Reviewed By:

M.W.Lawson P.E.
Materials & Research
Engineer

Prepared By:

Peter C. Winters
January 8, 1990

RESEARCH UPDATE

NUMBER U90-1

3M 350 TAPE ON OPEN GRADED PAVEMENT - I 91 LYNDON-BARTON

REFERENCE: Work Plan 88-R-3, Update U88-10

HISTORY:

During construction of Lyndon-Barton IR 091-3(10), an experimental durable pavement marking, 3M 350 Tape, was specified for 8" white gore markings and 4" white lanelines on the accel/decel lanes at interchanges 23, 24, 25, and at a rest area and two scenic turnouts on this resurfacing project.

APPLICATION:

During a six week period ending on August 17, 1989, the experimental product was installed using the manufacturer's recommendations. A manufacturer's representative was present during some of the application to assure that contractor personnel were following proper application procedures.

As a control, durable markings of an approved thermoplastic material were installed at the following locations.

NB MM. 156+/- Barton Interchange (25) Off ramp
NB MM. 154+/- Glover Scenic Overlook On & Off ramps
NB MM. 143+/- Wheelock Scenic Overlook On & Off ramps

A total of 2430 LF of 8" and 2545 LF of 4" white tape markings was installed.

A total of 2499 LF of 8" and 1152 LF of 4" white thermoplastic markings was installed.

COST:

The price for 4" white line Tape was $1.50 per LF.
The price for 4" white thermoplastic line was $0.50 per LF.
The price for 8" white line Tape was $3.00 per LF.
The price for 8" white thermoplastic line was $1.00 per LF.

STATUS:

The project was inspected on January 3, 1990. Substantial loss of product had occurred as shown below.

8" White tape lines - 48% loss.
4" Laneline dashes - 52% loss.

STATUS:continued

The thermoplastic gore markings showed no loss and only very slight scuffing at one location where the long lines and diagonal chevrons overlapped.

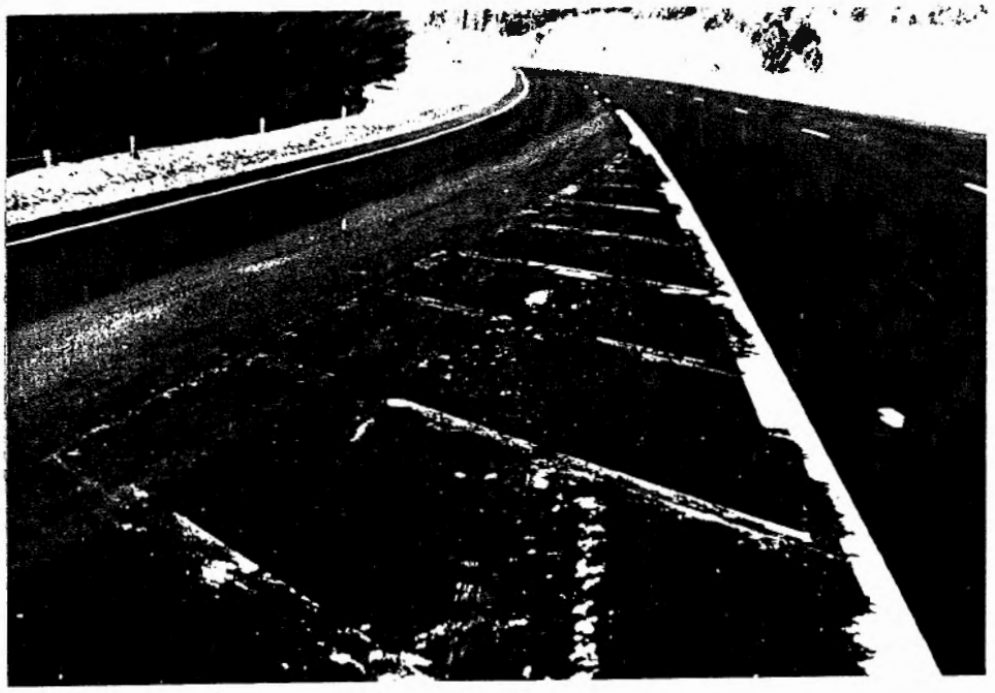
The painted edgelines on this project were all visible although somewhat yellowed as is expected for a first application over new asphalt.

At each location a test of the bond of the tape to the pavement was made by grasping the tape with a finger and thumb, then gently pulling. Only minimal pressure was required to remove the tape from the surface. After removal of the tape, standing water was found in all of the depressions in the pavement. The lack of bond suggests that most of the remaining product will be lost by the end of the winter season.

CONCLUSIONS:

The percentage of product lost after only four months is unacceptable. Further use of 3M 350 Tape on open graded friction courses is not recommended.

FOLLOW UP: A final evaluation will be made in the spring.



Distribution A,B,C,D,E

Reviewed By:

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Prepared By:

Peter C. Winters
January 9, 1990

RESEARCH UPDATE

NUMBER U90-2

3M 350 TAPE ON OPEN GRADED PAVEMENT - I 89 COLCHESTER

REFERENCE: Work Plan 88-R-4, Update U88-10

HISTORY:

During construction of Colchester IR 089-3(14), an experimental durable pavement marking, 3M 350 Tape, was specified for the Northbound and South bound 10' x 4" white dashed centerlines on this resurfacing project. Control was to be the edgelines which were painted with standard traffic paint.

APPLICATION:

The application of the experimental product occurred during July of 1989, ending on July 31, 1989. The special provisions of this contract called for the presence of a manufacturer's technical representative on the project at all times. Although the manufacturer did not provide full time coverage, a representative was present during some of the application to assure that contractor personnel were following proper application procedures. Fifteen thousand nine hundred LF of 4" white line was applied between milemarkers 92/00 and 98/00 on both Northbound and Southboundlanes.

COST: The price for this product was $1.50 per LF. installed for a total cost of $23,850.00.

STATUS:

On August 18, 1989 a cursory inspection of the product was made. At that time it was noted that the two week old product was "slightly gray - about equal to the painted edgelines." Only one damaged stripe was noted with a loss of approximately 6" of tape. It was also noted that the tape could be lifted from the surface of the pavement with only gentle pressure using fingers only.

