Performance Evaluation of a Cement-Stabilized Fly Ash Base

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The performance of a compacted, aggregate-free, cement-stabilized fly ash base beneath a highway shoulder is described. A 2.7-m (9-ft) wide, 457.5-m (1,500-ft) long fly ash test section was placed on both sides of State Highway M-54 near Grand Blanc, Michigan, in May 1987. The test section base was constructed using a high carbon, Class *F* fly ash that was stabilized with 12 percent by weight portland cement. A number of tests were used to monitor and evaluate the performance of the base. These tests included (*a*) Clegg impact readings on the compacted surface of the fly ash, (*b*) moisture-density and unconfined compression tests on core samples, (*c*) elevation and vertical deflection measurements on the pavement, (*d*) edge break surveys, (*e*) a crack pattern analysis, and (*f*) leachate analyses. The results of monitoring and evaluation tests conducted to date show that in general the fly ash test section has held up reasonably well where design and compaction specifications have been met. No widespread major problems (e.g., crumbling, disintegration, excessive heave or settlement of the pavement) have occurred during the 5-year period since construction in these areas. Problems with heave and cracking of the pavement on top of the fly ash have so far been restricted to a few local areas. Heaving and cracking in these areas occur mainly during the winter and are associated with frost effects. The presence of a joint that was purposely cut in the surface wearing course between the road shoulder and traveled way on one side of the highway greatly exacerbated arcuate cracking that developed next to the joint.

The Michigan Department of Transportation, which is responsible for 15456 km (9,600 mi) of federal and state trunkline highways, expects to complete 225 km (140 mi) of road shoulder replacement annually. This shoulder replacement program could use 300,000 tons of fly ash annually, or 1/8 of the total annual production in southern Michigan's lower peninsula. Use of fly ash in a roadway base course requires careful design of a fly ash-cement mix suitable for Michigan's demanding traffic loads and winter climate.

Previous research (*1*) has shown that raw fly ash exhibits unsatisfactory freeze-thaw characteristics, thus requiring stabilization with cement. Results of tests on a high-carbon fly ash (*2*) showed that a cement content of 12 percent by dry weight of solids would be sufficient to meet strength-durability criteria and minimize frost heaving. Design thickness calculations indicated that the required thickness of a compacted and trimmed base course was 2.54 cm (10 in.). This design was based on achieving an in-place density of 98 percent of the maximum dry density based on the Modified Proctor test. Protocols (*2*) were also developed for mixing, handling, and placement of the fly ash mixture in the field. The laboratory work indicated the importance of thorough mixing of cement and fly ash to achieve uniform properties and satisfactory performance. Laboratory findings showed that uniform cement dispersion could be achieved by pre-blending a dry 40:60 fly ash and cement mixture. This mixture was then proportioned with fly ash and water to produce the desired base course product containing 12 percent cement by weight of dry solids. A full-scale field trial confirmed the need for this pre-blended concentrate to achieve uniform dispersion.

Fly ash used for this project was a high-carbon, Class *F*, dry hopper ash from Consumers Power Company's D.E. Karn Plan located at Essexville, Michigan, 80.45 km (50 mi) north of the demonstration site. The site selected for the demonstration was a new construction project: a four-lane state highway connecting I-75 directly south of the city of Flint. The trial road shoulder base course was placed on both sides of 457.5-m- (1,500-ft-) long section of the four-lane highway.

The cement-stabilized fly ash base course, mixed on-site using pre-blended 40:60 fly ash-cement mix and conditioned fly ash, was placed and compacted in two lifts. Control of compaction was critical. The first passes on each lift were made with a 10-ton steel wheel roller without vibration, whereas subsequent passes used vibration. A rubber tire roller was then used to close all cracks, thereby producing a smooth base surface. The final dimension of the base course was 2.54 cm (10 in.) thick by 2.74 m (9 ft) wide. The compacted finished surface was protected from drying and rain by application of a sprayed asphalt emulsion. An asphalt concrete leveling course was placed over the fly ash shoulder 7 days after construction and the surface or wearing course was completed 3 days later. The finished M-54 highway was opened to traffic in June 1987. Details about the field construction of the fly ash base course and preliminary findings are presented elsewhere (*2-4*).

A 5-year post-construction monitoring and testing program was implemented to evaluate the performance of the base course under operating conditions. These tests included

1. Moisture-density and unconfined compression tests on core samples from the fly ash base,
2. Elevation and vertical deflection measurements on the pavement,
3. Edge break surveys,
4. Crack pattern analysis, and
6. Analysis of samples from both groundwater monitoring wells and a leachate collection system installed beneath the fly ash test section.

The results and findings of this monitoring program are summarized in the following sections.

