 d. The design procedure, which includes selection of the optimum percentage of lime, rapid cure,**

durability testing, and residual strength requirements, uses common laboratory tests and requires no sophisticated equipment. It is appropriate for expedient and nonexpedient construction situations. Residual-strength curves were constructed from the results of a computer study that used a nonlinear finite-element code capable of simulating multiple wheel-gear loadings.

Mixture design procedures for stabilized pavement layers must consider the type of stabilizing agent, the optimum amount of it, and the strength and durability. The design sequence should be applicable to a wide variety of types of soil and relate to the strength requirements of the structural layer.

The Soil Stabilization Index System (SSIS) (2) was developed for the U.S. Air Force as a basis for a stabilized soil-mix design. It was conceptual in nature and required validation for a wide range of soil types. The SSIS provides various alternatives for the selection of the stabilizer and the determination of the optimum amount.

## OBJECTIVES

The overall objective was to validate the SSIS through laboratory testing and field evaluation. Other objectives were

1. To evaluate the pH test proposed by Eades and Grim (12) for estimating the optimum lime content;
2. To verify the correlation between accelerated curing and the normal curing of lime-stabilized soils and ascertain the validity of curing times and temperatures;
3. To evaluate the three-cycle, freeze-thaw strength as an indicator of the field durability in varied environments;
4. To evaluate the strength after vacuum soak as an indicator of the field durability in varied environments; and
5. To correlate the durability and residual strength requirements for stabilized airfield layers.

Ultimately, the purpose was to develop a design process that used simple laboratory tests and equipment and could be applied by military engineers in expedient and nonexpedient situations. The validation methods for lime, cement, and asphalt stabilization and a literature survey of previous research in stabilized mix design have been given by Currin and others (1).

## LABORATORY TESTING PROGRAM

### Materials

The soils listed in Table 1 were selected as representing a wide range of soil classifications and a variety of soil-forming processes. Many were taken from military bases where stabilized layers have been used.

### Eades and Grim pH Test

The analysis of the Eades and Grim pH test (12) was an essential phase in the testing program. Theoretically, the percentage of lime that gives a pH of 12.4 for the lime-soil mixture can be used to obtain the optimum strength conditions.

The percentage of lime obtained by the Eades and Grim test was compared to that obtained by using standard 28-d unconfined compressive-strength tests. For 3 of the soils (Craig, Moody, and Cannon), the inconclusive strength data available prohibited determination of the optimum percentage of lime. The pH test predicted the optimum percentage of lime within 1 percent of that obtained by using strength data for 16 of the soils and within 2 percent for the other 3 soils (Tyler, Coari, and Robbins). Regression analysis showed an excellent correlation between the percentages of lime determined by the two tests.

The majority of the soils investigated in this program could not be considered problem soils as a result of high sulfate or organic concentrations. Although the Tyler soil had high sulfate and organic contents (3.5 and 3.0 percent respectively), no conclusions could be reached about the effects of these on the pH test. The Robbins soil, which had low sulfate and organic contents (0.04 and 0.22 percent respectively), showed similar variations between the pH lime percentage and the strength lime percentage.

These results indicate that the Eades and Grim pH test is acceptable for the determination of an initial percentage of lime to be used in stabilization analysis.

### Strength Development Using Rapid Cure

Both curing time and curing temperature affect the strength of soil-lime mixtures (3). Higher than normal temperatures activate the strength-producing pozzolanic reactions, which shortens the curing time required for strength development. Generally, higher temperatures produce higher strength over a given time interval, and the strength gain at a particular temperature initially increases at a rapid rate and gradually decreases with increasing curing time.

Dunlap and Biswas (3) have tried to correlate normal 28-d curing with accelerated curing. They found the curing times given below to be equivalent to a 28-d normal cure for lime-soil mixtures [°C = (°F - 32)/1.8].

| Temperature (°C) | Time (h) |
|---|---|
| 40 | 65 |
| 49 | 30 |
| 60 | 10 |

They also found that accelerated curing temperatures appeared to produce chemical reaction products similar to those obtained in normally cured specimens.

For this study, the unconfined compressive strengths of lime-soil mixtures molded at the optimum percentage of lime, the optimum moisture content, and the maximum dry density were measured after 28 d of cure at 23°C (73°F) and 100 percent relative humidity. Identical specimens were cured at 49 and 40°C (120 and 105°F) until the 28-d normal cure strength was duplicated.

Although the 60°C (140°F) curing temperature produces chemical reaction products similar to those produced by a normal cure, the 40 and 49°C temperatures are more realistic and conservative. The 49°C curing temperature gave more consistent data than did the 40°C curing temperatures and reduced the curing time appreciably with what seemed to be very little risk of altering the chemical reaction products. Therefore, the 49°C curing temperature was used.

