TRANSPORTATION RESEARCH RECORD 593

Use of Waste Materials and Soil Stabilization

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Use of Waste Materials in Embankment Construction

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The impact of environmental constraints and economic considerations
compels the engineer to seek new and novel techniques for using waste
materials in embankment construction. This paper describes the use of
sanitary landfill and nonbiodegradable waste (discarded tires) and de-
sign criteria for incorporating waste material into California highway
embankments. Construction guidelines and theoretical considerations
are presented. One case history and plans for a test embankment that
will be stabilized by tire sidewall mats are described.

The California Department of Transportation, like most
other road-building organizations, has in the past placed
severe restriction on the incorporation of unsuitable ma-
terials into highway embankments. Clearing and grub-
bing were an important and rigidly adhered to first step
in the highway construction process. The burial of logs
and stumps was prohibited and, indeed, in some cases,
knots and twigs were picked out of embankments as part
of the construction process. When sanitary landfills were
crossed, the waste was normally stripped to original
ground and disposed of before construction of the em-
bankment began.

Recently, environmental restrictions, economics,
and concern for visual impact have necessitated con-
struction of highways over marginal to extremely diffi-
cult terrain. The options with respect to development
of borrow and waste disposal sites have been severely
restricted and have thus compounded the problem. Thus,
reevaluation of past highway practice with respect to
waste or unsuitable materials has become necessary.

This paper discusses the use of two types of waste
materials incorporated into embankments constructed
along California highways: sanitary landfill waste and
nonbiodegradable waste (discarded tires).

Recently several case histories (1, 2), concerning the
crossing of sanitary landfills with highway embankments,
have appeared in the literature. They describe the con-
struction technique and the results of measures aimed
at minimizing postconstruction settlements.

On Calif-73 in Orange County, California DOT is near-
ing completion of a project in which sanitary landfill
waste is incorporated into embankment construction.
This project is described as is a test embankment yet to
be constructed, in which, it is believed, a systematic
incorporation of tire waste will serve to benefit the fill
and thus permit steeper than normal side slopes and in-
crease resistance to seismic loading.

SANITARY LANDFILL WASTE

General Design Criteria

No general design criteria are described because the con-
tract specifications are described in the discussion of the
case history.

Case History

The sanitary landfill waste project consists primarily of
an interchange in Newport Beach, California, near the
Irvine campus of the University of California (Figure 1).
One segment of the interchange provides ramp access to
MacArthur Boulevard, which was relocated to accommo-
date future full alignment.

The results of a foundation investigation revealed that
foundation soils were generally soft and compressible,
and this necessitated 2:1 side slopes, stabilizing berms,
waiting periods, and controlled rates of loading for em-
bankment construction.

The general pattern of foundation soils consists of
alternating strata of compressible clay and fine to coarse
sands that appear to be free draining. Figure 2 shows
the boring locations and the log of the borings for a por-
tion of the realigned MacArthur Boulevard, including a
portion of the sanitary landfill.

As indicated by the borings, a sanitary landfill con-
taining 152 910 m^3 (200 000 yd^3) of refuse occupied a sec-
tion of the line along which the realigned MacArthur
Boulevard and University Drive would be constructed.
Construction of this landfill began in 1954 and was com-
pleted with 0.6 m (2 ft) of earth cover in 1961.

As the design of the interchange was nearing com-
pletion, it became apparent that removal of this huge

quantity of waste would involve a tremendous expenditure. Construction of embankments over the landfill would subject the roadway to intolerable long-term settlement and compound the problem already present because of the nature of the subgrade soils.

Inquiries by designers with respect to disposal of the waste revealed that the only available option was placement in another sanitary landfill. Since the project was deficient in embankment, the 152 910 m³ (200 000 yd³) would have to be replaced by material obtained from outside the project limits and delivered to the site at an estimated cost of $3.92/m³ ($3.00/yd³). Thus, the potential net savings available by using the waste in embankment construction was estimated at $900 000. This finding prompted further study of the possibility of using the waste in the embankments. There was little information to draw on, except for recent limited experience in the burial of wood waste in embankments. The specifications ultimately developed by the District Design, Construction, and Transportation Laboratory personnel for this purpose are as follows:

Those areas shown on the plans as "Refuse Removal Area" are areas of unsuitable material. The Contractor shall excavate the refuse cover and refuse material and construct embankments within the excavated refuse area with material obtained from excavation within the project limits (except excavated refuse material) or borrow.

At the option of the Contractor, excavated refuse material may be used in embankment construction in the areas shown on the plans as "Refuse Embankment Areas."

In addition to the requirements in Section 19-5, "Compaction," and Section 19-6, "Embankment Construction," of the Standard Specifications, the placement of excavated refuse material in embankments shall conform to the following:

1. Excavated refuse material shall be thoroughly mixed with suitable embankment material at a rate not to exceed 50 percent of the mixture.

2. Each layer of the refuse material mixture shall be covered with at least two layers of suitable embankment material.

3. No layer of the refuse material mixture shall be placed within four feet of finished grade.

4. Rock, portland cement concrete, asphalt concrete, ferrous and nonferrous metals shall not exceed one foot in the vertical dimension when placed in embankments.

5. All other material including biodegradable material shall not exceed one-half foot in greatest dimension.

A typical embankment cross section is shown in Figure 3.

The heterogeneous nature of the waste precluded compaction control by conventional means. However, it was reasoned that placement of waste in relatively thin lifts sandwiched between layers of soil would minimize the risk of low densification, since a relatively firm working table would be necessary to achieve the specification compaction requirement in the soils layers.

Refuse embankment construction requirements of the special provisions to the contract include stripping surface materials at refuse embankment sites to an elevation of 1.2 m (4 ft) and constructing embankments to a finished embankment height subject to the following rates of loading:

1. Place 2.7 m (9 ft) of embankment at a rate of 0.41 m (1.33 ft) per week followed by a 60-day waiting period;

2. Construct the embankment to an elevation of 6.7 m (22 ft) at a rate not to exceed 0.41 m (1.33 ft) per week followed by a 60-day waiting period; and

3. From 5.5 m (18 ft) to finished grade elevation, construct at a uniform rate not to exceed 0.9 m (3 ft) per week.

Heave stakes, piezometers, settlement platforms, benchmarks, and inclinometers were installed for construction control. Additional benchmarks were installed at the top of the fills above the settlement platforms at original ground to monitor compression occurring within the fill itself.

Excavation of the landfill exposed a composition of wood, stumps, paper, fibrous wastes, cans, bedsprings, pipe, wire, glass containers, plastics, tires, bricks, and concrete debris. Organic materials encountered were generally in a good state of preservation. Newspapers, dated in the late 1950s, were clear and readable. As had been anticipated, based on the exploration of the fill in late 1970, groundwater was encountered from 4.6 to 6.1 m (15 to 20 ft) below ground surface and was ponded and later pumped into tank trucks for use in the compaction operation. No discharge of groundwater was permitted to enter into San Diego Creek. Leachate was not considered to be a problem, and no program to monitor leachates from the embankment was initiated since the refuse was to be incorporated into embankments several meters above the water table and sandwiched between layers of relatively impermeable soil. The refuse was excavated from the locations shown in Figure 1 and hauled to the embankment with rear-dump trailer and tractor trucks.

The device that ultimately proved most successful for refuse excavation and loading was a hydraulic backhoe. This had several advantages, including a digging action from the top downward into the saturated refuse that penetrated the rags and paper on the initial thrust and filled the bucket. Wet soft areas were worked by reaching out and down; the machine carriage did not enter the area and bog down. The backhoe capacity was found to be approximately 229 m³ (300 yd³) of refuse per hour. After the refuse was hauled to the embankment location and dumped, bulldozers spread the material in 15.2-cm-thick (6-in) lifts as shown in Figure 4. At this point, unsuitable pieces including tires (Figure 5) were picked out, stockpiled, and eventually hauled away for disposal at a public dump.

Embankment soils for blending with the refuse were hauled to the site in twin-bottom dump trailers. The soil was spread over the in-place refuse with rubber-tired bulldozers and a motor grader as shown in Figure 6. Mixing was accomplished with either a sheepsfoot roller pulled by bulldozers or a self-propelled sheepsfoot compactor as shown in Figure 4. The compactor spikes penetrated the soil and rubbish and pulled, ripped, and split the rubbish as it was mixed with the soil and compacted. The principal problem was the tendency of the compactor to become plugged with refuse. The sandy soil that was used for the embankment proved to be an asset for the blending operation because of its low cohesion. A similar attempt to mix cohesive or clayey soils with the refuse would have been extremely difficult, if not impossible.

The moisture content of the refuse buried was from dry to saturated. Saturated refuse was spread and allowed to air dry before it was blended with the soil. Specifications were included for odor control of the refuse during handling operations. A commercial deodorant, available for use if obnoxious odors were encountered on this project, was not necessary.

Compaction control of the soil lifts sandwiched between the blended refuse lifts was maintained with nuclear gauges. A relative compaction requirement of 90 percent, according to California Test Method 216, was specified and achieved for the soil portion of the embankment. Compaction control of the blended refuse layers was achieved by visual inspection. Inspectors observed the blending and compaction of the refuse layers and directed the modification of the operation where inadequate compaction or mixing was observed. Exposed

layers of the blended refuse were seen as a result of an excavation for a drainage culvert as shown in Figure 7. The blended refuse layers appear across the center of Figure 7, sandwiched between two soil layers. No cavities were observed in the exposed layer. The soil and refuse were moist and thoroughly mixed and could be separated only by using a handpick. The layer appeared to be well compacted.

The sandy soil used for embankment eased the problem of mixing considerably; the success of such an operation with cohesive materials is doubtful. As of June 1976, no significant amount of compression has been detected within the soil-waste fills.

NONBIODEGRADABLE WASTE

General Design Criteria

Engineers have long been aware of the stabilizing effects of inclusions of various materials in earthworks. The first disciplined, and by far the most extensive and successful, application of soil reinforcement was developed by Vidal (3) in the late 1950s. Vidal's system of reinforced earth consists of placing steel reinforcing strips at predetermined intervals within the fill mass for the purpose of providing tensile or cohesive strength in a relatively cohesionless material. For a soil to be satisfactory for reinforced earth construction, Vidal suggests that it be granular and have an angle of internal friction of at least 25 deg so that adequate friction resistance can be developed between the soil and the reinforcing material.

The stabilizing effect of materials with relatively high tensile strength in soil has been observed since ancient times. Increased shear strength with certain types of nonbiodegradable materials was noted during a laboratory study by California DOT (4).

One of the most perplexing solid waste disposal problems involves automobile tires. It has been estimated that approximately 200 million tires are discarded each year in the United States. Air quality legislation precludes burning as a solution. A major problem with respect to burial of tire carcasses in soil is their tendency to eventually work up to the surface. The problem of tire disposal was of sufficient magnitude in California to prompt passage of House Resolution 37 in the 1973 California legislative session, which charged the California DOT to study the problem of abandoned tires and develop possible solutions for their disposal or recycling.

Investigation of the problem of tire disposal revealed that equipment is now commercially available to economically separate tire sidewalls and treads, the latter having been found to be a commercially valuable commodity. The sidewalls alone, having a nearly flat configuration and extremely high tensile strength, are an obvious possibility for soil reinforcement and, if they are placed in strips or mats, could serve to greatly increase the internal stability of an embankment, based on the reinforced earth principle. To go one step further, it was speculated that embankments stabilized in this manner could be constructed at much steeper side slopes than would otherwise be possible and could provide a means of disposal of this troublesome waste product.

To study this possibility further, California DOT Transportation Laboratory conducted an analysis to determine the theoretical effects of tire reinforcement on earthquake resistance of embankments. This analysis assumed tire placement in mats extending for widths of 0.8 of the embankment height at vertical intervals of 1.2 m (4 ft). It was accomplished with the Quad-4 finite

element program developed at the University of California, Berkeley. The finite element mesh (Figure 8) consisted of elements representing the reinforcing mat and boundary soil.

The embankment was assumed to have a relative density of 90 percent and a density of 2082 kg/m³ (130 lb/ft³). Shear modulus G was assumed to vary with overburden height as shown by the following equation (4):

$$G = K_2 (\sigma_o')^{\frac{1}{2}} \tag{1}$$

where

G = shear modulus in pascals,
K_2 = function of relative density D_r, and
σ_o' = effective overburden stress in pascals.

The foundation soil was also assumed to be sandy and had a relative density of 75 percent and a density of 2082 kg/m³ (130 lb/ft³). From equation 1, the K_2 of the 1.5 m (5 ft) of foundation soil is 61. For the composite material, a constant shear modulus of G = 6.37 MPa (133 kips/ft²) was used, based on the results of tests on rubber tire specimens. A constant damping factor of 25 percent was also used. The embankment was assumed to be 7 m (23 ft) in height with 1½:1 side slopes. The earthquake selected was the California Institute of Technology type C-1 with a maximum acceleration of 0.3 g, a period of 0.35 s, and a duration of 12 s applied at the base of 1.5 m (5 ft) of foundation material. This would correspond to an earthquake measuring 7 on the Richter scale at a distance of 24 km (15 miles) from the fault. The results in terms of change or reduction in the dynamic shear stress resulting from reinforcement are shown in Figure 8. Under these conditions, dynamic shear stress would be reduced in the embankment soil by 20 to 62 percent, at an average of about 33 percent. The greatest reduction occurs in the interior; this would indicate that failure, if it did occur, would probably be on the surface. Shear strain would experience a similar trend and would be reduced by about 33 percent in the embankment soil. These values would, of course, vary with side slope, type of soil, earthquake intensity and duration, and fill height. The results of this analysis and the earlier laboratory study of the stabilizing effect of waste led to a decision to construct a prototype test embankment in which tire sidewall mats were used for reinforcement. Federal Highway Administration approval for the instrumentation and analysis portions as a Highway Planning and Research (HPR) project was received on August 8, 1973.

Plans for Embankment

In early spring 1976, a test embankment was suggested by California DOT that, although not ideal from a research standpoint, would definitely be constructed during the 1976 construction season. It is located on Calif-236, about 24 km (15 miles) north of Santa Cruz (Figure 9). The proposal stated that a sidehill fill slipout would be corrected by constructing an embankment approximately 91.5 m long and 15 m high (300 ft long and 50 ft high).

The slide is located on the northwest slope of a narrow, densely forested ridge. This area is underlain by the Rices mudstone member of the San Lorenzo formation, Oligocene Age, and consists of poorly cemented mudstones, siltstone, and sandstone. Bedding planes dip steeply northeastward parallel to the ridge. An investigation of the slide mechanism revealed a depth of unconsolidated and loose slide material and freewater from 18.3 to 21.3 m (60 to 70 ft) below roadway elevation. It was concluded that the primary cause of the slide

Figure 1. Sanitary landfill waste project location map.

Figure 2. Boring locations and soil profile along portion of realigned MacArthur Boulevard.
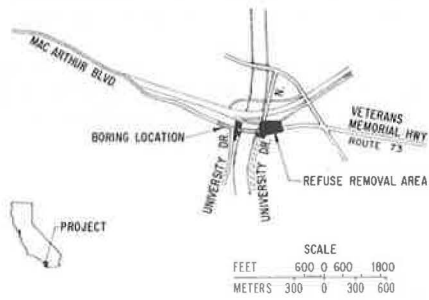
Figure 3. Cross section of engineered refuse fill and substrata.
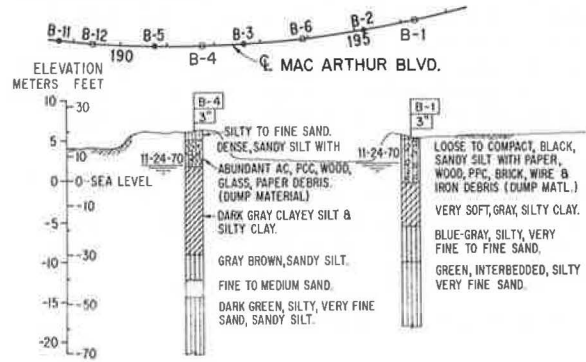
Figure 4. Sheepsfoot compactor processing refuse-soil layer.

Figure 5. Refuse rejected as unsuitable.

Figure 6. Bottom dump delivering load as rubber-tired bulldozer spreads soil over refuse.

Figure 7. Blended refuse layer exposed by 1.5-m-deep trench excavation.

Figure 8. Percentage reduction of maximum dynamic shear stress determined by finite element mesh.
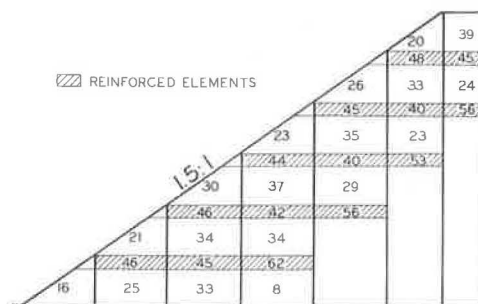
Figure 9. Nonbiodegradable waste project location map.

PROJECT LOCATION
(04-SCr-236, PM 7.26)
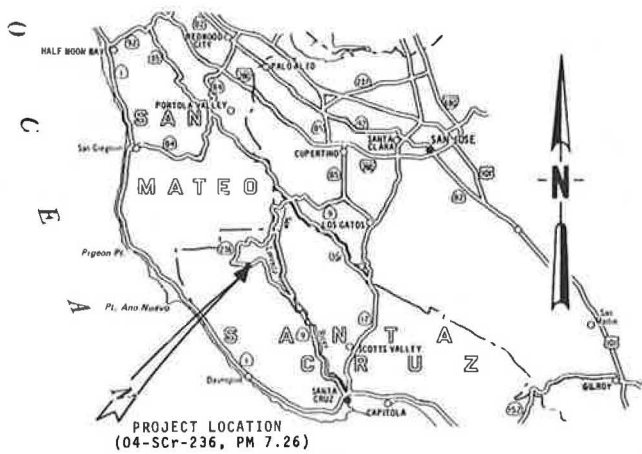
Figure 12. Theoretical peak connector strength versus required strength of 9.5-mm steel bar in cohesive and cohesionless soils for embankment heights to 18 m.
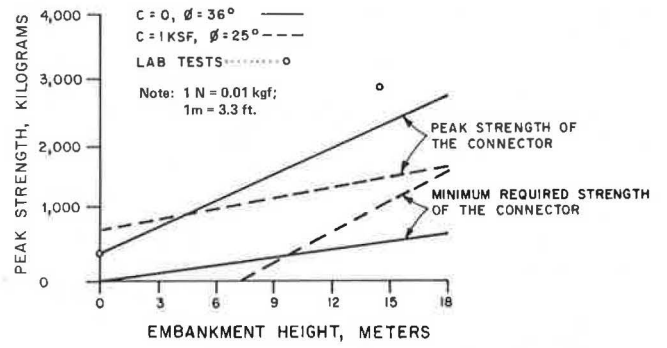
Figure 10. Cross section of tire reinforcement and instrumentation.

INSTRUMENTATION LEGEND

S	Slope Indicator
↔	Tire strain gage
⊥	Settlement platform
o	Pressure cell cluster
+	Extensometer
o	Line and elevation bench mark

Figure 11. Plan view of tire placement and tie arrangement.

TIRE CLIP

was subsurface water that, over a period of years, had saturated and weakened the earth mass supporting the roadway and ultimately had caused failure. The generally unfavorably bedded fractured planes were also a factor.

The experimental embankment is to be constructed on a side slope of $\frac{1}{2}$:1. It is estimated that the steepened slope, made possible by stabilization, will save approximately 68 810 m^3 (90 000 yd^3) of embankment that would have been necessary with the conventional $1\frac{1}{2}$:1 side slope because of the sloping nature of the terrain.

The essential elements of the test embankment include removal of the slide debris to well below the apparent slide plane, construction of a positive subsurface drainage system to relieve the cause of initial failure, and extensive instrumentation of the central section to monitor fill behavior (Figure 10). The outer 1.8 m (6 ft) of the embankment will be treated with straw. Seed and mulch will be air blown onto the face of the slope when construction is completed.

The tire sidewall mats will extend 10.2 to 15.2 cm (4 to 6 in) beyond the edge of the embankment to minimize erosion until permanent growth is established. The resulting artificial serrations should serve as energy dissipators for surface runoff. Mat embedment depth will be sufficient so that the reinforced portion of the embankment, if considered as a gravity system, will have sufficient mass to resist overturning and sliding.

In all large direct shear tests of the tire sidewall mats embedded in soil, the critical element was the connector rather than slippage between soil and tire mat or tensile failure of the tire sidewall. Initial consideration was given to the use of heavy (14-gauge), pneumatically fired staples. The results of pull tests using up to four such staples revealed an inconsistent performance, due primarily to the difficulty in obtaining consistently tight staple closure on the bottom side of the mat.

The clip type of connector that ultimately evolved is shown in Figure 11. Pull tests conducted on this type of connector, using 6.4, 7.9, and 9.5-mm-diameter ($\frac{1}{4}$, $\frac{5}{16}$, and $\frac{3}{8}$-in) cold rolled steel, revealed that the 9.5-mm-diameter ($\frac{3}{8}$-in) clip provided adequate tensile strength; this was true even when the estimated corrosion loss during the design life of the embankment was considered. Figure 12 shows the peak connector tensile strength versus strength required for embankment heights up to 18.3 m (60 ft) for the range of embankment soil shear strength properties anticipated on the project. The results of two actual laboratory tests are superimposed.

In addition to ease of installation, another important advantage of the clip connection is increased rigidity of the mat since the clips will grip the tire sidewall bead.

In April 1976, FHWA was requested to provide demonstration project funds to cover the costs of tire sidewalls, clips, and placement. This request was subsequently approved. Instrumentation analysis of data will be accomplished under the aforementioned ongoing HP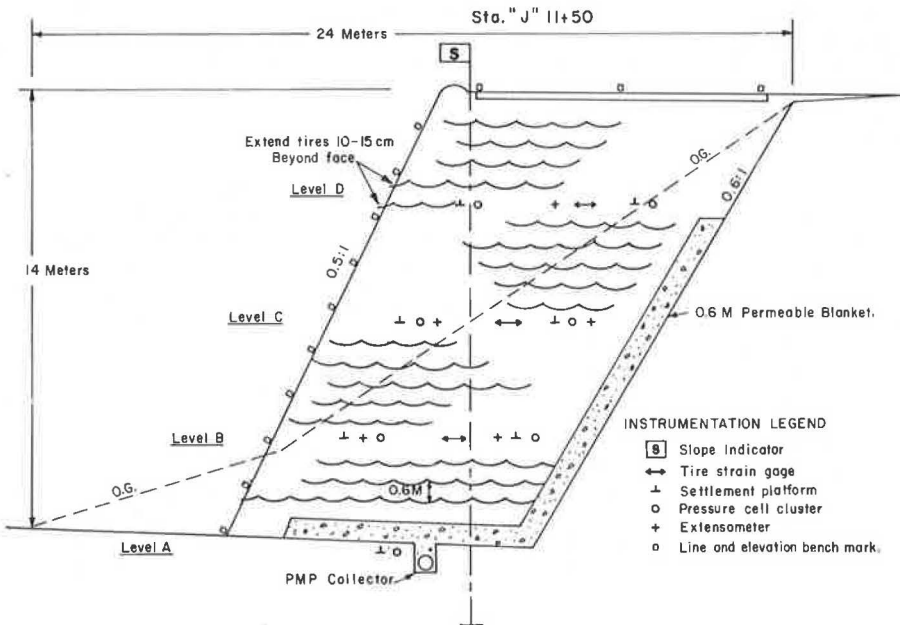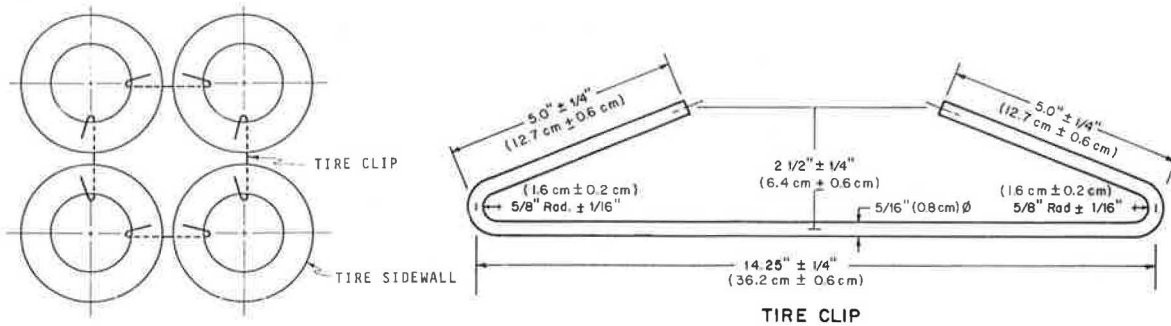R project. Project construction is expected to begin between August 15 and September 1, 1976, and to be completed within 1 month to 6 weeks. Instruments will be monitored for 2 years after construction.

SUMMARY AND CONCLUSIONS

Environmental constraints and economic considerations recently have necessitated a reevaluation of past highway practice with respect to inclusion of waste materials in embankments.

Experience with the Calif-73 project in Orange County thus far has demonstrated that satisfactory embankments can be constructed by using landfill waste. Whether landfill waste should be used must depend on an evaluation of engineering feasibility and aesthetics, based on availability of disposal sites, volume of landfill wastes, waste composition, state of waste decomposition, possible deleterious effect of the use of landfill waste on water quality, nature of embankment soil, and time constraints (effect of waiting periods).

A primary concern is the heterogeneous nature of the material. For obvious reasons, relative compaction cannot be used as a control test. Thus, the engineer must judge and supervise the operation and be prepared to make modifications to the character of the waste. Shear strength and consolidation characteristics, if necessary, must be determined by in situ testing. Instrumentation is of fundamental importance in controlling or modifying the operation.

Laboratory studies and dynamic response analysis have indicated that the systematic inclusion of certain nonbiodegradable wastes (tire sidewalls) could possibly benefit a fill and thus permit steeper side slopes and increase resistance to earthquake loading.

A test embankment to evaluate this premise is now planned for construction on Calif-236 in Santa Cruz County in early autumn 1976. It will be constructed at a $\frac{1}{2}$:1 side slope reinforced with tire sidewall mats at 0.6-m (2-ft) intervals. The performance of the completed embankment will be monitored by instrumentation installed during its construction.

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Waste Use in Highway Construction

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This paper discusses engineering and economic considerations of using solid waste in highway embankment construction. Both milled refuse and the residuals of various resource recovery systems are discussed as substitutes for costly highway building material. Included are an exploration of the problems and advantages of this concept and suggestions for further research.

In this paper, the following questions about the engineering and economic considerations of using solid waste in highway embankment construction are addressed:

1. Can refuse perform adequately as an embankment construction material?
2. Is the use of refuse economically justified when compared to the availability of other materials?
3. What role does resource recovery play in the economic and engineering evaluations?
4. What research activities should be undertaken to further investigate the use of refuse as an embankment construction material?

REFUSE

Disposal of waste materials has always been an unpleasant problem. From the distant to the recent past, the only concern has been disposal at the lowest possible cost. Concerned individuals have from time to time pointed out the damage done to air and water quality by open and sometimes burning dumps. Increased environmental awareness during the 1960s and 1970s, however, stimulated waste managers to clean up landfill operations and thereby decrease the objectionable effects of landfills.

Just as decisions were made and funds committed to improve the situation, shortages of every kind occurred and had a significant effect on waste management. Added to the costs of complying with new environmental regulations are increased fuel prices and skyrocketing land costs. Hence, the cost of getting garbage to landfills and disposing of it properly is increasing.

ROADS

Although the era of massive road-building projects is over, new highways to further serve transportation needs are continually being built. Many of these projects occur around large cities where the source of embankment material is usually far away and may also be in short supply. Therefore, obtaining and transporting this material back to the city are also becoming more and more expensive.

REFUSE ROADS

A reasonable course of action then would be to try to lower the total cost of each project by combining refuse and roads. The use of municipal refuse in local road embankment construction would unquestionably increase the life of the landfill dramatically and, therefore, reduce the cost of disposal.

Depending on the percentage of refuse by volume in an embankment, road-building costs would likewise be reduced. Of course, the first consideration in the use of refuse in embankments is its engineering suitability and not its economy.

ENGINEERING CAPABILITIES OF REFUSE AS CONSTRUCTION MATERIAL

Raw refuse directly after collection certainly is not a desirable building material. It is not uniform; therefore, it is difficult to handle and to compact. Larger particles tend to bridge when attempts are made to compact the material. When further loading is applied, this bridging effect causes uneven and unpredictable settlement. Organic fractions of the refuse are subject to decay, and this causes further settlement. As decay continues and water infiltrates, contaminating leachate is formed that pollutes ground and surface water. Flammable and noxious gases also build up in the fill and

present the possibility of explosion or health hazards.

For all of the seemingly insurmountable problems associated with using this material in embankment construction, it is surprising that refuse is currently being used successfully in a highway realignment project in California (1, 2), where garbage from an existing landfill, through which the highway must pass, is used in construction of the embankment. The refuse is laid down in thin lifts, mixed with dirt, and compacted. Then two lifts of traditional embankment material are laid down and compacted over the refuse. This sandwiching of garbage and traditional embankment material proceeds to within 1.2 m (4 ft) of the finished elevation.

The contractor on the job is saving money because the volume occupied by refuse carries only a transportation and placement cost. Traditional material, if it were not available from cuts, would have to be purchased as well. Logistically the project is a success; however, performance of the embankment has not yet been evaluated.

From an engineering standpoint at this time, only recommendations about methods of placement can be made. Extremely limited data have been collected on this subject. Even data on the settlement and load bearing performance of sanitary landfills are lacking because, until recently, there has been no need to use landfill sites for any type of construction and because few completed landfills have been operated according to current strict specifications. As a result, data from these covered dumps are useless (3, 4, 5).

Recommendations for placement of waste material in embankments are given below.

1. Because of the high probability of leaching and the unrealistic measures necessary to collect and treat leachate along a roadway, refuse embankments should only be constructed in areas where the water table is low and where the underlying soil is capable of naturally attenuating leachate to an acceptable condition before it reaches groundwater.

2. Care should be taken to ensure that daily refuse is covered with compacted material to prevent excessive infiltration of precipitation during construction. Familiarity with standard sanitary landfill operation procedures will be helpful when one is attempting to use refuse as a construction material.

3. Refuse should be milled or shredded before placement. Settlement will be unavoidable, but it can at least be predictably uniform. Densities of 830 to 950 kg/m^3 (1400 to 1600 lb/yd^3) may be expected. Milled refuse is inert; hence, it will not attract annoying and unsanitary flies and rodents during construction or after placement (6).

4. Refuse is extremely active biologically and chemically. Daily production should be compacted and covered. It will be impossible to stockpile refuse material for even short periods of time (days). Piles of refuse begin to decompose quickly and create a health hazard. This is a severe limitation in areas where seasonal inclement weather completely halts road construction activity.

5. Placement of refuse should occur in about 0.3-m (1-ft) compacted lifts, and suitable embankment material should be mixed into the refuse as it is compacted and covered by two layers of compacted earth material.

6. Refuse should be placed at least 1.2 m (4 ft) away from the profile of the embankment.

7. A period of settlement should be allowed before subsequent layers are placed.

In relation to recommendations 5, 6, and 7, specifications for the MacArthur Boulevard project in California are as follows (1):

1. Mix one part trash with five parts suitable material from excavation other than type A;

2. Cover each layer of mix with two layers of embankment material;

3. Do not use more than 15.2 cm (6 in) of biodegradable material;

4. Allow the first 2.7 m (9 ft) of fill to settle for 60 days;

5. Allow the second stage (also with a 60-day settlement) to go from 2.7 to 5.4 m (9 to 18 ft) high, but not at a rate of more than 0.4 m (1.3 ft) per week;

6. After the second stage, do not allow placement to exceed 0.9 m (3 ft) per week; and

7. At a bridge abutment excavation, allow the fill to settle for 365 days.

It is the position of this paper that refuse should not be used within 30.5 m (100 ft) of bridge abutments and then the refuse embankment should not end abruptly. Rather, over the next 30.5 to 61 m (100 to 200 ft) from the bridge abutment, the use of refuse should be decreased gradually to zero at 30.5 m (100 ft). These recommendations are general; however, until some practical experience is gained (as in the MacArthur Boulevard project) and more research is completed, general recommendations are the best that can be offered.

ECONOMICS OF PLACING MILLED REFUSE IN HIGHWAY EMBANKMENTS

In a typical situation today, a municipality pays for waste disposal services of the solid waste authority. Based on a city with a population of 300 000, refuse disposal costs are as follows ($1/Mg = $0.907/ton):

Item	Cost ($/Mg)
Milling	4.40
Transportation to landfill, assuming round-trip distance of 40 km (25 miles) from shredder station	2.63
Landfill operation	3.31
Total	10.34

A contractor enters the scene with a highway embankment project to supplement a completed municipal bypass. The average distance between the mill and the road project is the same as that between the shredder and the landfill.

