University of Alberta

PERFORMANCE EVALUATION OF 2002 AND 2003 ALBERTA ASPHALT RUBBER PROJECT

By

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ABSTRACT

Highway agencies are faced with the challenge of maintaining pavement infrastructure, which is crucial to both economic and social activities. The overlays using Asphalt Rubber Mixture (ARM) have evolved as an effective rehabilitation strategy over the last four decades. In the summer of 2002 and 2003, pavement sections using ARM were paved in Alberta to investigate the effectiveness of ARM in Alberta weather condition.

This thesis presents the performance evaluation of the 2002 and 2003 Alberta Asphalt Rubber pavement sections. The performance evaluation of more than ten asphalt rubber pavement sections, laid in 2002 and 2003, are discussed.

A laboratory experiment was conducted at the University of Alberta to reconfirm the 2003 ARM design and to investigate the moisture resistance of ARM. The laboratory experiment revealed open-graded ARM to be more prone to moisture damage compared to dense-graded ARM.

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I wish to dedicate this thesis to my parents for their continuous love, support and encouragement throughout all phases of my life.

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CHAPTER 1 INTRODUCTION

1.1 Background

Highway agencies are faced with the challenge of maintaining the pavement infrastructure, which is very crucial to both economic and social activities. Applying an effective maintenance strategy is important in reducing the pavement maintenance and rehabilitation costs, and also in minimizing the traffic delays and inconveniences resulting from frequent maintenance activities. Highway agencies are continuously looking for different solutions to maintain and extend the useful service life of pavement. One suggested solution to enhance the service life of pavement is by using paving materials with improved engineering properties.

Asphalt Rubber Mixture (ARM), a mixture of asphalt cement, Crumb Rubber (CR) and aggregate, has been used for maintenance and rehabilitation of pavements for over four decades. The incorporation of CR, recycled from waste tires, in asphalt cement is believed to enhance the engineering properties of the binder and mix, in addition to the secondary benefit of consuming tire waste. More than three million tires are discarded annually in Alberta alone (Asphalt Rubber Information Database, 2004). The successful use of ARM in Alberta for pavement maintenance and rehabilitation can result in consuming substantial amounts of discarded tires. Although different states like California, Arizona, Texas, Alaska and other transportation agencies have reported effective applications of ARM for pavement maintenance and rehabilitation, there is no study or experimental data to guarantee a similar success in the Alberta's weather condition.

In the summer of 2002 and 2003, Tire Recycling Management Alberta (TRMA) partnered with Alberta Infrastructure and Transportation (AI&T), the City of Edmonton, the City of Calgary, Strathcona County and the City of Lethbridge and paved a total of 14 pavement overlays using ARM. The objective of this experiment was to study the application of ARM for pavement maintenance and rehabilitation in Alberta conditions. During the 2002 project, each project location paved a control section with conventional or other common asphalt mixtures, and the ARM test sections to compare their performances. However, during the 2003 project, only the ARM test sections were paved.

A total of four different aggregate gradations, four different mix designs and three different binder designs were used during the 2002 and 2003 Alberta Asphalt Rubber project. This thesis attempts to summarize the performance data from the 2002 and 2003 project locations, and analyze the potential problem faced by the 2002 and 2003 ARM sections.

In the summer of 2005, a field survey was conducted by the University of Alberta to measure the cracks and other surface distress of most of the 2002 and 2003 project locations. A similar evaluation was conducted on the 2002 project locations during the summer of 2004. The 2005 field survey suggested that some of the sections paved in 2003 were showing signs of moisture damage while they utilized the same mix design, aggregate gradation and binder design. The recommended asphalt content for the 2003 project was relatively less compared to the 2002 project. The Tensile Strength Ratio (TSR) of 2003 ARM, as an indication of moisture sensitivity of the mix, was also low with a value of 0.45. Based on these observations, it was decided to examine the moisture sensitivity, and to reconfirm the recommended asphalt content for 2003 ARM. In addition, it was decided to investigate the effect of aggregate gradation on the moisture sensitivity of ARM. It is felt that open-graded ARM allows water to permeate more easily than dense-graded asphalt mixtures. This increases the possibility of severe water damage due to freeze and thaw cycles. A laboratory experiment was conducted, at the University of Alberta, to investigate the above mentioned issues.

1.2 Scope and objectives

The main objective of this study was to conduct a performance evaluation on the 2002 and 2003 Alberta Asphalt Rubber pavement sections. More than ten asphalt rubber pavement sections were studied. Pavement conditions before and after overlay with ARM were monitored. In addition, mixture properties and designs of all these projects were measured. Pavement conditions, after overlay, were monitored by its roughness, rutting and cracks. For some locations, it was possible to compare the ARM with conventional asphalt mixtures at the same location.

A forensic study was conducted to reconfirm the 2003 mix asphalt rubber mixture design and to investigate the moisture resistance of ARM used in Alberta. The impact of moisture sensitivity on ARM was also investigated.

1.3 Organization of the thesis

The thesis has been structured in five different chapters. Chapter one gives a general background to the 2002 and 2003 Alberta Asphalt Rubber project.

Chapter two presents a brief literature review on the use of ARM for pavement maintenance and rehabilitation. In addition, literature reviews on the performance evaluation of ARM from other highway agencies is presented. The literature review focuses on the performance of the ARM overlays.

Chapter three summarizes the short-term performance evaluation of the ARM overlays laid in Alberta in the summer of 2002 and 2003, and identifies the problems related to the pavement performance based on the short-term performance evaluation.

Chapter four presents the designs and results of laboratory experiments to examine the moisture sensitivity of the open-graded and dense-graded ARM. Finally, Chapter five contains the conclusions and recommendations.

CHAPTER 2 LITERATURE REVIEW

Several highway agencies have conducted studies to characterize ARM for pavement maintenance and rehabilitation. Literatures suggest that the results from these numerous studies are mixed. Some agencies have reported substantial benefits from using ARM overlays for pavement maintenance and rehabilitation, while others do not find the performance of ARM overlays promising enough to justify its extra cost. There are numerous literatures related to the application of CR for pavement maintenance and rehabilitation and it is not possible to include all of them in this chapter. This chapter primarily consists of two main parts. The first part provides brief background information on the incorporation of CR as a paving material for pavement maintenance and rehabilitation. The second part presents the literature review on the field performance and laboratory studies conducted by different highway agencies. Only the literatures on the ARM (wet process) overlays have been considered. The emphasis was on the aggregate gradation and performance of the ARM overlays.

2.1 Background on application of CR for pavement maintenance and rehabilitation

The incorporation of CR, from scrap tires, for producing asphalt paving products is not a new technology and has been in use since the 1960's. Different highway agencies in the United States and various other countries have routinely used the CR modified asphalt paving products for pavement maintenance and rehabilitation (Epps, 1994).

The CR modified asphalt paving materials has been successfully used for crack filling, chip seals, Stress Absorbing Membrane Interlayer (SAMI), friction course and overlays. The application of CR for producing asphalt paving products can be accomplished by using several techniques including wet and dry processes which are discussed in the subsequent paragraphs.

Wet process: In the late 1960's, Charles McDonald, while working with City of Phoenix, Arizona, introduced the wet process or reacted CR process. In the wet process, the CR is blended and partially reacted with the asphalt cement and the resulting product is used as binder in the hot mix. Typically, the asphalt cement and the CR are reacted at a high temperature and diluents, aromatic oils and polymers are sometimes added. The American Society of Testing and Materials (ASTM) has defined the AR produced by wet process as "a blend of asphalt cement, reclaimed tire rubber, and certain additives in which the rubber component is at least 15 percent by weight of the total blend and has reacted in the hot asphalt cement sufficiently to cause swelling of the rubber particles" (ASTM D8, 2001).

Dry process: The dry, or non reacted, process was developed by the U.S. Rubber Reclaiming of Vicksburg, Mississippi in the late 1960s. The dry process adds shredded tires to the aggregate in a hot-mix central plant operation before adding the asphalt cement. The dry process is used in dense, open and gap-graded mixtures (Epps, 1994). The CR paving products, in this thesis, refers to products produced by wet process.

2.1.1 Advantages and limitations of AR

2.1.1.1 Advantages of AR

It is believed that incorporating CR in Hot Mix Asphalt (HMA) enhances the engineering properties of HMA, thereby making it suitable against rutting, fatigue cracking and low temperature cracking. The performance of ARM, at the different temperatures, is summarized in the subsequent paragraphs.

High temperatures: At high temperature the viscosity of the asphalt binder decreases thereby making it flow easily, and hence, prone to rutting. The addition of CR to HMA increases viscosity, thus stiffening the HMA at higher temperatures (Takallou et al., 1997 and Chipps et al., 2001).

Intermediate temperatures: At the intermediate pavement temperature, the HMA is prone to the reflective cracking as a result of the following:

- Tensile stains at the bottom of the asphalt layer due to excessive bending, and
- Load repetitions due to high traffic.

By adding CR to the HMA, an increase in resilience within the layer occurs and provides more elasticity during bending (Thomas et al., 2004). The addition of CR aids in the energy absorption properties of the HMA, thereby reducing the potential for failure due to cyclic loadings (Gopal et al., 2001).

Low temperatures: At low temperature, the HMA is prone to low temperature cracking, due to low stiffness and higher creep. Results from a number of researcher studies have shown that the addition of CR decreases the stiffness and increases the creep properties of the ARM at low temperatures (Takallou et al., 1997, Gopal et al., 2001, and Bahia and Davies, 1994).

Aging of asphalt mixtures: The hardening of asphalt binder and mixture starts at the mixing plant where the HMA is heated to temperatures ranging from 135 to 180°C. After the mixing, the age hardening continues in field at a much slower rate. The addition of CR to HMA has also been found to resist age hardening. Raad et al. (2001) found the stiffness of recently placed ARM and ten year old ARM to be similar, thereby indicating that ARM has undergone lower age hardening during its service.

The advantages of using AR can be summarized as follows (Hicks, 2002):

- Resistance to the reflective cracking because of added resilience.
- Resistance to rutting because of increased viscosity.
- Resistance to low-temperature cracking due to lower stiffness and higher creep at low temperature.
- Environmentally friendly as it consumes the discarded tires.
- Can be used in different maintenances (chip seal, SAMI, crack filling) and rehabilitations (overlays, open-graded friction course).
- Despite the higher initial construction cost, the ARM has proved to be cost effective because of reduced maintenance and rehabilitation costs. In addition, some highway agencies believe that the thickness of the overlay can be reduced to half the thickness of conventional HMA (Shatnawi, 2003).
- Many studies have found that pavements overlaid with ARM produce less noise than the conventional overlays.

2.1.1.2 Limitations of ARM

ARM has some limitations which makes highway agencies reluctant to use it. Some of the major limitations of ARM are as follows:

Higher initial cost: Several studies have indicated that AR binder alone can cost twice as much compared to conventional asphalt binders (Way, 2003). Typical cost increases for ARM are 1.5 to 2.0 times the cost of conventional mixtures (Epps, 1994).

Construction: The viscosity of AR is higher than the conventional asphalt binder, thus requiring higher mixing and construction temperatures. The restrictive temperature requirements along with the sticky AR make the construction more challenging. Similarly, the use of AR as binder requires several changes in the construction process (Epps, 1994). The ARM paving requires extra equipments to existing conventional HMA paving equipments. Some of the extra equipments include crumb rubber hopper, reaction vessel, blender etc.

2.2 Different agencies experiences with ARM overlays

2.2.1 Arizona experience with ARM overlays

Arizona Department of Transportation (ADOT) has a rich experience of AR applications. The first use of AR in Arizona, which dates back to 1964, was used as band aid maintenance. In Arizona, normally CR size passing 2 mm sieve is used (Way, 2002). The AR binder used in Arizona consists of 80% asphalt cement and 20% CR. The aggregate gradation commonly used with AR binder in Arizona is either open-graded or gap-graded (Morris and Carlson, 1992).

The open-graded mix is used solely as a surface course with air void of a minimum 15%. Generally the mix has 9% or more AR binder and has high friction characteristics. The open-graded overlays vary from 38 mm (1-1/2") to 50.8 mm (2") thickness and are used in conjunction with the gap-graded mix if additional structural strength is required (Morris and Carlson, 1992).

The other ARM is essentially a dense-graded mix designed to have 5% air voids. However, portions of small aggregate and fines are reduced to provide a gap-grading. The AR content in the gap-graded mix is usually 7.5 to 8.5% (Morris and Carlson, 1992). Figure 2.1 shows the aggregate gradation used in Arizona for open-graded and gap-graded gradations.

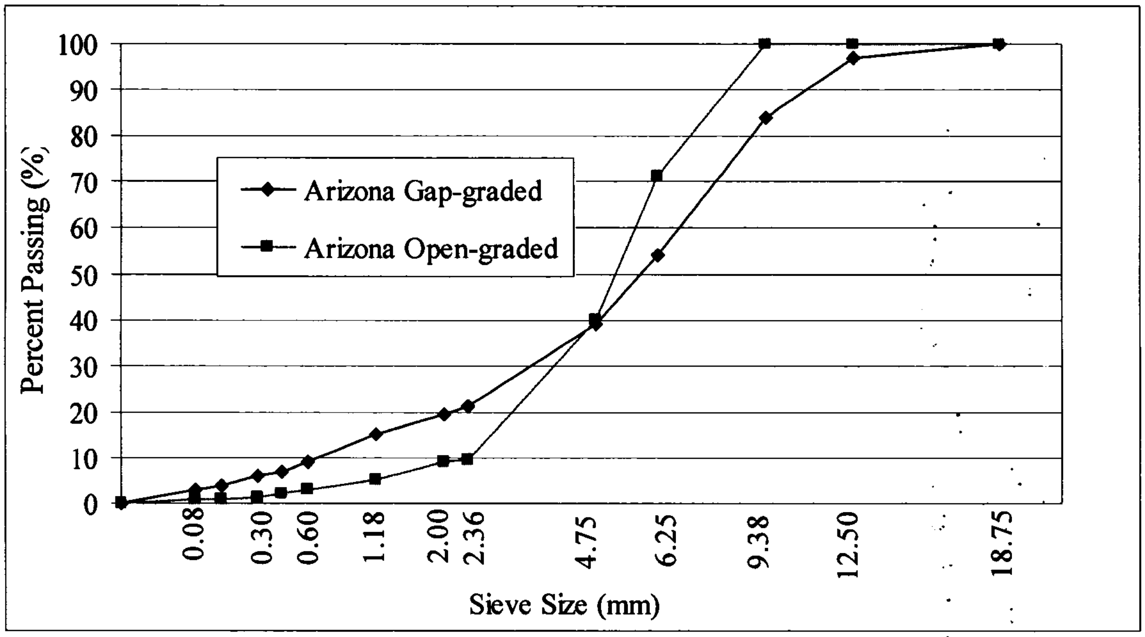

Figure 2.1 Arizona DOT aggregate gradation for AR projects

During 1990 and 2000, over 2500 miles of ARM overlays were placed in Arizona. In 2000, a study was conducted to investigate the performance of ARM overlays placed in Arizona between 1988 and 1992. This study included 14 projects consisting of open-graded and gap-graded overlays placed over Portland Cement Concrete (PCC) and flexible pavements. These pavements were located in diverse climatic conditions ranging from cold and wet to hot and dry regions and under both heavy and low traffic conditions. The overall observations from the performance study are summarized below (Morris and Carlson, 1992):

1. All the open-graded overlays placed over PCC and flexible pavements have, in general, performed excellently ten years after the construction under both hot and cold climatic conditions, as well as both heavy and light traffic conditions.

2. In a specific project, where open-graded and gap-graded overlays were placed together over PCC pavement, the pavement, after 10 years of service in severe climatic conditions (low temp of -34°C and high up to +32°C) and under heavy traffic condition performed excellently.
3. The performance of gap-graded overlays over flexible pavements is rated between good and excellent, with a percent of cracking measurement 10 years after construction being less than the values before overlay construction in all projects. It is to be noted that the gap-graded overlays are located in diverse climatic conditions ranging from hot and dry to cold and wet regions.
4. Almost 10 years after construction, in all fourteen projects (both open-graded and gap-graded) a significant reduction in maintenance cost is noticed, compared to the values before overlay construction.
5. No bleeding or flushing has occurred in any of the projects.

Other than the field performance evaluation, ADOT has also done laboratory studies to characterize the engineering properties of the ARM. An accelerated test using the Model Mobile Load Simulator (MMLS) was conducted to estimate the rutting potential of ARM and conventional asphalt concrete mixes. The rut depths were monitored regularly over a million load repetitions to estimate the rate of rutting of ARM and conventional hot mixes. It was found that the rutting rate of ARM was less than the conventional hot mix for the high load level. For low level loads, the rate of rutting of the conventional mix was less than the ARM. The profile of the rut suggested that in case of asphalt concrete pavement, the rutting was principally through densification, while for ARM it was mostly due to shoving of the surface course (Nourelhuda et al., 2003).

In short, the overall performance of ARM overlays has been outstanding in Arizona based on the long-term performance evaluation.

2.2.2 Texas experience with ARM overlays

Texas has a long history of ARM application in pavement construction and rehabilitation. The first reported application of AR in Texas dates back to 1976. The Texas Department of Transportation (TxDOT) uses AR for four different applications including a) Chip seal, b) SAMI, c) Hot mixes for overlay, and d) Porous Friction Course (PFC) with open gradation (Tahmoressi, 2001).

The experimental sections with ARM for overlays consist of dense-graded and gap-graded gradation. Prior to mid 1992, TxDOT used dense-graded gradation for ARM overlays. Either Type C (15.8 mm) or Type D (12.5 mm max size) dense-graded gradation were used. However, after 1992 the dense gradation was modified into gap-graded gradation (Tahmoressi, 2001). Figure 2.2 shows the different aggregate gradation used in ARM overlay projects in Texas.

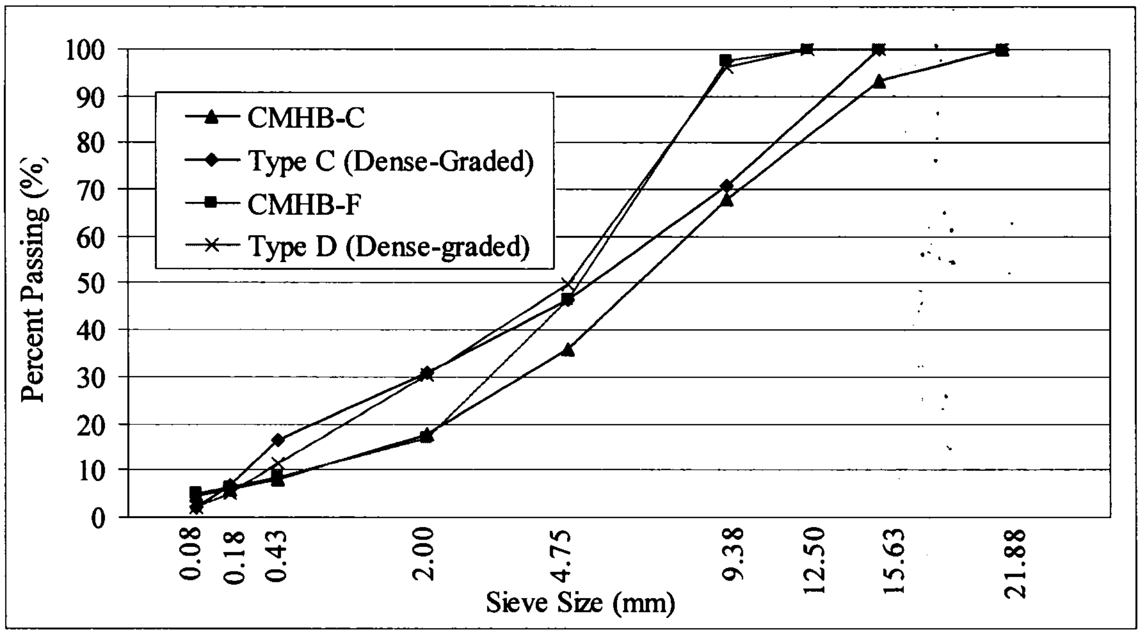

Figure 2.2 Texas DOT aggregate gradation for ARM overlays

The PFC has an open-graded gradation and is similar to the open-graded friction course used in Arizona. In 2001, the performance of the ARM overlay and PFC were evaluated and the age of the evaluated projects varied from one to nine years. The findings from the study are summarized as follows (Tahmoressi, 2001):

1. The dense-graded ARM overlays are performing in good condition nine years after construction. Of the two sections, one project, which used Type C gradation experienced early raveling problems and chip seal was done to protect from further

raveling. However, it is important to note that the binder content in this project was very low at just 4.2%.

2. All ARM projects that utilized CMHB-F gradation are in excellent condition. However, it is important to note that the projects utilizing the CMHB-F gradation were still young at the time of evaluation (1, 3 and 6 years old).
3. With the exception of two projects, all other projects that utilized CMHB-C gradation are in excellent condition. The two projects which showed pre-mature failure revealed base failure to be the reason, rather than the ARM. Resistance to cracking exhibited by the CMHB-C overlay projects is impressive. The ages of the projects are six to eight years old.
4. All PFC projects are exhibiting excellent performance properties. Resistance to cracking and raveling is particularly impressive. From the cost and benefits stand point, PFC represents the best application for ARM.

In addition to the field performance evaluations, laboratory evaluations of the ARM were conducted by TxDOT. Asphalt Aggregate Mixture Analysis System (AAMSA) was used as a tool to evaluate the CR mixtures. The mixtures evaluated included open-graded ARM. Two different rubber contents, 10% and 18% by weight of the asphalt cement, were used and for each rubber content two different sizes of CR (passing #10 and #80 mesh) were used in this study. The major findings from the laboratory evaluation can be summarized as follows (Rebala and Estakhri, 1995):

1. All the ARM had resilient modulus values lower than the control mix (dense-graded mixture with virgin asphalt). This interprets that the thickness required for the open-graded ARM will be higher than required for dense-graded mixtures for a particular traffic level.
2. The fatigue potential evaluated using indirect tensile strain at failure and diametral resilient modulus indicated all ARM, except for ARM with 18% fine CR to be inferior to the control mixture (dense-graded with asphalt cement).
3. The rutting potential evaluated using uni-axial creep data showed all mixtures to have moderate rutting potential. The mixture, with 18% coarse CR, appeared to have the least rutting resistant.

