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SMA
(Stone Matrix Asphalt)
Colfax Avenue Viaduct

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Prepared in cooperation with the
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The Colfax viaduct project was Colorado's second SMA project but their first attempt to use it on a bridge deck. The first project was located on SH 119, west of Longmont and successfully demonstrated design, production and placement of the European SMA. The Colfax viaduct project was unique from the project placed on SH 119 in that it was constructed on a bridge deck requiring a different paving paving technique. This project successfully demonstrated the placement of the European SMA on a bridge deck. This report documents the construction of the Colfax viaduct project. Documentation of construction on the SH 119 project can be found in CDOT report No. CDOT-DTD-R-95-1.

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1.0 Introduction

In 1994, the Colorado Department of Transportation placed its first Stone Matrix Asphalt (SMA) pavement. The project was located in Boulder County on State Highway 119. The project extends approximately 5 miles between State Highway 52 on the southwest end to Hover Road in Longmont on the northeast end. This project contains five different mix designs:

1) standard dense graded hot bituminous pavement (HBP) (Grading C),
2) Stone Matrix Asphalt (SMA) with Vestoplast S,
3) SMA with polymer modified asphalt, PM-ID, (AASHTO Task Force 31, Type ID polymer, Reference 1),
4) SMA with cellulose fiber pellets, and
5) Grading C with AC-20R (AASHTO Task Force 31, Type II-B polymer).

Approximately 11,000 tonnes of SMA was placed on this project. The placement of this project was a success. Documentation of the construction is reported in report no. CDOT-DTD-R-95-1 titled "Demonstration of the Placement of Stone Matrix Asphalt in Colorado". (2)

For this project field performance data will be evaluated on an annual basis and final results will be available in approximately 3 years.

Since SMA pavements are durable and can be placed in thin lifts other applications for SMA construction were reviewed. Asphalt overlays on bridge decks are common in Colorado and the replacement of the existing asphalt overlay is complicated by the fact that the maximum thickness of the asphalt wearing surface is limited to 4 inches. Based on the success of the SH 119 construction project it was decided to place an SMA pavement on a bridge deck in Denver.

2.0 Project

Typically, Colorado bridges are designed to have a maximum of 4 inches of hot bituminous pavement (HBP) on the surface (CDOT design dead load requirement). When the riding surface on the bridge deck requires rehabilitation, the HBP must be removed before an additional surface treatment is placed so that the maximum of 4 inches of HBP is not exceeded. The existing surface on this project C 0404-030, Colfax Viaduct consisted of a 2" HBP with membrane.

Since SMA pavements are very durable and can be placed in thin lifts it could be advantageous to use them on bridge decks. Using SMAs on bridge decks allows for less milling and the existing deck membrane does not have to be replaced.

Project No. C 0404-030 was a good candidate to try the SMA pavement. This project is located in downtown Denver on the Colfax viaduct between Federal Blvd. and Osage Street (Figure 1). This project is approximately 1 mile long and has an Average Daily Traffic (ADT) of 46,100 with 9% trucks. The existing pavement surface on this project had began to ravel and needed to be removed. The cost to either remove the existing HBP and membrane by planing and overlaying, or to raise the expansion joints would have been extremely high and were not considered to be an option. The decision to mill and replace with a SMA was considered the most cost effective solution.

This project consisted of milling 1-1/4 inches of HBP and replacing it with 1-1/4 inches of SMA pavement. Because of the thin lift being placed on this project (CDOT design guidelines require 2 inch minimum for dense graded mixes) no control section using a dense graded mix was established on this project. This project contained approximately 2700 tonnes of SMA mix.

2.1 Evaluation Section

A 1000 foot evaluation section was established in the eastbound driving lane. A location map of the evaluation section is shown in Figure 2. The bridge is three lanes in each direction. For traffic control concerns only the two outside lanes in the eastbound direction will be evaluated under this study.

2.1.1 *Existing Distress*

A preconstruction evaluation was performed on the project which consisted of measuring ruts and cracks.

Rut depths were measured every 15 meters (50 feet) throughout the test section in both the right and left wheel paths of the two outside lanes. The ruts were measured with a two-meter (six-foot) straight edge and were measured to the nearest 2.5 mm (0.1 in). Rutting in the evaluation section was fairly low. The average in all the wheel paths was around 8 mm (0.3 in). The highest rutting measurement was 18 mm (0.7 in). According to CDOT's standard an average measurement of 8 mm (0.3 in) is considered low.

Crack maps were prepared for the evaluation section. Cracking was very extensive so only the transverse cracks were recorded. On the average there were about 14 transverse cracks per 100 foot of pavement. It was observed that on the average there was only 1 transverse crack per 100 foot that ran the entire width of the pavement. This type of cracking does not follow the typical thermal cracking pattern that is found in roadways. Thermal cracking in roadways tend to extend the full width of the pavement.

The cracks had began to deteriorate on the edges and there were a number of areas on the mat where the membrane was exposed. These areas ranged in size from 25cm^2(4in^2)to 0.1m^2(1ft^2) square. Prior to paving the larger areas were repaired. The membrane was removed and a cold pour material was applied. Typical distress found in the existing overlay on the deck surface is shown in Figures 3 and 4. The removal and repair of the exposed membrane is shown in Figures 5 and 6.

2.2 Bids

Excluding the patching, a SMA mix was used on the entire project. Table 1 shows the tonnage used and the cost per ton of the SMA mix.

Table 1. Bid Cost of the SMA.

	TONNAGE	COST PER TON
SMA PM-1D	2749	$60.00

PM-1D - Polymer Modified, Type 1-D

This type of project did not lend itself to the economy due to the size and complexity of the construction involving a bridge structure with expansion joints. However the bids were consistent with the SMA PM-ID used on SH 119 (2).

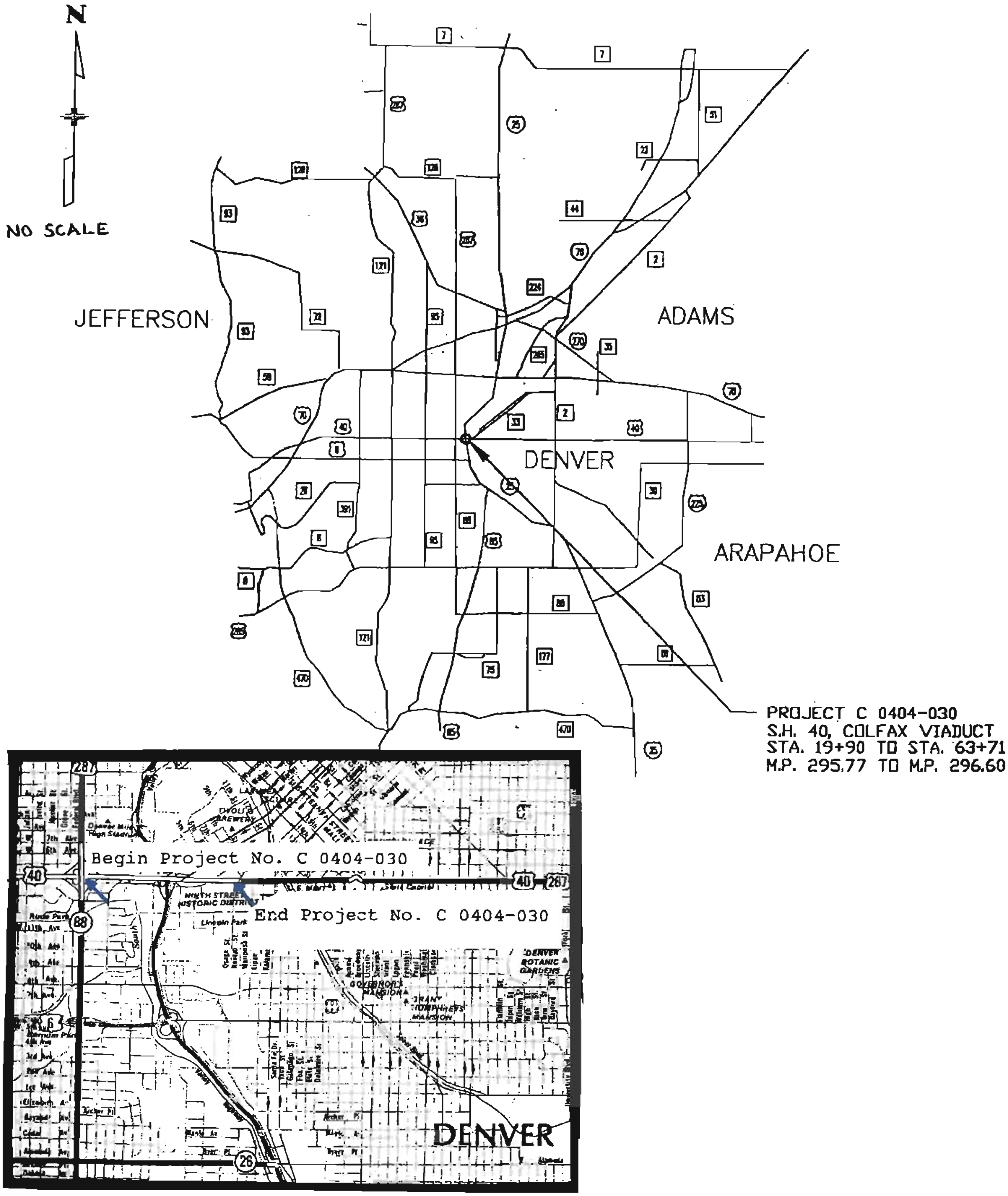


Figure 1. Location Map of Project No. C040-030.