A logical pattern to follow should the refuse be acceptable within the specifications of the highway project would be for the waste authority to deposit milled refuse at the road site rather than at the landfill. There would be no charge to the contractor for this material.

The city, at this point, would still be paying exactly the same fee per megagram to the waste authority, less landfilling costs. Net cost to the city would then be $7.04/Mg ($6.39/ton).

With this arrangement, each interest benefits. The contractor may avoid purchase and transportation costs of as much as 30 percent of the necessary fill material. The resultant lower cost per kilometer of highway is a direct benefit to the municipality. This is in addition to the aforementioned 30 percent reduction in total refuse disposal costs. The capacity of the landfill site is extended because it will be used primarily for bulky materials rejected from the mill and special industrial wastes.

Sanitary landfill sites are extremely difficult to find. When the property is purchased, extensive engineering studies are required for approval. Expensive site preparation is necessary before operations may begin. Hence it is in the owner's best interest to extend the life of an established site for as long as possible.

It is appropriate to mention at this time that, although shredding nearly doubles the cost of refuse disposal, the benefits and potential value of this process make the expenditure worthwhile; the small, even particle size makes it easy to handle with little blowing. Shredded refuse does not attract flies or rodents. With the advent of resource recovery, shredding is a necessary first step in all phases, such as in the use of refuse as a construction material.

A hidden cost in the shredding of gargage is the cost of transferring the garbage to the landfill because the shredder station also acts as a transfer station. In moderate to large cities, these facilities are an essential part of the waste disposal system.

IMPACT OF RESOURCE RECOVERY ON CONCEPT OF SOLID WASTE IN HIGHWAY EMBANKMENT CONSTRUCTION

Resource recovery involves the removal of specific materials from refuse. From an engineering standpoint, the residual product would generally be of higher quality as an embankment material but would have significantly less volume.

Until the recent shortages of fuel materials, resource recovery could not compete economically with landfills as a means for refuse disposal, nor were products of recovery competitive with products manufactured from virgin materials. However, as the prospect of long-term shortages increases and as the costs of finding and operating landfills go up, the value of recovered resources is quickly reaching a break-even point and will soon become profitable (7, 8, 9).

The cost of embankment construction was decreased by the use of refuse; therefore, the cost to the municipality was proportionately reduced. The city also had lowered the cost of refuse disposal by eliminating much of the landfill operational costs.

If a resource recovery plant is used, the most economical programs at this time are material recovery (fiber, metal, and glass) or pyrolysis to produce fuel oil.

Both systems reduce the volume of residual waste substantially. In the case of total materials recovery, in which a wet process is used, the residual product is a sludge that would be unsuitable for use in the embankment and would necessarily have to be disposed of in a sanitary landfill. The financial benefits of the materials recovery system would then have to be balanced with the losses, namely, landfill costs and a more expensive roadway.

Pyrolysis reduces the organic fraction of refuse to a No. 6 grade fuel oil and gases. The remaining char is a glassy aggregate called frit. The gases given off during the process are burned to provide energy for the plant, and the oil is sold commercially. The remaining char, although it has not been studied formally, would seem to be an adequate material for earth construction. Certainly it is superior to pure refuse that is subject to substantial settlement. The physical properties of this char are probably similar to bottom ash, which is a product of coal combustion. Bottom ash is used successfully in secondary and utility road construction (10, p. 108). Again, however, the financial rewards gained from the sale of oil would need to be compared with the greater expense of constructing embankments without the contribution of no-cost refuse.

An important property of these two residual materials is their biological inactivity. Unlike raw refuse, this material can be stored or stockpiled during periods of surges or when working conditions are impossible. Therefore, even if resource recovery can lower the total cost of refuse disposal, an alternative use for the refuse such as embankment fill may still be more economical. In such an application, refuse may be considered a resource in and of itself.

Other resource recovery processes include incineration. The residuals are fly ash, bottom ash, and glass and metals if they have not been removed. The bottom ash itself would suffice as an embankment material. Metal and glass included would contribute to volume and overall mechanical properties. Fly ash itself has been used as a component in the making of certain types of concrete.

Fuel recovery, similar to pyrolysis, segregates the refuse via air classification into fibrous energy-rich materials and heavy material, consisting of large amounts of glass and metal, that would be suitable embankment material with little differential settlement when placed properly.

Currently a severe problem with resource recovery is the unpredictability of markets. A municipality may invest heavily in a particular resource recovery concept based on temporarily high values of the product. Extended periods of low demand are not unusual, however, in current markets, and this is especially true of recycled resources. Generally, recycled materials, such as wood fiber, glass cullet, and scrap metal, can only be used in a small proportion to virgin materials. Therefore, when the demand for a finished product slips slightly, the effect on the recycled components is more severe (11).

CONCLUSION

The concept of combining refuse disposal and highway construction is attractive enough to merit further research. Under present circumstances, refuse in embankment construction would always be less expensive than any other material, simply because there is no charge for it. The only costs that may be incurred are temporary storage in a sanitary landfill situation and transportation costs greater than those already paid for by the municipality for transport of refuse to the landfill.

The following areas need to be investigated.

1. Completed refuse embankments should be researched to determine how total settlement is affected by lift thickness, mixtures of soil and refuse, settlement intervals, and total embankment thickness.
2. Leachate production should be monitored to determine whether the asphalt or concrete roadway is impervious to excessive infiltration of water.
3. Costs of spreading and compacting refuse material should be compared with the costs of preparing traditional road-building material. In this paper, these costs are assumed to be the same. This assumption should be investigated.

Resource recovery residues in many cases appear to have excellent road-building qualities. In view of the growing economic incentives for resource recovery, high-quality residue materials will be increasingly available. The disposal of these in road embankments would effectively complete the recycling process. Therefore, it would be wise to initiate research in the use of these materials.

Cost studies should be performed to answer the following questions:

1. Assuming that a municipality donates the refuse at a mill for no charge and likewise agrees to transport the material distances less than or equal to the landfill distance, for what additional distance would the cost of transport still be less than the cost of procuring conventional embankment material?

2. Assuming that a particular resource recovery system becomes a legitimate competitor for the refuse, what price would the contractor be willing to pay for the shredded refuse versus allowing it to be processed through resource recovery? What value should be placed on the residues of recovery as highway embankment materials?

The political implications of refuse embankments must also be considered in highway projects where environmental impact statements are required. It is already difficult enough to win approval for certain highway construction projects without the issue of an additional environmental threat. The possibility of a refuse embankment becoming a polluter could easily become a serious political issue. This consideration again stresses the importance of research in the area of leachate production in the embankment.

These are some of the areas that offer research possibilities for the future. However, until more data are evaluated, it would be unwise to embark on wide-scale refuse embankment construction. Performance data simply are not yet available to prove that refuse embankments provide a stable foundation for highways.

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Use of Ash in Embankment Construction

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This paper summarizes the annual quantities of fly ash, bottom ash, and boiler slag recently produced by major electric utilities in the United States and makes projections for future production. Similar data for England are also presented. Available data on the chemical, index, and engineering properties of these materials and on the use of fly ash, bottom ash, and boiler slag by the construction industry in the United States and England are presented. Since the use of fly ash as a highway embankment fill material in the United States appears to be extremely promising, the paper describes several projects in the United States, England, and Scotland in which large quantities of fly ash have been used successfully and economically.

The magnitude of global energy problems combined with dwindling natural resources has made it clear to a substantial and growing segment of the engineering community that the use of power industry by-product resources, such as fly ash, bottom ash, and boiler slag, must be substantially increased over the next several decades if we are to effectively resolve these serious problems. In fact, the need for developing new technologies of conservation and use becomes more apparent and urgent with each passing month, as we are confronted by the challenge of rapidly diminishing natural construction materials.

Fortunately, through the combined efforts of the National Ash Association, the Bureau of Mines, universities, private practitioners, and others, the technology for using fly ash, bottom ash, and boiler slag in highway construction is, by and large, available. Fly ash is currently being used as a fill material in the construction of highway embankments, and fly ash, bottom ash, and boiler slag are being used as fillers in asphalt pavements. In addition, fly ash is being used as a pozzolan in cement-aggregate or lime-aggregate base course construction, and bottom ash is used as the aggregate in many cases. The technology for the design and construction of cement-bound fly ash base courses was developed in Europe and is being further developed in the United States through a project funded by the National

Ash Association. Bottom ash and boiler slag are also being used as antiskid materials for roadways during winter.

A logical and potentially promising area for the large-volume use of fly ash is the construction of highway embankments. This paper describes several projects in which fly ash has been used as a fill material in highway embankment construction. In addition, quantities of ash collected and used both in the United States and England are summarized, and the chemical, index, and engineering properties of fly ash, bottom ash, and boiler slag are discussed.

COLLECTION AND USE

Figure 1 shows by state the approximate quantities of coal burned by major electric utilities during 1973. More than 85 percent of the coal is burned by utilities located east of the Mississippi River. Figure 2 shows the approximate quantities by region of coal burned and the amount of ash produced and used in the United States during 1973, and Figure 3 shows a compilation by year of coal consumption and ash production by electric utilities in the United States based on data from the National Ash Association (1) from actual data up to 1975 and projected for subsequent years.

Table 1 gives data compiled by the National Ash Association on the use of ash in the United States during 1974. Table 2 gives a comparison of ash collected and used in the United States from 1966 through 1974. These data indicate that ash production during this period increased by about 135 percent and that use increased by more than 180 percent. They also indicate that a substantial volume of ash must currently be placed in storage areas each year. For example, approximately 46 Tg (51 million tons) of ash went unused in 1974. Therefore, the stockpiles of these valuable by-product resources are growing throughout the United States and will continue to do so, especially with the recent order by the Federal Energy Administration to many utilities to convert from oil to coal. It seems logical, therefore, to direct our efforts toward increasing ash use in highway construction.

As given in Table 1, the U.S. highway construction industry in 1974 used about 2 Tg (2.2 million tons) of

14

Figure 1. Approximate quantities of coal burned by electric utilities by state.

NOTE:

NUMBERS ARE IN TERAGRAMS
I TERAGRAM = 1.1 MILLION TONS
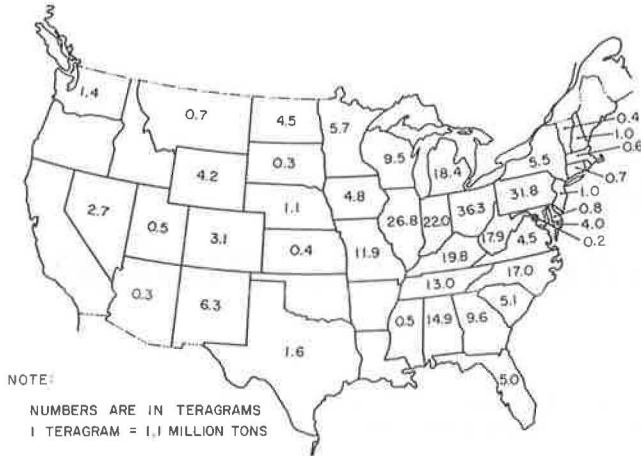
Figure 2. Approximate quantities of coal burned and ash produced and used by electric utilities by region.

NOTE:

NUMBERS ARE IN TERAGRAMS
I TERAGRAM = 1.1 MILLION TONS

C = COAL USED
A = ASH PRODUCED
U = ASH UTILIZED
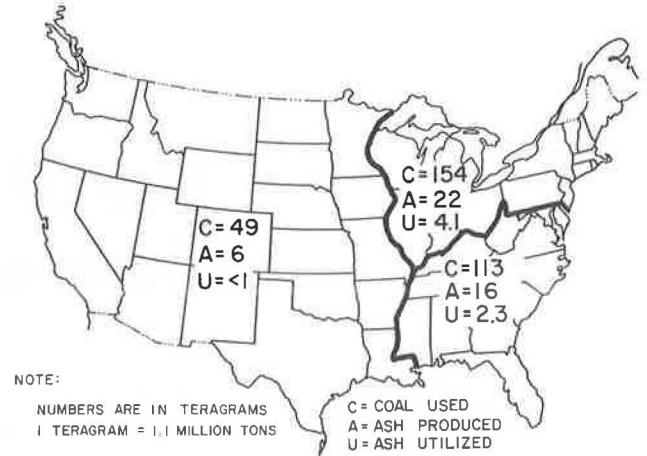
Figure 3. Coal consumption and ash production by U.S. electric utilities.
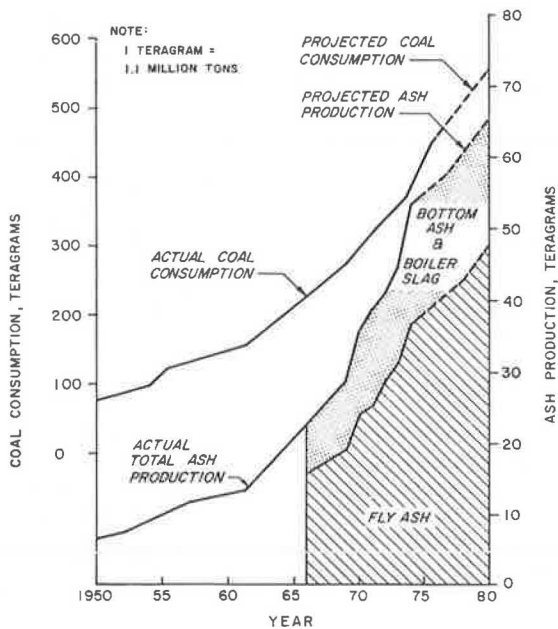
Table 1. Ash uses, ash removed from the plant site, and ash used from storage in the United States in 1974.

Item	Fly Ash (Tg)	Bottom Ash (Tg)	Boiler Slag[a] (Tg)	Total Ash (Tg)
Ash use				
In type 1-P cement ASTM 595-71 or mixed with raw materials before forming cement clinker	0.4	<0.1	—	0.4+
Partial replacement of cement in concrete or concrete products	0.5	—	—	0.5
Lightweight aggregate	0.1	0.1	—	0.2
Stabilization and roads	0.3	0.5	1.1	1.9
Filler in asphalt mix	0.1	<0.1	<0.1	0.1+
Miscellaneous	0.4	0.8	0.9	2.2
Total	1.8	1.5+	2.0+	5.3+
Ash removed from plant site at no cost to utility	0.4	0.5	0.1	1.0
Ash used from storage	0.9	0.6	0.1	1.6
Total use by weight				
Amount	3.1	2.6	2.2	7.9
Percent	8.5	20.0	50.0	14.6

Note: 1 Tg = 1.1 million tons.
[a]If separated from bottom ash.

Table 2. Ash collected and used in the United States from 1966 through 1974.

Year	Ash Collected (Tg) Fly Ash	Bottom Ash	Boiler Slag	Total	Ash Used Fly Ash Amount (Tg)	Percent	Bottom Ash Amount (Tg)	Percent	Boiler Slag Amount (Tg)	Percent	Total Amount (Tg)	Percent
1966[a]	15.5	7.4	—	22.9	1.3	8.4	1.5	20.3	—	—	2.8	12.3
1967	16.7	8.2	—	24.9	1.3	7.8	2.1	25.6	—	—	3.4	13.7
1968	18.0	6.6	2.4	27.0	1.7	9.4	1.6	24.2	1.4	58.3	4.7	17.4
1969	19.1	6.9	2.6	28.6	1.7	8.9	1.8	26.1	0.9	34.6	3.7	12.9
1970	24.0	9.0	2.5	35.5	2.0	8.3	1.6	17.8	1.0	39.4	4.6	13.0
1971	25.2	9.2	4.5	38.9	3.0	11.9	1.5	16.3	3.4	75.6	7.9	20.3
1972	28.8	9.7	3.5	42.0	3.3	11.5	2.4	24.7	1.2	34.3	6.9	16.4
1973	31.4	9.7	3.6	44.7	3.5	11.1	2.1	21.6	1.6	44.4	7.2	16.1
1974[b]	36.6	13.0	4.4	54.0	3.1	8.5	2.6	20.0	2.2	50.0	7.9	14.6

Note: 1 Tg = 1.1 million tons.
[a]First year for data.
[b]More comprehensive data collection program was developed, thus resulting in substantial increase over previous year.

ash, or approximately 25 percent of the total ash used. By contrast, as given in Table 3, the highway construction industry in England used about 1.6 Tg (1.8 million tons) in 1973-74 in roads and embankments alone, or 35 percent of the total ash used. In the past, this figure has been as high as 67 percent. Without a doubt, the highway construction industry is the number one potential bulk user of ash in the United States. To this end, the Federal Highway Administration is currently sponsoring two projects related to the use of ash in highway construction. Ohio State University was awarded a research contract to study power plant bottom ash in black base and bituminous surfacing. The objectives of the study are to investigate the feasibility of using bottom ash in bituminous mixtures and to develop technical data on the physical and engineering properties of the material.

GAI Consultants, Inc., was awarded the second research contract to study the use of fly ash as a construction material for highways. As with the Ohio State University study, the main objective of the GAI study is to develop a user's manual encouraging design and construction engineers to provide for the use of fly ash on highway projects. The manual is intended to supply data and information on the use of fly ash in the following applications: as a material for subbase and base course construction with cement or lime stabilization, as a soil stabilizing material, as embankment fill material, as a lightweight fill in slope stabilization projects, in grouting operations, and for structural backfill.

CHEMICAL PROPERTIES OF ASH

Fly Ash

Figure 4 shows the variations in chemical composition of fly ash produced in the United States (8). As shown, the principal constituents of fly ash are silica (SiO_2), alumina (Al_2O_3), and iron oxide (Fe_2O_3), and there are smaller amounts of calcium oxide (CaO), magnesium oxide (MgO), sulfur trioxide (SO_3), sodium oxide (Na_2O), and unburned carbon. The constituents most likely to affect the index and engineering properties of fly ash are free lime and unburned carbon. Free lime influences the pozzolanic activity of fly ash, and unburned carbon affects compaction and strength characteristics. The water soluble constituents are calcium and sulfur.

Bottom Ash and Boiler Slag

Figure 5 shows the variation in chemical composition of bottom ash and boiler slag (9). As with fly ash, the major constituents of bottom ash and boiler slag are silica, alumina, and iron oxide, and there are smaller quantities of calcium oxide, magnesium oxide, sodium oxide, potassium oxide (K_2O), sulfur trioxide, and other compounds.

INDEX PROPERTIES OF ASH

Fly Ash

Fly ash is characterized by low specific gravity, uniform gradation, and lack of plasticity. The specific gravity of fly ash particles varies with chemical composition. The results of 46 tests conducted on fly ash produced in western Pennsylvania indicate that the specific gravity can vary from about 2.3 to 2.6 and averages about 2.4. In contrast, the specific gravity of most soils ranges from about 2.6 to 2.8.

The range of grain-size distribution for fly ash is shown in Figure 6, which also indicates the relatively uniform grain-size distribution of fly ash compared with that of several types of soil. Because of its spherical shape, small surface area, and the uniform silt size of individual particles, fly ash has no plasticity.

Bottom Ash and Boiler Slag

Table 4 (2) gives the specific gravity of several samples of bottom ash and boiler slag. In general, boiler slag tends to have a higher specific gravity than bottom ash. Since the specific gravities of these ashes are a function of their chemical constituents, it is apparent that ashes with high iron contents (Fe_2O_3) have correspondingly high specific gravities.

Figure 7 (2) shows the range of grain-size distribution for bottom ash and boiler slag. These materials range in size from fine sand to fine gravel. The boiler slags, however, tend to be more uniform.

ENGINEERING PROPERTIES OF ASH

Fly Ash

Figure 8 shows the dry unit weight-water content relationships for seven compaction tests conducted in accordance with AASHTO T-180 (3). The shape of the compaction curves is generally similar to that obtained for cohesive soils. In other words, fly ash displays an optimum water content at which the greatest density is achieved for a given compaction energy in much the same manner as cohesive soils. The maximum density varied from a high of about 1426 kg/m^3 (89 lb/ft^3) at an optimum water content of 19 percent to a low of 1233 kg/m^3 (77 lb/ft^3) at 29 percent water content.

The results of six relative density tests (ASTM D 2049-69) on dry samples of fly ash are also plotted along the zero water content axis in Figure 8. The maximum relative densities vary from about 1250 to 1410 kg/m^3 (78 to 88 lb/ft^3). The corresponding minimum relative densities vary from approximately 961 to 1073 kg/m^3 (60 to 67 lb/ft^3).

The shear strength of fly ash depends on the degree of compaction. The results of direct shear tests and triaxial shear tests conducted on freshly compacted samples of fly ash are shown in Figure 9 (3) and indicate the general range of the relationship between the angle of internal friction and dry unit weight for fly ash produced in western Pennsylvania. These strength data indicate that a loosely placed, completely drained fly ash embankment can be safely constructed by using a fill slope of about 4 horizontal to 1 vertical; a well-compacted and drained fly ash embankment can be constructed at a 2 horizontal to 1 vertical slope. In addition, it has been shown that fly ash possesses significant cohesive strength because of capillary stresses in the pore water (3) and that the shear strength of fly ash can change significantly with time because of age hardening or pozzolanic behavior (4). Age hardening has been best correlated to the amount of free lime present in fly ash.

Fly ash behaves much like a cohesive soil in terms of consolidation; that is, on application of vertical pressure, the stress is initially shared by the soil structure and pore water. The excess pore water pressure gradually decreases as the water is squeezed out of the pores, and, as the pore water pressure decreases, the load is transferred to the fly ash structure; this produces a volume change. Laboratory consolidation tests have indicated that compaction can significantly reduce the compressibility of fly ash (3).

The coefficient of permeability for fly ash depends

Table 3. Ash collected and used in England from 1973 through 1974.

Item	Ash[a] (Tg)
Total ash collected	8.8
Ash use	
Cement manufacture, prekiln and postkiln	<0.1
Concrete	0.1
Concrete block	1.3
Lightweight aggregate	0.2
Grouting	0.1
Fill	
Roads and embankments	1.6
Building sites	0.9
Other	0.1
Total use	
Weight	4.3+
Percent	48.9

Note: 1 Tg = 1.1 million tons.

[a]From stations fired with pulverized fuel.

Figure 4. Variation in chemical constituents of fly ash in United States.
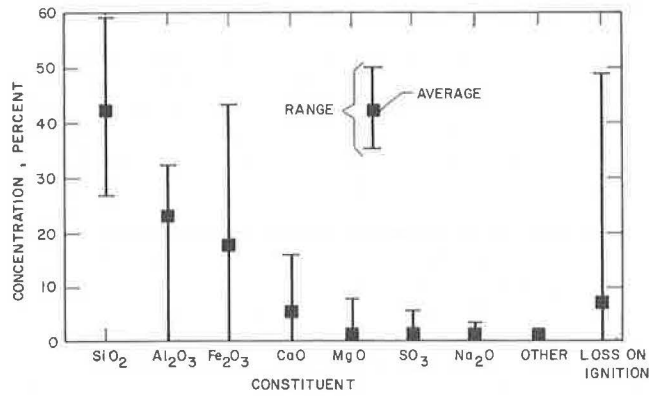
Figure 5. Variation in chemical constituents of bottom ash and boiler slag in United States.
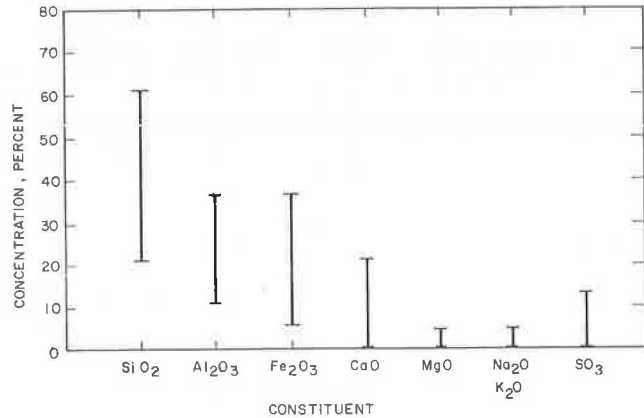
Table 4. Specific gravity of bottom ash and boiler slag.

Source	Bottom Ash	Boiler Slag	Specific Gravity
Fort Martin			
Unit 1	Dry bottom		2.35
Unit 2	Dry bottom		2.48
Kammer		Wet bottom	2.72
Kanawha River	Dry bottom		2.28
Mitchell	Dry bottom		2.78
Muskingham		Wet bottom	2.47
Willow Island		Wet bottom	2.61

Figure 6. Grain-size distributions for fly ash.

Note: 1 mm = 0.04 in.
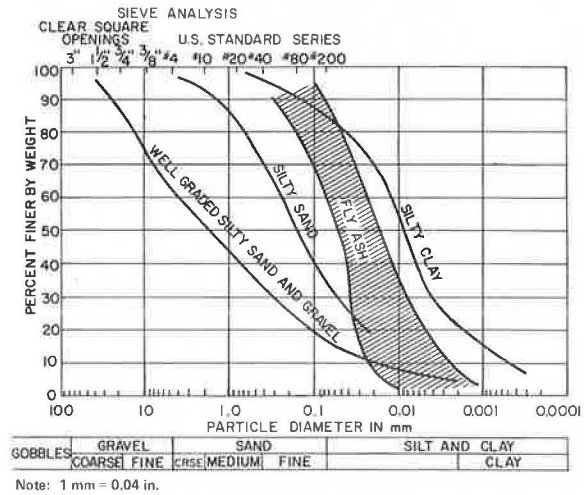
Figure 7. Grain-size distributions for bottom ash and boiler slag.

Note: 1 mm = 0.04 in.
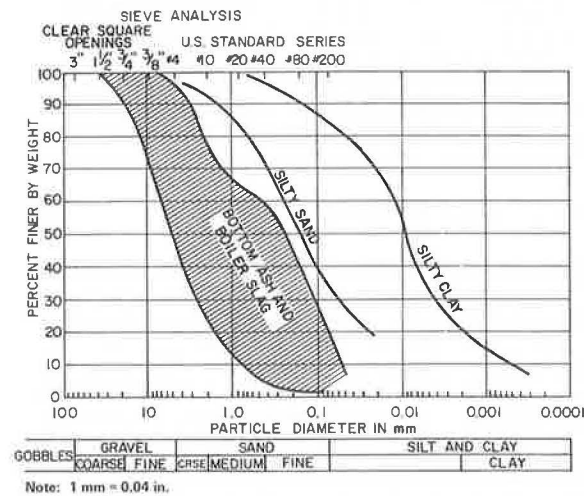
Figure 8. Results of laboratory compaction tests for fly ash.
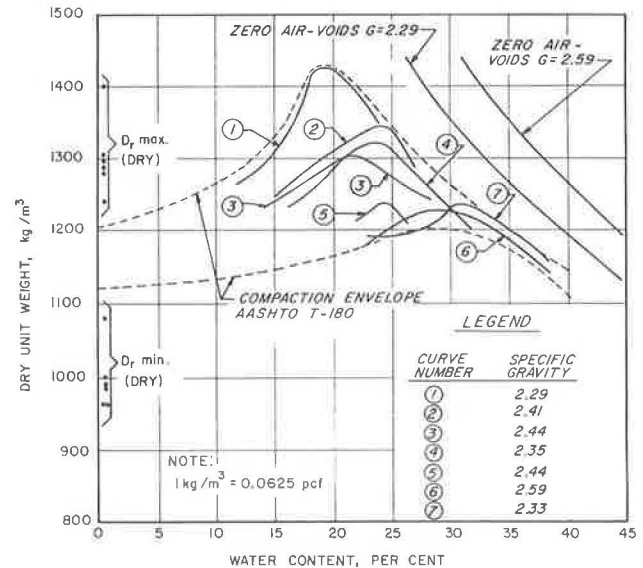
on its degree of compaction and the pozzolanic acitivity. The coefficient of permeability for fresh fly ash in the United States has been found to range from 1 to 5 μm/s (0.04 to 0.2 μin/s). The addition of 10 percent lime or cement to fly ash can reduce the coefficient of permeability by a factor as high as 10 (5).

Bottom Ash and Boiler Slag

Published data on the range of compaction, strength, and permeability properties to be expected for bottom ash and boiler slag are somewhat limited. The maximum and minimum relative densities for bottom ash were found to range from 1105 to 1860 kg/m³ (69 to 116 lb/ft³) and from 800 to 1458 kg/m³ (50 to 91 lb/ft³) respectively (2). The maximum and minimum relative densities for boiler slag were found to range from 1458 to 1762 kg/m³ (91 to 110 lb/ft³) and from 1137 to 1410 kg/m³ (71 to 88 lb/ft³) respectively (2).

As with fly ash, the shear strength of bottom ash and boiler slag varies with the degree of compaction. The angle of internal friction for bottom ash and boiler slag

Figure 9. Angle of internal friction versus dry unit weight for fly ash.

Figure 10. Results of field vibratory compaction tests for fly ash.

in a loose condition can vary from 38 to 42.5 deg and can average 41 deg. The coefficient of permeability has been shown to vary from 0.3 to 0.9 mm/s (0.012 to 0.035 in/s) (2).

ROADWAY EMBANKMENT PROJECTS IN UNITED STATES

Melvin E. Amstutz Expressway

The Melvin E. Amstutz Expressway project (6) involved the construction of a fill embankment for a four-lane highway divided by a 12.8-m-wide (42-ft) median between Grand and Greenwood avenues in Waukegan, Illinois. The contractor submitted a bid of $2.42/m³ ($1.85/yd³) for alternate D, the construction of a fly ash embankment. The alternate bids indicated that a fly ash embankment would realize a savings of approximately $62 000 over an earth fill embankment. The contract called for the placement and compaction of 188 845 m³ (247 000 yd³) of fly ash. The average height of the fly ash embankment was 1.1 m (3.5 ft), although 5.5 to 6.1-m-high (18 to 20-ft) embankments were constructed in ramp areas. The fly ash embankment was covered by 0.6 m (2 ft) of earth fill in the median area and by 2.4 m (8 ft) of earth fill on the outside of the fly ash slopes.

The fly ash was trucked to the site, either from closed dry storage silos or from stockpiles located outside the power plant, and placed in 15.2-cm (6-in) lifts. Each lift was compacted with a vibrostatic roller to a minimum density of 85 percent of the maximum laboratory density (AASHTO T-99). The maximum dry density for the fly ash used on the project was 1426 kg/m³ (89 lb/ft³) at an optimum water content of 25 percent. The contractor added water when necessary to obtain the required density.

Before the fly ash embankment was constructed, unsuitable in-place soils were removed and replaced with a granular fill to a level 0.6 m (2 ft) above the groundwater table.

Based on the experience gained through this project, the Illinois Department of Transportation (6) concluded:

All indications are that the fly ash embankment being placed on the project will provide a lightweight stable fill which will be stronger than most natural soils, due to the age hardening characteristics of the material. When the moisture content is controlled within a range of 20-25 percent, the material is workable and stable provided the proper construction methods are utilized. Excellent compaction and penetrometric results were obtained when the fly ash was compacted at a moisture content of 2-5 percent below optimum.

The contractor was able to work and place embankment on the job, on many days when other jobs with earth embankment could not be worked due to wet conditions. The lightweight material may find use in many areas where it is desirable to bridge weak soils. In order to utilize fly ash economically, an available source of sufficient quantity must be located close to the proposed job site.

US-250

A slide caused by poor drainage had occurred along US-250 (7). Engineers of the West Virginia Department of Highways decided to remove the slide mass, install subsurface drainage, and replace the slide material with compacted fly ash. The fly ash was hauled to the site in open trucks; no dusting problems were encountered in the the operation. The fly ash was tailgated by the trucks and spread into 23-cm-thick (9-in) lifts by a road grader. Each lift was compacted by a rubber-tired vibratory roller to a density equal to or greater than 98 percent of the maximum laboratory density (AASHTO T-99).
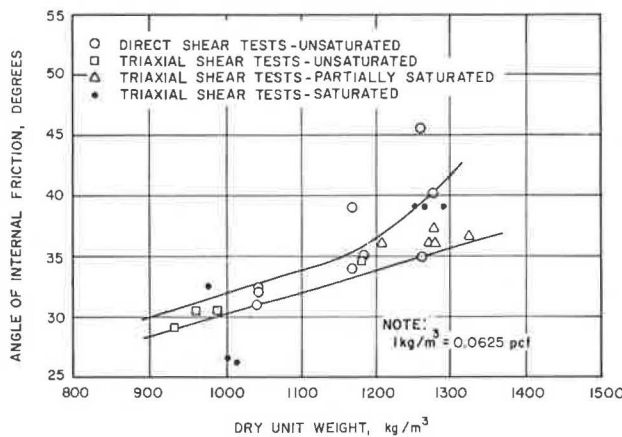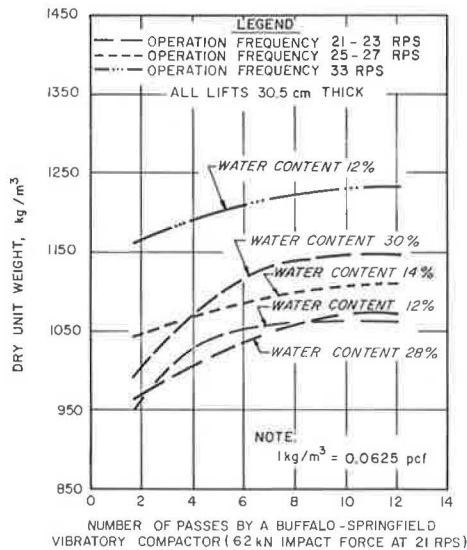
Fly Ash Storage Area Access Road

An eastern utility decided to construct an access road into its fly ash storage site by using compacted fly ash and bottom ash. Before any fill was placed, the access road embankment site was cleared and stripped of all unsuitable material. A 0.9-m-thick (3-ft) coarse bottom ash drainage blanket about 30.5 m (100 ft) wide was placed beneath the toe of the downstream slope, and a 1.8-m (72-in) corrugated metal drainage conduit with concrete head walls was placed in the valley to carry the small stream under the embankment. The access road embankment had a maximum height of 23 m (75 ft) and upstream and downstream slopes of 3 horizontal to 1 vertical and was covered with about 0.3 m (1 ft) of topsoil.