Unconfined compressive strength versus hours of curing at 49°C data were collected. The curing time to produce the equivalent of a 28-d normal cure for all soils tested was determined statistically to be 30 h, although the data varied over a wide range.

The correlation obtained by regression analysis between the rapid and normal cures showed that a 30-h cure at 49°C is a valid substitute for a normal cure and may be used before durability or strength tests. Rapid-cure values are generally slightly conservative.

## Freeze-Thaw Test as Durability Indicator

Both exposed-surface and vacuum-flask freeze-thaw tests were used. However, the vacuum-flask method appears to be more realistic and thus is discussed here. The strength loss was determined at 3, 6, and 9 freeze-thaw cycles. A second-order regression curve relating the strength loss to the number of cycles was then plotted for each soil.

The mean of the points on the best fit curves for all of the soils at which the slopes were equal to zero (i.e., zero strength loss with increasing number of cycles) is 6.74 cycles. There is remarkably little scatter about the mean $dy/dx = 0$ value for the various soils. The data show that after approximately 7 cycles of freeze thaw, there is no further significant reduction in strength.

A family of second-order curves of the general equation of $y = ax + bx^2$ was developed on the basis of the shape of the actual data curves. These curves represent the freeze-thaw behavior of lime-soil mixtures that are within 2 percent of the optimum percentage of lime as the difference between their 28-d strength and their strength after a certain number of freeze-thaw cycles. Figure 1 was developed from the family of design curves and shows the strength loss over the range of 3 to 7 cycles.

On the basis of these data, the freeze-thaw strength loss can be predicted from knowledge of the 28-d normal cure strength of a lime-soil mixture and the 3-cycle freeze-thaw strength. The following example is illustrative: unconfined compressive strength ($q_u$) after 28-d cure = 1448 kPa (210 lbf/in$^2$) at 23°C (73°F) and 100 percent relative humidity, and $q_u$ after 3 freeze-thaw cycles = 827 kPa (120 lbf/in$^2$). Therefore the strength loss is 621 kPa (90 lbf/in$^2$). If Figure 1 is entered with a 621-kPa (90-lbf/in$^2$) freeze-thaw strength loss at 3 cycles and projected to 7 cycles, a freeze-thaw strength loss of 880 kPa (127 lbf/in$^2$) is predicted. The 28-d $q_u$ minus the 7-cycle freeze-thaw strength loss = residual strength = 568 kPa (83 lbf/in$^2$).

**Table 1. Soils tested.**

| Soil | AASHO Classification | Unified Classification |
|---|---|---|
| Dyess, Texas | A-7-6(12) | CL |
| Altus, Okla. (subgrade) | A-7-6(12) | CL |
| Tyler, Texas | A-7-5(15) | OH |
| Houma, La. | A-7-6(20) | CH |
| Perrin A, Texas | A-7-6(20) | CH |
| Perrin B, Texas | A-7-6(20) | CH |
| Perrin AB, Texas | A-7-6(20) | CH |
| Bergstrom, Texas | A-6(7) | CL |
| Kelly, Texas | A-7-5 | CL |
| Carswell, Texas | A-7-6(20) | CH |
| Tinker, Okla. | A-6 | CL |
| Ellington, Texas | A-7-6(20) | CH |
| Barksdale, La. | A-2-4 | CL-ML |
| Ellsworth, S.D. | A-2-7 | SW-SC |
| Craig, Ala. | | CH |
| Moody, Ga. | A-2-5 | SM |
| Robbins, Ga. | A-2-4 | ML |
| LeMoore, Calif. | A-7-6(16) | CH |
| Malmstrom, Mont. | A-6 | CL |
| Cannon, N.M. | A-1-b | SM |
| Estirado, Equador-Brazil | A-7-5(8) | CL |
| LaBrea, Brazil | A-7-5(14) | CH |
| Coari, Brazil | A-7-5(4) | CL |
| Eirunepe, Brazil | A-7-5(1) | CL |