TEST SECTION DESCRIPTION

The Michigan Department of Transportation (MDOT) participated actively in the site selection and construction planning phases of

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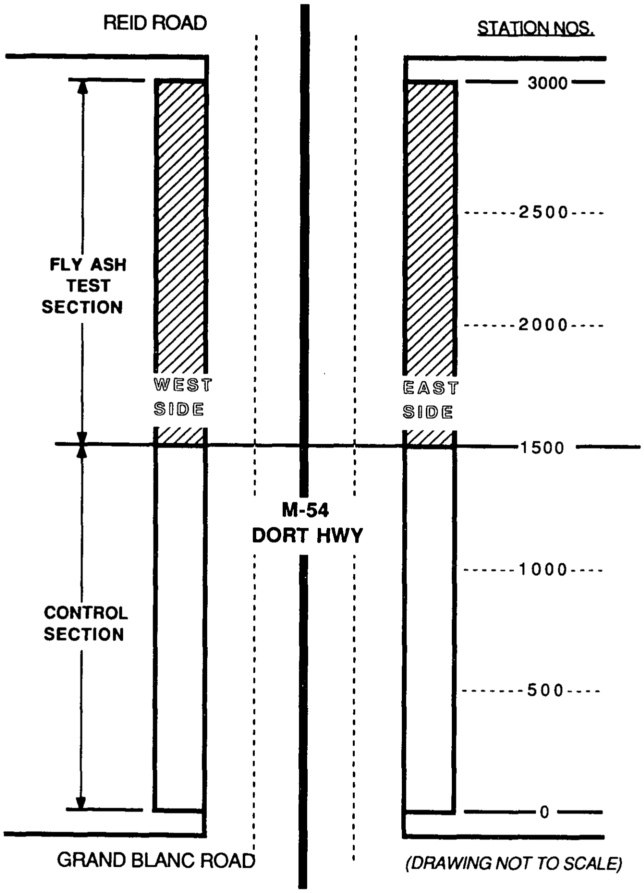


FIGURE 1 Schematic diagram showing plan and layout of road shoulder test.

the project. A new highway construction project on Michigan Route M-54 emerged as the most promising location from a number of candidate sites considered. A 457.5-m- (1,500-ft-) long section between Grand Blanc Road and Reid Road (see Figure 1) was selected for a trial fly ash shoulder. The combined length of the fly ash shoulder on both sides totaled 915 m (3,000 ft). An additional 915-m (3,000-ft) length of shoulder adjacent to the fly ash test section was established as a control section. At present the traffic is of medium intensity, but is expected to grow with time. To accommodate the traffic, the asphalt pavement was made 14.6 m (48 ft) wide with two 3.66-m (12-ft) lanes in each direction. The total width of the roadway, including the 2.7 m (9-ft) shoulders on each side, was 20.1 m (66 ft).

The required thickness of the fly ash base was calculated using the American Association of State Highway and Transportation Officials (AASHTO) design equation. The soil support value was assumed to be 3, which is equivalent to a California Bearing Ratio of about 3, and the minimum required sand subbase thickness was taken as 30.4 cm (12 in.). The final thickness of the compacted cement stabilized fly ash base course for the M-54 shoulder was calculated to be about 25.4 cm (10 in.). This design thickness was reasonable, based on previous experience. The adjacent control section had a standard Michigan Class *A* design with a 12.7-cm (5-in.) asphalt concrete base, 10.1-cm (4-in.) aggregate base and a 6.3-cm- to 7.6-cm- (2.5 to 3-in.-) thick asphalt surface supported on a sand subbase. A stratigraphic profile of both the fly ash and control section is shown in Figure 2.

PERFORMANCE EVALUATION TESTS

Field Moisture-Density Tests

The compacted, cement-stabilized fly ash base course was constructed during the period May 14 to 19, 1987. MDOT drilled 10.1-cm- (4-in.-) diam core samples from the fly ash test section on August 1987, December 1987, and April 1988. These dates correspond approximately to 90-, 180-, and 270-day test samples. The cores were sealed in plastic bags and transferred to the laboratory for determination of their moisture content, density, and unconfined compressive strength.

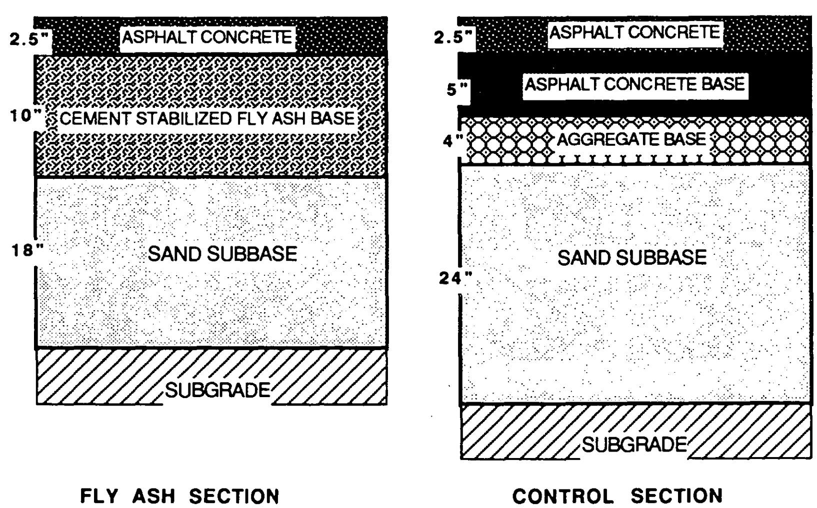


FIGURE 2 Stratigraphic profile of fly ash test section and control section.