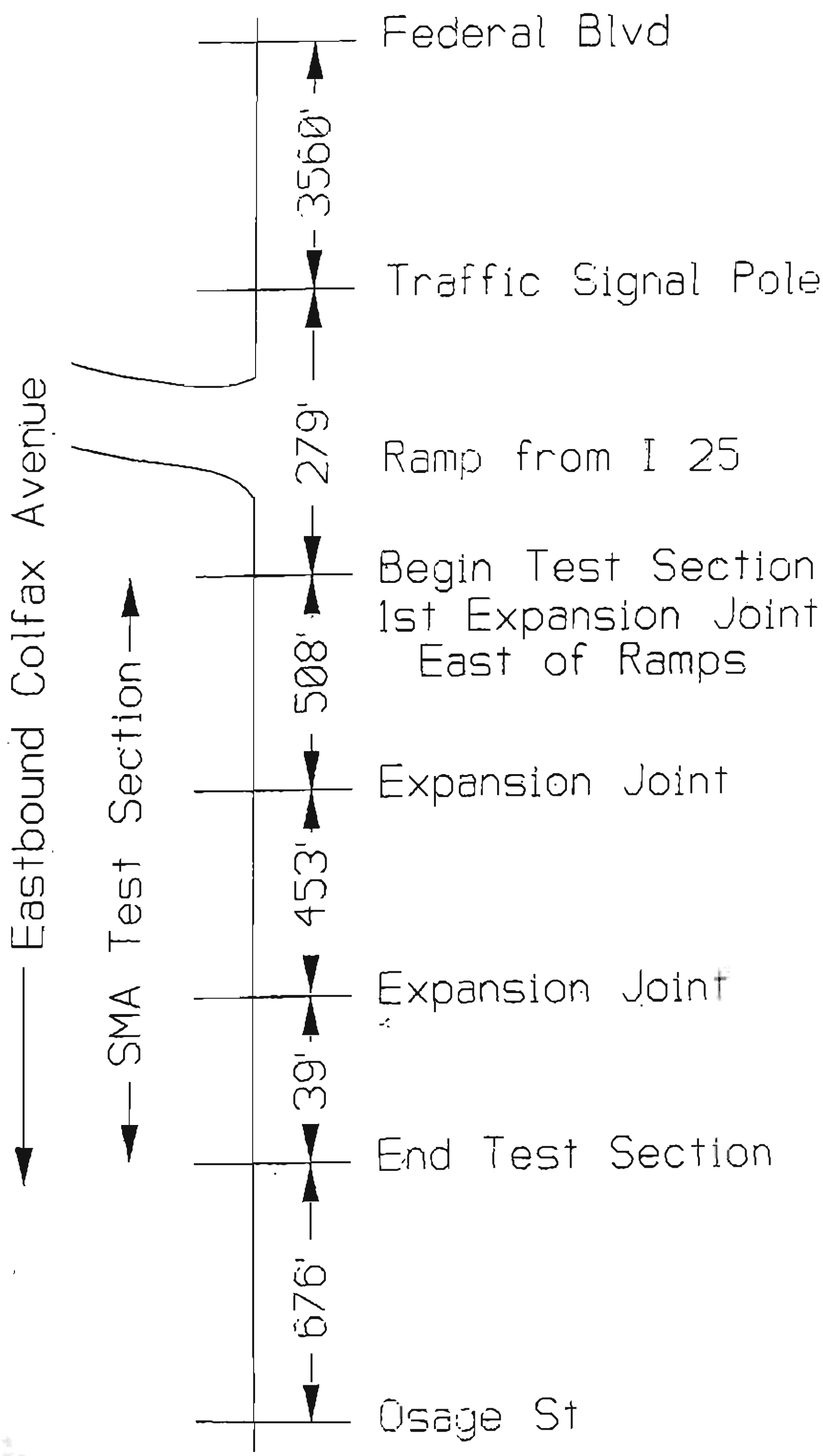


Figure 2. Location Map of the Evaluation Section.

Figure 3. Typical cracking pattern prior to paving.

Figure 4. Extent of distress prior to paving.

Figure 5. The exposed membrane was removed.

Figure 6. A cold-pour material was used to cover the area where the membrane was removed.

3.0 SMA Mix Designs

The specifications used for the project can be found in Appendix A.

3.1 Aggregate Tests

All the aggregates were granite and came from the Meridian Pit in Meridian, Wyoming. The stockpiles used for SMA included a 19-mm (3/4-in) rock, a 12.5-mm (1/2-in) rock, and a granite sand.

3.1.1 Gradation

The percentage of each stockpile is shown in Table 2.

Table 2. SMA Trial Blending Percentages.

Stockpile	Percent of Blend
19.0-mm Rock	27%
12.5-mm Rock	49%
Granite Sand	18%
Limestone Dust	5%
Hydrated Lime	1%

Table 3 shows the SMA composite gradation.

Table 3. SMA Composite Gradation.

Sieve Size	Percent Passing	CDOT Specification	FHWA Recommendations
19.0 mm (3/4")	100	100	100
12.5 mm (1/2")	91	90 - 100	85 - 95
9.5 mm (3/8")	70	75 maximum	75 maximum
4.75 mm (No. 4)	24	20 - 30	20 - 28
2.36 mm (No. 8)	19	16 - 24	16 - 24
600 μm (No. 30)	13		
300 μm (No. 50)	11		
75 μm (No. 200)	7.7	7 - 11	8 - 10

The target values of the SMA design were within CDOT's Master Range. The tolerances for the various sieve sizes were: 9.5 mm ($\pm$5), 4.75 mm and 2.36 mm ($\pm$4) and 75 μm ($\pm$2).

3.1.2 Physical Properties

The tests results on the fine and coarse aggregates are shown in Table 4.

Table 4. Aggregate Test Results.

Test	Procedure	Result	Specification
AASHTO T 96	LA Abrasion	23%	30% max
AASHTO T 104	Sodium Sulfate Soundness	2%	12% max
CP-45	Fractured Faces One or more Two or more	 100% 98%	 100% 90% min
AASHTO T 89	Liquid Limit	NP	NP
AASHTO TP 33	Fine Aggregate Angularity	45.7	45 min

All the test results were acceptable. Not all the tests were specified on the project; however, all the tests in Table 4, in addition to ASTM D 4791 (Flat and elongated, 3 to 1 and 5 to 1) are recommended by FHWA (3).

3.2 Additive

A polymer modified asphalt cement was used on this project to prevent the asphalt cement from draining down during hauling and placement.

The polymer modified asphalt cement was supplied directly to the contractor's asphalt plant from an independent polymer modifying company (Koch Materials Co.). The polymer met the AASHTO Task Force 31 Type 1-D specification (1). The polymer supplier was the same as on the SH 119 SMA project. The purpose of the polymer in an SMA mix is to stiffen the asphalt cement and prevent draindown.

3.3 Asphalt Cement Tests

The PM-1D was manufactured by Koch Materials Co. using Conoco asphalt and SB copolymers. The asphalt cement was tested to AASHTO MP1 Superpave binder specification. The material conformed to Superpave PG 76-28.

3.4 Mixture Tests

During construction on SH 119 the test results using AASHTO T 283 to evaluate moisture resistance on the SMA only passed marginally. However further evaluation using the Hamburg wheel-tracking device to test the SMA for moisture resistance indicated that the SMA was resistant to moisture. It was concluded that possibly the AASHTO T 283 tests may not accurately represent the moisture susceptibility of the SMA (2). For this reason only the Hamburg wheel-tracking device was used to test the SMA for moisture resistance on the Colfax viaduct project. The Hamburg wheel-tracking device results can be found in section 7.1.

Draindown tests were not performed on the mix. However, it should be noted that during construction no draindown problems were observed.

3.4.1 *Marshall Results*

The tests for the mix design were performed by the contractor. The Marshall mix design used 50 compaction blows on each side of the specimen. The Marshall test results are shown in Table 5.

