Design, Construction, and Performance of Lime, Fly Ash, and Slag Pavement

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A pavement (approximately 54 000 yd^2) that consists of a 10-in thick lime and fly ash aggregate base and a 3-in thick asphalt concrete surface was constructed in 1976. The pavement serves heavy coal trucks that haul to a power plant. Ten thousand tons of fly ash and 30 000 tons of slag were used in the construction of the project. A brief description of the materials and mixture design process, a discussion of the thickness design approaches used, a brief description of the construction operations and quality control procedures, a summary of results of postconstruction testing, and a description of the performance trends over the past five years are presented. The structural capacity of the pavement has not decreased since construction. Although some transverse and longitudinal cracking and very limited fatigue cracking have occurred, pavement performance as of the summer 1981 has been good.

A pavement (approximately 54 000 yd^2) that consists of a 10-in thick base of lime and fly ash aggregate and a 3-in thick asphalt concrete surface was constructed in 1976. The pavement serves the Central Illinois Public Service Company (CIPS) electric generating station, approximately 3 miles south of Coffeen, Illinois. The principal traffic on this pavement is multiple-unit trucks that carry coal to the plant or remove the bottom ash and fly ash from the plant to a dump site.

This paper presents a brief description of the materials and mixture design process, a discussion of the thickness design approaches used, a brief description of the construction operations and quality control procedures, a summary of results of postconstruction testing, and a description of the performance trends over the past five years.

MATERIALS

The pavement consisted of an asphalt concrete surface and a lime, fly ash, and slab base layer placed directly on the prepared subgrade.

Asphalt Concrete

The asphalt concrete surface layer met Illinois Department of Transportation (IDOT) specifications for a class 1 surface. A class 1 surface is a high-type asphalt concrete with a minimum Marshall stability of 1700 lb.

Base Material

The base material was a lime and fly ash aggregate (LFA) mixture blended in proportions to provide the densest possible mix for the materials used. The lime used in the LFA was a monohydrated, high-calcium lime supplied by the Mississippi Lime Company, St. Genevieve, Missouri. The fly ash was obtained from the Commonwealth Edison electric generating plant near Kincaid, Illinois. The plant burns crushed coal, and the fly ash is collected by using cyclone-type collectors. The fly ash was conditioned with approximately 20 percent moisture prior to stockpiling. The fly ash is of average quality with moderate pozzolanic reactivity.

The aggregate used in the mixture was a boiler bottom slag produced by quenching the utility plant bottom ash with water. The slag was produced by the CIPS electric generating plant near Coffeen, Illinois, and was taken randomly from the slag disposal area. A typical gradation of the slab is given in the table below.

Sieve	Percentage Passing
1/2 in	99.5
3/8 in	98.4
No. 4	91.1
No. 10	51.0
No. 20	9.1
No. 50	4.5

Subgrade

The subgrade soils are medium-to-heavy clays derived from weathered thin loess deposits over Illinoian till. The soils generally classify as CL or CH in the unified classification system. Typical soaked California bearing ratio (CBR) values are 3-5, and moduli of subgrade reaction values vary between approximately 75 and 100 lb/in^3.

Mixture Design

Criteria for LFA mix design are based on material durability and strength. To ensure good performance the material must be (a) durable, (b) placed with adequate thickness for its strength and anticipated traffic loads, and (c) constructed by using proper placement, compaction, and curing techniques.

LFA mixture design involves four major steps:

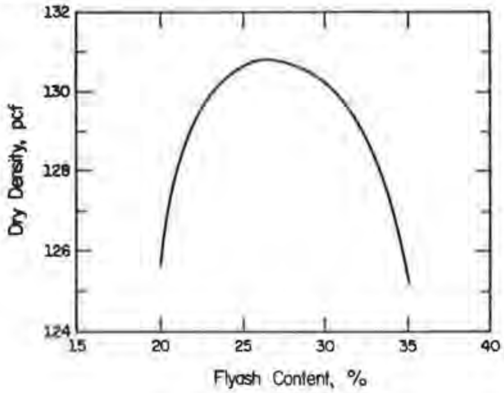
1. Ensure that the lime and fly ash will react to form the necessary cementitious bonds,
2. Ensure that the aggregate is sound and will bond to the lime fly-ash matrix produced by the reactions of the lime and fly ash,
3. Provide adequate fly ash to completely fill all voids in the aggregate, and
4. Provide an adequate amount of lime to produce the desired chemical reactions.