4. The moisture resistance evaluated using the tensile strength and resilient moduli ratio indicated all mixtures to have a ratio of over 0.9 showing adequate moisture resistance. The ARM with fine CR appears to have a better moisture resistance.

2.2.3 California experience with ARM overlays

California Transportation (Caltrans) has extensive experience of ARM application for pavement overlays and is one of the major consumers of scrap tire for pavement maintenance and rehabilitation. In 1995, field reviews of 88 ARM projects in California were conducted. The projects reviewed consisted of dense-graded and gap-graded ARM overlays. The review revealed that thin ARM overlays would give good performance when properly designed and constructed. A few poor performances found in some ARM projects were attributed to poor mix design and/or construction. Similarly, field reviews of ARM projects conducted throughout California in 1999 revealed that out of 113 ARM projects reviewed, over 90% of the projects exhibited little to no distress and are expected to achieve their design life. Two common mixes used in California to produce ARM overlays are gap-graded and dense-graded mix. Figure 2.3 shows the typical dense and gap gradations used in California (Shatnawi and Long, 2000).

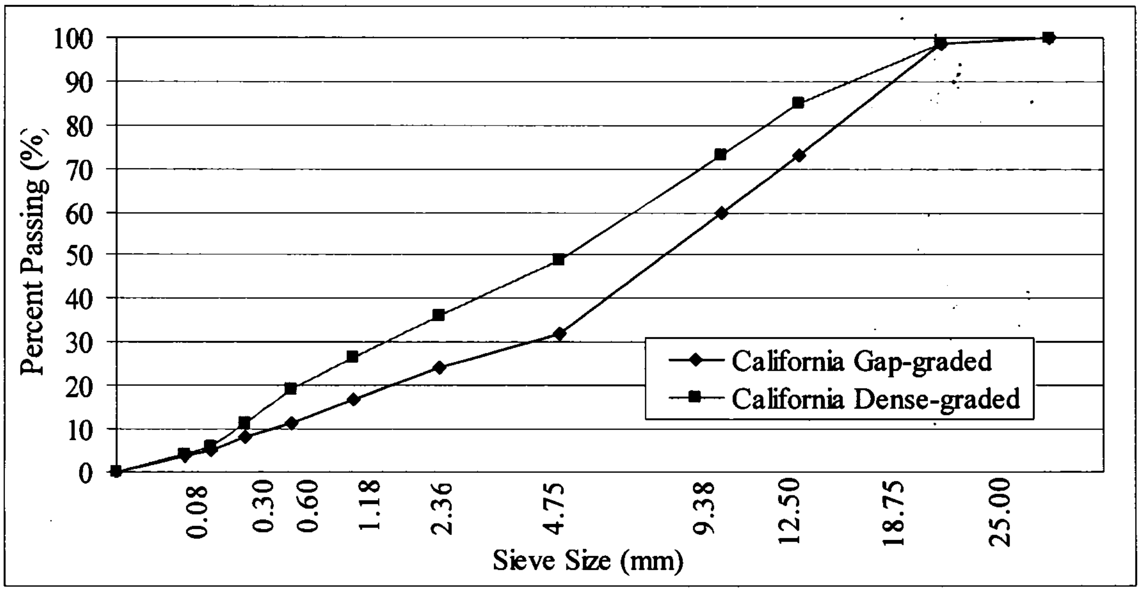

Figure 2.3 Typical Caltrans gap and dense gradation for ARM projects

2.2.3.1 Ravendale Project

The first application of ARM in overlay was done in 1983 in northwestern California approximately about 100 miles north of Reno, Nevada. The annual precipitation in the project location is between 200 and 255 mm. The air temperature exceeds 32°C in the summer and drops below freezing in the winter. The AR consisted of 18% CR, 4% extender oil and 72% asphalt cement. The rubber gradation had 98% passing the 0.6 mm sieve and 8.0% passing the 0.15 mm sieve. The overlay constructed using AR as binder consisted of dense-graded aggregate gradation (Shatnawi and Long, 2000). Table 2.1 shows the aggregate grading used in the ARM Dense-Graded Asphalt Concrete (DGAC) overlays with and without SAMI at this project location (Doty, 1988).

Table 2.1 Aggregate grading specification for Ravedale project (percent passing)

Sieve Size (mm)	ARM (DGAC)	Conventional DGAC
25.00	-	100
19.00	100	95-100
12.50	95-100	-
9.50	80-85	65-80
4.75	55-65	46-56
2.36	38-48	33-43
0.60	18-28	14-14
0.075	3-8	3-8

The distress survey conducted four years after the construction revealed that the ARM overlay performed relatively better compared to the conventional dense-graded overlays and plus ride overlays (overlays constructed using dry process) (Doty, 1988). In 1993, lab testing was conducted on the field cores for the reflective cracking testing. Lab results indicated ARM overlays to perform better that the conventional dense-graded overlays from reflective cracking perspective (Shatnawi and Long, 2000).

The ARM overlay sections with SAMI failed their intended design life of 10 years and experienced over 510,000 Equivalent Single Axle Loads (ESAL) before failure. The other ARM overlay without SAMI failed after 450,000 ESAL applications. It is to be noted that the thickness ratio (thickness provided/thickness required) was 0.45 and 0.30 for ARM overlay with SAMI and 0.6 for ARM overlay without SAMI. The conventional DGAC overlays failed well before experiencing 510,000 ESAL except for one conventional overlay where the thickness ratio was 1.25 (Shatnawi and Long, 2000).

2.2.3.2 Newberry Springs SPS-5 Project

The results from Caltrans field sections and laboratory performance tests in Newberry Springs SPS-5 projects have revealed that gap-graded ARM overlay could provide adequate rutting resistance when properly constructed and designed (Shatnawi and Long, 2000).

Laboratory tests on the cores from the gap-graded ARM overlay and conventional dense-graded overlay for rutting resistance reveled that the conventional dense-graded mix has more resistance to permanent resistance compared to gap-graded ARM mix. However, this does not mean that the gap-graded ARM overlays are prone to rutting. The measured rut depth of the test sections five years after construction was less than 2 mm for both conventional dense-graded and gap-graded ARM overlay.

2.2.3.3 Laboratory tests

Additional laboratory tests were conducted to justify the application of ARM use in California. Caltrans utilized the South African Heavy Vehicle Simulator (HVS) to perform the accelerated pavement testing on the conventional dense-graded asphalt mixture and gap-graded ARM overlay sections. The HVS test results indicated that a reduction of at least 50% in layer thickness to obtain similar fatigue performance over flexible pavements can be justified when conventional dense-graded asphalt mixture is replaced with gap-graded ARM. Similarly, controlled-strain fatigue tests conducted at the University of California indicated that the gap-graded ARM would provide considerably higher fatigue life when compared to the same thickness of conventional dense-graded asphalt mixtures (Shatnawi and Long, 2000).

Reflective cracking tests were conducted on three laboratory mixes (gap-graded ARM, dense-graded ARM and conventional dense-graded asphalt mixtures) at 20°C to assess the reflective cracking resistance of the mixes. The findings of the tests showed that in terms of the reflective cracking gap-graded ARM ranked first, dense-graded ARM ranked second and conventional dense-graded asphalt mixture ranked third. Research conducted at the University of California showed that ARM would have superior thermal cracking resistance when compared to the conventional dense-graded asphalt mixes. The tests conducted included the Thermal Stress Restrained Specimen Test (TSRST) and thermal fatigue using the flexural fatigue tests at low frequencies. Moisture sensitivity tests using AASHTO T283 conducted by Caltrans on gap-graded ARM, dense-graded ARM and conventional dense-graded asphalt mixtures containing the same aggregate type, showed a lower TSR for gap-graded ARM and dense-graded ARM compared to conventional dense-graded asphalt mixes. Field performance indicated that moisture damage occurred early in the service life of some ARM pavement projects (Shatnawi and Long, 2000). Similarly, the moisture sensitivity testing using Environmental Conditioning System (ECS) on conventional dense-graded asphalt mixture and gap-graded ARM was conducted to assess the resistance to water damage in these mixes. The binder content of 4.5% and 5.0% was used for conventional mix and 7.0 and 8.0% for the gap-graded ARM. The results of the test indicated a low resistance to water damage in the conventional mix but not in the gap-graded ARM (Doty, 1988).

2.2.4 Minnesota Department of Transportation

In 1984 the Minnesota Department of Transportation (MnDOT) paved four test sections using dense-graded ARM overlays and four control sections using conventional dense-graded overlays. The thickness of the test and control sections varied from 25.4 mm (1") to 63.5 mm (2.5"). Field performance evaluation and lab tests on the field cores were conducted to evaluate the performance of dense-graded ARM overlays in the cold climatic condition of Minnesota. The lab test on the cores from the test and control sections were conducted to measure the stiffness and in place air voids. The results from the lab tests indicated that the dense-graded ARM overlay, in general, had less stiffness compared to the conventional dense-graded overlay. Similarly, the in place air voids in the test sections were, in general, higher compared to the control sections (Trgeon, 1991).

The field performances were evaluated in terms of rutting, International Roughness Index (IRI) measurements and transverse and longitudinal cracking measurements. The test and control sections consisting of 38 mm (1.5") overlay were severely affected by transverse and longitudinal cracking occurrence five years after construction and chip seal was applied in these sections in 1991. The other test section showed excellent performance in the first few years after construction from cracking perspective, but were moderately affected by cracking occurrence five years after construction. The rutting and IRI measurements conducted in 1991 revealed that both test sections and control sections were performing well from the rutting perspective, as the rutting in all the sections was below the acceptable value. The IRI measurements indicated that the performance of both ARM overlays and conventional overlays were comparable with no sections outperforming the other (Trgeon, 1991).

In short, the formulation of the used ARM provided little or no perceived benefits to the roadways at much higher costs.

2.2.5 Alaska Department of Transportation and Public Facilities (AKDOT & PF)

The first application of rubberized overlays using the wet process was first applied at Fairbanks, Alaska, in 1988. The binder of ARM overlay consisted of 18% CR and 72% AC-2.5 asphalt cement. The pavement structure underneath the overlay was exactly the same for both ARM and HMA overlays. The aggregate gradation for both ARM and HMA was also the same and is shown in Figure 2.4. The optimum binder content for ARM and HMA overlays were 6.4% and 5.3% respectively, by the weight of the total mix (Saboundjian and Radd, 1997).

In 1996, the field evaluations and lab tests were conducted on the HMA and ARM sections to evaluate the performance of ARM in Alaska weather condition. The field observations showed that ARM sections outperformed the HMA sections from a cracking perspective with transverse crack spacing of 37 m in ARM sections compared to 6 m in HMA control sections. The rutting resistance of HMA control section was found to be better than the ARM section with a rutting value of 1.1 mm compared to 3.5 mm in ARM sections (Saboundjian and Radd, 1997).

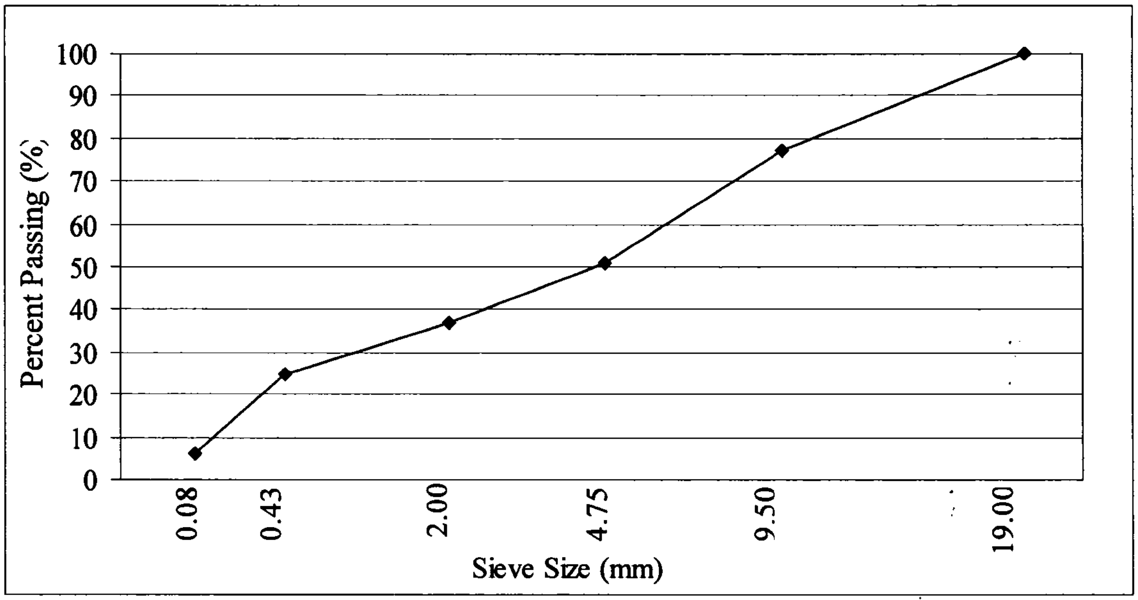

Figure 2.4 Alaska DOT aggregate gradation for ARM projects

In addition to the field performance evaluation, lab tests on the ARM and HMA specimens were conducted for fatigue, thermal cracking and rutting resistance. Controlled-strain flexural fatigue test indicated that at 22°C the fatigue resistance of ARM overlay was better than that of conventional HMA. The fatigue life calculation, using the analysis results from ELSYM5 and laboratory developed relationships, indicated the ARM overlay to live three times longer than the HMA overlay. As low temperature cracking is a dominant distress in Alaska, the thermal cracking test (Thermal Stress Restrained Specimen Test) was conducted on the ARM and HMA specimens. The test results indicated that ARM overlay exhibit a significant improvement in low temperature compared with the conventional mixes. These results were consistent with the field performance. The permanent deformation resistance of the ARM and conventional mixes were also evaluated in the lab using the Georgia Loaded Wheel Tester (GLWT). The test results indicated that the HMA mix rut depth of 3.26 mm outperformed the ARM with a rut depth of 5.07 mm after 8,000 cycles (Saboundjian and Radd, 1997).

2.2.6 Florida experiences with ARM overlays

Between 1989 and 1990, a total of three test projects, two open-graded and one using dense-graded friction course mixtures, were constructed in Florida with an objective of evaluating the long-term field performance of the ARM for the Florida condition. In each project location pavement test sections using ARM (wet process) were paved along with a control section using conventional asphalt binder. The CR percentage used for different pavement test sections varied from 3 to 17%. One project location also consisted of a test section where rubber was added using dry process (Choubane et al., 1998).

The performance evaluation of the pavement sections was conducted over a period of ten years. The performance was judged based on the amount of distresses, rideability, skid resistance, rutting, cracked area and patching. A major finding of the performance evaluation over a period of ten years was that the wet process addition of rubber improved the crack resistance of the surface mixtures. The findings of the performance evaluation can be summarized as follows (Choubane et al., 1998, and Smith et al., 2000):

- The addition of the rubber has no observable beneficial or detrimental effect on field performance.
- Less rutting was measured in the dense-graded mix when rubber was added.
- The wet process addition of rubber significantly improved the cracking resistance of the open-graded friction course mixes. Test sections with ARM resulted in approximately one to six percent cracked areas, depending on the amount of rubber, while those with asphalt cement or dry-mixed ARM showed about thirty percent cracked areas.
- The wet process addition of rubber, of up to fifteen percent, resulted in higher ride ratings on open-graded surface mixes as determined using Mays ride meter.

Based on the initial findings of these test projects, the Florida Department of Transportation initiated the implementation of specifications requiring the use of CR in all asphalt surface mixes. These specifications define the ground tire rubber, the blending process and the amount of CR (Choubane et al., 1998).

2.2.7 Ontario experience of ARM overlays

Between 1990 and 1992 eleven demonstration projects were paved in Ontario to assess the performance of CR modified overlays. Although the demonstration projects primarily focused on assessing the performance of CR application using dry process, one project utilized AR containing 7% CR (by the weight of asphalt) produced by wet process as binder in the ARM. The performance evaluation conducted three years after the construction revealed ARM sections to be in excellent condition compared to control sections and the other projects constructed using dry process of CR addition. The ARM sections showed 50% less transverse cracking compared to the control sections (Emery J., 1995).

2.2.8 Virginia experience with ARM overlays

In September 1993, a four lane, 7.2 km section road was paved with ARM in Virginia. The test sections included dense-graded and gap-graded ARM with and without SAMI application. The control section was a dense-graded mix (same gradation as dense-graded ARM) with conventional AC-30 binder. Table 2.2 shows the aggregate gradation for dense-graded and gap-graded mixes. The AR binder contained 18% of minus 2 mm CR and the design binder content for control section, dense-graded and gap-graded ARM section were 5.4%, 6.5% and 7.8% respectively (Maupin, 1996).

Table 2.2 Mix design gradation bands for Virginia mix

Sieve Opening (mm)	Percent Passing	
	Dense-graded	Gap-graded
19.0	100	100
12.5	97-100	95-100
9.5	82-94	79-87
4.75	48-62	32-40
0.6	18-24	9-12
0.075	4-7	2-6

After eight months, samples were sawed from the pavement and examined visually. Moisture was detected in the gap-graded rubber mix and at the interface immediately beneath the mix. No moisture was detected in the dense-graded mix there by suggesting gap-graded mix might be more susceptible to moisture damage. Some transverse cracks have reflected through all the sections. Based on the short-term performance evaluation, none of the ARM sections show superior performance to control section. The use of rubber added approximately 50 to 100% to the cost of conventional mixes and it appeared unlikely to justify the use of the ARM overlays in Virginia (Maupin, 1996).

2.2.9 Nevada experience with ARM overlays

During the early 1990s the Nevada Department of Transportation (NDOT) paved several ARM overlays in four project locations to evaluate the performance of the overlays in Nevada using the aggregates from Nevada sources. The experiment consisted of overlays laid at reduced thickness and design thickness to evaluate if using ARM overlays permits a reduction of the overlay thickness ratio of upto 50% as reported in California. The performances of the overlays were monitored for ten years and were judged based on the roughness, rutting, cracking and rutting. Based on the long-term performance of the ARM overlays, it was concluded that the reduction of the ARM overlay thickness resulted in a very unsatisfactory performance and required ARM overlay thickness comparable to the conventional HMA thickness for satisfactory performance. In summary, the NDOT's experience suggested the ARM overlays to be effective in Nevada condition must be constructed similar to the conventional HMA with a minimum thickness of 50 mm and 19 mm open-graded course. This requirement, which resulted in no additional benefits compared to conventional HMA, made ARM overlays too expensive to be considered as a rehabilitation alternative in Nevada's condition (Sebally et al., 2003).

2.3 Summary from the ARM overlay experiences

Based on the literature review of the studies conducted on the performance of ARM overlays, the following observations were notable from the view point of the scope of this thesis:

- The experience of ARM overlays has been mixed with some states claiming high benefits of using ARM as rehabilitation alternative, while others could not see any perceived benefits to justify the additional costs.
- Both gap-graded and dense-graded aggregate gradation has been used for ARM overlays and has been reported as successful projects in Texas, California and Alaska. The open-graded gradation, primarily used as friction course, has also performed excellently in the states reporting benefits of using ARM overlays.
- There has not been an aggregate gradation accepted as the best for use with ARM overlays. There exist variations in the aggregate gradation even among the states that have reported success with ARM overlays.
- Most of the lab performance studies conducted showed the reflective cracking, rutting and thermal cracking resistance of ARM to be adequate.
- Moisture related distresses (raveling) have been reported to be an early distress prevalent in the ARM overlays. The study in Virginia has reported that gap-graded gradation can trap moisture compared to dense gradation, thereby making gap gradation more prone to moisture damage in the field.

CHAPTER 3 PERFORMANCE EVALUATION OF 2002 and 2003 ASPHALT RUBBER PROJECTS IN ALBERTA

3.1 Introduction

Tire Recycling Alberta (TRA) and Alberta Infrastructure and Transportation (AI&T) initiated a project to study the feasibility and performance of ARM overlays in Alberta. During the summer of 2002, AI&T, the City of Edmonton, the City of Calgary and the County of Strathcona each paved pavement sections using ARM. At each project location, two ARM pavement sections with two different thicknesses, and at least one control section using conventional HMA or other asphalt mixture types, such as Shingle Asphalt Mixture (SAM), were used. Generally, the ARM reduced thickness sections were half as thick as the full-depth ARM sections at each project location. Continuing with the 2002 trial sections, 10 more pavement sections were laid in the summer of 2003. The City of Edmonton paved four pavement sections at four locations compared to paving one pavement section in 2002. Strathcona County paved three pavement locations with ARM, a total of 12.6 km, compared to one pavement section of 1.6 km in 2002. In 2003, AI&T paved two pavement sections totaling 32 km with ARM. For the first time, an 18.4 km pavement section was repaired with AR chip seal. The City of Lethbridge joined this pilot project in 2003 and paved one 0.55 km pavement section with ARM. Finally, the City of Calgary, which paved one 1.7 km pavement section in 2002, did not participate in the 2003 project.

For both the 2002 and 2003 project, EBA Engineering Consultants Ltd. was responsible for the technical coordination and testing of this project, the University of Alberta performed data analysis and performance evaluation for the project, and aci Acoustical Consultants Inc. performed sound measurements to evaluate the noise reduction of the ARM sections.

This chapter provides a summary of design, performance evaluation and the field observations for the 2002 and 2003 Alberta Asphalt Rubber project. It is premature to come to a conclusion about the performance of ARM in Alberta's conditions based on the short-term performance, however, the short-term performance evaluation will provide an indication of where the ARM pavement performance is heading towards.