## Immersed Strength as Durability Indicator

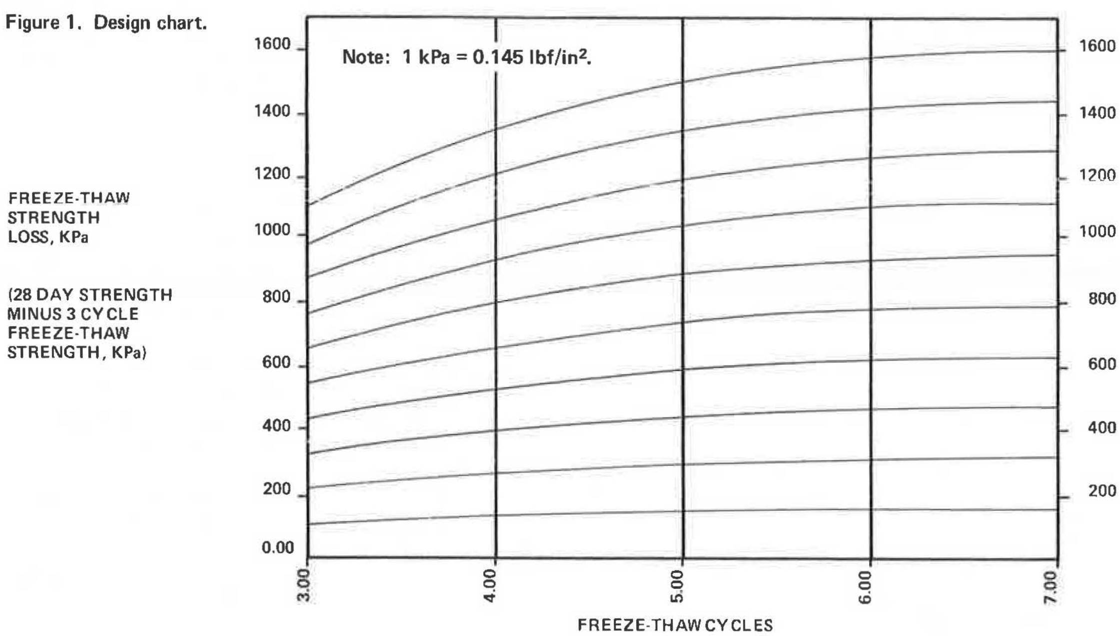
Another objective was to substantiate whether or not immersion testing, particularly vacuum saturation, can be used as an alternative to freeze-thaw testing.

The vacuum immersion test may be conducted after a normal 28-d cure at 23°C (73°F) or a rapid cure of 30 h at 49°C (120°F). If a rapid cure is used, a 2-h equilibrium period is required to permit the stabilized mixture to cool. The specimens must be sealed during this period to prevent moisture loss.

To remove air from the voids, the specimens are placed in an upright position within the vacuum vessel and the chamber evacuated to 81.6 kPa (24 in of mercury) for 30 min. The specimens are placed on a perforated plexiglass plate so that all surfaces will be equally exposed to the chamber environment. After the 30-min period, the vacuum vessel is flooded with water to a depth sufficient to cover the soil specimens. The vacuum is removed, and the specimens are soaked for 1 h at atmospheric pressure.

At the end of the soak period, the specimens are removed from the water and allowed to drain for approxi-

**Figure 1. Design chart.**



mately 2 min on a nonabsorptive surface. After the free surface water is drained away, the specimens are immediately tested for unconfined compressive strength. The entire immersion or saturation procedure can be carried out in less than 4 h after the rapid cure.

The strength after vacuum saturation was compared to the strength after 3 cycles of freeze-thaw testing. Preliminary regression analysis showed that there was a distinct change in slope of the regression line at a vacuum-saturation strength of 945 kPa (137 lbf/in$^2$) [a 3-cycle freeze-thaw strength of 345 kPa (50 lbf/in$^2$)]. On the basis of this, separate regression lines were constructed for vacuum-saturation strengths between 0 and 945 kPa (segment 1) and for vacuum-saturation strengths between 945 and 3792 kPa (550 lbf/in$^2$) (segment 2). The correlation coefficients of segments 1 and 2 are 0.82 and 0.79 respectively.

Figure 2 shows the design chart. From the vacuum-saturation strength, by using the proper segment of the design curve, the 3-cycle freeze-thaw strength is obtained. This value is then used with Figure 1 to determine the 7-cycle freeze-thaw strength loss.

Long-term (21-d) immersion as well as short-term (8-d) immersion were considered as alternatives to the vacuum-saturation test. However, 21-d immersion was not considered to be a suitable alternative because of the length of the test and the probability of unconservative results because of hydration while the specimens are submerged. Limited data on 8-d immersion showed that this test may possibly be used as an alternative to vacuum immersion or 3-cycle freeze-thaw, but no conclusive statement can be made yet. The vacuum-saturation test seems to be the most acceptable alternative to freeze-thaw durability testing because it is expedient and requires no sophisticated equipment.

## RESIDUAL-STRENGTH REQUIREMENTS FOR AIRFIELD PAVEMENTS

Analysis of the stresses and strains in pavements subjected to moving wheel loads can be correlated with the performance of those pavements (4, 5, 13). Full-scale tests conducted at the Waterways Experiment Station indicate that, despite the present limitations in computational techniques and materials characterization, it is possible to predict the performance of a pavement from the results of computer analyses (5). Such procedures hold promise for the development of rational design schemes.