To determine the behavior of fly ash under vibratory field compaction, several series of tests were conducted on 30.5-cm-thick (12-in) lifts compacted with a vibratory compactor. The static weight of the compactor was about 5900 kg (6.5 tons), and the dynamic force was about about 62.3 kN (7 tons) per impact at 21 revolutions/s. Typical results of the test program are shown in Figure 10 (3), which is a plot of the compacted dry unit weight versus the number of passes by the roller. Each curve in Figure 10 indicates the results of successive density tests on the same lift after various numbers of roller passes. Each series of tests (each curve in Figure 10) was conducted at the same water content and vibratory frequency as indicated in the figure. The results show that the density of fly ash increases rapidly for the first five or six passes of the compactor but increases little after eight passes. Neither the maximum density achieved nor the rate of density increase with number of passes correlates well with the water content of the fly ash. Although this lack of dependence on water content is an important and beneficial feature of vibratory compaction, since it can be difficult and costly to control the water content of fly ash, the maximum densities achieved in the field were generally less than those achieved in the laboratory. The laboratory compaction characteristics of the fly ash used on the project are similar to those shown by curves 3 and 4 in Figure 8.

The higher vibrating frequencies were more effective in compacting fly ash. It was not determined whether frequencies higher than the maximum value of 33 revolutions/s used in this series would be more or less effective.

ROADWAY EMBANKMENT PROJECTS IN ENGLAND AND SCOTLAND

Production of fly ash in England for 1973-74 was about 8.8 Tg (9.7 million tons) (Figure 3). The Central Electricity Generating Board (CEGB), which designs, builds, and operates power stations in England, faced a tremendous disposal problem because of limited land area. Consequently, the CEGB embarked on a program of developing commercial outlets for large volumes of fly ash to appreciably reduce the disposal costs and help preserve the environment.

As indicated in Table 3, the main use of fly ash in England has been as fill on road construction projects and building sites. A short summary of some roadway embankment projects undertaken in England and Scotland follows.

Motorway M9

In February 1966, construction began on a fly ash embankment for a portion of Motorway M9 between Edinburgh and Sterling, Scotland, at the Earlsgate Interchange.

The alluvium beneath the proposed route was as much as 39.6 m (130 ft) thick and very compressible, and this made construction of an earth fill embankment difficult because of stability and settlement problems. The maximum height of the embankment was 7.9 m (26 ft).

Construction began with the removal of unsuitable materials and the placement of a 1.07-m-thick (3.5-ft) gravel drainage blanket. The fly ash was placed in thin lifts and compacted to 95 percent of the maximum laboratory density determined by British Standard Test Methods, with a total of 0.54 Tg (0.6 million tons) of fly ash being used on the project. The embankment was constructed with side slopes of 2 horizontal to 1 vertical and covered with 15 cm (6 in) of top soil.

Alexandria Bypass

This truck road diversion was built in two stages and bypasses Alexandria and three other towns situated in the valley of the River Leven in Dumbartonshire, Scotland. The 9.2-km-long (5-mile) diversion provides access from the heavily industrialized area of the River Clyde into the rural highlands and the shores of Loch Lomond.

The first stage of the project consisted of crossing the floodplain of the River Leven, the outlet from Loch Lomond. A high approach embankment was required on both sides of the river so that navigational clearance on the river could be maintained. Poor subsoil conditions consisting of saturated silt dictated that a lightweight fill for the embankment and more than 0.41 Tg (0.45 million tons) of fly ash be used for this stage. Even with the use of the fly ash, a settlement of more than 0.9 m (3 ft) has been recorded near the river, most of it having occurred during construction.

Construction of the second stage began in 1973, and started near the edge of the floodplain and extended along the hillside above Alexandria. The high embankment begun in the first stage had to be continued onto the rising ground, this time to a bridge over an electric railway and a local road. An additional 54 Gg (60 000 tons) of fly ash were used for this purpose since the subsoils consisted of the same weak silts encountered in the first stage. Two years after construction, the 11-m-high (36-ft) embankment had settled only 25 cm (10 in). Working in a rather confined area, the contractor used a D6 bulldozer for spreading the fly ash and a D4 bulldozer towing a 3600-kg (4-ton) vibrating roller for compaction and successfully placed 1.1 Gg (1 200 tons) of fly ash per day at an average dry density of 1121 kg/m^3 (70 lb/ft^3).

Motorway M5

The 7.2-km-long (4.5-mile) Avonmouth section of Motorway M5 starts at the southern end of Filton Bypass and ends at the Portway between Bristol and Avonmouth, England. The western 3.2 km (2 miles) of roadway are built on the plain of compressible alluvium with some peat layers. This portion of the roadway is built on an embankment about 2.1 m (7 ft) above the original ground. High 6.1-m (20-ft) embankments above the motorway carry Lawrence Weston Road, Kings Weston Lane, and the link road over it. High embankments also carry the motorway at its western end on the approach to Avonmouth Bridge. The motorway has dual 11.0-m-wide (36-ft) pavements and 3.05-m-wide (10-ft) shoulders; the overall width at the top of the embankment is 39.4 m (129 ft).

A trial fly ash fill embankment was constructed in August 1965. The embankment was planned to be 9.1 m (30 ft) high and 30.5 m (100 ft) square at the top and had side slopes of 2 horizontal to 1 vertical. The base of the

embankment was a 46-cm-thick (18-in) layer of sand to allow for the relief of pore water pressures in the subsoils. The building of the trial embankment proved that a high quality of motoring could be achieved after the subsoils were temporarily surcharged to accelerate the dissipation of pore water pressure and strengthen the subsoils.

SUMMARY

The use of power industry by-product resources such as fly ash, bottom ash, and boiler slag in highway construction can solve two significant problems: (a) in the highway construction industry, rapidly diminishing natural construction materials, and (b) in the power industry, ever-increasing quantities of fly ash, bottom ash, and boiler slag requiring disposal. As indicated by the highway embankment projects described in this paper, the technology for using fly ash as a fill material is well developed. In addition, the technologies for using all of these materials in many other aspects of highway construction are also available. It is clear that the time has come for engineers in both industries to direct their efforts toward solving each others' problem.

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Construction of Fly Ash Roadway Embankment in Illinois

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Electrostatically precipitated fly ash is a readily available source of structural fill in many metropolitan areas of the United States. A site was chosen by the Illinois Division of Highways near a large power plant for both a trial embankment and a highway embankment constructed with fly ash. A methodology for construction of fly ash embankments was developed that included spreading fly ash in thin lifts, scarification, and compaction by a vibratory roller. A clay seal was used to aid in confining the fly ash during construction and to provide a means of supporting vegetation. The performance of this roadway embankment has been satisfactory in respect to both structural stability and aesthetics.

Much fly ash is produced annually in the United States, as elsewhere in the world. Disposal of this fly ash not only has become an increasing burden on the producer of the fly ash (primarily, the power industry) but also results in a cost ultimately transferred to the power consumer. Superficially, fly ash may be considered as the waste product of a procedure necessary to reduce pollutants in the air and thus help to preserve our environment and as a waste product by its origin suspect and deserving of no more than careful disposal. However, the transportation engineer may, when faced with increasing shortages of building materials, choose to explore and evaluate the potential of fly ash as an engineering asset.

In urban areas particularly, earth borrow is often in scarce supply and available only at premium cost. Often these same urban areas are served by large power-generating stations that consume great quantities of coal and consequently produce great quantities of fly ash. A favorable combination of circumstances is created, provided the technology is available, for the use of fly ash in structural fill.

CONSTRUCTION OF TEST EMBANKMENT

In spring 1965, the Illinois Division of Highways, in cooperation with the Commonwealth Edison Company and the Chicago Fly Ash Company, planned and supervised the construction of a trial fly ash embankment on a site adjacent to an electrical generating plant at Waukegan, Illinois.

The trial embankment was 61 m (200 ft) long, 12 m (40 ft) wide, and 1.8 m (6 ft) high and was constructed primarily to develop a construction methodology for future use of electrostatically precipitated fly ash in state projects.

The fly ash used consisted of two types: a stockpiled fly ash (approximately 60 days old) used in the lower 1.2 m (4 ft) of embankment, and a considerably younger silo-stored fly ash used for the upper 0.6 m (2 ft).

Initial procedures consisted of dumping the fly ash in 15.2-cm (6-in) loose lifts from tandem dump trucks, grader spreading, and compacting by means of a 9072-kg (10-ton) self-propelled pneumatic roller. Because of the formation of a crust and the presence of large lumps of hardened fly ash from the stockpiles, compaction of the full lift depth was apparently not possible. The value of further efforts, with disking and sheepsfoot tamping roller compaction, also proved to be dubious because the tamping roller tore the surface and did not provide any substantial increase in compaction.

At this point, procedures were altered to include rotary tilling, scarification, and six to eight passes with the pneumatic-tired roller. This sequence of operations proved successful in obtaining at least 85 percent relative compaction (AASHTO T-99) and was continued to completion of the embankment.

The effect of compaction at moisture contents both above and below optimum moisture (±25 percent), as determined by AASHTO T-99, method C, was also evaluated. Attempts at significantly low moisture contents (optimum ± 13 percent) were totally ineffective; the roller lost mobility, and dusting was a severe problem to personnel and equipment. The most effective range of compaction moistures was from optimum + 8 percent to optimum + 4 percent; this provided compaction in a narrow range

of from 85 to 88 percent. Other observations included the tendency of the fly ash to harden with age, that is, gain strength with time; a relatively high apparent permeability in respect to other similarly graded natural soils; and the inability of fly ash to support vegetation.

ENGINEERING PROPERTIES OF FLY ASH

There are several significant properties of fly ash that must be considered when it is used in structural fills or roadways. Individual fly ash particles are spherical in shape, generally solid, though sometimes hollow. In bulk quantities, fly ash possesses a silty texture, a specific gravity somewhat less than that of naturally occurring soils, and no plasticity.

Fly ash exhibits shear characteristics similar to those of a cohesionless soil, a significant undrained angle of internal friction (+25 deg), and a minimal cohesion intercept in a dry condition (1). A latent strength development or age-hardening process does often occur because of inherent pozzolanic activity within the fly ash.

The gradation of the fly ash used in this project is as follows: 19 percent sand, 71 percent silt, and 10 percent clay. It is nonplastic, has a maximum density of 1362 to 1474 kg/m^3 (85 to 92 lb/ft^3), and has an optimum moisture of from 20 to 26 percent. The California bearing ratio (CBR) for three conditions of fly ash is as follows: 20 percent for unsoaked, 2 percent for soaked, and 5 percent for swell. In that the CBR seems to be reduced as the fly ash swells when it is soaked, zones known to become saturated would most likely not be considered for placement of fly ash.

CHEMICAL PROPERTIES OF FLY ASH

The primary constituents of the fly ash used on this project were oxides of silica (45 percent), iron (19 percent), and lesser percentages each of aluminium, sodium, calcium, magnesium, and potassium. The presence of variable quantities (5 to 7 percent) of unburned carbon in the fly ash was noted throughout construction, and on at least one occasion an even higher carbon content was suspected because of the black color, and the fly ash was rejected.

The combination of available lime (CaO, MgO), ash pozzolans, an alkaline environment, and moisture may produce the apparent age hardening of the fly ash. The fly ash here had a pH of 10 and sufficient available lime to appear on the surface after drying. Recent use of certain western coals (Wyoming and Montana) in Illinois has produced fly ash with accelerated pozzolanic activity, apparently because of the higher natural lime content of the coal.

CONSTRUCTION OF HIGHWAY EMBANKMENT

In 1972, a contract was let by the Illinois Division of Highways to construct a major highway improvement between Grand and Greenwood avenues in Waukegan, 64 km (40 miles) north of Chicago. The portion of improvement covered here is now known as the Melvin E. Amstutz Expressway.

The proposal for the project included 2.33 km (1.45 miles) of four-lane concrete pavement with concrete median, the 188 080-m^3 (246 000-yd^3) embankment for which was to be constructed from one of the alternate materials, including fly ash.

The fly ash construction alternate was chosen by the contractor because a nearby power plant offered an available source of material, at a cost savings. The types of fly ash available were the same as those used in the test embankment phase, i.e., stockpiled and silo-stored material.

The height of the embankment was nominally 1 m (3.3 ft), except in the ramp areas at Grand and Greenwood avenues, where fills in excess of 8.5 m (28 ft) were constructed.

Site Conditions

The site conditions consisted of approximately 1980 m (6500 ft) of swampy vegetation to be removed to an average depth of ±0.6 m (±2 ft) underlain by clean sand (Figure 1). The removal areas were then to be backfilled to an elevation of 0.6 m (2 ft) above the mean water table with a porous granular backfill. The mean water table at this location is controlled entirely by the adjacent Lake Michigan.

Fly Ash Handling

Particular problems were encountered in handling the drier (±8 percent moisture), silo-stored material. As when used previously, this silo-stored material created a severe dusting problem during dumping and manipulation and required large quantities of water to bring it to compaction moisture, especially on hot, windy days.

When the stockpiled material arrived at the job site, its moisture content ranged from 12 to 40 percent; consequently, the material presented fewer problems in respect to blowing and compaction. Exceedingly large lumps of hardened fly ash from the stockpile were routinely broken up at the loading site by an end loader (Figure 2).

Manipulation of the fly ash after it was spread on grade by trucks (Figure 3) consisted of blading the fly ash in 15-cm (6-in) lifts (Figure 4), water spreading it if necessary, and disking it to achieve uniformity of moisture content and reduce the effects of lensing and crusting (Figure 5).

Lensing (the formation of small, shallow, transverse shear cracks) and crusting of the surface are considered detrimental, and disking or tilling through the loose lift into the surface of the preceding compacted lift is seen as a practical means of eliminating any adverse effects of lensing.

Field Compaction

The contractor was not bound to a particular means of compaction but was given some degree of flexibility, provided that a relative compaction of 85 percent (AASHTO T-99, method C) was attained on each lift. He chose to use a vibratory, single steel drum, 9072-kg (10-ton) roller (Figure 6). This roller performed adequately and often attained compaction well in excess of 85 percent with as few as two passes. Previous findings about the required range of compaction moisture were somewhat verified, although the response of stability and density at moisture contents in excess of optimum appeared to be much more sensitive with vibratory compaction than with pneumatic-tired roller compaction.

A 2.4-m (8-ft) envelope of cohesive soil was placed on the outside of the embankment to serve as an erosion protection device and to provide a means of vegetation support (Figure 7). This envelope also provided the secondary benefit of containing the outside edges of the embankment, thus allowing complete compaction at this point.
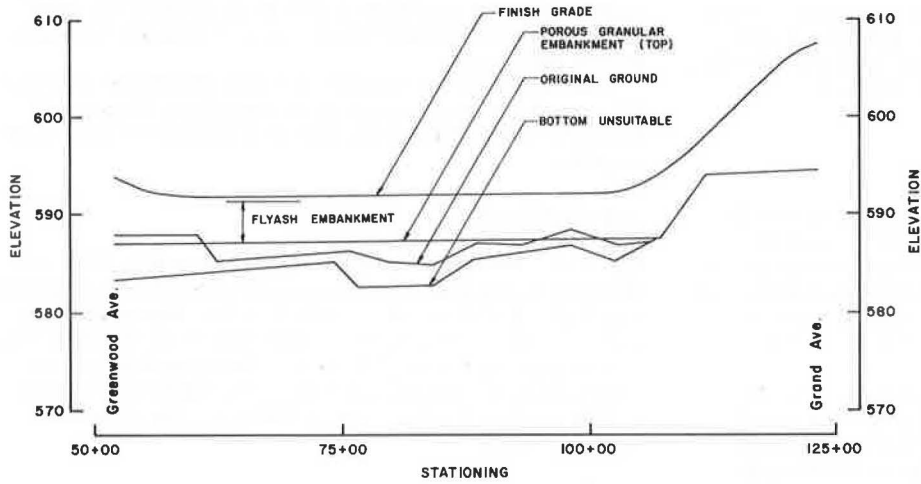
Figure 1. Roadway profile.

Figure 2. End loader breaking up hardened fly ash from stockpile.

Figure 3. Spreading fly ash on grade.
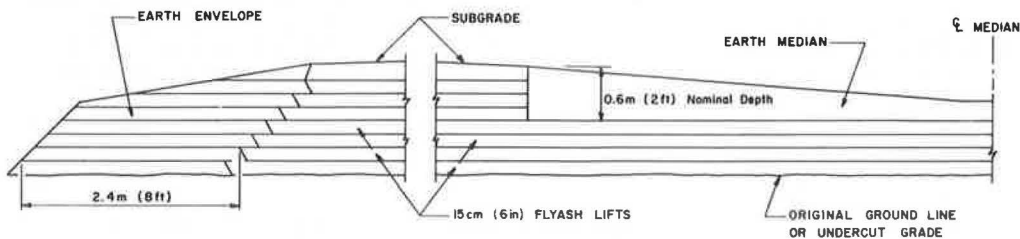

Figure 4. Blading fly ash in lifts.

Figure 5. Disking fly ash.

Figure 6. Single steel drum roller.

Figure 7. Section of fly ash embankment.

Other Observations

Because of an apparent high permeability and often a high demand for water when fly ash is compacted, contractor operations were often unimpeded by moderate rainfall. Even after periods of heavy rain, work could resume much sooner than would be possible with more cohesive soils.

Freezing occurred to a depth of 45.7 cm (18 in), less than one-half of the 107 cm (42 in) normally expected in this area. During the thawing period, instability of this zone was evident because of the formation of ice lenses that prevented the downward flow of water. However, when thawing was complete, suitable stability was restored in 2 to 3 days.

Erosion of exposed fly ash was severe because channels several meters deep were developed during heavy storms on unprotected slopes. After periods of heavy rainfall and on drying, free lime appeared on the surface, accompanied by the development of longitudinal tension cracks in roller wheel tracks.

There is no evidence at this time of slope or subgrade instability. Slopes as steep as 2:1 are performing satisfactorily. Instrumentation has been installed, in both the high embankment areas and subgrades, to measure settlement, frost depth, and capillary rise. Because of the recent mild winter in this area and a relatively short period of observation, no significant trends have been observed.

CONCLUSIONS AND
RECOMMENDATIONS

1. Electrically precipitated fly ash is an acceptable alternative to naturally occurring soils in both subgrades and fills above the water table.

2. Methods of construction control, such as compaction in thin lifts, scarification of the preceding lift surface, and compaction to at least 85 percent relative compaction, may be essentially those used for natural soils.

3. Compaction characteristics of fly ash are similar to those of noncohesive soils and are more responsive to vibration than to kneading or tamping.

4. Variations in standard densities of fly ash should be expected and provided for in control procedures.

5. Provisions must be made in contracts for the use of large quantities of water, as a means of controlling dusting and obtaining compaction.

6. Environmental hazards, real or purely speculative, must be solved or fly ash use may never reach its full potential. Dusting is a real problem, particularly to contractor's equipment. Groundwater pollution by fly ash is currently in the speculative category, and its significance can only be assessed by realistic laboratory and field measurements on a case-by-case basis.

ACKNOWLEDGMENT

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Part 2
Erosion Control, Stabilization, and Compaction Control

Improvement of Dispersive Clay Erosion Resistance by Chemical Treatment

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Erosion damage caused by dispersive clay soils has created many problems in compacted earth structures and natural cut slopes. Although methods exist for dispersive soil identification, there has been only limited investigation of chemical treatment to improve erosion resistance of these soils. Thus, chemical stabilization of such soils was investigated by using mixtures of hydrated lime, sodium chloride, and aluminum sulfate, which were representative of the calcium, sodium, and aluminum ion groups. This paper describes the effect of chemical treatment on the dispersion potential and erosion resistance of six Oklahoma soils. After a review of literature concerning methods of dispersive clay soil identification and chemical reactions that occur during treatment, experimental test results are presented and discussed. Test results further support the physical erosion test as an adequate means of correctly evaluating the erosion potential of dispersive clay soils. The use of soil engineering properties to determine dispersive erosion potential was evaluated and found to be unsatisfactory. Flocculation through chemical stabilization was an alternative for reducing or eliminating dispersive clay erosion problems. All three chemical stabilizers provided successful treatment and are ranked in order of decreasing effectiveness.

Damage to earth embankments and cut slopes from dispersive clay soil erosion is of increasing interest to geotechnical engineers. A dispersive clay soil erodes in the presence of flowing water, when the particle-to-particle repulsive forces exceed the van der Waals attractive forces. Thus, individual clay platelets are split off (dispersed) and carried away. Such erosion may start in a drying crack, settlement crack, or other channel of high permeability in a soil mass. When water flows through the crack and the clay is eroded at a faster rate than it can swell and close the crack, erosion damage occurs.

Current procedures for identification of potentially dispersive clay soils include several methods of finding their dispersion potential. Although the problem of identifying potentially dispersive soils has received considerable attention and successful identification techniques have been developed, modification of such soils to improve their erosion resistance has received little

attention.

The School of Civil Engineering at Oklahoma State University, Stillwater, has been engaged in a study to determine the effects of chemical treatment on the dispersion potential of cohesive soils. The project was sponsored by the Bureau of Reclamation, U.S. Department of the Interior. This paper presents results obtained during this study, in which hydrated lime, sodium chloride, and aluminum sulfate were used as chemical stabilizers. In addition to discussions of test results, a review of the basic concepts and theories used in the identification and treatment of dispersive clay soils as well as present theories and hypotheses concerning effects of chemical treatment are presented.

LITERATURE REVIEW

Identification of Dispersive Clay Soils

The first widely accepted method of identification of a potentially dispersive clay soil was proposed by Volk (1). His procedure, based on results of tests conducted on dispersive soils from Arizona, involved both qualitative and quantitative measurements of the dispersion potential of a clay soil. Fletcher and Carroll (2), while conducting tests on similar soils, discovered that the soils contained extremely high exchangeable sodium and calcium carbonate. Further research indicated that the dispersion of the clay fraction of dispersive soils resulted from the high percentage of sodium ions surrounding clay particles, relative to other cations (3). Several other researchers investigated the relationship between the concentration of sodium ions and other exchangeable cations present (4, 5, 6, 7). These researchers felt that the relationship involving sodium versus other cations present could be used to predict field behavior.

In 1967, Emerson (8) proposed a test to determine the degree of dispersion based on the observed reaction of soil aggregates immersed in water. The rapid dispersion test or crumb test, which was introduced by Sherard (9) in 1972, was a refinement of the coherence test presented by Emerson. Sherard believed his test was more closely associated with expected field behavior.

A slight modification of the test proposed by Volk is

the laboratory dispersion test (LDT), which has been adopted by the U.S. Department of Agriculture Soil Conservation Service. The LDT is currently the most widely used method of dispersive clay identification. Based on the LDT, the degree of dispersion is classified according to expected field behavior ranging from nondispersive to highly dispersive.

After examining previously proposed tests, the authors developed a physical erosion test (PET), to better simulate dispersive soil erosion under field placement conditions (10, 11).

Chemical Treatment

Previous researchers have demonstrated that the difference between a dispersive and a nondispersive soil results from the relative concentration of sodium compared with other exchangeable cations on or surrounding the clay particles. Since the differences are chemical, chemical stabilization might be a solution to the dispersive clay problem. The first chemical treatment considered is the addition of lime (calcium oxide or hydroxide) and its effect on cohesive soils. The effect of lime, when added in the small amounts used in this research study, was to partially modify the soil rather than to stabilize it (12). The addition of lime usually results in the following changes: increase in plastic limit, decrease in plasticity, decrease in number of clay-sized particles, decrease in maximum compacted density, increase in optimum moisture content, decrease in volume change potential, and increase in shrinkage limit (13, 14, 15, 16).

Mechanisms usually presented in literature to explain the changes in physical properties from lime treatment are cation exchange, flocculation, carbonation, and pozzolanic reactions. The last two mechanisms are believed to be long-term effects that control strength gain from addition of lime. Cation exchange and flocculation, which should control soil dispersion potential, have been found to increase with increasing lime content (17, 18, 19, 20). Cation exchange and flocculation are also produced by chemical treatment with sodium chloride and aluminum sulfate.

In the United States, sodium chloride has been widely used in road construction (21, 22, 23). Several changes in the physical properties of soils and soil-aggregate mixtures are thought to occur from sodium chloride treatment (24). The following changes are generally accepted: reduction in volumetric shrinkage, reduction in expansion, decrease in permeability, increase in strength, and increase in compacted density with lower optimum moisture content. The flocculation and exchange capacity of clay soils for sodium ions depends on two factors: original ions associated with the clay mineral and concentration of sodium ions in solution (25, 26).

Use of aluminum sulfate as a chemical stabilizer has received limited attention. To some extent, aluminum sulfate has been used with phosphoric acid to provide metallic ions in soil stabilization (27). The use of aluminum sulfate as a chemical additive was investigated more fully in this study.

MATERIALS, EQUIPMENT, AND SAMPLE PREPARATION

Materials

Materials used in this study were six soils of varying physical properties and geologic histories, native to Oklahoma. Four of these soils were chosen because their soil masses exhibited dispersive clay erosion in the field. The remaining two samples were added to provide a measure of control because they were known to be nondispersive. Pertinent engineering properties of the samples are given in Tables 1 and 2. The sample locations, geologic histories, and chemical properties of the samples are given elsewhere (10).

High-calcium hydrated lime, reagent grade sodium chloride, and laboratory grade aluminum sulfate were used in the study.

Physical Erosion Test

The physical erosion test (PET) was developed to simulate field situations and to accelerate dispersive erosion so that it could be measured during a relatively short test period (10, 11). PET apparatus, shown in Figure 1, simulates field conditions by back pressure saturation and intermittent flow of water through samples compacted to expected field moisture and density conditions. Erosion testing was accelerated by drilling longitudinal holes in the soil samples. The relationship between the weight of dry soil at the start of the testing sequence and that remaining after testing was used to indicate the percentage of erosion, i.e.,

$$\text{Percentage of erosion} = \frac{\text{initial dry weight} - \text{ending dry weight}}{\text{initial dry weight}} \times 100 \quad (1)$$

PET is believed to give better results than other chemical and engineering property tests currently used to identify potentially dispersive clay soils. Interpretation of PET data used for predicting potential dispersive erosion damage is as follows.

1. Erosion values of 0 to 15 percent indicate that little, if any, erosion should occur.
2. Erosion values of 16 to 35 percent indicate that slight to moderate erosion may occur, probably as a result of slaking or surface erosion or loss of mass structural coherence. Visual observation of PET specimens should help classify the expected behavior.
3. Erosion values of 36 to 50 percent indicate that moderate to severe erosion may occur, primarily as a result of clay dispersion and that there is some lag between construction and the time erosion damage is noted. Visual observation of PET specimens should help to determine the behavior expected.
4. Erosion values of 51 to 65 percent indicate that severe to very severe erosion may occur, primarily as a result of dispersion of a clay binder that contains an appreciable sand-silt fraction with low erosion resistance. If the soil is primarily clay, these values indicate extremely high dispersion potential. Visual observation of the PET specimens will aid predictions of the type of expected field erosion behavior.
5. Erosion values of 66 percent or more indicate soils similar to those in the previous item, except that a faster rate of erosion occurs. Very severe erosion problems should be expected with these soils, during or shortly after construction.

Although the above criteria are tentative, they nevertheless represent our best judgment about interpretation of PET results in evaluation of expected field erosion behavior (10).

Methods based on use of soil engineering properties to predict dispersive soil erosion behavior are usually not valid. For example, a typical engineering relationship developed by Gibbs and Holtz (28), involving natural density versus liquid limit, was evaluated during PET development (10). Data for 21 natural dispersive and nondispersive soils were plotted on the erosion chart of

shown in Figure 2, which indicates PET groups. Because the research samples were received in a disturbed state, standard Proctor maximum dry density was used rather than natural dry density in the graph, but the error is not believed to be significant. When the relationship shown in Figure 2 is examined, both the nonerosive, nondispersive clay soils and the highly erosive dispersive clay soils fall in and along the zone of supposedly highest erosion resistance. Another evaluation of erosion prediction was made by using soil plasticity and the A-line chart of the Unified Soil Classification System (28). The shaded areas in Figure 3 were thought by Gibbs and Holtz to represent the erosion potential of fine-grained cohesive soils. When the results obtained from PET were plotted according to major groups, no definite relationship was observed between results obtained from PET and those based on soil plasticity.

The data shown in Figures 2 and 3 confirm the newly accepted theory that conventional soil engineering properties may not be used to correctly predict erosion resistance of potentially dispersive clay soils. Soil chemistry techniques have not been completely useful either, but PET (10) is believed to give reasonable predictions. Besides determining the amount of soil erosion expected for given soil placement conditions and worst case erosion water conditions, visual observation of samples during and after testing may give considerable qualitative insight about the type and rate of ex-

Table 1. Engineering properties of research samples.

Sample Number	Specific Gravity	Liquid Limit	Plastic Limit	Plastic Index	Shrinkage (%) Linear	Shrinkage (%) Volumetric	Standard Proctor γ_d (kg/m³)	Standard Proctor ω_{opt} (%)	Modified Proctor γ_d (kg/m³)	Modified Proctor ω_{opt} (%)
201	2.76	43	18	25	11	30	1823	12.6	2055	8.6
202	2.73	46	21	25	17	43	1723	13.5	1944	10.0
203	2.75	48	20	28	15	39	1717	13.7	1930	10.3
204	2.75	60	19	41	17	43	1534	20.5	1765	16.9
205	2.72	47	16	31	17	46	1706	16.9	1852	15.0
206	2.70	45	18	27	14	37	1661	19.0	1841	15.4

Note: 1 kg/m³ = 0.063 lb/ft³.

Table 2. Grain size analysis of research samples.

Sample Number	Percentage Passing (sieve size) 0.002-mm	0.05-mm	0.02-mm	0.05-mm	No. 200	No. 140	No. 60	No. 40
201	23	33	47	61	80	86	98	100
202	33	51	79	89	92	95	99	100
203	33	54	81	90	93	95	99	100
204	30	73	83	86	87	100	—	—
205	36	51	69	82	86	87	100	—
206	31	41	70	90	97	98	100	—

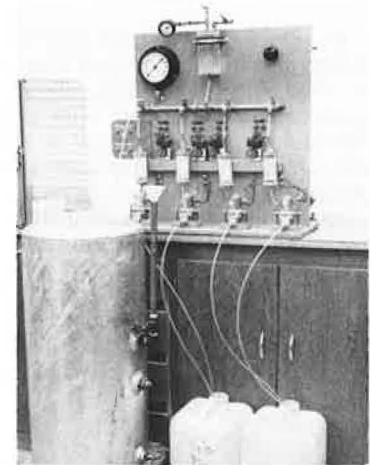Note: 1 mm = 0.04 in.

Figure 1. PET apparatus.

Figure 2. Trends shown by natural density versus liquid limit relationship.
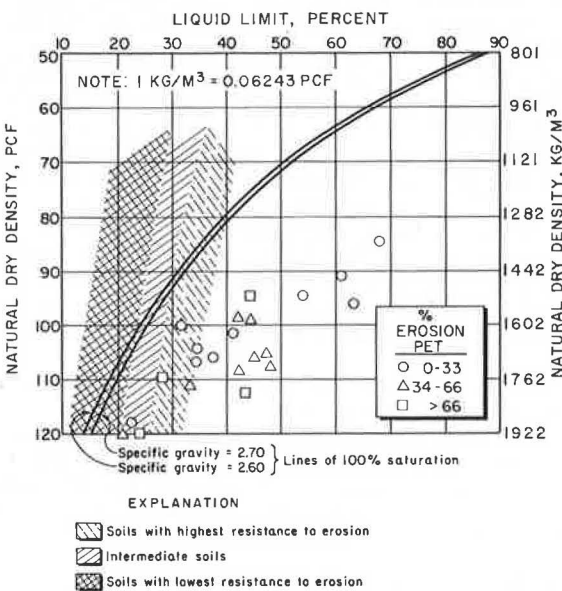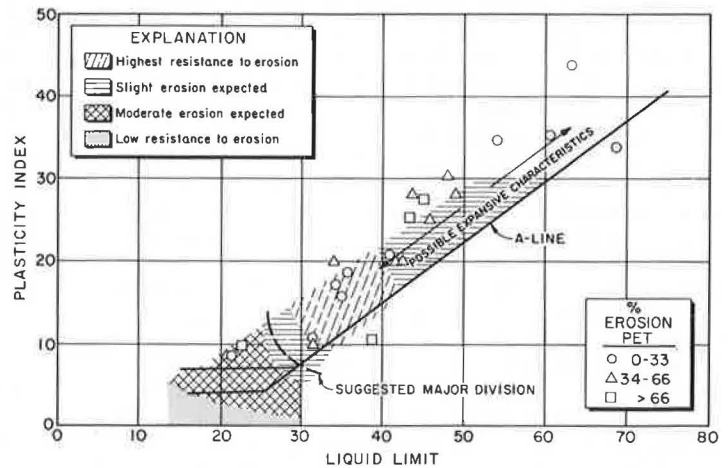

Figure 3. Suggested trend of erosion characteristics for fine-grained cohesive soils with respect to plasticity.

pected field erosion.

