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Large-Stone Hot Mix Asphalt Pavements

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16. Abstract

Large-stone HMA pavements have been promoted by various associations including the National Asphalt Pavement Association (NAPA) as an answer to rutting problems which exist with current pavement designs. Until 1988 the Colorado Department of Transportation (CDOT) had little or no experience with the advantages or disadvantages of large-stone HMA pavements.

This report investigates problems encountered in the construction of three large-stone HMA construction projects. Problems encountered with large-stone HMA include segregation of the mix and the validity of mix designs produced with standard test equipment. The study examined the following; lab tests in use by the CDOT for large-stone pavements, constructability, field performance, and cost required to produce the mix.

Implementation:

Grading G should be allowed on projects when practical. Grading G should be used as a base coarse. Segregation and smoothness should be addressed before paving. If used as a wearing surface, it should be for experimental projects where pavement smoothness and segregation of the mix are addressed extensively before paving. Grading G should be placed in a minimum lift of 75mm (3 inches).

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1.0 INTRODUCTION

Hot mix asphalt (HMA) pavements with large stones have been promoted by various asphalt associations including the National Asphalt Pavement Association (NAPA) as an answer to rutting problems which exist with current pavement designs. Until 1988 the Colorado Department of Transportation (CDOT) had little or no experience with the placement of large-stone HMA pavements.

Expected problems with large-stone pavements include segregation of the mix and the validity of mix designs produced with standard test equipment. Thin overlays can also be difficult to place as the mat thickness approaches that of the maximum aggregate size. Large-stone HMAs are generally unsuitable for the top (finished) mats, and they may cause excessive plant wear, negating some of the cost benefits.

The objectives of this study are to investigate the following:

A. To examine the lab tests in use by the CDOT for large-stone HMA pavements.

B. The constructability of a large-stone HMA pavement. This included the tendency of the mix to segregate, the measures needed to prevent segregation, and the suitability of existing batching and laydown equipment as well as overall pavement smoothness and compaction.

C. The performance of a large-stone HMA pavement. This included rutting and stripping.

D. The cost required to produce the mix. Specifically the cost of producing a large-stone HMA compared with a standard HMA.

2.0 BACKGROUND

The idea of using large sized aggregates (up to 1-1/2") in asphalt pavements is an old idea that appears to show increasing promise in modern pavement designs. Large-stone HMAs were popular from the turn of the century until the 1950's when more uniform and easily handled small-stone HMAs became favored for asphalt production. Large-stone HMAs placed in Tennessee in the 1950's and 1960's are still in service.

Recent problems in a number of states with mix stability and early pavement rutting have since created a need for more rut-resistant pavement designs.

With the use of large-stone HMAs bid prices should be somewhat reduced. Since the total surface area of the aggregate is reduced, the quantity of asphalt required is lower. The large-stone requires less crushing again reducing costs on a per ton basis over standard mix. The cost comparison of large-stone HMAs is shown in section 6.0.

3.0 MIX DESIGN FOR LARGE-STONE MIXES

Specifications and mix design procedures were developed by the CDOT for large-stone HMAs based upon information from the National Asphalt Pavement Association (1) and the Kentucky Department of Transportation (2).

3.1 Specifications

3.1.1 Aggregates

The master range for the gradation of the Grading G large-stone HMA is shown in Table 1.

Table 1. **Master Range Gradation for Grading G.**

Sieve Size (mm)	Sieve Size (In.)	Percent Passing
37.5	1-1/2	100
19.0	3/4	63 - 85
12.5	1/2	46 - 78
4.75	#4	22 - 54
2.36	#8	13- 43
0.300	#50	2 - 20
0.075	#200	0 - 8

These aggregates shall have a percentage of wear of not more than 45 when tested in accordance with AASHTO T 96. All material retained on the 4.75 mm (No. 4) sieve shall have at least two fractured faces, and the amount of natural fines shall not exceed 15 percent of the total weight of the aggregate.

3.1.2 Mixtures

The large-stone HMA shall be designed within an air void range of 3 to 5% and with a minimum asphalt cement content of 3.8%. For durability the large-stone mix shall have a minimum tensile strength ratio of 0.60 when tested according to CPL-5109, a modified Lottman procedure.

No requirements for rut resistance have been established. It is customary to specify a minimum Hveem stability for the design of the standard sized aggregate mixtures. However, the large-stone HMA requires the use of a 150-mm (6-in.) diameter sample that will not fit into the 100-mm (4-in.) diameter stabilometer.

Based on the aggregate requirements and information from other states for the large-stone HMA, it was believed the large-stone HMA should be more rut resistant than the standard smaller sized aggregate mixtures. Some creep compliance testing was performed to subjectively evaluate the large-stone HMAs. These test were not conclusive. An additional study may be undertaken to determine acceptable values.

3.2 Laboratory Testing Procedure

The standard sized samples used for the smaller top-sized aggregate mixtures are 100-mm (4-in.) diameter. The large-stone HMAs required a minimum sample diameter of 150-mm (6-in.) Individual test procedures had to be adjusted to account for this difference in diameter.

3.2.1 Sample Preparation

Trial and error procedures were performed to determine how to obtain a well mixed sample. A 4000 gram sample is a very large quantity to mix. The quantity was able to fit in the Hobart 19L (20-quart) mixer. Originally a dough hook was used for mixing and this was unsuccessful since partially coated aggregate was common. Also, the aggregate would bind between the hook and the bowl and either stop the mixer or fly out of the bowl. A special cover over the bowl was required for safety.

A wire whisk proved to be an excellent mixing instrument. Since the aggregate was well coated, this is the process that is currently used.

3.2.2 Sample Compaction

Several states were performing designs on large-stone HMAs by using the Marshall method. The weight of the drop hammer used for the 100-mm (4-in.) diameter sample was increased in order to apply the same pressure on a 150-mm (6-in.) diameter sample. It is understood that the combination of the added weight of the hammer and the larger aggregate has made crushing during compaction a problem.

The CDOT does not use the Marshall method, but uses the Hveem method. Compaction of the sample required some modification to the California kneading compactor. The area of the foot required for a 100-mm (4-in.) diameter sample is 2000 mm^2 (3.1 $in.^2$). This area was proportioned upwards by volume of the sample to 4520 mm^2 (7.0 $in.^2$), and the shape remained similar. When the new foot was manufactured, it was necessary to get the foot attachment at the location which would permit the outside edge of the foot to be virtually flush with the mold. This precise machining was accomplished at the machine shop.

It was desired to use the same compactive effort for the large-stone mixes and the standard smaller sized aggregate mixes. Currently, a 3.1 MPa (450 psi) pressure is used. The kneading compactor displays the pressure of the hydraulic reservoir during compaction. The displayed pressure is identical to the foot pressure when the foot has an area of 2000 mm^2 (3.1 $in.^2$). When the area of the foot is changed, the pressure displayed must be adjusted to get the actual foot pressure. With a 4520 mm^2 (7.0 $in.^2$) foot, a pressure on the gauge of 7.0 MPa (1016 psi) would be equivalent to an actual foot pressure of 3.1 MPa (450 psi). A pressure of 3.1 MPa (450 psi) is the compactive effort used for the smaller aggregate sizes.

After these modifications, compaction of the sample was generally the same as that for the 100-mm (4-in.) diameter sample. Since the area of the foot was proportioned upward by volume, the same number of tamps, 90, and the same pressure 3.1 MPa (450 psi) were used. This gives the 150-mm (6-in.) diameter sample the same force applied per volume of specimen as the 100-mm (4-in.) diameter sample.

3.2.3 Resistance to Moisture Damage

This test was performed by using AASHTO T 283 as modified by CPL-5109. This did not change. Since the samples were 150-mm (6-in.) in diameter, loading strips with a radius of 75 mm (3 inches) were used during the loading. When using these strips, no end correction to account for flattening was required.

4.0 SITE DESCRIPTIONS

Three sites were monitored during construction for problems due to the large-stone HMA. The equipment and procedures used for the test sites are listed for each of these three sites.

4.1 I-70 @ Cedar Point

The first large-stone HMA mix was placed on construction project CXIR 34-0070-03 located in the west bound lanes of I-70 from M.P. 348.24 to M.P. 348.62 in August of 1989, a total of 600m (2,000 ft). The contractor for this project was Asphalt Paving. The Project Location Map is shown in Appendix A-1. The driving lane and passing lane were paved with 83mm (3-1/4 inches) of Grading G, the shoulder received 108mm (4-1/4 inches). Figure 1 shows the typical section for this project.