Table 3.1 provides a summary of the 2002 and 2003 project locations.

Table 3.1 Summary of the Alberta Asphalt Rubber project

Project Location	Project year	Agency	Estimated Lane-Km of Roadway Paved	ARM Produced (tons)	CR Used (tons)	Passenger Tires Equivalent
17 Street/ Baseline Road	2002	Strathcona County	1.6	742	18.7	3,117
137 Avenue	2002	City of Edmonton	3.2	2,771	44.7	7,450
Highway 630:02	2002	AI&T	2.0	1,368	19.8	3,300
112 Avenue NW	2002	City of Calgary	1.7	935	18.6	3,117
50 Street	2003	City of Edmonton	156.1	11,666	156.1	26,010
111 Avenue	2003	City of Edmonton	31.8	2,850	31.8	5,297
102 Avenue	2003	City of Edmonton	10.8	888	10.8	1,792
Stony Plain Road	2003	City of Edmonton	33.9	2,860	33.9	5,652
Baseline Road*	2003	Strathcona County	50.9	4,307	50.9	8,488
Baseline Road**	2003	Strathcona County	10.8	432	10.8	1,792
Kaska Road	2003	Strathcona County	9.4	729	9.4	1,568
Highway 623:04	2003	AI&T	86.9	5,500	86.9	14,477
Highway 507:04	2003	AI&T	31.1	191***	31.1	5,188
4 Avenue	2003	AI&T	11.9	815	11.9	1,983
Total				35,863	535.4	89,231

** Chelsea Way to Highway 21 ** W of Broadmoor Blvd to Broadview Dr ***No ARM produced*

3.2 General information for 2002 and 2003 Asphalt Rubber projects in Alberta

3.2.1 17 Street /Baseline Road

The project on 17 Street in Sherwood Park involves a four-lane (two-lane North Bound [NB] and two-lane South Bound [SB]), semi-urban roadway with high truck traffic loading. A portion of pavement, near a busy intersection, makes it an ideal section for ARM rutting resistance evaluation. The project limit at this location starts at km 0.135, ends at km 1.004, and includes sections in both directions of 17 Street. In the NB direction, a test section running 120 m to the southern project limit at km 0.135 was paved with 60 mm of ARM pavement starting at km 0.255. In the SB direction, a 150 m stretch of test section was paved with 60 mm of ARM just south of 105 Avenue. Immediately south of this test section, a 231 m long section was paved with 100 mm of Polymer Modified Asphalt (PMA). This PMA section is followed by another test section, which ends at Baseline Road (measured south to north) and was paved with 60 mm of ARM covering 40 mm of PMA. Immediately south of this Baseline Road intersection, another ARM test section, stretching 243 m, was paved with 30 mm of ARM on top of 40 mm of PMA. A rail track intersects the project sections diagonally, just north of Baseline Road. The 130 m long left turn lane east of 17 Street on Baseline Road (with urban cross was paved as another test section with 60 mm ARM on top of 40 mm of PMA. The layout of the pavement section at this project location is presented in Appendix A, Figure A-1 and A-2.

3.2.2 137 Avenue

The test sections 137 Avenue in Edmonton stretch between 113A Street and 127 Street. It is a divided arterial roadway, running east-west, with two lanes of urban cross-section (curb and gutter) in each direction. For the 137 Avenue project, both EB lanes are ARM test sections. The WB lanes are control sections paved with 100 mm of conventional asphalt mixtures. The project limits are km 0.000 and km 1.600 on 137 Avenue, respectively, as shown in Appendix A, Figure A-3. At the east end of the project, bordering 127 Street, a test section extending 800 m to the east and including both EB lanes was paved with 50 mm of ARM. The second test section, 550 m in length and paved with 100 mm of ARM, is located immediately east of this section. This 100 mm

ARM section ends at km 1.350. Another 250 m long section of 50 mm ARM was paved to the western project border at 113A Street. In addition to the main lanes, several turning lanes along 137 Avenue were also paved with ARM.

3.2.3 Highway 630:02

Highway 630:02 is a two-lane rural highway. The two test sections on Highway 630:02 span from east of North Cooking Lake to west of Range Road 205. The ARM project starts at km 27.700 on Highway 630:02. The first ARM test section, which stretches 500 m toward the southeast from the project limit, was paved with 80 mm of ARM and was constructed in two lifts in both lanes. Immediately east of this section, another test section was paved with a single lift 40 mm of ARM in both lanes. The east end of the second ARM test section marks the ARM project limit, located at km 28.700 on Highway 630:02. At the eastern limit of the ARM project, the ARM abutted a conventional asphalt overlay, which was placed in 2000. The control section for this project consisted of two thicknesses. One control section for this project, stretching west to the project limit at km 27.700, was paved for 500 m with 40 mm of conventional asphalt mixture. The other control section, located west of the 40 mm HMA section, was paved for 500 m with 80 mm of conventional asphalt mixture.

The layout of the pavement section at this project location is presented in Appendix A, Figure A-4.

3.2.4 112 Avenue NW

112 Avenue NW in Calgary is a two-lane road running east and west between 69 Street and 85 Street NW. The first test section, starting at the intersection of 69 Street and 112 Avenue and stretching approximately 425 m west, was paved with 75 mm of ARM. Abutted to the west is another 425 m long test section paved with 40 mm of ARM. The control section at this project location is a 75 mm thick pavement with Shingle Asphalt Mixture (SAM) located immediately west of the 40 mm ARM test section. The control section is approximately 830 m in length. The layout of the pavement section at this project location is presented in Appendix A, Figure A-5.

3.2.5 50 Street

50 Street in Edmonton is a two-way, two-lane road of which a 5.2 km section was paved with ARM in 2003. This overlaid section stretches from 13 Avenue NW to 41 Avenue SW (Southern city limit of Edmonton) and is intersected by Ellerslie Road. The layout of the pavement section at this project location is presented in Appendix A, Figure A-6. The thickness of the ARM overlay at this project location was 100 mm, laid after 50 mm milling of old Asphalt Concrete Mix (ACM).

3.2.6 111 Avenue

111 Avenue in Edmonton is a two-direction urban collector roadway with three lanes in each direction. The ARM section at this project location stretches from 124 Street to East of Groat Road and is 0.950 km long. The presence of four signalized intersections along this section makes this project location appropriate to study the performance of ARM overlay at intersections. The old asphalt pavement at this road was milled 50 mm and overlaid by 75 mm ARM in both directions. The layout of the pavement section at this project location is presented in Appendix A, Figure A-7.

3.2.7 102 Avenue

The section paved in the 102 Avenue in Edmonton stretches from 111 Street to 116 Street and is 0.6 km in length. This two-lane, two-way road is a local residential road and is the first ARM overlay paved in a residential area. The old asphalt pavement at this project location was milled to 50 mm and overlaid with 75 mm ARM in both lanes. The performance data for this project location was not available. As a result this project location has been excluded from further analysis in this thesis.

3.2.8 Stony Plain Road

The section paved on Stony Plain Road in Edmonton is a two-lane, two-way road with several controlled intersections and traffic islands. The ARM section stretches from 156 Street to 166 Street and is 1.1 km in length. Seventy-five mm ARM overlay was paved in both directions and a 50 mm thickness milling was performed before the overlay. It should be noted that there is an Edmonton Transit System (ETS) transit centre located close to the east limit (156 Street) of this project location. The layout of the pavement section at this project location is presented in Appendix A, Figure A-8.

3.2.9 Baseline Road (Chelsea Way to Highway 21)

A section in Baseline Road stretching from Chelsea Way to Highway 21 (1.6 km) in Strathcona County, was one of the three ARM sections paved in 2003. Baseline Road is a divided urban collector road with three lanes in each direction. The thickness of the ARM overlay at this project location was 40 mm, laid after 40 mm milling of old ACM. The layout of the pavement section at this project location is presented in Appendix A, Figure A-9.

3.2.10 Baseline Road (Broadmoor Blvd/Broadview Drive)-Strathcona County

The WB direction of Baseline Road, stretching from Broadmoor Blvd. to Broadview Drive (0.250 km) in Strathcona County, was another ARM project in Strathcona County paved in 2003. The thickness of the ARM overlay at this project location was 40 mm, laid after 40 mm milling of old ACM. The performance data for this project location was not available. As a result, this project location has been excluded from further analysis in this thesis.

3.2.11 Kaska Road-Strathcona County

The section paved in Kaska Road is a two-way, two-lane local urban road stretching from Sioux Road to Chippewa Road (0.54 km) in Strathcona County. The overlay thickness at this project location is 40 mm ARM, laid after 40 mm milling of the old ACM. The layout of the pavement section at this project location is presented in Appendix A, Figure A-10.

3.2.12 Highway 623: 04

Highway 623:04 is a two-lane, two-way rural highway and is the only project location where both conventional asphalt mixture and ARM were paved in 2003. A total of 21.5 km of pavement section was paved, which consisted of two different thickness of ARM (total of 7 km) and 3 different thicknesses of ACM (total of 14.5 km) sections. Table 3.2

presents the thickness and location of the ARM and ACM overlays. The layout of the pavement section at this project location is presented in Appendix A, Figure A-11.

Table 3.2 ARM thickness for Highway 623:04 project

Begin (km)	End (km)	Overlay Type	Overlay Thickness (mm)
0.000	1.800	ACM	40 over 20 mm ACM leveling course
1.800	2.070	ACM	60 over 60 mm ACM leveling course
2.070	3.500	ACM	50 over 20 mm ACM leveling course
3.500	5.000	ARM	60 over 10 mm ACM leveling course
5.000	9.500	ARM	40 over 10 mm ACM leveling course
9.500	10.500	ARM	40
10.500	14.000	ACM	50 over 20 mm ACM leveling course
14.000	18.500	ACM	40 over 20 mm ACM leveling course
18.500	21.500	ACM	50 over 30 mm ACM leveling course

3.2.13 Highway 507: 04

Highway 507:04 is the only project location where AR chip seal was applied. This 10 km pavement section is located east of Highway 6 at Pincher Creek, Alberta. Since this thesis focuses on the performance of ARM overlay, this project location has been excluded from further analysis in this thesis.

3.2.14 4 Avenue (Lethbridge)

The 4 Avenue project is the only ARM pavement section paved in the City of Lethbridge. This project limits for this project are 10 Street E to 13 Street E and the pavement section is 0.550 km in length. The thickness of the ARM overlay at this project location was 60 mm which was applied after 60 mm milling. The performance data for this project location was not available. As a result, this has been excluded from further analysis in this thesis.

3.3 Mix design and materials used in Alberta Asphalt Rubber project

3.3.1 2002 project

Three project locations including 17 Street, 137 Avenue and Highway 630:02 of 2002 Asphalt Rubber project in Alberta utilized the same binder, aggregate gradation and mix design. However, the 112 Avenue project location in Calgary utilized a different aggregate gradation and mix design. In the subsequent paragraphs the aggregate gradation and mix design used in Calgary has been referred to as Calgary gradation and Calgary mix design respectively. The aggregate gradation and mix design for other 2002 project location have been referred to as 2002 gradation and 2002 mix design respectively. The AR binder used was the same for all projects and has been referred to, in this study, as 2002 AR binder.

3.3.1.1 2002 AR binder

The asphalt cement used for both conventional and rubber asphalt mixtures was 150/200A penetration grade asphalt cement supplied by Husky Energy. The CR used for AR was a 2.0 mm maximum size scrap tire crumb produced by Alberta Environmental Rubber. The proportions of base asphalt cement and CR were 81% asphalt cement and 19% CR (by weight of total binder). The gradation of the CR, in accordance with Arizona Department of Transportation's (ADOT) specifications, is shown in Table 3.3 and Figure 3.1.

The asphalt cement was heated to 204°C before the CR was slowly added to the hot asphalt cement. The results of various tests on binder are presented in Table 3.4. Note that for the 24-hour reaction period, the binder was maintained at 135°C overnight and then reheated to 177°C.

Table 3.3 CR Gradation (2002)

Sieve Size (mm)	Percent Passing	
	Result	Specified Limits (ADOT Specification)
2.36	100	-
2.00	99.2	100
1.18	30.1	65-100
0.60	0.5	20-100
0.30	0.2	0-45
0.075	0.0	0-5

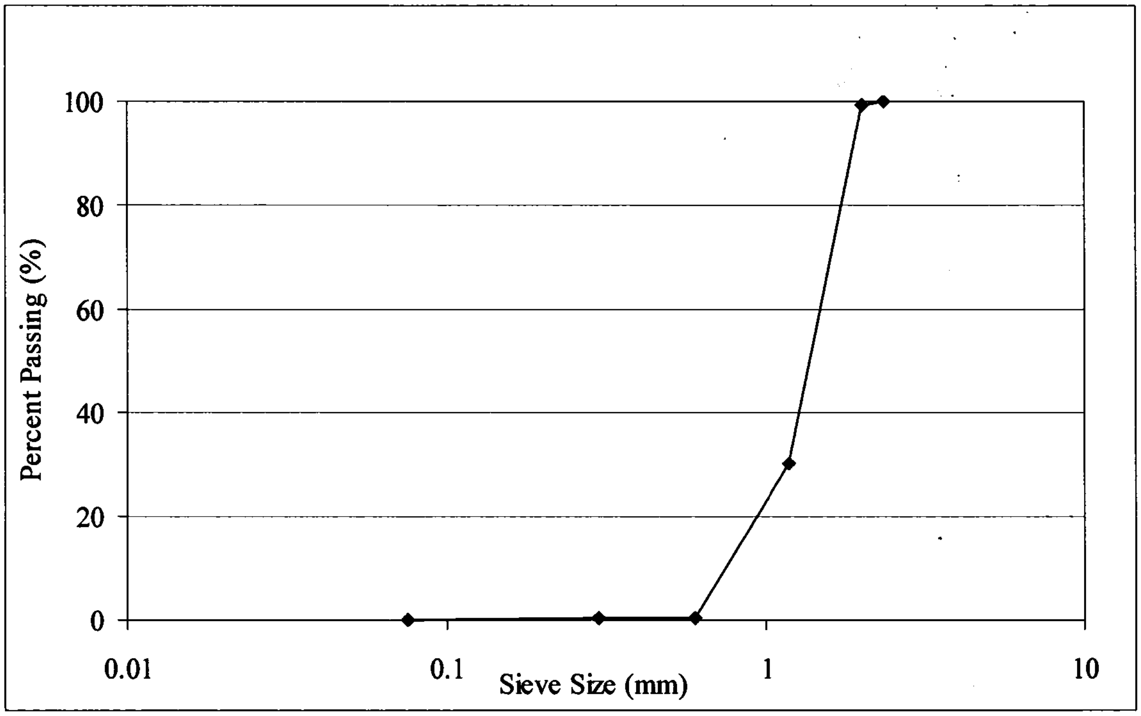

Figure 3.1 Gradation of CR (2002)

Table 3.4 Laboratory test results for CR (2002)

Test Performed	Reaction time (minutes)					ADOT Specified Limits
	60	90	240	360	1440	
Viscosity, Haake at 177°C, cP	2200	2700	3200	3500	2900	1500-4000
Resilience at 25°C, % Rebound (ASTM D5329)	41	-	49	-	45	$\geq$25
Ring & Ball Softening Point, °C (ASTM D36)	62.78	63.33	64.44	67.5	65.83	$\geq$130
Needle Penetration at 4°C, 200g, 60 sec., 1/10 mm (ASTM D5)	30	-	32	-	37	$\geq$15

3.3.1.2 2002 aggregate gradation and mix design

The aggregates used in this mix design included four stockpiles of mineral aggregate supplied from the Stollery Pit and Onaway Pit of Edmonton. Figure 3.2 illustrates the aggregate distribution of the design blend. The mix design for this asphalt gradation and binder reported an optimum binder content of 8.9%. Table 3.5 provides the ARM properties at optimum binder content. In addition, the aggregate component gradations and the design blend gradations are also tabulated in Table 3.6. The moisture sensitivity test was also conducted at the optimum binder content using the AASHTO T283 procedure. The results of the test indicated a TSR value of 0.75.

Figure 3.2 Aggregate gradation for ARM design (2002 projects)

Table 3.5: Properties of ARM at recommended asphalt content (2002)

Property	Value	ADOT Specification Requirement
Asphalt Content (%, by total weight)	8.9	-
Bulk Specific Gravity	2.178	-
Stability (kN)	10.8	-
Flow (0.25 mm Units)	19	-
Voids in Mineral Aggregate (%)	23.40	≥19
Air Voids (%)	5.5	5.5±1.0
Voids Filled with Asphalt (%)	76.4	-

Table 3.6: Aggregate gradations for ARM design (2002 projects)

Sieve Size (mm)	Percent Passing				
	12.5 mm Coarse Aggregate	5 mm Buckshot	Washed Manufactured Fines	Manufactured Fines	Design Blend
19	100	100	100	100	100
12.5	90	100	100	100	94
9.5	57	100	100	100	74
6.3	19	96	100	100	51
4.75	6	66	93	94	36
2.36	3	6	58	65	15
2.00	2	4	52	59	13
1.18	2	3	40	48	11
0.600	2	2	30	39	9
0.425	2	2	24	34	7
0.300	2	2	17	27	6
0.150	2	1	5	16	4
0.075	1.2	0.7	1.0	8.8	1.8
Proportion	60%	20%	10%	10%	100%

3.3.1.3 Calgary aggregate gradation and mix design

The aggregate gradation was achieved by blending 60% coarse aggregate (12.5 mm nominal) and 40% fine aggregate, as supplied by the City of Calgary Manchester Yard asphalt plant facility. Figure 3.3 illustrates the aggregate distribution curve for the design mix. The aggregate component gradations and design blend gradation are tabulated in Table 3.7.

The mix design for this aggregate gradation and AR binder reported an optimum binder content of 7.6% (by total weight of the mix). Table 3.8 provides the ARM properties at optimum binder content.

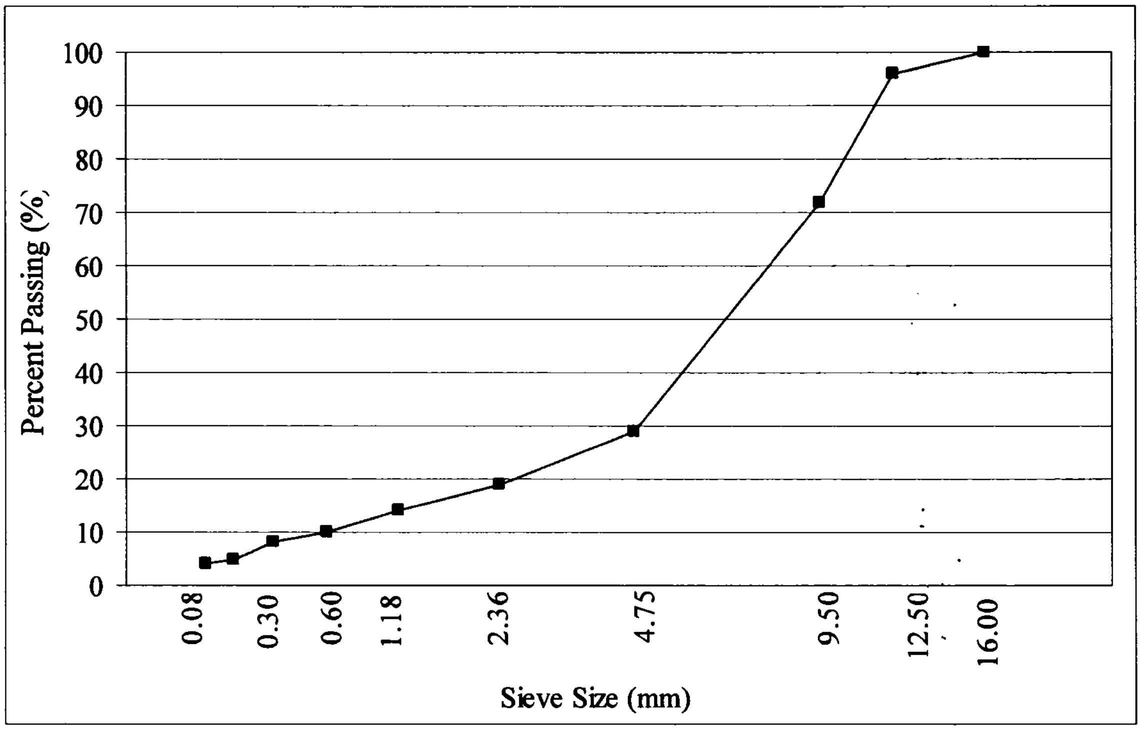

Figure 3.3 Aggregate Gradation for Calgary aggregate gradation

Table 3.7 Aggregate gradations for ARM design (Calgary project)

Sieve Size (mm)	Percent Passing		
	12.5 mm Coarse Aggregate	Fine Aggregate	Design Blend
16	100	100	100
12.5	93	100	96
10	53	100	72
5	4.0	67	29
2.5	2.2	44	19
1.25	1.8	32	14
0.630	1.4	24	10
0.315	1.2	17	8
0.160	1.0	12	5
0.080	0.9	8.9	4.1
Proportion	**60%**	**40%**	**100%**

Table 3.8 Properties of ARM at recommended asphalt content (Calgary project)

Property	Mix Design Value	ADOT Specification Requirement
Asphalt Content (% total weight)	7.6	-
Bulk Specific Gravity	2.247	-
Marshall Stability @ 60°C (kN)	10.5	-
Marshall Flow @ 60°C (0.25 mm)	16.0	-
Voids in Mineral Aggregate (%)	19.8	≥19
Air Voids (%)	5.5	5.5±1.0
Binder Film Thickness (μm)	19.6	-
Voids Filled with Asphalt (%)	72	-

3.3.2 2003 project

In the 2003 Alberta Asphalt Rubber project, all ARM overlay projects utilized the same AR binder, aggregate gradation and mix design. However, the ARM overlay project for Highway 623 utilized a different binder, aggregate gradation and mix design. In the subsequent paragraphs the binder, aggregate gradation and mix design used in Highway 623 have been referred to as Highway 623 binder, gradation and mix design respectively. The binder, aggregate gradation and mix design for other project locations has been referred to as 2003 binder, gradation and mix design respectively.

