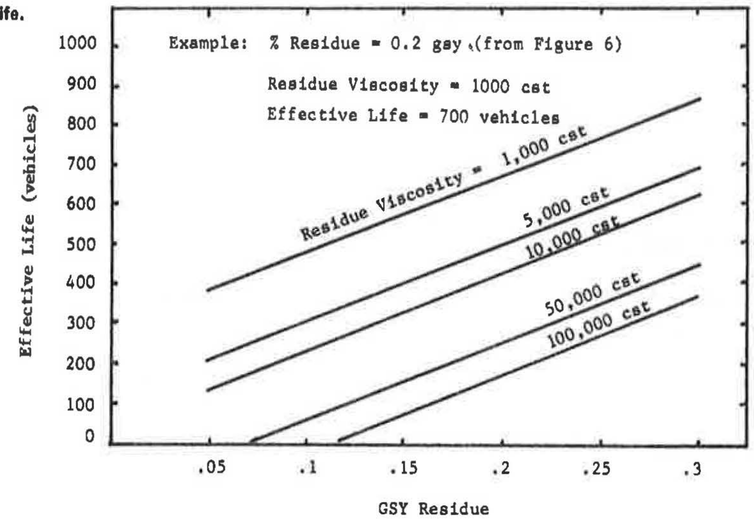
Figure 8. Residue application rate versus effective life.



REFERENCES

1. B. Langdon. An Evaluation of Dust Abatement Materials Used in Region 6, Final Report. Forest Service, U.S. Department of Agriculture, Region 6; Oregon State Univ., Corvallis, Transportation Engineering Rept. 80-3, Jan. 1980.
2. E.M. Shelton. Diesel Fuel Oils, 1979. Bartlesville Energy Technology Center, U.S. Department of Energy, Rept. BETC/PPS-79-5, Dec. 1979.
3. R.D. Stephens and others. A Study of the Fate of Selected Toxic Materials in Waste Oils Used for Dust Palliation on Logging Roads in the Plumas National Forest. Hazardous Materials Section, California Department of Health Services, Sacramento, Aug. 1980.
4. F.S. Rostler. The Chemical Aspects of the Rationale in Selecting, Using, and Specifying Products for Dust Control. Paper presented to the Forest Service, U.S. Department of Agriculture, Region 6, Portland, OR, May 1978.
5. Bituminous Dust Palliative Treatment. In Forest Service Standard Specifications for Construction of Roads and Bridges, Forest Service, U.S. Department of Agriculture, Section 412, 1979.

Recycling Bituminous Concrete Through Use in Cement-Treated Base: Case Study, Fredonia Streets

KATHRYN M. DREASEN AND JOHN E. LAWSON, JR.

The reconstruction of a low-volume road through Fredonia, Arizona, was the first pavement structure constructed under contract to the Arizona Department of Transportation (ADOT) that incorporated an old road base, including ground bituminous concrete, into a cement-treated-base (CTB) design. The shortage of nearby suitable road-base material resulted in the consideration of stabilization of the existing pavement structure. If cement stabilization was to be used, ADOT experience indicated that an asphalt-rubber stress-absorbing membrane interlayer would be required to inhibit reflective cracking through the asphalt surface. Three different structural sections were constructed based on varying soil types and availability of recyclable road base. On this particular project, the use of a CTB that incorporated reprocessed bituminous concrete and aggregate base from the previous pavement structure saved ADOT approximately $120 000. This savings is based on a structurally equivalent pavement that consisted of full-depth asphaltic concrete and costs based on the contractors' bid prices for this item on this job. An additional consideration that made this alternative attractive was the simplicity of construction techniques. It is believed that cement stabilization of old, recyclable pavements will prove to be a low-cost, high-strength alternative to thick asphalt concrete sections when aggregate availability is a problem.

In many rural areas of Arizona, the cost of roadway development is often escalated by the lack of accessible quality material. Fredonia is one such area. For economic reasons, a design was developed that recycled bituminous concrete through use in a cement-treated-base (CTB) mixture. The pavement structures consisted of either 6 or 8 in of CTB covered by 2 or 6 in of asphalt concrete (AC), respectively, both topped by 0.5 in of asphalt concrete friction course (ACFC). Based on historical evaluations, an asphalt-rubber stress-absorbing membrane interlayer (SAMI) was incorporated into this system between the top of the CTB and the bottom of the AC.