Table 5. Marshall Test Results.

Property	Specification	PM-1D
VTM (%)	3-4	3.3
Asphalt Content (%)		6.43
VMA (%)	16(min)	15.3
Stability, N (lb)	6200 min (1400)	10400 (2340)

PM-1D - Polymer Modified, Type 1-D
VTM - Voids in the Total Mix (Air Voids)
VMA - Voids in the Mineral Aggregate

3.4.2 *Specification Comments*

Based on the results from the SH 119 project, the VMA specification was increased from a minimum of 15% to 16%. FHWA currently recommends 17% (3), however when the SMA project on SH 119 was constructed FHWA's recommendation was a minimum of 15%. For the Colfax viaduct project the VMA specification was set at 16. Although FHWA recommends a VMA value of 17, CDOT's current specification for VMA is 16 because it is felt that a level of 17 can not be consistently achieved.

3.5 Mineral Filler Tests

The mineral filler used for this project was a crushed grey limestone (CAL 200) dust. The limestone dust properties measured were particle size (AASHTO T 88) and plasticity index (AASHTO T 90). Test results are shown in Table 6.

Table 6. Test Results on the Mineral Filler.

Test	Result	Recommendation
Particle size smaller than 20 μm	44%	< 20%
PI	Non-plastic	< 4%

The particle size was measured by the contractor using the hydrometer analysis (AASHTO T 88). The mineral filler was finer than recommended by FHWA (3). However, it was similar in gradation to the mineral filler used on SH 119. The test results are shown in Table 7.

Table 7. Hydrometer Analysis (AASHTO T 88) Results on the Mineral Filler.

Size (μm)	Percent Passing
75	83
20	44
2	5

4.0 Construction

4.1 Plant Description

A Gencor continuous mixer with a capacity of 500 tonnes per hour was used on this project. The fuel source was natural gas. The SMA mix required four cold feed bins, with a retrofit for the addition of mineral filler (limestone). The silo used for the mineral filler had a 60 tonne capacity. Lime was added with a weigh pod and vane feeder and mixed with damp aggregate in an approved pugmill. A baghouse was used for emission control. The storage silo for the HBP had a 200 tonne capacity. The average time the HBP was in the silo was 15 minutes.

4.2 Plant Modifications for SMA

Unlike the other modifiers that are used in SMA designs the polymer modifier used on this project did not require any modification to the plant to properly add the additive. However, a cement silo was set up with a metering device to add the mineral filler. The specifications required the mineral filler be added at the same point as the asphalt cement. Both the mineral filler line and the asphalt cement line entered the rear of the mixing drum and were discharged into a mixing head. This allowed the asphalt cement to coat and capture the mineral filler, which helped to prevent blowing the mineral filler out of the drum and into the baghouse.

The rate of production of the plant was virtually cut in half from a capacity of 500 tonnes per hour to 250 tonnes per hour. However, according to plant personnel the SMA did not reduce production at the plant. The rate of production was reduced to match the placement rate.

4.3 Haul Trucks

The HBP was delivered to the project with end-dumps. The haul time from the plant to the project was approximately 35 minutes. The haul trucks were not covered with a tarp. The temperature of the mix behind the paver was 138°C (280°F) to 146°C (295°F).

4.4 Laydown Operation

One Blaw Knox 510 paver was used. The majority of the paving was done in a 12.5 foot width. Three rollers were used to compact the SMA. The final rolling pattern established used three steel-wheeled rollers. A 7 ton Hyster 350D was used for breakdown and was kept right behind the lay down operation. This roller made two coverage. Two 10 ton Hyster C766Bs were operated in tandem right behind the breakdown roller. Each of these rollers made three coverages a piece. Rolling was stopped when the surface temperature reached 88°C (190°F). All the rollers were operated in the static mode.

The specification for density of an SMA is 94% of rice, densities obtained using this rolling pattern and using the thin lift nuclear gage to measure densities only produced densities of 92% at the highest.

4.5 Trial Placement

The project plans require the contractor to place a test section prior to construction to evaluate the contractor's ability to both produce and place the SMA. Two days prior to the start of the project the contractor placed a short section of the SMA mix in the driving lane. On this test strip, cores were used to calibrate the thin lift nuclear gage. Since this project was entirely on a bridge deck the number of cores taken were limited. During the placement of this test strip no problems were encountered. However, during placement the mix appeared to be rich and the materials engineer lowered the asphalt cement content from 6.7% to 6.5%. Reduction in the asphalt content was the only adjustment made.

This trial placement also gave the contractor an opportunity to develop the best technique to work with the SMA mix at the expansion joints.

4.6 Construction Techniques

The construction schedule was designed such that traffic disruption was minimal. The existing pavement was milled on one weekend and the following week-end the SMA pavement was placed. This schedule was altered because of weather conditions but as a whole the work schedule caused little disruption to traffic.

Tapers were placed at all expansion joints once the pavement was milled. These tapers were removed prior to placing the SMA pavement. Although hand work is difficult with SMAs the contractor did not have much difficulty working with the SMA material at the expansion joints.

Figures 7 and 8 show the preparation at the expansion joints. Figure 9 shows the required hand work at the expansion joint.

Figure 7. Removing the taper at the expansion joint.

Figure 8. Preparing the joint prior to SMA placement.

Figure 9. Hand work is required at the expansion joint.

5.0 Post-Construction

5.1 Post-Construction Observations

A visual inspection of the mat following paving showed the surface of the mat to be uniform throughout the project. The only quantitative test performed on the finished mat was smoothness.

5.1.1 Smoothness

The plans for this project contained Colorado's 1995 smoothness specification. The smoothness specification requires that the contractor takes the measurement using a computerized California type profilograph. Smoothness measurements were taken on the existing paving, on the milled pavement and on the finished mat. Typically smoothness is measured down the center of each lane and are taken following each day's paving, however due to the small quantity of SMA material on this project each lane was measured and recorded separately. The 1995 CDOT specification uses a 2.5 mm (0.1 inch) blanking band. In urban areas smoothness is measured on percent improvement. Smoothness results are shown in Table 8.

Table 8. Smoothness Results.

	Existing Pavement in/mile	Milled Surface in/mile	Finished Mat in/mile	Percent Improvement
Eastbound Right Lane	40.19	32.41	37.60	6
Westbound Right Lane	38.53	31.80	33.97	12
Eastbound Middle Lane	52.86	42.55	34.28	35
Westbound Middle Lane	49.90	36.37	24.79	50
Eastbound Left Lane	78.09	44.83	47.68	39
Westbound Left Lane	66.54	43.69	46.45	30

5.2 Future Post-Construction Evaluations

This evaluation section will be evaluated for a three year period. Evaluations will be made twice a year. In the spring cracks will be noted. In the fall rutting measurements will be taken. During each evaluation visual inspection of the pavement will be made.

Upon completion of each evaluation, short field notes will be written documenting the performance to date.

At the end of the three year evaluation, data obtained from this project will be incorporated into the final report on the SH 119 SMA. This report will document, evaluate and make recommendation as to the future use of SMA mixes in Colorado.

6.0 Field Verification Test Results

6.1 Asphalt Content, Field Compaction and Gradation

The design AC content was 6.5%. The density requirement was 94% to 96% of the Rice (AASHTO T 209 value)

6.1.1 Test Results

The field verification test results are summarized in Tables 9, 10, and 11.

Table 9. Asphalt Content and Field Compaction Test Results.

Additive	Asphalt Content (%)				% of Maximum Density			
	Avg	S.D.	n	Q.L.	Avg	S.D.	n	Q.L.
PM-1D (6.5%)	6.39	.12	6	98.9	92.6	.61	6	*

Avg - Average
S.D. - Standard Deviation
n - Number
Q.L. - Quality Level
PM-1D - Polymer Modified, Type 1-D

* Although measured densities were lower than the specification no price reduction was applied as per project special provision.

On the SH 119 project only cores were used to determine density. It was determined early in construction that the nuclear gage density and the cores did not have any correlation.

Since this project was located on a bridge deck cores were limited to the compaction test section. A thin lift nuclear gage was used to determine densities and provided a good correlation with the cores.

Table 10. Quality Level of Gradation Test Results.

Additive	Gradation						
	Quality Level						n
	19.0 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	75 μm	
PM-1D (6.5% AC)	100	77.8	97.4	6.5	100	100	4

PM-1D - Polymer Modified, Type 1-D

Table 11. Gradation Test Results.

Additive	Approved Project Gradation					
	Sieve Size (mm)					
	19.0	12.5 (90-100)	9.5 (65-75)	4.75 (20-28)	2.36 (15-23)	.075 (5.7-9.7)
PM-1D (6.5% AC)	100	90	68	28	19	9.4
	100	92	70	30*	19	8.8
	100	93	70	29*	19	9.0
	100	90	75	30*	20	9.5

* outside of specification range
PM-1D - Polymer Modified, Type 1-D

6.2 Volumetrics

Three replicate samples were compacted by the contractor for field quality control. The volumetric test results are shown in Table 12. The volumetric properties were acceptable.