### Indicators of Pavement Performance

Five factors that are indicative of the response of a pavement to moving loads were studied. The surface deflection and tensile strain in asphalt concrete can be correlated with the fatigue cracking of asphalt surface courses. In addition, the vertical stress and strain within the subgrade can be correlated with subgrade rutting, which can ultimately relate to distress of the pavement surface. Limiting values of the vertical subgrade strain are the basis for the design procedure for flexible pavements developed by the Koninklijke-Shell Laboratorium in Amsterdam (6).

Because stabilized layers in most pavement sections act in flexure, it was necessary to investigate the tensile stresses in such layers. Flexural cracking in stabilized layers leads to loss of strength and increased stresses applied to the subgrade.

A nonlinear, finite-element analysis technique (AFPRE-AFPAV) capable of simulating multiple wheel loadings was used.

To correlate the performance of pavement sections with the calculated indicators, it was necessary to devise a method of quantitatively assessing the present condition of the section. Accordingly, a rating form was devised that considered the major types of distress mechanisms that could be linked to the structural adequacy of the pavement (1, 7, 8). Each distress category was measured at a particular section, and the summation was used to determine the relative condition of the section. Those sections with higher numerical ratings were considered to be in poorer condition than those with lower ratings.

The test sections were selected from Air Force and Navy bases having stabilized and nonstabilized pavement sections that had been subjected to approximately the same traffic loadings. Information about their construction histories, layer thicknesses, and material properties

**Figure 2. Design chart: 3-cycle freeze-thaw strength from vacuum-immersion strength.**

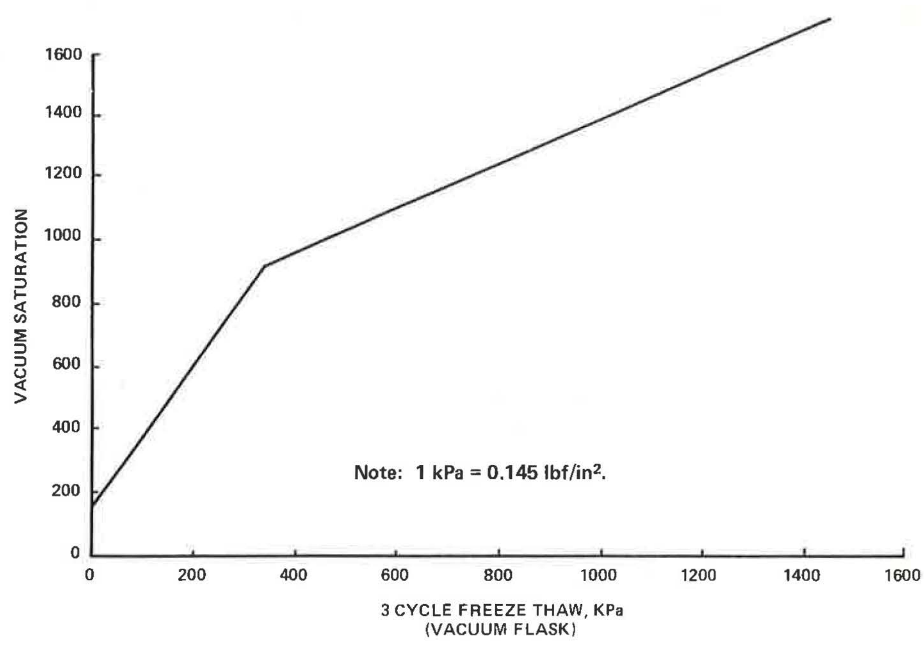


Figure 3. Deflection basins at Seymour Johnson Air Force Base.

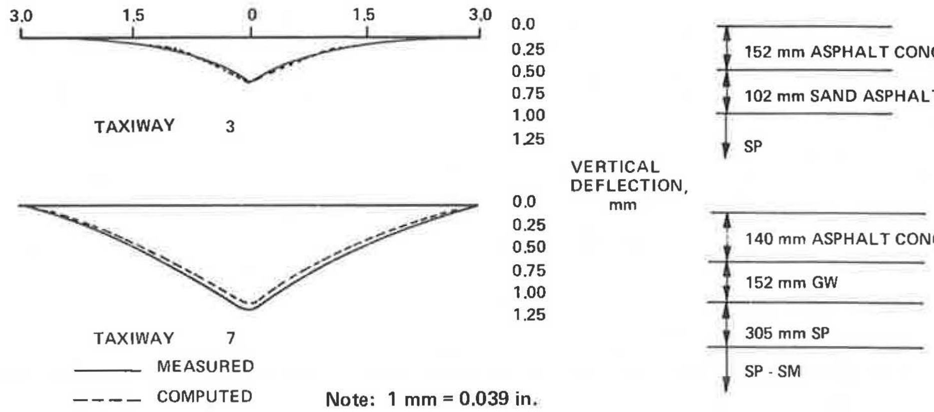



Table 2. Computed responses.

| Section | Max Deflection (mm) | Max Subgrade Stress (kPa) | Max Tensile Strain in Surface Course (mm/mm) | Max Tensile Strain in Stabilized Layer (mm/mm) |
|---|---|---|---|---|
| Taxiway 3 | 0.86 | 90 | 0.000 240 | 0.000 542 |
| Taxiway 7 | 1.65 | 56 | 0.000 697 | — |

Note: 1 mm = 0.039 in; 1 kPa = 0.145 lbf/in$^2$; 1 mm/mm = 1 in/in.

was taken from the Airfield Pavement Evaluation and Condition Survey Reports published by the Corps of Engineers and the Air Force Civil Engineering Center.