BACKGROUND

The decision to use CTB on Fredonia streets was influenced by the performance and evaluation of existing roadways constructed with soil-cement. During the summer of 1977, a section of AZ-169 was constructed by using soil-cement. Construction was efficient and economical, and an evaluation after five years of service has shown it to be struc-

Figure 1. Typical section of AZ-169.

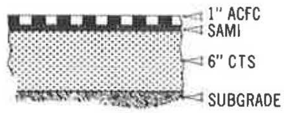
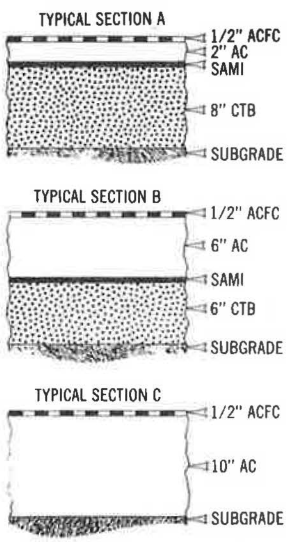


Figure 2. Structural coefficient for CTB.

ADDITIVE STRUCTURAL COEFFICIENT	MIXING	PASS NO. 8	PASS NO. 4	STRENGTH	P.I.	THICKNESS OF A.B. OVER C.T.B.
.05	CENTRAL PLANT					
.00	ROAD MIX					
.01		30-65				
.02			45-75			
.07				500 PSI		
.05				300-500 PSI		
.00				300 PSI		
.01					N.P.	
.01						4"
.02						6"

Figure 3. Typical A, B, and C sections.



turally sound and performing well.

One test section of AZ-169 consisted of 6 in of cement-treated subgrade (CTS), SAMI, and 1 in of ACFC (see Figure 1).

This particular section has served for five years with no visible defects and no maintenance. Other test sections and the control section have experienced various degrees of distress. A recent contract was awarded to reconstruct these failing sections. Evaluation of this project indicated that the SAMI controlled the reflection of shrinkage cracks that are formed in the rigid soil-cement base and that tend to migrate through the asphalt paving surface (1,2).

PRELIMINARY MIX DESIGN CONSIDERATIONS

CTB Mix Design

The design of the CTB mixture was formulated from test hole logs, which identified the average thickness of material in the existing pavement. The initial mix design consisted of five parts by weight of the base material mixed with three parts by weight of the crushed bituminous concrete. The laboratory mix design required that this mixture be crushed or pulverized until 85 percent passed a 1-in sieve and at least 40 percent passed the No. 4 sieve. Mixtures were evaluated at 6 and 8 percent cement in accordance with ASTM C-593. The optimum moisture content was 6.7 percent. The results of the testing are as follows:

Cement (%)	Avg Compressive Strength (psi) Saturated	Vacuum
6	640	655
8	960	975

Based on these results, it was determined that 6 percent cement by dry weight of aggregate would be sufficient to achieve adequate 7-day strengths. Because of such high strengths, the pavement design engineer was able to award a relatively high structural coefficient to the CTB, thus reducing the required thickness of the AC.

Pavement Design

Arizona uses the American Association of State Highway and Transportation Officials (AASHTO) flexible pavement design procedure to determine required pavement thickness. Preliminary sampling and testing consisted of some 15 locations. It was determined that the project could be separated into two design sections based on soil conditions. One section consisted primarily of A-2 soils; a soil-support value of 5.42 (R-value = 39) was selected to represent this section. The other section consisted of A-4 and A-6 soils; a soil-support value of 2.50 (R-value = 8) was chosen to represent this section.

A 20-year AASHTO analysis indicated required structural numbers of 2.77 and 4.16 based on 18-kip single-axle equivalent loads (413 575) and a regional factor of 2.2.

The closest known source of suitable aggregate is located a distance in excess of 25 miles from Fredonia. Therefore, an analysis of the feasibility of using the in-place material in the pavement section was conducted. As previously mentioned, a mix design that consisted of cement stabilization of the existing pavement and base was performed. The results of this mix design were considered satisfactory for further development of this concept.