Preparation and Curing of Test Specimens

A standard mixing and curing procedure was used for each chemical treatment and produced nearly constant moisture contents and densities. During initial preparation, each sample was oven-dried, ground, and sieved through a No. 40 sieve. Sufficient moisture for complete chemical reaction was ensured by mixing the sample and admixture at 2 percent wet of optimum moisture content for 24 hours before compaction. The samples were compacted to standard Proctor maximum dry density at optimum water content by using a Harvard miniature mold and the Ohio State University miniature impact compaction device. Details of procedures used in sample preparation and curing are given elsewhere (10, 11, 29).

RESULTS

The primary objectives of the investigation involved the effects of lime, sodium chloride, and aluminum sulfate treatment on the dispersive erosion potential of cohesive soils; however, the dispersive erosion potential of the cohesive soils had to be determined without any chemical treatment. In the following sections, the effects of lime, sodium chloride, and aluminum sulfate treatment on six cohesive soils are presented and discussed.

No Chemical Treatment

Each sample was tested for internal erodibility a total of 16 times: 8 times at standard Proctor density and 8 times at modified Proctor density. Test results obtained from samples compacted at both standard and modified compaction and corresponding water contents were approximately the same. The explanation of this is based on a combination of two factors: The increase in dry density as a result of increased compactive effort leads to a decrease in susceptibility to internal erosion, and lower water contents associated with optimum compaction under increased compactive effort tend to increase erodibility. When both these influences on the behavior of a soil mass occurred simultaneously, the result was no general change in the average percentage of erosion.

Lime Treatment

Results obtained from physical erosion testing of lime-treated samples showed a substantial decrease in the percentage of erosion as the percentage of lime was increased, even for the nonerosive control samples.

Figures 4 and 5 show typical effects of lime treatment on erosion of dispersive soil sample 202. These photographs were taken during the pilot study phase, and, because little erosion took place at the higher lime percentages, the production research was carried out with 0.5, 1.0, and 1.5 percent lime.

The effects of lime treatment on soil erodibility, as shown by a plot of the percentage of erosion versus chemical treatment for each soil tested, are shown in Figure 6, in which a relationship is seen to exist between the percentage of lime treatment and PET percentage of erosion. Although this relationship is not linear, the percentage of erosion definitely decreases as the percentage of lime treatment increases.

An explanation for the effectiveness of lime as a chemical treatment for dispersive clay soils is based on several factors. One factor is the increase in Ca^{++} cation exchange with the increase in soil pH. Another factor believed to cause decrease in erosion with increase in lime

treatment percentage is compression of the ionic atmosphere around the clay particles. Pozzolanic reactions between the lime and clay particles may also have caused an increase in strength, and the cementation reduces measured erosion. Although this last reaction was observed to occur, it was not directly measured.

Sodium Chloride

Percentages of sodium chloride chosen were 1, 2, and 3 by dry weight of soil. Two methods were used in adding the sodium chloride to the soil sample. One method involved mixing the sodium chloride and the soil before water was added; the other method involved dissolving the sodium chloride in water before it was mixed with the dry soil. The least effective mix procedure in the treatment of dispersive erosion would provide better insight into the use of sodium chloride in either form. Since the wet mix method gave a less effective treatment, as seen in Figure 6, the dry mix procedure was abandoned after three soils had been tested. Relationships between the percentage of sodium chloride, both wet and dry mixed, and the average PET percentage of erosion are shown in Figure 6.

When these results were analyzed, three different patterns of change in erosion were noted when sodium chloride was added. The first erosion pattern was observed in sample 201 in which the percentage of erosion initially decreased, then increased as the percentage of sodium chloride increased. The second pattern was observed in samples 202, 203, and 206, in which the average percentage of erosion decreased as the percentage of sodium chloride increased. The final pattern was observed in control samples 204 and 205, in which the average percentage of erosion increased as the percentage of sodium chloride increased.

The different patterns of erosion may be explained when sodium chloride is considered to act as a peptizing agent. Van Olphen (19) notes that, as the concentration of ions increases when sodium chloride is added, a charge reversal on the clay edges is likely to result because of anion adsorption on the clay edges that causes clay dispersion. Then, according to Mysels (30), further sodium chloride causes the concentration of sodium cations to increase on the clay surface as the adsorbed anions are replaced, and a flocculated structure results.

The behavior of control samples 204 and 205 seems to demonstrate that an initial increase in sodium chloride causes an increase in dispersion potential. These samples possessed an initially low ESP (clay), which is the exchangeable sodium percentage (ESP) (3) divided by the percentage of the clay fraction. ESP (clay) is expressed as a percentage (11). A low ESP (clay) indicates a small amount of adsorbed sodium ions when compared with the total cation exchange capacity of the clay fraction. Although we initially thought that sodium chloride addition should cause additional flocculation of the nondispersed soils, especially at the sodium chloride percentages added, it appeared that the samples exhibit charge reversal and dispersion rather than flocculation from high sodium concentrations in the pore water. However, a point probably exists after which the addition of more sodium chloride will result in flocculation rather than in dispersion.

Samples 202, 203, and 206, which possessed an initially high ESP (clay), demonstrated a reduction in dispersion and thus a reduction in the percentage of erosion as sodium chloride was added. The high initial sodium concentration of these soils had caused a charge reversal on the clay edges; thus when additional sodium was added it became a flocculating agent rather than a peptizing agent.

Sample 201 was observed to first decrease and then

Figure 4. PET observations at start of testing for sample 202 with lime treatment.

Figure 5. PET observations after 4 hours of testing for sample 202 with lime treatment.
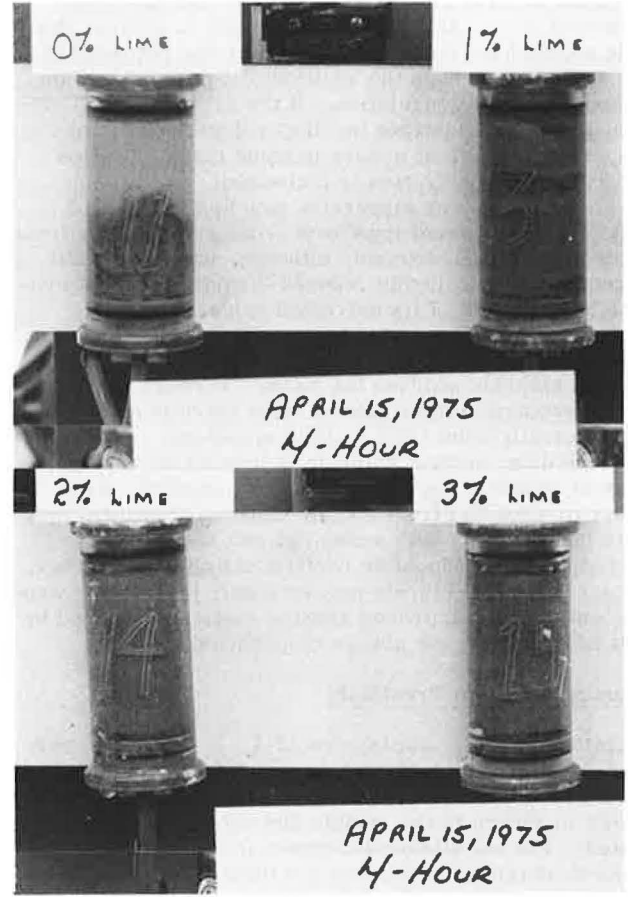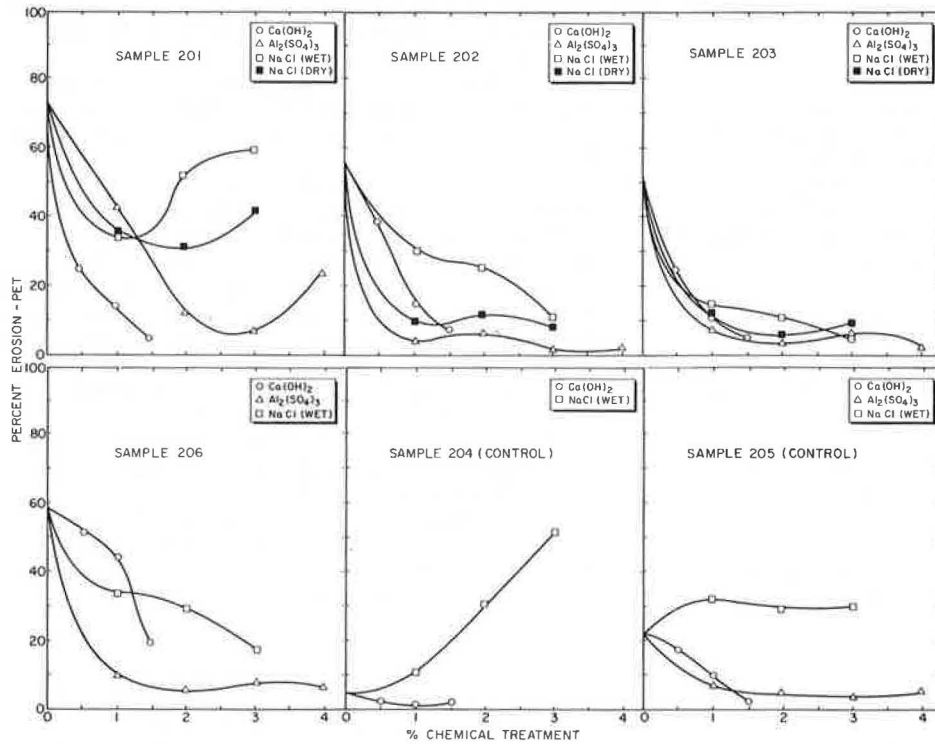

Figure 6. Percentage of erosion, based on PET, versus chemical treatment for each soil sample.

increase in erosion potential as the sodium percentage increased. Sample 201 is the anomaly of the study and is highly erosive as indicated by ESP and ESP (clay) values of zero. Adding a small amount of sodium chloride should have increased the dispersion potential of the soil, and perhaps the addition of 1 percent sodium chloride caused flocculation. If the data are not in error, they may illustrate the dispersion-flocculation-dispersion cycle that occurs in some clays as cation electrolyte concentration is increased.

Three of the four dispersive samples tested (202, 203, and 206) showed improved erosion resistance from sodium chloride treatment, although, for sample 201, 1 percent sodium chloride reduced the percentage of erosion to about half of its untreated value. However, higher treatment percentages caused increased erosion. The two low-sodium control soils were dispersed by sodium chloride addition but as they were not dispersively erosive in their natural state there is no reason to chemically treat them in field situations. According to these data, sodium chloride is less effective than lime in reducing dispersive erosion potential. Also, the behavior expected from sodium chloride treatment may vary considerably with amount of salt used; thus expected behavior should be confirmed by PET. Further, because sodium chloride may be easily leached by seepage waters, the improved erosion resistance caused by salt addition may not always be permanent.

Aluminum Sulfate Treatment

Aluminum sulfate admixtures of 1, 2, 3, and 4 percent by dry weight of soil were used. The effect of aluminum sulfate treatment on erosion resistance is shown in Figure 6; the sample 204 control soil was not tested. The significant decreases in the percentage of erosion shown in the figures are thought to result from flocculation of the soil. Although aluminum sulfate treatment appears to be an effective method of reducing dispersive soil erosion, its unit cost is considerably higher than that of either lime (which works as well or better) or sodium chloride.

CONCLUSION

The objective of this study was to determine whether chemical treatment could be used to adequately prevent erosion caused by dispersive clay soils. This objective was accomplished by evaluating the effectiveness of various chemical treatments on the erosion pattern determined by PET. From the analysis of the data obtained during this study, the following may be concluded:

1. Results obtained from this study further verified that PET is an adequate indicator of potentially dispersive clay soils;
2. Use of soil density or plasticity is not an adequate means of predicting the dispersive erosion potential of clay soils;
3. Flocculation through chemical stabilization is a reasonable alternative for reducing or eliminating dispersive clay problems; and
4. All three chemical stabilizers evaluated, hydrated lime, aluminum sulfate, and sodium chloride, provided successful treatment and are ranked in order of decreasing effectiveness.

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Chemical Stabilization for Control of Dust and Traffic Erosion

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Forty-six commercially available chemicals were tested. Specimens of a dune sand sprayed with chemicals were subjected to simulated wind velocities up to 145 km/h (90 mph). Specimens of compacted granitic soil treated with chemicals by either spraying or mixing were subjected to simulated traffic abrasion forces under simulated tire pressures up to 414 kPa (60 psi). Selected chemical treatments were subjected to various environmental-durability conditions including freeze-thaw cycles, wet-dry cycles, rain-dry cycles, and various curing temperatures. Based on the results of this laboratory testing, several chemical stabilizers were selected for use in a large-scale field application. Eleven chemicals were sprayed on untraffickable areas to control dust and wind erosion. Five chemicals were sprayed on an unpaved road to control erosion and dust behind traffic. Three chemicals were also mixed with the surface of an unpaved road. Methods of field application and monitoring techniques including dust collection by a high-volume air sampler, dust fall collection in cups, and extraction tests are discussed. Preliminary comparisons of the chemical applications with themselves and with control sections, where water was used, are given. Evaluation will continue for approximately 12 months.

In arid and semiarid climates, soil erosion due to wind causes movement of cohesionless sandy soils and development of sand storms and high levels of dust particulates. In addition, unpaved gravel secondary roads require periodic grading and replacement of material lost through erosion due to traffic. Soil erosion due to both wind and traffic causes a significant increase in dust particulates in the atmosphere. The problem has already posed severe safety, health, and public relations problems and will continue unless positive measures for erosion control are developed and implemented.

EXPERIMENTAL PROGRAM

The scope and objectives of this laboratory investigation were multifold and are outlined as follows:

1. Screen the commercial market for chemical stabilizers that are potentially suitable for soil erosion control;

2. Select two soils, a wind-blown sand for studies of erosion due to wind and a common subgrade soil for studies of erosion due to traffic;

3. Determine the ability to reduce erosion due to wind of various chemicals sprayed on dune sand;

4. Determine the ability to reduce erosion due to traffic of various chemicals sprayed on or mixed with compacted subgrade soil;

5. Determine the durability of the stabilized soils under adverse environmental conditions (these tests were limited to those chemicals that performed best in the preliminary tests); and

6. Select several chemicals, based on the laboratory testing program, for use in field tests.

The durability tests include tests of erosion due to wind and traffic under freeze-thaw, wet-dry, and rain-dry cycles and various curing temperatures.

Chemical stabilizers were solicited from major manufacturers, suppliers, and formulaters. The letter of solicitation specified that the products should be nontoxic, nonflammable, noncorrosive, easy to handle and apply, and safe for plant or animal life should they leach out of the treated soil.

Cost of application was limited to 18 cents/m² (15 cents/yd²) for nontraffickable areas (this was later reduced) and 90 cents/m² (75 cents/yd²) for stabilization of traffickable unpaved roads.

Of approximately 170 manufacturers and suppliers contacted, 36 indicated willingness to participate in the project and forwarded 45 chemicals, which were used in this study (Table 1). Detailed information regarding these chemicals is given elsewhere (1).

TESTS OF EROSION DUE TO WIND

Initial Testing

Loose dry sand was placed in 15-cm-diameter by 5-cm-high (6 by 2-in) polyvinyl chloride (PVC) molds, at a nominal density of 1.55 g/cm³ (97 lb/ft³). Specimens were sprayed evenly with the recommended rate of chemical mixture by using a spray gun. Specimens were cured in an environmental room at 21°C (70°F) and 50 percent

relative humidity for 1, 3, and 7 days. After curing, separate specimens were tested for 3 min at 72.4 and 145-km/h (45 and 90-mph) simulated wind from wind blowers. The weight of material loss was corrected to dry basis and calculated as a percentage of the original weight of dry sand in the mold. Details of the testing procedures are given elsewhere (1).

Based on their performance in the initial test (resulting in less than 5 percent erosion), 27 chemicals were selected to undergo further testing to evaluate the durability of their stabilization potential after being subjected to adverse environmental conditions.

Freeze-Thaw Cycles

After being sprayed, the sand specimens were cured for 3 days in the environmental room. Specimens were then placed for 6 hours in a humid room [21°C (70°F)] where access to moisture was made available through continuous moisture spray and vapor. Specimens were then subjected to three freeze-thaw cycles. Each cycle consisted of 6 hours in a freezing room at -12.2°C (10°F) and 18 hours in a humid room at 21°C (70°F).

At the end of the third cycle, specimens were allowed to air dry to a constant weight in the environmental room. Duplicate specimens were then tested under 72.4 and 145-km/h (45 and 90-mph) wind velocity. The percentage of erosion was calculated as discussed above.

Wet-Dry Cycles

Similar specimens were subjected to three wet-dry cycles. Each cycle consisted of 6 hours in the humid room and 18 hours in the environmental room. The testing was the same as that for wind erosion.

Rain-Dry Cycles

Similar specimens were subjected to three rain-dry cycles. Each cycle consisted of 1 hour of rain at 6.04 cm/h (2.38 in/h) and 23 hours in the environmental room. Details of the Rotadisk Rainulator and procedures for testing are given elsewhere (1, 2, 3, 4). Specimens were tested for erosion due to wind, and the percentage of loss caused by erosion due to rain and wind was determined and reported separately.

Variable Curing Temperatures

Similar specimens were cured at 4.4 and 60°C (40 and 140°F) instead of 21°C (70°F) and then tested as were the specimens in the initial testing. This test was conducted to evaluate the influence of low and high temperatures on the stabilization effects of the chemicals.

Discussion of Test Results

The results of these environmental tests indicated that 20 chemicals (of 27 tested) successfully endured the conditions imposed. The rate of application of the chemical was reduced, and thus the cost of application was reduced to about 9.5 cents/m² (8 cents/yd²).

The same tests were conducted on another set of treated specimens at the reduced rates, and the specimens were tested at a 145-km/h (90-mph) wind velocity. The results of the tests at the reduced rates were considered for further investigations.

Fourteen chemicals successfully endured the various environmental conditions to which they were subjected, and they provided a good measure of erosion control under a wind velocity of 145 km/h (90 mph). A selection criterion was arbitrarily set that eliminates any chemi-

cal treatment that resulted in a percentage of erosion due to wind equal to or greater than 5. The rain-dry cycles proved to be the most severe type of durability test condition in that it generally resulted in higher erosion than the other conditions.

The 14 chemicals that endured the tests were

1. Aerospray 70,
2. Petroset SB,
3. Dresinate DS-60W80F,
4. Terrakrete No. 2,
5. Aquatain (powder),
6. Plyamul 40-153,
7. Foramine 99-434-2,
8. Surfaseal,
9. Coherex,
10. Paracol 1461,
11. Dust Stop,
12. Foramine 99-194,
13. Polyco 2460, and
14. Norlig 41 + F125.

When we requested more Polyco 2460, Foramine 99-434-2, and Plyamul 40-153 for the field application, the suppliers reported that these chemicals had been discontinued mainly because their basic ingredients were unavailable during the energy shortage (January 1974). Aquatain (powder) was not considered for field testing because it is biodegradable and would lose effectiveness with time. Dust control oil was added to those chemicals used in the field application because of its superior field performance in another study completed at that time (5).

Most of the lignin-based products, e.g., Orzan GL-50, Norlig 41, and Soiltex, experienced their highest erosion after being subjected to the rain-dry cycles. This was expected because of the usually high solubility of the lignin products.

An attempt to reduce the solubility of a lignin product (Norlig 41) was made by mixing the Norlig 41 solution with a solution of a waterproofer (F125) that has sodium methyl siliconate as its major constituent. This mixture successfully reduced the solubility of the Norlig 41. This mixture was used throughout the study as chemical No. 46.

Chemicals Selected for Field Test of Erosion Due to Wind

Based on the test results, 11 chemicals were used in the field test phase of this project. The rates of application selected were the reduced rates. The chemicals used in the field application (water was used as the control section) were

1. Aerospray 70,
2. Petroset SB,
3. Terrakrete No. 2,
4. Foramine 99-194,
5. Dresinate DS-60W-80F,
6. Dust control oil,
7. Surfaseal,
8. Paracol 1461,
9. Dust Stop,
10. Coherex, and
11. Norlig 41 + F125.

TESTS OF EROSION DUE TO TRAFFIC

Spray Application

Tests were conducted to evaluate the degree of stabilization effected by spraying the chemicals on a compacted road surface subjected to the abrasive action of traffic.

The spray treatment simulates a postconstruction application for unpaved (dirt) roads on which it may not be feasible to mix the chemical with the subgrade before compaction. Granitic soil was used exclusively in tests of erosion due to traffic.

The apparatus used was a modification of a design reported by Gallaway and Jimenez (6). The apparatus consists of four rubber rollers mounted on a shaft that, in turn, is mounted on a small steel frame. Bolted to the top of the frame is an 11-cm-long (4.5-in) shaft. The top portion of the shaft was inserted in the rotating sleeve of a mechanical bituminous mixer that can rotate at a speed of 30 rpm. Tare weights were machined out of steel cylinders and steel plates such that they can be slipped onto the small frame through the vertical shaft. The weights were calibrated such that the resulting imprint contact pressure between the rubber rollers and a flat soil surface can be varied at 27, 310, and 414 kPa (30, 45, and 60 psi). Granitic soil mixed at optimum moisture content was compacted in the PVC molds to a maximum dry density of 2.05 g/cm^3 (128 lb/ft^3) as determined by AASHTO T-180. The specimens were then placed in the environmental room for 7 days to reach constant weight. Specimens were then sprayed evenly at the recommended rate of chemical by using a spray gun. One set of specimens was then cured for 3 days and another for 7 days in the environmental room.

After curing, the specimen was weighed, placed under the abrasion apparatus at a contact pressure of 414 kPa (60 psi), and tested for 10 min. During the test, a small wind blower was directed at the top of the specimen to remove abraded particles. At the end of the abrasion test, the specimen's final weight was recorded. The ratio of the weight loss, corrected to dry weight, to the weight of dry soil in the compacted specimen was reported as the percentage of erosion. Chemical treatments resulting in an abrasion loss of less than $\frac{1}{2}$ percent were considered worthy of further testing.

Nineteen chemicals (of 46) were selected to undergo further testing after being subjected to adverse environmental conditions.

Freeze-Thaw Cycles

Specimens were compacted, cured for 7 days in the environmental room, sprayed with chemicals, and then allowed to cure for 7 days in the environmental room, as described before. Specimens were placed for about 2 hours on moist pads where water was made accessible to the bottom of the specimens. Specimens were then subjected to three freeze-thaw cycles. Each cycle consisted of 6 hours in the freezing room and 18 hours in the environmental room. At the end of the third cycle, the weight of each specimen was recorded and then tested under a 414-kPa (60-psi) contact pressure for abrasion. At the end of the test, the ratio of the corrected weight loss to the weight of the dry soil in each compacted specimen was reported as the percentage of erosion.

Wet-Dry Cycles

Similar specimens were subjected to three wet-dry cycles consisting of 6 hours in the humid room and 18 hours in the environmental room. They were then tested for abrasion.

Rain-Dry Cycles

Similar specimens were subjected to three rain-dry cycles consisting of 1 hour of rain at 6.04 cm/h (2.38 in/h) and 23 hours in the environmental room and were then tested for abrasion.

Variable Curing Temperatures

After being sprayed, similar specimens were cured at 4.44 and 60°C (40 and 140°F) instead of 21°C (70°F) for 7 days and were then tested as were the specimens in the abrasion test.

Of the 19 chemicals tested, eight effectively endured the test conditions and resisted the simulated tire abrasion under 414 kPa (60 psi) of pressure with erosion losses of less than $\frac{1}{2}$ percent. These chemicals were

1. Aerospray 70,
2. Polycol 2460,
3. Plyamul 40-153,
4. Foramine 99-434-2,
5. Curasole AE,
6. Foramine 99-194,
7. Ashland oil stabilizer, and
8. Norlig 41 + F125.

In addition, one chemical, Dust Bond 100, resulted in similar effective degrees of control, except at the curing temperature of 4.44°C (40°F).

Results

Both the 4.44°C (40°F) curing temperature and the rain-dry cycles proved to be the most severe types of durability test conditions inasmuch as they generally resulted in the highest erosion.

Again, several chemicals could not be delivered by manufacturers in sufficient quantities for the field application. These were Polyco 2460, Foramine 99-434-2, Plyamul 40-153, Ashland oil stabilizer, and Norlig 41. Therefore, only three of the eight chemicals that proved effective were available for field testing. Inasmuch as Dust Bond 100 (which is also a lignin sulfonate product) was available, it was used in place of Norlig 41 in the mixture with F125 as chemical No. 46. In addition, because of the successful experience in field application of dust control oil (5) and its reasonably good laboratory results, this chemical was included in the chemicals used in the field test. Accordingly, five chemicals were used in the field application, in addition to water (control section):

1. Aerospray 70,
2. Dust Bond 100 + F125,
3. Foramine 99-194,
4. Curasole AE, and
5. Dust control oil.

Mixing Application

Traffic erosion tests were conducted to evaluate the degree of stabilization effected by mixing the chemicals with the subgrade material before placement and compaction. This application is intended to produce a stabilized road surface for secondary roads that resists the abrasive forces of traffic and reduces the dust clouds produced by traffic on unpaved dirt roads. Granitic soil was used exclusively in this test. The traffic abrasion simulator discussed previously was also used in this phase of the testing program.

In this case, the soil, water, and chemical solution were mixed together before compaction. After compaction, one set of specimens was cured for 3 days and another for 7 days in the environmental room. At the end of the curing period, the specimens were tested for abrasion at a simulated tire pressure of 414 kPa (60 psi), as discussed before. Chemical treatments resulting in abrasion loss of less than $\frac{1}{2}$ percent were considered

worthy of further testing.

Based on their performance in this test, only seven chemicals were selected to undergo further testing to evaluate their stabilization potential under adverse environmental conditions. These chemicals were

1. Redicote E-52,
2. Norlig 41,
3. Soiltex,
4. Norlig 41 + F125,
5. Orzan Gl-50,
6. Dust Bond 100, and
7. Ashland oil stabilizer.

It is interesting to note that five of these seven chemicals have lignin sulfonate as a base material.

The environmental-durability tests included freeze-thaw cycles, wet-dry cycles, rain-dry cycles, and various curing temperatures. Details of the durability tests are similar to those discussed previously.

Under 414 kPa (60 psi) of simulated tire pressure, only Norlig 41 + F125 was effective (with erosion loss of less than $\frac{1}{2}$ percent) under all durability conditions. A similar degree of effectiveness was given by Redicote E-52, Norlig 41, Dust Bond 100, Soiltex, and Ashland oil stabilizer except under one or two of the durability conditions imposed.

Chemicals with a lignin sulfonate base provided the best performance in the traffic abrasion tests of the mixed specimens; we were also successful in water-proofing (reducing solubility) the Norlig 41 treatment by adding F125.

Because it was unrealistic and unnecessary to use three or four different lignin-based chemicals in the field application, Norlig 41 was chosen to represent this group of chemicals. However, because the manufacturer was unable to deliver large quantities at that time, Dust Bond 100 was used instead of Norlig 41 in composition of chemical No. 46. Accordingly, Redicote E-52 and a mixture of Dust Bond 100 plus F125 were recommended for field application. At this time the supplier of dust control oil indicated his willingness to donate the chemical for field application by spraying and mixing. Therefore, the field application included the Redicote E-52, Dust Bond 100 + F125, and dust control oil.

FIELD TESTING PROGRAM

Dust Control Test

Two sites were used to test the application of the chemical spray to control dust on untraffickable areas. Each site was cleared, leveled, and smoothed before spraying. Each chemical was allocated a designated area of 6 by 12 m (20 by 40 ft). Chemical solutions were prepared in and sprayed with a mobile sprayer having a 190-liter (50-gal) capacity. The rates of application were adjusted beforehand by calibrating the output of the sprayer.

Two months after the application of chemical on the first site (a former farm area), weeds started to grow profusely, aided by the heavy summer thunderstorms. In an actual field application this may not be considered a problem because weeds tend to provide an additional measure of dust control. However, on a test site, they did present a problem because they obscured the conditions of the sprayed surfaces. Therefore, another site was selected and sprayed with the chemicals to which Princep-80W, a weed control agent containing 80 percent Simazine as an active ingredient, was added. The recommended rate of application for Princep-80W was set at 11.2 kg/ha (10 lb/acre).

The techniques used for evaluating the performance of the chemical applications on the dust control sites (untraffickable) were developed by the writer (1, 5). These methods, briefly outlined below, were conducted on a bi-weekly basis.

Sampling of Windblown Dust

A small blower was used as a wind simulator to stir dust particles off the surface. The blower was placed on an inclined steel support such that the air flow would hit the ground surface at an angle of about 40 deg with the horizontal (1, 5). The wind velocity at the mouth of the blower was about 19 km/h (12 mph) and decreased to approximately 13 km/h (8 mph) at the point of impact on the ground. A high-volume air sampler (HiVol)(7) was placed 1.2 m (4 ft) away from the blower along the direction of wind flow. A glass-fiber filter paper 20 by 25.4 cm (8 by 10 in) in size was used to collect the dust particulates on it.

Sampling was conducted with the wind blower on, and the HiVol drawing air at 1.4 m³/min (50 ft³/min) over a 5-min period. The amount of dust collected was computed in micrograms per cubic meter. The development and modification of this test and the reasoning behind the chosen parameters are given elsewhere (1).

Sampling for Extraction Test

Soil samples from the surface of the treated zones were obtained and used in an extraction test to determine the amount of benzene or water-soluble organic matter or both present. Comparing the amounts extracted from samples at different periods after application gives a quantitative evaluation of the degree of leaching of the chemical. The weight of the extracted organics was measured to the nearest milligram and converted, according to the area of extraction sample, into grams per square meter. The extraction procedure is outlined elsewhere (1, 5) and is very similar to that outlined by the Public Health Service (8).

Visual Inspection and Evaluation

In addition to the quantitative evaluation techniques discussed above, a qualitative evaluation is being made periodically on the condition of each test plot. This evaluation includes condition of the surface, thickness and firmness of crust, color change, cracks, and vegetation growth. This inspection supplements the other tests and helps to spot erratic or unexpected results.

Road Test

Spraying Application

A road test site was selected on a dirt (gravel) road just south of I-10 east of Tucson. Ten sections 183 m long (600 ft) and 8.5 m wide (28 ft) were marked along the road. One section was used for each chemical treatment. For applications by spraying, the surface of the road was usually prepared by surface blading (no ripping), which left a nominally loosened surface layer. The chemical solution was prepared in a boot truck and sprayed on the surface through the spray bar. The boot truck was equipped with a circulating pump that continued to mix the chemicals during application. After it was sprayed, the surface was usually rolled with a rubber-tired roller.

Mixing Application

For the mixing application, the road surface was sprayed

lightly with water, and then the surface was ripped up, by using a ripper attached to the grader, to a depth of about 7.6 cm (3 in). A mixed and compacted mat of this thickness was selected because a Seaman mixer was not available and based on field results reported by Hoover (9). Because Hoover (9) had difficulty in mixing and compacting a ripped 10-cm-thick (4-in) layer, he recommended future use of 7.6-cm (3-in) thickness. After the road surface was ripped up, additional water was sprayed to reduce surface tension effects, and then a portion of the required chemical application was sprayed

on the surface. The loosened surface soil was then bladed to the sides of the roads to form two windrows. Each windrow was then spread back on the road surface, sprayed with more chemical and water if necessary, and then bladed to form a windrow in the middle of the road. When all the required chemical and enough water (to reach optimum moisture in the field) had been added, a continuous operation of surface mixing by the blade was done. After complete mixing, two side windrows were formed. The mixed soil was then spread on the surface and compacted in two lifts, forming a slight crown near the center. The field techniques used for evaluating the performance of the chemical applications were developed and discussed elsewhere (1). The methods are briefly outlined below.