3.3.2.1 2003 AR binder

A 150-200 Pen. asphalt cement supplied by Imperial Oil, and minus 2 mm CR, supplied by Alberta Rubber Environmental were used for producing the AR binder. The binder consisted of 81.5% asphalt cement and 18.5% CR by total binder. Table 3.9 and Figure 3.4 present the CR gradation used for the 2003 project. Various physical properties of binder were measured and the results are presented in Table 3.10.

Table 3.9 CR gradation (2003 Project)

Sieve Size (mm)	Percent Passing	Specified Limits (ARIZ 714)
2.00	99	100
1.18	28.7	65 - 100
0.60	1	20 - 100
0.30	0.3	0 - 45
0.075	0	0 - 5

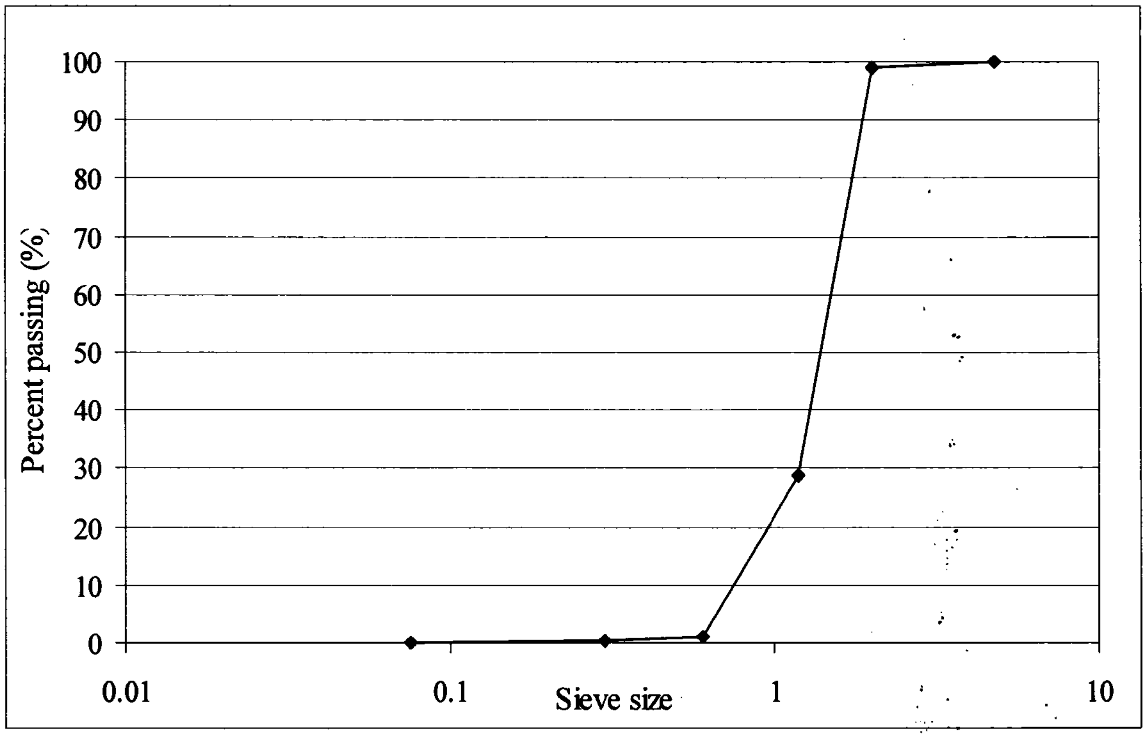

Figure 3.4 Gradation of CR (2003)

Table 3.10 Physical properties of the binder (2003 project)

Tests performed	Reaction time (minutes)					ADOT Specified Limits
	60	90	240	360	1440	
Viscosity, Haake at 177°C, cP	2000	2800	3500	3300	2700	1500-4000
Resilience at 25° C, % Rebound (ASTM D5329)	43	-	51	-	49	≥25
Ring & Ball Softening Point °C (ASTM D36)	59	64	66.67	65	66.67	≥130
Needle Penetration at 4°C, 200 g, 60 sec, 1/10 mm (ASTM D5)	29	-	32	-	33	≥15

Note: The AR was held overnight at 135 °C and heated back to 177°C for the final 24-hour reaction period.

3.3.2.2 2003 aggregate gradation and mix design

Five different aggregates were used to achieve the job mix formula gradation for this project location. The source of the aggregate was Villeneuve Pit 240 at Edmonton, Canada and was supplied by Inland. Table 3.11 shows the proportion and gradation of the five different aggregates used for the ARM design. Figure 3.5 is a power 0.45 chart showing the aggregate gradation of the ARM.

The recommended binder content for the ARM was 6.82% by weight of mix. Properties of the ARM at the optimum design AR content are summarized in Table 3.12. It is to be noted that this value of optimum AR content for the 2003 project is less than 8.9% used in the 2002 project.

A TSR test, based on AASHTO T283 was conducted to assess the moisture susceptibility of the ARM. Test results indicated a TSR value of 0.40. This low value of TSR is alarming compared to a value of 0.75 for the 2002 project.

Table 3.11 Aggregate gradation for ARM design (2003 Project)

Sieve Size (mm)	Aggregates gradation					Blend	ADOT 413-2 Spec.
	#1 Manufactured Fines	#2 Washed Manufactured Fines	#3 Chips	#4 12-10	#5 16-10		
19	100	100	100	100	100	100	100
12.5	100	100	100	100	47	95	80-100
9.5	100	100	100	55	7	67	65-80
6.3	100	100	100	5	3	41	
4.75	98	95	88	4	2	37	28-42
2.38	70	67	7	3	2	15	14-22
2	65	62	4	3	2	14	
1.18	53	51	3	2	1	11	
0.6	43	40	2	2	1	9	
0.425	38	32	1	1	1	7	
0.3	32	24	1	1	1	6	
0.15	20	8	1	1	1	3	
0.075	11.9	1.5	0.6	0.7	0.4	1.6	0-2.5
Proportion (%)	8	10	20	52	10	100	

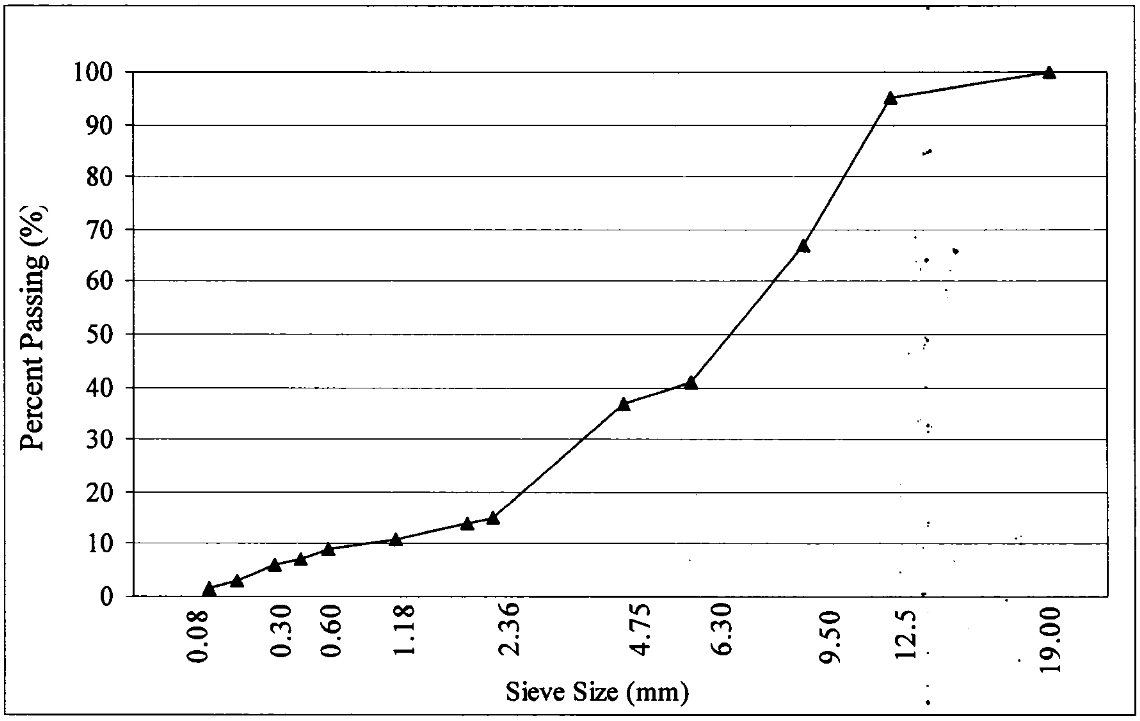

Figure 3.5 Aggregate gradation for the ARM design (2003 project)

Table 3.12 Properties of ARM at recommended asphalt content (2003 project)

ARM Properties	Value	ADOT 413-1 Specification
Percent of AR (by weight of mix)	6.82	
Bulk Specific Gravity	2.237	
Bulk Specific Density (kg/m^3)	2232	
Theoretical Max Specific Gravity (G_{mm})	2.367	
Stability (kN)	12.6	
Flow (0.25 mm)	14	
Air Voids (%)	5.5	4.5-6.5
VMA (%)	19.39	>19
Voids Filled with Asphalt (%)	71.7	
Effective Asphalt (%)	6.53	
Dust to Effective Asphalt Ratio	0.25	
Effective Specific Gravity	2.605	

3.3.2.3 Highway 623 binder

Binder for Highway 623 consisted of 82% Pen Grade 200-300 asphalt cement and 18% CR (by total weight of binder). Table 3.13 and Figure 3.6 show the CR gradation used in the Highway 623 project location.

Table 3.13 CR gradation for Highway 623

Sieve Size (mm)	Percent Passing	Specified Limits (ASTM D5644)
2.00	100	100
1.18	34	20-65
0.60	1	10-40
0.30	0	0-30
0.075	0	0 - 5

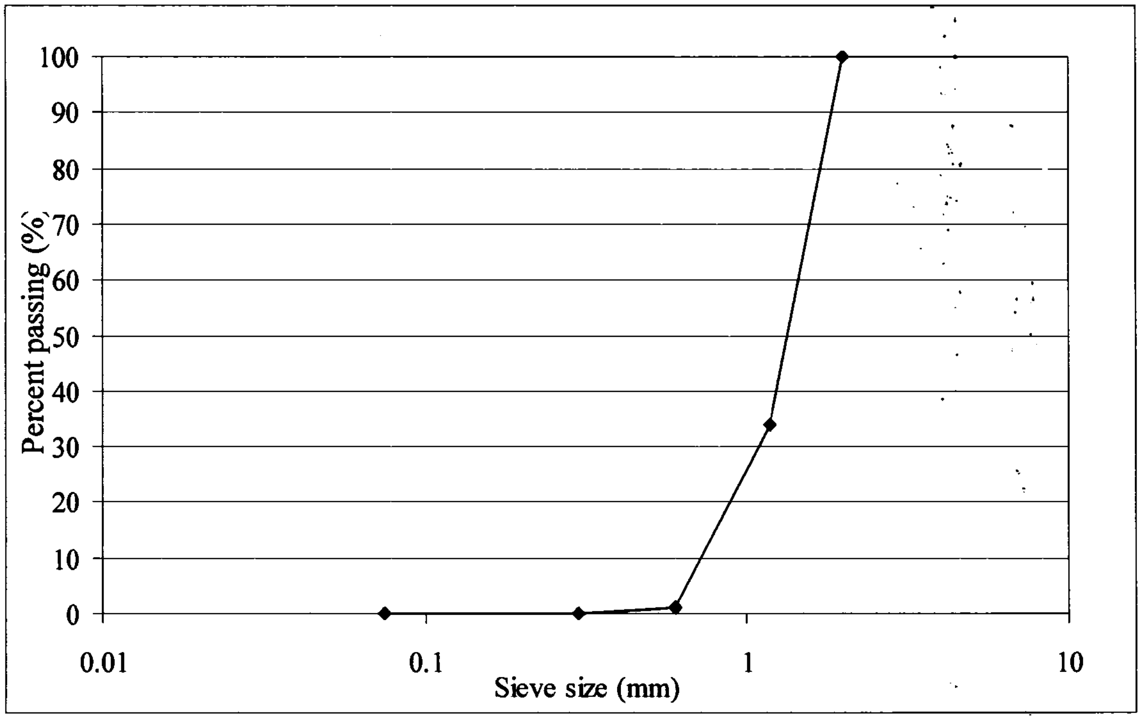

Figure 3.6 Gradation of CR (Highway 623 project)

Various physical properties of the binder were measured and are reported in Table 3.14.

Table 3.14 Physical properties of binder (Highway 623 project)

Test performed	Reaction time (minutes)					ADOT Specified Limits
	60	90	135	240	1440	
Apparent Viscosity at 177°C, cP (Brookfield, Spindle No.3 at 20 RPM) Modified ASTM D2196, Method A	1900	2000	2075	2075	2725	1500-5000
Penetration at 25°C, 100 g, 5 sec, ASTM D5	73	-	77		79	50-100
Penetration at 4°C, 200 g, 60 sec, ASTM D5	49	-	49		51	$\geq$25
Pinsky-Martins Flash Point, °C ASTM D93	284	-	-	-	-	232.2
Softening Point, °C ASTM D36	62.0	62.0	64.5	65	66	$\geq$51.7
Resilinece at 25°C, % Rebound ASTM D5329	32	-	47		40	$\geq$10
TFOT Residue Penetration at 4°C, 200g, 60 sec, dmm ASTM D1754	45	-	-	-	-	-
Penetration Retention at 4°C (%)	92	-	-	-	-	$\geq$75
Specific Gravity of ARB at 25°C/25°C	1.037	-	-	-	-	-

Note: The AR was held overnight at 135°C and heated back to 175°C for the final 24 hour reaction period.

3.3.2.4 Highway 623 aggregate and mix design

Two different mineral aggregate (washed manufactured fines and coarse aggregate) were used for the mix design. Figure 3.7 shows the aggregate gradation for the Highway 623 project. In addition, Table 3.15 shows the aggregate gradation and proportions of the two aggregate types used to obtain the job formula gradation.

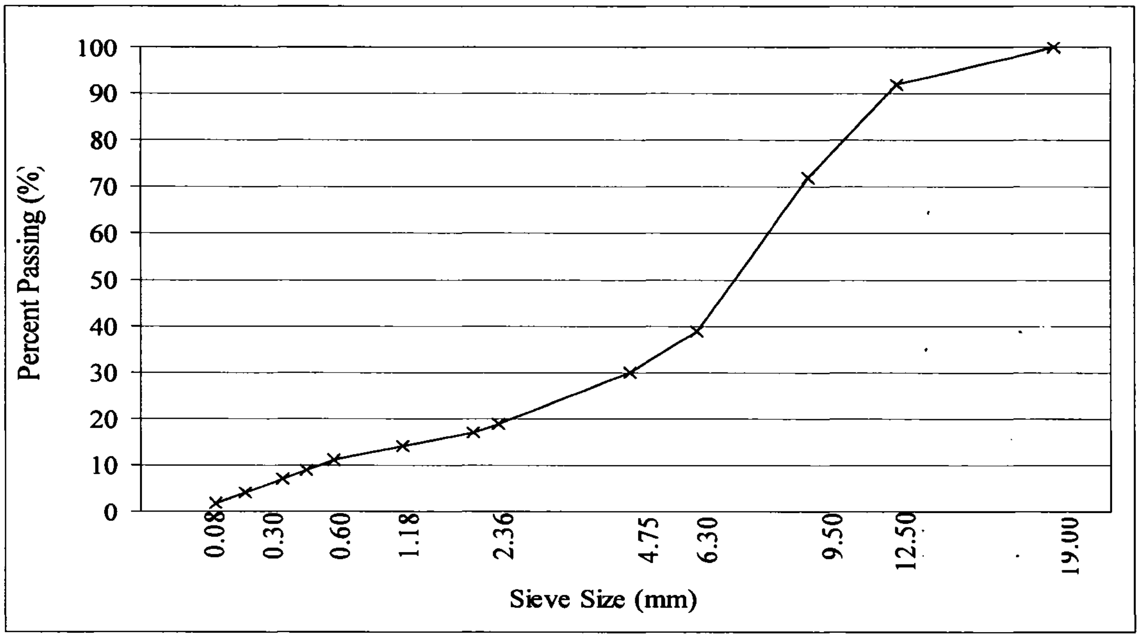

Figure 3.7 Aggregate gradation for the ARM design (Highway 623:03)

Table 3.15 Aggregate gradation for ARM design (reported Highway 623:03)

Sieve Size (mm)	Washed Manufactured Fines	Coarse Aggregate	Combined Blend	ADOT 413-2 Specification
19	100.0	100.0	100	100
12.5	100.0	89.1	92	80-100
9.5	100.0	62.0	72	65-80
6.3	97.9	18.7	39	-
4.75	94.5	8.1	30	28-42
2.38	64.0	3.6	19	14-22
2	57.4	3.4	17	-
1.18	46.9	3.1	14	-
0.6	36.4	2.9	11	-
0.425	29.0	2.7	9	-
0.3	20.3	2.5	7	-
0.15	7.8	2.1	4	-
0.075	2.6	1.6	1.9	0-2.5
Proportions (%)	25.0	75.0		-

The recommended binder content was 7.9% (by weight of mix). Table 3.16 presents the mix design data for the Highway 623:04 project.

Table 3.16 Properties of ARM at recommended asphalt content (Highway 623)

Property	Value	ADOT 413-1 Specification
Percent of AR (by weight. of mix)	7.9	-
Bulk Specific Gravity	2.195	-
Bulk Density (kg/m^3)	2191	-
Theoretical Max Specific Gravity (G_{mm})	2.326	-
Air Voids (%)	5.6	4.5-6.5
Voids in Mineral Aggregate (%)	21.6	$\geq$19
Voids Filled with Asphalt (%)	73.9	-
Effective Asphalt (%)	7.535	-
Dust to Effective Asphalt Ratio	0.25	-
Effective Specific Gravity	2.603	-
Film Thickness (microns)	29.1	-

3.3.3 Summary of the materials and mixtures design for Alberta Asphalt Rubber 2002 and 2003 project

In total, four different aggregate gradations and mix designs were used during the Alberta 2002 and 2003 projects. Similarly, three different AR binders were used during the 2002 and 2003 projects. Tables 3.17 and 3.18 summarize the properties of the AR binder used in the 2002 and 2003 projects. Figure 3.8 compares the various aggregate gradations used in the 2002 and 2003 Alberta Asphalt Rubber projects.

Table 3.17 Summary of AR binder used in Alberta Asphalt Rubber projects

Property	2002 binder	2003 binder	Highway 623
Base Asphalt	150-200 Pen	150-200 Pen	200-300 Pen
CR content (by total weight of binder)	19%	18.5%	18%

Table 3.18 CR gradation of AR binder used in Alberta Asphalt Rubber projects

Sieve Size (mm)	Percent passing		
	2002 AR binder	2003 AR binder	Highway 623
2.36	100	100	100
2	99.2	99	100
1.18	30.1	28.7	34
0.6	0.5	1	1
0.3	0.2	0.3	0
0.075	0	0	0

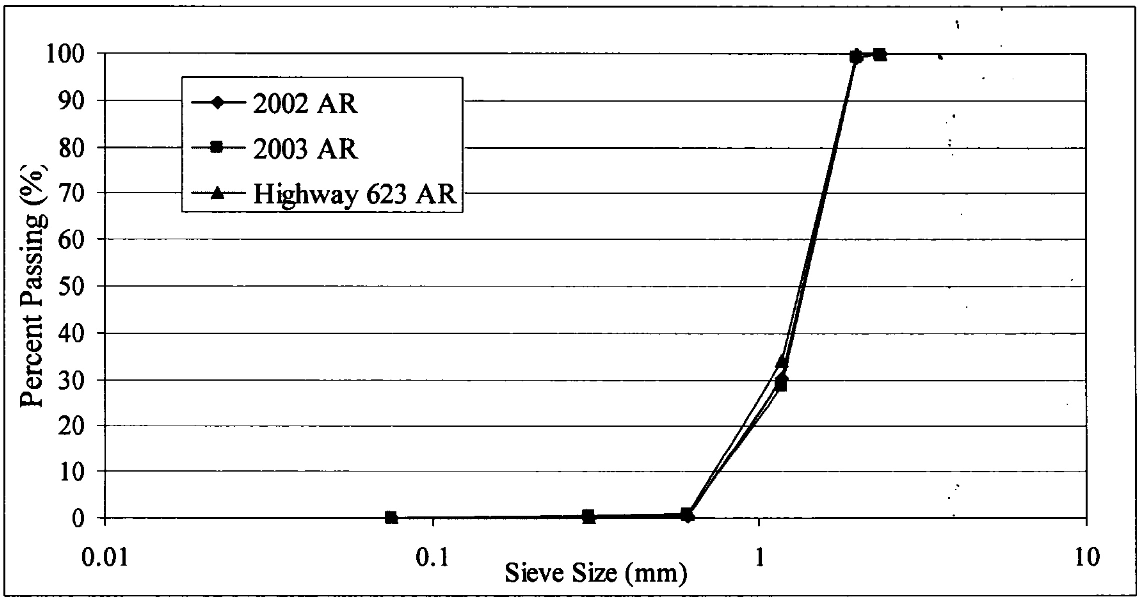

Figure 3.8 Gradation of CR for Alberta Asphalt Rubber project

No substantial difference can be seen in the gradation of the CR used during the 2002 and 2003 Alberta Asphalt Rubber projects. The base asphalt used in Highway 623 was, however, of soft consistency compared to the asphalt cement used in other project locations.