In order to perform a preliminary economic analysis, it was necessary to assign a structural coefficient to the CTB. At that time, the Arizona Department of Transportation (ADOT) used the values in Figure 2 to determine the structural coefficient of CTB. (Note, for CTB use base of 0.12.) By using this figure, a coefficient of 0.23/in was obtained; however, considering the fact that strengths in

Table 1. Details of items for cost comparison.

Item	Unit Cost ($)	Quantity	Total Cost ($)
AC bid items			
AC	13/ton	19 900 tons	258 700
Asphalt cement	218/ton	1033 tons	225 200
Cement additive	0.70/lb	20 674 lb	14 500
Tack coat	475/ton	56 tons	21 000
Total			519 400
CTB bid items			
CTB	$0.74/yd^2$	$54\ 515\ yd^2$	40 000
AB-3 (graded aggregate)	10/ton	1500 tons	15 000
Cement	105/ton	1160 tons	121 800
Curing seal	335/ton	56 tons	18 800
Total			195 600
SAMI bid items			
Asphalt-rubber material	650/ton	110 tons	71 600
CM-11 (chips)	18/ton	550 tons	9 900
Total			81 500

excess of 600 psi were developed, an additional 0.02 was added, thereby giving a coefficient of 0.25/in. The high strength obtained also somewhat alleviated concerns regarding frost susceptibility of the CTB.

A coefficient of 0.38 (at a 3-in thickness) was assumed for AC. The typical design sections were as follows (see Figure 3):

1. Section A: 8-in CTB, SAMI, 2-in AC, and 0.5-in ACFC;
2. Section B: 6-in CTB, SAMI, 6-in AC, and 0.5-in ACFC; and
3. Section C: 10-in AC and 0.5-in ACFC.

It was determined that, if the existing bituminous concrete was incorporated into the existing base, enough material should be available for 6-8 in of CTB. A preliminary costs analysis was performed, and it was estimated that a cost savings of approximately 10 percent could be realized by the use of a CTB design compared with a full-depth AC design. A cost analysis based on actual bid prices indicates that approximately 14.5 percent ($120 000) was saved, which assumes that full-depth AC would have been bid at the same cost as the AC bid on the job. Details of this cost analysis are given in Table 1.

The following items should be used in conjunction with the information in Table 1 for a complete breakdown of the cost comparison:

1. For the AC sections, 58 515 yd² was required at an average thickness of 5.4 in. The cost for the AC was $1.65/yd²-in.
2. The CTB cost included pulverization, mixing with cement, and placing of the material. The average thickness was 6.7 in. The cost for the CTB was $0.52/yd²-in.
3. For the SAMI sections, 54 515 yd² was required. The cost for the SAMI was $1.67/yd². The SAMI cost in terms of inches of CTB was $1.67/6.7 = $0.25/yd²-in. Therefore, the cost of the CTB and the SAMI was $0.77/yd²-in.
4. The cost comparison for the project was as follows. The cost if full-depth AC had been used throughout the project would have been $916 000. The cost for the as-built design was $796 500. Therefore, $120 000 (14.5 percent) was saved.

CONSTRUCTION TECHNIQUES

During the fall of 1981, the existing pavement and base were pulverized. This material was removed from the roadway, with depths of removal varying from 8 to 12 in, and it was placed to the side within the right-of-way. In areas where the grade was deficient, graded aggregate was added to this stockpiled material. This material was mixed and then laid down as the new base. This base was compacted to 100 percent of maximum density and remained in this condition without a seal throughout the winter.

In spring 1982, the subgrade was repulverized and additional aggregate was added to soft areas and to areas in which winter traffic and snow plows had lowered the grade.

The following is a sequence of typical stabilization processes that were performed daily:

1. A day prior to stabilization, the existing base and surfacing materials to be treated were scarified and prewetted to optimum +2 percent moisture.
2. The next day, this material was shaped to the approximate crown and grade.
3. Cement was pumped from a surge tank on the cement truck to a contractor-devised cement spreader with a second surge tank on top (see Figure 4).
4. The cement was mixed by using a Bomag mixer, and any necessary additional water was added by using a water truck. Moisture had to be maintained at optimum or slightly dry of optimum (see Figure 5).
5. Compaction was achieved in 2 h by means of a vibratory steel-wheeled roller. Finishing was completed with a pneumatic roller (see Figure 6).
6. In accordance with the specifications, the top 0.25 in of material was graded off and wasted.
7. The surface of the compacted layer was sealed with a 0.25-gal/yd² application of MC-250.