Experience with the Kincaid fly ash and the lime used indicated that these components were mutually reactive. Also, experience with slags produced from other wet-bottom-boiler installations indicated that the aggregate had the potential to produce a high-quality paving material. Thus, the mix design process was reduced to choosing the proper fly ash and lime contents.

The amount of fly ash required to produce a quality paving material is a function of the voids in the aggregate. To produce a quality LFA mixture, sufficient lime and fly ash matrix material must be in the mix to slightly overfill the voids in the aggregate.

The quantity of fly ash required to fill the voids is determined from the density of the compacted mix. Figure 1 shows that, as the quantity of fly ash in the mix is increased, the density of the mix first increases and then decreases. The fly ash content that produces the maximum density in the mix is the optimum fly ash content. This optimum corresponds to the point where all the voids in the aggregate are filled. Additional fly ash tends to separate the aggregate particles, thus reducing the overall density of the mixture. The optimum fly ash content (dry fly ash from the stack) was 27.5 percent.

Figure 1. Mixture-density and fly ash-content relations under AASHTO T-99 compaction.



Dry fly ash from the stack is very finely divided (more than 95 percent passing the No. 200 sieve). When conditioned with water, some fly ashes tend to set up and produce fly ash balls or lumps in the stockpile. It is the fine fraction (No. 200 sieve) of the fly ash that primarily reacts with the lime. Many of the fly ash balls are larger than some aggregate particles, thus the large fly ash agglomerations cannot serve either as filler for the voids or to react with the lime. Thus, it is necessary to either break up these agglomerations or to screen the fly ash and discard the coarse fraction.

Experience shows that quality LFA mixes can be produced if 100 percent of the fly ash passes the 3/8-in sieve and approximately 85 percent passes the No. 4 sieve. With this gradation control, however, some adjustments must be made in the fly ash quantities to compensate for the coarse fly ash agglomerations in the mix.

For this particular project, a screen was placed on the mixing plant fly ash hopper to eliminate the coarse fly ash agglomerations from the mix. Also the amount of fly ash in the mix was increased from the optimum content of approximately 27.5 percent to 32.5 percent to compensate for the nonuniform distribution of the fly ash throughout the mix, for changes in coarseness in the fly ash, and for nonuniform moisture distribution in the fly ash. The resulting mix was slightly fat with fly ash, which is the ideal design for achieving high mix quality.

Lime content in LFA mixes is based on the amount required to produce a mix of adequate strength and durability. In addition, experience shows that, for high-caliber mixing plants, there will be a variation in lime feed of about 0.25 percent, although less-efficient plants may have a variation in lime feed of up to 0.5 percent. Since a portable plant was to be set up just for this job (as contrasted with a permanent and continuously operating plant), it was determined that the minimum lime content for the mix should be around 3 percent by weight. A check of the mix by using the 3 percent lime content indicated that, after curing at 100°F for 7 days, the mix developed a strength of approximately 1800 lb•f/in². This strength is adequate for structural considerations and durability requirements. Comparable flexural strengths for this mix were between 325 and 350 lb•f/in² after 7 days at 100°F curing. Thus, the lime content for the mix was set at 3 percent by dry weight.

PAVEMENT DESIGN

Thickness design of pavements with LFA mixes can be accomplished by using the American Association of State Highway and Transportation Officials (AASHTO) procedures and assigning an equivalency value (A_2) to the LFA mix. The LFA pavement can also be considered as a slab and the Westergaard elastic slab theory or the Meyerhoff ultimate load theory used for design. For this project, the results from all three approaches are presented.