Sampling of Windblown Dust

This test is the same as that discussed for the dust control sites and was conducted on each section of the road test.

Dust Collectors Across the Road

Dust collectors were installed across the road at the middle of each section. The dust collectors consisted of plastic cups that were taped to the top of 5-cm-wide (2-in) plywood sticks, with their top approximately 0.9 m (3 ft) above the ground. The containers were half-filled with distilled water and covered at the top with a wire screen with square openings 2 mm (0.78 in) in size. The screen was taped to the side of the cup to prevent insects from crawling into the cup, which occurred when only a rubber band was used.

The cups were placed at a spacing of 6.1 m (20 ft) for a distance of 43 m (140 ft) and at 15.2-m (50-ft) spacing

Table 1. Chemicals used in laboratory and field test.

No.	Chemical	No.	Chemical
0	Water (control)	24	Enzymatic SS-2
1	Soil stabilizer 801	25	Dresinate DS-60W-80F
2	Compound SP-301	26	Paracol 1461
3	White soil stabilizer	27	Terra-Krete No. 2
4	Stikvel P 65	28	Terra-Krete No. 1
5	Velsicol W-617	29	Ecology control M-binder
6	Redicote E-52	30	Triton X-114 SB
7	Aerospray 70	31	Corexit 7740
8	Aerospray 52	32	Super Crete 100
9	Curasol AE	33	Aliquat H226
10	Polyco 2190	34	Petroset-RB
11	Polyco 2460	35	Biobinder
12	Orzan GL-50	36	Surfax 5107
13	Surfaseal	37	Dust control oil
14	Formula 125	38	Dust stop
15	Enzymatic SS-1	39	Aquatain (liquid)
16	RTD-SS-X	40	Aquatain C (powder)
17	Norlig 41	41	Foramine 99-194
18	Dust Bond 100	42	Plyamul 40-153
19	Sodium silicate grade No. 9	43	Ashland soil stabilizer
20	Petroset SB	44	Compound SP-400
21	Coherex	45	Foramine 99-434-2
22	Soiltex	46	Norlig 41 + F125
23	Thermoset 401		

Table 2. Preliminary observations of dust control and road tests.

Item	Chemical No.	Highest HiVol Collection ($\mu g/m^3$)	Description[a]
Dust control test	0	64 301	Natural color, thin soft crust, some cracks
	7	8 751	Light brown, hard crust 4.8 mm thick, some cracks
	13	2 972	Brown, very hard crust 6.4 mm thick, some cracks
	13	6 397	Light brown, hard crust 4.8 mm thick, some cracks
	20	8 312	Natural, hard crust 4.8 mm thick, some cracks
	21	4 857	Natural, medium crust 4.8 mm thick, some cracks
	25	7 122	Natural, hard crust 4.8 mm thick, cracks
	26	8 949	Natural, hard crust 4.8 mm thick, cracks
	27	2 478	Natural, hard crust 6.4 mm thick, light cracks
	37[b]	3 441	Black, soft crust 7.9 mm thick, light cracks
	38	5 862	Natural, hard crust 4.8 mm thick, some cracks
	41	8 921	Natural, very weak crust 1.58 mm thick, many cracks
	46	4 490	Light brown, hard crust 4.8 mm thick, some cracks
Road test, spraying application	0	63 367	Natural color, soft when wet, worn, rutted, large amount of loose material, large cloud of dust behind traffic
	7	4 932	Brown color, medium hard surface, medium wear and few ruts, small amount of loose material, light dust behind traffic
	9	4 191	Dark brown, hard surface but worn, rutted with several potholes, substantial loose material on surface, moderate dust behind traffic
	18	5 286	Brown, hard surface, little wear, smooth surface with little loose material, very light dust behind traffic
	37	1 352	Black, very hard surface, some potholes near shoulders, minimal loose material, extremely light dust behind traffic
	41	16 918	Natural color, soft, worn and rutted surface, poor riding quality, large amount of loose material, almost as if untreated
Road test, mixing application	0	41 334	Natural color, soft when wet, worn, rutted, substantial amount of loose material, large dust cloud behind traffic
	6	1 246	Black, very hard, asphaltlike surface, little wear, fairly smooth, no loose material, no dust behind traffic
	18	1 897	Brown, hard surface, smooth, little wear, some loose material, very light dust behind traffic
	37[c]	3 618	Black, hard in spots, many potholes and ruts, large loose material on surface, moderate dust behind traffic
	37[d]	5 111	Black, hard in spots, very worn with ruts and potholes, substantial loose material, moderately heavy dust behind traffic

[a]Descriptions are of condition of plot for dust control test and condition of road for road test. [b]Application was 0.45 dm³/m² (0.1 gal/yd²). [c]Section 10a. [d]Section 10b.

for an additional 31 m (100 ft) at both sides of the road. The cups were left in place for 21 days and were periodically checked to ensure that there was sufficient water in them. This test was considered to be relatively simple yet conforms, as nearly as possible, to ASTM D 1739 for collecting and analyzing dust fall. The distance adopted for dust collection across the road, 73 m (240 ft) on both sides, was based on the results of similar testing reported by Hoover (10), where the dust collected showed a very rapid drop-off from the road shoulder out to 9 to 12 m (30 to 40 ft), followed by a more gradual drop to about 46 m (150 ft). Beyond 46 m (150 ft) a nearly constant low deposition rate was reported (10).

At the end of the collection period, the cups were sealed and taken to the laboratory. Details of the laboratory filtration and determination of nonvolatile solids (dust particles) are given by Sultan (1). This test will be conducted three or four times during the entire monitoring period of 15 months.

Visual Inspection and Evaluation

In addition to the quantitative evaluation techniques described above, a qualitative evaluation is made periodically of the condition of each test section. This evaluation includes condition of the road surface, degree of dust control during traffic, riding quality, ruts, potholes, and surface cracking.

PRELIMINARY FIELD RESULTS

The following field data, results, and observations are reported after only 3 months of field monitoring. Therefore, they should be viewed as preliminary.

Dust Control Test

Preliminary observations for the various treated plots are given in Table 2. The highest value was reported because the site has been abnormally wet during the observation period.

Based on the data given in Table 2 and general field conditions, the top five of the applied chemicals in terms of performance are as follows:

1. Terrakrete No. 2,
2. Surfaseal 1:10,
3. Dust control oil, 1.13 liter/m² (¼ gal/yd²),
4. Norlig 41 + F125, and
5. Coherex.

As stated previously, because of the short period of evaluation and the moist condition of the surface due to heavy rains, these data are preliminary.

Road Test

Spraying Application

Preliminary observations for the various sections of the road treated with chemicals applied by spraying are also given in Table 2. Based on the data given in Table 2 and the general road conditions, a preliminary performance rating of the chemicals is as follows:

1. Dust control oil,
2. Dust Bond 100 + F125,
3. Aerospray 70,
4. Curasol AE, and
5. Foramine 99-194.

Mixing Application

Preliminary observations for the various sections of the road treated with chemicals applied by mixing are given in Table 2. The observations include the road condition, color, riding quality, observed dust behind traffic, loose material on the surface, and highest recorded concentration based on HiVol readings.

Based on the data given in Table 2 and the general road conditions, a preliminary performance rating for the applied chemicals may be given as follows.

1. Redicote E-52,
2. Dust Bond 100 + F125, and
3. Dust control oil.

CONCLUSIONS

1. Based on laboratory test results, many commercially available chemical stabilizers proved their capability in significantly reducing erosion of dune sands under simulated wind velocities of up to 145 km/h (90 mph) after being subjected to adverse environmental conditions.
2. Laboratory studies indicate that several chemicals can be applied either by spraying or mixing and the stabilized surfaces can resist simulated traffic abrasion under a simulated contact pressure of 414 kPa (60 psi).
3. Preliminary field results indicate that the chemicals adequately control dust and resist erosion due to wind on untraffickable areas, at application costs of less than 10.8 cents/m² (9 cents/yd²).
4. Preliminary field results for the road application are less encouraging than those for untraffickable areas. This indicates less direct correlation between laboratory results and field performance for treatments subjected to traffic effects. However, a few chemicals successfully reduced erosion under traffic on an unpaved road, at a treatment cost of less than 71.8 cents/m² (60 cents/yd²) for a 7.6-cm (3-in) mat.

ACKNOWLEDGMENTS

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Elastic and Viscoelastic Behavior of a Chemically Stabilized Sand

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Samples of clean, fine sand saturated with a sodium silicate grout (mix sample No. 7) were prepared and cured in a moist room for 35 days. Four types of tests were performed on identical samples to investigate their elastic and viscoelastic behavior. Results indicate that the isotropic, linear-elastic constitutive law provides a reasonable approximation for characterizing the chemically stabilized fine sand under moving load. Furthermore, for applied vertical stress levels of less than 50 percent, the mixture may be treated as a linear-viscoelastic medium for computing time-dependent deformation under sustained loading.

Although chemical grouting has traditionally been used to form cutoff barriers for seepage control, injected chemical grout in many instances solidifies within the soil matrix to form a treated soil mass considerably different from its original material. Warner (12) concluded that a significant increase in strength can be achieved if desirable chemical grouts are used. These grouts should generally provide rigid gels and longer gel time. Inasmuch as construction activities in the Arctic and subarctic region have increased in recent years, more attention has been directed toward frost heave and spring thaw of soil masses, which can detrimentally affect engineering structures such as highway subgrade, earth embankment, and supporting pedestal of pipelines. The use of chemical grouts to fill the soil voids, thus preventing moisture migration and formation of ice lenses in the soil mass, is considered a possible solution to these problems (1, 11).

The introduction of chemical grouts into a soil matrix by either injection or mixing affects the mechanical behavior of the soil. It is therefore important that the load-deformation characteristics of the chemically stabilized soil be properly determined for the design and construction of any structures founded on chemically stabilized soil masses.

This paper presents the results of a preliminary study of the elastic and viscoelastic behavior of a chemically stabilized sand tested under controlled laboratory conditions. A limited number of important parameters were considered and the results should be applicable only to the conditions described in the tests. However, accumulated information of this type will provide needed knowledge for proper design of foundations on chemically stabilized soils in many parts of the world.

NOTATION

The following symbols are used in this paper:

D = damping ratio,
E = elastic modulus,
G = shear modulus,
G^* = complex modulus = τ_0/γ_0,
G_1 = storage modulus = $G^* \cos \delta$,
G_2 = loss modulus = $G^* \sin \delta$,
e_0 = initial void ratio,
f = frequency of oscillation,
$\sigma_{ZZ}, \sigma_{RR}, \sigma_{\theta\theta}$ = axial, radial, and tangential stresses,
σ_d = deviator stress,
$\epsilon_{ZZ}, \epsilon_{RR}, \epsilon_{\theta\theta}$ = axial, radial, and tangential strains,
ϕ_d = friction angle from drained test results,
δ = phase angle,
$\nu, \nu(t)$ = Poisson's ratio,
τ_0 = maximum shear stress amplitude,
γ_0 = maximum shear strain amplitude,
$\psi_G(t)$ = creep compliance in shear, and
$\psi_{EZ}(t)$ = modular creep compliance.

LABORATORY TESTING PROGRAM

Material

The soil used was a uniform fine sand with subrounded particles. The sand was washed through a No. 30 sieve to produce a grain size distribution with a median grain size of 0.49 mm and a uniformity coefficient of 1.4. The specific gravity of the sand particles was 2.64. The maximum and minimum void ratios of this sand, in a dry state, were approximately 1.23 and 0.59 respectively (2).

A sodium silicate grout (SIROC) was chosen as the chemical grout. SIROC is a three-component system;

its base chemical is a modified sodium silicate that mixes with the reactant and catalyst solution known as SIROC No. 2 and No. 3 respectively. The manufacturer's formulation directions (8,9) indicate 18 different suggested mixes of the three solutions for obtaining different grout properties and gel times. SIROC mix sample No. 7 was chosen for this study. The proportions in percentage by volume of the three SIROC components and water were as follows:

Component	Proportion	Component	Proportion
SIROC No. 1	50	SIROC No. 3	9
SIROC No. 2	31	Water	31

Mix sample No. 7 was mixed at a temperature of 20°C and a gel time of 50 min. The SIROC has been shown to provide the best overall solidified mass (12).

Sample Preparation

Two sizes of samples were made. The majority were 5-cm-diameter by 10-cm-high cylindrical samples; however, some cake-shaped specimens of 8-cm-diameter by 3.2-cm-high were prepared for cyclic simple shear testing. The dry density of all the samples was 1.6 g/cm^3, corresponding to a relative density of approximately 90 percent. The sample fabrication followed the procedure recommended by Warner (12). It involved pouring a known quantity of dry sand into a waxed cardboard mold about one-third full of the fluid grout. The sand was slowly poured into the mold, and vibration was intermittently applied along the wall of the mold to ensure a uniform, grout-saturated, densely packed sample. Specimens prepared this way have been shown to yield unconfined compressive strengths comparable to those of core samples obtained from the field (12).

All samples were cured in a room with constant humidity (98 percent) and temperature (21°C). The cardboard molds were stripped off after 7 days of curing. The total curing period was 35 days. At the end of this curing period, samples were well solidified and ready for testing.

Types of Tests Performed

The elastic and viscoelastic behavior of the solidified samples was studied by conducting four laboratory tests: repeated uniaxial compression test, resonant column test, uniaxial creep test, and cyclic simple shear test. Table 1 gives information pertinent to the various tests. All tests were performed in a room with a constant temperature of 21°C. A more detailed discussion of the tests and their results follows.

TEST RESULTS

Repeated Compression Test

The conventional repeated triaxial compression test apparatus (6) was used to determine both the axial and tangential elastic (resilient) deformation of a specimen (5 cm diameter and 10 cm height). The rate of repeated loading was 20 cycles/min with a load-on time of 0.3 s/cycle. Because the material was very stiff and rigid (Figure 1), bonded strain gauges were used to ensure accurate response measurements. The strain gauges used for the measurement of both axial and tangential strain were SR-4, type FAE-50-12 SO (2.5-cm active length). Eastman 910 bonding cement was used to bond the strain gauges to the solidified specimen. Two gauges mounted at midheight were used in each of the axial and tangential directions so that average values of strain could be measured. Each pair of gauges was connected to a set of compensating dummy gauges to form a full bridge. The specimen was properly seated by gluing the ends to the base and cap with hydrostone paste.

Preliminary test results indicated that, to a stress level (applied axial stress/unconfined compressive strength) of approximately 40 percent or 621 kPa, the magnitude of elastic deformation appeared to be independent of the number of load applications (to as many as 12 000 load repetitions). It was therefore decided that multistress level, repeated loading tests could be performed on a given specimen. Tests conducted in this study had applied stress levels ranging from 104 to 621 kPa. At each stress level, 1000 load repetitions were applied. By averaging three sets of test results, the elastic axial and tangential strain values for various repeated stress levels were recorded.

Creep Test

Uniaxial creep tests were performed on cylindrical samples. Both the axial and tangential strains were recorded as shown in Figure 2. Again, bonded strain gauges were used to measure the change in strains with time under a constant load. A total creep time of 1200 s was chosen, and constant stress levels of 276, 552, and 828 kPa were used. Because the creep stress levels were less than 50 percent of the static strength of these specimens, no creep failure was anticipated. The primary creep behavior of the chemically solidified soil mass was adequately described within the 1200-s testing period.

Resonant Column Test

Cylindrical specimens were also used in the resonant column tests to determine the shear modulus and damping ratio. A detailed description of the resonant column apparatus and testing procedures is given by Hardin (3). By applying a forcing torque of given amplitude to the vibration end of the specimen about its axis, the resonant frequency of the soil column can be established. Calculations can then be made to compute, at a given shear strain, the shear modulus and damping ratio of the solidified soil mass. Resonant frequencies were established for various forcing torque amplitudes with axial loads held at 103.5 and 172.5 kPa respectively. For each axial loading condition lateral pressures of 0, 138, and 276 kPa were applied.

Cyclic Simple Shear Test

The cake-shaped specimens were used to determine the viscoelastic behavior of the chemically stabilized soil mass. Tests were performed by using the modified NGI simple shear apparatus (5). A sinusoidal shear displacement was applied to the specimen through a motor-driven, adjustable, eccentric cam. The sinusoidal shear stress was determined by measuring the variation of force transmitted to the base plate by the input motion. From the sinusoidal force and displacement traces of different frequencies (0.5 and 1.5 Hz) and amplitudes (0.01 and 0.05 percent), the variation of phase angle (δ) was computed. All specimens were properly seated and glued to the top and base plates with hydrostone paste. However, vertical pressures of 69 and 138 kPa were applied to ensure that no slippage would take place between the specimen and the plates.

ANALYSIS OF THE TEST RESULTS

Test results obtained from this study were analyzed to

provide preliminary information concerning the elastic and viscoelastic responses of the chemically stabilized sand mass.

Resilient (Elastic) Modulus and Poisson's Ratio

Elastic layered systems of analysis can be used to determine stress and deformation of a pavement system under moving traffic. Therefore, for the last 2 decades,

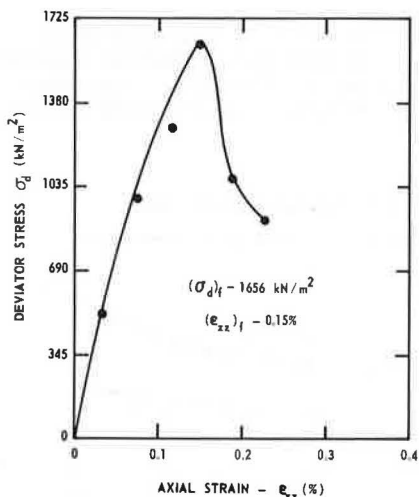Figure 1. Unconfined stress-strain relationship for treated sand.
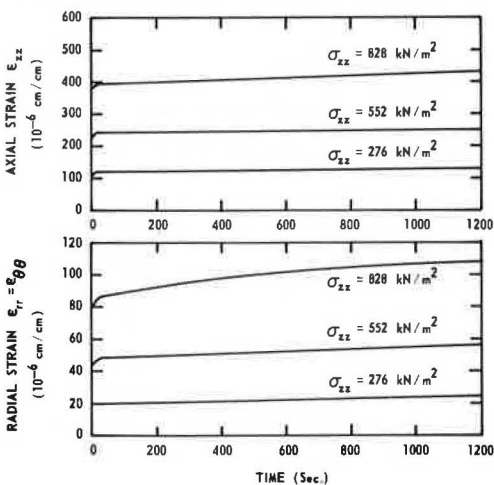

Figure 2. Uniaxial creep test results.

the resilient and fatigue properties of pavement component materials such as asphalt concrete, concrete, compacted clay, and stabilized soils have been extensively studied.

For an isotropic, linear-elastic material, only two elastic constants are needed to characterize its elastic behavior ($\underline{4}, \underline{10}$); and in the elastic layer system of analysis, Young's modulus (E) and the Poisson's ratio (ν) are often used. The uniaxial, repeated load test results were examined and the elastic parameters were computed assuming the material behaves as an isotropic, linear-elastic medium. Figure 3 shows the values of E and ν with respect to the applied repeated stress level. Although it is evident that both E and ν vary with the applied stress level, the variations are rather limited considering the wide range of applied stress levels used. Therefore, it appears that a linear-elastic constitutive law is a valid and adequate representation for describing the behavior of the chemically stabilized soil under traffic loading.

Shear Modulus and Damping Ratio

Both the resonant column test and the cyclic shear test results were used to compute the variations of shear modulus and damping ratio with shear strain. These relationships are shown in Figure 4. The shape of these curves is similar to that of curves reported by Seed and Idriss ($\underline{7}$) for untreated soils. The shear modulus and damping ratio are strain dependent: The smaller the strain is, the higher the modulus is and the lower the damping ratio is. Computation of the shear modulus and damping ratio values from cyclic simple shear tests was based on the linear-viscoelasticity theory of a steady-state sinusoidal response. The elastic shear modulus was computed from

$$G_1 = G^* \cos \delta \qquad (1)$$

where the symbols are as defined previously. G_1, the storage modulus, is the shear modulus associated with storage energy of the system ($\underline{10}$).

Also shown in Figure 4 is the shear modulus computed from uniaxial repeated loading tests.

Figure 5 shows the effect of frequency and vertical load on the variations of shear modulus and damping ratio with shear strain. Their influence is relatively small compared with the effect of shear strain.

Linear Viscoelastic Parameters

The data obtained from uniaxial creep tests were used to compute $\psi_R(t)$, $\psi_{Ez}(t)$, and $\nu(t)$ under the assumption that the material is linear viscoelastic ($\underline{4}$). Figures 6, 7, and 8 show the variations of these parameters with time and applied stress levels. For a wide range of applied stress levels (from 276 to 828 kPa), the variations of linear

Table 1. Laboratory testing program.

Type of Test	Test Conditions	Measurements	Parameters Determined
Repeated uniaxial compression	$\sigma_{zz} = 103.5, 155.3, 207, 310.5, 414, 621$ kPa $\sigma_{\theta\theta} = \sigma_{RR} = 0$	ϵ_{zz} $\epsilon_{\theta\theta} = \epsilon_{RR}$	E, ν
Resonant column	$\sigma_{RR} = 0, 138, 276$ kPa $\sigma_{zz} = 103.5, 172.5$ kPa Varying the amplitudes of a forcing torque	Resonant frequencies	G, D
Uniaxial creep	$\sigma_{zz} = 276, 552, 828$ kPa $\sigma_{\theta\theta} = \sigma_{RR} = 0$	$\epsilon_{zz}(t)$ $\epsilon_{\theta\theta}(t)$	$\psi_{Ez}(t)$, $\psi_R(t)$, $\nu(t)$
Cyclic simple shear	$\sigma_{zz} = 69, 138$ kPa Frequency = 0.5, 1.5 Hz $\gamma = 1 \sim 5 \times 10^{-4}$ cm/cm	$\tau(t)$ δ	D, G

viscoelastic parameters remain relatively small such that average values of these parameters may be approximated in linear-viscoelastic formulation. A linear-viscoelastic solution may be used to determine the permanent deformation of a pavement structure containing a chemically stabilized soil layer or to estimate the time-dependent deformation of a pipeline support foundation resting on a chemically treated soil mass.

SUMMARY AND CONCLUSIONS

Fine sand saturated with SIROC was studied in the laboratory to determine its elastic and viscoelastic behavior under different loading conditions. The mechanical behavior of the chemically stabilized soil is affected by curing temperature, moisture condition, and physical environment. Therefore, both the elastic and viscoelastic behaviors could differ greatly in different environments. Based on this study, the following conclusions may be stated.

1. The mixing of a chemical grout into a fine sand drastically changes the hydraulic and mechanical behavior of the soil. In fact, it forms a solidified soil mass that is much more hydraulically impervious and mechanically stronger and stiffer.

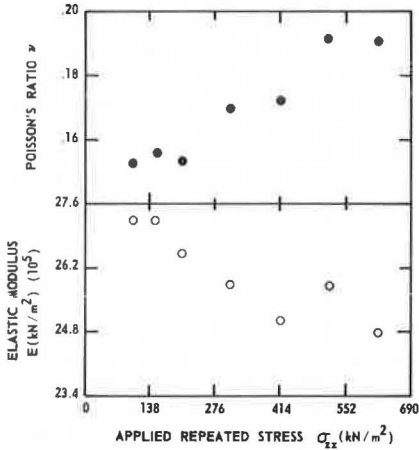**Figure 3. Elastic properties under repeated loading.**

Figure 4. Variations of damping ratio and shear modulus with shear strain.
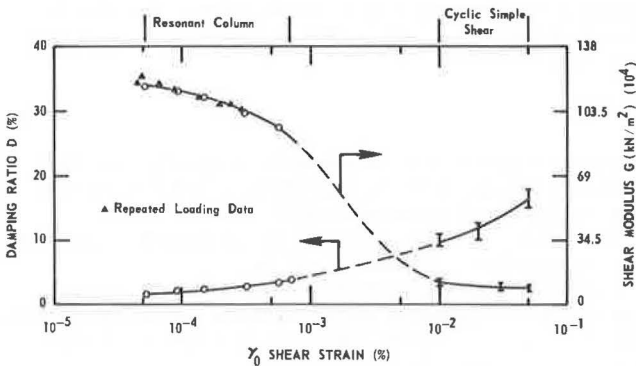

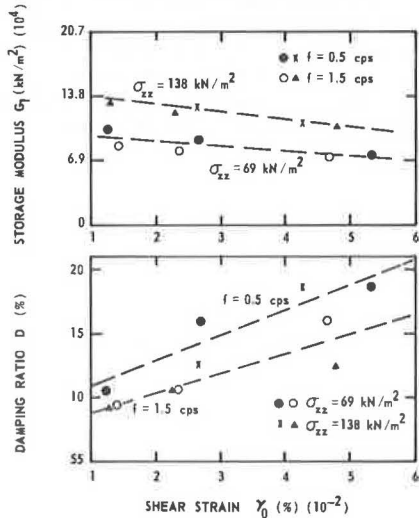**Figure 5. Cyclic simple shear test results.**

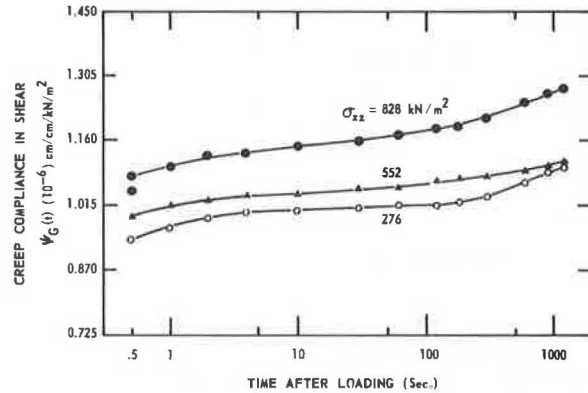**Figure 6. Creep compliance in shear for uniaxial creep test.**

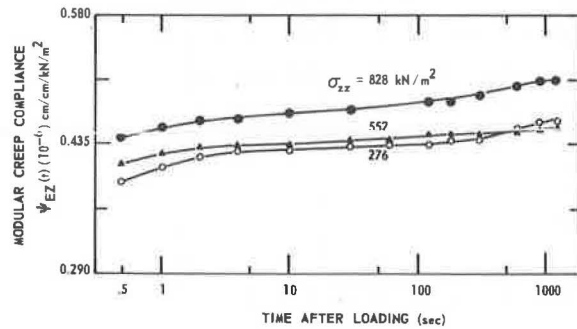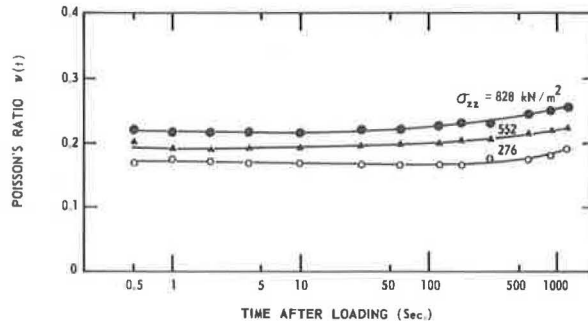**Figure 7. Modular creep compliance for uniaxial creep test.**

Figure 8. Poisson's ratio for uniaxial creep test.

2. The isotropic, linear-elastic constitutive law appears to provide a reasonable approximation for characterizing the chemically stabilized fine sand under moving load.

3. The magnitude of shear modulus and damping ratio under dynamic testing is dependent on the shear strain applied. The smaller the strain is, the higher the modulus is and the lower the damping ratio is.

4. For applied vertical stress levels of less than 50 percent, the chemically stabilized soil may be treated as a linear-viscoelastic medium for computing time-dependent deformation under sustained loading.

ACKNOWLEDGMENTS

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Electrochemical Hardening of Expansive Clays

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This paper is concerned with a large-scale swelling soil stabilization project that was undertaken by the Arizona Department of Transportation in July 1973. The site chosen for the study was approximately 146 m (480 ft) of the westbound lane on I-40 about 56 km (35 miles) east of Holbrook, Arizona. The stabilization technique used was electroosmosis and a 0.4 N KCl solution for inundating the clayey mass of the site. This resulted in an average 36 percent decrease in the percentage of swell and a 50 percent reduction in swell pressure of the Chinle clay. X-ray diffraction and electron micrograph data indicated possible causative factors of the reduction in swell pressure and percentage of swell.

The object of this research was to use existing electrochemical soil treatment technology and, if necessary, to modify portions of it for the purpose of creating a viable soil stabilization technique that could be implemented by Arizona highway maintenance personnel.

PHYSICOCHEMICAL ASPECTS

Base Exchange

The absorbed ions on a clay surface are present in a diffuse double layer. Each of the ions required to neutralize the charge on the particle surface oscillates because of Brownian motion; thus the ion is assumed to be oscillating in a cell, called an oscillation cell, adjacent to a charged area on the particle surface. Other ions from an added electrolyte may enter the oscillation cells or may remain in the external phase. A given ion with a large naked radius, i.e., K^+, will have a smaller hydration radius and thus be able to approach the charged surface closer than an ion with a smaller naked radius, i.e., Na^+, and a corresponding larger hydration radius. Thus, the potassium ion will, on the average, bond to the surface with a correspondingly greater energy than a sodium ion.

Such concepts have been evolved into mathematical models that depict the ion exchange as a stochastic process. These theories, in particular the one by Jenny, lead to mass-law or mass-action equilibrium equations. However, these models of base exchange imply that the exchange phenomenon is essentially a complex redistribution of ions both between an external phase and the ion swarm and also within the ion swarm. Unfortunately this process depends on several factors. The ion redistribution cannot be regarded as simple metathesis, precisely defined by a simple equation of the mass-law type. The mass-law equations often quoted in the literature, with regard to base exchange phenomena, must be considered as only approximations from which there may be considerable variation in unfavorable cases.

Because of the stochastic nature of base-exchange phenomena, any soil treatment based on these phenomena must also be regarded as a stochastic process. Of course the chance of successful treatment along these lines can be increased by preparing the external phase to be rich in the preferred ions, e.g., use a high-concentration KCl solution to treat the soil.

It must be borne in mind that an ion-exchange soil treatment can alter the physicochemical properties of the swelling clay and that such a treatment is one of the most effective ways to combat the problem over a relatively small, localized region on which an expensive in-place structure, say a pavement, rests.

The ability of a clay to absorb ions on its surfaces or edges is called its base or cation (anion) exchange capacity, which is a function of the surface chemistry of the clay and the size of the clay particles. Thus, the term base exchange is widely used even though hydrogen ions and even organic ions may be involved in the exchange.

LABORATORY WORK

The purpose of the laboratory work was to determine the expansive and swell characteristics of the untreated soil obtained from the test site.

Site Selection and Sampling

The field work for this study began in July 1973; the selected test site was at milepost 323.8± on the westbound

lane of Interstate 40, about 64 km (40 miles) east of Holbrook, Arizona. A plan view of the site is shown in Figure 1. Because this project was initiated as an implementation study, an engineering decision on pretest sampling techniques was made at the onset of the project.

The engineering decision was based on the following general considerations: Most problem areas of swelling clays traversed by a roadway are easily identifiable visually. After an area has been identified as a possible problem area, a quick economical way of sampling must be used. It is clear that, in a study oriented purely toward investigating soil phenomena, a sampling technique used over a selected region should be based on a random procedure. However, the primary goal was to use knowledge of past electroosmotic work and apply the same techniques, properly modified, to enable a rapid evaluation of the swelling problems of a given area.

The sampling procedure would be undertaken so that the drill rig would have to move only longitudinally; this would save a great deal of time and therefore money. Moreover, the rig was positioned as close to the center of the Interstate as possible without interfering with traffic in one travel lane of the westbound roadway. Twenty-six test holes were drilled, and samples were obtained to a depth of 4.6 m (15 ft) in the clay subgrade. The samples were transported to the Arizona Highway Department Materials Division where they were tested.

Soil Properties of Untreated Material

The average values for the Atterberg limits on the samples were liquid limit 39, plastic limit 17, and plastic index 22. The average percentage passing the No. 200 sieve was 70, and 20 percent was 2 μm in size. The specific gravity of the soil was 2.75. The soil would classify as a CL material based on the Unified Soil Classification system.

The expansive pressure of the untreated soil was determined for material passing a No. 40 sieve in an expansometer. The density of the soil was approximately 1700 kg/m³ (106 lb/ft³), and the moisture contents were 10 and 15 percent. The percentage of swell was determined on the untreated soil by using the expansometer and percentage of swell apparatus. The density was maintained at 1700 kg/m³ (106 lb/ft³), and the moisture contents were approximately 10 and 15 percent. The results of some of these tests are discussed below.

These tests indicated that the selected area is representative of a region with moderate swelling characteristics.

FIELD WORK

The scope of the field work of the electrochemical stabilization of Chinle clay was

1. To design, install, and operate a full-scale field test on the test section;
2. To sample the electrochemically treated section; and
3. To evaluate the ability of existing maintenance resources to carry out this work with a minimum of specialized personnel in attendance.