Figure 3.9 provides a comparison of the different aggregate gradations utilized in the 2002 and 2003 Alberta Asphalt Rubber projects. Table 3.19 summarizes the four different aggregate gradations and mix design utilized during the 2002 and 2003 Alberta Asphalt Rubber projects.

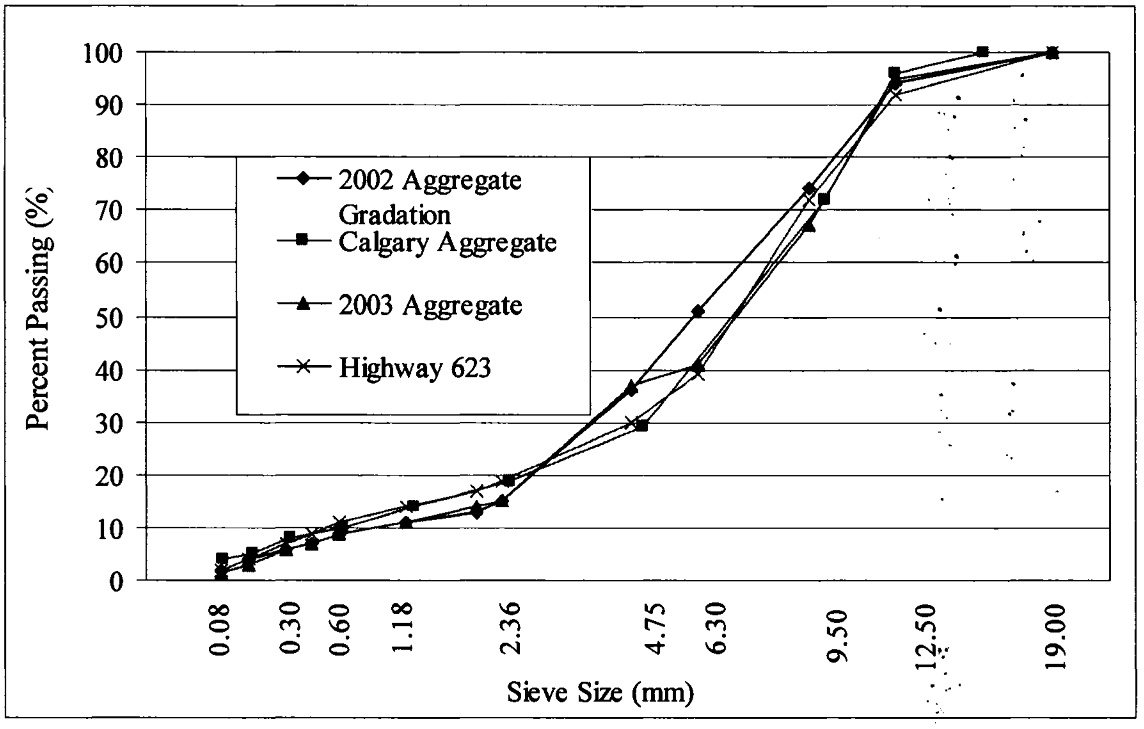

Figure 3.9 Aggregate gradation for 2002 and 2003 Alberta Asphalt Rubber project

Table 3.19 Summary of aggregate gradation and mix designs for 2002 and 2003 Alberta Asphalt Rubber projects

Property	2002	Calgary (2002)	2003	Highway 623 (2003)
Sieve Size (mm)	Percent passing	Percent passing	Percent passing	Percent passing
19.00	100	100	100	100
12.50	94	96	95	92
9.50	74	72	67	72
6.30	51	-	41	39
4.75	36	29	37	30
2.38	15	19	15	19
2.00	13	-	14	17
1.18	11	14	11	14
0.60	9	10	9	11
0.425	7	-	7	9
0.30	6	8	6	7
0.15	4	5	3	4
0.075	1.8	4.1	1.6	1.9
Design Binder content (by total weight)	8.9%	7.62%	6.82%	7.9%
Bulk Specific Gravity	2.178	2.247	2.237	2.195
Voids in Mineral Aggregate (%)	23.4	19.8	19.39	21.6
Air void (%)	5.5	5.5	5.5	5.6

Although there is no substantial difference among the aggregate gradations, it is evident that the 2003 aggregate is more open compared to other aggregate gradations. The amount of large coarse aggregate is less in the 2002 mix compared to the 2003 mix. The 2003 aggregate gradations had less fine aggregate compared to the Calgary aggregate and Highway 623 aggregates.

An important difference between the 2003 mix design and other mix designs was the lower binder content which could impact the long-term performance of pavement sections paved with this mix. The recommended binder content for the 2003 mix was 6.9% compared to 8.9%, 7.62% and 7.9% in the 2002, Calgary and Highway 623 mix designs, respectively. It is to be noted that the binder content in mix is directly proportional to the binder film thickness which subsequently affects the moisture sensitivity of the mix. A lower binder content and hence the lower film thickness might result in moisture related problems like stripping, raveling or even loss of aggregate. This lower asphalt content is alarming, considering the fact that there was no substantial difference between the aggregate gradation and the binder used in the 2002 and 2003 projects. The AASHTO T283 moisture sensitivity test on the 2003 mix also resulted in a lower TSR value of 0.40 compared to a value of 0.75 for the 2002 mix. A laboratory study was conducted at the University of Alberta to validate the lower asphalt content used in the 2003 mix design. The results are presented in Chapter four of this thesis.

3.4 Performance evaluation of Alberta Asphalt Rubber project

The performance of the ARM sections paved in 2002 and 2003 are being monitored for their rut depths and roughness. IRI is being used as a measure of the roughness of the pavement sections. In addition, the crack measurements of the 2002 ARM sections were measured manually during the summer of 2004 and 2005. Similarly, the cracks of the 2003 ARM sections were measured manually during the summer of 2005. The IRI and rut were measured by EBA using an International Cybernetics Corporation MDR 4087 Road Profiler, which was equipped with eleven laser sensors and two wheel path accelerometers. The IRI was collected as an indication of pavement roughness before and after construction for all project sections. The values for both the Inner Wheel Path (IWP) and the Outer Wheel Path (OWP) in a single lane were measured. However, cracks were measured by the University of Alberta. Some of the pictures taken during the crack measurement are presented in Appendix B of this thesis.

This section presents the short-term performance evaluation of the 2002 and 2003 ARM sections based on the IRI, rut and crack measurements. In addition, this section attempts to compare the performance of the ARM sections.

3.4.1 17 Street project (2002)

In the 17 Street project one pre-construction and three post-construction IRI and rut measurements were collected. The post-construction measurements were made on August 26, 2002, October 27, 2003 and October 24, 2005 respectively. Figures 3.10 and 3.11 present the summary of the IRI and rut measurements at the 17 Street project location respectively.

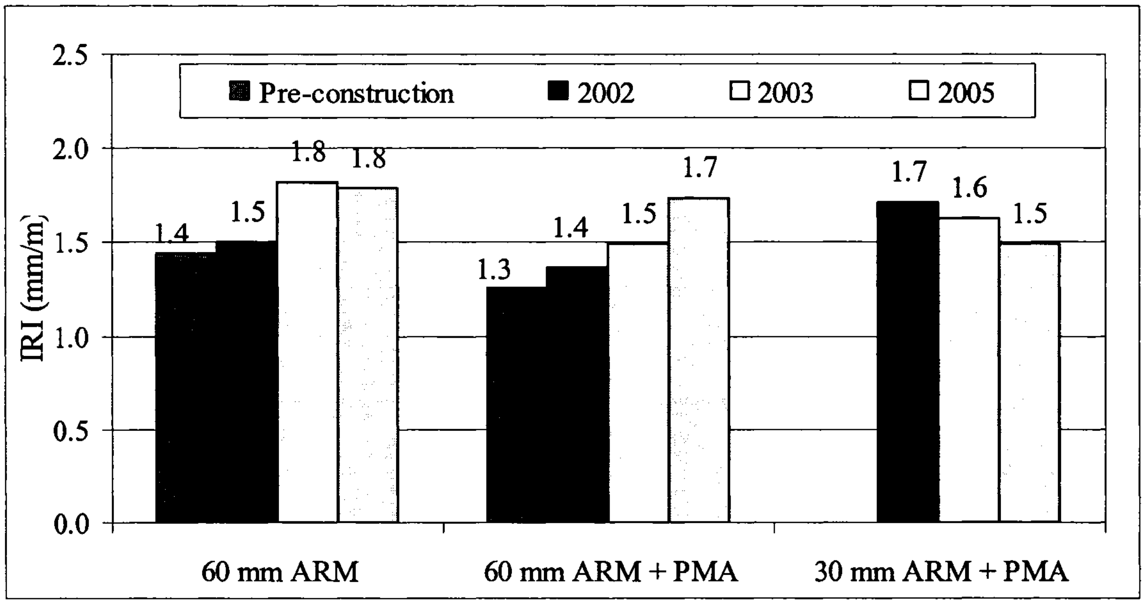

Figure 3.10 Summary of the IRI measurements at the 17 Street project location

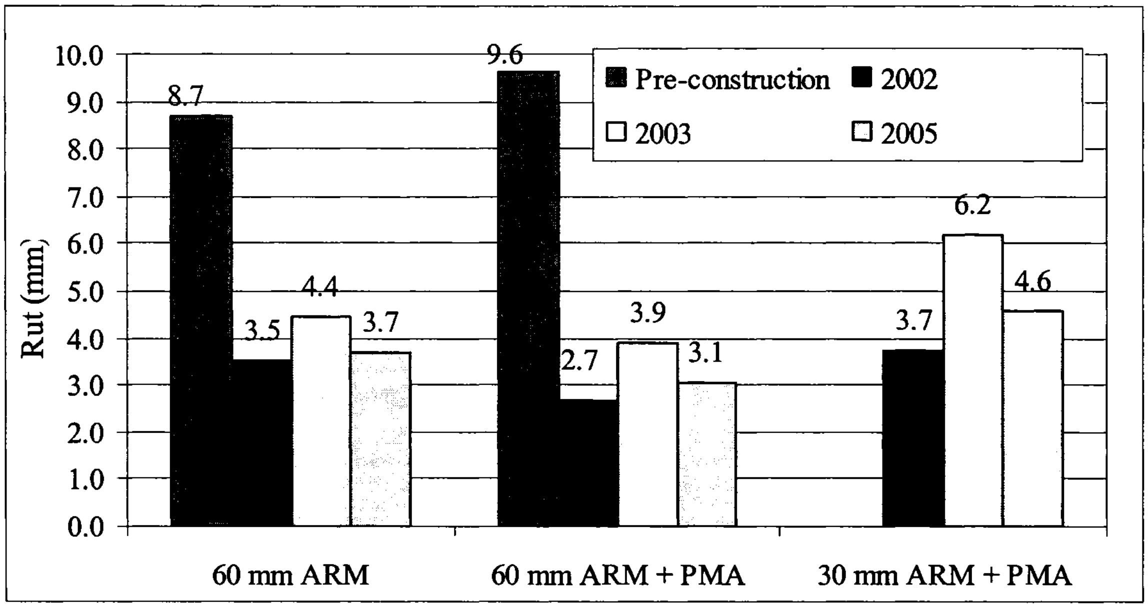

Figure 3.11 Summary of the rut measurements at the 17 Street project location

The IRI measurements reveal that the post-construction IRI of the ARM sections increased gradually at both 60 mm ARM overlays at a slow rate. However, the 30 mm ARM overlay showed a decreasing IRI trend. Generally, all before and after IRI values were in the range of 1.3 to 1.7, which suggests roughness was not the main reason for overlay and it is in good range three years after overlay. The high rutting measurement before overlay can be attributed to heavy truck traffic and the presence of a signalized intersection at this project location. After overlay with ARM, rutting decreased at all locations of this project. It seems 60 mm ARM overlay and combination of PMA and ARM overlay showed a better rutting performance.

In the summer of 2004 and 2005, the crack and distress measurement were recorded manually. No additional distresses or cracks were observed in 2005 compared to the 2004 measurements in all pavement sections at this project location. Table 3.20 presents a summary of the crack measurements at this project location in the summer of 2004. Figure 3.12 presents the crack map for this project location.

Table 3.20 Normalized (per 100 m) crack measurements for 17 Street

Crack Type	Severity	60 mm ARM	60 mm ARM over 40 mm PMA	30 mm ARM over 40 mm PMA
Transverse Crack (no of Cracks)/100m	High severity	0.31	0.00	0.00
	Medium Severity	0.00	0.00	0.00
	Low Severity	0.00	0.00	0.00
Longitudinal Cracks (m)/100m	High severity	0.00	0.00	0.00
	Medium Severity	0.00	0.00	0.00
	Low Severity	0.00	0.00	0.00

Based on the visual surface distress evaluation, it can be concluded that cracking or other surface distresses (ravelling, stripping, patches and pot holes) were not a major problem at the ARM section at the 17 Street project location.

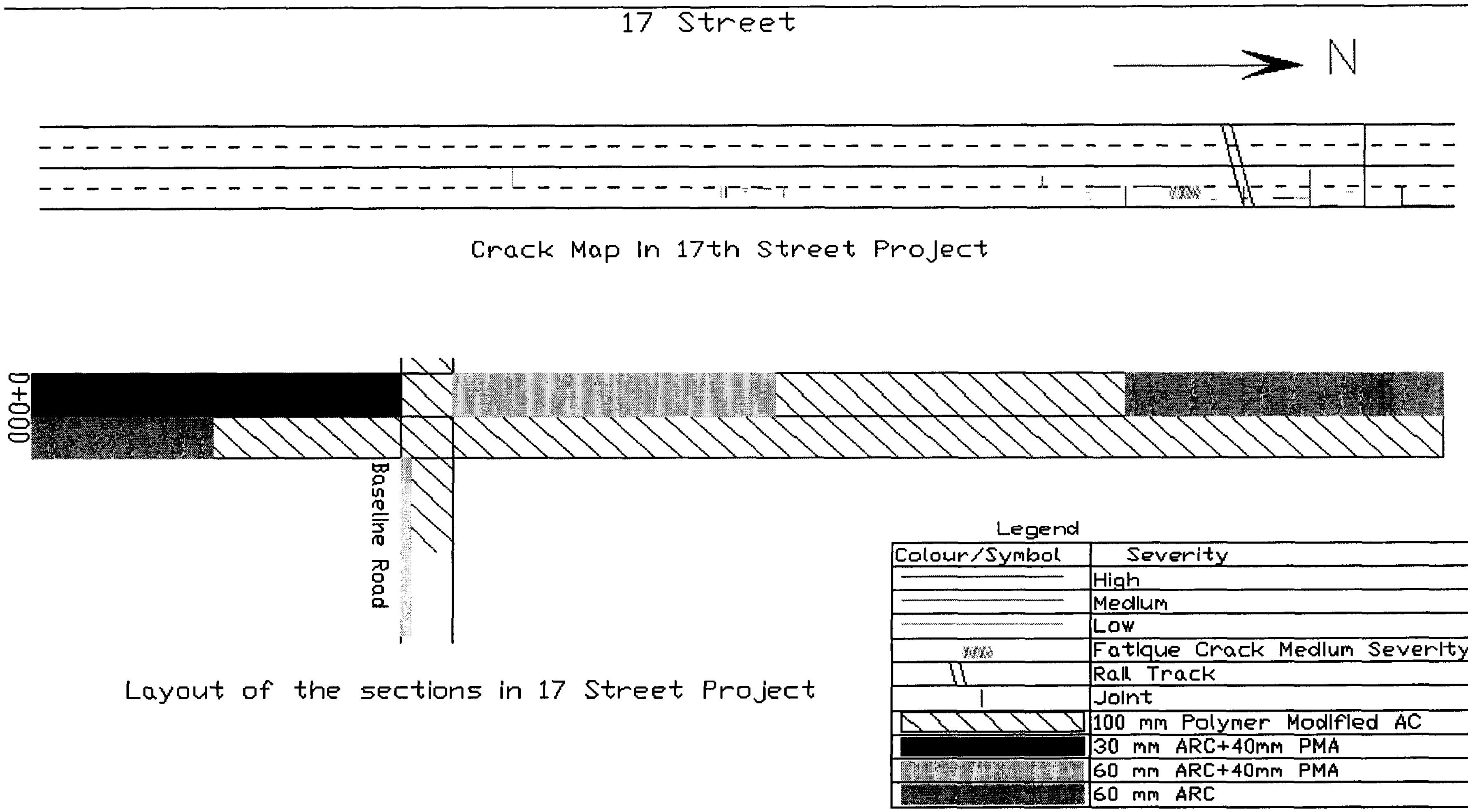

Figure 3.12 Crack measurements at 17 Street project location

IRI and rut were also measured in the left turn lane east of 17 Street on Baseline Road. The post-construction measurement on the Baseline Road was conducted on August 26, 2002 and October 27, 2003. Figures 3.13 and 3.14 present the summary of the IRI and rut measurements at Baseline Road.

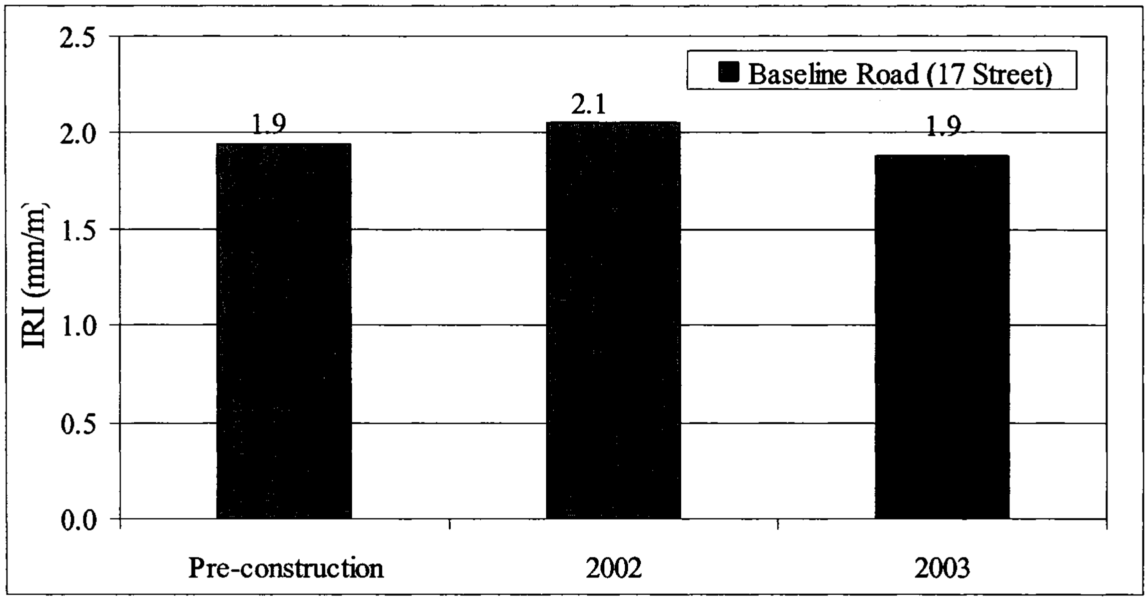

Figure 3.13 Summary of the IRI measurements at the Baseline Road (17 Street)

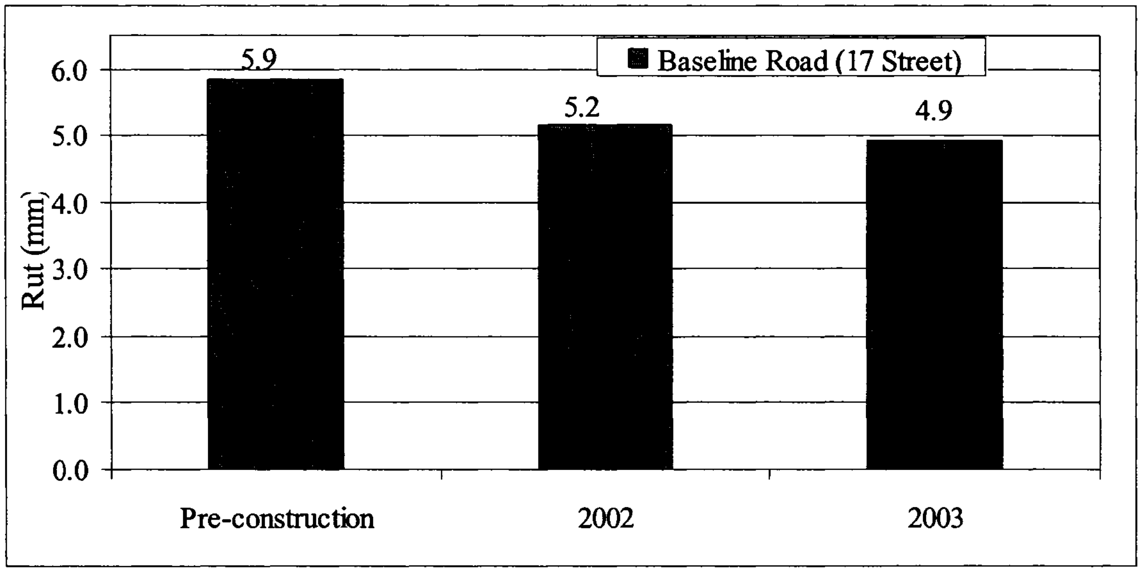

Figure 3.14 Summary of the rut measurements at the Baseline Road(17 Street)

No substantial increase in IRI was observed one year after construction on the Baseline section. Of all sections paved in 2002, the rutting measurement at this project location is the highest. As previously mentioned, this can be attributed to the heavy truck traffic and the presence of a signalized intersection.

3.4.2 137 Avenue project (2002)

In the 137 Avenue project, one pre-construction IRI and rut measurement and three post-construction IRI and rut measurements were collected. The post-construction measurements were made on October of 2002, 2003 and 2005 respectively. In the summer of 2004 and 2005, manual field surveys of the 137 Avenue pavement sections were conducted to measure the cracks and other surface distresses.

Figures 3.15 and 3.16 present the summary of the IRI and rut measurements conducted at this project location.