Traffic Projections

The principal traffic on this pavement was the construction traffic associated with the modification and expansion of the CIPS electric generating station, trucks that haul coal to the completed plant, and trucks that haul slag and fly ash from the plant to the disposal area. In addition there was to be some local farm-to-market traffic.

Projected average daily traffic over the life of this pavement was as follows:

Passenger cars	600
Single-unit trucks	350
Multiple-unit trucks	250

The terms single- and multiple-unit trucks are consistent with the IDOT method for computing a traffic factor (TF) [total equivalent 18-kip single-axle loads (ESAL)] based on axle load equivalencies. For class 3 roads in Illinois, assuming a 20-year analysis period, the IDOT procedure for calculating the traffic factor by using the above traffic data yields the following:

For rigid pavements,

$$\begin{aligned} TF &= 20[(0.73 \times 600) + (22.445 \times 350) + (206.955 \times 250)] \times 10^{-6} \\ &= 1.19 = 1.19 \times 10^6 \text{ 18-kip ESAL} \end{aligned} \quad (1)$$

and for flexible pavements,

$$\begin{aligned} TF &= 20(0.073 \times 600) + (17.885 \times 350) + (144.905 \times 250) \times 10^{-6} \\ &= 0.8506 \text{ for flexible pavements} = 850\,600 \text{ 18-kip ESAL} \end{aligned} \quad (2)$$

Typical Designs

With a TF of 0.85 for flexible pavements and an Illinois Bearing Ratio (IBR value is similar to CBR) of 5.0 (AASHTO soil support equal to 4.0-4.5), the required structural number value (from IDOT design nomograph) is 3.6. This value was used for developing comparative designs.

According to the IDOT design procedure for flexible pavements, the structural value for the pavement is given by the equation:

$$D_t = A_1D_1 + A_2D_2 + A_3D_3 \quad (3)$$

where D_1, D_2, and D_3 are the thicknesses in inches of the surface, base, and subbase layers, respectively, and A_1, A_2, and A_3 are coefficients that are a function of material type and properties.

Based on the data reported earlier (compressive strengths well in excess of 1000 lb•f/in² after 7 days of curing at 100°F), the proposed LFA mix has the potential to be above average in quality. The material coefficient value (A_2) assigned by IDOT for LFA mixes of this quality is 0.28. Assuming the high-quality class 1 asphalt concrete has A_1 coefficient value of 0.40, the structural number for the typical pavement section selected is as follows:

3-in AC surface at 0.40 = 1.20
10-in LFA base at 0.28 = 2.80
SN for entire section = 4.00

The SN value of 4.0 compares with a required SN value of 3.6 for a subgrade with an IBR of 5.0, and a required SN value of 3.8 for a subgrade IBR of 4.0. Thus, even if the subgrade was weaker than expected, the design section with a SN of 4.0 should be adequate to carry the design traffic.

Based on the laboratory test results and assuming a reduction factor of 0.6 for going from the laboratory to the field, the LFA material used for the design can be assumed to have a compressive strength in excess of 1000 lb•f/in² (1800 x 0.6 = 1080), and a corresponding flexural strength greater than 200 lb•f/in². Preliminary mix design data indicated that 28-day compressive strengths of 1200-1500 lb•f/in² under summer field conditions are not unrealistic with corresponding flexural strengths in the range of 250-300 lb•f/in².

The pavement was analyzed by using the Westergaard model for flexural stresses and fatigue failure in the LFA base and for ultimate load capacity by using the Meyerhof theory. Ahlberg and Barenberg previously recommended the use of a Meyerhof-based procedure (1).