The project was inspected on January 4, 1990 by Research & Development personnel. Substantial loss of product had occurred as shown below.

Percent Loss	None	1 to 10%	10 - 90%	90 - 100%	Overall
Northbound					
No. Dashes	439	56	83	214	33%
Southbound					
No. Dashes	551	125	57	66	12%

STATUS:continued

An independent inspection was made by Maintenance division personnel on January 5, 1990. This inspection confirmed the percentages lost.

As can be seen, the southbound is in better shape but 12% is still a significant loss. On the southbound lane much of the loss is leading edge damage in the 1 - 10% category. On the northbound many of the stripes are in the 90 - 100% loss category and occurred in groups which accentuates the lack of delineation in some areas. Many of the 90-100% lost stripes have 2 to 4" of the leading edge still present. These were tamped down during application to obtain a lower leading edge profile.

The painted edgelines on this project were all visible although somewhat yellowed as is expected for a first application over new asphalt.

CONCLUSIONS:

The percentage of product lost after only four months is unacceptable. Further use of 3M 350 Tape on open graded friction courses is not recommended.

FOLLOW UP: A final evaluation will be made in the spring.

Distribution A,B,C,D,E

Reviewed By:

Milan W. Lawson, P.E.
Materials & Research Engineer

Prepared By:

Ronald I. Frascoia
January 29, 1990

RESEARCH UPDATE

NUMBER U90-3

CARBON BLACK MODIFIED PAVEMENT

Reference

Research Investigation P85-4, Work Plan 86-R-8, Update U88-1

Purpose

This update reviews the performance of carbon black modified and high stability control mixes in resisting the redevelopment of wheel path rutting.

History

A 3500'+ section of Route 7 paved in 1979, Court Street in Middlebury, developed distress in the form of rutting and shoving shortly after placement. By June of 1985, rutting in the wheelpaths averaged 10/16" with maximum values of 1 11/16". An investigation, completed in July 1985 (P85-4), concluded that the pavement distress was the result of a number of factors, but that the major cause was due to instability of the plant mixed base course.

In 1986, a decision was made to reconstruct two 600+ foot sections of Court Street at the School Street and Charles Avenue intersections in an attempt to determine what would be required to rehabilitate the entire project in future years. Construction was carried out under project Middlebury F019-3(39)S in August and September, 1986. The procedure included the stepped removal of the existing pavement courses with full removal over the mid section. Both mixes used AC20 asphalt with a 75 blow Marshall test criteria in place of the standard 50 blow requirement. Stability requirements were increased from 1000 lb. to 1500 lb. for Type I and to 1800 lb. for Types II and III. The pavement typical substituted two 2 1/2" courses of Type I for the standard plant mixed base course with one 1 3/4" course of Type II and one 1 1/4" course of Type III. Carbon black (MICROFIL 8 by Cabot Corp.) was added at a rate of 25 lb. per ton to the mix placed at the Charles Avenue intersection. Traffic was kept off the new pavement until the surface temperatures had cooled to 100°F.

Cost

The 730 tons of bituminous mix with carbon black added was bid at $78.00 per ton. The high stability control mix was bid at $35.00 per ton which was only slightly higher than the project average unit price for the item. The Cabot Corporation reports that carbon black (MICROFIL 8) was available at a cost of $0.33 per pound FOB Franklin, LA or $0.37 per pound at Middlebury, VT. The cost of the product has not changed through March of 1990. Cabot's Project Manager currently recommends that carbon black be added at the rate of 15% of the asphalt content if AC 20 asphalt is used. That would amount to a loading of 18 lbs., of carbon black with a reduction in the asphalt content of 11 lbs. per ton. Such usage would result in a $6.00 per ton increase in the material cost of the modified mix.

Status

Traffic volumes have increased over the test sections and are now exceeding 16,000 vehicles per day. The sites have been examined for signs of distress or rutting each fall since placement. The results have been documented as follows:

	Carbon Black	Control
Pavement Cracking	1 diagonal crack	no cracks

Ride Quality - Mays values in inch/mile at 35 mph

	1 Year (11-87)
Carbon Black	117
Control	111
Old Pavement	198

Rut Measurements - in 1/16 inch increments

Depth Replaced	1 Year (10-87)		2 Years (11-88)		3 Years (10-89)	
	CB	Control	CB	Control	CB	Control
0	6.2	14.2	11.0	17.5	11.8	15.5
1	2.8	--	5.5	--	6.2	--
1 1/2	1.2	3.0	3.2	6.5	3.2	7.0
3	0.8	2.2	2.2	4.2	2.0	4.2
5 1/2	0.5	1.2	2.2	3.8	2.2	4.2
8	0.5	1.2	1.7	3.9	2.2	3.4

Summary

Significant findings noted through three years of service include the following:

One inch of carbon black modified mix was unable to prevent rutting

which averaged 6/16 of an inch.

One and one half inch of the high stability control mix was unable to prevent rutting which averaged 7/16 of an inch.
One and one half inch or more of the carbon black modifier mix and three inches or more of the high stability mix were able to resist above normal rutting through three years of service.

There were no increases in the amount of rutting from the second to the third year of service on either type of mix at the three inch to eight inch locations.

Preliminary Conclusions

All thicknesses of the carbon black modified mix developed less rutting than equal thicknesses of the high stability control mix.

There was no abnormal rutting in either the carbon black or control mixes where they were placed at a thickness of three inches or greater.

There was no increase in rutting at the three inch or greater thickness with either mix between the second and third years. Such results suggest the 4/16 inch average rutting in the control mix and the 2/16 inch average rutting in the carbon black mix may be the result of normal pavement consolidation rather than the plastic flow or migration of mix which is associated with serious rutting problems.

Recommendation

Based upon the performance information gathered through three years of service, it appears that removal and replacement of the binder and surface course with either a carbon black modified or high stability mix would prevent the reoccurance of the rutting and poor riding quality present on the remainder of the original 1979 construction project.