Samples were obtained from 9 to 10 coring locations on either side of the highway. The locations were selected randomly, with the exception of two locations that were cored each time in the same vicinity of stations with known high and low Clegg impact readings, respectively; namely, Stations 26+00 (west) and 15+00 (east), respectively. Test specimens were recovered from both the top and bottom portions of the fly ash field cores.

Moisture contents for the 90-day cores samples ranged from 26 to 44 percent (dry weight basis) with an average value of 33.5 percent. With one or two possible exceptions, all of the cores samples exhibited moisture contents well in excess of the Modified Proctor optimum moisture determined in the laboratory investigation to lie between 24 and 26 percent of dry weight for cements contents ranging from 0 to 15 percent dry weight. Moisture contents measured on the 180-day (6-month) cores were even higher, ranging from 29 to 53 percent with an average value of 36.5 percent.

Dry densities for the 90- and 180-day samples averaged 11.0 and 11.5 kN/cu.m (69 and 72 pcf) respectively. These values lie below the target density of 12.4 kN/cu.m (78 pcf), and correspond to a relative compaction of slightly under 90 percent based on the modified AASHTO test. These dry densities were also lower than the dry densities measured on the compacted base using a nuclear gauge [average = 12.8 kN/cu.m (80.2 pcf), range = 11.8 to 14.3 kN/cu.m (74.3 to 89.7 pcf)]. Cement contents in the field cores varied from 2.8 to 19.1 percent dry weight. The target cement content was 12 percent. This variation is consistent with cement contents measured on pugmill samples during construction, which varied from 5.9 to 18.5 percent by weight with an average of 11 percent.

Unconfined compression tests were run on 5.1-cm (2-in.) cube samples that were trimmed from the cores by dry sawing. The average unconfined compressive strength at 90 and 180 days was 3369 and 2825 kPa (489 and 410 psi) respectively. These averages exceed the minimum required 7-day strength of 2756 kPa (400 psi). The average unconfined compressive strength at 270 days was unreliable because the sample population was too small and included samples that were either shattered or too soft, and hence not usable. Although the average strength met or exceeded the minimum value required, there was considerable variation among samples. Strengths ranged from less than 689 kPa to about 6890 kPa (100 to 1,000 psi). Only about half the samples tested met or exceeded the minimum target strength of 2756 kPa (400 psi). No additional strength gain was observed after 90 days of elapsed time.

Correlations were examined among unconfined compressive strength and other soil properties such as density, moisture content, cement content, and Clegg impact readings. The relationship between unconfined compressive strength versus dry density at 180 days is plotted in Figure 3. Fairly good correlations were obtained, with the strength increasing with dry density in an exponential fashion as shown. The coefficient of correlation for 90- and 180-day strength versus dry density was 0.61 and 0.95, respectively.

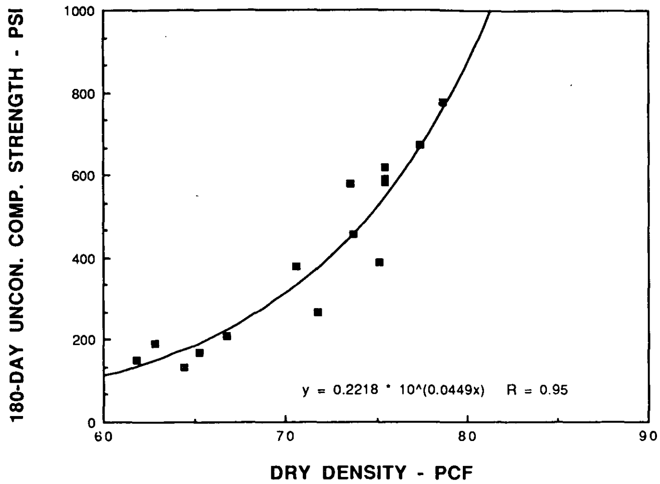


FIGURE 3 Unconfined compressive strength versus dry density: 180-day samples (1 psi = 6.89 kPa).

Clegg Impact Tests

Impact tests were run on the compacted surface of the fly ash base course using a Clegg impact tester. This device records the deceleration of a standard weight dropped from a fixed distance onto the base course surface. The Clegg "impact" reading obtained in this way can be correlated (5) with the modulus and strength of a compacted base. This test was run every 15.2 m (50 ft) along the surface and at different elapsed times up to 4 days following compaction. The test is nondestructive, fast, and provides a good record of the spatial variation and development of strength or stiffness with time.