The evaluation procedure consisted of the following steps:

1. The test sections were selected from the survey reports,
2. The present conditions of the sections were evaluated by using the rating forms,
3. The deflections under aircraft loadings were obtained by using a precise level capable of measuring vertical motions to 2.5 $\mu$m (0.0001 in), which was sighted on a target placed along the outer edge of the wheel path of the main gear as the aircraft was towed along the predetermined path, and
4. The pavement sections from selected bases were analyzed by using the AFPRE-AFPAV code. Measured and computed deflection basins were matched, and calculated indicators of pavement performance were compared.

Pavement sections at five airfields—Reese, Cannon, Kelly, and Seymour Johnson Air Force bases, and Point Mugu Naval Air Station—were evaluated.

## Seymour Johnson Air Force Base

Although lime-stabilization had not been used at this base, the data are illustrated and show the effectiveness of computer structural analysis of pavement sections.

Taxiways 3 and 7 were evaluated. The nonstabilized section (taxiway 7) had extensive transverse and longitudinal cracking, intermittent rutting, and some alligator (fatigue) cracking. It received a rating of nine. The stabilized section (taxiway 3) had only isolated rutting and cracking and received a rating of three. The deflections were obtained under an F-4 aircraft [gross weight = 114 Mg (52 000 lb) and tire pressure = 1724 kPa (250 lbf/in$^2$)]. The deflections were measured and computed along a line offset 25.4 cm (10 in) from the center of the tire.

Figure 3 shows the measured and computed deflection basins for the two sections. Table 2 summarizes the computed responses of the two sections.

The significantly smaller deflections and strains in taxiway 3 account for its better surface condition as opposed to that of taxiway 7. The high subgrade stress calculated for taxiway 3 may account for the isolated rutting observed. The higher subgrade stress for this section compared to that of taxiway 7 can be attributed to the much smaller thickness of pavement.

The field performance evaluation showed the accuracy with which a powerful analytical tool such as the AFPRE-AFPAV code can be used in computing deflection basins.

## Development of Residual-Strength Curves

The strength requirements for highway pavements and their associated loadings derived by linear analytical techniques have been given by Dempsey and Thompson (9). Because airfield pavement thicknesses and wheel loadings are significantly different from those of highways, it was necessary to determine the strength requirements for airfield pavement sections. Furthermore, because high-quality stabilized layers have greater stiffness than do underlying natural materials, they act in a flexural mode. The limiting value of strength for these layers to be investigated is the flexural strength.

The AFPRE-AFPAV nonlinear computer codes were used to account for the nonlinear stress and strain relations of paving materials, particularly natural subgrades and unbound granular layers.

The objective of this phase of the investigation was to determine the flexural strength required of stabilized pavement layers. These values were correlated with the unconfined compressive strengths ($q_u$) that would be required in the pavement after the first freeze-thaw season. The procedures described above were then used to determine the required $q_u$ before freeze thaw on the basis of the anticipated number of freeze-thaw cycles.

## Analysis Procedure: Lime-Stabilized Layers

Typical flexible and rigid pavements were analyzed for aircraft loading in the three design categories described in the Air Force Manual. The F-4E aircraft was used for the light-load category because it has the highest gear load in that category. The C-141 was used for the medium-load category, and the B-52 was used for the heavy-load category. Flexible pavements were not analyzed in the heavy-load category, nor were rigid pavements analyzed for the light-load category. A wide range

Figure 4. Residual-strength requirements for lime-stabilized layers in airfield pavements (flexible pavements: light-load design).