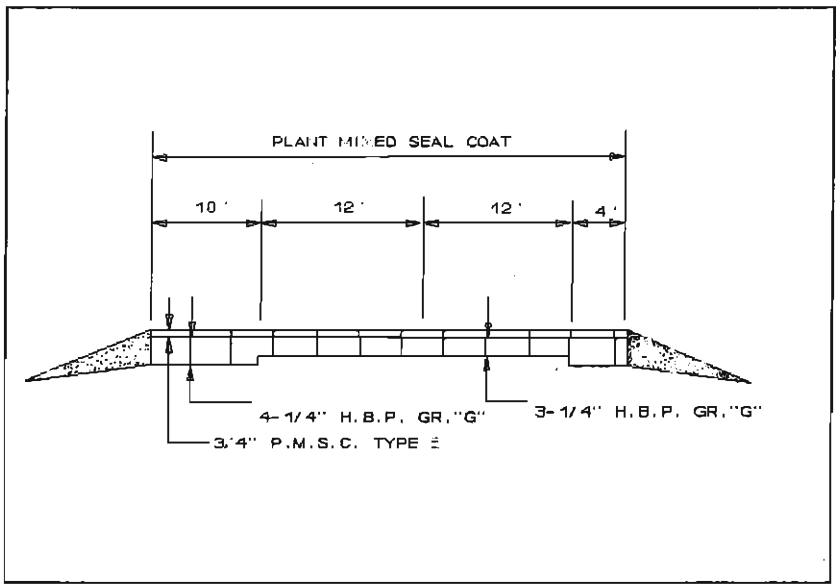


Figure 1 - I-70 LARGE STONE TYPICAL SECTION

Plans for the project called for a 19mm (3/4 inch) thick Plant Mix Seal Coat Type B to be place over the entire project. However the Grading G did not appear to be as open and susceptible to moisture as originally anticipated, so it was decided to add to the Grading G test section. A plant mix seal coat was not placed over the first 90m (300 ft.) leaving this area open to weather. This is prior to the 300m (1000 ft.) that had been set up for evaluation. This section was evaluated as well as the test (Grading G with PMSC) and the

control section (Grading C) sections. 1,637 tons of large-stone HMA was placed at this location. The design asphalt content was 4.7% with 4.9% air voids at optimum. The job mix formulas for each of the projects evaluated are shown in Table 2.

Table 2 Job Mix Formula

	Percent Passing		
Sieve	I-70	SH-14	SH-40
37.5mm (1-1/2in)	100	100	100
19.0mm (3/4in)	77	77	77
12.5mm (1/2in)	62	65	59
4.75mm (No.4)	44	46	45
2.36mm (No.8)	34	31	35
0.30mm (No.50)	10	10	14
0.075mm (No.200)	4.4	4	4.8
% AC	4.7	4.2	4.6
Source & Grade of AC	Elf AC-20R	Conoco AC-10F	Conoco AC-20

4.1.1 Construction Techniques

The Grading G was delivered from the asphalt plant approximately three miles from the site in bottom dump trucks. The large-stone HMA was laid down in a windrow in front of the elevating loader that placed the material into the laydown machine's hopper. The contractor, Asphalt Paving felt that this was the best way to control segregation of the mix during paving.

The large-stone HMA left the plant at approximately 150 degrees C (300 degrees F). During the fourth roller pass, mat temperature was 115 degrees C (240 degrees F). Two steel rollers were used for achieving compaction. Five passes (each pass equals one run over the section) with a vibratory roller and two without vibration produced a percent of relative compaction ranging from 92.2 to 94.2. The minimum percent relative compaction specification for this project was 92.0% based on the maximum specific gravity (AASHTO T 209).

4.1.2 Post-Construction Appearance

The overall appearance of the finished project was similar to that of a standard mix. Some small pockets of segregation could be seen, but there were no severe problem areas with segregation. The project appeared to be smooth and in overall good condition. The overall smoothness of the project is discussed in section 5.2.1 where the results of the Profilograph testing are located.

4.2 SH-14

FR 014-2(17) was constructed in April of 1990 and was located from M.P. 143.5 to M.P. 147.3 on SH-14 east of Ft. Collins. Site location map is shown in Appendix A-1. The test section was 55m (1800 ft.) of Grading G placed at a width of 17.1m (56 ft.). A depth of 127mm (5 inches) was placed in two 64mm (2-1/2 inch) lifts. A total of 4,102 tons of Grading G was used on this section. The Grading G was then covered with a 38mm (1-1/2 inch) thick Grading E rubberized lift for a surface coat. The control section for this project was a 114mm (4-1/2 inch) overlay of Grading E (maximum 19mm (3/4 inch) aggregate). This area is used as a control section for comparing the Grading G and the Grading E. Specifications for the Grading G on this project are as follows: air voids - 4-6%, Lottman - 70(min), asphalt content - 4.3% (AC-10). The typical section is shown in Figure 2 below.

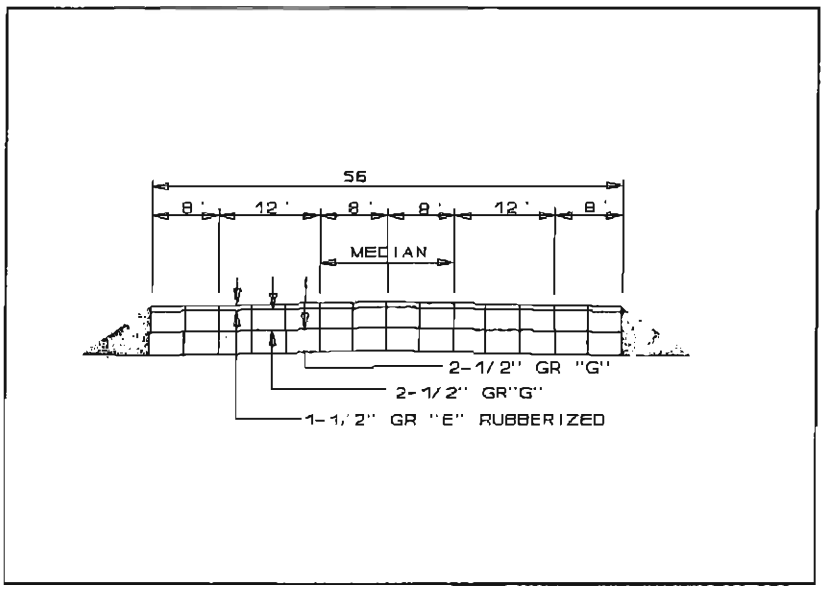


Figure 2 - SH-14 LARGE STONE TYPICAL SECTION

The test and the control sections are not ideal for comparison because the test section was all new construction,(base, pavement) and the control section was an overlay on top of the existing pavement. The two sections carry identical traffic so construction and pavement distress can still be compared.

4.2.1 Construction Techniques

Paving operations began on the morning of April 27, 1990 with the drum mixer located at I-25 and Harmony Road in Ft. Collins approximately ten minutes travel time from the construction site. The contractor Western Mobile, Inc. chose to use end dumps for hauling the mix and placed the mix directly into the laydown machine's hopper. Early in the construction the segregation problem was severe. The majority of the problem appeared to be at the point where the mix was transferred from the haul truck to the laydown hopper. To help reduce segregation the contractor had the end dumps raise their beds to a point where the mix would slide against the tail gate before releasing the gate and thereby allowing the mix to drop uniformally into the hopper of the laydown machine. This appeared to help greatly in reducing the segregation occurring at the transfer point. The other method used to reduce segregation was for the laydown operator not to raise the wings on the hopper until the end of the day at which time the material was discarded. Figures 3 and 4 show the haul truck, laydown machine and how they were being used on this project. Figures 5 and 6 show the segregation occurring inside the laydown hopper at the edge of the wings.

Three rollers were used on this project in order to achieve compaction. The first roller was a Dyna Pac-CC50 steel vibratory roller. This roller made two passes in the vibratory mode. Each pass is defined as once over the mix. Next a large rubber tired roller (BROS-SP-1000) made two passes. The finish roller was a REX 910 steel wheel that made two passes on the section in the non-vibratory mode. Temperatures directly behind the paver averaged 120 degrees C (250 degrees F). Compaction readings ranged from 92% to 93%. Minimum compaction specification was 92%.

Figure 3 - Pavement Equipment used on SH-14

Figure 4 - End dump used to place mix directly into paver hopper on SH-14.

Figure 5 - Segregation of large-stone mix
can be seen at the edge of the paver hoper wings.

Figure 6 - Same as figure 5.

4.2.2 Post-Construction Appearance

The overall appearance of this project showed several pockets of segregation. Many of these locations are where the laydown machine stopped to reload its hopper. Figures 7 and 8 show an area of heavy segregation mostly due to equipment problems with the laydown machine. The laydown machine placed too thin of a lift at this point and began to drag larger aggregate behind the laydown screed. Figure 9 shows a closeup of one of the large-stone segregated pockets seen on the project. Figure 10 shows the segregated pocket. A 38mm (1-1/2 inch) thick Grading E rubberized mix was placed over the top of the large-stone HMA, giving an overall appearance of a smoothly completed project.