No traffic was allowed on the treated base for 48 h after the curing seal had been applied. An asphalt-rubber SAMI was placed by the Arizona Refining Company (ARCO) across the full roadway width in accordance with specifications (see Figure 7). The chip spreader followed immediately behind the ARCO asphalt-rubber spreader. Several weeks later, the AC was laid in 2-in lifts over the three sections.

FIELD QUALITY CONTROL

The key to a successful stabilized base is close field supervision. Parameters such as spread rate, moisture content, and compaction must be continuously monitored. On average, the cement spread rate was checked three times per run. A run consisted of 300 ft. If the cement spread rate was found to vary from 6 percent by dry weight of aggregate, tests were taken more often until the correct rate was achieved. Also, the moisture content was monitored three times per run. Best compaction and mixing were achieved if the soil moisture content was above optimum prior to the addition of cement and kept near or below optimum during compaction. The depth of stabilization was checked after compaction by using a phenolphthalein solution. Exact cement content could not be determined by using this test, but whether cement was present in any quantity could be determined. When the solution was sprayed on the soil-cement mixture, the cement could be detected by a pink coloration.

Percent compaction was checked at random in one location in the 300-ft run. Special provisions specified compaction at 95 percent of maximum density by using ARIZ Test Method 230 (Field Density by the Sand Cone Method). Compaction was easily achieved by three passes of a 9T vibratory steel-wheeled roller. Surface irregularities were smoothed out by using a rubber-tire roller.

The SAMI was accepted with a manufacturer's certificate. Compaction of the 2-in lifts of AC was monitored according to ARIZ Test Method 412a (Density of Compacted Bituminous Mixtures--Nuclear Method). Density compliance was based on the results of seven random tests per lot taken with a nuclear gauge. Although close supervision eliminates many problems, several construction problems did require special attention.

CONSTRUCTION PROBLEMS AND RECOMMENDATIONS

The finished CTB surface was rough, due primarily to the presence of oversized aggregate in the mix. Most potholes and gouges resulted during the grading of the top 0.25 in. Oversized aggregate exposed by the grader and dragged along the surface was the primary reason for irregularities. Some of the deeper potholes were filled with cold mix and compacted prior to the application of the rubber seal. Many of these problems might have been eliminated if special provisions and construction control had strictly eliminated +1 in of material.

Another problem came up in regard to the allow-

Figure 4. Spreading of cement.

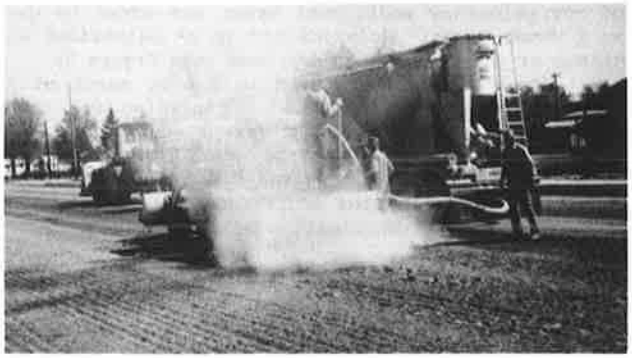
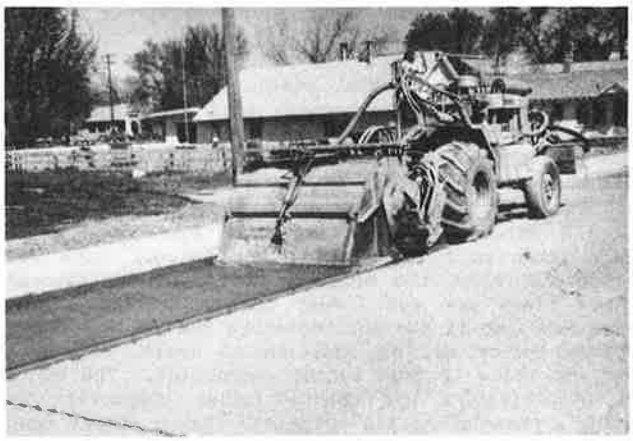


Figure 5. Mixing of cement.



able time frame between the addition of water to the soil-cement mixture and final grading. The special provisions allowed 2.5 h for this process. Due to this short time frame, the contractor was restricted to very short runs of 300 ft. Had this time period been lengthened to 4 h, which is typically recommended by the Portland Cement Association, longer runs would have been attempted and more contractor efficiency would probably have resulted.