Table 12. Volumetric Test Results of Field Produced SMA.

Air Voids (%)			VMA (%)			Marshall Stability			Marshall Flow		
Avg	S.D.	n	Avg	S.D.	n	Avg	S.D.	n	Avg	S.D.	n
3.4	.38	7	16.7	.39	7	2146	156	7	16.5	.67	7

VMA - Voids in Mineral Aggregate
Avg - Average
S.D. - Standard Deviation
n - number

7.0 European Torture Test Results

Laboratory tests were performed to identify rutting, moisture damage, and thermal cracking. Fatigue cracking was not investigated in the laboratory as part of this study.

All tests were performed on material that was produced at the plant and sampled from behind the augers in the test section. Replicate samples were tested and the averages were reported.

7.1 Hamburg Wheel-Tracking Device

7.1.1 Description of Test Equipment

The Hamburg wheel-tracking device is used to evaluate the resistance of the HBP to moisture damage. It is manufactured by Helmut-Wind Inc. in Hamburg, Germany as shown in Figures 10 and 11.

A pair of samples are tested simultaneously. A sample is typically 260 mm (10.2 in.) wide, 320 mm (12.6 in.) long, and 40 mm (1.6 in.) deep. a sample's mass is approximately 7.5 kg (16.5 lbs.), and it is compacted to 6% $\pm$ 1% air voids. For this study, samples were compacted with the linear kneading compactor. The samples are submerged under water at 50°C (122°F), although the temperature can vary from 25°C to 70°C (77°F to 158°F). A steel wheel, 47 mm (1.85 in.) wide, loads the samples with 705 N (158 lbs.). The wheel makes 50 passes per minute over each sample. Each sample is loaded for 20,000 passes or until 20 mm of deformation occurs. Approximately 6-1/2 hours are required for a test.

The results from the Hamburg wheel-tracking device include the creep slope, stripping slope and stripping inflection point as shown in Figure 12. These results have been defined by Hines (4). The creep slope relates to rutting from plastic flow. It is the inverse of the rate of deformation in the linear region of the deformation curve, after post compaction effects have ended and before the onset of stripping. The stripping slope is the inverse of the rate of deformation in the linear region of the deformation curve, after stripping begins and until the end of the test. It is the number of passes required to create a 1 mm impression from stripping. The stripping slope is related to the severity of moisture damage. The stripping inflection point is the number of passes at the intersection of the creep slope and the stripping slope. It is related to the resistance of the HBP to moisture.

7.1.2 Test Results and Discussion

The test results from the Hamburg wheel-tracking device are shown in Table 13. The mm of deformation after 20,000 passes are shown. The results are shown graphically in Appendix B.

Table 13. Test Results (mm of Deformation After 20,000 Passes) from the Hamburg Wheel-Tracking Device.

Temperature (°C)	SMA (PM-1D)	Specification
50	3.05 mm	10.0 mm (max)

PM-1D - Polymer Modified, Type 1-D

The Hamburg test results from the SMA mix placed on this project were acceptable. The result from this project were consistent with the SMA containing the PM-1D on the SH 119 project (2).

Figure 10. The Hamburg Wheel-Tracking Device.

Figure 11. Close-Up of the Hamburg Wheel-Tracking Device.

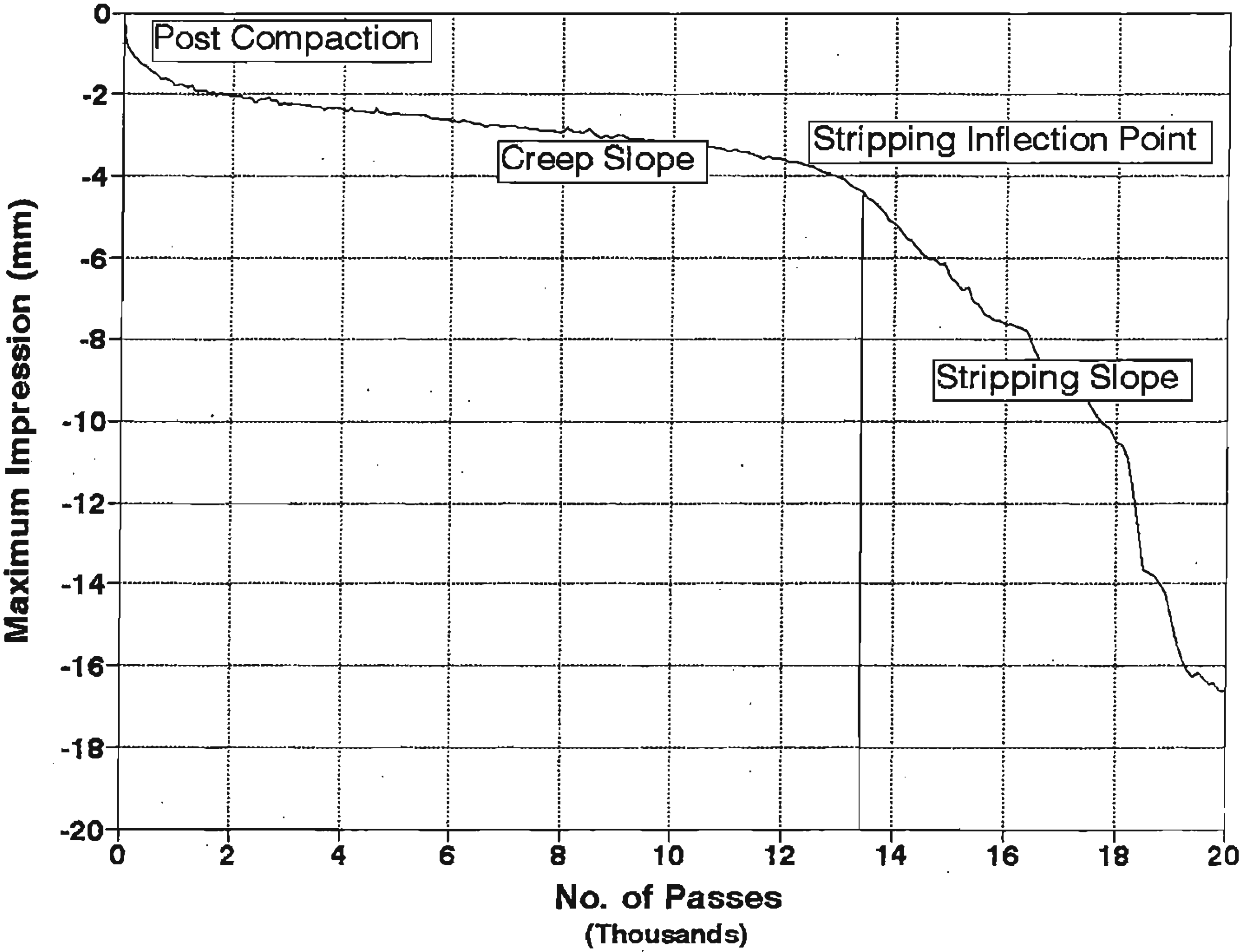


Figure 12. Typical Results from the Hamburg Wheel-Tracking Device.

7.2 French Rutting Test

7.2.1 Description of Test Equipment

The French rutting tester is used to evaluate the resistance of the HBP to permanent deformation. It is manufactured by the Laboratoire Central des Ponts et Chaussees (LCPC) and is shown in Figure 13: a close-up is shown in Figure 14. The samples tested are 500 X 180 mm (19.7 X 7.1 in.) and can be 50 or 100 mm (2 or 4 in.) thick. Two samples can be tested simultaneously.

The samples are tested by having a tire roll back and forth over the sample at elevated temperatures. The samples are loaded with 5000 N (1124 lbs.) by a pneumatic tire inflated to 0.6 MPa (87 psi). The tires load each sample at 1 cycle per second; one cycle is two passes. The chamber is heated to 60°C (140°F) but can be set to any temperature between 35 and 60°C (95 and 140°F).

When a test is performed on a laboratory compacted sample, it is aged at room temperature for as long as 7 days. It is then placed in the French rutting tester and loaded with 1000 cycles at room temperature. The deformations recorded after the initial loading are the "zero" readings. The sample is then heated to the test temperature for 12 hours before the test begins. Rutting depths are measured after 100, 300, 1000, 3000, 10,000, 30,000 and possibly 100,000 cycles. The rutting depth is reported as a percentage of the sample thickness. After a given number of cycles, the percentage is calculated as the average of 15 measurements, (five locations along the length and three along the width) divided by the original slab thickness. A pair of slabs can be tested in about 9 hours.