4.3 SH-40

Project CX 33-004-13 was located on SH-40 beginning approximately 17.5 Km (11 miles) southeast of Hugo and extending approximately 17.5 Km (11 miles) southeasterly to the Cheyenne/Lincoln county line at M.P. 422.18. The project map can be seen in Appendix A-1. The contractor was Kiewit Western. The project was constructed in August of 1990. The project consisted of a leveling course followed by two layers of HMA. The first lift was 89mm (3 inches) Grading G covered with a 38mm (1-1/2 inch) lift of rubberized Grading C. A 300m (1000 ft.) section was placed at M.P. 416 consisting of two 38mm (1-1/2 inch) lifts of Grading C covered with a 38mm (1-1/2 inch) lift of rubberized Grading C. This section is the control section for evaluating the performance of the Grading G material placed on the majority of the project. Specifications for Grading G on this project are as follows: air voids 3-4%, Lottman -80(min), asphalt content 4.5% (AC-20). Figure 11 below shows the typical section for the Grading G test section.

Two 180m (600 ft.) test sections were setup for evaluation. One section was the Grading G which was the test section, located in the eastbound lane at M.P. 416 + 2000. The other section was located within the 300m (1000 ft.) Grading C material at M.P. 416 + 00, which was used as a control section.

Figure 7 - Segregated areas due to equipment problems and too thin of a lift causing aggregate to be dragged.

Figure 8 - Same as Figure 7.

Figure 9 - Close-up of a segregated pocket on SH-14.

Figure 10 - Segregated pocket on SH-14.

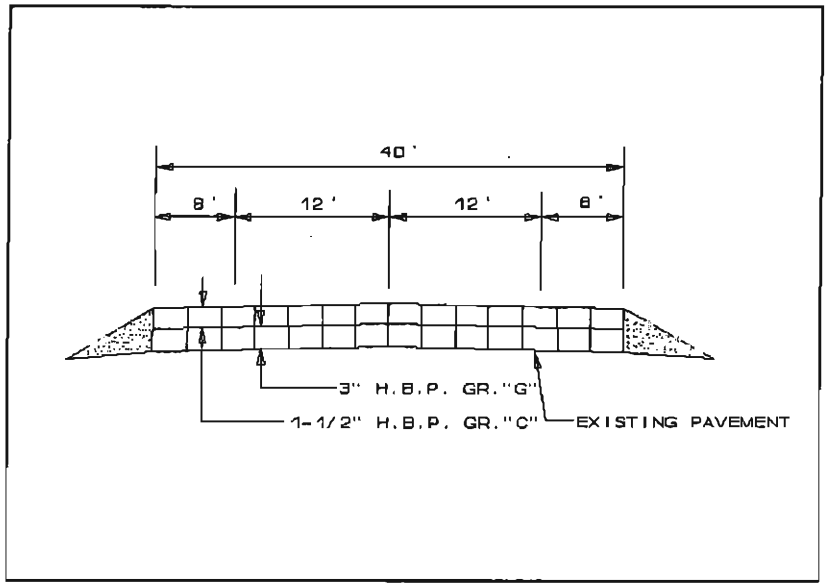


Figure 11 - SH-40 LARGE STONE TYPICAL SECTION

4.3.1 Construction Techniques

Paving operations began in August 1990 with the drum mixer located approximately 4.8 Km (3 miles) from the construction site. The contractor, Kiewit Western Co., chose to use bottom dump style trucks and an elevating loader to place the material into the laydown machine hopper. Using this equipment eliminated some of the segregation that was anticipated, however, segregation could be seen when the equipment came to a stop both at the end of the day and while waiting for material to arrive on site. Laydown hopper wings were again kept from being raised as seen in the first two large-stone HMA projects to eliminate some of the mix segregation. Figures 12, 13, and 14 show the laydown operation.

Two steel wheel rollers were used to achieve compaction. The first was a large vibratory roller that made three passes in vibratory mode. This was followed by a smaller steel wheel roller that made two passes bringing the average compaction to 94%. This was well within the minimum 92% specification. Temperatures directly behind the paver averaged 127 degrees C (260 degrees F). Figure 15 shows the large vibratory roller used for this project.

This project required 20,200 tons of large-stone HMA, approximately four times more material than had been placed previously. Because of the large quantities, some equipment wear may have been observed. The contractor had problems at the plant and with the paving equipment. A total of five clutches had to be replaced in the laydown machines during the course of the project. According to the contractor, this was attributed to the added weight of the mix (the lifts of the Grading G on this project were 76 mm (3 inches) thick while the Grading C was 38mm (1-1/2 inches) thick) plus the shoulder and the driving lane were paved in one pass. Had the driving lane and the shoulder been paved separately some the paver problems may have been eliminated. The contractor also mentioned that the bearings in the auger appeared to wear faster when placing the Grading G than when placing the Grading C HMA. The plant foreman stated that modifications to the drum mixer fins were required early in the project. The fins bent over and slowed production. These fins had to be straightened and reinforced before production could resume. These problems may not be totally due to the Grading G HMA but are mentioned because of the possible relationship.

4.3.2 Post-Construction Appearance

The contractor appeared to have problems with segregation on the west end of this project. In this area, paving was intermittent due to new bridge construction that was to be performed at a later date. Short paving spans led to many starts and stops in the paving operation, causing many heavily segregated areas. As the paving operation approached the east end of the project where paving was continuous, fewer segregated areas were noticed. The finished project appeared to be smooth and similar in appearance to a project paved with a standard mix.

Figure 12 - Bottom dump placing mix in windrow for an elevating loader to place material into the paver hopper.

Figure 13 - Elevating loader used to load mix into paver hopper on SH-40.

Figure 14 - Paver operation on SH-40.

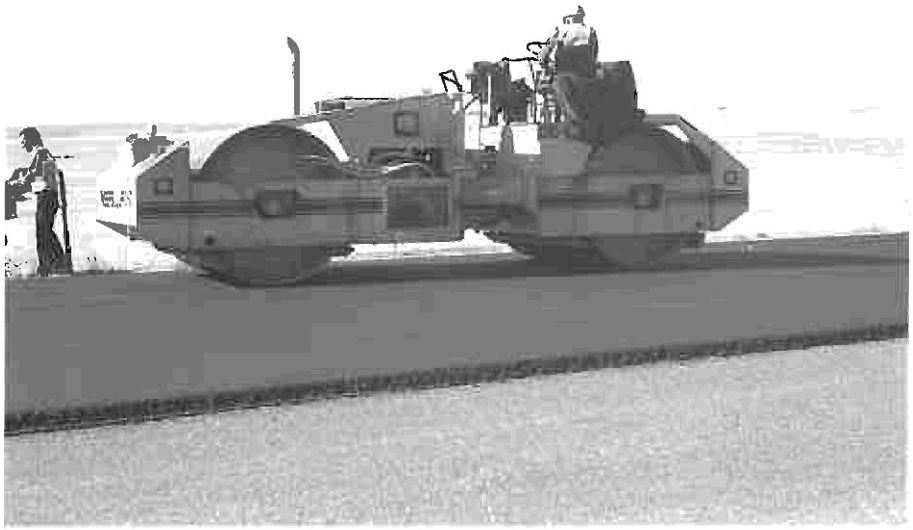


Figure 15 - Vibratory roller used directly behind paver.

5.0 SITE EVALUATIONS AND FIELD PERFORMANCE

Field evaluations were performed on three sites following construction for a period of three years. However, the I-70 project was prematurely rehabilitated after the first year due to problems encountered with the Plant Mix Seal Coat that was placed over the mix being evaluated. Evaluations are shown for this section up to the period where the site was lost due to the removal of the Plant Mix Seal Coat. Each of the three sites were tested for rutting, smoothness, moisture damage.

5.1 Rutting

Rutting was measured at each site by placing a 1.8m (6 ft.) straight edge across each wheel path within the lane to be tested. A calibrated wedge was placed between the straight edge and the pavement, allowing a reading in millimeters to be taken. Each site had a control section and a test section. The test section is the large-stone HMA, with the control section being the mix that was normally used. Both the test and control sections were 183m (600 ft.) in length. Rut measurements were taken every 15m (50 ft.) within the section, giving a total of 13 readings per site. Rutting depths equal to or greater than 8mm are considered unacceptable.

Each site was also cored, with the samples of the pavement sent to the CDOT Materials Lab for air void testing. Air voids for each site were monitored over the evaluation period. This test is an indicator to rutting. Rutting is related to the loss of air voids in the pavement, thereby air void testing gives a reasonable comparison to rut measurements.