The highway field installation was designed to reproduce the simple electrode configurations used in previous studies. To this end, a design was chosen that incorporated the desirable characteristics of previous work with the realities of conducting the field work on a much larger scale and with relatively inexperienced personnel.

The electrical design was such that three sections of a 152-m (500-ft) test section could be electrified simultaneously with the same voltage gradient.

The electrode configuration for the whole test site is shown in Figure 2. The center section has, as anodes, vertical No. 8 rebar about 1.5 m (5 ft) long. This is in contrast to the other two sections, which have horizontal anodes made up of 6.1-m (20-ft) sections of No. 8 rebar welded together end to end to form the anodes. After the horizontal anodes were formed in this manner they were manually placed into a previously prepared 1.2-m-deep (4-ft) trench.

The cathode was formed of 6.1-m (20-ft) sections of No. 8 rebar welded end to end and then carefully lowered into the trench.

The electrode configuration was designed to be easy to assemble and yet to be an effective item. Although many sophisticated design patterns exist, the simple design used in this project requires a minimum amount of expertise for proper installation at the site.

As a result of past field tests using electrochemical methods for stabilization of Chinle clay, the field installations were designed such that the clay would be treated only to a depth of 0.9 m (3 ft). This estimate was based on laboratory tests that showed that, if the clay were effectively treated to this depth, the site could be judged as stabilized. Moreover, because of previous study results, it was obvious that to treat the clay to a greater depth than 0.9 m (3 ft) would result in an overkill in the first 0.9 m (3 ft).

In order to properly suffuse the soil pores with the KCl solution it was decided to drill 15.2-cm (6-in) diameter auger holes on 2.4-m (8-ft) centers, approximately 1.7 m (5.5 ft) deep throughout the test section. The positioning of the auger holes was based on previous studies. There was a total of 285 of these auger holes positioned throughout the site.

To prevent caving of the blowsand subbase material each auger hole was sleeved with 15.2-cm (6-in) O.D. steel pipe, 68.6 cm (27 in) long, topped with a 25.4-cm-diameter (10-in), 2.5-cm-thick (1-in) steel plate. The plate had a 4-cm (1½-in) hole in the center to permit introduction of the KCl solution into the auger hole. The 25.4-cm-diameter (10-in) steel plate was obviously necessary to give a stable platform for the steel sleeves and to provide a sufficiently rigid surface for vehicular traffic.

The 285 protruding 25.4-cm-diameter (10-in) steel plates presented a potential traffic problem and made it necessary to countersink each auger hole with a concentric 25.4-cm-diameter (10-in), 2.5-cm-deep (1-in) depression. The countersinking operation was accomplished easily by welding to the top of the auger drill stem one of the 25.4-cm (10-in) steel plates.

After this was accomplished, six old drilling teeth were set flat against the bottom of the plate, spaced about 60 deg apart in a symmetrical pattern and then welded into place. The teeth then provided an abrasive surface for the countersinking operation.

Each of the 285 auger holes was sleeved down 43 cm (17 in) below the asphalt surface. The sleeving did prevent the base course from extensively caving in and KCl solution from wetting the base course. However, there was some caving and wetting. District 4 personnel suggested 69 cm (27 in) of base course material as a best estimate to use throughout the site.

The total drilling and sleeving operation required approximately 2 workweeks of the drill crew's time.

Upon completion of the drilling and sleeving, the electrical installation was initiated. This operation consisted of trenching for horizontal electrodes and drilling for a section of vertical anodes. This operation required about 3 days with a crew of five.

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District 4 personnel mixed and placed solution by using a modified 9.5-m^3 (2500-gal) capacity goose neck water truck. The water and KCl were mixed so that a 0.4 N solution was obtained.

At the outside ambient water temperatures in the Holbrook area at that time of year [May through August, 23°C (73°F)], this was the maximum amount of KCl soluble in water.

After mixing in the tanker truck the 0.4 N KCl solution was placed into the auger holes under pressure by using ordinary gasoline nozzle and hose fixtures leading from the solution truck. In this manner the auger holes were filled twice a day for a period of approximately one month, in which time about 136 m^3 (36 000 gal) of 0.4 N solution was introduced into the site region.

It was determined early in the project that prior to the introduction of the electric field the solution would be delivered to the site for about 30 to 35 days. With this procedure the clay could be presaturated with the KCl solution by utilizing the relative ease with which an electrolyte moves through the clayey material and simultaneously the expense of running an electrical generator during the initial soil saturation period could be avoided. During this period about 64 m^3 (16 894 gal) of solution was delivered to the site.

On July 8, 1973, the 60-kW dc generator was started, and an overall current of about 400 A was recorded with a voltage gradient of about 0.1 V/cm. On the average about 133 A flowed through each of the three sections during the field test. After approximately 30 days of continuous operation, an electrode polarization phenomenon was noted, which caused a rapid power loss. Electrical operations were discontinued immediately thereafter.

At this time, a total of 136 m^3 (36 000 gal) of solution was delivered to the site. After the electrical system was shut down, sufficient soil samples were obtained and shipped to the Materials Services Laboratories for testing and evaluation.

RESULTS AND ANALYSIS OF
FIELD TEST

After completion of the field test operations on August 16, 1974, soil samples were gathered for the following laboratory analyses:

1. X-ray diffraction analysis,
2. Transmission electron microscope (TEM) techniques, and
3. Expansive pressure and percentage of swell tests.

Data obtained from X-ray diffraction analysis yield information on the effects on the lattice structure of the clayey soil. TEM techniques identify any effects on the soil structure arising from the electrochemical treatment that are too gross (i.e., macromicro effects) on the crystalline structure for the X-ray diffraction to resolve. Results of the pressure and swell tests give data on the effects of the electrochemical treatment on the engineering properties of the soil mass.

X-Ray Diffraction Analysis

Identification of the crystalline soil minerals was performed by X-ray diffraction analysis. Both clay and nonclay minerals that are crystalline have a long-range order in their atoms or ions. By bombarding a mount containing the sample with high-energy X-rays, the spacing of the crystalline structure can be determined from the wavelength of the X-ray.

The X-ray data clearly indicated that the clay had

been affected by the electrochemical treatment; moreover, these data are supported by electron micrographs and tests on the engineering properties of the oil. Based on both electron micrograph and X-ray diffraction data, it appears that the stacking geometry of the clay particles was modified by the electrochemical treatment.

Because no additional minerals were found in the treated materials as opposed to the untreated, it is not possible to conclude that the smectites were transformed into an illiticlike clay. Thus, the reduction in the diffracted X-ray intensity of the treated clay and the reduction in the stacking number, as shown by the electron micrographs, indicate that the treatment affected primarily the 0-0 bonds between the basic units but did not cause significant diagenesis of the clay. Moreover, because the engineering properties of the clay are highly dependent on the nature and stability of the interlayer environment between the unit cells, the soil tests reveal that the electrochemical treatment of chinle clay also alters the interlayer complex, which is composed of inorganic ions, water, and even organic complexes, and interferes with the basic 0-0 bonding between the unit cells.

Transmission Electron Microscope
Analysis

Bentonite types of materials were observed by using transmission electron microscope techniques. Standard bentonite minerals (Wyoming bentonite and USP bentonite) were compared to pretest clay samples (40133 and 40188) and posttest clay samples (40415A).

An example of electron diffraction from the untreated and treated clay is shown in Figure 3.

The micrographs seem to show that the treated sample has mineral flakes that are thinner than those of the pretest samples. This is presumably because the nature and stability of the bonding between the successive basic structural units (Lamella) have been altered causing the stacking of the units to change geometrically. This is supported by the X-ray diffraction data, which showed the untreated material to have a smectite peak of greater intensity than the treated material (Figures 4 and 5).

Results of Data Analyses

All of the evidence clearly indicates that the crystalline fabric of the clay was affected by the treatment. This is substantiated by the electron and X-ray diffraction patterns, which for the pretreated material depicted a thick layered structure. This is noted by the wider diffraction rings or high-intensity X-ray diffraction peaks of the pretreated material as opposed to the relatively thin diffraction rings or low-intensity X-ray peak obtained from the treated material. The wide rings or high scattered intensity is characteristic of the thick crystal. If the crystal is thin, a pattern of thin diffraction rings or low intensity is obtained.

Expansive Pressure and Percentage of
Swell of Posttest Samples

Upon completion of the electrochemical treatment, samples were obtained and returned to the Materials Services Laboratory in Phoenix where tests of expansive pressure and percentage of swell were performed in the expansometer and percentage of swell apparatus designed by Arizona State University.

The data indicated that the expansive pressure decreased somewhat more than 50 percent and the percentage of swell decreased by more than 36 percent.

SUMMARY

This study focused primarily on implementing electrochemical soil treatment procedures developed in previous FHWA-sponsored work.

Initially a site for the field work was chosen along a section of I-40 that was experiencing swelling problems. The site was along a roadway of high traffic volume so that traffic control problems during an operation of this sort could be fully considered, inasmuch as this method of soil stabilization is intended to become part of Arizona DOT maintenance procedures.

In this work the primary considerations were

1. The relative ease of field implementation of established electrochemical soil treatment technology and

2. The effectiveness of soil treatment of electrochemical methods when the field operation is conducted largely by unskilled and semiskilled maintenance personnel.

Figure 1. Plan view of site.
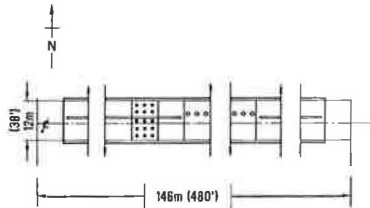
Figure 2. Electrode configuration for site.
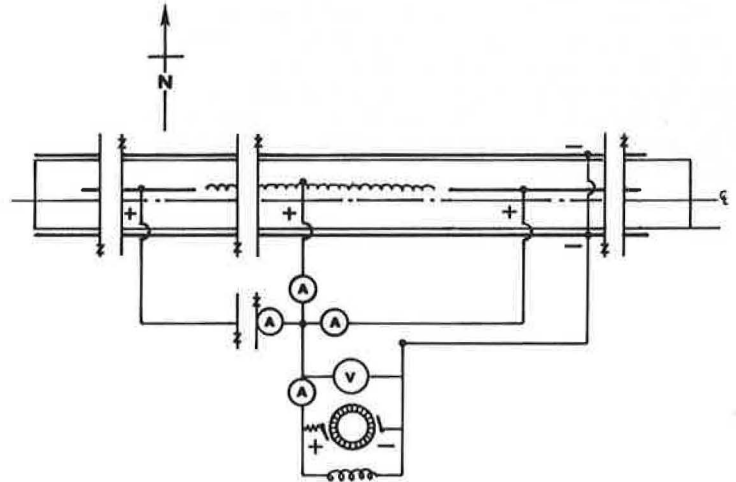
Figure 3. Electron diffraction from (a) untreated and (b) treated clay.
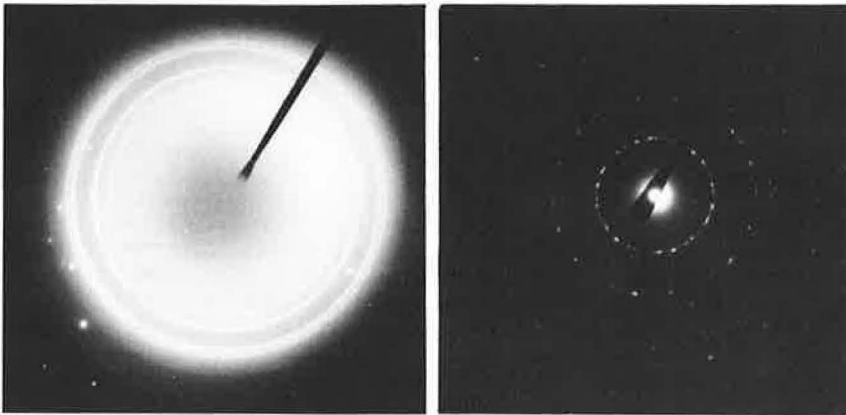

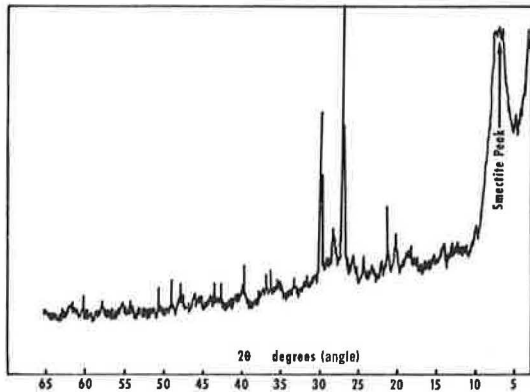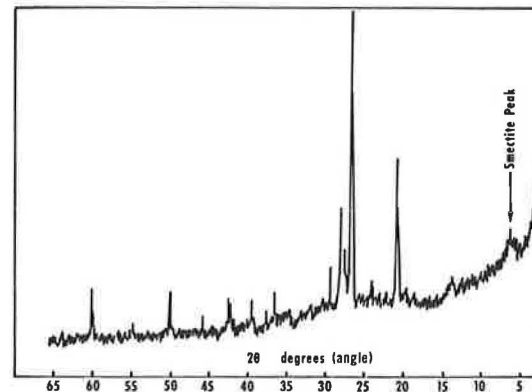Figure 4. X-ray diffraction pattern of untreated clay.

Figure 5. X-ray diffraction pattern of treated clay.

Based on this study it can be stated that the use of electrochemical soil treatment technology can be effectively implemented by using existing work force and material resources of the Arizona DOT, supplemented by minor purchases of specialized hardware and software.

ACKNOWLEDGMENTS

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the state of Arizona or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

Stabilization of Sands by Asphalt Emulsion

K. P. George, Department of Civil Engineering, University of Mississippi

This laboratory study investigates whether sands and sand-clay aggregates are amenable to stabilization by asphalt emulsion. Marshall tests were used to evaluate the stability of emulsion-treated sand. Microscopic examination of the treated aggregates revealed the nature and extent of bitumen coating and spreading characteristics of bitumen binder. The results show that well-graded sands with sufficient silt-clay material respond well to emulsions. Portland cement in trace quantities is required, however, to improve the water susceptibility of the mixtures. Because of its superior bonding to siliceous aggregate and its rapid setting tendencies, cationic emulsion is preferred in sand stabilization. Recommendations concerning mixing and compaction moistures are presented. This paper shows that either too little or too much mixing time will result in less than optimum coating. A hypothesis concerning the mechanism of emulsion bonding to aggregates—dry as well as prewet—is proposed.

Bitumen, the most widely used binder for road surfacings, does not carry an electrical charge; its adhesion to aggregates is, therefore, predominately mechanical. Because asphalt emulsions are designed to carry an electrical charge (positive or negative), they are preferentially absorbed by aggregates of opposite charge, which establishes an electrochemical bond.

Early bituminous emulsion contained anionic emulsifying agents such as iron oleate and could only be used with dry aggregates and in good weather. The unreliability of the weather led to the development of the cationic emulsions. Their development was based on the concept that the positively charged surface of the minute globules of asphalt would be strongly attracted to the surface of electronegative asphalt.

Asphalt treatment of sands or aggregates with asphalt emulsions offers several advantages over hot-mixed asphalt treatment. The most important advantage is that mixing can be done at ambient temperatures either in place or in portable plant mixers, which allows the use of low-cost aggregate with a considerable saving in petroleum products. According to the U.S. Bureau of Mines, the total amount of energy-producing petroleum products used in the road-building industry reached a staggering 5 174 000 m³ (1.367 billion gal) in 1973.

One shortcoming of emulsion-treated mixes (ETM), however, is that strength development is limited by the rate of water loss in the structure. Ultimate cure condition may be achieved by 120 days of curing.

Because emulsified asphalt has been used on a limited scale only, insufficient data are available on the response of emulsion to various aggregates; for this reason, select aggregates have, for the most part, been used in roads during the last 2 decades. For instance, of the 30 projects that Finn and others (7) surveyed in seven states, only seven of the bases included sandy or fine-grained soils. Kerston and Pederson and Korfhage (10, 11) reported poor performance with SS-1 in Minnesota loess and a poor-quality aggregate. Scrimsher and others (16) reported that two cold asphalt emulsion mixtures—one dense graded and the other open graded—placed as 25-mm (1-in) overlays on an existing pavement showed noticeable raveling and the surface developed a rough ride. Recently, Meier (13) reported three projects in the Northwest where fine sand was stabilized with slow-setting grade emulsified asphalt. Again, the performance of two of the three projects was less than satisfactory. One problem encountered involved the difficulty in aerating the mixture, a circumstance that was attributed to the finer gradation. Nevertheless, successful use of emulsified asphalt in sand and cohesive graded sands has been reported (3, 8, 12). As Bratt (2) remarked, however, numerous problems exist, for example, finding a specification that guarantees consistent behavior of emulsions. The numerous failures reported in the literature suggest the lack of a system for evaluating the amenability of a soil to stabilization with asphalt emulsion. To evaluate this problem, facts were sought regarding the following areas:

1. Desired characteristics of aggregate,
2. Type of emulsion best suited for sandy materials,
3. Desired moisture for mixing, compaction, or both,
4. Sensitivity of ETM to water action, and
5. Mechanism of emulsion-aggregate bonding.

MATERIALS AND METHODS

Soils

Four sandy soils were selected for study. The percentage of fines (which refers to the amount of material passing the No. 200 sieve) of these soils varies widely—5, 10, 17, and 20 percent—as does the uniformity coefficient. Soil K37 (Unified Soil Classification symbol SP), a rounded sand with 5 percent fines, is from Baxter, Mississippi; K38 (SW-SM) with 10 percent fines, K40 (SM) with 17 percent fines, and K42 (SM) with 20 percent fines come from the sand-clay topping used in the base construction of the Miss-6 bypass at Oxford.

Asphalt

Emulsified asphalts of slow-setting grades CSS1 and SS1 were used. The properties of the asphalts, as furnished by the manufacturer, are given in Table 1.

Specimen Preparation

Air-dried aggregate was moistened with water before mixing with emulsion. The ingredients (aggregate and emulsion) were hand-mixed for a minute, after which they were mixed by machine until the aggregate was evenly coated. To facilitate even coating, excess moisture (2 to 4 percent) was added during mixing and subsequently evaporated by a blower.

Marshall test specimens 64 mm (2.5 in) high and 102 mm (4 in) in diameter were prepared according to ASTM D 1559, except that 75 blows were applied on both sides instead of 50. These specimens were air dried for 7 days at 50 percent relative humidity and 26°C (73°F) before testing at a loading rate of 51 mm/min (2 in/min). The vacuum soaking test developed by Dunning and Turner (5, 14) was adopted here for evaluating the moisture sensitivity of the mixtures. The tentative ASTM testing method (18) was followed to compact the prepared mixture by using the gyratory testing machine (GTM).

RESULTS AND DISCUSSION

Aggregate-Sand

Important properties of aggregate and sand from the standpoint of emulsion stabilization include (a) mineralogical composition or surface chemistry or both, (b) particle shape and surface texture, and (c) gradation.

The popular theory that siliceous aggregates tend to be electronegative and carbonates of calcium, magnesium, or aluminum tend to be positive has been refuted by recent research studies. Sherwood (17) reported that all aggregates he studied had negative surface charges. By determining the zeta potential of 87 naturally occurring aggregates obtained worldwide, the Highway Chemicals Department of Armak (6) reported negative surface charges in all of those aggregates. Of those 87 aggregates, 34 were limestone and dolomite, and the remaining 53 were various siliceous types. In naturally occurring aggregates, therefore, cationic emulsion is preferred over anionic emulsion.

Microscopic studies as well as Marshall stability values of the various sands reveal that sands exhibiting certain properties are unsuitable for emulsion stabilization. Polished or smooth-textured sands from water-transported deposits and poorly graded sands with few fines are unsuitable. K37 is a typical sand having these attributes. The main problem encountered in K37, especially with cationic emulsion, was that either too long a mixing time (more than 1 min of hand mixing) or drying-

back accompanied by gentle mixing resulted in a reduction of coating or stripping (Figure 1b). Lack of sufficient fines in aggregates could be another reason for stripping. To substantiate this point, soil K37 was modified by adding 4 percent by weight of silty clay fines from soil K38. As shown in Figure 1c, the coatability and mixing stability of modified K37 are substantially improved. Similar results have been obtained by Kallas, Puzinauskas, and Kreger (9), who recommended filler material to improve the stability of hot asphalt mixtures.

The use of emulsion in soil stabilization is restricted to fine granular soils, sands, and silty sands low in clay content. The higher the fines content is (clay in particular), the greater will be the difficulty of obtaining sufficient emulsion mixing stability. For example, if the moisture in soil K42 during mixing is less than ideal, the emulsion will ball up; the resulting ETM exhibits only a fraction of the strength that can be attained under ideal conditions. Although in the range of 5 to 20 percent fines the dry stability increases with fines, as indicated by retained strength after soaking (Figure 2), the resistance to moisture attack decreases. One cautionary note is that neither sands with high fines content (> 20 percent by weight) nor those uniformly graded with low fines content (< 8 percent by weight) should be considered for emulsion stabilization.

Emulsion Type: Anionic or Cationic

Because siliceous aggregates (for that matter, most other highway aggregates) are electronegative, cationic emulsion should be preferred over anionic emulsion. The results of a comparative study of cationic and anionic emulsions in several sands and sandy soils are presented below. The cationic mix requires 1 to 2 percent more moisture than its anionic counterpart. The additional moisture is necessary in cationic emulsions to inhibit premature breaking of emulsion and to promote uniform coating.

In a series of tests, Marshall specimens were molded with moisture content 1 percent less in the anionic mixes than in the cationic mixes. The stability values after 7, 14, 21, and 28 days of air curing showed that the dry stability of anionic-sand mixtures exceeded the cationics. Table 2 gives a comparison of the stability values of specimens of both emulsions after 7 and 14 days of air drying with those of specimens after air drying and vacuum soaking. Contrary to what has been observed in air-dried mixtures, the anionic mixtures, upon soaking, became highly sensitive to water attack as indicated by the lower soaked stability values. The explanation for this anomalous result could be that, because cationic emulsion is positively charged, it is electrochemically bonded to the negatively charged sand grains. The bond between the anionics and the sand grains is purely physical and results simply from the drying process. Therefore, the latter mixtures are more susceptible to water attack than the former, in which stripping action is inhibited because of strong bonds.

Insofar as curing is concerned, cationic emulsion performs better than the anionic one. Experimental results, although not reported here, illustrate that cationic emulsions dehydrate and cure at a considerably faster rate than do anionic emulsions. This advantage is important when aggregates containing variable and excessive amounts of moisture must be used.

In summary, in sandy soils cationic emulsions offer many benefits not available with the conventional anionic emulsions and perhaps cutback asphalts. These benefits include faster setting rates, superior stripping resistance, and, more important, greater resistance to the

deleterious effects of moisture. In general, cationic asphalt emulsions adhere well to a wider variety of aggregates of different chemical composition than do anionic emulsions.

Factors Affecting Emulsion Mixing

Moisture Content

A literature review indicates that moisture contents for mixing and compaction are controversial, to say the least. A survey of emulsion usage in the 50 states indicated that an overwhelming majority of the states favored mixing at the Proctor optimum and compacting slightly below the optimum. Only three states and a few other agencies such as the Bureau of Indian Affairs (13) favored moisture above optimum for mixing.

The moisture for compaction is evaluated in accordance with the following tests:

1. Proctor moisture density test (ASTM D 698-66T) using untreated soil;
2. Moisture-density relation using gyratory testing machine with 690-kPa (100-psi) vertical pressure, 104-kPa (15-psi) air roller pressure, and 3-deg gyratory angle [the density resulting from these settings and 12 revolutions in the GTM machine is shown to be similar to that obtained in the field with a steel wheel roller (15)]; and
3. A factorially designed experiment in which the emulsion and water content were simultaneously varied (by plotting the Marshall stability values in an x-y plot of emulsion versus water content, an optimum stability value can be detected and the formulation at which the best properties are found may be derived).

The results are given in Table 3. The Proctor density of untreated soil in all of the soils is lower than the GTM counterpart; the difference tends to decrease with the increase in fines content. If GTM density is obtainable in the field, the use of Proctor density for field control is not justified. The Proctor optimum moisture, however, compares favorably with the liquid content required for optimum GTM density. Had the Marshall stability been the criterion, the liquid content requirements of soils K37, K38, and K40 are 8, 10, and 12 percent respectively, which agrees with the GTM liquid contents. It is recommended that compaction liquid content be equal to the liquid content for optimum density in GTM.

The amount of moisture present during mixing also has a decided influence on the final distribution of emulsified asphalt in the soil. When the percentage of fines exceeds approximately 5, at the moisture content recommended for maximum density emulsion, the cationic type, especially, breaks, causing mixing difficulties. On the other hand, increasing the moisture by 3 percent and prewetting the aggregate or diluting the emulsion with water facilitates mixing and distribution of binder (Figure 3). As expected, those mixes, when compacted after drying-back to the compaction moisture content (as per Table 3), exhibited higher stability values in the dry state as well as after vacuum soaking (Figure 4). In the light of these results, a recommendation is in order that soil and emulsion be mixed at a moisture content 2 to 3 percent above that specified for compaction.

Uniformity of Mixing

Insofar as mixing is concerned, the researcher is faced with a dual problem. First, how well does the selected emulsion coat the given aggregate during mixing operations? Although slow-setting emulsions are used in this investigation, they, especially cationics, failed to mix with the sand aggregates in dry state. The photographs in Figure 3 clearly show that thoroughly moistening a soil before the emulsion is added facilitates mixing. Presented below is a discussion of how prewetting improves the uniformity of mixing. During mixing, the fine fraction of sand, if dry, preferentially absorbs the water from the emulsion, causing premature breaking of the emulsion. Besides, if the capacity of the aggregate surface for absorption is large, the amount of emulsifier transferred may be sufficient to destroy the stability of the emulsion. The fresh bitumen that precipitates, instead of coating the aggregates, tends to ball up. In dry aggregates, break can also be produced by coagulation. In cationic emulsion the colloidal particles carrying charges of the same sign (positive) tend to repel each other. This is one reason that they do not settle. These charged particles in the cationic emulsion, upon mixing with dry sand, attract charged particles of opposite sign (negatively charged finer particles), and the charges on the emulsion particles are neutralized. Many such particles then unite to form a precipitate, a process called coagulation. The instant ball-up of emulsion in dry or sparingly moistened sand may well be a coagulation phenomenon. With anionic emulsion, however, because of similar charges on the adherend and adherent, coagulation will be absent; the bitumen, therefore, spreads more uniformly yet is only weakly absorbed by the aggregate surface.

On the other hand, when emulsion is added to moistened sand, the asphalt droplet establishes a bond with the aggregate through water molecules as shown in Figure 5b. However, because the affinity of the surface active emulsifying agent is stronger than its affinity for water, in time the former will displace water from the aggregate surfaces irreversibly and establish electrochemical bonds. The schematic in Figure 5 shows how the surface active agent links the bitumen binder and the aggregate.

In summary, it may be concluded that moisture content before mixing, percentages of fines, and porosity of the aggregates, in that order, have the greatest influence on the aggregate coating with emulsion.

Having obtained a uniform mix, the researcher is now confronted by the second aspect of the dual problem: how well the asphalt remains coated (or mixing stability) under adverse conditions of mechanical abrasion during mixing or compaction. If, during the process of mixing, the cohesive forces in the asphalt are greater than the adhesive forces at the asphalt-aggregate interface, the mechanical stresses of mixing cause the asphalt phase to assume a minimum surface area and ball up. Stripping of asphalt due to vigorous machine mixing is shown in Figure 1b. Scrimsher and others recommended that the mixing time be limited to 2 to 15 s for open-graded mixtures and 15 to 20 s for dense-graded mixtures. Depending on soil texture, however, it is recommended that laboratory mixing time be 1 to 2 min.

Moisture Susceptibility of Emulsion-Treated Sand

In spite of high air-dry stability, ETM is highly susceptible to water attack, as indicated by the loss in stability after vacuum soaking (Table 2). The water sensitivity of soil-emulsion mixtures was also measured by the loss in tensile strength, as shown by the split tension test after vacuum soaking.

As stated previously, adding moisture to the aggregate before blending with emulsion improves soaked stability

of all the sand-emulsion mixtures. This improvement is due to the fact that uniformity of mixing is enhanced. To further improve the soaked stability of sand-emulsion mixtures, lime and, subsequently, cement were added in trace quantities. So that spreading of the cement would be uniform it was dry mixed with the sand before water and emulsion were added, in that order. Stability values after vacuum soaking (Figure 4) show that cement treatment, 1 to 1½ percent by weight, further improves the water susceptibility of ETM. For example, with 1½ percent cement, K37 retained 77 percent of its stability after soaking. Cement or lime enters into a surface reaction whereby the hydrogen, sodium, potassium, and so forth on the aggregate surfaces are replaced by calcium from

Figure 1. (a) Mix stability of K37 and cationic emulsion hand-mixed for 1 min, (b) asphalts stripped when machine mixed further, and (c) increased mix stability in K37 when 4 percent fines added.
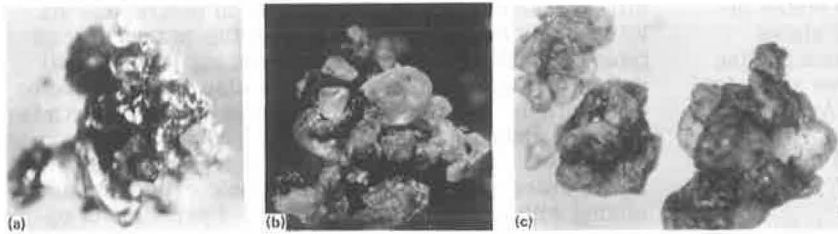
Figure 2. Dry strength, strength after soaking, and moisture absorbed related to percentage of fines.
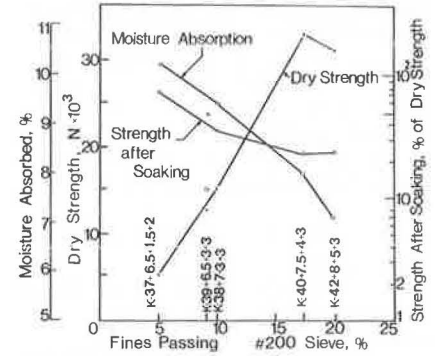
Note: (1 N = 0.225 lbf).

Table 1. Properties of emulsions.

Property	Cationic CSS-1	Anionic SS-1
Emulsion		
Furol viscosity at 28°C	35 to 65	67.7
Settlement, 5 days, percent	—	0.6
Cement mixing, percentage broken	0.1	0.1
Residue (by distillation), percent	64.0 to 68.0	67.7
Base asphalt		
Penetration at 28°C, 100 g, 5 s, cm	149 to 180	146
Solubility in CS_2, percent	—	99.95
Ductility at 28°C, 5 cm/min, cm	100+	40+

Note: 1°C = (1°F - 32)/1.8; 1 g = 0.035 oz; 1 cm = 0.39 in.

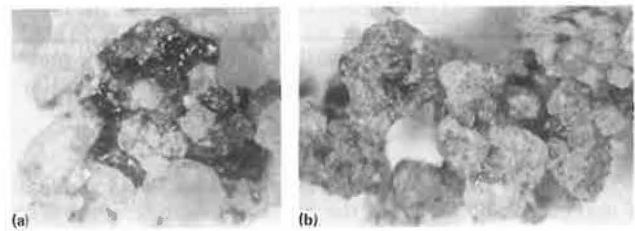Figure 3. Increased mix stability with increasing mixing moisture.

Table 2. Comparison of Marshall stability values of air-dried samples with those of air-dried and vacuum-soaked samples.

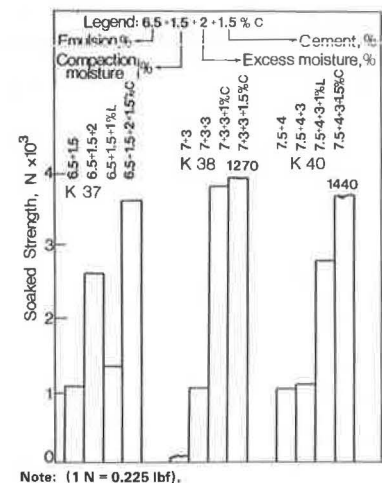| Soil No.	Emulsion Type	Percent	Moisture (%)	7-Day Air Cure	7-Day Air Cure and Soak	14-Day Air Cure	14-Day Air Cure and Soak
				Stability (N)			
K37	Cationic	5.5	2.5	4 180	1710	4 335	2200
	Anionic	5.5	1.5	7 070	135	10 430	845
K38	Cationic	7	3	10 365	0	12 455	510
	Anionic	7	2	23 710	380	19 840	735
K40	Cationic	7.5	4	23 350	955	25 330	1335
	Anionic	7.5	3	24 485	710	26 890	510

Note: 1 N = 0.225 lbf.

Figure 4. Marshall stability after vacuum soaking, cationic emulsion.

Note: (1 N = 0.225 lbf).

Table 3. Moisture-density relations using various test techniques.