Last, the 8-in CTB was covered by one 2-in lift of AC. If the pavement design had called for two lifts of AC, many of the CTB surface irregularities may have been smoothed out.

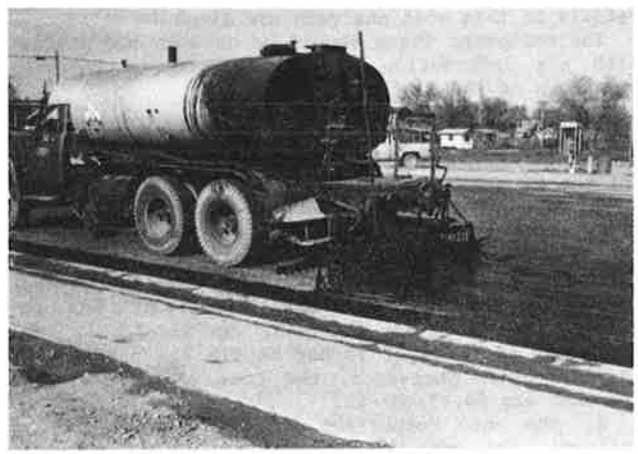
CORRELATION OF FIELD CORES TO LABORATORY CYLINDERS

Eight cylinders were fabricated during each shift. Approximately 40 lb of treated base were sampled from the roadway just prior to compaction. The material was placed in sealed 5-gal plastic buckets and transported to the field laboratory. All eight cylinders were fabricated within 1 h after field sampling. Moisture contents were monitored during fabrication from the residual material shaved off the top of each cylinder. A rapid moisture determination was used to ascertain moisture content. Immediately following fabrication of each cylinder, the specimen was placed on a shelf in the moist room. The moist room consisted of a large stock tank partly filled with water that was covered with plastic and temperature controlled by using a small electric heater.

Four specimens were broken from each set at seven days after fabrication. Two specimens from each set

Figure 6. Finishing with pneumatic roller.

Figure 7. Placing of asphalt-rubber SAMI.



were broken at approximately 28 days. Two cylinders from sets 1-14 were broken at 56 days; the remaining cylinders from sets 15-23 were broken at approximately 180 days.

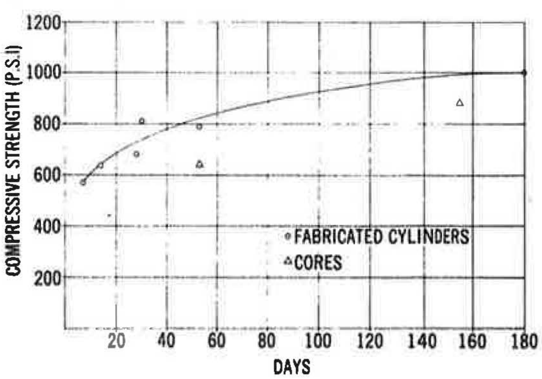
Table 2 illustrates these compressive strengths. The averages and standard deviations of the compressive strengths were determined for the five different curing periods. A plot of the average gain of compressive strength with time is shown in Figure 8. This graph represents a 1.7 percent strength gain/day from the 7th to the 14th day, a 0.5 percent strength gain/day from the 14th to the 28th day, a 0.5 percent strength gain/day from the 28th to the 56th day, and a 0.2 percent strength gain/day from the 56th to the 180th day. This gradual gain of strength is to be expected in cement-stabilization projects, provided that cold weather is not encountered early in the curing process.

In addition to this testing, cores were taken from the roadway at various time intervals and tested for compressive strength. A 53-day-old core taken from the roadway broke at 640 psi. Cylinders fabricated in the laboratory from roadway-mixed material broke at 732 psi at 56 days. This difference represents a 12.6 percent increase in strength from laboratory compaction and curing to field compaction and curing. Cores were again taken from the roadway at approximately 155 days; the average compressive strength was 880 psi. The remaining cylinders fab-

Table 2. Average compressive strength of laboratory specimens.