The ultimate load capacity for the section with a 3-in surface and 10-in LFA base was used on a flexural strength of 200 lb•f/in² for the LFA material and was calculated for both interior and edge-loading conditions. Assuming a k for the subgrade of 100 lb/in³ and discounting the contribution of the 3-in asphalt concrete surface, the ultimate load capacities were 27 kips for edge loading and 40 kips for interior loading. Relating these values to 10-kip wheel loads (20 000-lb single-axle loads) gives factors of safety against failures of 2.7 for edge loading conditions and 4.0 for interior loading conditions.

Analysis of the 10-in thick section by using the Westergaard slab theory and a 10-kip wheel load, again assuming a subgrade k of 100 lb/in³, and ignoring the contribution of the 3-in surface layer, gave flexural stresses in the LFA base of 110 lb•f/in² for interior loading and 160 lb•f/in² for edge loading. The maximum stress ratio for interior load conditions is 110/220 = 0.55 and for edge loading conditions 160/200 = 0.80. Field experience with LFA materials indicates that, as a result of the normal strength gain characteristics of these materials with time, a pavement that has a stress ratio of less than 1.0 will generally not fail in fatigue.

The pavement is considered adequate by using all three thickness design criteria.

CONSTRUCTION AND QUALITY CONTROL

The LFA mix was blended in a mixing plant capable of producing up to 600 tons/h. The plant consisted of one feed hopper each for the slag and the fly-ash components, a major storage silo, and a smaller secondary feed silo for the lime, a storage tank for water, a continuous mixing, twin-shaft pugmill, and the necessary belts, gates, and controls to feed and control the amount of components that go into the pugmill.

The mixing plant was set up in the slag-disposal area for the CIPS generating station and the slag loaded directly from the disposal pile into the hopper with a front-end loader. No attempt was made to control or adjust the slag.

Conditioned fly ash from the Kincaid ash-disposal area was delivered and stockpiled at the mixing plant site. The fly ash was scalped on a 3/8-in screen and fed directly into the fly ash feed hopper. Moisture content of the fly ash varied from 9 to 12 percent.

Lime was delivered in pneumatic trucks on a daily basis as needed. The lime was transferred from the delivery trucks to the lime storage silo by using compressed air. The storage silo was large enough to hold one 20-ton truckload of lime.

The lime, fly ash, and aggregate components were fed from their respective hoppers through calibrated gates onto a main feeder belt and delivered into a continuous-flow pugmill. Enough water was added during pugging to bring the mix to its optimum moisture content (approximately 7.5 percent). The mix flowed through the pug into a surge hopper and was then loaded into open dump trucks.

The mix was delivered to the road site, spread with a dozer-mounted spreader box, and compacted with vibrating steel wheel rollers. The entire 10-in thickness of LFA mix was spread and compacted in a single lift. Curing was accomplished by using a bituminous prime coat or a sealer.

The asphalt concrete surface course was placed by using conventional paving procedures.

Quality control procedures included the collection and testing of grab samples of the mix as delivered to the road site and the conducting of in situ density tests on the compacted materials. Specific tests on the grab samples included compacting Proctor-sized specimens and measuring the strength of the mix after 7 days of curing at 100°F and conducting titration tests on fresh samples of the mix to determine its lime content.

The number of Proctor samples and titration tests per day varied from two to four depending on the amount of material produced. An attempt was made to get at least three samples each day regardless of production. Because of some problems with the mechanical reliability of the plant, this was not always possible.

Results from the quality control tests are presented in Figures 2 and 3. Figure 2 shows the daily average and range for lime contents in the mix determined from the titration tests. Note that, for the first three weeks, the lime contents were highly variable and somewhat below the design value of three percent. These data reflect the inexperience of the contractor with this type of construction, the poor mechanical condition of the plant, and improper calibration techniques. After September 1 the lime content control was considerably improved.

Figure 3 shows the daily ranges and averages for LFA compressive strengths. These data also reflect the poor quality and lack of uniformity during the first few weeks of construction. As with the lime content, the compressive strength data were much improved after September 1.