A successful test will typically have a rutting depth that is less than or equal to 10% of the slab thickness after 30,000 cycles. The results are plotted on a log-log graph paper. The slope and intercept (at 1000 cycles) are calculated using linear regression. The equation is:

$$Y = A\left(\frac{X}{1000}\right)^{B} \qquad (Equation\ 1)$$

where:

Y = rutting depth (%),
X = cycles,
A = intercept of the rutting depth at 1000 cycles and
B = slope of the curve.

7.2.2 Test Results and Discussion

The test results for the French rutting tester are shown in Table 14. The percent rut depth after 30,000 cycles is shown. The results are shown graphically in Appendix C.

Table 14. Test Results (% Rut Depth After 30,000 Cycles) from the French Rutting Tester.

Temperature °C	SMA PM-1D	Specification (Maximum)
60	3.21	10%

PM-1D - Polymer Modified, Type 1D

The test results indicate that this SMA mix will be rut resistant. These results were consistent with the SH 119 project.

Figure 13. French Rutting Tester.

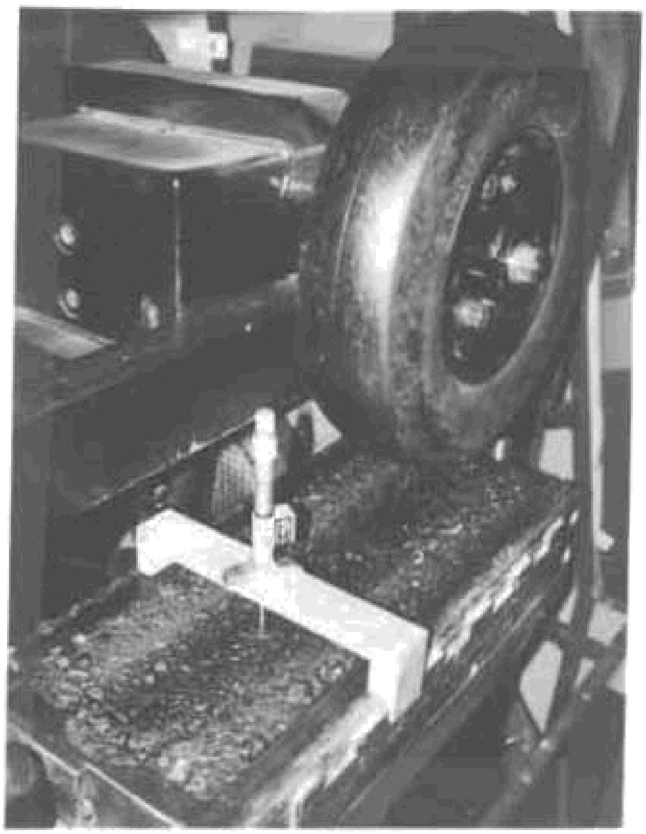


Figure 14. Close-Up of the French Rutting Tester.

7.3 Thermal-Stress, Restrained-Specimen Test

7.3.1 Description of Test Equipment

The thermal stress restrained specimen test (TSRST) is used to evaluate the resistance of the HBP to low temperature thermal cracking. The TSRST was developed at Oregon State University as part of SHRP. The TSRST is manufactured by OEM, Inc. in Corvallis, Oregon. The device is shown in Figures 15 and 16. A schematic of the sample is shown in Figure 17. The device is fully automated.

Vinson (5) evaluated numerous tests used to identify the low-temperature thermal cracking characteristics of HBP. Based on the evaluation, the TSRST as modified by Arand (6) was determined to be the best. This test has been evaluated by Jung (7,8).

The loose HBP was short-term aged for 4 hours at 135°C (270°F) and then compacted. Samples were compacted in the linear kneading compactor for this study. The compacted HBP was then long-term aged for 120 hours (5 days) at 85°C (185°F) in a forced draft oven. Samples tested were 50-mm (2-in.) diameter and 250-mm (10-in.) long.

After a sample is mounted in the TSRST, it is cooled at a rate of 10°C (18°F) per hour. Liquid Nitrogen is used to provide the cooling. The sample is not allowed to contract during the cooling period. The sample length is monitored with LVDTs and the use of invar steel rods. Since the sample is not allowed to contract as it cools, stresses develop within it. A closed-loop system keeps the sample at a constant length. When the developed stress exceeds the strength of the sample, the sample breaks. The temperature and stress at fracture are recorded. A typical plot of the test results is shown in Figure 18.

The repeatability of the test was studied by Jung (8). The coefficient of variation was 10% for the fracture temperature and 20% for the fracture strength. This was considered to be excellent and reasonable, respectively. One standard deviation, 68% of replicate samples will have a fracture temperature within $\pm$ 2 or 3°C ($\pm$ 4 or 5°F). Likewise, $\pm$ 400 to 600 kPa ($\pm$ 60 to 90 psi) would be representative of fracture stresses of 68% of identical samples.

7.3.2 Test Results and Discussion

The fracture temperature and fracture strength of each of the mixtures tested are shown in Table 15. The tests results are similar to the PM-1D test results for the SMA containing the PM-1D used on the SH 119 project (2).

Table 15. TSRST Test Results

	Fracture	
	Temperature (°C)	Strength (kPa)
SMA PM-1D	-35.7	4335
SMA PM-1D	-42.4	3492

PM-1D - Polymer Modified, Type 1-D

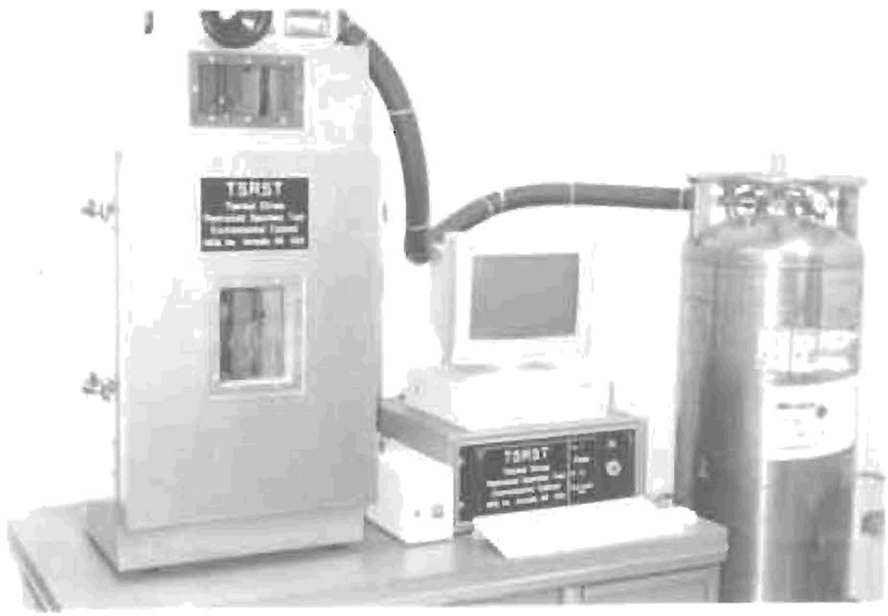


Figure 15. The TSRST Device.

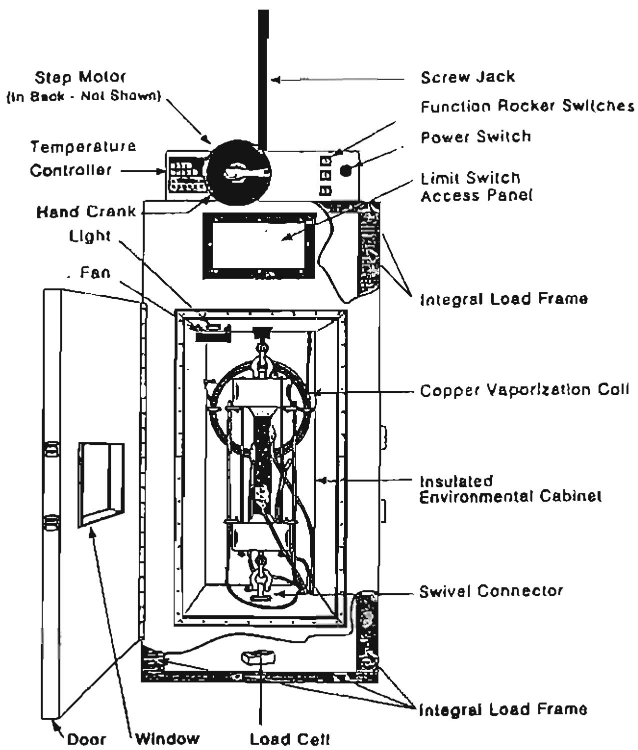


Figure 16. Schematic of the TSRST Device.