Soil No.	Proctor on Untreated Sand Density (kg/m³)	Proctor on Untreated Sand Moisture Content (%)	GTM Compaction on Treated Sand Density (kg/m³)	Liquid Content (%) Cationic Emulsion	Liquid Content (%) Water	Factorial Exponent Liquid Content (%)
K37	1738.0	10.0	1986.3	6.5	2.0	5.5/2.5
K38	1736.6	12.0	1915.8	7	4.3	7.0/3.0
K40	1906.2	10.0	2050.4	7.5	4.5	7.5/4.5
K42	1949.4	10.7	2037.6	8	4.5	—

Note: 1 kg/m³ = 0.62 lb/ft³.

the cement. The calcium-rich surface formed is shown to react with long chain organic acids to form water-resistant surfaces.

Mechanism of Emulsion Bonding

Asphalt emulsions are intimate mixtures of two immiscible liquids (asphalt and water). An emulsifying chemical, compatible with asphalt and water, has to be added to make the system stable. These chemicals are characterized by having a long hydrophobic (water repelling) hydrocarbon tail and a polar hydrophilic (water attracting) head. As schematically shown in Figure 5a, the hydrophobic tail of the molecule is oriented toward the nonpolar medium, such as asphalt. The surface active molecule acts as a bridge between the two phases, lowering the surface energy and making any transition between the phases less abrupt.

Because cationic emulsions are more favored in stabilization than are anionic emulsions, the following discussion focuses on cationic emulsion and aggregates. When an emulsified asphalt is mixed with an aggregate, the binder is precipitated by the positively charged group of the emulsifier attaching itself to the surface of the aggregate; the surface active agent thus forms a link between the bitumen binder and the aggregate (Figure 5c). Because cationic emulsifying agents carry a positive charge, they adhere to negatively charged aggregates; but in the case of positively charged alkaline surfaces, they are converted into negatively charged areas by the free acid present in the cationic emulsion.

A hypothesis concerning how the emulsified bitumen establishes bond in moistened aggregate was presented earlier and is shown in Figure 5b and 5c.

Another question that remains unanswered is how the stability of ETM increases with age. It has been asserted that in the beginning the bond between the bitumen and the aggregate is partly mechanical and partly electrochemical. Upon curing, however, the moisture content decreases, and the bitumen establishes stronger electrochemical bonds. The strength increase with curing may, therefore, be attributed to added bonds. Second, as the water and solvent evaporate from the mix, either before compaction or after compaction during the curing process, it becomes possible for the asphalt to spread out and adhere to the surfaces of the mineral particles. The microscopic pictures shown in Figure 6 were taken of K40 with 7.5 percent emulsion and 4 percent water after air drying and subsequent drying under a lamp. The first picture was taken 15 min after mixing, the second one after 3 min of additional drying under a heat lamp. These pictures strongly suggest that, as the water is evaporated from the mixture by moderate heating, the bitumen in the crevices is drawn into coatings on the soil grains by the force of surface tension. The water-resistant films prevent the soil from taking up an amount of water sufficient to cause it to lose stability. Permanent stabilization is thereby accomplished.

CONCLUSIONS

1. Well-graded sands and soils, with fines contents (smaller than 74 μm) between 8 and 20 percent, are amenable to stabilization by emulsion. Uniformly graded sands can be made to perform satisfactorily if blended with silt-sized fines.

2. Because of the superior adhesion property, sands stabilized with cationic emulsion show high early strength and exhibit superior soaked stability. The mixing stability of anionics, however, excels that of cationics.

3. It is recommended that compaction liquid content be equal to the liquid content for optimum density in GTM. Mixing moisture of 2 to 3 percent above that for compaction will ensure adequate mixing stability to give a uniform distribution of binder.

4. The presence of moisture in a soil before the emulsion is added facilitates mixing. Excess silt and clay in a soil adversely affect uniformity of mixing. A reasonable mixing time is necessary to ensure proper coating of the aggregate; too long a mixing time, however, may result in stripping.

5. Portland cement in trace quantities (1 to 2 percent), acting as a stabilizing agent, greatly enhances the soak-stability of sand-emulsion mixtures without the accompanying brittleness of cement-stabilized material.

Figure 5. (a) Asphalt droplets suspended in water. Surface active agent linking (b) bitumen binder and (c) aggregate.
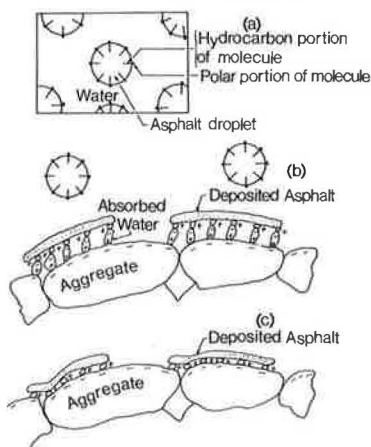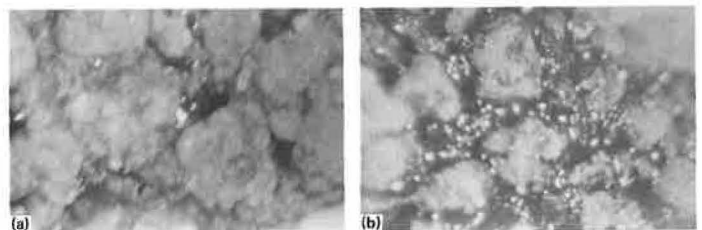

Figure 6. Bitumen spreading due to evaporation and moderate heating after (a) 15 min of air drying and (b) additional 3 min of heat drying.

Mississippi, and Armak Highway Chemicals Department for supplying the emulsion for this research.

The opinions, findings, and conclusions expressed in this report are those of the author and not necessarily those of the state or the Federal Highway Administration.

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Establishing a Construction Cutoff Date for Placement of Aggregate-Lime-Pozzolan

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Materials, Testing and Research, Pennsylvania
Department of Transportation

Aggregate-lime-pozzolan has been, for the most part, successfully used as
a base course and shoulder material in flexible pavement construction in
Pennsylvania. However, a number of recent failures associated with late-
season construction have occurred. These failures prompted the Pennsyl-
vania Department of Transportation to impose a construction cutoff date
for the placement of the material. In this paper, failure criteria based on
attained tensile strengths and the probabilities of attaining these tensile
strengths before the freezing season are established. Total probabilities
of failure of a typical 3 percent lime, 15 percent fly ash, and 82 percent
limestone aggregate (regular mix) are determined and plotted against
placement dates. An acceptable failure probability is chosen, and the
corresponding placement date is designated as the cutoff date for place-
ment of the typical aggregate-lime-pozzolan material. Two field dem-
onstration projects were monitored for one freezing season to verify
the chosen cutoff date.

As part of a program of testing and evaluation of trouble-
some materials, the Bureau of Materials, Testing and
Research of the Pennsylvania DOT conducted in-depth
research on aggregate-lime-pozzolan (ALP) material.
This research confirmed that the poor performance of
ALP resulted because the material had not been placed
far enough in advance of probable freezing conditions to
allow adequate strength gain (curing). Consequently,
the bureau decided to determine a construction cutoff
date to be incorporated into Pennsylvania DOT specifi-
cations on the use of ALP material in highway construc-
tion.

The research work presented here was performed to
develop a failure criterion for ALP, based on strength
characteristics, with which to statistically evaluate ac-
tual temperature data in certain areas of the state and
to determine ALP material failure probabilities given
a placement date and location. Ultimately, the work
was done to develop a technique for establishing real-
istic construction cutoff dates that when implemented
would eliminate or substantially reduce recently experi-
enced material failures.

TENSILE STRENGTH FAILURE CRITERION

For tensile strength testing, the double punch test de-
veloped at Lehigh University (2) is a relatively quick and
reproducible test that simulates actual field failure con-
ditions better than the more commonly used unconfined
compression test (1). Failure during freezing occurs
when interparticulate, cementitious bonds break under
tensile stresses induced by freezing pore water or the
formation of ice lenses or both. Consequently, 231 cyl-
inders were tested to determine a relationship between
double punch tensile strengths and the number of freeze-
thaw (F-T) cycles at failure as described in a previous
publication (1).

Figure 1 shows probability curves for a sample fail-
ing during F-T cycling for any tensile strength value
achieved prior to freeze-thaw testing. The solid and
dashed curves shown in this figure were deduced from
the results of tests on the 231 samples mentioned above.

1. The solid line represents failure defined as 14
percent weight loss after 12 freeze-thaw cycles (AASHO
T-136-70). The probability of failure was determined by
dividing the number of samples having initial tensile
strengths in a particular range 345 to 414, 414 to 483,
and 483 to 552 kPa (50 to 60, 60 to 70, and 70 to 80 psi)
that failed by the total number of samples in the same
range. For example, approximately 67 percent of the
samples with tensile strengths prior to F-T cycling of
414 to 483 kPa (60 to 70 psi) failed.
2. The dashed line represents failure defined as
eventual total loss of tensile strength after some rea-
sonable number of F-T cycles greater than 12. A prob-
ability of failure was determined as previously described
by using the same range of tensile strengths before F-T
cycling. By this failure definition, all samples in the
range of 414 to 483 kPa (60 to 70 psi) or lower failed.

Using varying failure probabilities over a tensile
strength range of 345 to 552 kPa (50 to 80 psi) is more
realistic than using a single tensile strength of 483 kPa
(70 psi) as the criterion for acceptance or failure. When
the standard AASHO T-136-70 was used to define failure,
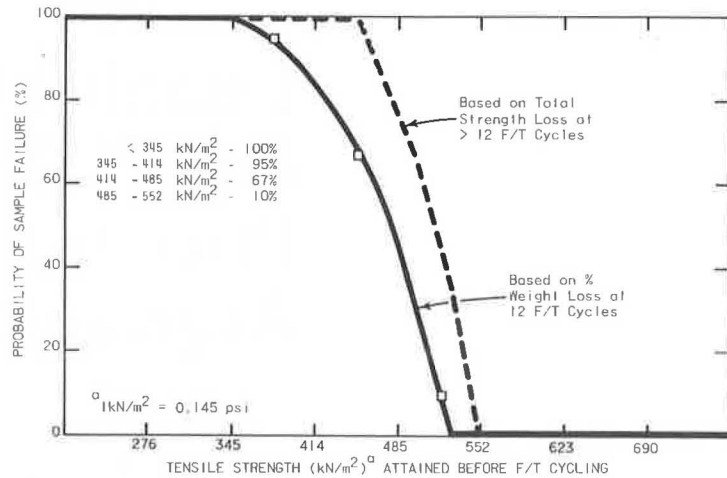
the results were as follows:

Strength (kPa)	Percentage of Failure
<345	100
345 to 414	95
414 to 483	67
483 to 552	10
>552	Negligible

Frequency Analysis

The goal was not only to determine material failure
probabilities based on attained tensile strengths but also
to determine what probabilities were associated with
attaining a given tensile strength before the start of the
freezing season in Pennsylvania. Because curing is
primarily a function of temperature and time, a degree-
day concept based on 255.4 K (0°F) was used in the pre-
diction of tensile strength to combine the temperature
and time parameters into a single unit.

A statistical analysis of frequency, typically used in
hydrological studies, was performed on predicted ten-
sile strengths with 26 years of maximum-minimum
daily temperature data from nine first-order weather
stations. Three weather stations (one with exception-
ally cold temperatures, one with exceptionally warm
temperatures, and one with moderate temperatures)
were used from Pennsylvania DOT districts in the Phil-
adelphia, Harrisburg, and Pittsburgh areas in which
most of the ALP material is produced and placed. Av-
erage daily air temperatures (ADAT) were calculated
for each day from August 1 through December 31. This
period was chosen to ensure the ultimate construction
cutoff date would be included. Because a 255.4 K base
was used, the ADAT was simply equated to degree-days.

The temperature data were computerized so that the
total tensile strength gained by December 31 was pre-
dicted for placement dates of August 1 and 15, Septem-
ber 1 and 15, and October 1 and 15 for each of the 26
years at each of the nine weather stations. Tempera-
ture data were manipulated as follows.

1. ADATs or degree-days were cumulated in three
different registers beginning on the ninth day after place-
ment. Because of a delay in the start of the chemical
reaction, the initial cure rate is much slower; and,
therefore, an average tensile strength gain of only 55
kPa (8 psi) resulted from the first 8 days of curing. The
three registers correspond to the temperature ranges
of 285.9 to 292.6, 292.7 to 297.0, and greater than 297 K

(55 to 67, 68 to 75, and greater than 75°F) used in de-
veloping the cure rate curves depicted in Figure 2. At
temperatures below 285.9 K (55°F), no appreciable gain
in tensile strength was achieved under field conditions.

2. The cumulated real degree-days (heat energy
index) in each temperature range were converted to a
partial tensile strength for that range by means of a
linear equation. Because all total cumulated degree-
days were within the straight-line portions of the cure
rate curves in Figure 2, the following linear equations
were used.

$$\sigma x = 0.041x \qquad \sigma y = 0.049y \qquad \sigma z = 0.078z \qquad (1)$$

where x, y, and z are the cumulated real degree-days
for the temperature ranges 285.9 to 292.6, 293.2 to 297,
and greater than 297 K (55 to 67, 68 to 75, and greater
than 75°F) respectively and σx, σy, and σz are the as-
sociated partial tensile strengths. It should be noted
that all lines (as indicated by these equations) intercept
the ordinate axis at zero. The authors felt that the
second linear portion of the curves for each temperature
range was the true cure rate although the first linear
portion was a result of the initial delay in the start of
true curing. Compensation for strength gain during this
initial delay period is made below.

3. The partial tensile strengths for each tempera-
ture range were then summed to obtain the total tensile
strength (σ_t) gained. An additional 55 kPa (8 psi) was
included in this total to represent the average strength
gained during the first 8 days of curing.

$$\sigma_t = 55 + 6.89[0.041(x) + 0.049(y) + 0.078(z)] \qquad (2)$$

4. The average of the tensile strengths for the three
weather stations was obtained for each of the three dis-
tricts. Therefore, an average tensile strength was tab-
ulated for each of the three districts, for each of the six
placement dates, and for each of 26 years.

Figure 3 shows theoretical frequency curves based
on some of the cumulative tensile strength data described
above. The data for each district and each date (one
point for each of the 26 years) are placed in ascending
numerical order and assigned a frequency or probability
of occurrence based on a 26-point sample group. From
Figure 3, a tensile strength of 310 kPa (45 psi) or higher
should occur 50 percent of the time or every other year
in District 6-0 if the material is placed on October 1.
Likewise, the probability that the tensile strength will
be 345 kPa (50 psi) or less is approximately 64 percent,

and the probability that the tensile strength will be 414 kPa (60 psi) or less is 86 percent. Therefore, the probability that the tensile strength will be in the 345 to 414 kPa (50 to 60 psi) range for the October 1 placement date in District 6-0 is 86 percent minus 64 percent or 22 percent.

Air and Base Course Temperature Correlation

The initial frequency curves shown in Figure 3 were derived by using air temperatures; however, ALP cures from base course temperatures, which are generally

Figure 2. Tensile strength gain for various curing temperatures.
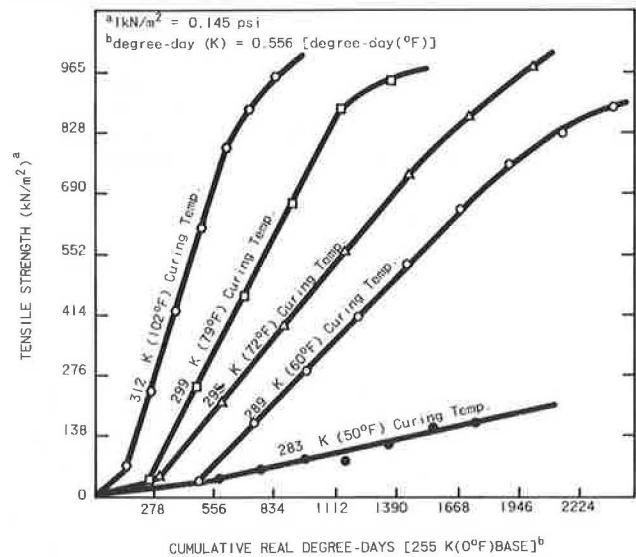
Figure 3. Cumulated tensile strengths versus frequency of occurrence (District 6-0).
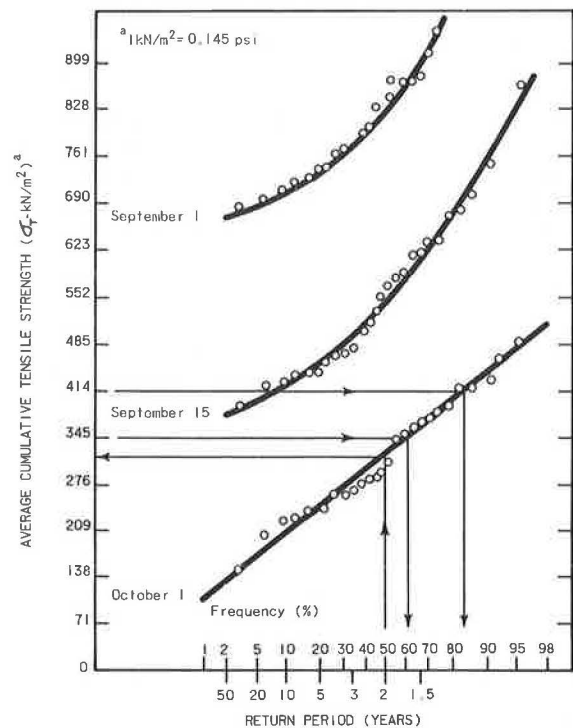
Figure 4. Relationship between monthly ADAT and average base temperature increase over air temperature for various depths to the midpoint of the base layer.
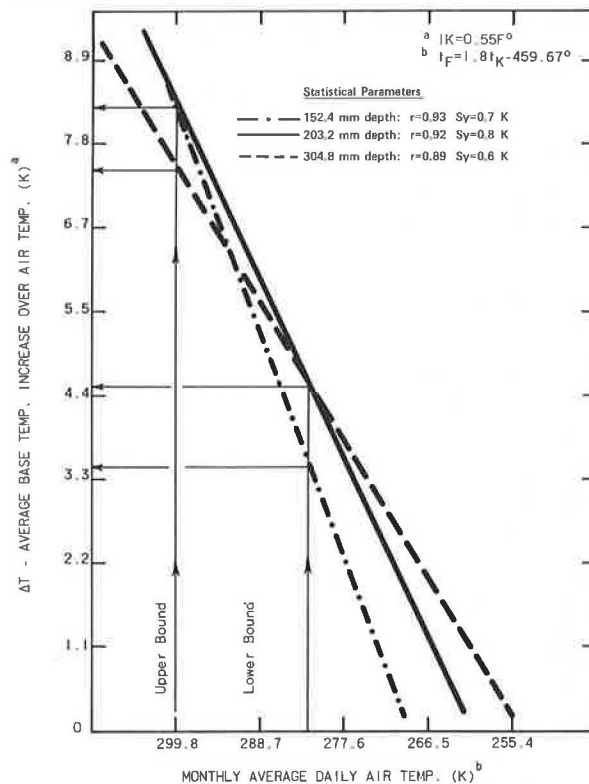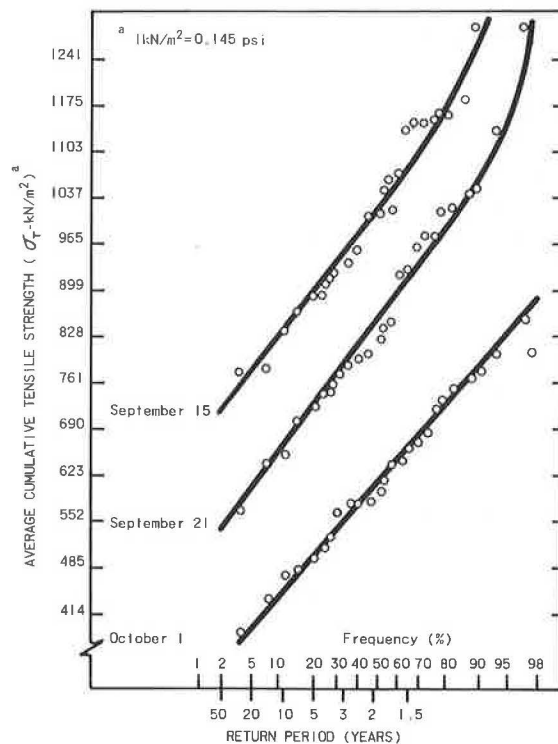

Figure 5. Cumulative tensile strength versus frequency of occurrence.

higher during the investigated season. Air, surface, and pavement profile temperatures (available from a research project entitled Freezing Indices and Regional Climates) were compared statistically to provide a method for legitimately adjusting air temperatures within the pavement system. Four years of data were collected at 14 flexible pavement installations located throughout the state. Monthly ADATs were regressed with average increases of base temperatures (thermocouple measurements at the top, midpoint, and bottom of the base layer) over air temperatures. Initially, the data were grouped according to surface layer thicknesses; however, this procedure resulted in low correlation coefficients. Grouping of the data according to total depths from the surface to the base layer midpoints resulted in good correlation (Figure 4).

Figure 4 shows that average temperature increases at 152.4, 203.2, and 304.8-mm-deep (6, 8, and 12-in) base midpoints are very similar within the range of monthly average daily temperatures used. When the figure is entered on the abscissa at 281.5 K (47°F) (lower bound), increases of approximately 3.3 K to 4.4 K (6 to 8°F) are realized. Therefore, by increasing an ADAT of 281.5 K (47°F) by 4.4 K (8°F) to simulate base temperature, the minimum temperature of 285.9 K (55°F) for appreciable curing to take place is obtained. Moreover, an average daily air temperature of 299.8 K (80°F), which corresponds to increases of approximately 7.2 to 8.3 K (13 to 15°F) in the pavement system, was never exceeded in the investigated ADAT data.

A second frequency analysis of tensile strengths, using simulated base temperatures, was performed and resulted in the curves shown in Figure 5. This frequency analysis was made as previously described except that each average daily air temperature of 281.5 K or greater (47°F or greater) was adjusted up by the maximum temperature increase (ΔT) corresponding to the 203.2-mm (8-in) depth to base midpoint line shown in Figure 4.

Failure Probabilities for Particular Placement Dates

The overall probability that a material would fail given a particular placement date was calculated by multiplying the probability of failure during freeze-thaw cycling for an initial tensile strength by the probability of attaining that initial tensile strength before the freezing season under Pennsylvania temperature conditions (Table 1). For example, if ALP material is placed on October 1 in District 6-0, the probability that it will have a tensile strength value between 414 and 483 kPa (60 and 70 psi) by December 31 is 15 percent less 8 percent or 7 percent; and the probability that a material with an initial strength of 414 to 483 kPa will fail is 67 percent. Therefore, the partial probability of failure associated with a strength range of 414 to 483 kPa is 7 times 67 or 4.7 percent. The total probability that a material placed on October 1 in District 6-0 will fail is the sum of all the partial probabilities for the tensile strength ranges. A summary of total probability data is given in Table 1 for all three districts. A plot of these total probabilities of failure against the corresponding material placement dates for each district is shown in Figure 6.

Choice of Construction Termination Date

The curves of construction cutoff dates versus total probability of failure shown in Figure 6 indicate a negligible probability of failure for material placed on September 15. However, excessive failure probabilities of 10 to 16 percent exist for the October 1 placement date. An evaluation using September 21 as the placement date re-

sulted in probabilities of failure of approximately 0.5 to 1.0 percent. Acceptance of slightly higher probabilities of failure could have been tolerable; however, upon consideration of the possibly idealized optimum mix design and maximum densities used in developing these failure probabilities, the choice of the September 21 construction cutoff date and the apparently low failure acceptance seems justified. Moreover, an increase of the accepted total probability of failure to as much as 5 percent would have extended the construction season by only a few days. Therefore, September 21 has been specified by the Pennsylvania DOT as the termination date for construction with ALP material in Districts 6-0, 8-0, 11-0, and 12-0. This construction cutoff date is intended for the remaining mountainous and northern portions of the state when increased use of ALP material in those areas justifies it. Until additional investigation is completed, a September 1 construction cutoff date has been specified for the remaining colder portions of the state.

These cutoff dates are based on the regular ALP mix as stated earlier. Adding small percentages of cement or other agents that substantially increase the rate of curing enables acceptable strengths to be attained in shorter time periods. Hence, the construction termination date may be extended to later in the season in these situations.

Field Demonstration Projects

Two demonstration test sections were installed during fall of 1974 to verify the specified September 21 cutoff date and to observe the performance of new mix designs with special additives. One project was a 305-m-long (1000-ft) section of County Line Road in North Philadelphia (District 6-0) where two sections of the regular ALP mix and three sections of new mixes with additives to increase the rate of curing were placed. The second field installation was located in Ephrata Borough, Lancaster County (District 8-0), where 90-m-long (300-ft) sections of the regular mix and a new mix design were placed.

Regular mix ALP material was placed at the County Line Road project on September 14 before the specified construction cutoff date. The computer model, using actual maximum and minimum daily air temperatures at the site, predicted that this material would attain a tensile strength of 931 kPa (135 psi). The material placed on September 14 had an in-place tensile strength of 593 kPa (86 psi) by December 31 (Figure 7). This in-place tensile strength was obtained by averaging the strengths of a minimum of three core samples for each drilling date. Although the ALP material placed before the cutoff date did develop adequate tensile strength by December 31, this strength was only about two-thirds of that predicted by the model. Two possible reasons for this discrepancy are (a) lack of sufficient moisture to propagate the hydration process during the latter stages of the ALP field curing and (b) an alteration of the curing characteristics of ALP after the 4-day cold period during the third week in October. Laboratory cure rate curves were developed under ideal conditions in which environmental moisture was controlled. If the material dried too rapidly under field conditions, all of the potential chemical reaction would not take place and slower cure rates would result. During the fifth week of curing, subfreezing air temperatures occurred on 4 consecutive days. The ADATs on these days were 276.5 to 277.6 K (38 to 40°F). The cure rate after this cold period was apparently slower than the rate before. This slower rate may have been caused by a delay in the resumption of the curing process after the cold period or a breakdown of some of the previously formed bonds. Some adjustments to the computer model appear warranted and will be made

Table 1. Total probabilities of material failure by placement dates and districts.

Placement Date	Tensile Strength Range (kPa)	Probability That Cumulative Tensile Strength Will Be Within Range (%)			Probability of Failure Given a Strength Within the Range (%)			Total Probability of Failure (%)		
		6-0[a]	8-0[a]	11-0[a]	6-0[a]	8-0[a]	11-0[a]	6-0[a]	8-0[a]	11-0[a]
September 15	<345	0.0	0.0	0.0	100	100	100	—	—	—
	345 to 414	0.0	0.0	0.0	95	95	95	—	—	—
	414 to 483	0.0	0.0	0.0	67	67	67	—	—	—
	483 to 552	<1.0	<1.0	<1.0	10	10	10	<0.1	<0.1	<0.1
								<0.1	<0.1	<0.1
September 21	<345	<0.1	<0.1	<0.1	100	100	100	—	0.1	—
	345 to 414	0.1	0.2	0.1	95	95	95	0.1	0.2	0.1
	414 to 483	0.4	0.6	0.5	67	67	67	0.2	0.4	0.3
	483 to 552	1.1	1.7	1.5	10	10	10	0.1	0.2	0.2
								0.4	0.8	0.6
October 1	<345	1.3	2.3	2.3	100	100	100	1.3	2.3	2.3
	345 to 414	3.0	5.0	4.3	95	95	95	2.8	4.8	4.1
	414 to 483	7.0	11.0	9.0	67	67	67	4.7	7.3	6.0
	483 to 552	13.0	17.7	14.4	10	10	10	1.3	1.8	1.4
								10.1	16.2	13.8
October 15	<345	38.0	51.0	38.0	100	100	100	38.0	51.0	38.0
	345 to 414	23.0	23.0	24.0	95	95	95	21.9	21.8	22.8
	414 to 483	20.0	15.0	21.0	67	67	67	13.4	10.0	14.0
	483 to 552	11.0	8.0	11.0	10	10	10	1.1	0.8	1.1
								74.4	83.6	75.9

Note: 1 kPa = 0.145 psi.

[a]District of Pennsylvania Department of Transportation.

Figure 6. Total probabilities of material failure by material placement date.
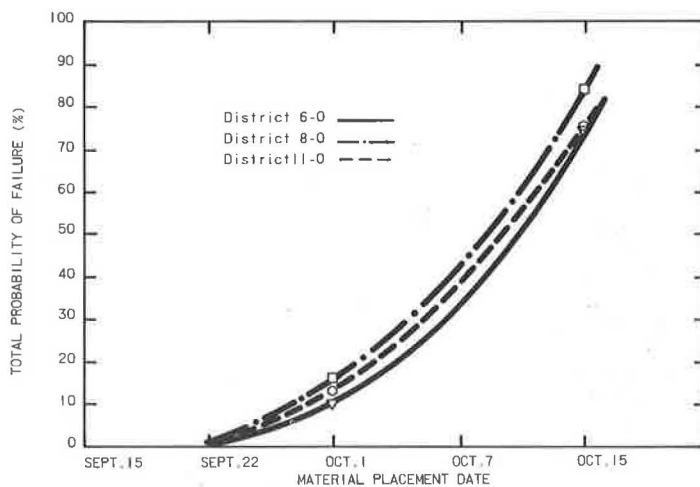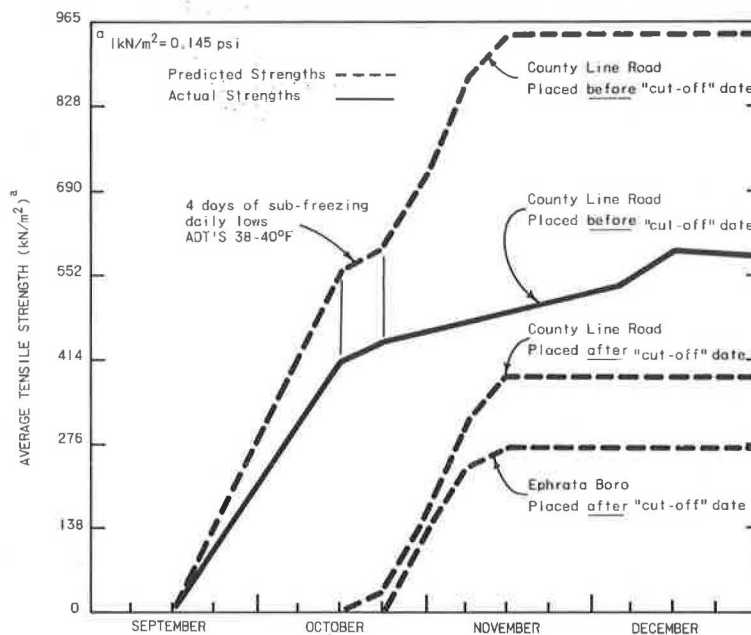

Figure 7. Predicted and actual tensile strengths at field demonstration sites.

as additional field curing data become available.

A second section of regular mix ALP material was placed at the County Line Road project on October 14, 3 weeks after the specified construction cutoff date. A tensile strength gain to December 31 of only 386 kPa (56 psi) was predicted for this material. The material placed after the cutoff date never attained a tensile strength high enough to obtain cores suitable for actual tensile strength testing. At least three attempts to obtain cores were made in each test section on a biweekly schedule. Previous drilling experience has shown that ALP with tensile strengths of about 345 kPa (50 psi) or less breaks apart when cored.

Cores obtained after the spring thaw on April 24, 1975, showed that the regular mix placed before the cutoff date had an average tensile strength of 580 kPa (84 psi), i.e., no appreciable strength loss during the freezing season. It was still impossible on April 24 to obtain cores of the ALP placed after the cutoff date. Moreover, all the lime and fly ash fines were washed from the limestone aggregate by the drill water, which indicated that few or no cementitious bonds remained. The material was apparently degraded by the freeze-thaw action of the previous freezing season.

The results were similar on the demonstration project in Ephrata Borough where the regular ALP mix was placed on October 22, approximately 1 month after the cutoff date. A tensile strength gain by December 31 of 267 kPa (39 psi) was predicted. Again, only broken pieces of cores were obtained after numerous drilling attempts. Again, almost total disintegration of the cementitious bonds was apparent during coring operations after the spring thaw.

These two field demonstration projects did verify that the cutoff date of September 21 for regular mix ALP is reasonable for Districts 6-0 and 8-0. The material placed 1 week before the cutoff date achieved adequate strength to withstand the freeze-thaw action of the initial freezing season. However, the material placed 3 and 4 weeks after the cutoff never attained a suitable tensile strength and deteriorated substantially from the freeze-thaw action of the initial freezing season.

The four sections of new mix designs with special additives for late-season construction provided significant data. These new designs showed accelerated early strength gains but attained ultimate strengths the same as or slightly lower than those of the regular mix. It appears that these new mix designs may be used for late-season construction after the currently specified September 21 cutoff date. However, further testing and analyses of curing properties at the considerably lower curing temperatures of October and November and of long-term effects of multiple freezing seasons on the new designs are warranted before the use of these materials is specified by the Pennsylvania DOT.