Follow-Up

Inspections and testing will continue on the test sections.

Distribution: A,B,C,D,E,F,G

Reviewed By:
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Materials & Research
Engineer

Prepared By:
Ronald Frascoia
January 29, 1990

RESEARCH UPDATE

NUMBER U90-4

NEED FOR PAVEMENT SCARIFICATION

References - Work Plan 86-R-3, U87-6

Purpose - To determine if it is necessary to scarify an existing bituminous pavement prior to overlaying it with additional subbase material and a new pavement system to insure there is no slippage or other distortion of the overlay materials.

History - Two 1,000 foot test sections were left unscarified on Vermont Route 100 (Project Stowe-Morristown F029-1(9)(S) in 1986. The test sections are located in Morristown at MM 0.426-0.615 and MM 1.561-1.75.

Reconstruction in the non-scarified test sections included placement of 6" of subbase of gravel, 3" of plant mixed base course, 1 1/4" of Type III binder and 3/4" of Type IV surface course.

The scarification treatment on 1.73 miles of the project resulted in an average size of 1 square foot or less on 50% of the bituminous pavement with the remainder ranging up to 1 by 3 lineal feet in dimension.

Cost Information - The pavement scarification was bid at $1.50 per square yard for a total of 23,750 square yards.

Initial Performance - The following information summarizes the condition of the pavement at scarified and non-scarified test locations through three years of service.

Pavement Cracking (in lineal feet per 100 feet)

	Preconstruction (5/86)	3 Year Evaluation (6/89)
Non-scarified	774	85
Scarified	776	94

Pavement Rutting (in 16ths of an inch)

Non-scarified	7.6	2.6
Scarified	7.5	3.2

MAYS Roughness (in inches per mile)
The trailer mounted MAYS meter was not available for testing prior to the reconstruction of the project.

	2 Year Evaluation (6/88)	3 Year Evaluation (7/89)
Non-scarified	68	80
Scarified	62	84

Miscellaneous
There was no visible sign of any shoving, rippling or other pavement distortion in the 1,000 foot non-scarified sections.

Preliminary Conclusion - Field measurements recorded to date suggest there has been no reduction in performance on the roadway sections where the underlying pavement was not scarified.

The elimination of the scarification requirement should be considered on future projects which are reconstructed to a similar standard. The resulting cost savings could be used to increase the pavement design thickness.

Follow-Up - Annual inspections will continue for the life of the project and reports will be prepared on a biennial schedule.

Dist: A,B,C,D,E,F,G,H

Reviewed By:

M.W.Lawson P.E.
Materials & Research
Engineer

Prepared By: PCW

Peter C. Winters
April 17, 1990

RESEARCH UPDATE

NUMBER U90-5

3M 350 TAPE ON OPEN GRADED PAVEMENT - I 91 LYNDON-BARTON

(FINAL)

REFERENCE: Work Plan 88-R-3, Update U88-10, Update U90-1

HISTORY:

During repaving of a section of I 91 (Project Lyndon-Barton IR 091-3(10)), an experimental durable pavement marking, 3M 350 Tape, was specified for 8" white gore markings and 4" white lanelines on the accel/decel lanes at interchanges 23, 24, 25, and at a rest area and two scenic turnouts on this resurfacing project.

APPLICATION:

The experimental product was installed during a six week period ending on August 17, 1989.

As a control, durable markings of an approved thermoplastic material were installed at interchange 25 NB off ramp, and the two scenic turnouts.

A total of 2430 LF of 8" and 2545 LF of 4" white tape markings was installed.

A total of 2499 LF of 8" and 1152 LF of 4" white thermoplastic markings was installed.

COST:

The price for 4" white line Tape was $1.50 per LF.
The price for 4" white thermoplastic line was $0.50 per LF.
The price for 8" white line Tape was $3.00 per LF.
The price for 8" white thermoplastic line was $1.00 per LF.

STATUS:

The project was inspected on January 3, 1990. Substantial loss of product had occurred as shown below.

8" White tape lines - 48% loss.
4" Laneline dashes - 52% loss.

STATUS:continued

A final inspection was conducted on April 16, 1990. The loss of 3M 350 tape had increased to the following:

8" White tape lines - 64% overall (40% to 100%)
4" Laneline dashes - 99% loss.

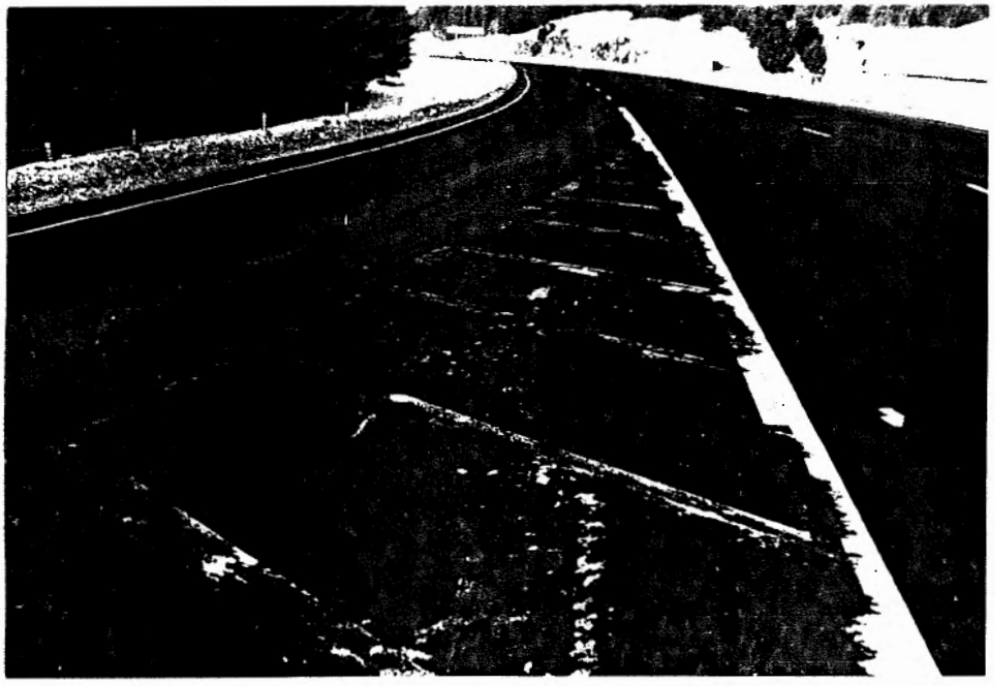
The thermoplastic gore markings showed no loss and only minor scuffing at some locations.