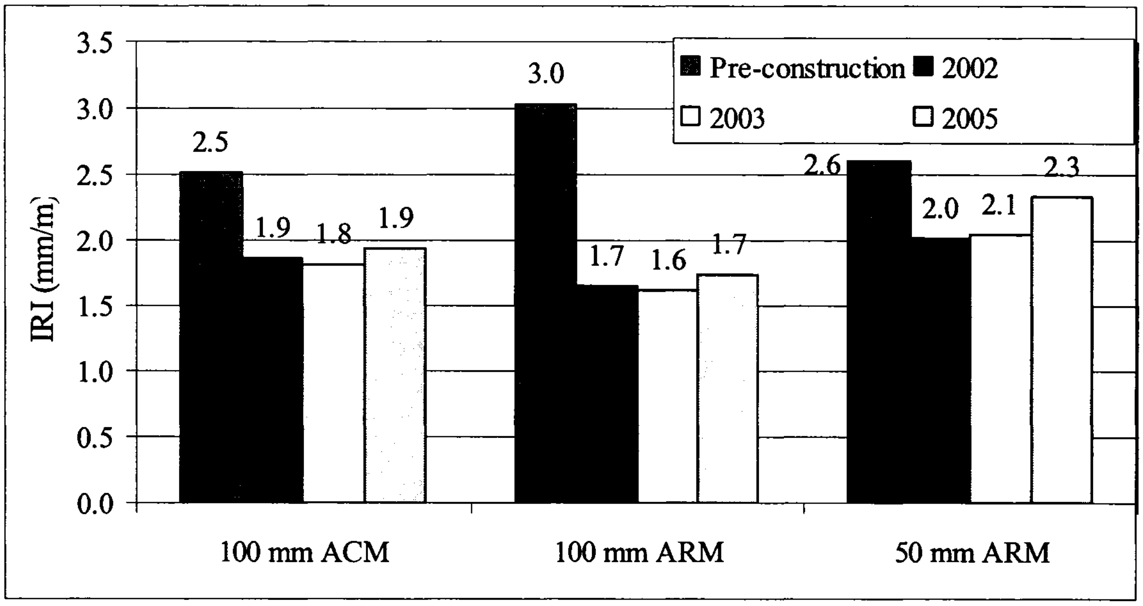

Figure 3.15 Summary of IRI measurements at the 137 Avenue project location

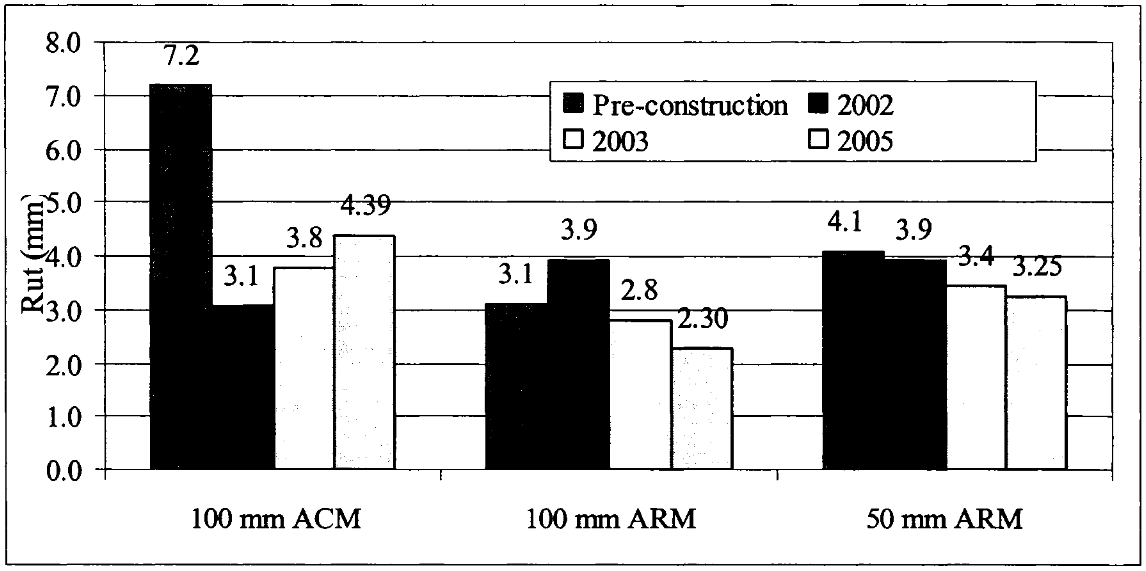

Figure 3.16 Summary of rut measurements at the 137 Avenue project location

As expected, increase in IRI for 100 mm ARM and ACM sections are relatively less compared to the 50 mm ARM section. However, there is no substantial difference in the performance of the 100 mm ACM and ARM sections from an IRI perspective. The rate of IRI increase is also very low at this project location. Regarding rutting, the pre-construction rutting of pavement section overlaid with ACM was higher compared to pavement sections overlaid with ARM. The post-construction rutting of ACM overlay increased by time. However; the rutting of ARM sections are showing a decreasing trend. This rutting decreasing trend can be attributed to changes in measurements year by year and the fact that the accuracy of the rut measuring equipment is ± 1 mm.

Table 3.21 presents the summary of the crack measurements conducted at the 137 Avenue project location during the summer of 2004 and 2005. For convenience, the measurements in Table 3.21 have been presented in Figures 3.17 and 3.18 as well. Figure 3.19 presents the crack measurements at the 137 Avenue project location.

Table 3.21 Summary of crack measurements survey at 137 Avenue project location

Crack type	Severity	Normalized crack length /100 lane m					
		50mm ARM		100mm ARM		100mm ACM	
		2004	2005	2004	2005	2004	2005
Transverse Crack	High	1.03	1.03	0.00	0.00	5.03	5.03
	Medium	23.33	23.50	20.47	20.47	2.06	2.06
	Low	8.92	12.11	9.25	15.28	0.46	19.82
	Total	33.28	36.64	29.72	35.75	7.55	26.91
Longitudinal Crack	High	0.00	0.00	0.00	0.00	0.00	0.00
	Medium	2.71	2.71	0.00	0.00	0.00	0.00
	Low	3.19	7.89	.85	4.34	0.00	15.59
	Total	5.89	10.60	0.85	4.34	0.00	15.59

The ARM sections at this project location were most severely affected by cracks compared to other sections paved in 2002. Most of the cracks observed in 2004 in the ARM sections were spaced uniformly, suggesting it might be because of the PCC base underneath. No significant cracks were measured in the ACM sections in 2004. In the 2005 measurements, it was observed that all the cracks measured in 2004, and the cracks measured in 2005 in the ACM section, were sealed. It is evident from the measurements in 2005 that more cracks appeared in the ACM sections compared to the ARM sections. Almost all of the new cracks observed in 2005, in the rubber section, were of low severity.

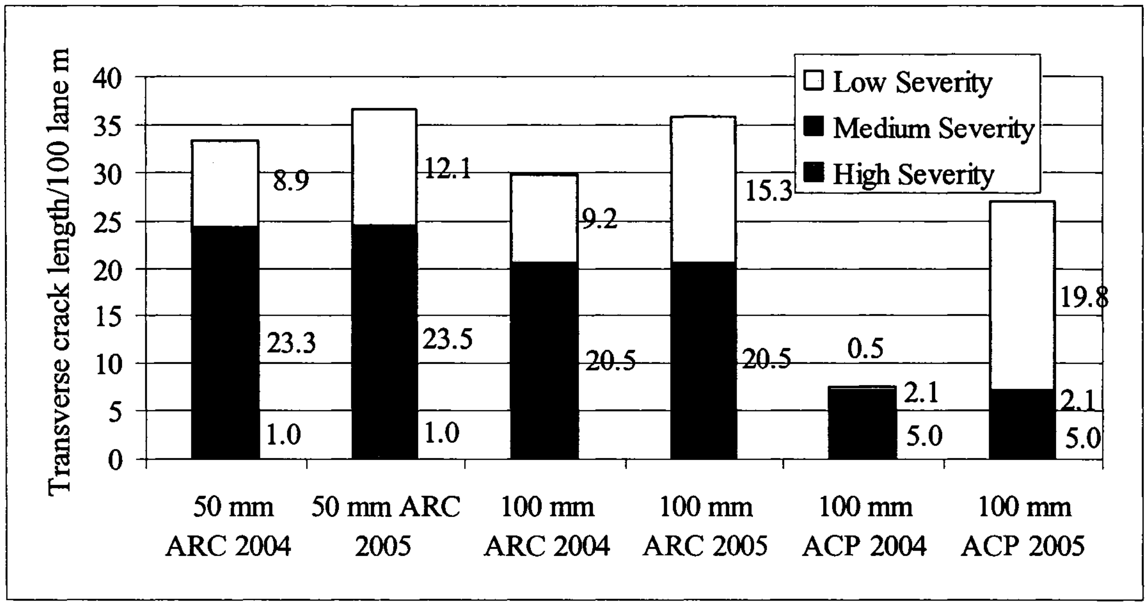

Figure 3.17 Transverse crack measurements at 137 Avenue project locations

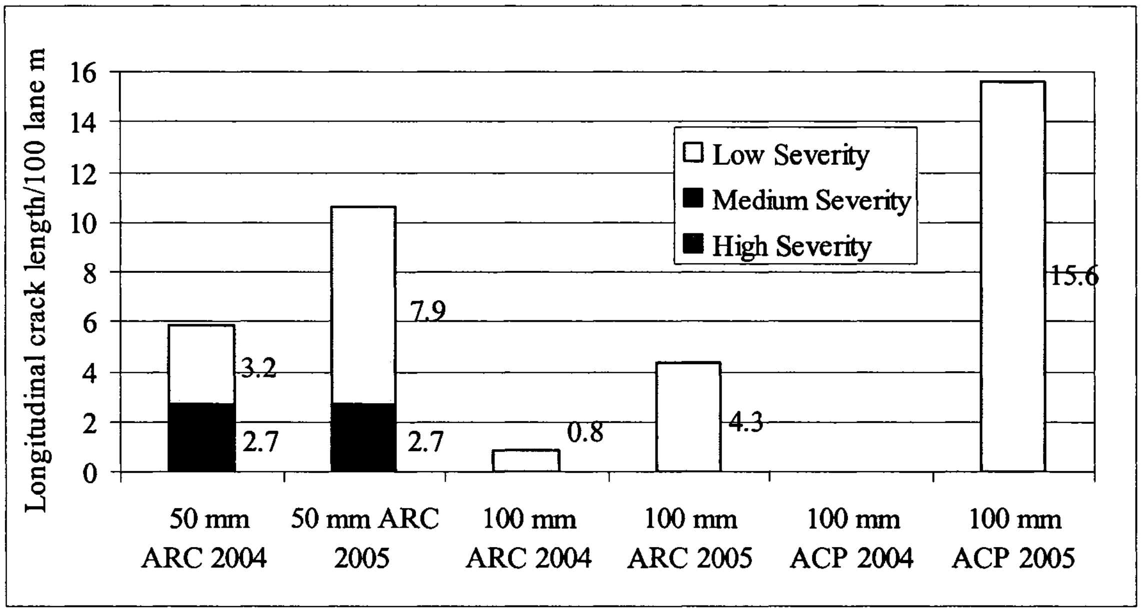

Figure 3.18 Longitudinal crack measurements at 137 Avenue project locations

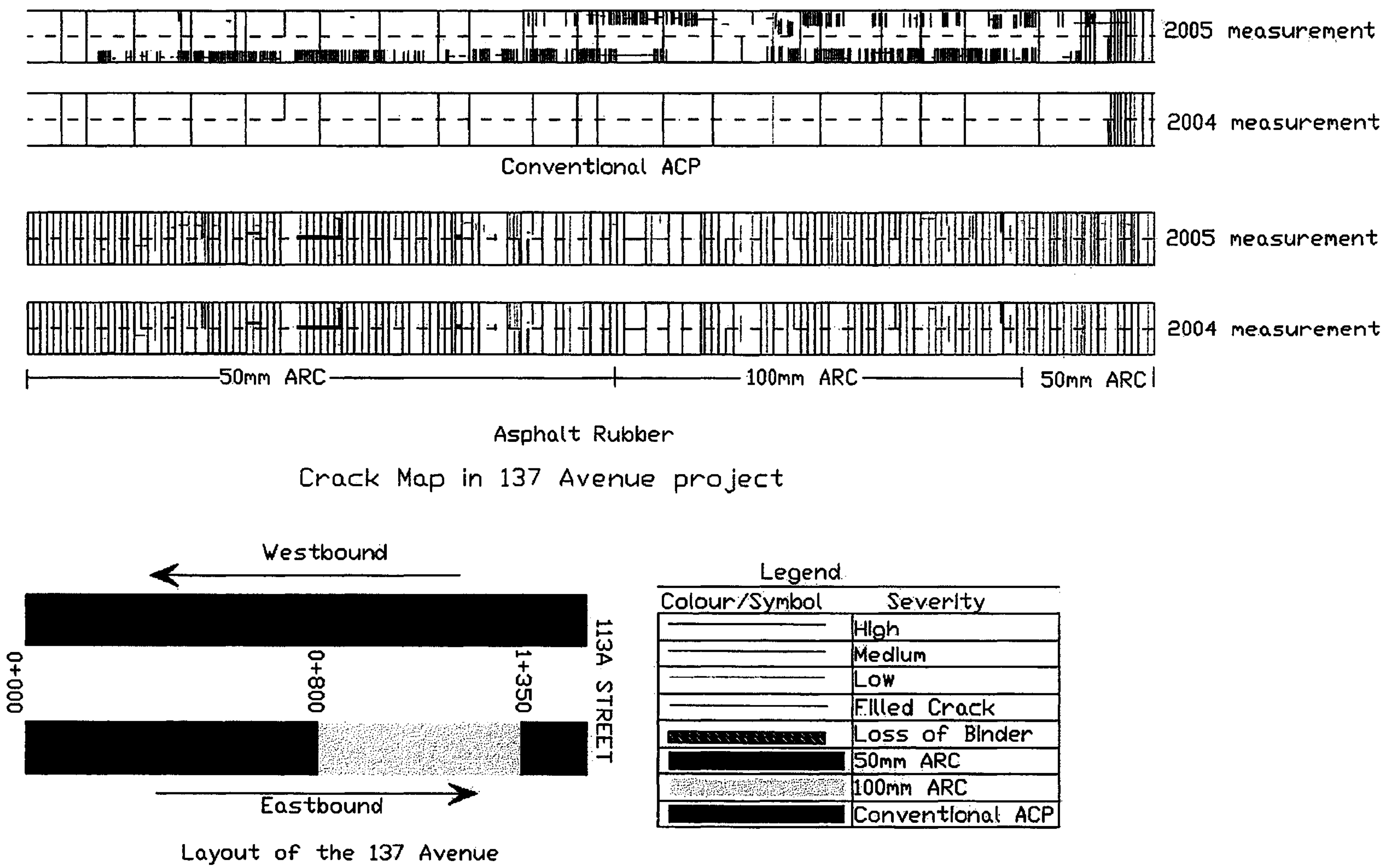

Figure 3.19 Crack measurements at 137 Avenue project location

3.4.3 Highway 630 project (2002)

In Highway 630:03 project one pre-construction and two IRI and rut measurements have been made after the construction. The post-construction measurements were made on August 26, 2002 and October 27, 2003 respectively. In addition to the IRI and rut measurements, a manual field survey was also conducted during the summer of 2004 and 2005 to measure the surface cracks and distresses at this project location.

As expected, the 80 mm ARM and ACM section has performed better than the 40 mm ARM section from both rutting and IRI perspective. However, the difference in the IRI values between the 40 and 80 mm pavement sections is not significant. The rate of IRI increase at this project location is extremely low. The rutting measurements at this project location indicate that the ACM pavement section performed better than the ARM section. However, the difference in the rutting measurement between the 80 mm ARM and ACM sections is not substantial.

Figures 3.20 and 3.21 present the summary of the IRI and rut measurements at this project location.

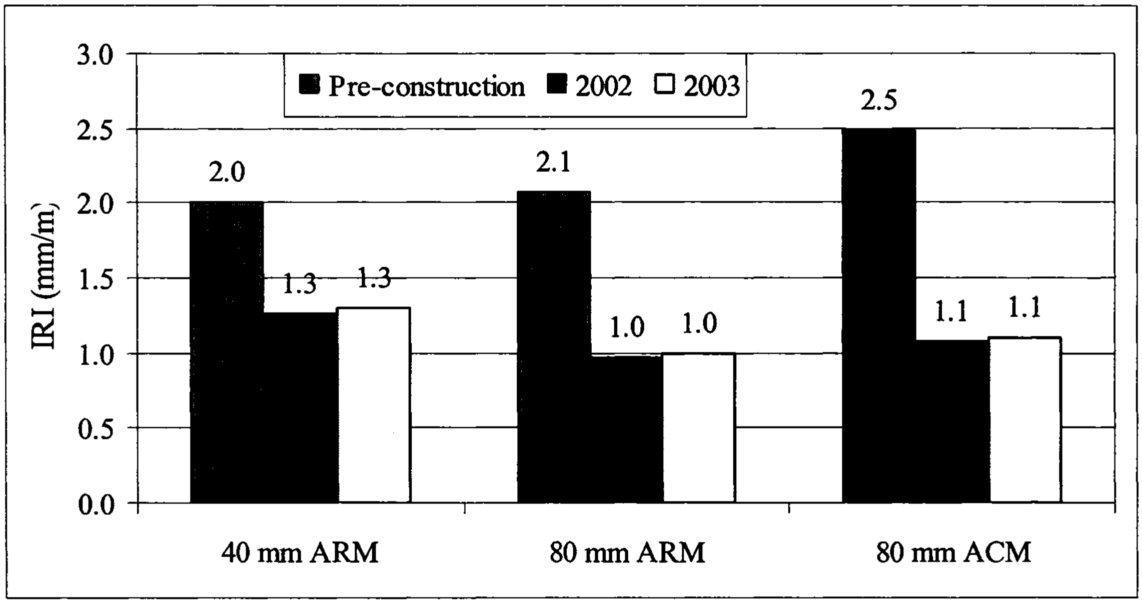

Figure 3.20 Summary of IRI measurement at Highway 630 project location

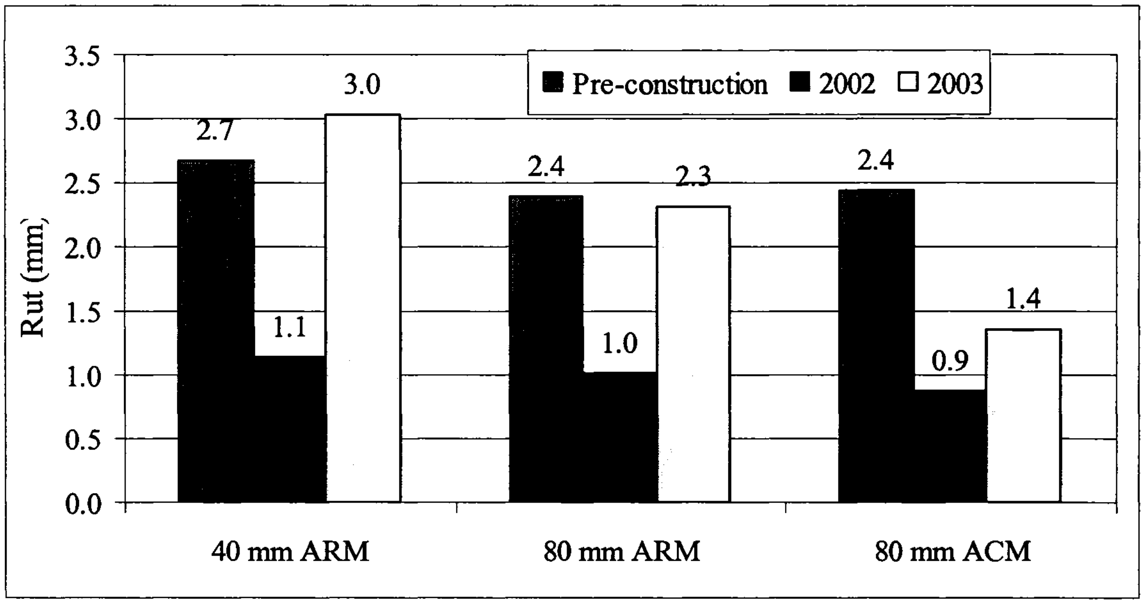

Figure 3.21 Summary of rut measurement at Highway 630 project location

Table 3.22 summarizes the visual distress measurements conducted in the summer of 2004 and 2005 at this project location. Figure 3.22 presents the crack measurement map at the Highway 630 project location.

Table 3.22 Summary of the distress measurement at Highway 630 project location

Distress Type	Severity	Normalized distress measurement/100m							
		40 mm ACM		80 mm ACM		40 mm ARM		80 mm ARM	
		2004	2005	2004	2005	2004	2005	2004	2005
Transverse cracks (m)	High	4.90	4.90	3.50	3.50	4.90	4.90	4.90	4.90
	Medium	1.75	1.75	5.95	6.30	1.40	1.40	0.35	0.35
	Low	0.00	0.47	0.00	0.00	0.00	0.00	0.00	0.00
	Total	6.65	7.12	9.45	9.80	6.30	6.30	5.25	5.25
Longitudinal cracks (m)	High	37.96	37.96	0.00	0.00	7.54	7.54	14.00	14.00
	Medium	11.20	11.20	42.90	42.90	55.18	55.18	33.67	33.67
	Low	4.63	29.14	7.02	13.82	8.22	23.92	0.00	5.93
	Total	53.79	78.29	49.91	56.71	70.94	86.64	47.67	53.59
Patch (m^2)		0.00	0.00	0.00	0.00	0.00	0.07	0.00	0.05

It is evident from the measurements that most of the cracks in 2004 were of high severity, except for the 80 mm ARM section where more medium severity crack was observed. It is to be noted that the majority of the longitudinal cracks observed at this project location were construction joints between two lanes. Based on the crack measurements from 2004 no sections can be considered superior to the other one. No substantial increases in cracks were observed in the pavement sections at this project location in 2005. However, few patches were observed in the ARM section in 2005.