Typical results of the Clegg impact test along the length of the fly ash test section on the west side of the highway are shown in Figure 4. Strength gains tended to be higher on the south end of the project. In addition, east side readings were generally higher than the corresponding west side readings on the opposite side of the highway. A pronounced dip in Clegg impact values was recorded in the vicinity of Stations 26+00 to 27+00 on both sides of the highway. These low values indicated a weaker base at this location and helped to identify an area that required close scrutiny as the study progressed.

There are several possible reasons for the low values at Stations 26+00 to 27+00. Low cement contents are a possible explanation; however, the coincidence of low values at exactly the same location but on opposite sides of the highway suggest other explanations as well. The highway traverses a topographic low at this location. A small amount of standing water was observed in the road shoulder trench before placement of the fly ash base. This moisture may have adversely affected compaction of the fly ash and the cement set, thus explaining a much lower rate of strength increase relative to other locations.

Edge Break Survey

Another indicator of stability and durability of the road shoulder is the extent of "edge breaking" along the outside edge of the pavement. The extent of edge breaking was assessed along the fly ash and control sections, respectively. Edge breaking was classified as either major or minor. "Major" refers to crumbling, disintegration, or excessive settlement at the edge of the pavement, whereas "minor" refers to a slight cracking or deflection at the edge.

The amount of edge breaking in these two categories was paced off along the entire length of the control and fly ash sections in June of 1989. The results of the survey showed that edge breaking was not as extensive along the fly ash test section compared with the control section. This finding was true for both categories of severity. It is interesting to note the pronounced contrast in major edge breaking along the fly ash test section between the east side (15 paces) versus the west side (109 paces). This difference in resistance to serious disintegration and crumbling at the edge is also consistent with the findings of the Clegg impact tests (see Figure 4) on the surface and unconfined compression tests on core samples. In general, impact readings were much higher on the east than on the west side. The same also holds true for unconfined compression test results.

Vertical Displacement Measurements

During the winter of 1988 to 1989, a visual inspection of the fly ash road shoulder in March appeared to indicate that some vertical heave had occurred at certain locations. The heave was manifest most visibly in the form of a localized, slightly raised crown or mound with an associated cracking pattern on the west side road shoulder in the vicinity of Station 25+00. A slight amount of apparent differential heave also took place over some distance along the joint that had been cut in the pavement between the traveled way and the shoulder. This vertical heave subsided, however, to a very small residual value by May 1989.

In order to study and document this suspected vertical heave more thoroughly, two series of vertical displacement measurements were initiated in the fall of 1989. The first consisted of measuring the vertical separation distance or offset every 7.6 m (25 ft) along the joint. The second consisted of running a line of levels every 30.5 m (100 ft) down the middle of the fly ash test section road shoulder on either side of the highway. Both these measurements were repeated periodically during succeeding years to see if frost heaving was occurring, and if so, its magnitude and extent.

Results of these measurements show that a considerable amount of vertical movement (or heave) has taken place along some portions of the shoulder. Vertical deflections ranged as high as 6.8 cm (2.7 in.) along the joint in some locations after 5 years. The temporal and spatial distribution of vertical heave or offset along the joint are shown in Figures 5 and 6. Compared in Figure 5 are the vertical offsets measured in 1990, some 3 years after construction, and compared in Figure 6 are offsets measured in 1992, approximately 5 years later. Vertical offsets were measured during the winter and in the late spring or early summer. The residual vertical heave along the joint measured in late spring or early summer was approximately half that recorded in February during the height of the winter when the ice lenses in the base course were fully developed. Vertical heaving or deformation increased progressively with time, as can be seen by comparing vertical deflections in 1990 (Figure 5) versus 1992 (Figure 6).

The vertical offset measurements at the joint are a good surrogate for heave-induced elevation changes. This equivalency was demonstrated by a strong correspondence between vertical offset at the joint versus elevation change along the centerline of the shoulder. East and west side elevation changes along the center

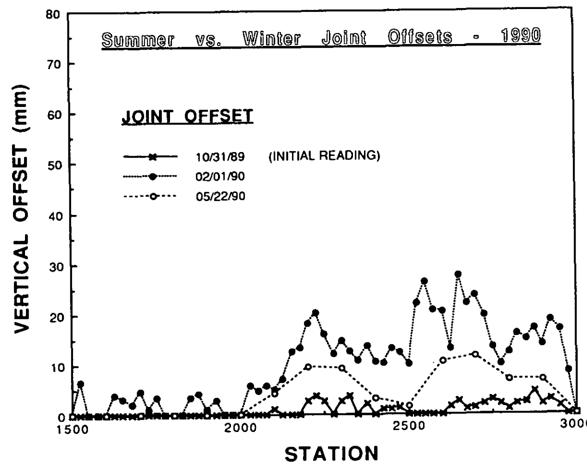


FIGURE 5 Vertical offsets measured in 1990 at joint in pavement between shoulder and traveled way.