The painted edgelines and centerline dashes on this project were all severely worn and must be rated poor to "faint".

CONCLUSIONS:

The percentage of 3M 350 pavement marking tape lost after only seven months is unacceptable. Further use of 3M 350 Tape on open graded friction courses is not recommended.

FOLLOW UP: No further evaluations are expected.



Typical Gore Area as of January 1990

Distribution A,B,C,D,E

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M.W.Lawson P.E.
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Engineer

Prepared By:
Peter C. Winters
April 17, 1990

RESEARCH UPDATE

NUMBER U90-6

3M 350 TAPE ON OPEN GRADED PAVEMENT - I 89 COLCHESTER

(FINAL)

REFERENCE: Work Plan 88-R-4, Update U88-10, Update U90-2

HISTORY:

During repaving of a section of I 89 (Project Colchester IR 089-3(14)), an experimental durable pavement marking, 3M 350 Tape, was specified for the 10' x 4" white dashed centerlines. Control was to be edgelines which were painted with standard traffic paint.

The application of the experimental product occurred during July of 1989. Fifteen thousand nine hundred LF of 4" white line was applied between milemarkers 92/00 and 98/00 on both northbound and southbound lanes.

Performance through January 4, 1990 was reported in Research Update U90-2. By January 4 there had been a 33% loss of product on the northbound lane and a 12% loss on the southbound lane. Later that month a Vt. Agency of Transportation pavement marking crew repainted the lost dashes.

STATUS:

On April 9, 1990 a final inspection was performed. The loss of product had increased to 64% on northbound and to 35% on southbound.

COST:

The price for this product was $1.50 per LF. installed for a total cost of $23,850.00.

CONCLUSIONS:

The percentage of 3M 350 pavement marking tape lost after only seven months is unacceptable. Further use of 3M 350 Tape on open graded friction courses is not recommended.

FOLLOW UP: No further evaluation is expected.

Distribution A,B,C,D,E

Reviewed By:

M.W.Lawson P.E.
Materials & Research
Engineer

Prepared By:

Peter C. Winters
July 23, 1990

RESEARCH UPDATE

NUMBER U90-7

PAVETECH (KOCH) BRIDGE JOINT SYSTEM I 89 BRIDGE 16 S PUTNEY

REFERENCE: Work Plan 90-R-2; Research Report 90-2

HISTORY:

Bridge 16 S, I89 over Sackett's Brook in Putney was rehabilitated during 1988-1989. The immediate failure of the installed joints led to the need for replacement. A PAVETECH Joint had been installed on a Bridge on Route US 2 in Waterbury in January of 1990 and appeared to have been successful. This system was selected as an experimental replacement on Bridge 16 S. The design, construction and early performance of the Joint in Waterbury was reported in research report 90-2 in January 1990, and is referenced for details of this system. Recently KOCH inc. has acquired the PAVETECH system and it has now been renamed "KOCH Bridge Joint System (BJS) TM.

INSTALLATION:

The system which consists of a modified mastic asphalt binder incorporating pre-weighed select granite aggregate and rubberized asphalt, was installed on March 14, 1990. Two 30 ft long joints were installed at the abutment joints. Air temperature was 66+ Deg. F. The width of the south joint was 25"+/- and the width of the north joint varied from 30" to 32". The only installation problem encountered was at the northerly joint in the shoulder area where the pavement depth was less than designed. This caused the system to taper from 3 to 2 lifts at the curb line. The system representative indicated that this would not affect the system's waterproofing ability.

COST:

The installation of this joint was performed at a per foot cost of $172.15 for 60 feet. The total cost of the contract was thus $10,329.00. Additionally,removal of the previous joint and traffic control by state forces was $1,519.88.

STATUS:

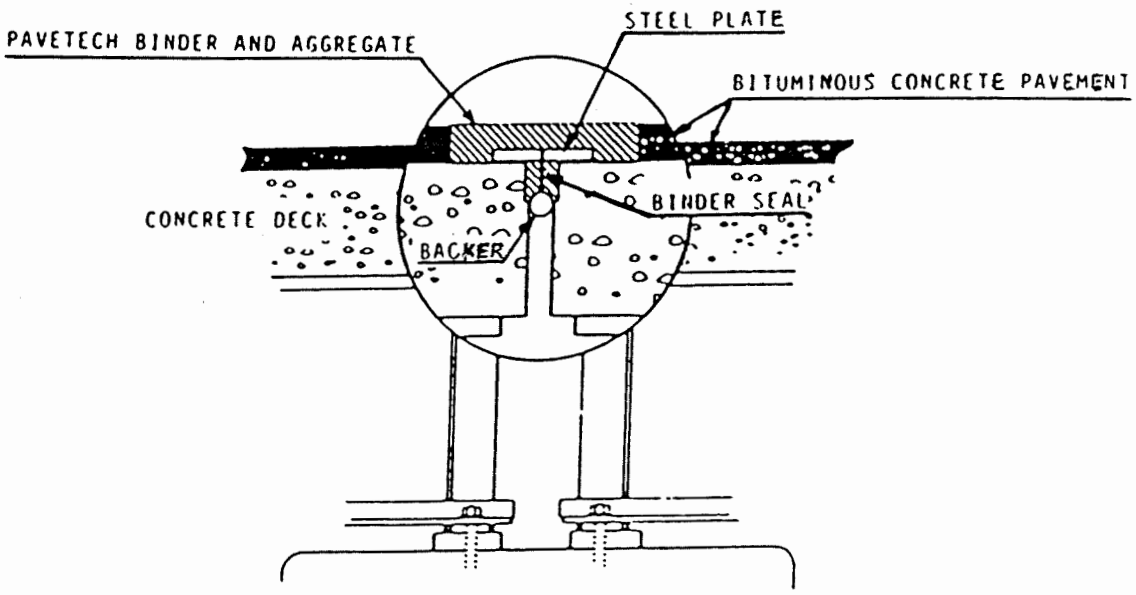
As of July 11, 1990 the joints are still flexible enough that an indentation can be made by a bootheel. Under traffic however, the indentation immediately disappears. The joints ride well. A slight, built in, "hump" does not appear to have increased with span expansion with hot weather.