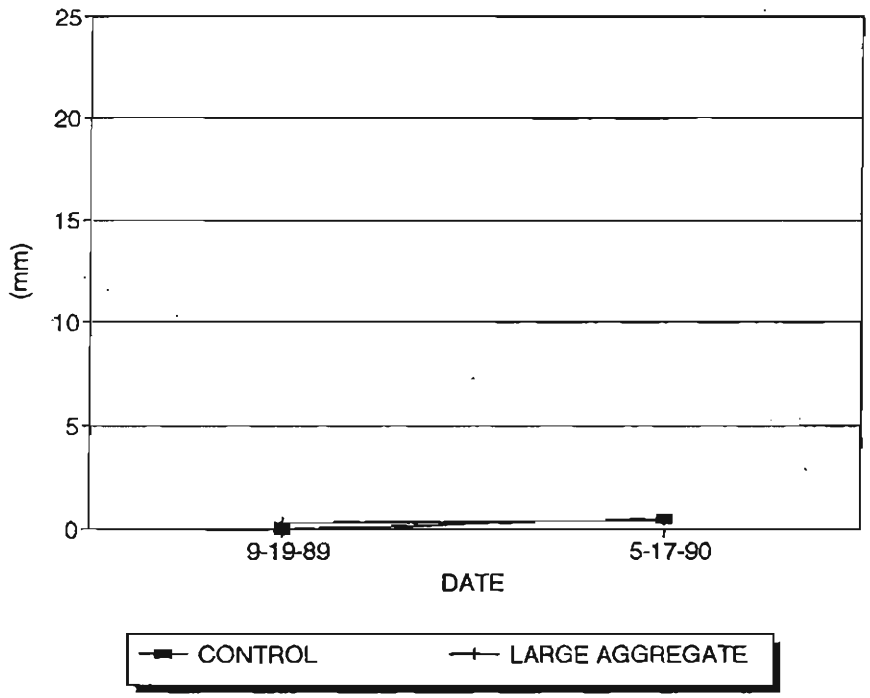
5.1.1 I-70 @ Cedar Point

Construction on the I-70 project was performed in 1989, in 1990 the plant mix seal coat was removed from the surface of the control section and part of the test section. This stopped any trend line in respect to rutting. Rut measurements were only taken for the post construction period and one year following construction. These results are shown in Figure 16. The large-stone HMA shows little if any rutting for this period (approximately 0.2mm (.008 inches)), while the control section (Grading C) also had little if any rutting (approximately 1.2mm (.048 inches)). However, in the spring of 1994 rutting was found in

RUT DEPTH MEASUREMENTS
LARGE AGGREGATE STUDY
I-70 @ CEDAR POINT

CONTROL (mm)

DATE	LWP	RWP	OVERALL
9-19-89	0	0	0.00
5-17-90	1.3	1	1.15
TEST		(GRADING G)	
9-19-89	1.6	0	0.80
5-17-90	1.5	0.5	1.00



* P.M.S.C. WAS REMOVED FALL OF 1990 DESTROYING THE TEST AND CONTROL SECTIONS.

Figure 16 - I-70 Rut Depth Measurements

I-70
% AIR VOIDS

	GRADING G	PMSC OVER G	GRADING C	PMSC OVER C
LIFT THICKNESS	83mm	19mm	76mm	19mm
1990	5.4	10	8	10
1991	4.8	9.6	N/A	N/A
1992	4.1	9.6	N/A	N/A
1993	3.4	9	N/A	N/A
Total change	2%	1%	0	0

*25.4mm = 1 inch

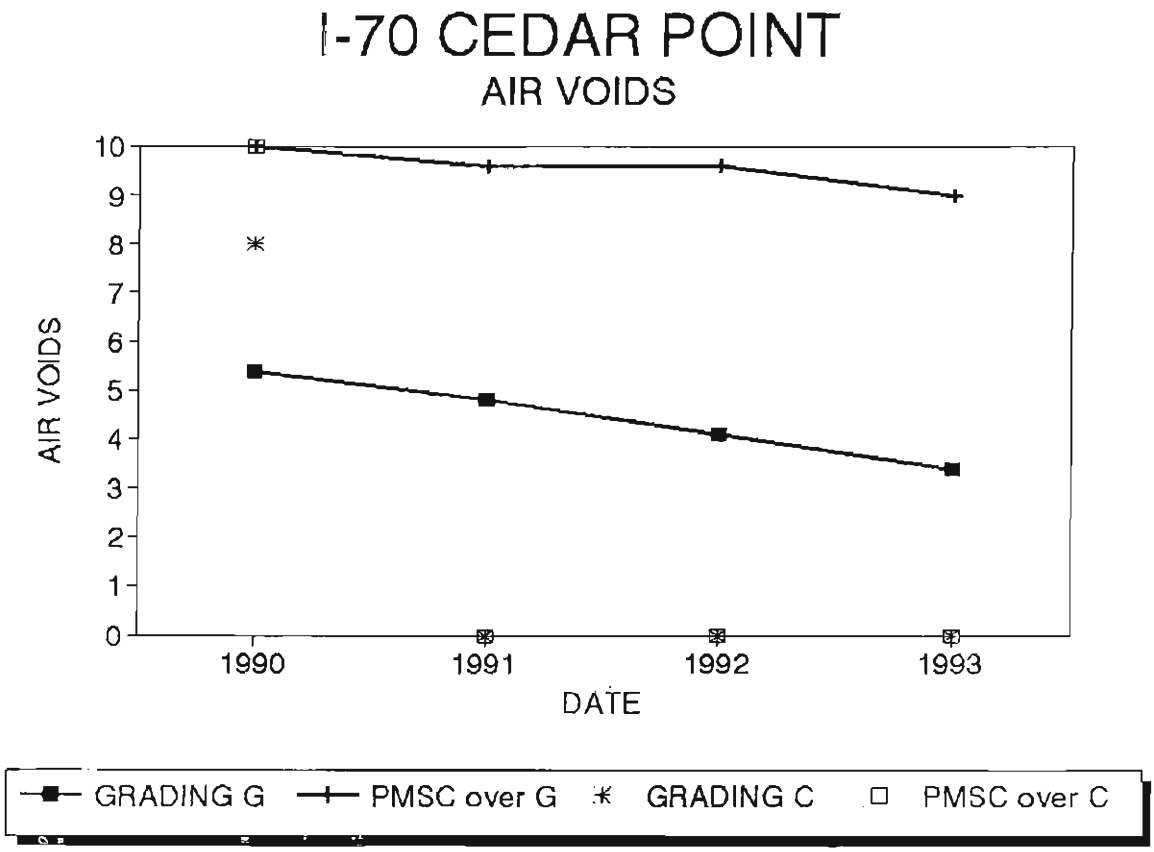


Figure 17 - I-70 Air Voids

the Grading G test section with the PMSC. Maximum rutting showed 11mm (0.44 inches) with a minimum of 3.75mm (0.15 inches). At this time there was a minimum of rutting in the uncovered Grading G. This shows the possibility that the PMSC is causing stripping within this section. The rutting in the PMSC section was not continuous (pockets within the section), which also gives credence to the possibility of PMSC causing stripping to occur within the section below the PMSC. The problem discussed here is documented with data from other sites in CDOT Report No. CDOT-DTD-R-91-4, "Rut-Resistant Composite Pavement Design".

Cores were taken for a period of four years in the Grading G section. Air voids in the pavement for these samples are shown in Figure 17. The Grading G section shows a decrease in air voids from 5.4% to 3.4%, a loss of 2% air voids over a three year period. This normal amount of consolidation taking place within this material is in direct relationship to the amount of rutting which occurred within the large-stone HMA section. The Grading C which is the control section was monitored for only one year, not allowing for good comparison with the Grading G.

5.1.2 SH-14

Rut measurements were taken for a period of three years, showing very little change in pavement rutting in both the control and test sections. The control section showed basically the same rutting for the three year period (0.4mm). The test section also showed very little change (0.1mm). Both sections rutting results can be seen in Figure 18. The test and control sections are both rut resistant.

Cores were removed from the pavement and the results of lab tests for air voids are shown in Figure 19. This Figure shows similar results to the field rut testing. The Grading G showed a drop of 0.4% in air voids over the four year period. This section was constructed at a depth of 64mm. The change in air voids relates to approximately a 0.3mm rut. The measured rutting is virtually all due to traffic densification. The control section shows a 1.7% drop in air voids which relates to a 1.3mm rut in the 76mm thick Grading E. The rutting measured on this project is virtually all due to traffic densification.

5.1.3 SH-40

Rut measurements were taken for a period of three years, once again showing little change in rutting in either test or control section. Both the control section and the test section both showed approximately the same amount of rutting at the same rate which can be seen in Figure 20. The amount of rutting approaches the maximum acceptable levels of 8mm.

Cores were removed from the pavement and the results of the lab test for air voids are shown in Figure 21. This Figure shows similar results to the field rut measurements. The biggest change in air voids can be seen in the Grading G toplift. This lift is a 38mm lift of Grading C rubberized material. Air voids on this material dropped 4.9% over three years. This shows that a large part of the rutting occurring in this section may be caused due to the top 38mm lift densifying and not the large-stone HMA below. Looking at Figure 21 it can also be seen that this holds true on the control section. The control section showed a drop of 2.3% air voids over the three years.

The majority of the rutting appears to have occurred within the top surface mat and not in the test and control material below the surface. The rutting has been caused by traffic densification, not plastic flow. When comparing the two sections, both have equal ability to resist rutting.

5.1.4 Additional Sites

Three additional Grading G sites were visited and checked for rutting. Neither of these three sites were formally evaluated, but it was felt that additional information on rutting could be valuable.