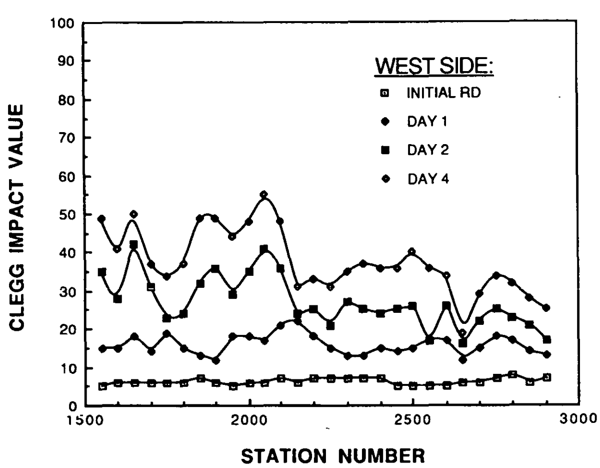


FIGURE 4 Clegg impact readings at different elapsed times and locations along the west-side shoulder.

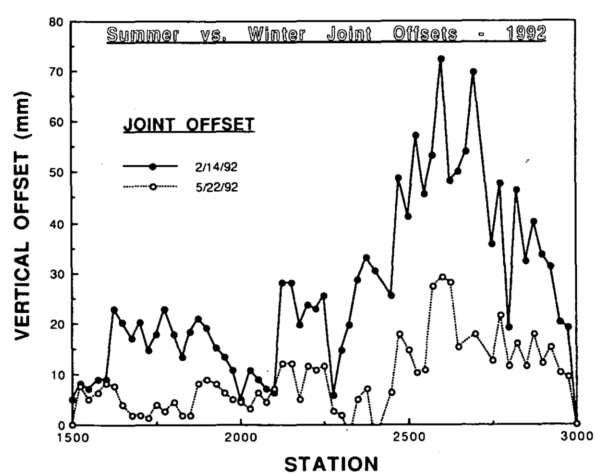


FIGURE 6 Vertical offsets measured in 1992 at joint in pavement between shoulder and traveled way.

of the shoulder measured on February 1, 1990, are compared in Figure 7. The two curves are virtual mirror images, with the exception that vertical heave tends to be greater (approximately 2 times) between stations 15+00 to 24+00 on the west side compared with the east side. This finding is particularly significant because in spite of the generally higher heave on the west side, there is little or no sign of cracking or distress where the shoulder abuts the roadway. In other words, it appears that cutting a joint—the procedure adopted on the east side—has contributed largely to the subsequent vertical offset measured along the joint and associated cracking and deformation problems (discussed in the next section). The presence of the joint has allowed runoff from the roadway to flow down into the subbase. The problem is exacerbated when the shoulder heaves (relative to the roadway) and presents a vertical face that diverts runoff vertically downward directly into the base. This infiltrating water becomes a source of water for ice lense formation in the underlying base during the winter.

Vertical displacement or heave is generally lower from Stations 15+00 to 24+00 compared with the area from Stations 24+00 to 30+00. The stabilized base in the former area tended to have higher cement contents and compacted densities. Analysis of field cores revealed relatively high cement contents in this area (e.g., 19.1 percent at Station 16+36 and 15.4 percent at Station 21+00, respectively). From Stations 25+00 to 28+00, the measured frost heave was virtually the same, but much higher for both shoulders compared with the area between Stations 15+00 to 24+00. This larger frost heave correlated with low cement contents in the base (e.g., 2.8 percent at east side Station 26+07). This is also the same area in which relatively low Clegg impact readings were recorded immediately after construction, as shown previously in Figure 4.

Crack Pattern Survey

Periodic inspection visits were made to the site following construction of the road shoulder base in May 1987. One of the main purposes of these visits was to look for any crack development in the asphalt pavement cap. The first crack was observed on April 13, 1988, in the vicinity of Station 25+10 on the west side shoulder. The cracking consisted of two narrow approximately parallel cracks with a combined length of 1.82 m (6 ft) at a distance of about 0.91 m (3 ft) from the traveled way pavement. No other structural cracks were observed in either shoulder at that time.

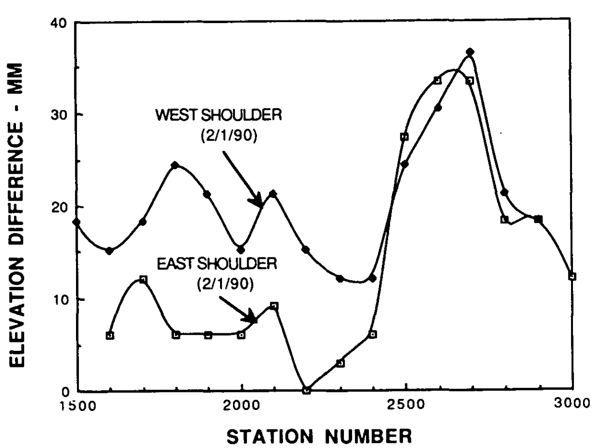


FIGURE 7 Comparison between elevation changes along centerline of east and west shoulders, respectively.