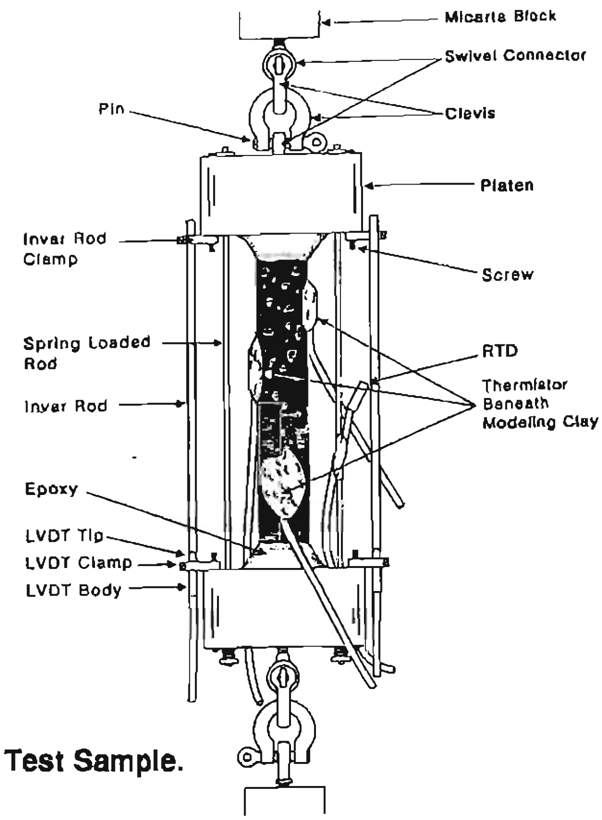


Figure 17. Schematic of the Test Sample.

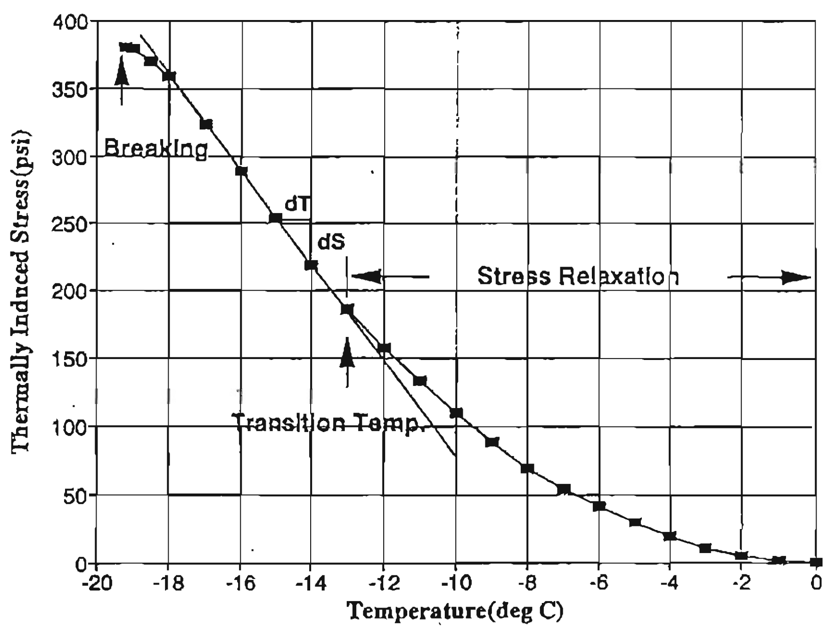


Figure 18. Typical TSRST Test Results.

8.0 Summary and Recommendations

The Colfax viaduct project was Colorado's second SMA project but their first attempt to use it on a bridge deck. The first project was located on SH 119, west of Longmont and successfully demonstrated design, production and placement of the European SMA. The Colfax viaduct project was unique from the project placed on SH 119 in that it was constructed on a bridge deck requiring a different paving technique. This project successfully demonstrated the placement of the European SMA on a bridge deck.

8.1 Design

On the Colfax viaduct project the VMA level was set at 16 which was an increase from the VMA level set on SH 119 of 15. The VMA level of 15 was FHWA's recommendation when the SH 119 project was constructed. FHWA currently recommends a VMA level of 17 (3). CDOT's current specification for VMA is 16 because it is felt that a level of 17 can not be consistently achieved.

The Master Range was widened to the FHWA recommendation (3). This allowed the contractor more flexibility in blending the aggregates to achieve the required VMA.

8.2 Construction

As on the SH 119 project, the specified 94% relative compaction was difficult to achieve on the Colfax viaduct project. Although densities were determined differently on the two projects, CDOT's current testing and construction procedures need further evaluation to determine the reason for the measured low densities.

The high asphalt content, lack of fines and the polymerized asphalt used in SMA mixes limits the ability to do extensive hand work. However, because of the hand work that is required at the expansion joints, additional care and caution must be taken when using a SMA mix on a bridge deck.

An extra awareness of the truck scheduling is necessary when paving with a SMA on a bridge deck to maintain a smooth paving operation and to avoid a back-up of paving trucks. Good scheduling that eliminates back-ups will in turn avoid drain down in the truck and cooling of the material.

8.3 Performance

The results from the European testing equipment (Hamburg wheel-tracking device and the French rutting tester) indicate that the SMA pavement will be rut resistant. These results were consistent with the SH 119 project (2).

The TSRST results indicate that the SMA pavement will resist low temperature cracking.

This project will be incorporated into the evaluation of the SH 119 project. Evaluations will be made twice a year. Final results will be available in approximately three years.

9.0 Future Research

The Colorado Department of Transportation has one SMA project planned for the 1996 construction season. The project is located on I-70 in Region 3. This project will contain approximately 30,000 tonnes of SMA mix. This project will specify polymers as the additive.

A control section will not be incorporated into this project, however the project will be evaluated under the SMA research study. Construction will be monitored, material testing will be performed, and paving and rolling techniques will be evaluated to determine any problems related to not achieving the specified density.

The evaluation will also include any variation in cores samples and nuclear gage density readings.

10.0 References

1. Shuler, T.S., Chairman (1991), AASHTO-AGC-ARTBA Joint Committee, Subcommittee on New Materials, Task Force 31, "Proposed Specifications for Polymer Modified Asphalt," 18 pages.

2. Harmelink, D., T. Aschenbrener, K. Wood (1995), "Demonstration of the Placement of Stone Matrix Asphalt in Colorado," Colorado Department of Transportation, CDOT-DTD-R-95-1, 90 pages.

3. Guidelines for Materials, Production, and Placement of Stone Matrix Asphalt (SMA) (August 1994), National Asphalt Pavement Association, Information Series 118, 18 pages.

4. Hines, Mickey (1991), "The Hamburg Wheel-Tracking Device," Proceedings of the Twenty-Eighth Paving and Transportation Conference, Civil Engineering Department, The University of New Mexico, Albuquerque, New Mexico.

5. Vinson, T.S., V. C. Janoo, and R.C.G. Haas (1989), "Low Temperature and Thermal Fatigue Cracking," SHRP Summary Report SR-OSU-A-003A-89-1.

6. Arand, W. (1987), "Influence of Bitumen Hardness on the Fatigue Behavior of Asphalt Pavements of Different Thickness Due to Bearing Capacity of Subbase, Traffic Loading, and Temperature," Proceedings of the 6th International Conference on Structural Behavior of Asphalt Pavements, University of Michigan, Ann Arbor, pp. 65-71.

7. Jung, D. and T.S. Vinson (1993), "Thermal Stress Restrained Specimen Test To Evaluate Low-Temperature Cracking of Asphalt-Aggregate Mixtures," Transportation Research Record 1417, Transportation Research Board, Washington, D.C., pp. 12-20.

8. Jung, D. and T.S. Vinson (1993), "Low Temperature Cracking Resistance of Asphalt Concrete Mixtures," Journal of the Association of Asphalt Paving Technologists, Volume 62, pp. 54-92.

Appendix A:
SMA Specification

REVISION OF SECTIONS 401, 403, AND 703
STONE MASTIC ASPHALT PAVEMENT

Sections 401, 403, and 703 of the Standard Specifications and Standard Special Provisions are hereby revised for this project as follows:

Subsection 401.02 shall include the following:

Recycled Asphalt Pavement (RAP) shall not be used in SMA mix.

Table 401-1 shall include the following:

**Stone Mastic Asphalt Pavement - Item 403

Passing 3/8" sieve	± 5%
Passing No. 4 and No. 8 sieves	± 4%
Passing No. 200 sieve	± 2%

In Subsection 401.02, second paragraph, delete items (1) and (2) and replace with the following:

(1) A proposed job-mix gradation for each mixture required by the contract, except stone mastic asphalt (SMA) mix, which shall be wholly within the master range table, Table 703-3 or 703-6, before the tolerances shown in Table 401-1 are applied. Also, a proposed job-mix gradation for SMA mix required by the contract which shall be wholly within the master range table, 703-3, before the tolerances shown in Table 401-1 for stone mastic asphalt pavement - Item 403 are applied.