A regression analysis was made by relating compressive strength to mixture lime content. The regression equation for the relation is as follows:

$S = 408 + 156L$

S = Compressive strength for 7-day cure at 100°F (lb•f/in²)

L = Lime content (%)

$R = 0.271$, significant at $\alpha = 0.025$

From these data, the importance of good control on lime content is apparent.

POSTCONSTRUCTION TESTING

After construction was completed (October 1976), 4-in diameter cores were taken in late November 1976. Full-depth cores were recovered in all of the sample sites. Data from the core samples are summarized in the table below.

Resilient modulus for asphalt concrete at 76°F was 360 000 lb•f/in².

For the LFA mixture,

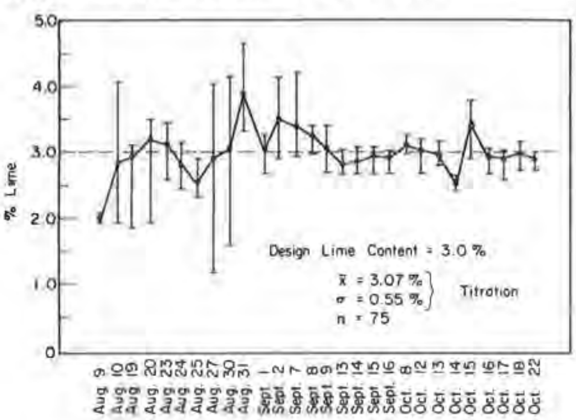
Item	Field Samples from Sept. 12, 1978 $(lb \cdot f/in^2)$
Unconfined compressive strength	1130
Split tensile strength	127
Resilient modulus	2×10^6

Postconstruction field cores in November 1976 were as follows:

Item	Age at Coring (days)	Compressive Strength $(lb \cdot f/in^2)$
Excessive	110	570
lumps in	110	740
fly ash	110	830
	110	1210
Avg		838
Fly ash	100	875
screened	96	3470
on 3/8-in	90	1230
sieve	48	1090
	40	738
Avg		1481

The pavement section described was subsequently selected for inclusion in the IDOT-University of Illinois IHR-508 flexible pavement research project (2). IHR-508 activities included Benkelman beam testing, IDOT road rater (8-kip peak-peak capacity) testing, and a soil and material sampling program. IHR-508 data were concentrated in a 100-ft long representative test section. A general full project length road rater (NDT) evaluation was conducted in April 1981.

Figure 2. Daily results from titration test for lime content.



Benkelman beam and road rater data are summarized in Table 1.

Soils and materials data were established for samples (bulk samples, cores, thin-walled tube subgrade samples) collected during September 1978 evaluation activities. Summaries of the data are presented in the preceding text table and below. A moisture-density-CBR plot for the subgrade is shown in Figure 4. Details of the testing procedures are presented elsewhere (2).

Gradation	Percentage
Sand, 2-0.05 mm	14.4
Silt, 0.05-0.002 mm	65.1
Clay, <0.002 mm	20.5

Liquid limit = 28.0
Plastic limit = 18.0
Plasticity index = 10.0
Unified classification = CL
Compaction characteristics (AASHTO T-99)
Maximum dry density = 102.2 lb/ft^3
Optimum water content = 15.3 percent

PAVEMENT PERFORMANCE

The pavement described has been in service for nearly five years. As of the summer 1981, the pavement has carried nearly 1.75×10^6 tons of coal, more than 100 000 tons of top and bottom ash, and considerable local traffic. The only maintenance thus far has been the placement of a double surface treatment (slag cover aggregate) to improve the skid

Figure 3. Field compressive strength data.