Four major types of cracking patterns have subsequently developed in the pavement overlying the fly ash base course during the 5-year monitoring period. These categories include the following:

Type I: Arcuate-shaped cracks along the joint between the shoulder and traveled way resulting from traffic passing over a raised and weakened portion of the shoulder;

Type II: General, random cracking associated with areas of under compaction or insufficient cement content in the fly ash;

Type III: Cracks originating in and propagating away from core holes in the pavement; and

Type IV: Cracks propagating away from joint corners.

Crack growth tended to progress with time. The character and development of arcuate (Type I) cracking along the joint at Station 25+30 on the east side is shown in Figure 8. The development of random (Type II) cracking at Station 25+10 on the west side is shown schematically in Figure 9. The crack growth versus time at Station 25+10 (west side) is illustrated in Figure 10.

The temporal pattern of crack development at Station 25+10 (west side) shown in Figure 10 indicates that crack growth is most rapid during the winter months and is associated with frost heaving in the base. The density and strength of cores from this area are generally much lower than those at other locations. The cement content of a core from this location was also quite low (2.8 percent by weight). Visual evidence during the winter indicated that localized heaving was occurring here as well. Snow plow striations in the pavement surface were clearly visible here as a result of a slightly raised "mound" in the pavement surface.

Influence of Pavement Joint

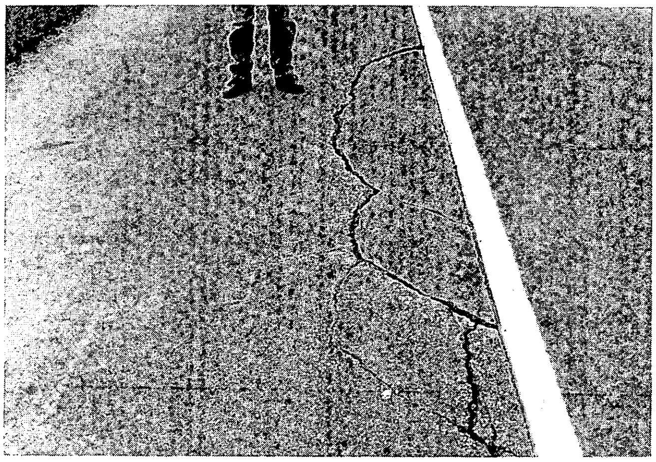
The traveled way wearing or surface course was continued and extended across the shoulder on the west side. A different procedure was followed, however, on the east side. Here a 0.6-cm- (1/4-in.-) wide and 3.8-cm- (1 1/2-in.-) deep groove was cut and sealed right above the construction joint where the traveled way pavement base and the fly ash base (shoulder base) meet. This procedure (grooving) is often specified when two different, adjacent base materials are used. This way the shoulder can move somewhat independently as a result of material and other influences. Unfortunately, the presence of this joint also contributed to localized (Type I) cracking problems observed in the east-side shoulder.

Pronounced arcuate (Type I) cracks developed adjacent to the joint on the east side at Stations 25+30, 26+90, 27+20, and 27+50. The cracks at these locations are crescent-shaped features oriented parallel to the joint. These cracks coincide with areas of maximum joint offset (see Figures 7 and 8) and generally lower strengths (see Figure 4). The cracks most likely developed in response to traffic loads during the spring thaw when the shoulder was raised in a sharp vertical discontinuity at the joint. This problem was not observed in the west-side shoulder in spite of generally higher elevation changes along the west side (see Figure 9). This finding indicates that it would have been advisable not to cut a joint in the pavement. The presence of the joint not only tended to divert run-off water downward into the fly ash base, it also contributed to the cracking problems observed along the east-side shoulder.

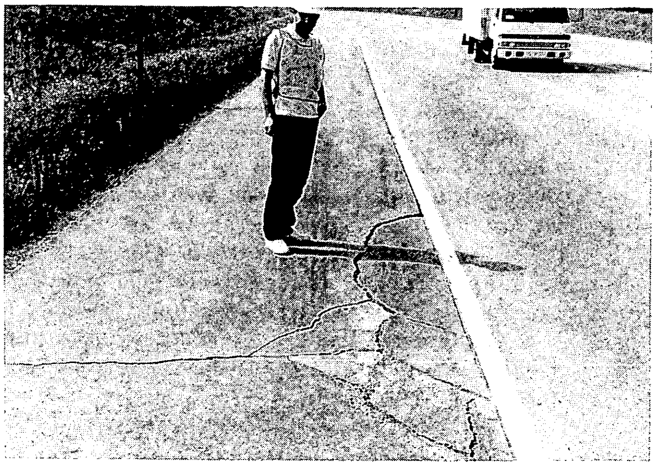
In general, any kind of penetration through the pavement (e.g., grooves, joints, or holes) adversely affected the pavement. Cracks originated in and propagated away from core holes and joint corners as well. The deleterious effect of the core holes was particularly pronounced; these holes were often the locus of major transverse cracks in the fly ash pavement.