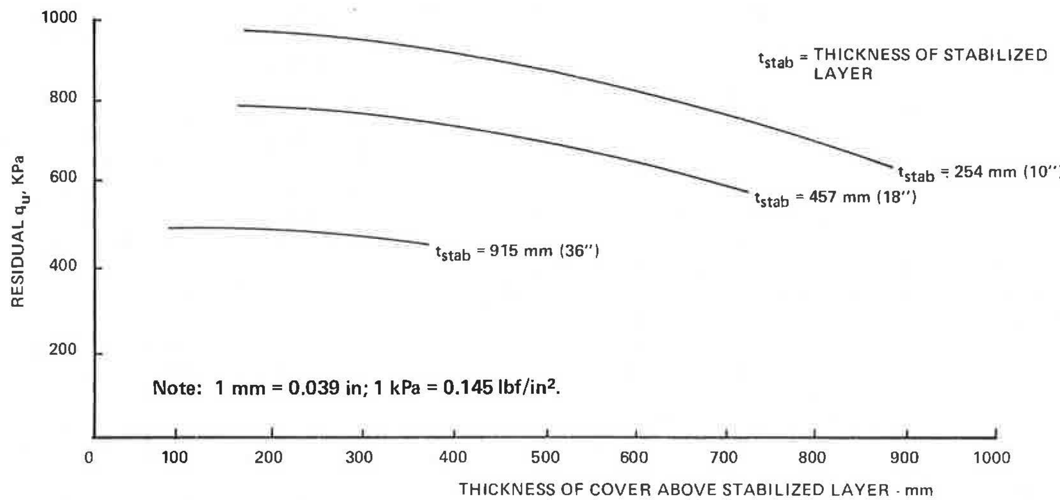



Figure 5. Residual-strength requirements for lime-stabilized layers in airfield pavements (flexible pavements: medium-load design).

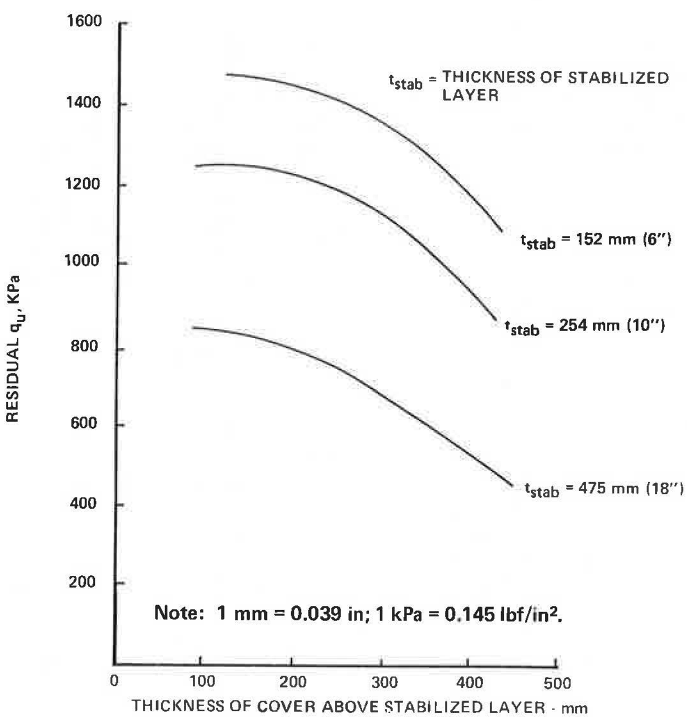
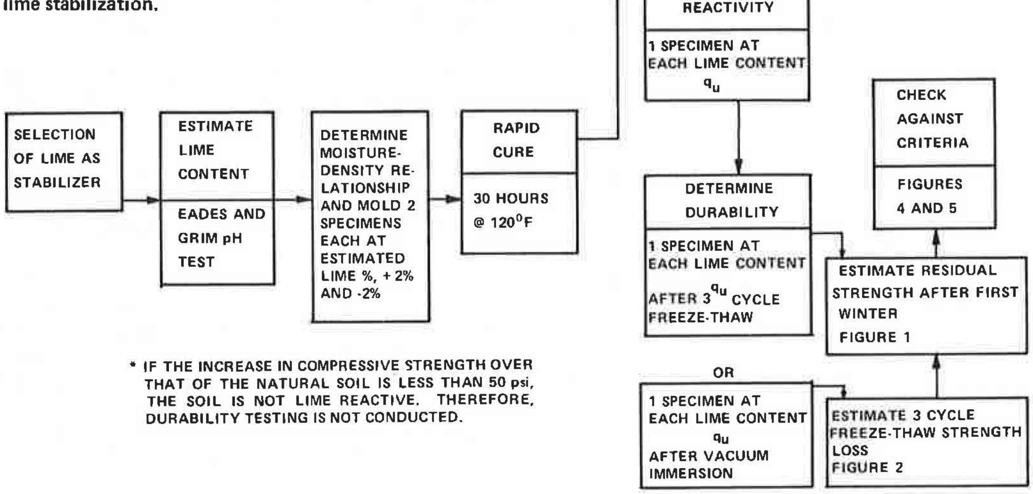



Figure 6. Design procedure for expedient and nonexpedient lime stabilization.



of subgrade types, pavement thicknesses, and stabilized-layer properties were investigated. The aircraft landing-gear configurations and wheel loads cover the range with which the military engineer is involved. The majority of civilian jetliners have similar characteristics.