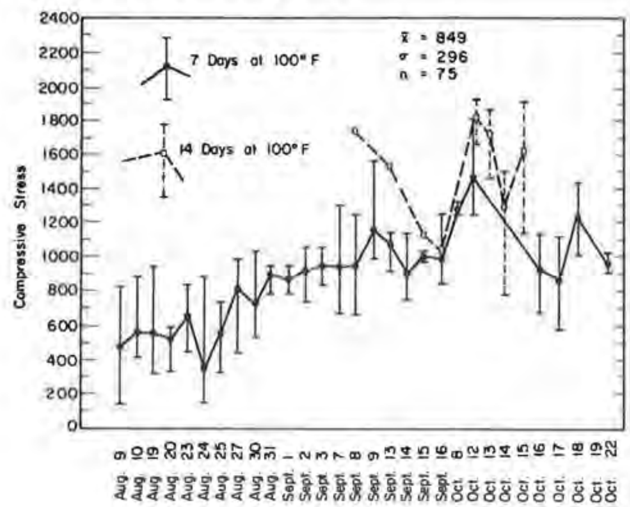


Table 1. Summary of deflection data.

Test Date	Asphalt Concrete Temperature (°F)	Benkelman Beam Δ (mils) Avg	Benkelman Beam Δ (mils) SD	Road Rater DO[a] Avg	Road Rater DO[a] SD	Road Rater Area[b] (in) Avg	Road Rater Area[b] (in) SD
5/23/78	93	12.1	2.2	10.7	0.98	29.4	0.96
9/12/78	85	14.5	3.2	9.5	0.70	28.8	0.88
10/26/78	50			7.2	1.33		
4/18/79	68			10.5	1.58	30.8	1.55
8/8/79	120			11.7	1.57	28.6	0.47
4/23/81				10.5	1.30	29.4	0.82
4/23/81				9.8	4.75	28.4	3.00

Note: Data are for the 100-ft long test section with the exception of the last data entry, which is for the entire project length.

[a]DO is the center of 12 in-diameter load plate deflection (8 kip peak-peak vibratory loading; 15 Hz).
[b]Area is a measure of the pavement surface deflection basin (2).

Figure 4. Moisture-density CBR relations for Coffeen subgrade soil.

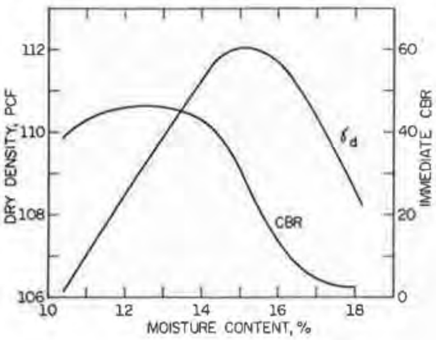


Figure 5. Typical transverse crack found at 30- to 40-ft intervals throughout the project.

resistance of the pavement.

The road rater data indicate that the structural capacity (maximum deflection and area) has not decreased over the 5-year life of the pavement. The pavement behavior is relatively insensitive to seasonal effects, as evidenced by the road rater data (Table 1) for several different periods of the year. Such behavior is typical of pavement sections that contain base courses that have a high rigidity.

The field coring data (September 12, 1978 information) indicate that the strength of the LFA mixture was greater than the project average construction compressive strength (Figure 3) of 849 lbf/in². There are no indications of any freeze-thaw durability distress in the LFA mixture as of that date.

Figure 6. Isolated longitudinal edge crack.

A close inspection of the project was conducted in the summer 1981. At that time transverse cracking had occurred; crack spacing ranged from approximately 50 to 150 ft. A typical transverse crack is shown in Figure 5. Most of the crack spacing intervals are greater than 75 ft. Limited longitudinal cracking has appeared in both the inner and outer wheel paths. A typical longitudinal crack is shown in Figure 6.

In isolated locations some fatigue (alligator cracking patterns) distress (shown in Figures 7 and 8) has occurred. Some of these areas have required surface patching and others are unmaintained. Studies to determine the factors that contribute to this distress are being planned. A likely cause for these disturbed areas is the significant variation observed in lime and fly ash mixture quality used (e.g., proportions and compaction). These variations were due in large part to inexperienced personnel of the contractor blending and placing the mix.