(2) The aggregate source, percentage of each element used in producing the final mix, and the gradation of each element.

When approved, laboratory test results submitted by the contractor may be used to modify the mixing and compaction temperatures.

Subsection 401.06 shall include the following:

Asphalt Cement shall be (Polymer Modified) (Type I-D).

Subsection 401.07 shall include the following:

Placement of SMA shall be permitted only when minimum air and surface temperatures are 50 F. or above.

Subsection 401.09 shall include the following:

The time between plant mixing and placement of SMA shall not exceed one hour.

REVISION OF SECTIONS 401, 403, AND 703
STONE MASTIC ASPHALT PAVEMENT

Subsection 401.17 shall include the following:

Compaction of SMA shall be accomplished using a minimum of two steel wheel rollers weighing 10 to 12 tons. Additional steel wheel rollers may be required by the Engineer. The initial breakdown roller shall follow the laydown operation as closely as feasible. All rollers must operate within 500 feet of the paver. The Engineer must approve, and may request changes in this distance. In-place density shall comply with Subsection 401.17 except the minimum acceptable level shall be 94 percent of voidless density. Price adjustments shall not apply.

Rollers shall not be used in a vibratory mode unless they are first used successfully in the demonstration control strip. Pneumatic wheel rollers shall not be used on SMA mix. Roller speed shall be between 1 and 3 mph.

Compaction shall be completed before the mix cools down to 275 F.

The method of measuring relative compaction for all SMA mixtures will be in accordance with CP-44 Method B (cores). The contractor shall provide all labor and equipment for the coring operation, and filling the core holes.

In-place density shall be expressed as a percentage of the maximum specific gravity determined for each lot of material.

Subsection 403.01 shall include the following:

This work includes placing a Stone Mastic Asphalt (SMA) pavement as shown on the plans. Before proceeding with the actual work, the contractor shall demonstrate that he can produce and place a satisfactory mix. The actual work may proceed when a full lane width demonstration control strip, having a minimum length of 400 feet, has been successfully placed. The control strip will be used by the Engineer to determine the compactive effort required for density. No other SMA production and placement will be allowed until densities are determined. The Engineer will designate the location of the control strip.

Subsection 403.02 shall include the following:

Mixture design and field control testing shall be performed using the Marshall Method (AASHTO T-245-90).

A minimum of two weeks prior to the proposed use of any stone mastic asphalt pavement on the project, the contractor shall submit to the engineer a mix design meeting the appropriate specification requirements, including the following:

Stability, Marshall Compactor (50 blow)	1400 lbs. minimum
% Voids in total mix	3-4%
VMA (% voids in aggregate)	16
Flow,0.25 mm(0.01 inch)	8-18
Lottman, CPL 5109, Min.	70
Dry Tensile Strength, PSI, Min. CPL 5109	30

A minimum of one percent hydrated lime by weight of the combined aggregate shall be added to the aggregate for all SMA pavement.

The SMA design must be approved by the Engineer before any pavement is placed on the project. In addition the Contractor will provide field control testing during production of the SMA mix. The following tests will be required for the design mix and their results provided to the Project Engineer during production:

TEST	FREQUENCY
Stability	1/400 ton or fraction thereof
Flow	1/400 ton or fraction thereof
% Voids in total mix	1/400 ton or fraction thereof
VMA, (% voids in mineral aggregate)	1/400 ton or fraction thereof
Lottman, CPL 5109	1/mix design
Dry Tensile Strength, PSI, CPL 5109	1/mix design

The person responsible for the SMA mixture designs and field control tests and the technicians performing them shall be identified at the preconstruction conference. The person responsible must possess one or more of the following qualifications:

1. Registration as a Professional Engineer in the State of Colorado

2. NICET certification at Level II or higher in the subfield of Highway Materials or Asphalt, Concrete and Soils.

3. A minimum of five years testing experience with soils, asphalt pavement and concrete.

Technicians performing the tests shall have previous design experience with the Marshall Method and must possess one or more of the following:

1. A minimum of two years testing in the specialty field.

2. Certification by a nationally recognized organization such as NICET.

3. For the appropriate specialty field, Certification by the American Concrete Institute (ACI), or by the Colorado Asphalt Producers Association (CAPA).

REVISION OF SECTIONS 401, 403, AND 703
STONE MASTIC ASPHALT PAVEMENT

Subsection 403.03 shall include the following:

Tack coat between the existing pavement and SMA shall be placed at a rate between 0.03 and 0.05 gallons per square yard.

Subsection 403.04 shall include the following:

Stone mastic asphalt pavement will be measured by the ton

Subsection 403.05 shall include the following:

Payment for Stone Mastic Asphalt Pavement will be full compensation for, mix design, furnishing, hauling, preparing, and placing all materials, limestone dust, hydrated lime, tack coat, and approved control strip; for labor, equipment, tools, setting of lines and guides where specified, and incidentals necessary to complete the item.

Subsection 703.04 shall include the following:Coarse Aggregate:

Aggregate for Stone Mastic Asphalt Pavement shall conform to the following:

Coarse aggregate for SMA shall meet the requirements of this subsection with the following additions:

1)	L.A. Abrasion Loss (AASHTO T96)	30% max
2)	Sodium Sulphate Soundness Loss (AASHTO T104)	12% max

100% crushed gravel shall be used in SMA mix. A minimum of 90% of the materials retained on the #4 screen shall have two or more fractured faces. Aggregates used in SMA shall be from a single source.

Fine Aggregate: Fine aggregate shall meet the following requirements:

Sodium Sulphate Soundness Loss (5 cycles, AASHTO T109) 12% max

Fine aggregate shall consist of 100 percent crushed aggregate and shall be nonplastic (AASHTO T-90).

REVISION OF SECTIONS 401, 403, AND 703
STONE MASTIC ASPHALT PAVEMENT

Subsection 703.04, Table 703-3 shall include the following:

Sieve Size	Grading SMA
3/4"	100
1/2"	90-100
3/8	75 (Maximum)
#4	20-30
#8	16-24
#16	
#30	
#50	
#100	
#200	7-11

Subsection 703.06 shall include the following:

Mineral filler for the Stone Mastic Asphalt pavement shall be limestone dust and shall meet the requirements of this subsection and the following:

Plasticity Index (AASHTO T-90)	4% max

The Contractor shall submit hydrometer analysis (AASHTO T88) for mineral filler.

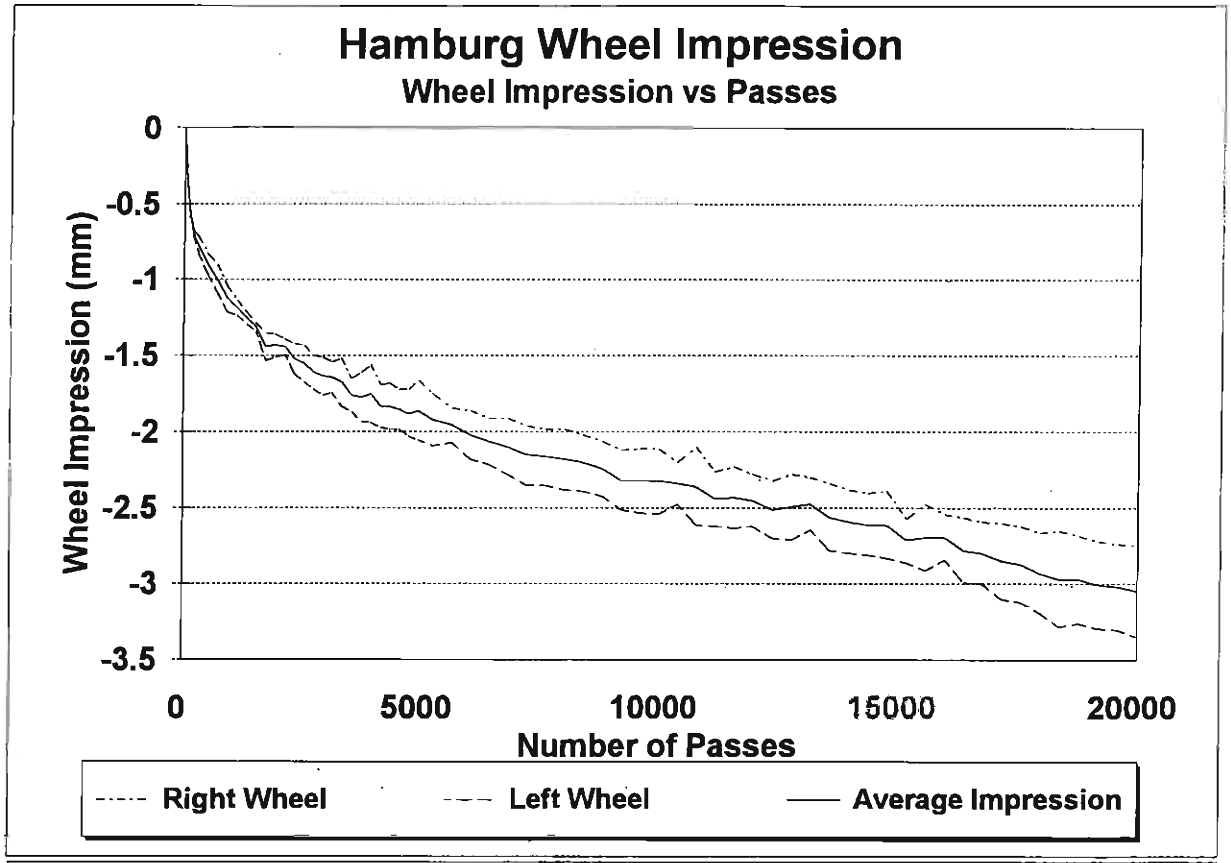
The mineral filler shall be stored in a silo and added automatically in the correct proportion. The mineral filler shall be added at the point the asphalt cement is added.