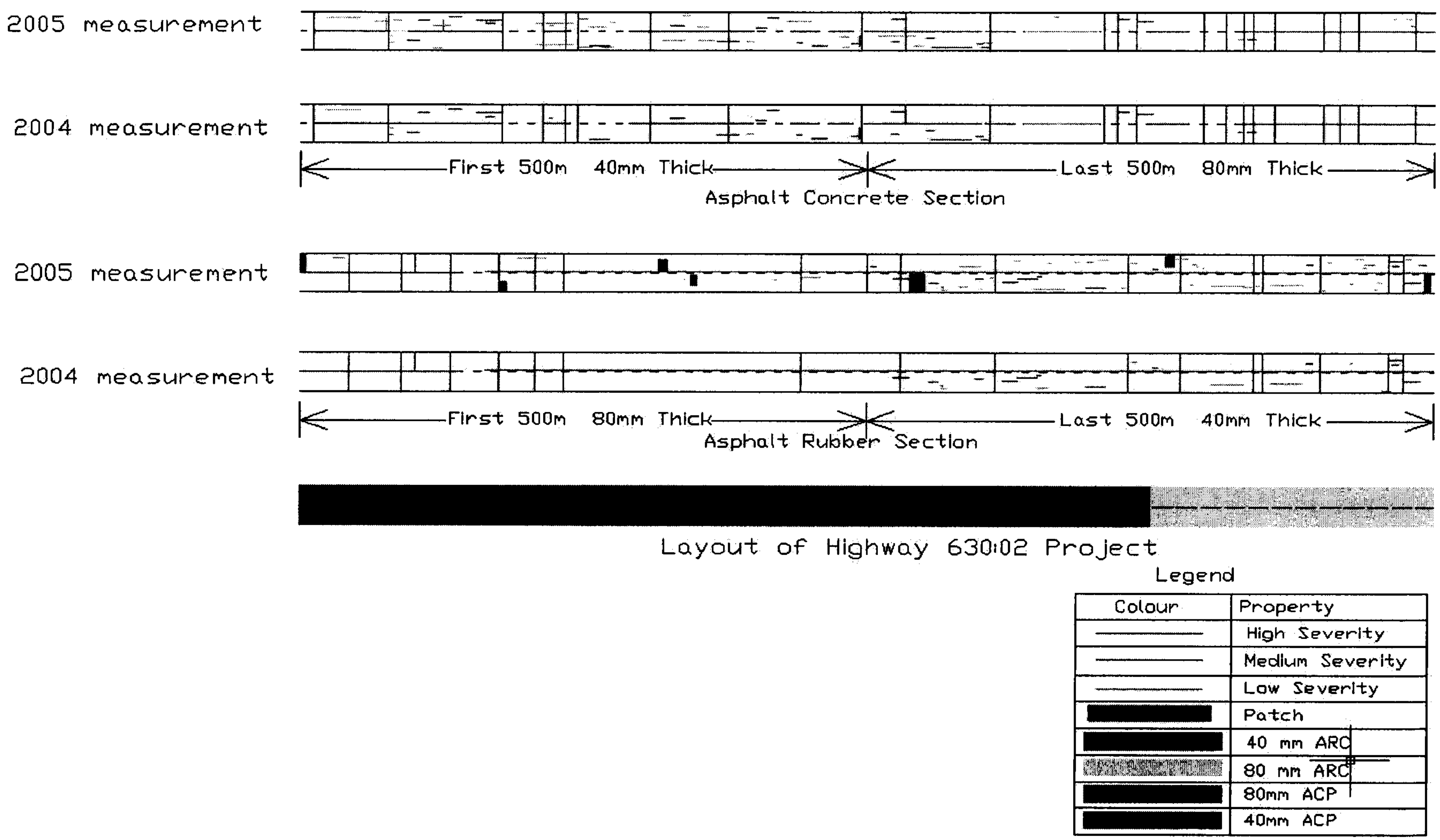

Figure 3.22 Crack measurement map at Highway 630:02 project location

3.4.4 112 Avenue (2002)

In the 112 Avenue project, one pre-construction IRI and rut measurement and two IRI and rut measurements have been conducted after the construction. The post-construction measurements were made on September 9, 2002 and September 30, 2003 respectively. In addition to the IRI and rut measurements, the crack and surface distresses were measured manually at this project location in the summer of 2004 and 2005.

Based on the IRI and rut measurement, the performance of the SAM can be considered superior to the ARM sections. The difference in the IRI and rut values between the SAM and ARM sections is, however, not substantial. In between the two ARM sections, the 75 mm ARM section, as expected, has performed better than the 40 mm ARM. Similar to the other 2002 projects, the rate of the IRI increased very slowly at this project location.

Table 3.23 presents the summary of the crack measurements conducted in the summer of 2004 and 2005 at this project location. Figure 3.25 presents the crack measurement map at this project location.

Figures 3.23 and 3.24 present the summary of the IRI and rutting measurements at this project location.

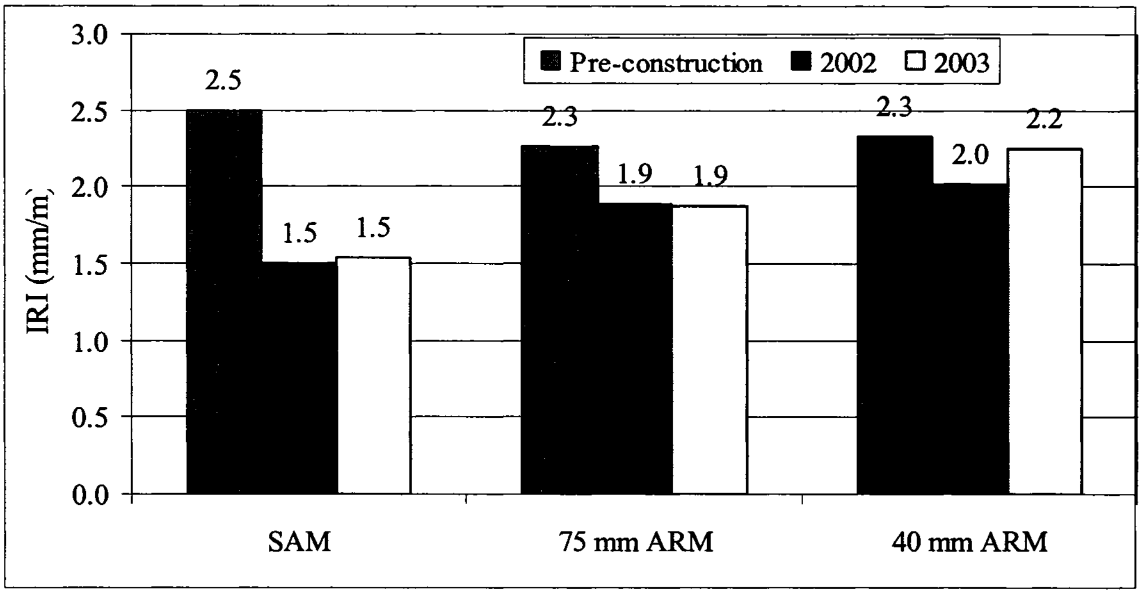

Figure 3.23 Summary of the IRI measurements at 112 Avenue project location

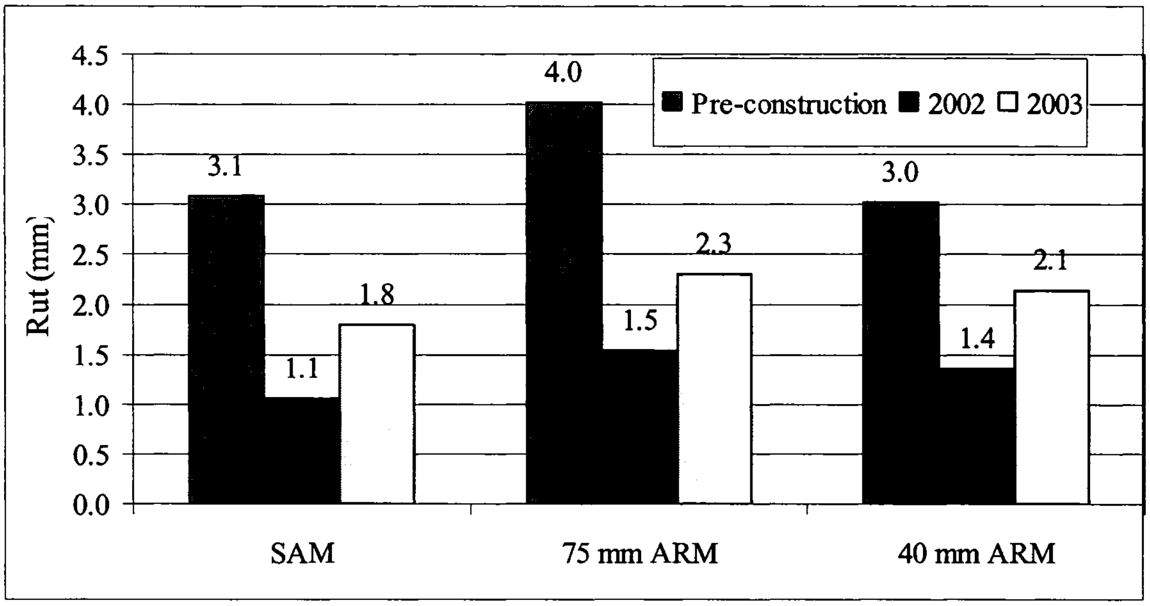

Figure 3.24 Summary of the rut measurements at 112 Avenue project location

Table 3.23 Summary of the crack measurements at the 112 Avenue project location

Crack Type	Severity	Normalized crack length/100 lane m					
		Shingle Asphalt Mixture		40mm ARM		75mm ARM	
		2004	2005	2004	2005	2004	2005
Transverse	High	0.00	0.00	0.00	0.00	0.00	0.00
	Medium	0.44	0.44	1.65	1.65	0.83	0.83
	Low	0.00	1.10	0.83	0.83	0.83	0.83
Longitudinal	High	0.60	0.60	2.00	2.00	0.46	0.46
	Medium	0.00	0.00	4.32	4.32	1.12	1.12
	Low	0.00	0.82	1.47	1.93	2.73	2.73

Among all the projects visually observed during the summer of 2004, the pavements sections at this project location showed the least amount of transverse and longitudinal cracks. As expected, both 75 mm ARM and SAM sections performed better compared to the 40 mm ARM section, from a cracking perspective. Almost no increase in the cracking was observed in the 2005 measurement.

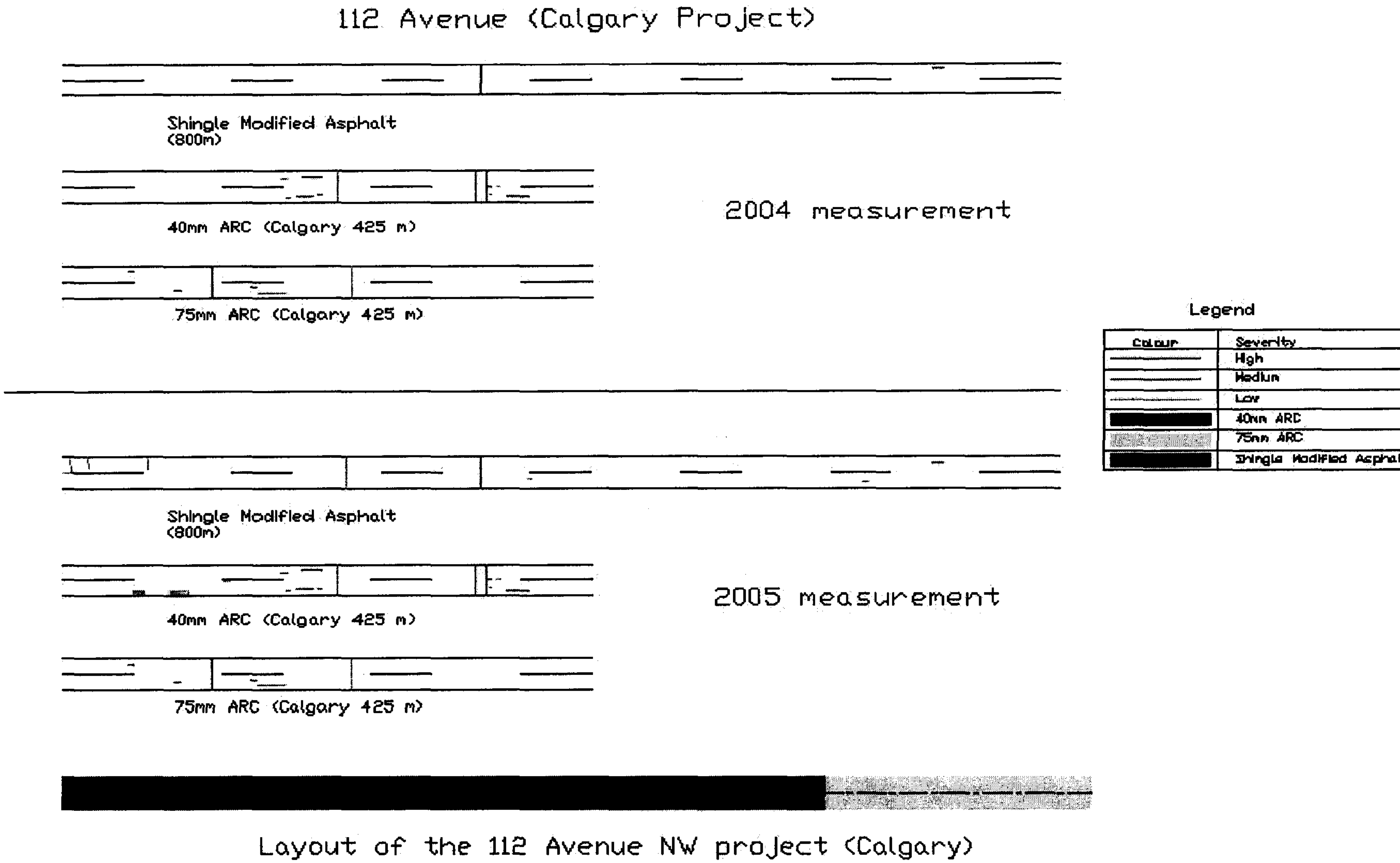

Figure 3.25 Crack measurements map at the 112 Avenue project location

3.4.5 50 Street (2003)

In the 50 Street project location, one pre-construction and two post-construction IRI and rut measurement were collected. The post-construction measurements were conducted on October 27, 2003 and October 24, 2005 respectively. In addition to the IRI and rut measurements, the crack and surface distresses were recorded manually at this project location during the summer of 2005.

The IRI increase at this project location is not substantial. Rutting measurement reveals a decreasing trend at this project location. This can be attributed to change in measurements year by year and the fact that the accuracy of rut measuring equipment is ±1 mm.

Figures 3.26 and 3.27 present the summary of the IRI and rutting measurements at this project location.

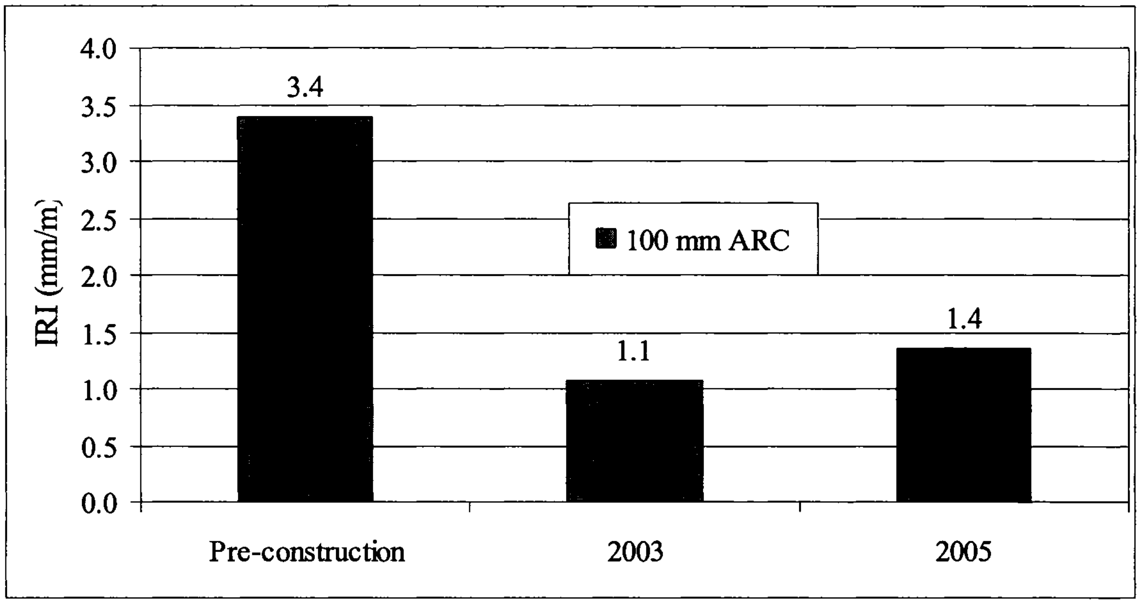

Figure 3.26 Summary of the IRI measurement at 50 Street project location

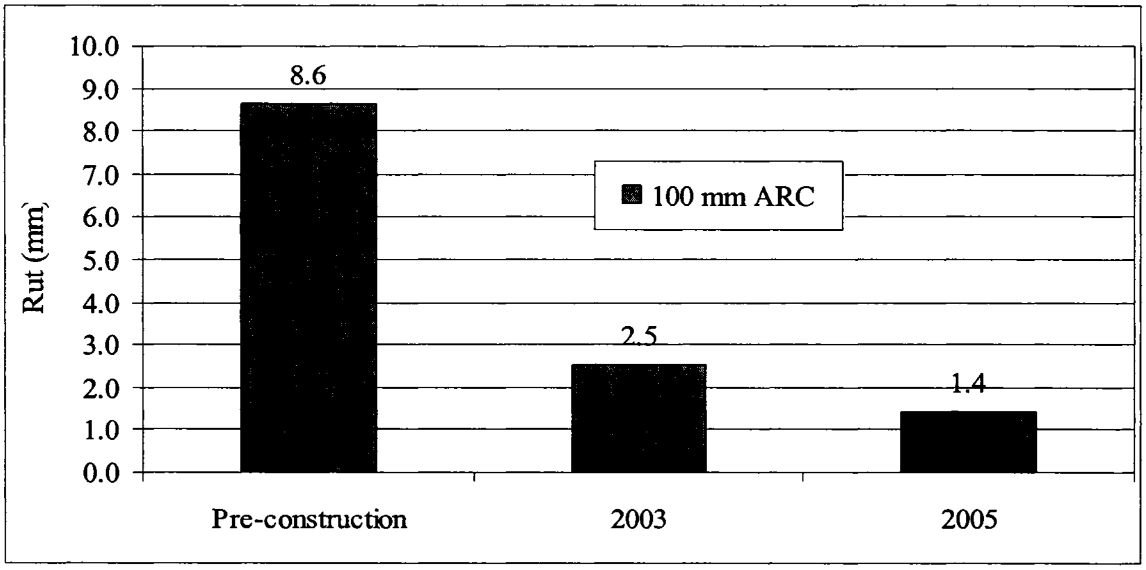

Figure 3.27 Summary of the rut measurement at 50 Street project location

Table 3.24 summarizes the crack measurements conducted at this project location in the summer of 2005. The crack map for this project location is presented in Figure 3.28.

Table 3.24 Summary of the distress measurement at the 50 Street project location

Distress Type	Severity	Normalized distress/100 lane m
Transverse crack (m)	High	0.00
	Medium	4.34
	Low	13.24
	Total	17.58
Longitudinal crack (m)	High	0.00
	Medium	0.00
	Low	8.58
	Total	8.58
Ravelling (m^2)		0.48

As shown in Figure 3.28 most of the cracks were concentrated in the section between 41 Avenue SW and Ellerslie Road. It is evident from Table 3.24 that a majority of the cracks are of low severity.