Set	Station	Avg of 4 Cores 7 Days	Avg of 4 Cores 14 Days	Avg of 2 Cores 28 Days	30 Days	56 Days	180 Days
1	2+50				669		
2	6+25	538			836	907	
3	15+75	570			844	992	
4	21+50	601			895	1 040	
5	26+75	412		575		609	
6	34+50	346		804		907	
7	37+00	364		515		561	
8	41+75	348		724		680	
9	0+10	671		694		892	
10	1175+80	268		495		591	
11	23+97	593		711		808	
12	16+50	637		758		812	
13	12+50		536	513		660	
14	8+50		722	724		831	
15	44+50		731	660			824
16	41+00		569	537			832
17		671		541			836
18		1 076		963			1504
19	6+00	588		732			1158
20	2+10	554		601			776
21	1181+00	666		784			1110
22	1178+00	599		694			816
23	1174+50	782		939			1162
Σ		10 284	2558	12 964	3244	10 290	9018
$\bar{x}$ (avg)		571.3	639.5	682.3	811	791.5	1002
SD, σ n-1		187.4	101.4	136.2	98.2	157.4	246.8

Figure 8. Average compressive strength of cores and laboratory-fabricated cylinders.



ricated in the laboratory were broken at 180 days. The average compressive strength for these cores was 1002 psi.

In several instances, a set of cylinders would have consistently low breaks. Set 10 is an example. When reviewing the data, the problem can likely be directly attributed to the moisture content. Most compaction occurred at 9-10 percent available moisture; in this particular case, moisture ranged from 4.9-6.3 percent, or approximately 2-2.5 percent below optimum. Tests were performed prior to construction to determine the effect moisture content had on compressive strength. The following tabulates this evaluation:

Cement (%)	Optimum (psi)	Optimum Plus 2 Percent (psi)	Strength Loss (%)
6	525	398	24
7	529	334	37

This loss in strength represents an average strength loss of 31 percent for a 2 percent increase in moisture. Similar results could be expected with a marked decrease in moisture content.

Future field testing will include an evaluation of the pavement over the CTB, and it will be compared with the full-depth AC sections. Evaluations will be based on maintenance costs, Mays roughness readings, dynaflect measurements, and percent cracking.

SUMMARY

1. On this project, CTB has proved to be an economical alternative to full-depth AC based on initial cost comparisons when aggregate locations are distant. Cost benefits of CTB can be more attractive if in-place material, including the old AC, can be incorporated into the design. Cost savings of CTB over full-depth AC for this particular project amounted to $120 000. Greater cost savings could easily be achieved if large quantities of suitable road base could be stabilized (stockpiled bituminous material from other projects could be incorporated into a CTB design) or if haul distances to suitable aggregate sources were greater than 25 miles.

2. Construction of CTB can be efficiently and easily accomplished. The contractor on this particular job had no previous experience with CTB. Despite this, close supervision and strict quality control by ADOT technicians ensured an end product that met required thickness and a minimum 7-day compressive strength of 500 psi.

3. If justified during the design process, a minimum of two lifts of AC should be used as the paving course over the CTB in order to smooth out surface irregularities.

4. CTB continues to gain strength with time. The 155-day-old cores had a compressive strength of nearly 900 psi. Strengths of this magnitude lead to skepticism concerning the conservativeness of the structural coefficients awarded to CTB. If long-term strengths in the range of 1000 psi were predictable, perhaps structural coefficients more compatible to those awarded to AC could be assigned to CTB. Part of the long-term evaluations will compare the performance of the so-called structurally inequivalent sections and determine if a higher structural coefficient could be awarded to the CTB based on its performance.

REFERENCES

1. G.R. Morris, N.J. Chen, and J.A. DiVito. Appli-

cation of Asphalt Rubber on New Highway Pavement Construction. TRB, Transportation Research Record 888, 1982, pp. 43-47.
2. G.B. Way. Dewey-Yarber Wash Membrane Encapsulated Subgrade Experimental Project, Final Report. FHWA, 1982.