STATUS Continued

Due to the location of the joints over the abutments, leakage will not be apparent unless there is visible surface damage such as delamination at the pavement/plug interface or significant cracking.

FOLLOW UP:

The joints will remain under observation and performance will be reported as significant results occur.



TYPICAL PAVETECH CROSS SECTION

Distribution A,B,C,D,E

Reviewed by:

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Prepared by: C. C. Benda, P.E.
Structural Concrete Engineer
May 10, 1990

RESEARCH UPDATE

Number U90-07

REPAIR OF SUBSTRUCTURE CONCRETE USING SHOTCRETE

Reference

Work Plan 87-C&R-24

Background

Inspired by reports that concrete forming and placement costs for bridge substructure rehabilitation could be reduced with pneumatically applied concrete, research was initiated to evaluate the performance of shotcrete.

Subsequently, it was found that Vermont was not alone in its desire to learn more about the durability and integrity of shotcrete used for bridge repairs. Following a series of discussions involving the Agency, private industry and the division office of the Federal Highway Administration, a wet mix shotcreting demonstration was set up for the Northeastern States Materials Engineers Association meeting on October 21, 1987.

The site chosen for the application was Bridge No. 61S over Vt. Rte. 2A on Interstate 89 at the Williston Interchange. Gilbertson Shotcrete Specialties, a division of Master Builders, Inc. agreed to furnish the labor, equipment and materials for the demonstration at no cost to the State.

A 4' by 12' section of the bottom of the northerly pier cap on Bridge 61S was prepared by District No. 5 maintenance forces. Preparation included the removal of unsound and delaminated concrete with a 15# chipping hammer in an area extending 4' from the easterly nose of the pier cap to the mid-point between the two columns, see Figure 1. As much of the bottom of the cap was severely delaminated, large segments up to 2 square feet in area, fell to the ground with little or no chipping effort. A saw cut was made at the mid-point to prevent concrete removal beyond the scope of the project. Concrete was chipped out 9" to 15" up each vertical face of the cap for the same 12' length as the bottom. The depth of removal was a minimum of 1" beyond the primary reinforcing steel into sound concrete. Ninety five percent of the exposed reinforcing steel was visibly corroded and the horizontal portion of the stirrups were rusted through, see Figure 2. At the completion of the removal operation the entire area was sand blasted to remove any remaining contaminants and rust from the reinforcing steel.

Materials

At the recommendation of Raymond Towne of Gilbertson Shotcrete Specialties, their prepackaged cement based mortar, GS-Shotpatch 10, was used for the repair. Gilbertson's product literature indicates that "GS-Shotpatch 10 is a ready-to-use, multi component, cement based mortar specially designed for durable repairs or overlays using either the wet or dry mix shotcrete application system." Thirty cubic feet of this material was delivered to the project site in 55 pound bags. One bag yields approximately 0.43 $ft.^3$.

Poor weather conditions created placement difficulties which resulted in a materials shortage. To complete the application, two cubic yards of ready-mixed mortar were delivered by the S. T. Griswold Co., Inc. of Williston, Vermont, the following day. The mix contained 970 lb. per cubic yard of Type I, Glens Falls cement and approximately 2700 lb. per cubic yard (SSD) of Griswold's concrete sand. An air entraining admixture and enough water to yield a 0.38 water/cement ratio was added at the batch plant.

In conjunction with the GS-Shotpatch 10 mortar and the ready-mixed mortar, Gilbertson's HPS Shotcrete Accelerator was incorporated into the mix at the nozzle. HPS Shotcrete Accelerating Admixture reportedly improves surface bond, decreases rebound and improves early strengths. It was used at a dosage rate of 3% by weight of cement.

Application

The shotcrete application was performed by United Gunite of Irvington, New Jersey. They used a Kaiser brand mixer and screw pump to deliver the concrete through a hose to the nozzle. At the nozzle, compressed air was injected into the material flow along with the set accelerating admixture.

Although the positive displacement pump was capable of discharging 7 $yd.^3/hr.$ the rate of application was between 1/4 $yd.^3/hr.$ and 1 $yd.^3/hr.$. The GS-Shotpatch 10 was placed in the mixing hopper, see Figure 3, along with approximately 0.7 gallons of water per 55 lb. bag. Each batch was thoroughly mixed before being dumped into the pump. When the ready-mixed mortar was applied, the mix was discharged from the truck directly into the mixing hopper and then into the pump.

The GS-Shotpatch 10 mortar was pneumatically applied to the pier cap on the afternoon of October 21, 1987. As temperatures were in the $40^{\circ}F$ to $45^{\circ}F$ range, the nozzleman had some difficulty with bond when material depths became too great, see Figure 4. When the supply of GS-Shotpatch 10 was depleted, the shotcreting operation was discontinued until the following day. The repair was completed with the ready-mixed mortar which was delivered in two - one cubic yard loads.

Following the completion of the troweling operation, the entire surface was covered with a liquid membrane curing compound, see Figure 5, and insulating blankets to protect the shotcrete from excessive evaporation and freezing temperatures. The overnight low temperatures on the 22nd and 23rd of October were 28°F and 26°F respectively.

Test Results

All specimens were cast in a vertical position using Gilbertson Shotcrete Specialty's GS-Shotpatch 10. Fabrication was completed with the same equipment and procedures used on the substructure repair, see Figure 6.

Given in Table 1 are results of testing for compressive strength, resistance to chloride ion penetration and freeze-thaw durability.