5.1.4.1 I-70 Arriba to Flagler. Project IR(CX)070-5(56) was constructed November 1991. This project consisted of a 75mm (3 inch) Grading G lift covered with a 37mm (1-1/2 inch) Grading C. Three years after construction, rut measurements in both wheel paths and both driving and passing lanes showed zero rutting.

5.1.4.2 SH-30 Piccadilly Rd. South. Project CX 10-0030-16 was constructed in August of 1989. This project consisted of a 75mm (3 inch) Grading G lift covered by a 19mm (3/4 inch) PMSC. Ruts were measured on this project in 1994, five years after construction. Maximum rutting was found to be 3.75mm (.15 inches), with an average rutting of 1.5mm (0.06 inches).

5.1.4.3 I-70 at Genoa. Project FRI-CX-CY070-5(059) was closed out in December of 1991. This project consisted of a 63mm (2.5 inch) to 100mm (4inch) Grading G lift inlaid in the driving lane with no surface coat applied. The Grading G was left open for a period of one year. Raveling within this section began to occur so a surface coat, (micro-surfacing) was applied. This section showed no rutting after three and one-half years.

RUT DEPTH MEASUREMENTS
LARGE AGGREGATE STUDY
SH-14

CONTROL (mm)

DATE	LWP	RWP	OVERALL
6-25-90	1.5	0.5	1.0
7-15-91	0.8	0.8	0.8
5-6-92	2.5	1.5	2.0
5-26-93	1.0	1.3	1.2
TEST		(GRADING G)	
6-25-90	1.5	3.3	2.4
7-15-91	1.0	0.8	0.9
5-6-92	0.8	0.8	0.8
5-26-93	0.0	1.5	0.8

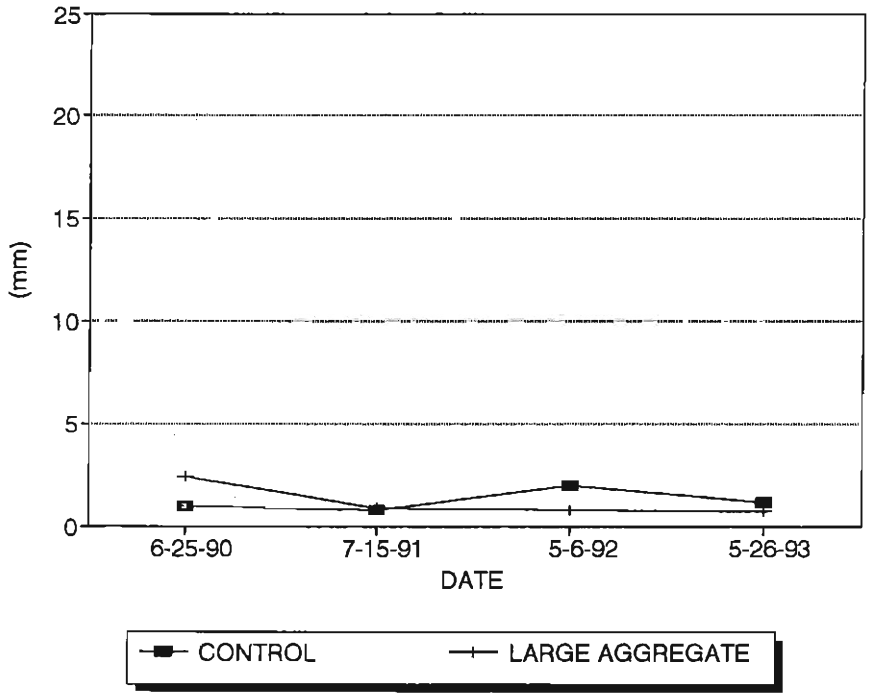


Figure 18 - SH-14 Rut Depth Measurements

SH-14
% AIR VOIDS

	GRADING G	G SURFACE COAT	GRADING E	E SURFACE COAT
LIFT THICKNESS	64mm	25mm	76mm	25mm
1990	7.4	4.3	6.3	4.1
1991	7	2.6	6	4.5
1992	7.2	3.9	5.8	3.7
1993	7	2.6	5.5	3.7
Change in Voids	0.4	1.7	0.8	0.4

*25.4mm = 1 inch

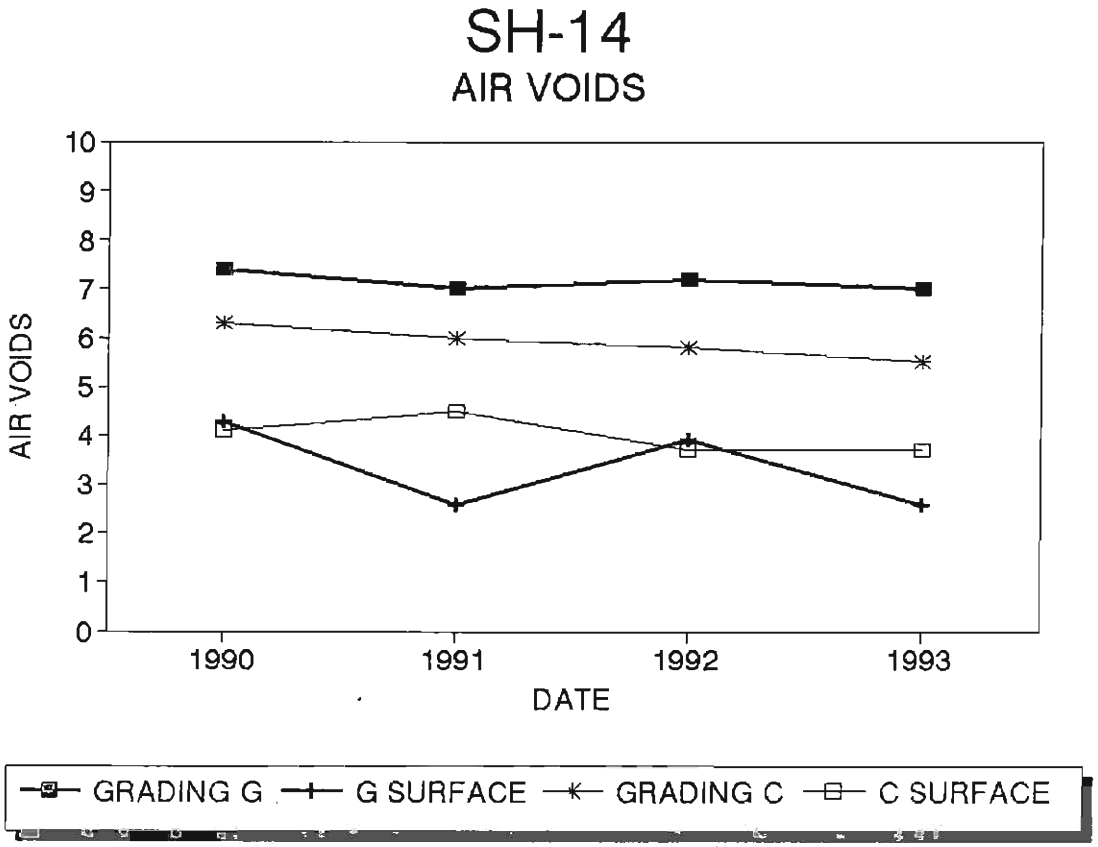


Figure 19 - SH-14 Air Voids

RUT DEPTH MEASUREMENTS
LARGE AGGREGATE STUDY
SH-40

CONTROL (mm)

DATE	LWP	RWP	OVERALL
5-24-90	14.7	16.5	15.60
11-30-90	3.6	0	1.80
6-20-91	6.4	2	4.20
6-5-92	9.7	4.8	7.25
8-4-93	5.1	9.4	7.25
TEST		(GRADING G)	
5-24-90	14	20.6	17.30
11-30-90	2	0	1.00
6-20-91	4.6	1.5	3.05
6-5-92	8.4	4.3	6.35
8-4-93	7.6	5.1	6.35

* 25.4mm = 1 inch
* 5-24-90 (Pre-Construction)