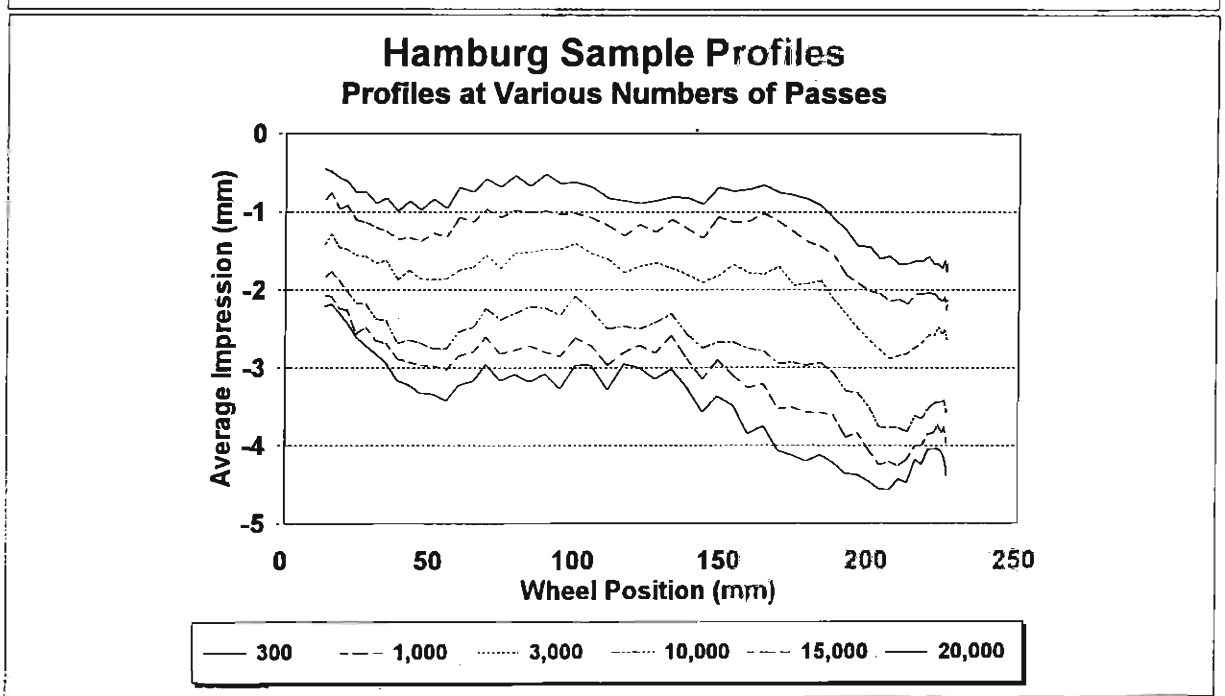
Appendix B:
Hamburg Wheel-Tracking Device Test Results

Colorado Department of Transportation	Date	06-Nov-95	
	Sub Account	0	
	Field Sheet No.	0	
Staff Materials	Location	Colfax	
	Region	0	
Item 403	Contractor		

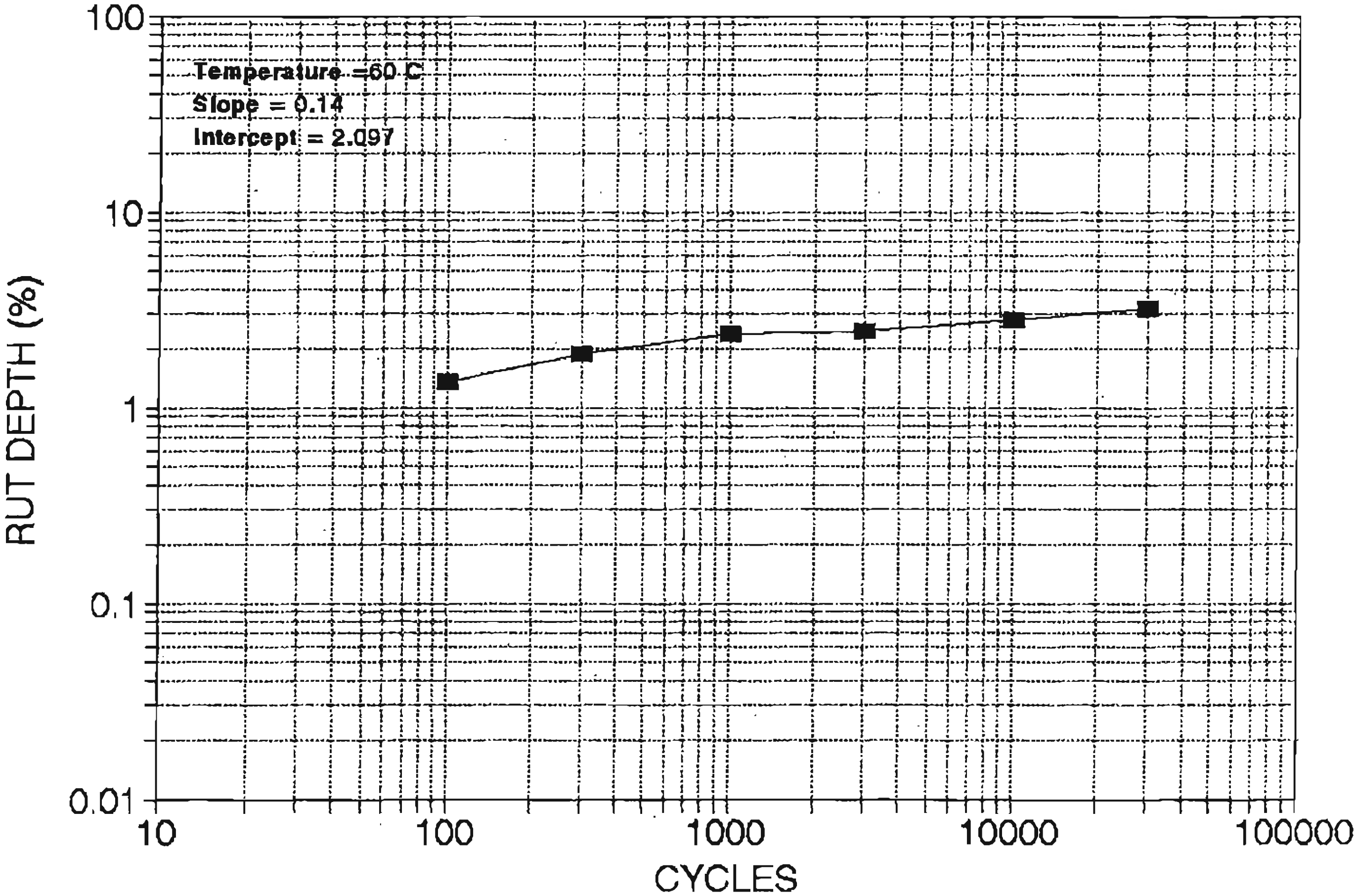
	Creep	Strip	Inflection
Average	11535	ERR	0
Left	10431	ERR	0
Right	12948	ERR	0
Creep	Lft 1000 - 20000		Rt 1000 - 20000
Strip	Lft 0 - 0		Rt 0 - 0

Temp: 50° C **SMA Colfax**





Appendix C:
French Rutting Tester Results



Temperature =60 C
Slope = 0.14
Intercept = 2.097
100
10
1
0.1
0.01
RUT DEPTH (%)
10
100
1000
10000
100000
CYCLES

SMA, COLFAX