Ten 4" diameter cores were extracted from two 18" by 18" unreinforced test panels measuring 6" in thickness. The cores were tested for compressive strength (AASHTO T24-86) at 7, 14, 28, 56 and 90 days following standard moist curing.

Compressive strength of the cores were 10 to 25 percent higher than typical values of 6" x 12" cylinders representing Class AA, Portland Cement Concrete currently used for structural repairs.

After 14 days of moist curing and 28 days air drying, the two specimens to be used for determining resistance to chloride ion penetration were tested for base level chloride ion content. Upon completion of 100 days of continuous ponding with a 3% NaCl solution, the specimens were resampled for total chloride ion content at depths of 0.25" to 1" and 1" to 2", in accordance with AASHTO T260-84.

The two specimens used to determine freeze-thaw durability were cycled from 40°F to 0°F and back to 40°F in a 3% NaCl solution 300 times following an initial 14 day moist curing period. The 3" x 3" x 16" samples were tested for weight loss and fundamental transverse frequency at 50 cycle intervals (AASHTO T161-86).

Results of both chloride ion penetration testing and freeze thaw evaluations compared favorably with previous studies conducted with Class AA concrete*. Freeze-thaw durability and permeability of the GS-Shotpatch 10 mortar was essentially equivalent to the Class AA concrete.

*Research Update Number 89-1

Field Performance

A field inspection of the project conducted on April 23, 1990 revealed a series of hairline cracks spaced at 10 to 15 inches along the vertical faces of the pier cap. Cracking extended down one face of the patched area across the bottom of the cap and up the other face. The cracks were interconnected in a random pattern on the bottom. The exact cause and depth of the cracks is unknown. In general, the finer the aggregate in the mixture, the more likely shrinkage cracking will occur. As the outer portion of the repair contained no material coarser than the 3/8" sieve size, this is one possible explanation for the cracking.

Soundings taken on the entire repair area indicated one delaminated section measuring 20" by 23" located 40" from the east face of the westerly column. The remainder of the shotcrete appeared very solid and quite well bonded despite the extremely poor condition of the adjacent concrete and the undesirable weather during application.

Recommendations

The use of pneumatically applied concrete should be permitted on a full scale rehabilitation project where the application and performance can be closely monitored. The shotcrete contractor should be prequalified and use of an accelerating admixture considered mandatory. Although no application was made with silica fume, this admixture should be given serious consideration when shotcrete is specified.

Until the Agency has gained experience with the dry mix process, the recommendations herein apply only to the wet mix process.

TABLE 1

GS-SHOTPATCH 10 (SHOTCRETE) MORTAR

Compressive Strength, Freeze-Thaw Durability
and Chloride Ion Concentration Test Results

Sample ID	GS10-A	GS10-B	Class AA
Compressive Strength, psi			
7 days	5670	5170	4310
14 days	6310	5830	5150
28 days	6510	6460	5960
56 days	6050	6970	----
90 days	7410	7010	----
Resistance to Freezing and Thawing			
Weight Loss, %			
@ 300 cycles	5.2	5.2	3.8
Durability Factor			
@ 300 cycles	113.5	114.1	103.4
Chloride Ion Penetration, PPM(lb./cy) of Concrete			
Base Level	132(0.5)	136(0.5)	47(0.2)
100 day Ponding			
1/4" to 1" depth	692(2.8)	556(2.2)	918(3.7)
1" to 2" depth	172(0.7)	148(0.6)	110(0.4)

FIGURE 1

Preparation of Existing Concrete Substructure

FIGURE 2

Prepared Surface

FIGURE 3

Mixing/Pumping Operation

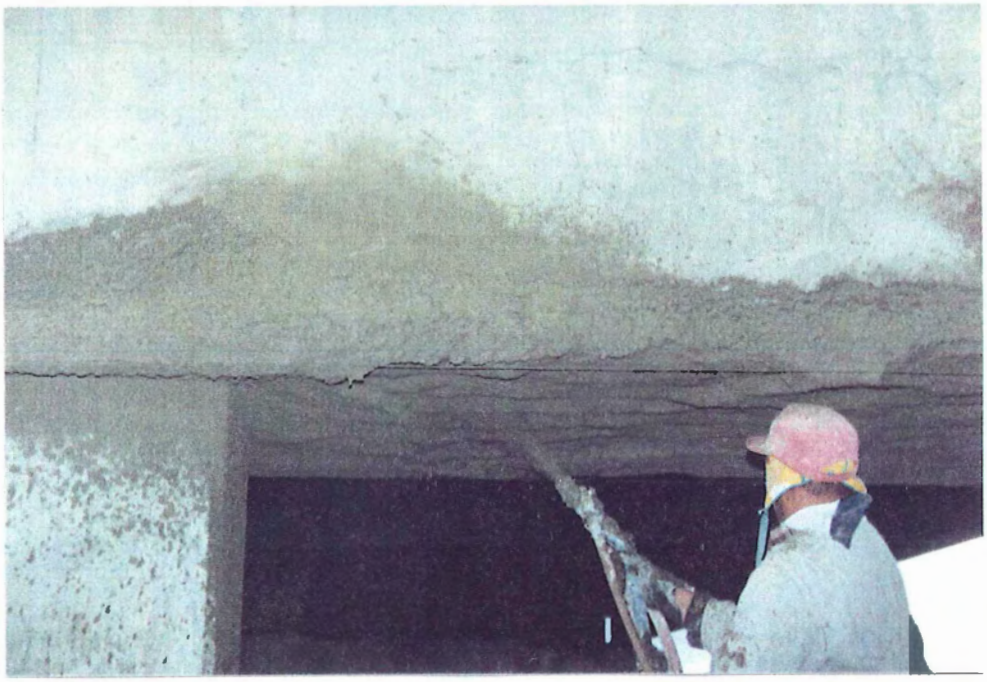

FIGURE 4

Shotcrete Application

FIGURE 5

Completed Repair

FIGURE 6

Test Specimen Fabrication

Reviewed By:
8-13-90
M.W. Lawson, P.E.
Materials & Research Engineer

Prepared By:
Lertchai Chaumrattanakul
Lertchai Chaumrattanakul
August 6, 1990

RESEARCH UPDATE

Number U90-8

THORMA-JOINT BRIDGE JOINT SYSTEM
(Initial Report)

REFERENCE: Work Plan 90-R-03

HISTORY:

Due to failure of many bituminous joints on bridges in the state of Vermont, investigations of possible alternate joint systems are being conducted. The Thorma-Joint system, applied on bridge #16 north on I-91 in Putney, Vermont by Linear Dynamics Inc. (LDI), is one of the alternates.