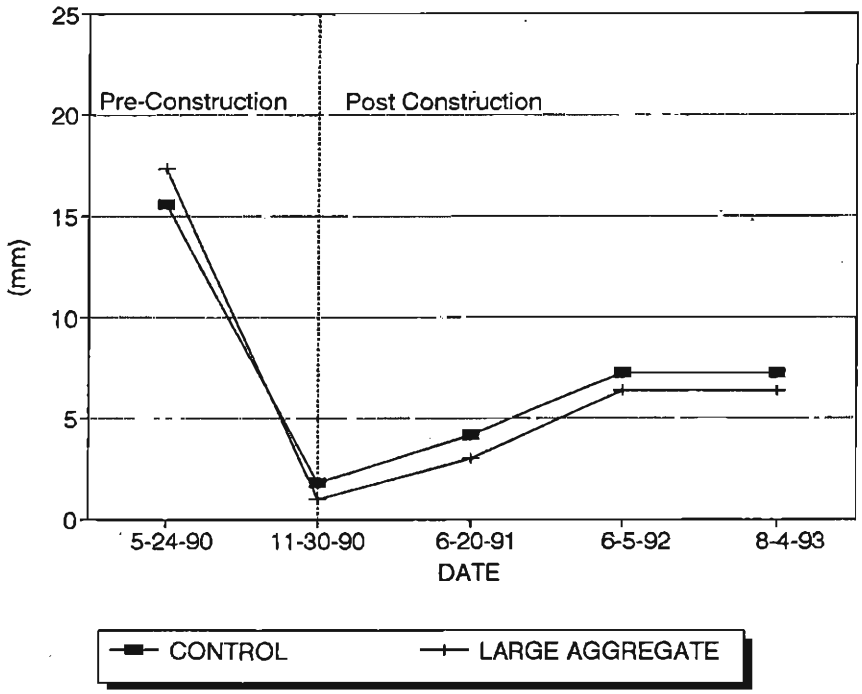


Figure 20 - SH-40 Rut Depth Measurements

SH-40
AIR VOIDS

	GRADING G	G SURFACE COAT	GRADING C	C SURFACE COAT
LIFT THICKNESS	89mm	38mm	76mm	38mm
1991	3.6	9.9	6	5.5
1992	4.5	5.1	5.3	3.5
1993	2.8	5	5.3	3.2
Change in Voids	0.8	4.9	0.7	2.3

*25.4mm = 1 inch

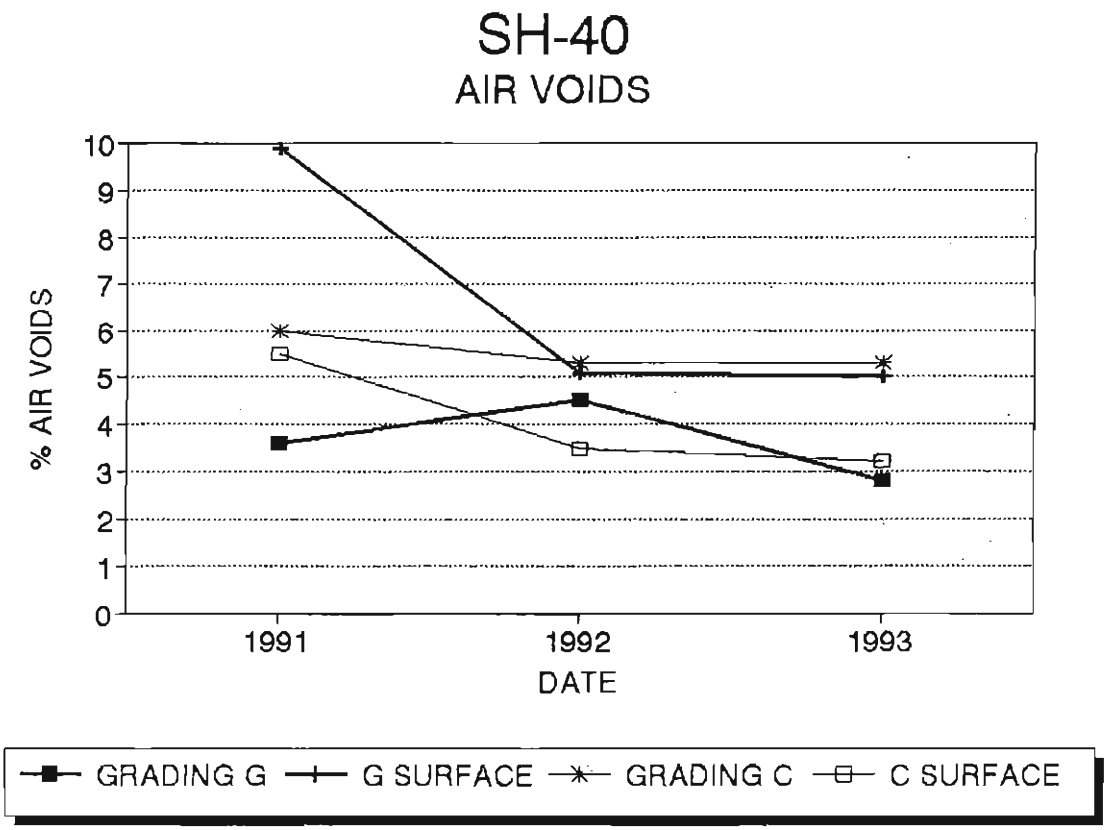


Figure 21 - SH-40 Air Voids

5.2 Smoothness

Smoothness is measured by using a Profilograph consisting of a frame 7.6m (25 ft.) in length supported on wheels at both ends. The profile is recorded from the vertical movement of a wheel attached to the frame at midpoint and is in reference to the mean elevation of the points of contact with the road surface established by the support wheels. For this study an Ames Profilograph was used to collect smoothness data. The CDOT has recently incorporated a specification for smoothness using the California Type Profilograph. For comparison purposes with projects tested for the large-stone HMA study, Table 3 below shows the incentive/disincentive specification for smoothness that is being used for 1994 CDOT projects.

Table 3 Incentive/Disincentive Payment for Smoothness using Profilograph

Millimeters/ Kilometer	Millimeters/ Meter	Inches/Mile	Payment Dollars/Square Meter
32 or less	0.032 or less	2 or less	+ 0.25
32 to 47	0.032 to 0.047	2 to 3	+ 0.20
47 to 63	0.047 to 0.063	3 to 4	+ 0.15
63 to 79	0.063 to 0.079	4 to 5	+ 0.10
79 to 95	0.079 to 0.095	5 to 6	+ 0.05
95 to 111	0.095 to 0.111	6 to 7	0.00
111 to 126	0.111 to 0.126	7 to 8	-0.05
126 to 142	0.126 to 0.142	8 to 9	-0.10
142 to 158	0.142 to 0.158	9 to 10	-0.15
158 to 174	0.158 to 0.174	10 to 11	-0.20
174 to 189	0.174 to 0.189	11 to 12	-0.25
over 189	over 0.189	over 12	-0.25, plus corrective work

+ indicates incentive
- indicates disincentive

5.2.1 I-70 @ Cedar Point

Figure 22 shows the results of smoothness testing starting with November 1989 (immediately after construction), and May 1990. Testing was performed for only two years because of the premature rehabilitation during the summer of 1990. Figure 22 shows that placing a smooth mat with the large-stone HMA was difficult. The large-stone HMA after construction showed 260mm and 290mm per kilometer, where the control section (Grading C) was constructed with a Profilograph reading of 28mm per kilometer.

Comparing the above results with the CDOT smoothness specification it can be seen that the large-stone HMA is more difficult to place smoothly, resulting in the contractor being required to perform corrective work to reduce the roughness and take a $0.25 per square meter disincentive pay. The control section (Grading C) was placed at 28mm per kilometer, thereby allowing the contractor to gain $0.25 per square meter on incentive pay, if the smoothness specification were used.

On this project smoothness was very difficult to achieve on the test (Grading G) section. The contractor showed no difficulties in producing a smooth pavement in the control section (Grading C).

5.2.2 SH-14

Figure 23 shows the results of smoothness testing starting immediately after construction with yearly testing for the following three years. This figure shows similar results to that seen on the I-70 site. Immediately after constructing the test section (Grading G), it showed 260mm per kilometer, well above the maximum 1994 CDOT specification. Corrective work and disincentive pay would be required. The control section (Grading E) was placed very smoothly at 35mm per kilometer. The contractor would be awarded an incentive.

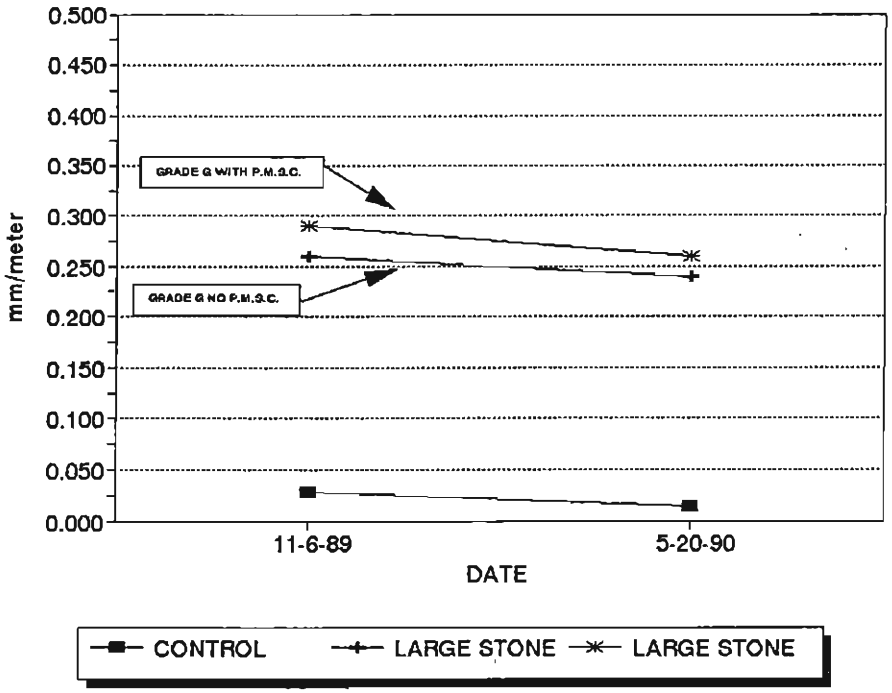
The test section (Grading G) showed again that smooth placement was very difficult to achieve in comparison to the control section (Grading E). The smoothness did not improve with time.