Economic Analysis of Soil-Cement Base Construction Compared with Crushed-Rock Base

TEERACHARTI RUENKRAIRERGSA

Cement stabilization involves some special equipment with a relatively high initial investment cost, such as a soil stabilizer; this equipment is generally more expensive than other conventional equipment. The economy of employing a soil-cement base to replace a crushed-rock base has been in doubt. An economic analysis is presented that accounts for the cost of renting and operating all equipment for soil-cement production, cost of the cement, and cost of the soil. Results of the analysis indicated that the soil-cement base will be cheaper than the crushed-rock base by about 35-46 percent when the hauling distance for crushed rock is 200-300 km from the job site. It is found that the most expensive item is the cost of the cement and the cheapest is that of the machinery. Cement cost accounts for 57 percent of the total cost; the cost of machinery is only 19 percent.

Soil-cement roads were introduced in Thailand in 1965 by the Siam Cement Company (1). At that time, a 5-km test road was constructed employing lateritic soil-cement as a base course. Lateritic soil and terrace gravel are local materials found in abundance in northeastern Thailand. Rock sources are deficient in this region. After 1965, there was a period of using soil-cement to replace crushed rock in road construction in northeastern Thailand for about five years (1967-1971). Some soil-cement roads, because of the traffic load, developed reflection cracks on the surface, which caused increased maintenance.

The quantity of reflection cracks tends to increase with time and number of heavy trucks. Reflection cracking is due both to the shrinkage of the soil-cement and to deflections from heavy trucks. Differentiation between both modes of crack formation could be made roughly by visual evaluation. For shrinkage cracks, orientation of the crack will be random and it might be longitudinal and transverse in direction. Sometimes it is considered as a surface crack or hair crack because it will form only on the top of the soil-cement road. Deflection cracks are formed because of overloaded trucks or too much flexibility in the lower layer. Again, the crack might be both transverse and longitudinal. However, if the raw soil is well graded, the cracked slab will interlock with the coarse fraction of the soil aggregates. The ones most available in northeastern Thailand are lateritic soil and terrace gravel.

Routine maintenance work to repair the reflection cracks is to seal them with liquefied asphalt or premix. However, it was found that the liquefied asphalt tends to fill the upper part of the crack only, due to the fact that the crack is too small or that viscosity of the liquefied asphalt is too high to penetrate into the lower layer. As a result, reflection cracks will be partly sealed and water penetration through the crack is not completely prevented. The incomplete reflection-crack sealing made the Department of Highways in Thailand doubtful about the performance of the soil-cement road. As a result, soil-cement road construction in Thailand has been terminated, probably since 1972. During 1967-1971, there were about 1400 km of soil-cement road being constructed.

Even though reflection cracks in soil-cement were not properly sealed, it was found that most soil-cement roads exhibited good performance. Periodic maintenance applied on those soil-cement roads is sealing with a single surface treatment (seal coat) after the roads had been serving traffic for five to seven years. Consequently, major or plastic failure is not apparent in this type of road, except for two soil-cement roads that were seriously damaged after being open to traffic for more than 10 years. Now both of them have been partly reconstructed. During the reconstruction phase, one existing soil-cement road was excavated and a silty-clay layer 30-50 cm thick was found. For additional proof, it was apparent that the elevation of the soil-cement layer was nearly the same as that of the surrounding soil and the water table under the embankment was relatively high in the rainy season; this increased the flexibility of the soil-cement base. From this evidence, it might be expected that both the intervening clay layer and the high ground-water layer tend to induce reflection cracks and failure of the two soil-cement roads.

An oil crisis has caused the cost of transportation to increase considerably. Different types of construction materials and goods are loaded onto heavy-duty trucks (10-wheel trucks in Thailand) as much as possible so that the number of hauls could be reduced. The total load of truck and goods will usually be more than 21 tons, which is the legal load limit for this type of truck in Thailand. Evidence from traffic counts and checks along some selected highways showed that the number of heavy buses and trucks varies in the range of 25-40 percent of the average daily traffic, and their gross weight could be as high as 25-40 tons. It can be imagined how much the pavement will deteriorate from the overloaded trucks. The service life of the road pavement was determined from the repetitions of standard axle loads; overloaded trucks increased the deteriorating effect on the pavement many times more than did the lighter ones and shortened the life of the road. From the average gross weight and the traffic composition, repetitions of the equivalent standard load could be estimated by using equivalent factors developed by using data from the road test of the American Association of State Highway Officials (2).

The overloaded trucks transporting the construction material from the crushing plant to the job site tend to destroy the pavement and to shorten its