DESCRIPTION:

The two joints were located at the abutments of bridge #16 north. The bridge has a reinforced concrete deck overlaid with bituminous concrete pavement. The joints are 35' long at a ninety degree angle to the center of the deck.

The joints were overlaid with asphalt pavement in the same manner as the rest of the deck prior to the Thorma-Joint application. In preparation for joint installation saw cuts 20' apart and 4" deep were made equal distance from the center of the joint. Since waterproofing membrane had already been installed on the deck, it was burned off after the saw cutting process.

COST:

The two Thorma-Joints were installed at a cost of $13,500.00 (or $192.50 per linear foot for the basic joint system) plus an additional $4,171.00 for a design modification.

INSTALLATION:

The joint installation, completed in one day by a 4-person team from LDI, was performed on July 11, 1990 with clear skies and air temperature of 75 to 85 degrees F.

The saw cutting on the joints started at 8:00 a.m. and was completed by 10:00 a.m. Two 2-parallel saw cuts were made 10" from the center of the joints at a depth of 4" (except on part of the southerly joint where the pavement was thicker and required a cut to a depth of 5").

Pavement removal began at 10:15 a.m. Concrete debris and residues from jackhammering were removed manually, and the open trough was air blasted clean. During removal of the sawed pavement at the southerly joint, the blast cleaning process delaminated the pavement course and broke the bond between base course of asphalt and the waterproofing membrane. The debonding occurred in the vertical face of the deck pavement.

The membrane on the bottom of the southerly trough was burned with a 1200 degree F lance to remove the membrane and asphalt residue. A 2" gap in the center of the joints was revealed.

One half inch aggregate was heated in a truck mounted drum mixer, to a temperature of 500 degrees F. The specified temperature is 375 degrees F, but 500 degrees F was used first in order to heat up the second drum mixer before the hot aggregate and rubberized bitumen binder were combined. The hot aggregate was then transferred to the second drum mixer, located at a lower position behind the truck.

The rubberized bitumen binder, preheated to 350 degrees F, was applied and uniformly poured into the joint over a blue board backer. Steel plates, 6" wide and 4'- 3" long with predrilled holes 7" center to center were put over the movement gap. Nails were pushed through the holes in the steel plates in order to keep the plates centered. More binder was then poured and squeegeed on the top of the plates until the plates, and all the surfaces of the trough, were completely covered to a depth of 3/8".

The 1/2" heated aggregate and the hot binder were mixed and poured into the joint in two 1" lifts. A layer of pure binder was applied between lifts. After the top course was installed, the joint was cooled down with water prior to thorough compaction, by a vibrating plate compactor. No roller was used.

A final layer of binder was applied on the top of the joint by 2:30 p.m. The surface of the joint was flat, but not quite smooth. The same procedure was followed on the northerly joint later that day.

STATUS

Due to the location of the joints over the abutments any leakage which may occur will not be visually apparent. The joints will be monitored visually for signs of surface damage such as delamination at the pavement/plug interface or significant cracking. The bridge has been open to traffic for a few days with no apparent distress.

FOLLOW UP

The joints will remain under observation and reports will be produced as needed whenever significant data is obtained.

Distribution A,B,C,D,E,F,G

Reviewed By: ML/GBA
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Prepared By: PCW
Peter C. Winters
August 8, 1990

RESEARCH UPDATE

NUMBER U90-9

PAVETECH (KOCH) BRIDGE JOINT SYSTEM FAILURE-WATERBURY

REFERENCE: Work Plan 89-R-12; Research Report 90-2

HISTORY:

On December 15, 1989 a PAVETECH Bridge Joint System was installed in Bridge #44, US-2 over the Little River in Waterbury Vt.

The weather was severe with air temperatures of -8 degrees F at 7:30 A.M. The installation of this joint was described in detail in Research Report 90-2, Jan 1990.

Since the installation of this joint, KOCH Mfg. Co. of Stroud Oklahoma has acquired the rights for the system and it is now called the KOCH Bridge Joint System.

By January 1990 some deterioration of the binder/grit wearing course had been noted, probably due to snow plow damage.

On January 18, 1990, following heavy rainfall, water was noted dripping from a plastic trough, which had been previously installed below the joint. It was theorized at that time that the water might be coming from the unsealed curb.

On February 9, 1990 the joints in the vertical granite curb were sealed with bridge membrane and water was poured on the joint. AT 10:15 A.M. 95 drops per minutes were observed falling from the end of the trough and by 10:30 a steady stream of water was pouring from the end of the trough.

On May 8, 1990 another test was made. The bridge was dry and water was applied with a garden sprayer. The test began at 9:40 A.M. By 10:15 water was dripping from the trough at approx. 1 drop per second. By 10:40 A.M. a steady stream was running from the trough. Testing was completed at 1 P.M. It was concluded that the joint was leaking.

On May 15, 1990, three 4" diameter cores were taken from the joint. Evaluation of one taken at the joint/asphalt interface revealed a loss of bond between the pavement and the joint material. The other cores revealed that some aggregate was poorly coated with binder and there were many visible voids. One of the other two cores was taken in a crack at the edge of a distressed area on the surface of the joint. The crack was full depth.

STATUS:

On August 2, 1990 the entire joint was removed and replaced by the original proponent Davis & Swanson of Tilton New Hampshire along with the KOCH Technical representative Mr. Richard J. Baker.