5.2.3 SH-40

Figure 24 shows the results of smoothness testing immediately after construction starting in 1990 and continuing for a three year period. Once again it can be seen that the test section (Grading G) was not smooth immediately following paving. The large-stone HMA section showed 309mm per kilometer after construction. Corrective action and disincentive pay would be required. The control section (Grading C) was better at 185mm per kilometer. The contractor would be assessed a disincentive, but no corrective action would be required. Neither section was acceptable when compared to the 1994 CDOT Smoothness Specification. The smoothness did not improve with time.

PROFILOGRAPH RESULTS
LARGE AGGREGATE STUDY
I-70 Cedar Point

CONTROL		(mm/meter)	
DATE	LWP	RWP	OVERALL
11-6-89	0.028	0.028	0.028
5-20-90	0.014	0.017	0.015
TEST	(Grading G)	with P.M.S.C.	
11-6-89	0.145	0.376	0.260
5-20-90	0.188	0.292	0.240
TEST	(Grading G)	no P.M.S.C.	
11-6-89	0.283	0.298	0.290
5-20-90	0.194	0.327	0.260



* Surface Coat (P.M.S.C.) was removed 10/10/90 preventing further profile testing.

Figure 22 - I-70 Profilograph Results

PROFILOGRAPH RESULTS
LARGE AGGREGATE STUDY
SH-14

CONTROL (mm/meter)

DATE	LWP	RWP	OVERALL
6-25-90	0.035	0.035	0.035
7-15-91	0.049	0.139	0.095
5-6-92	0.035	0.098	0.066
5-26-93	0.055	0.098	0.077
TEST			
6-25-90	0.145	0.376	0.260
7-15-91	0.188	0.292	0.240
5-6-92	0.275	0.417	0.346
5-26-93	0.167	0.376	0.272

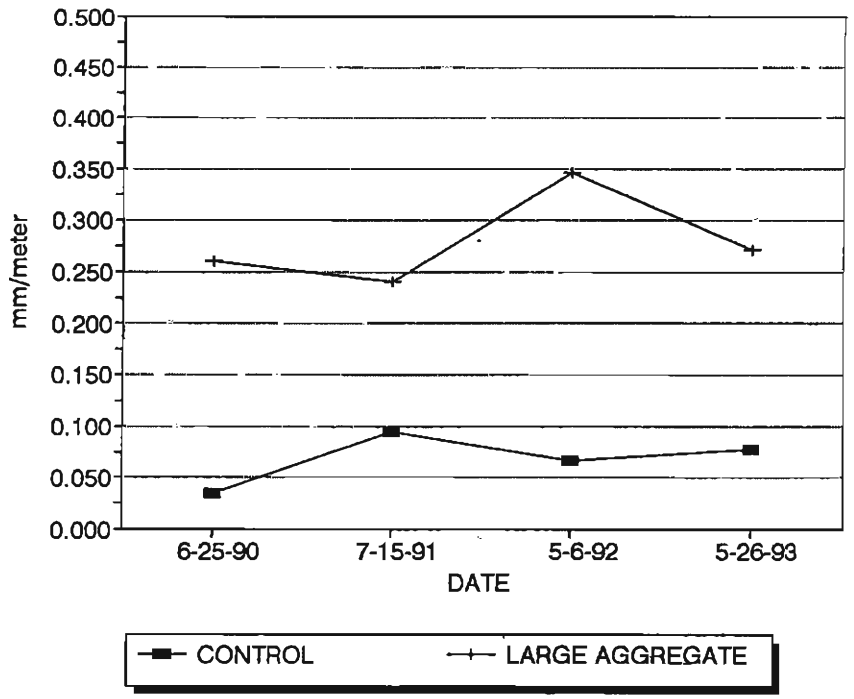


Figure 23 - SH-14 Profilograph Results

PROFILOGRAPH RESULTS
LARGE AGGREGATE STUDY
SH-40

CONTROL (mm/meter)

DATE	LWP	RWP	OVERALL
11-30-90	0.396	0.223	0.309
6-20-91	0.188	0.133	0.159
6-5-92	0.379	0.237	0.308
8-4-93	0.362	0.229	0.295
TEST			
11-30-90	0.188	0.180	0.185
6-20-91	0.229	0.368	0.298
6-5-92	0.237	0.194	0.302
8-4-93	0.202	0.229	0.216

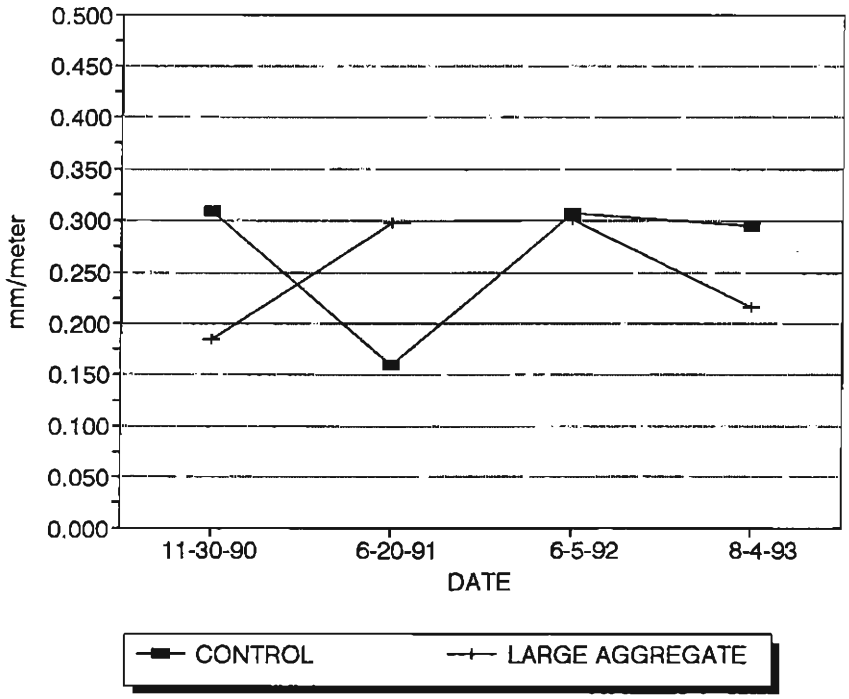


Figure 24 - SH-40 Profilograph Results

5.3 Moisture Damage

A modified Lottman indirect tensile stripping test has been used by the CDOT as the standard test for the moisture susceptibility of asphalt mixes. The test patterns the procedures outlined by AASHTO T 283, ASTM D 4867 and is defined by Colorado Procedure L-5109. This test is an indirect tensile stripping test which measures the effects of accelerated water conditioning by saturation and freezing of compacted bituminous mixtures.

CDOT specifications for field produced material is a minimum of 70% tensile strength ratio (TSR). As the tensile strength ratio rises, the mix becomes less susceptible to moisture. Table 4 shows the results of CPL-5109.

For all projects except SH-40, the TSRs were acceptable at the time of construction. Cores were taken from each project, remolded, and tested according to CPL-5109. For all projects, the TSRs of recompacted samples remained acceptable and unchanged. Test results from CPL-5109 indicated the pavements were not experiencing moisture damage. All of the surfaces of the cores were smooth. This is another indication that moisture damage was not occurring on these projects.

Table 4 Lottman Results (CPL-5109)

I-70 @ CEDAR POINT

	Grading G			Grading G no PMSC			Grading C		
YEAR	TSR	Voids	Satur.	TSR	Voids	Satur.	TSR	Voids	Satur
1990	91	8.2	36	No	Test	------	88	6.4	77
1992	117	5.9	62	99	6.2	66	88	6.2	68
1993	84	3.4	63	No	Test	------	80	*	*

SH-14

	GRADING G			GRADING E		
YEAR	TSR	VOIDS	SATUR.	TSR	VOIDS	SATUR.
1990	81	6.9	71	78	6.5	65
1991	95	6.1	*	No	Test	--------
1992	95	6.6	72	78	6.5	70

SH-40

	GRADING G			GRADING C		
YEAR	TSR	VOIDS	SATUR.	TSR	VOIDS	SATUR.
1991	61	6.1	57	87	5.8	72
1993	No	Test	--------	94	5.6	62

6.0 COST COMPARISON

The cost of Grading G was compared to Grading C on projects that used both materials. This allowed a closer comparison since the haul price should be equal, allowing for the only difference to be material costs. Table 5 shows eight projects that used both materials within the past five years. These costs were taken from the CDOT Cost Data Book published by the Cost Estimates Squad of the Staff Design Branch.