Asphalt concrete rut depths in the outer wheel path are approximately 0.45 in in the heavily loaded lane (entrance road to power plant) and 0.20 in in the less heavily loaded lane (exit from the plant area). Longitudinal roughness in the wheel paths is low and the overall ride quality is high.

Overall, the pavement has provided good performance. The county engineer has recently purchased a plant to set up in the slag disposal area to provide a constant supply of LFA mix to be used in future road construction in the county and adjacent counties.

SUMMARY

The design, construction, and performance of an

Figure 7. Areas of isolated alligator cracking.

Figure 8. Closeup of alligator cracking in isolated areas.

extensive (54 000 yd^2) lime and fly ash pavement project (10-in thick LFA base, a 3-in thick asphalt concrete surface) is documented. The structural capacity of the pavement has not decreased since construction. The pavement performance as of summer 1981 has been good. A major attribute of the LFA base course pavement was the extensive use of by-product materials (10 000 tons of fly ash, 30 000 tons of slag) in the project.

ACKNOWLEDGMENT

We wish to thank Terry Wells of CIPS and Anthony Georgeff, Superintendent of Highways for Montgomery County, for their assistance and support of this project.

REFERENCES

1. H.L. Ahlberg and E.J. Barenberg. Pozzolanic Pavements. Engineering Experiment Station, Univ. of Illinois, Urbana-Champaign, Bull. 473, Feb. 1965.
2. M.S. Hoffman and M.R. Thompson. Nondestructive Testing of Flexible Pavements-Field Testing Program Summary. Univ. of Illinois, Urbana-Champaign, Civil Engineering Studies, Transportation Engineering Series No. 31, June 1981.

Evaluation of Heavily Loaded Cement-Stabilized Bases

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A field evaluation was carried out to determine performance of heavily loaded cement-stabilized bases. Ten projects, located in Oregon, Idaho, and British Columbia, were surveyed. Bases at six projects were used as log-sorting yards and at the other four projects as container-port storage areas. Stabilized base thickness ranged from 6 to 18 in. Cement content of the stabilized base generally varied from 5 to 8 percent. Log-sorting yards carried wheel loads that exceeded 80 kips. Wheel loads at container ports ranged from 10 to 25 kips. Performance was evaluated visually. Properties of base and subgrade materials were determined in the laboratory from samples obtained at each project site. Pavement analysis was conducted to determine stresses in the base. Also, required base thickness was computed for each site. Thickness was chosen to just sustain the estimated number of wheel loads up to the time of survey. It was found that base thickness computed from existing design procedures was generally more than as-constructed thickness. Since bases at all project sites are performing well, it is concluded that present design procedures for conventionally stabilized materials are conservative for heavily loaded high-quality cement-stabilized bases investigated in this study.

Since 1935 thousands of miles of cement-stabilized bases have been constructed. Extensive laboratory and field testing has been done on stabilized bases that meet criteria for soil-cement. These bases ranged in thicknesses from 5 to 9 in (1). Compressive strength was generally 400-500 $lb{\cdot}f/in^2$ (2-4).

Very little information has been reported for stabilized bases that have thicknesses of 12 in or greater and have compressive strength in excess of 1000 $lb{\cdot}f/in^2$. Information is available on limited full-scale traffic tests conducted by the U.S. Army Corps of Engineers on soil-cement pavements 21 and 25 in thick (5).

Use of high-quality thick-cement-stabilized bases has been increasing for heavily loaded facilities such as log-sorting yards, container ports, and log-haul roads. However, present methods for design of such pavements are based on extrapolations of results from laboratory testing and field evaluations of 5-9 in thick soil-cement pavements. To improve design of future heavily loaded high-quality cement-stabilized bases, a field evaluation of several such facilities was conducted. Information obtained included data on materials, design, construction, performance, and maintenance. This information was analyzed to determine the feasibility of using high-quality cement-stabilized bases for very heavily loaded roadways. The adequacy of existing thickness design procedures for such bases was also evaluated.