ENVIRONMENTAL MONITORING

Both laboratory leaching tests and field monitoring of water quality in observation wells and leachate collection stations were implemented as part of the demonstration project. The purpose of



(a)



(b)

FIGURE 8 Arcuate cracking along joint at Station 25+30 (east side) showing progression of cracking with time: (a) July 19, 1991, and (b) May 5, 1992.

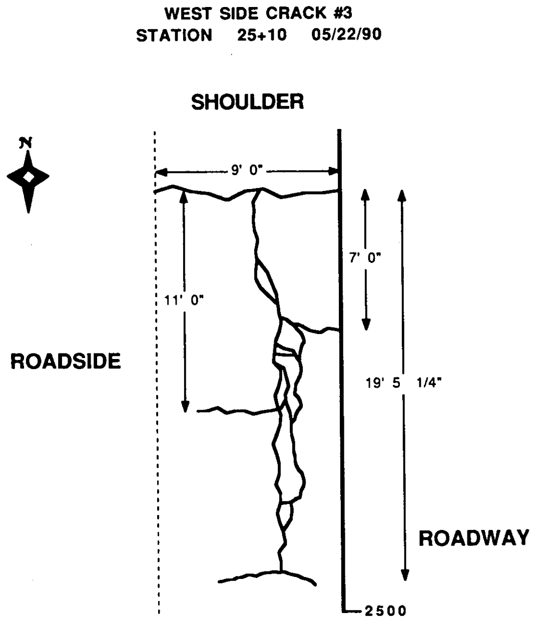


FIGURE 9 Schematic diagram of cracking pattern measured May 22, 1990, in vicinity of Station 25+10 on west-side shoulder.

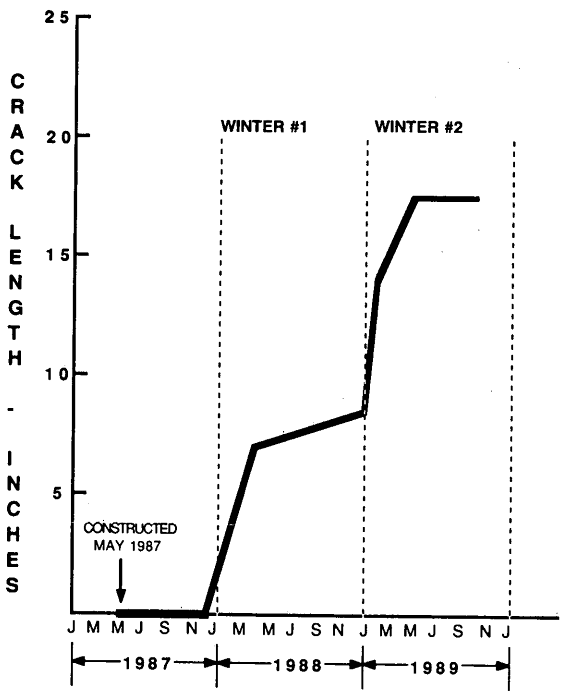


FIGURE 10 Crack growth with time at west-side Station 25+10 (1 in = 25.4 mm).

the laboratory leaching studies was to ascertain the potential for contaminant generation by cemented fly ash under extreme leaching conditions. The field monitoring system was installed in order to determine the impact, if any, of the cemented fly ash base course on water quality.

Laboratory Leaching Studies

In order to simulate the "worst case" condition, two relatively extreme leaching protocols were followed. In the first, two samples of cemented fly ash were subjected to the leaching procedures established by the Environmental Protection Agency (EPA) under the Resource Conservation and Recovery Act (EPA Method 1310). The analytical results obtained on the resulting leachate are summarized in Table 1. The recommended limits established under the Safe Drinking Water Act (SDWA) are included in this table for reference purposes. In general, the concentration of metal ions and in the leachates from the cement-fly ash samples are below the recommended limits established by the SDWA. Only chromium and selenium slightly exceeded the SDWA standards. Even these exhibit less than an order-of-magnitude concentration above the standards.

Field Monitoring

The laboratory studies indicated the potential for certain metal ions being released from pulverized, cemented fly ash. Thus, particular emphasis was placed in the field monitoring study on metals analysis. Groundwater monitoring wells were installed adjacent to the roadway in both the fly ash and control sections. Leachate samples were taken from sub drain pipes, placed directly beneath the fly ash base, which discharged into a man hole sampling box. The leachate samples were collected by positioning a carefully pre-cleaned stainless steel bucket at the pipe opening and catching the leachate as it flowed from the pipe.