The price ratio of Grading G to Grading C over the five year period showed that the large-stone HMA cost 102% of the standard HMA (Grading C). It can be seen that this varied from project to project. The highest cost of Grading G was shown at 110% of the standard HMA on a project in 1993. The lowest cost was with the Grading G being 92% of the standard HMA on a project in 1990.

A large-stone HMA theoretically should be less expensive than a standard HMA due to the fact that less crushing is required of the aggregate to meet the design parameters, and there is less use of asphalt. This does not account for specialized modifications to equipment and procedures to produce small amounts of large-stone HMA mix. Large-stone HMAs evaluated for this study use between 4.2% and 5.0% asphalt content compared to Grading C HMAs that are designed for asphalt content of 5.0 to 5.5%.

Table 5 shows that at present large-stone HMAs are no more or less expensive than CDOT standard HMAs. However, as usage of this mix increases its overall cost should decrease due to added knowledge about placement and production of the mix.

TABLE 5 - GRADING G VS. GRADING C COST COMPARISON

YEAR	PROJECT NO.	GRADING G ITEM NO.	TONS USED	PRICE PER TON	GRADING C ITEM NO.	TONS USED	PRICE PER TON	PRICE RATIO GRADING G TO GRADING C
1989	CXIR 34-0070-03	40307020	1637	$20.00	40305020	33457	$19.00	105.26%
1990	CXBRF 11-0285-23	40307020	12170	$22.00	40305020	3440	$24.00	91.67%
1991	BRF 040-5(11)	40307020	2183	$37.15	40305020	3109	$35.15	105.69%
	FC(CX) 40-5(9)	40307020	1461	$37.50	40305020	2287	$36.20	103.59%
1992	FR(CX) 014-2(21)	40307020	26141	$33.40	40305020	13786	$33.20	100.60%
	IM-NH-IR(CX) 070-3(16	40307020	4204	$24.00	40305020	37052	$24.00	100.00%
1993	FCU-NH(CX)-CX-CY 28	40307020	4847	$30.00	40305020	3659	$30.00	100.00%
	MR-STM 8256(2)	40307020	3616	$22.00	40305020	2958	$20.00	110.00%
	TOTAL TONS GRADING G USED		56259		TOTAL TONS GRADING C USED			99748
	AVERAGE PRICE PER TON		$28.26		AVERAGE PRICE PER TON			$27.69

PRICE RATIO GRADING G TO GRADING C FOR 5 YEAR PERIOD 102.03%

7.0 CONCLUSIONS

The following conclusions are taken from the evaluations of the three sites, each showing similar results to the testing performed and in construction evaluation.

1. The large-stone mixtures were very difficult to place. Segregation was always a problem. By using better construction techniques, segregation was reduced, but is was always present.

2. Rut depth measurements showed that the control sections built with the smaller-aggregate HMAs and the test sections built with the large-stone HMAs had the same rutting potential.

3. The control sections were consistently smoother than the test sections. All of the large-stone mixtures would have required corrective action with the 1994 Smoothness Specification; two of the three smaller aggregate HMAs would have received incentives.

4. The cost of the smaller aggregate HMAs are very similar to the large-stone HMAs. It was thought the large-stone HMAs would have lower costs due to less crushing of the aggregate and less asphalt content, but as a whole the large-stone HMA was not less expensive. These costs may lower if large-stone HMAs are used more frequently bringing up the overall tonnage.

8.0 IMPLEMENTATION

1. Grading G should be used in the future when practical.

2. Grading G should be used as a base course. Segregation and smoothness should be addressed before paving.

3. If used as a wearing surface, it should be for experimental projects where pavement smoothness and segregation of the mix are addressed extensively before paving.

4. Grading G should be placed in a minimum lift of 75mm (3 inches).

REFERENCES

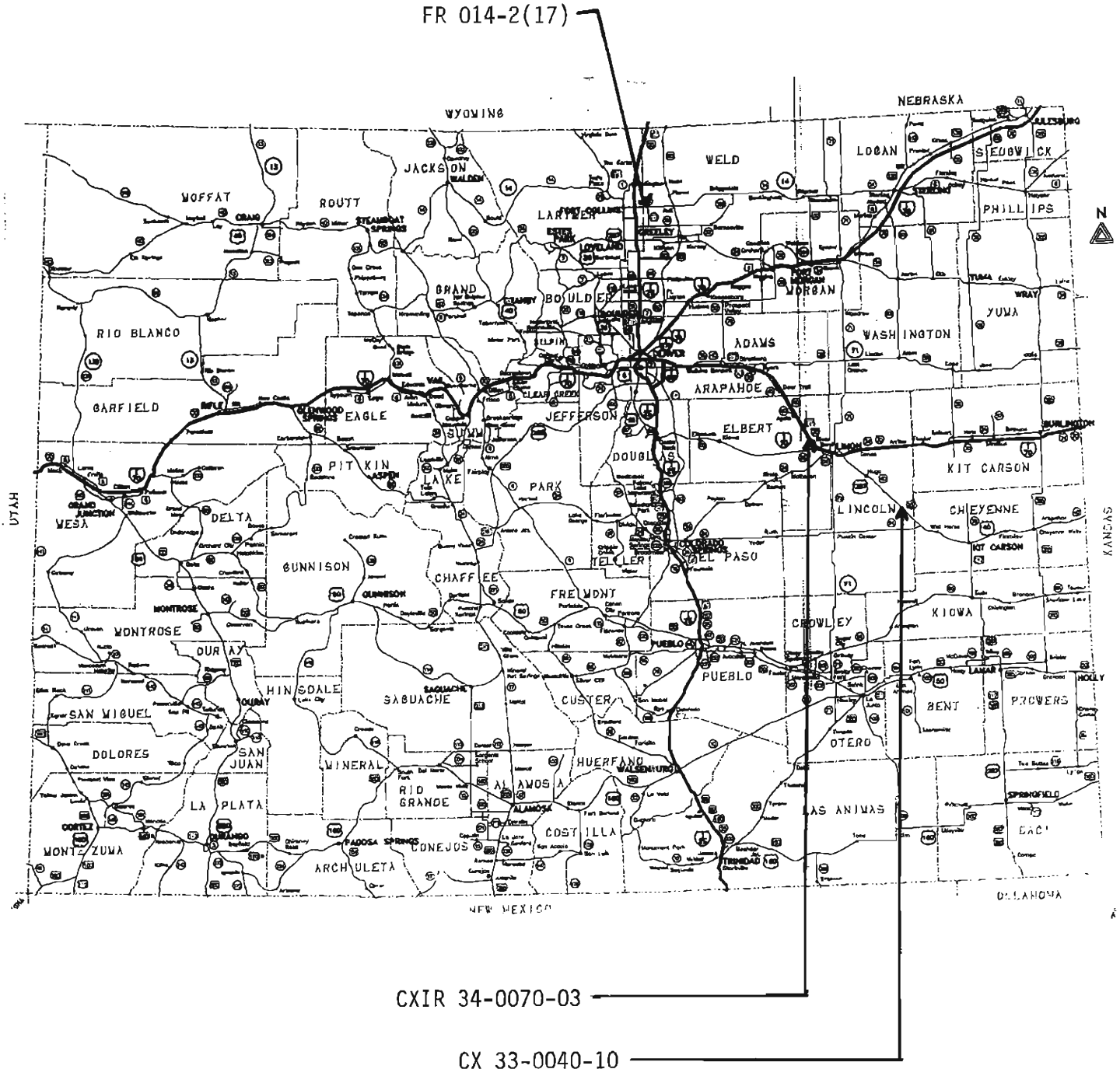
1. Acott, M. "Design of Hot Mix Asphalt for Heavy Duty Pavements," National Asphalt Pavement Association, QIS 111/86, October 1987, 16 pages.

2. Anderson, R., "Kentucky's Experience with Large Size Aggregate in Bituminous Hot-Mix," Association of Asphalt Paving Technologists, 1991, Vol. 60, pp. 1-18.

APPENDIX A

PROJECT MAP

Large-Stone Mix Project Map



APPENDIX B

GRADATION MASTER RANGE

Master Range Table for Hot Bituminous Pavement

Sieve Size	Percent by Weight Passing Square Mesh Sieves		
	Grading G	Grading C	Grading E
1-1/2"	100		
1"			
3/4"	63-85	100	100
1/2"	46-78	70-95	
3/8"		60-88	
#4	22-54	44-72	44-72
#8	13-43	30-58	30-58
#30	4-22	12-34	
#200	1-8	3-9	3-12

Master Range Table for Plant Mixed Seal Coat

Sieve Size	Range of Job-Mix Tolerances +or-	Percentage by Weight Passing Square Mesh Sieves		
		Type A	Type B	Type C
1/2"		100	100	100
3/8"		90-100	90-100	90-100
#4	8%	30-50	35-57	40-60
#8	6%	5-20	12-33	20-47
#50	6%	.....	3-15	4-18
#200	2%	2-7	2-8	2-9