STATUS Continued

During removal of the original joint it was noted that much of the aggregate was not coated (although it had appeared to have been coated when installed). There was water between the joint and the concrete slab in many areas. Some of the concrete surface was powdered and showed lack of bond to the deck although the dust and small particles were bound into the binder. The binder material was well adhered to the steel plates. One complete full depth cross section was removed intact. Even this complete section revealed uncoated stones, voids, and delamination between courses.

The jackhammering of the old joint caused some damage to the surface of the concrete.

Reinstallation of the joint proceeded in the same manner as the original installation.

PROBABLE CAUSE:

Even though it had previously been believed that the system could be installed in any weather, evaluation of the failure by the Manufacturer's (KOCH) personnel and Materials & Research division led to the conclusion that the probable cause of the failure was the severely cold (-8 to +20/- Degrees F) and wet condition of the bridge during installation.

The bridge was frozen (December was a record cold month) and the attempt to dry the concrete with the 1200 Degrees F lance was unsuccessful although the surface appeared dry. Frost within the concrete was thawed but the moisture present in the air and deck immediately condensed and may have frozen causing a frost plane at the interface of the binder and the asphalt and concrete surfaces.

The lack of aggregate coating may have also contributed to the failure and occurred because the stone absorbed moisture while being heated. The binder may then have flowed around but not been absorbed into the surface of the stone. The process of flooding with binder to fill the voids was also partially unsuccessful, probably due to rapid cooling of the binder in the sub freezing air temperature.

The manufacture representative, the contractor and R&D are now in agreement that this system should not be installed where air and deck temperature are below 40 Degrees F and rising.

FOLLOW UP:

The evaluation will continue with observation and evaluation of the performance of the newly reinstalled joint.

Distribution A,B,C,D,E,F,G

Reviewed By:

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Prepared By:

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July 23, 1990

RESEARCH UPDATE

NUMBER U90-10

PAVETECH (KOCH) BRIDGE JOINT SYSTEM I 89 BRIDGE 16 S PUTNEY

REFERENCE: Work Plan 90-R-2; Research Report 90-2

HISTORY:

Bridge 16 S, I89 over Sackett's Brook in Putney was rehabilitated during 1988-1989. The immediate failure of the installed joints led to the need for replacement. A PAVETECH Joint had been installed on a Bridge on Route US 2 in Waterbury in January of 1990 and appeared to have been successful. This system was selected as an experimental replacement on Bridge 16 S. The design, construction and early performance of the Joint in Waterbury was reported in research report 90-2 in January 1990, and is referenced for details of this system. Recently KOCH inc. has acquired the PAVETECH system and it has now been renamed "KOCH Bridge Joint System (BJS) TM.

INSTALLATION:

The system which consists of a modified mastic asphalt binder incorporating pre-weighed select granite aggregate and rubberized asphalt, was installed on March 14, 1990. Two 30 ft long joints were installed at the abutment joints. Air temperature was 66+ Deg. F. The width of the south joint was 25"+/- and the width of the north joint varied from 30" to 32". The only installation problem encountered was at the northerly joint in the shoulder area where the pavement depth was less than designed. This caused the system to taper from 3 to 2 lifts at the curb line. The system representative indicated that this would not affect the system's waterproofing ability.

COST:

The installation of this joint was performed at a per foot cost of $172.15 for 60 feet. The total cost of the contract was thus $10,329.00. Additionally,removal of the previous joint and traffic control by state forces was $1,519.88.

STATUS:

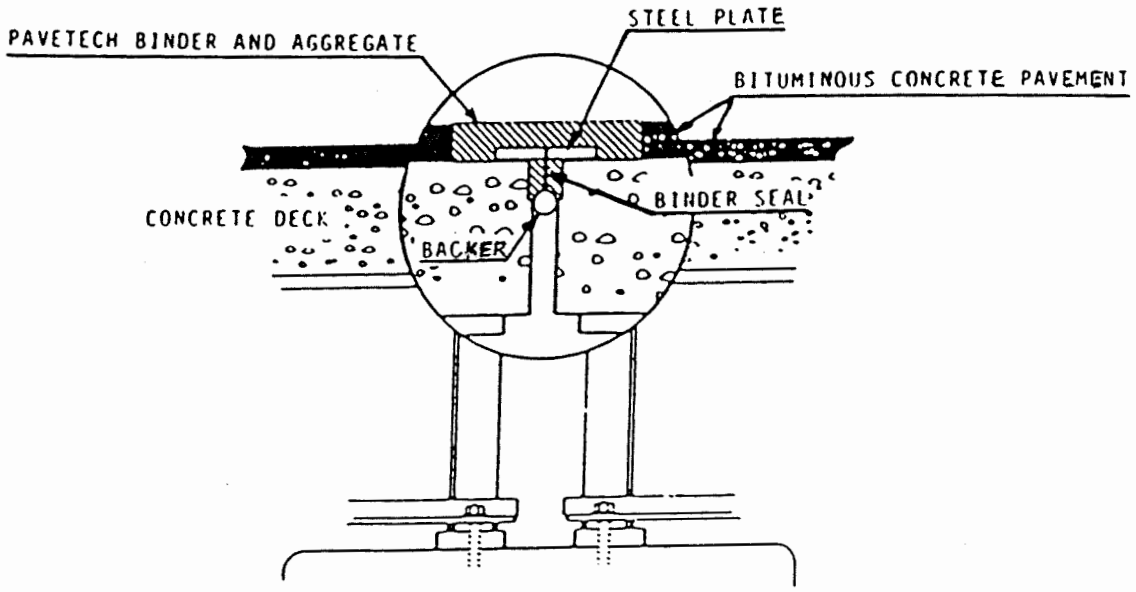
As of July 11, 1990 the joints are still flexible enough that an indentation can be made by a bootheel. Under traffic however, the indentation immediately disappears. The joints ride well. A slight, built in, "hump" does not appear to have increased with span expansion with hot weather.

STATUS Continued

Due to the location of the joints over the abutments, leakage will not be apparent unless there is visible surface damage such as delamination at the pavement/plug interface or significant cracking.

FOLLOW UP:

The joints will remain under observation and performance will be reported as significant results occur.



TYPICAL PAVETECH CROSS SECTION

Distribution A,B,C,D,E