### Residual Strength

It is usually assumed that the most critical period in the life of a pavement containing stabilized layers occurs immediately after the first freeze-thaw period. This is the point at which natural subgrades may be least stable because of high moisture contents and stabilized layers will have suffered the deteriorating influence of the winter freeze-thaw cycles. However, Thompson and Dempsey (10) have shown that stabilized materials will continue to gain strength with increased curing time after the first winter and that the freeze-thaw damage occurring in subsequent winters is not cumulative. Therefore, the flexural strength required after the first freeze-thaw season may be regarded as the minimum necessary for satisfactory pavement performance. The flexural strength was converted to the unconfined compressive strength by using the relation flexural strength = $0.25q_u$. The values of $q_u$ thus obtained represent the minimum required values of unconfined compressive strength (residual strength) that the stabilized materials must exhibit in the field immediately after the first freeze-thaw season.

### Results: Flexible Pavements

Figures 4 and 5 show the residual-strength requirements for lime-stabilized layers for flexible airfield pavements. The design procedure using these figures should include the following steps:

1. Use the standard California-bearing-ratio design procedures to determine the required pavement thickness,
2. Select the individual layer thicknesses,
3. Enter the appropriate figure (design category and type of stabilizer) with the thickness of cover (thickness of material above the top of the stabilized layer) and read the required residual strength from the appropriate curve,
4. Use the procedures discussed above to determine the strength loss for the number of freeze-thaw cycles anticipated for the first season, and
5. Add the anticipated strength loss to the residual strength.

This value represents the $q_u$ required in the field after construction and initial curing and before the first freeze-thaw season.

### Results: Rigid Pavements

Because of the thickness and high modulus of concrete surface courses, the calculated flexural strengths varied over a small range [<69 kPa (10 lbf/in$^2$)]. Therefore, it is recommended that residual strength values of 410 to 550 kPa (60 to 80 lbf/in$^2$) be required for stabilized bases for rigid airfield pavements. These values are higher than would be indicated by the relation that flexural strength = $0.25q_u$. However, lower strength mixes have lower values of flexural modulus, which would allow larger strains and possibly more severe cracking in the stabilized layer.

The preceding discussion has been concerned with pavement response to repeated dynamic wheel loadings; i.e., flexural fatigue was the main consideration. However, stabilized pavement sections develop ultimate strengths far in excess of the stresses that lead to initial cracking. For situations where low traffic volumes are anticipated, e.g., the expedient construction case, the required strengths of the stabilized layers may be significantly overestimated by the use of Figures 4 and 5. The data given by Suddath and Thompson (11) indicate that ultimate strengths may be at least two to three times as large as those predicted by Meyerhof's ultimate-load theory. Clearly, for these situations, the designer is justified in accepting lower strengths than those indicated by Figures 4 and 5.

## DESIGN SEQUENCE

The design procedure shown in Figure 6 was developed on the basis of the laboratory data and computer analysis. Selection of the stabilizing agent is determined by the grain-size distribution and the plasticity and is discussed by Currin and others (1).

The entire design process can be accomplished in approximately 1 week. If vacuum saturation is substituted for freeze-thaw testing, the time involved is shortened to 3 or 4 d.

## CONCLUSIONS

The design process described here uses common laboratory tests and requires no sophisticated equipment. It can be accomplished in a short time and is appropriate for use by engineers untrained in stabilization technology. The adequacy of a proposed mix is ultimately determined by comparing its strength after durability testing with the required residual strength in the field. This process has been validated for a wide range of soil types.

## ACKNOWLEDGMENT

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## REFERENCES

1. D. D. Currin, J. J. Allen, and D. N. Little. Validation of Soil-Stabilization Index System With Manual Development. Frank J. Seiler Research Laboratory, U.S. Air Force Academy, Colorado Springs, Technical Rept. FJSRL-TR-76-0006, 1976.
2. J. A. Epps, W. A. Dunlap, and B. M. Gallaway. Basis for the Development of a Soil-Stabilization Index System. Air Force Weapons Laboratory, Kirtland Air Force Base, N.M., Technical Rept. No. AFWL-TR-70-176, 1971.
3. W. A. Dunlap and B. R. Biswas. Accelerated Laboratory Curing of Lime Stabilized Soils. Air Force Weapons Laboratory, Kirtland Air Force Base, N.M., 1974.
4. H. B. Seed, F. G. Mitry, C. L. Monismith, and C. K. Chan. Prediction of Flexible-Pavement Deflections From Laboratory Repeated Load Tests. NCHRP, Rept. 35, 1967.
5. W. R. Barker, W. N. Brabston, and F. C. Townsend. An Investigation of the Structural Properties of Stabilized Layers in Flexible Pavement Systems. Air Force Weapons Laboratory, Kirtland Air Force Base, N.M., Technical Rept. No. AFWL-TR-73-21, 1973.
6. G. M. Dorman and C. T. Metcalf. Design Curves for Flexible Pavements Based on Layered System Theory. HRB, Highway Research Record 71, 1965, pp. 69-84.