Results from the groundwater monitoring wells are unremarkable. Insufficient time has elapsed for any leachate to reach the wells and no significant differences have been observed to date between the background and test section wells. Leachate samples from the sub drains were analyzed for a number of toxic ions including selenium, arsenic, chromium, and cadmium. Typical results are shown in Figure 11, where dissolved concentrations are plotted versus collection date for arsenic. Results were plotted only for the north collection station, which receives leachate from the fly ash test section. Leachate concentrations for dissolved cadmium and chromium have remained relatively constant over time. On the other hand, selenium and arsenic concentrations have tended to fluctuate over time. The latter trend is particularly noticeable in the case of dissolved arsenic, as shown in Figure 11. Concentrations peaked in June 1989 and again in November 1991. It is important to note, however, that the maximum dissolved arsenic concentration in the leachate is still well below the Safe Drinking Water Standards (SWDS), as shown in the following table:

Parameter	Sub Drain Leachate (mg/liter)	SDWS (mg/liter)
Arsenic (As)	0.013	0.050
Cadmium (Cd)	0.020	0.010
Chromium (Cr)	0.030	0.050
Selenium (Se)	0.015	0.010

Only a limited number of usable samples have been obtained from the south collection station, which services the control section. A comparison of leachate concentrations (Arsenic, Cadmium, Chromium, and Selenium) between the fly ash test section and control sections on three different dates (07/90, 10/90, and 11/91), reveal little difference in concentrations between the two sections. Arsenic and selenium leachate concentrations tended to be slightly higher in the fly ash test section and this difference appeared to increase with the passage of time. These undiluted concentrations are nevertheless still within or near SDWS.

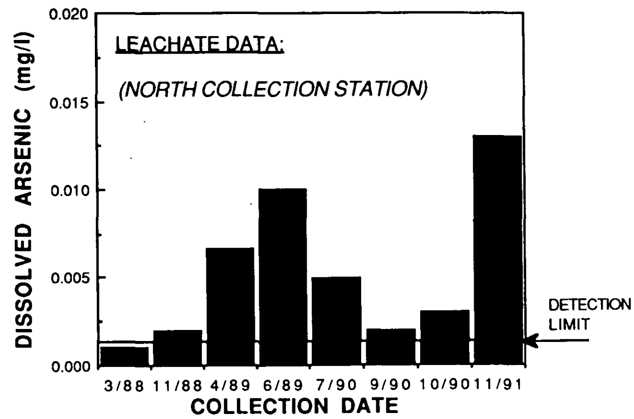


FIGURE 11 Sub drain leachate test results for dissolved arsenic.

TABLE 1 RCRA Leaching Test Results on Cement-Stabilized Fly Ash

COMPOUND	LEACHATE CONC - MG/LITER		
	SAMPLE "A"	SAMPLE "B"	SDWA LIMITS
Arsenic	0.009	0.005	0.050
Barium	0.070	0.460	1.000
Cadmium	<0.002	<0.02	0.010
Copper	<0.02	<0.02	1.000
Chromium	0.160	0.070	0.050
Lead	<0.02	<0.02	0.050
Mercury	<0.0002	<0.0002	0.002
Selenium	0.024	0.014	0.010
Silver	<0.01	<0.01	0.050
Zinc	<0.02	<0.02	5.000
Cyanide	<.005	<0.005	0.200

Notes:
1. Fly ash stabilized with 10% by wt. Portland cement
2. Molding w/c: "A" = 20 %; "B" = 28 % (dry wt. basis)

CONCLUSIONS

Post-construction monitoring of the fly ash-cement base has shown that if the fly ash is mixed correctly with the specified amount of cement and compacted to the specified density, it will perform well. Problems with surface heave and pavement cracking have so far been restricted to a few local areas. These localized problems appear to be the result of low density and strength in combination with a low cement content in the fly ash base and a thin asphalt cap.

A groove or joint should not be cut in an asphalt pavement between the shoulder and traveled way. The joint ultimately intercepts and diverts runoff and meltwater into the underlying fly ash base with serious consequences. The presence of a joint was the main cause of severe heave and cracking in sections of pavement on the east side.

Monitoring of groundwater well samples and sub drain leachate has not revealed any adverse impact of the fly ash-cement base on water quality to date.

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REFERENCES

1. Gray, D. H., and Y. K. Lin, Engineering Properties of Compacted Fly Ash. *Journal of Soil Mechanics and Foundations Division* (ASCE), Vol. 98, SM4, 1972, pp. 361–380.
2. Berry, W. H., D. H. Gray, U. W. Stoll, and E. Tons. *Use of Coal Ash in Highway Construction: Michigan Demonstration Project. Interim Report.* Electric Power Research Institute, EPRI GS-6155, Palo Alto, Calif., 1989.
3. Gray, D. H., E. Tons, and W. H. Berry. Design and Construction of a Cement-Stabilized Coal Ash Base Course in a Highway Pavement. In *Proc., International Symposium on Utilization of Coal Mining Wastes,* Glasgow, Scotland, Sept. 2–5, 1990.
4. Gray, D. H., E. Tons, and W. H. Berry. Post-Construction Monitoring and Performance Evaluation of a Cement-Stabilized Fly Ash Base. In *Proc., 9th International Ash Use Symposium,* Vol. 2: Stabilization and Aquatic Uses, EPRI GS-7162, Jan. 1991, pp. 45:1–16.
5. Clegg, B. An Impact Testing Device for In Situ Base Course Evaluation, In *Proc., 8th Australian Road Research Board Conference,* Perth, Australia, Vol. 8, No. 8, 1976, pp. 1–6.