CONCLUSIONS

Based on the results of this research work, the following conclusions have been drawn.

1. Tensile strengths gained by in-place ALP material before exposure to any freezing temperatures provide a good criterion for material acceptance or failure. Materials with tensile strengths less than 345 kPa (50 psi) have a negligible chance of surviving a moderate freezing season in Pennsylvania. Materials with tensile strengths ranging from 345 to 552 kPa (50 to 80 psi) have varying probabilities of failure according to curves shown in Figure 1. Materials with tensile strengths greater than 552 kPa (80 psi) before freeze-thaw cycling have a negligible chance of failure.

2. A frequency analysis based on readily available daily air temperature data may be used to predict construction termination dates for any material that requires temperature-time dependent curing. Cure rates for various temperature ranges must initially be established. A degree-day concept (based on any appropriate temperature) may then be used to convert temperature data to strength gain. Finally, a reasonable amount of tolerable failure (based on a strength criterion) must be chosen.

3. Air temperatures should be converted to pavement system temperatures to simulate realistic curing conditions for pavement materials. Because average pavement temperatures are substantially higher than air temperatures (Figure 4) during most of the investigated curing season, the strength gains from actual pavement temperatures will be accelerated over those predicted from air temperatures.

4. Based on two field demonstration projects that were monitored for one freezing season in Districts 6-0 and 8-0, the choice of the September 21 construction cutoff date appears reasonably good. Comparisons of predicted and in-place tensile strengths indicated some discrepancies; therefore, some adjustments to the computer model appear warranted.

5. Additives that increase rates of curing and strength development should be investigated when construction after the cutoff date is necessary. Economic analyses of additive costs versus longer construction seasons should be made.

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New Rapid Test Method for Earthwork Compaction Control

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The quality control procedures for earthwork compaction used by New York State and many others have been to compare in-place density (measured by the sand-cone method) with maximum density (determined by a one-point compaction test and a family of moisture-density curves). This paper describes a new rapid test that uses all these concepts. A time saving is accomplished by a combination of equipment and computation improvements. Through use of a special slide rule, in-place density is obtained from the volume of a hole as measured by a sand replacement volumeter. Maximum densities, taken from the moisture-density curves and compiled in compaction control tables, can be compared with in-place densities without any calculations or interpolations. Moisture determinations are usually not required, and conversions from wet to dry density are never necessary. New York State is now using this system with a high degree of success.

The field test for earthwork compaction now most widely used takes $1\frac{1}{2}$ to $2\frac{1}{2}$ h. The number of tests that can be accomplished is thus severely limited, and the reportable results are delayed at a time when rapid substantiation of conformance is essential. Controls are generally in terms of dry density, which requires measurement of field moisture content even when moisture is not an explicit specification requirement. Testing personnel must perform many complex observations that are based in part on making rational engineering judgments and interpolations.

This new test method retains the fundamental elements of compaction control that have gained widespread recognition and acceptance but eliminates nonessential procedures having no quantitative application. Test procedures are simplified and judgment errors eliminated. The test is rapid, but control is not relaxed or compromised.

TEST PROCEDURES

The method consists of the following basic steps:

1. Measuring in-place wet density of soil in a layer just placed and compacted;

2. Determining the net weight of 944 cm³ ($\frac{1}{30}$ ft³) of soil after it is compacted in a mold in accordance with AASHTO Designation T-99, Method C (standard Proctor density);

3. Reading off the highest required and lowest allowable wet densities from compaction control tables specially designed for this test;

4. Determining whether the layer passes or fails by comparing the measured field wet density with the highest required and lowest allowable wet densities; and

5. In some cases, where the field wet density is between the highest required and the lowest allowable wet density, finding the moisture content to determine whether the layer passes or fails.

THEORY

Modern earthwork practices and equipment generally provide field densities exceeding the minimum specified. A new computational system, based on families of compaction control curves, now permits compaction testing in such cases without determination of moisture contents. For example, Figure 1 shows one set of control curves developed for New York State soils, specifically those identified as sands or as sands containing minor amounts of gravel and silt. For a laboratory dry density of 1810 kg/m³ (113 lb/ft³) and a moisture content of 6 percent, a point can be plotted on this graph. Through it a curve is drawn parallel and similar to the adjacent curves. The maximum dry density (1874 kg/m³ or 117 lb/ft³) is obtained from the point where this curve intercepts the locus of maximum density. Because moisture content for the laboratory and field dry densities is identical, the intersection of this moisture content value and the maximum dry density value is the point where field dry density would plot to be equivalent to maximum dry density. In Figure 1, this point is called field dry density required. Any point on this graph also represents a certain wet density that is the product of the dry density value and 1 plus the moisture content value. Points of equal wet density arrange in curves tending from the upper left to lower right.

Figure 2 shows the relationship among the wet density curves, compaction control curves, and points as plotted in Figure 1. If the laboratory wet density (1922 kg/m³ or 120 lb/ft³) and the moisture content (6 percent) are known, Figure 2 can be used to determine the field wet density required (1986 kg/m³ or 124 lb/ft³) to obtain the maximum dry density.

Figure 3 shows that points of equal laboratory wet

density (in this case, 1922 kg/m³ or 120 lb/ft³) and varying moisture contents develop different values for the maximum dry density and field wet density required. The range of varying moistures and these corresponding values can be limited as follows:

1. A 2 percent minimum limit for sand and for sands containing minor amounts of gravel and silt and a 4 per-
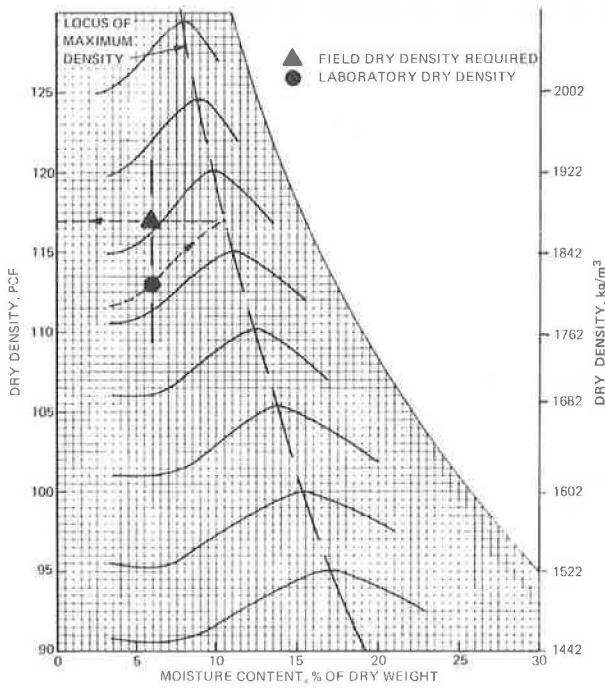
Figure 1. Compaction control curves.

Figure 2. Field wet density required to obtain 100 percent maximum dry density.
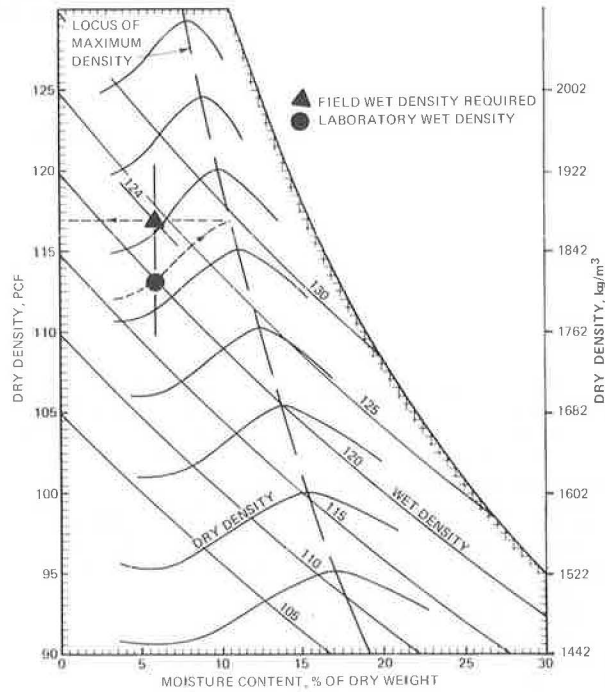

Figure 3. Field wet density required to obtain 100 percent maximum dry density for various moisture contents.
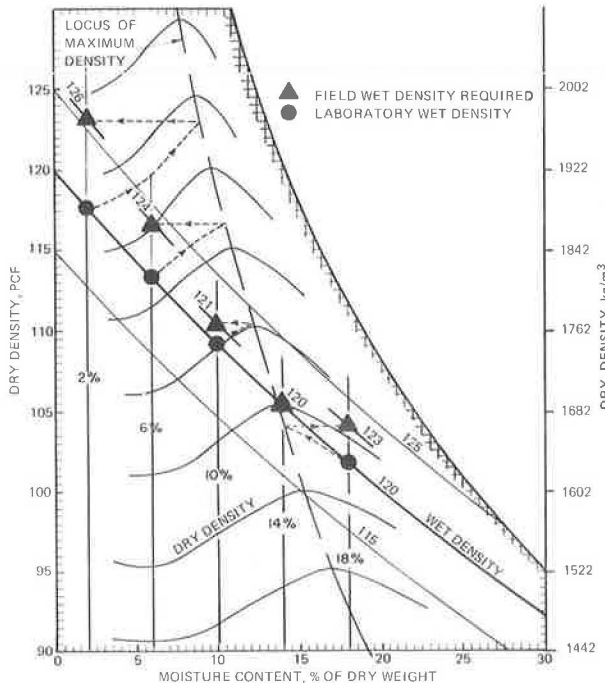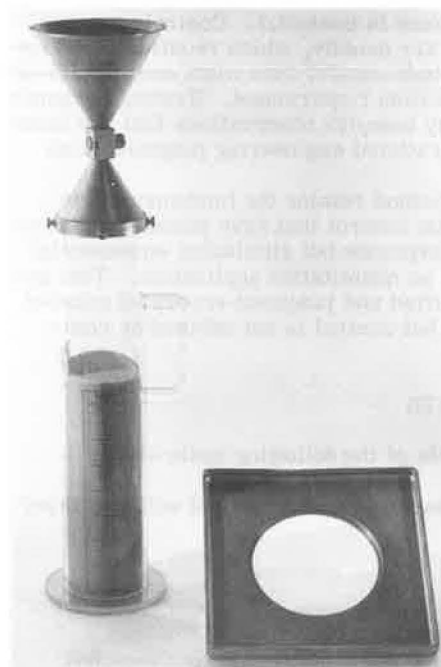

Figure 4. Sand replacement volumeter, including cone and base plate.

cent minimum limit for other soils. Usual embankment material will rarely be drier than these limits.

2. A maximum of +4 percent above optimum. Embankment materials with moisture approaching this limit will rut excessively and no compaction tests should be taken.

Within these limits, Figure 3 shows that the highest field wet density required (2018 kg/m³ or 126 lb/ft³) for a laboratory wet density of 1922 kg/m³ (120 lb/ft³) occurs at the minimum moisture content of 2 percent. Accordingly, if the measured field wet density is greater than the field wet density required at the 2 percent moisture content, the compaction test passes regardless of the actual moisture content of the soil.

As moisture content increases above 2 percent, the field wet density required to satisfy the specification requirements decreases until the lowest field wet density required (1922 kg/m³ or 120 lb/ft³) is reached. This is at the point where the wet density curve crosses the locus of maximum density (or optimum moisture). It then increases on the wet side of optimum moisture. This means that, if the measured field wet density is lower than the field wet density required at the optimum moisture content, the test fails regardless of the actual moisture content of the soil.

If the field wet density of the soil is between the highest and lowest field wet densities required, a moisture content determination is necessary to complete the test. From the known laboratory wet density and moisture content, the actual field wet density required can be obtained and compared with the measured field wet density for a pass-fail decision.

COMPACTION CONTROL TABLES

The field wet densities required to obtain a specified percentage of maximum dry density have been tabulated (Table 1) to eliminate the otherwise necessary computations and graphic interpolations. These tabulations thus replace the statewide compaction control curves, one of which was shown in Figure 1. They cover the full range of soil types and moisture contents normally encountered in earthwork construction in New York State. Similar tables can be developed to apply to any soil where moisture-density relationships are known.

NEW EQUIPMENT

Sand Replacement Volumeter

The sand replacement volumeter, a direct-reading apparatus (Figure 4), eliminates all weighings, corrections, and calculations usually associated with measuring the hole by the normal sand cone method. The apparatus is used as follows:

1. Fill the volumeter. Place the empty apparatus upright on a firm, level surface. Close the valve, and fill the cone with sand. Open the valve and fill the apparatus, keeping the cone at least half full of sand. Close the valve sharply, and empty the excess sand. Note that, as the volumeter is handled and transported, the sand will compact and the level within the volumeter will drop, but do not add sand. Measurement of the hole's volume is based on the loose volume of sand.

2. Fill the hole. Seat the apparatus on the base plate through which the hole was dug. Open the valve and, after the sand has stopped flowing, close the valve and remove the apparatus.

3. Read the volume. Hold the volumeter vertically with the cone end up. Invert it, and immediately return it to its original position. Gently shake the apparatus horizontally, just enough to level the sand. Read, average, and record the upper surface level of the sand, visible on the three vertical scales printed on the circumference of the volumeter.

When the simple techniques devised for this operation are used, the precision capability of the instrument has proved to be greater than the acceptable precision capability of the sand that may be used (±1 percent bulk density per AASHTO Designation T-191). Errors inherent in the normal sand cone method, due to the variability in unit weight of the sand caused by changes in moisture, gradation, and specific gravity, are eliminated. Measurements by this volumeter are easy, rapid, and (when compared under actual field conditions) more accurate than those by the older system.

Field Wet Density Calculator

A slide rule (Figure 5) is used to resolve the as-compacted density of the minus 19-mm (³⁄₄-in) fraction. Instructions are printed on the face of the slide rule. The limits set by the scales are designed to prevent errors that are outside the parameters of the test.

Fixed-Weight Containers

Weights of the moisture container and density cylinder have been standardized to simplify the test method further. With a container weight of 200 g (0.44 lb) and a moist soil weight of 500 g (1.1 lb), moisture content can be found from Table 2 by observing the total weight of the dry soil and container only. The fixed-density cylinder weight of 1814 g (4 lb) eliminates the need to compute laboratory wet density. The compaction control tables are arranged to find the required wet density from the total weight of the cylinder and soil.

Test Record Form

Figure 6 shows the complete procedure and some typical test results.

EVALUATION OF NEW METHOD

A new test method must exhibit real and significant benefits to be viable. To validate the new method, it was compared with the compaction control method previously used in New York State (Table 3). Test records from three major highway construction projects were selected to provide a large range of embankment materials, construction procedures, and manual testing techniques. The analysis included

1. Determining errors in computation and interpolations that are eliminated by the new method,
2. Finding the number of compaction tests in which moisture content determination would not have been required,
3. Estimating the time saved by using the new test method, and
4. Examining the simplification accomplished by this method.

Errors Eliminated

Of 1542 tests, 325 were found to have at least one error in computation or interpolation that would not have occurred if the new rapid test method had been used. For comparison purposes, a discrepancy that resulted in a value greater than ±1 percent from the true percentage

Table 1. Field wet densities required to obtain maximum dry density specified.

Tills, Silts, Clays, and Well-Graded Gravels — Densities (kg/m³) by Moisture Content Percentage

Sands or Sands Containing Minor Amounts of Gravel and Silt

Weight of Cylinder and Soil (kg)	Maximum Density Specified (%)	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
3.90	100	229.1	225.9	224.3	222.7	224.3										
	95	217.9	214.7	213.1	211.5	213.1										
	90	206.7	203.5	201.9	200.3	201.9										
3.86	100	224.3	222.7	219.5	216.3	219.5										
	95	213.1	211.5	208.3	205.1	208.3										
	90	201.9	200.3	197.0	193.8	197.0										
3.81	100	219.5	217.9	216.3	213.1	211.5	214.7									
	95	208.3	206.7	205.1	203.5	201.9	205.1									
	90	197.0	195.4	193.8	192.2	190.6	193.8									
3.76	100	214.7	213.1	211.5	209.9	206.7	208.7	209.9								
	95	205.1	203.5	201.9	200.3	197.0	198.6	200.3								
	90	193.8	192.2	190.6	189.0	185.8	187.4	189.0								
3.72	100	209.9	208.3	206.7	205.1	203.5	201.9	203.5	206.7							
	95	200.3	198.6	197.0	195.4	193.8	192.2	193.8	197.0							
	90	189.0	187.4	185.8	184.2	182.6	181.0	182.6	185.8							
3.67	100	206.7	205.1	203.5	200.3	198.6	197.0	197.0	200.3							
	95	197.0	195.4	193.8	190.6	189.0	187.4	187.4	190.6							
	90	185.8	184.2	182.6	179.4	179.4	177.8	177.8	181.0							
3.63	100	201.9	200.3	198.6	197.0	193.8	192.2	192.2	193.8	197.0						
	95	192.2	190.6	189.0	187.4	184.2	182.6	182.6	184.2	187.4						
	90	181.0	179.4	179.4	177.8	174.6	173.0	173.0	174.6	177.8						
3.58	100	195.4	195.4	193.8	192.2	190.6	189.0	187.4	189.0	189.0						
	95	185.8	185.8	184.2	182.6	181.0	179.4	177.8	179.4	179.4						
	90	176.2	176.2	174.6	173.0	171.4	169.8	168.2	169.8	169.8						
3.54	100	190.6	190.6	189.0	187.4	185.8	185.8	184.2	184.2	184.2	185.8					
	95	181.0	181.0	179.4	177.8	176.2	176.2	174.6	174.6	174.6	176.2					
	90	171.4	171.4	169.8	168.2	166.6	166.6	165.0	165.0	165.0	166.6					
3.49	100	185.8	185.8	185.8	184.2	182.6	181.0	179.4	177.8	177.8	179.4					
	95	176.2	176.2	176.2	174.6	174.6	173.0	171.4	169.8	169.8	171.4					
	90	166.6	166.6	166.6	165.0	165.0	163.4	161.8	160.2	160.2	161.8					
3.45	100	179.4	179.4	179.4	179.4	177.8	176.2	174.6	174.6	174.6	173.0					
	95	171.4	171.4	171.4	171.4	169.8	168.2	166.6	166.6	166.6	165.0					
	90	161.8	161.8	161.8	161.8	160.2	158.6	157.0	157.0	157.0	155.4					
3.40	100	174.6	174.6	174.6	174.6	173.0	171.4	169.8	169.8							
	95	166.6	166.6	166.6	166.6	165.0	163.4	161.8	161.8							
	90	157.0	157.0	157.0	157.0	155.4	153.8	152.2	152.2							
3.36	100	169.8	169.8	169.8	169.8	169.8										
	95	161.8	161.8	161.8	161.8	161.8										
	90	152.2	152.2	152.2	152.2	152.2										
3.31	100	166.6	166.6	166.6	166.6											
	95	158.6	158.6	158.6	158.6											
	90	150.6	150.6	150.6	150.6											
3.27	100	161.8	161.8	161.8												
	95	153.8	153.8	153.8												
	90	145.8	145.8	145.8												

Tills, Silts, Clays, and Well-Graded Gravels

Weight of Cylinder and Soil (kg)	Maximum Density Specified (%)	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
4.08	100		243.5	240.3	241.9											
	95		232.3	229.1	230.7											
	90		219.5	216.3	217.9											
4.04	100		240.3	235.5	235.7	241.9										
	95		229.1	224.3	224.3	230.7										
	90		216.3	211.5	211.5	217.9										
3.99	100		237.1	233.9	230.7	233.9										
	95		225.9	222.7	219.5	222.7										
	90		213.1	209.9	208.3	209.9										
3.95	100		233.9	230.7	227.5	225.9	232.3									
	95		222.7	219.5	216.3	214.7	217.9									
	90		209.9	208.3	205.1	203.5	206.7									
3.90	100		230.7	227.5	224.3	221.1	221.1	227.5								
	95		219.5	216.3	213.7	209.9	209.9	216.3								
	90		208.3	205.1	201.9	198.6	198.6	205.1								
3.86	100		227.5	224.3	221.1	217.9	216.3	217.9	222.7							
	95		216.3	213.1	209.9	206.7	205.1	206.7	211.5							
	90		205.1	201.9	198.6	195.4	193.8	195.4	200.3							
3.81	100		224.3	221.1	217.9	214.7	213.1	211.5	214.7	217.9						
	95		213.1	209.9	206.7	205.1	203.5	201.9	205.1	206.7						
	90		201.9	198.6	195.4	193.8	192.2	190.6	193.8	195.4						
3.76	100		221.1	217.9	213.1	211.5	208.7	206.7	206.7	208.3	213.1					
	95		209.3	206.7	203.5	201.9	198.6	197.0	197.0	198.6	203.5					
	90		198.6	195.4	192.2	190.6	187.4	185.8	185.8	187.4	192.2					
3.72	100		217.9	213.1	209.9	208.3	206.7	205.1	203.5	201.9	201.9	205.1	208.3			
	95		206.7	203.5	200.3	198.6	197.0	195.4	193.9	192.2	192.2	195.4	198.6			
	90		195.4	192.2	189.0	187.4	185.8	184.2	182.6	181.0	181.0	184.2	187.4			
3.67	100		216.3	211.5	208.3	205.1	203.5	200.3	198.6	198.6	197.0	197.0	198.6	201.9		
	95		205.1	201.9	198.6	195.4	193.8	190.6	189.0	189.0	187.4	187.4	189.0	192.2		
	90		193.8	190.6	187.4	185.8	182.6	181.0	179.4	179.4	177.8	177.8	179.4	181.0		
3.63	100		211.3	208.3	205.1	201.9	198.6	198.6	197.0	195.4	193.9	193.9	192.2	192.2	193.8	
	95		201.9	198.6	195.4	192.2	189.0	189.0	187.4	185.8	184.2	184.2	182.6	182.6	184.2	
	90		190.6	187.4	184.2	181.0	179.4	179.4	177.8	176.2	174.6	174.6	173.0	173.0	174.6	
3.58	100		208.3	203.5	201.9	198.6	197.0	195.4	193.8	192.2	192.2	190.6	190.6	189.0	189.0	187.4
	95		198.6	193.8	192.2	189.0	187.4	185.8	184.2	182.6	182.6	181.0	181.0	179.4	179.4	177.8
	90		187.4	182.6	181.0	179.4	177.8	176.2	174.6	173.0	173.0	171.4	171.4	169.8	169.8	168.2
3.54	100		203.5	200.3	198.6	195.4	192.2	192.2	190.6	189.0	189.0	187.4	185.8	185.8	184.2	184.2
	95		193.8	190.6	189.0	185.8	182.6	182.6	181.0	179.4	179.4	177.8	176.2	176.2	174.6	174.6
	90		182.6	181.0	179.4	176.2	173.0	173.0	170.4	169.8	169.8	168.2	166.6	166.6	165.0	165.0
3.49	100		200.3	197.0	195.4	192.2	190.6	189.0	187.4	187.4	185.8	184.2	182.6	182.6		
	95		190.6	187.4	185.8	182.6	181.0	179.4	177.8	177.8	176.2	174.6	174.6	174.6		
	90		179.4	177.8	176.2	173.0	171.4	169.8	168.2	168.2	166.6	165.0	165.0	165.0		
3.45	100		197.0	193.8	190.6	189.0	187.4	185.8	185.8	184.2	182.6	181.0	181.0			
	95		187.4	184.2	181.0	179.4	177.8	176.2	176.2	174.6	174.6	173.0	173.0			
	90		177.8	174.6	171.4	169.8	168.2	166.6	166.6	165.0	165.0	163.4	163.4			
3.40	100		192.2	190.6	187.4	185.8	184.2	182.6	181.0	181.0	179.4	177.8				
	95		182.6	181.0	177.8	176.2	174.6	174.6	173.0	173.0	171.4	169.8				
	90		173.0	171.4	168.2	166.6	165.0	165.0	163.4	161.8	160.2					
3.36	100		189.0	185.8	184.2	182.6	181.0	179.4	179.4	177.8	176.2	176.2				
	95		179.4	176.2	174.6	174.6	173.0	171.4	171.4	169.8	168.2	168.2				
	90		169.8	166.6	165.0	165.0	163.4	161.8	161.8	160.2	158.6	158.6				
3.31	100		185.8	182.6	179.4	177.8	176.2	176.2	174.6	174.6	174.6					
	95		176.2	174.6	171.4	169.8	168.2	168.2	166.6	166.6	166.6					
	90		166.6	165.0	161.8	160.2	158.6	158.6	157.0	157.0	157.0					
3.27	100		181.0	179.4	177.8	176.2	174.6	174.6	173.0	173.0						
	95		173.0	171.4	169.8	168.2	166.6	166.6	165.0	165.0						
	90		163.4	161.8	160.2	158.6	157.0	157.0	155.4	155.4						

Note: 1 kg = 2.2 lb; 1 kg/m³ = 0.06 lb/ft³.

of maximum density obtained was considered an error. Although only a small percentage of errors found would have affected a pass-fail decision, the high incidence of errors clearly shows the need to reduce the complexity of the older method.

Moisture Content Tests Virtually Eliminated

For project 1 (which appeared, in comparison with many other construction projects, to have particularly typical test results), 98 percent of the compaction tests taken would have required no moisture determinations had the newer method been used. This is significant in view of the extensive research that has been devoted to equipment improvements and time savings for moisture determinations, which now prove to be unnecessary.

Time Saved

A comparison of the time required by each method gave the following approximations.

1. The time necessary to travel to the test site, dig the hole, fill it with sand, and return to the soils laboratory varies considerably but is similar for both test methods.

2. Assuming a soil sample can be dried in about 45 min by the open flame method, the time necessary for a test result by the older method (after the inspector returns to the soils laboratory) is 1 h.

3. A test result using the new method, which does not require moisture content, can be obtained (after the inspector returns to the soils laboratory) in 10 min.

Figure 5. Field wet density calculator.
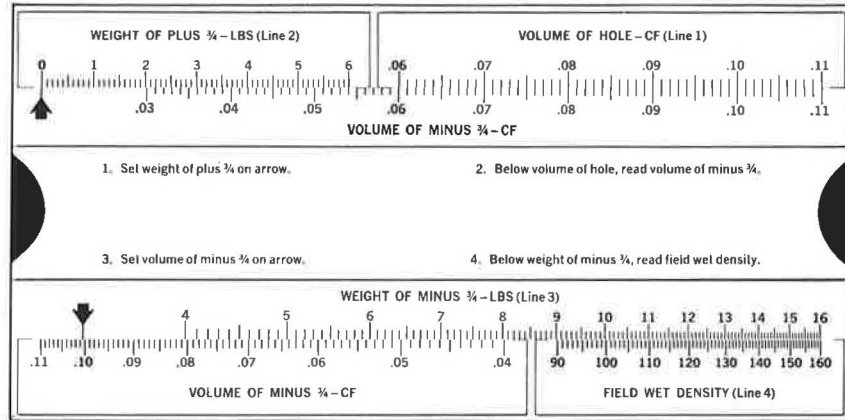
Table 2. Weight related to moisture content.

Dry Soil and Tare Weight[a] (g)	Moisture Content (%)	Dry Soil and Tare Weight[a] (g)	Moisture Content (%)
700	0	613	21
695	1	610	22
690	2	606	23
685	3	603	24
681	4	600	25
676	5	597	26
672	6	594	27
667	7	591	28
663	8	588	29
659	9	585	30
655	10	582	31
650	11	579	32
646	12	576	33
642	13	573	34
639	14	570	35
635	15	568	36
631	16	565	37
627	17	562	38
624	18	560	39
620	19	557	40
617	20		

Note: 1 g = 0.035 oz.

[a]Based on a 500-g moist sample and a 200-g container weight.

Table 3. Testing on three projects.

	Project			
Total Tests	1	2	3	Total
Observed	541	699	302	1542
With errors	70	112	143	325
Compared	471	587	159	1217
Not requiring moisture determination	461	438	136	1035

Figure 6. Compaction control data sheet with typical entries.

Table 4. Comparison of old and new test methods.

Old Test Method		New Test Method	
Step	Weighing or Calculation	Step	Weighing or Calculation
Calibrate for Sand Density (about every 10 tests)			
Calibrate cone			
1	Weigh jar, sand, and cone (before)		
2	Weigh jar, sand, and cone (after)		
3	1 − 2 = weight of sand in cone		
Find weight of sand in container			
4	Weigh jar, sand, and cone (before)		
5	Weigh jar, sand, and cone (after)		
6	4 − 5 = weight of sand in cone and container		
7	3 = weight of sand in cone		
8	6 − 7 = weight of sand in container		
Find volume of container			
9	Weigh container filled with water		
10	Weigh container		
11	9 − 10 = weight of water to fill container		
12	11 ÷ 28.3 kg (weight of 0.028 m^3 of water) = volume of container		
Determine sand density			
13	8 ÷ 12 = density of sand		
Find Volume of Hole			
14	Weigh sand and cone (before)		
15	Weigh sand and cone (after)		
16	14 − 15 = weight of sand used		
17	3 = cone volume correction		
18	16 − 17 = net sand in hole		
19	Record sand calibration factor (step 13)		
20	18 ÷ 19 = net volume of hole	1	Record volume of hole by using volumeter
Find Density in the Field (minus 19-mm material)			
21	Weigh soil and tare	2	Weigh plus 19-mm material
22	Record weight of tare	3	Weigh minus 19-mm material
23	21 − 22 = weight of soil		
24	23 ÷ 20 = field wet density of total sample		
25	Weigh plus 19-mm material and tare		
26	Record tare weight		
27	25 − 26 = weight of plus 19-mm material		
28	(27 ÷ 23) × 100 = percentage of material (in total sample)		
29	Using 24 and 28 interpolate from control charts = field wet density of minus 19-mm material		
30	Weigh wet soil and tare		
31	Weigh dry soil and tare		
32	Record tare weight		
33	30 − 31 = weight of water		
34	31 − 32 = weight of dry soil		
35	(33 ÷ 34) × 100 = moisture content (percent)		
36	29 ÷ [1 + moisture content (percent)] = field dry density of minus 19-mm material	4	Record field wet density of minus 19-mm material by using slide rule (Figure 5)
Find Density in the Laboratory			
37	Weigh cylinder and soil	5	Weigh cylinder and soil
38	Record cylinder weight		
39	37 − 38 = net weight of soil		
40	39 × 1039 (cylinder capacity = 955 cm^3) = laboratory wet density		
41	40 ÷ [1 + moisture content (percent)] = laboratory dry density (Proctor)		
Analyze Test Results			
42	Use 35 and 41 to interpolate from control charts = maximum density (control)	6	Record percentage of maximum density required
43	Interpolate from control charts for optimum moisture content	7	Record highest wet density required (Table 1)
44	(36 ÷ 42) × 100 = percentage of maximum density obtained	8	Record lowest wet density allowed (Table 1)
45	Record percentage of minimum density required		

Note: 1 kg = 2.2 lb; 1 m^3 = 35.3 ft^3; 1 mm = 0.04 in; 1 cm^3 = 0.06 in^3.

The older method also requires additional time to calibrate the sand and more time to check the test results; therefore, the total time saved is estimated at more than 1 h. By saving 1 h per test (when moistures are not required), a total of 530 person-hours (98 percent × total number of tests) could have been saved for more productive use on project 1. Inasmuch as the contractor is sometimes delayed awaiting test results, the time saved by this method can also increase the productivity of his operations.

Simplification Accomplished

Data given in Table 4 demonstrate the reduction in steps required by this method. Test results by the older method depended on the accuracy of 45 separate entries, including 16 weighings and 21 calculations. Test results by the rapid method are dependent on the accuracy of eight entries, including three weighings and one calculation.

SUMMARY AND CONCLUSIONS

Instructions for this test method are uncomplicated and straightforward. Moisture content determinations for compaction control purposes are virtually eliminated. Examination of the steps required by the two methods clearly shows that the time necessary to perform a test has been reduced and that simplification of the test provides a corresponding reduction in the probability of errors.

Although the new equipment and procedures described in this paper were designed for this test, if desired they can be independently evaluated and incorporated into

other test methods. The direct-reading volumeter is accurate and rapid; it is simple to operate, relatively maintenance-free, and inexpensive. The slide rule is specially designed to reduce the steps and errors now involved in determining field wet density corrected for ±19-mm ($\pm\frac{3}{4}$-in) material. Fixed-weight moisture containers and compaction cylinders contribute to the simplicity of the test. Development of the required wet density tables from moisture-density curves is a major evolution in quality control of earthwork construction.

This system of compaction control has been tried and proved by thousands of tests performed on various projects throughout New York State during the 1974, 1975, and 1976 construction seasons. The time-saving benefits of this procedure have been enthusiastically endorsed by project engineers and contractors alike.

ACKNOWLEDGMENTS