7. G. Y. Sebastyan. Discussion of Airfield Evaluation Procedures by Phillip P. Brown. Journal of the Aerospace Transport Division, Proc., ASCE, April 1965.
8. R. V. LeClerc and T. R. Marshall. Washington Pavement Rating System: Procedures and Applications. HRB, Special Rept. 116, 1971, pp. 112-121.
9. B. J. Dempsey and M. R. Thompson. Durability Properties of Lime-Soil Mixtures. HRB, Highway Research Record 235, 1968, pp. 61-75.
10. M. R. Thompson and B. J. Dempsey. Final Report on Durability Testing of Stabilized Materials. Civil Engineering Studies, Transportation Engineering, Univ. of Illinois, Urbana, Series No. 11, June 1974.
11. L. P. Suddath and M. R. Thompson. Load-Deflection Behavior of Lime-Stabilized Layers. U.S. Army Construction Engineering Research Laboratory, Champaign, Ill., Technical Rept. M-118, 1975.
12. J. L. Eades and R. E. Grim. Quick Test to Determine Lime Requirements for Lime Stabilization. HRB, Highway Research Record 139, 1966, pp. 61-72.
13. J. P. Nielsen and G. T. Baird. Nondestructive Pavement Load Rating. Proc., Symposium on Nondestructive Test and Evaluation of Airport Pavements. U.S. Army Waterway Experiment Station, Corps of Engineers, Vicksburg, Miss., May 1976.

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# Frost Action in Cement-Stabilized Colliery Shale

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**This paper describes a laboratory investigation of frost action in cement-stabilized colliery shales in which their performance is evaluated in terms of the heave developed during prolonged freezing. Tests of nine unburnt and four burnt shales showed that the addition of cement reduced heave, except in the case of some fine-grained unburnt shales. These results are discussed in relation to the effect of cement on pore size, on permeability, and on strength. Of the strength tests undertaken, only the direct tensile test provided data that could be related to the behavior observed during a freezing test. Significant heave occurred only when the heaving pressure generated at the freezing front was greater than the tensile strength. It is concluded that freezing behavior is consistent with an energy balance between the work done in heaving and the energy liberated by supercooled freezing.**

In recent years, there has been increasing interest in the use in road construction of nontraditional materials, such as industrial waste. A major source of such material in Great Britain is in the colliery tips that are probably the greatest single cause of dereliction of industrial land, so that their removal is also desirable in environmental terms. The material in these tips is the residue after coal has been removed in the washery and is generally referred to as colliery shale. It is customary to further describe the material as either unburnt or burnt because burning occurs in some tips under certain conditions. Broadly, burning converts the clay-like shale into a brick-like material having greatly changed properties.

Selected burnt shale has been used extensively in highway construction in Great Britain for many years as fill and as subbase material, but unburnt shale has been used significantly only during the past 8 years, following the issue of technical memoranda (1, 2) by the Department of the Environment that allow its use as common fill. However, because of its clay-like nature, it is unlikely that unburnt shale in its natural state will be suitable for use in pavement structures; the work reported in this paper is part of an investigation that examined its value when stabilized with cement. A preliminary study (3) showed that some unburnt shales could produce an acceptable soil-cement in terms of the strength criteria imposed in practice (4), and this prompted an extension of the work to the effect of frost action on materials of this type (5).

## OVERALL APPROACH

The aim of the investigation was to test colliery shales in their natural state and when stabilized with cement to identify the characteristics that influence their behavior when subjected to prolonged freezing. Because the principal interest was in the effect of cement treatment, the emphasis was on the measurement of tensile strength as specimen fracture appears to be an essential prerequisite for the growth of substantial ice lenses. Samples of nine unburnt and four burnt shales were studied to compare the behavior of the two types of shale and to evaluate their response to cement treatment.

The behavior of test specimens subjected to frost action was evaluated by the heave that occurred and by the pressure generated when the heave was restrained. Permeability tests were made to obtain data on the water-transport potential of the various materials.

### Preparation of Test Specimens

Cylindrical specimens, 152.4 mm high by 101.6 mm in diameter, were produced in constant-volume molds by using static compaction (6). To permit comparison among the results, the cement-stabilized specimens were made at the same moisture content as the unbound specimens [the optimum value determined on the unbound shale by using the 2.5-kg hammer in the British standard compaction test (7)]. The dry densities were adjusted so that the total air voids were the same for both the unbound and the cement-stabilized specimens of each shale.