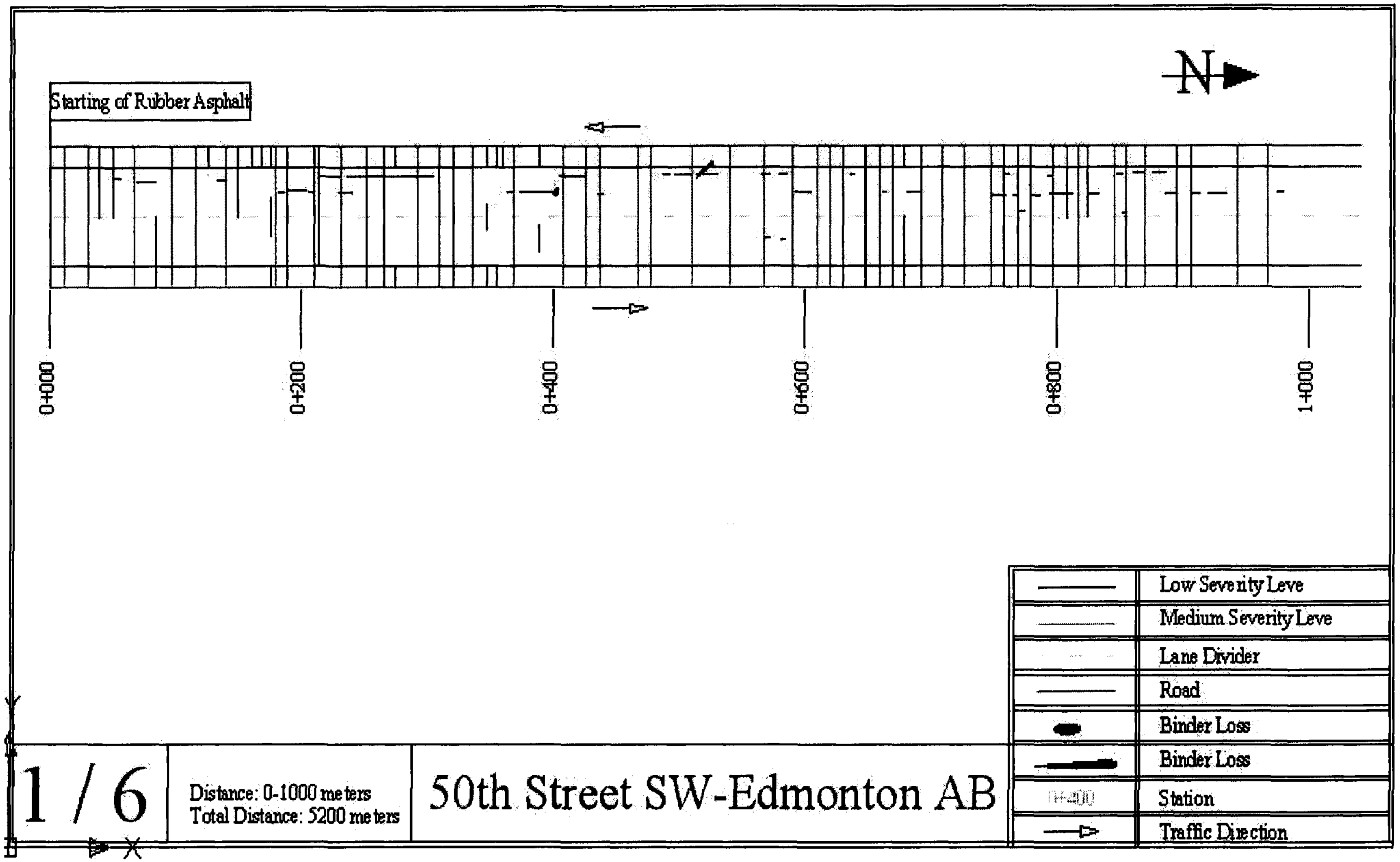

Figure 3.28A Crack measurements map at the 50 Street project locations (0-1000 m)

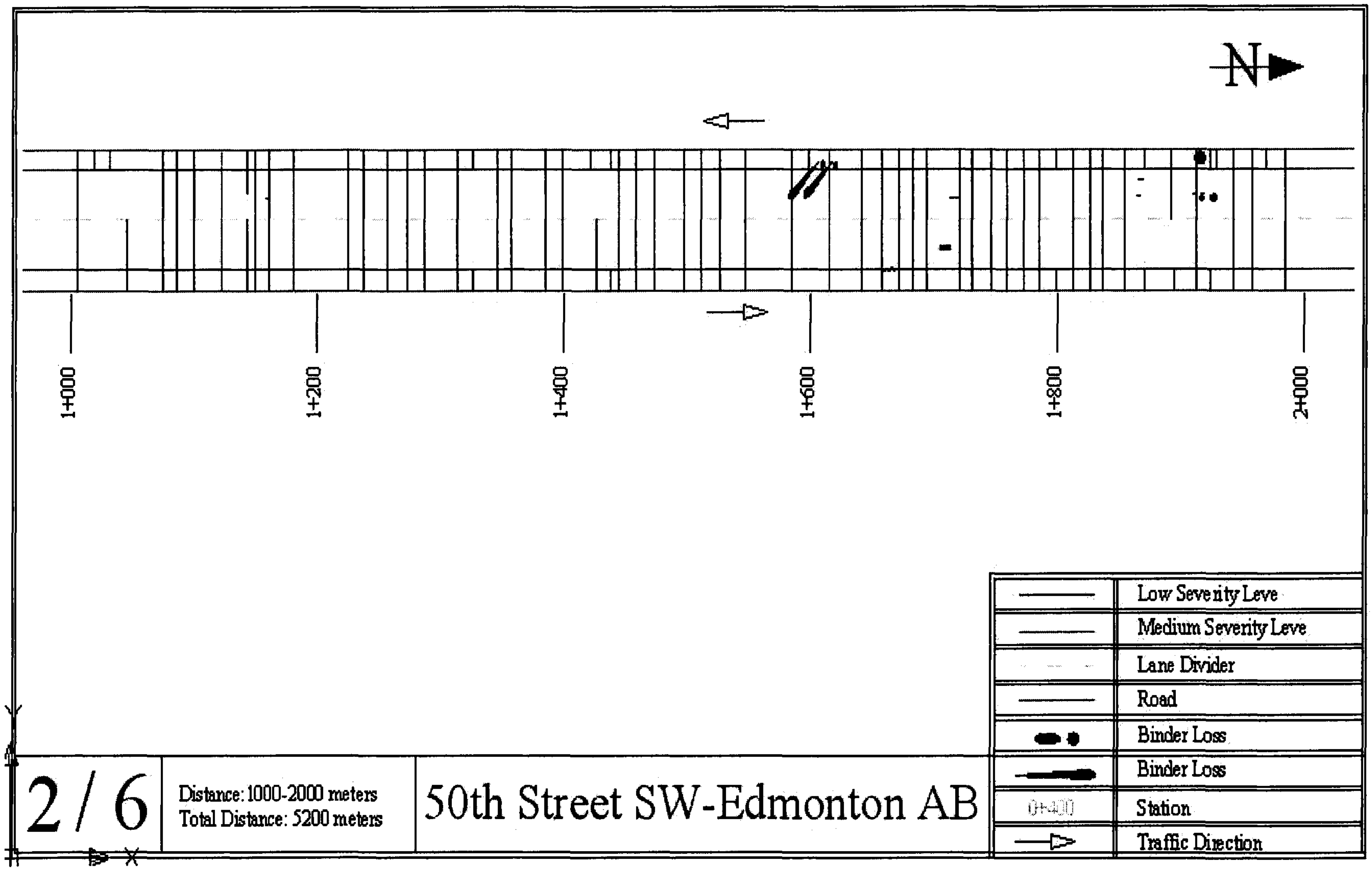

Figure 3.28B Crack measurements map at the 50 Street project locations (1000-2000 m)

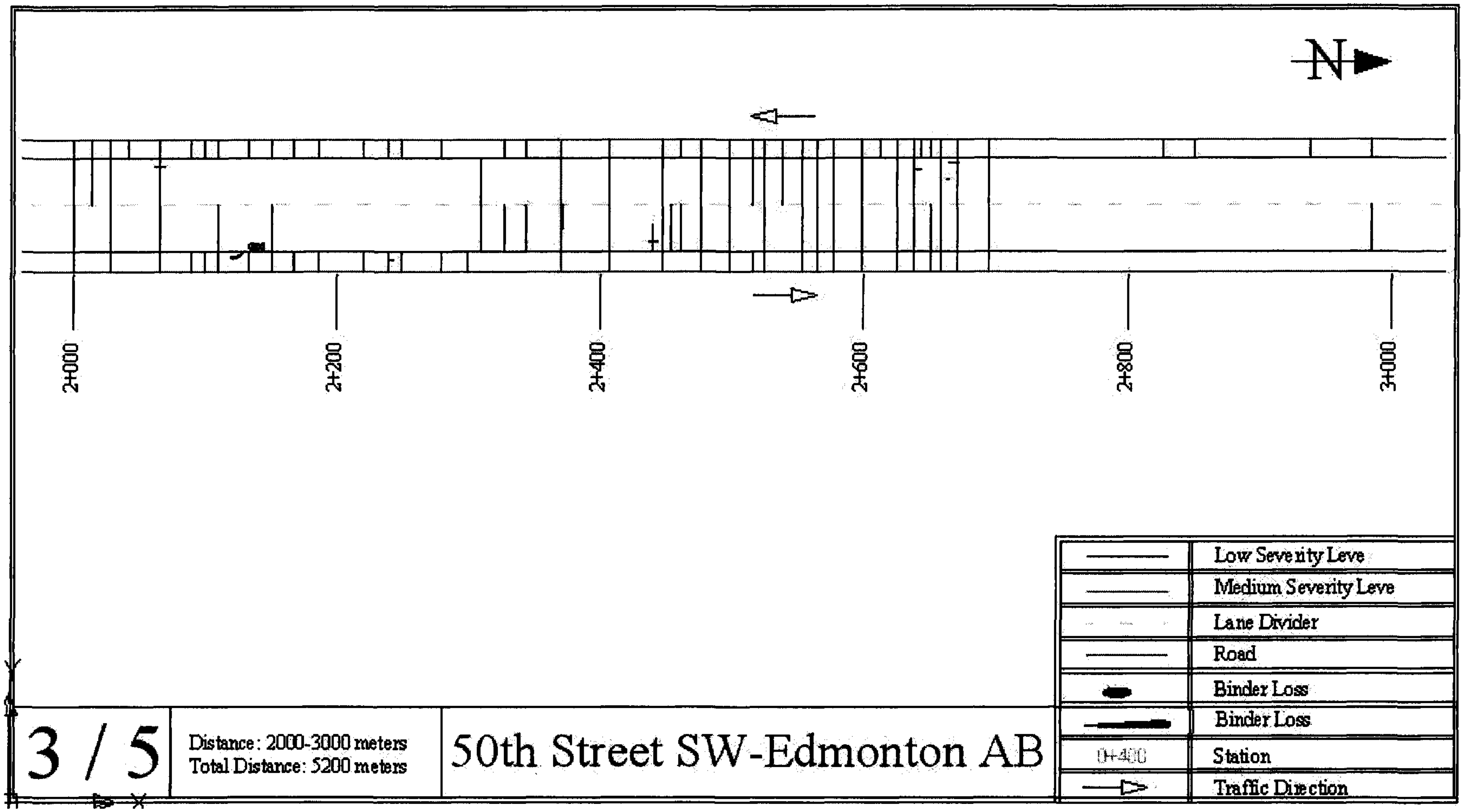

Figure 3.28C Crack measurements map at the 50 Street project locations (2000-3000 m)

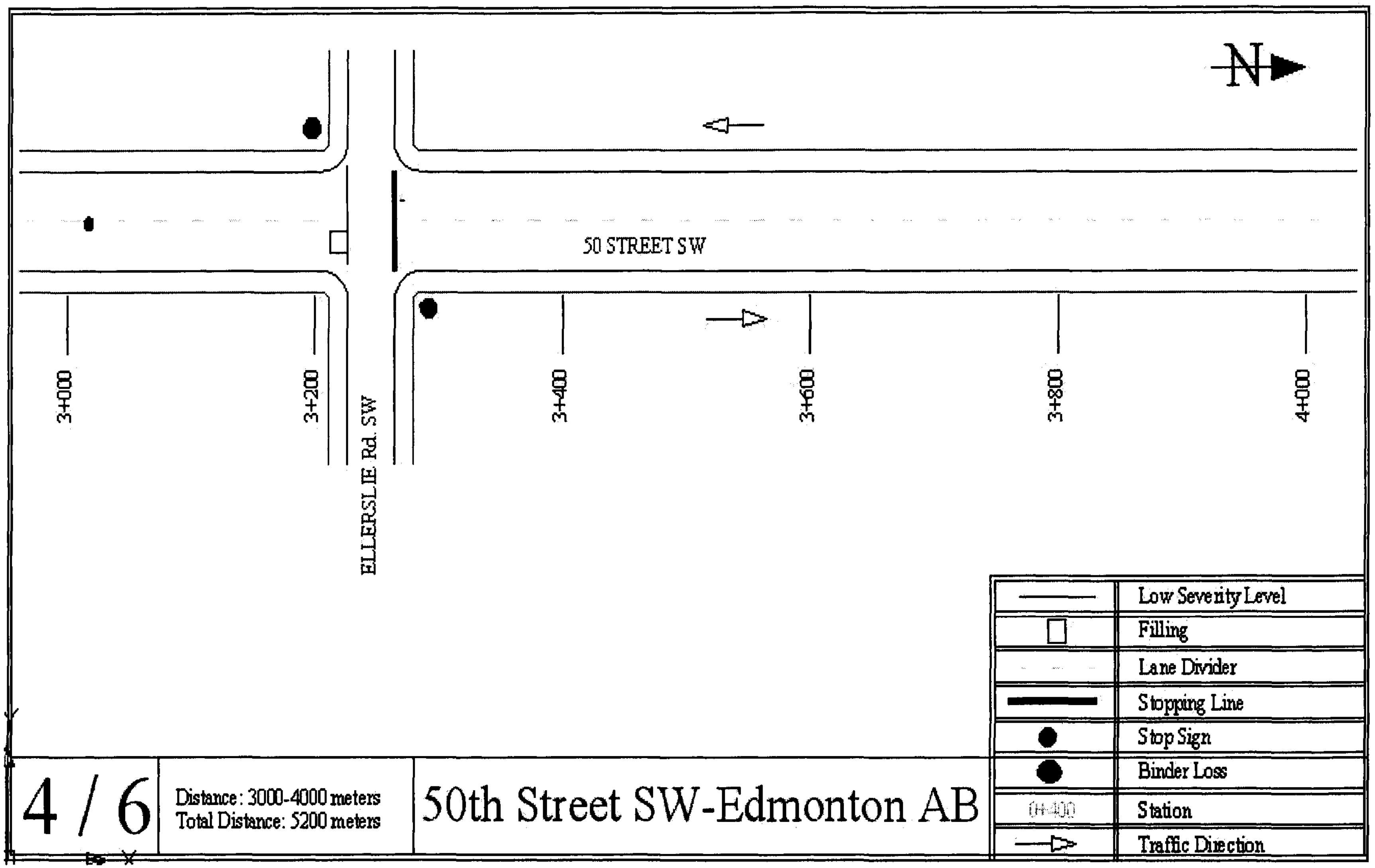

Figure 3.28D Crack measurements map at the 50 Street project locations (3000-4000 m)

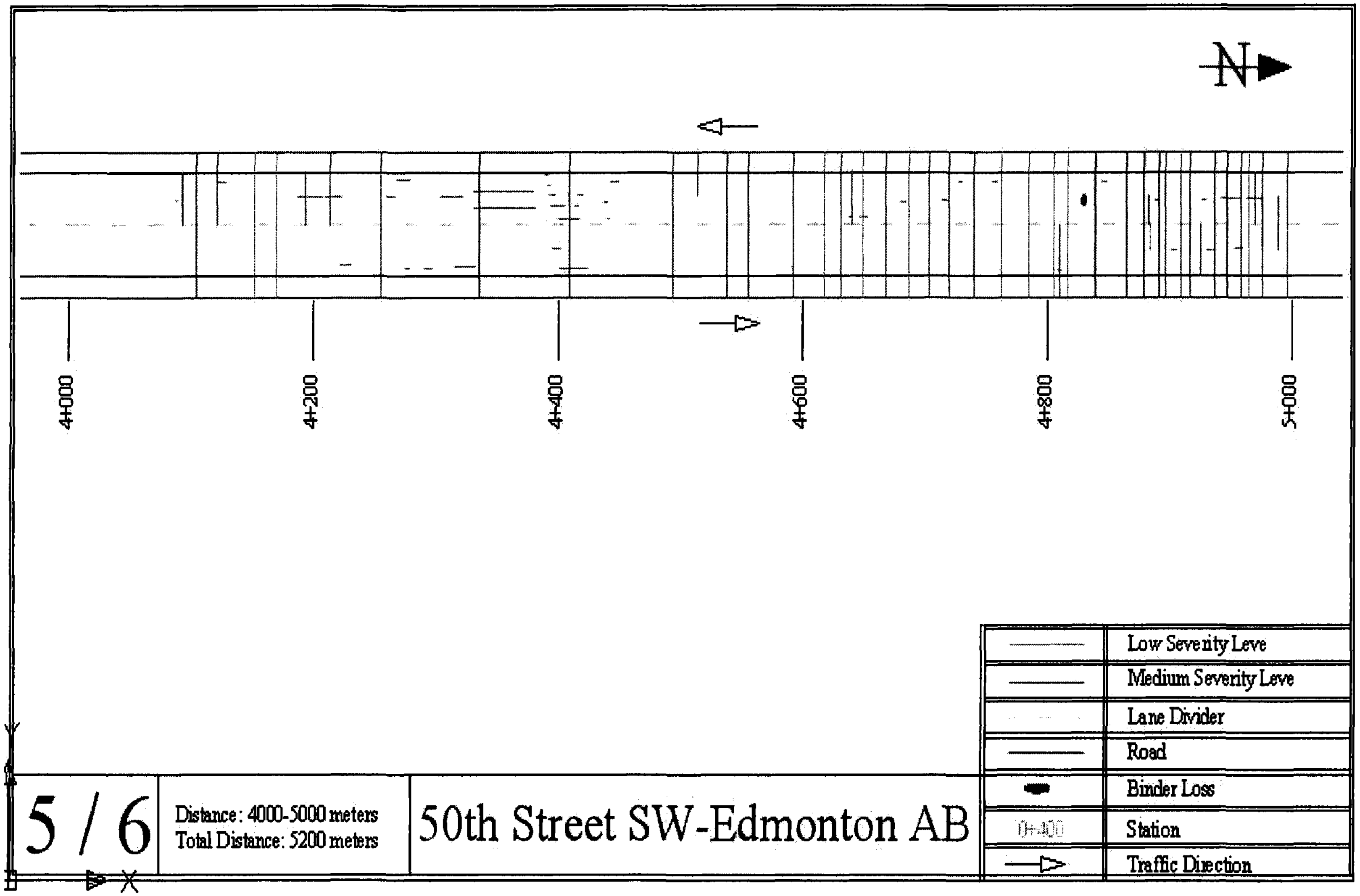

Figure 3.28E Crack measurements map at the 50 Street project locations (4000-5000 m)

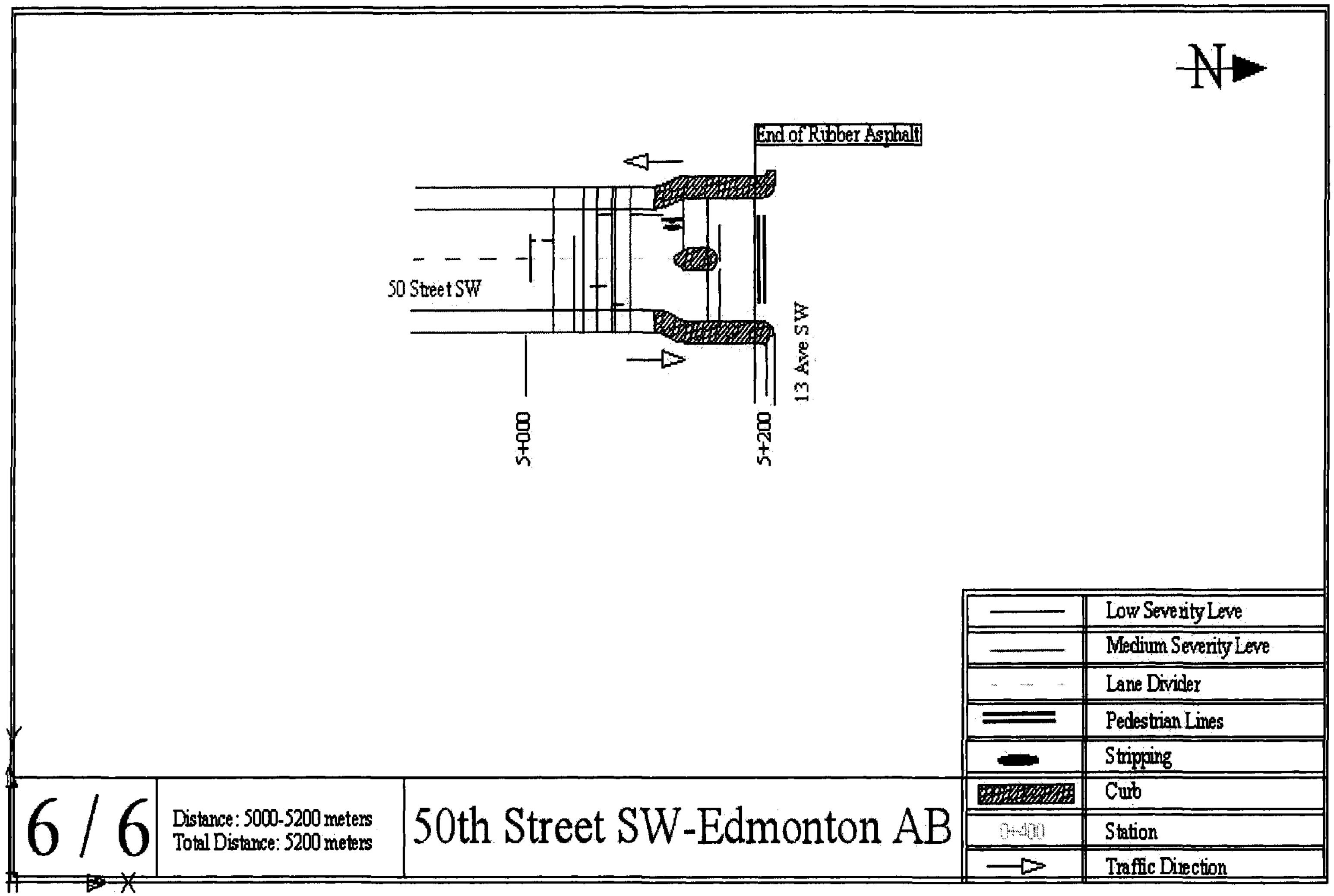

Figure 3.28F Crack measurements map at the 50 Street project locations (5000-5200 m)

3.4.6 Stony Plain Road (2003)

In the Stony Plain Road project location, one pre-construction and two post-construction IRI and rut measurements were collected. The post-construction measurements were made on October 19, 2003 and October 26, 2005 respectively. In addition to the IRI and rut measurements, crack and surface distresses were recorded manually at this project location during the summer of 2005.

Both the IRI and rut measurements at this project location appear to be slightly higher compared to the 50 Street project location.

Figures 3.29 and 3.30 present the summary of the IRI and rutting measurements at this project location.

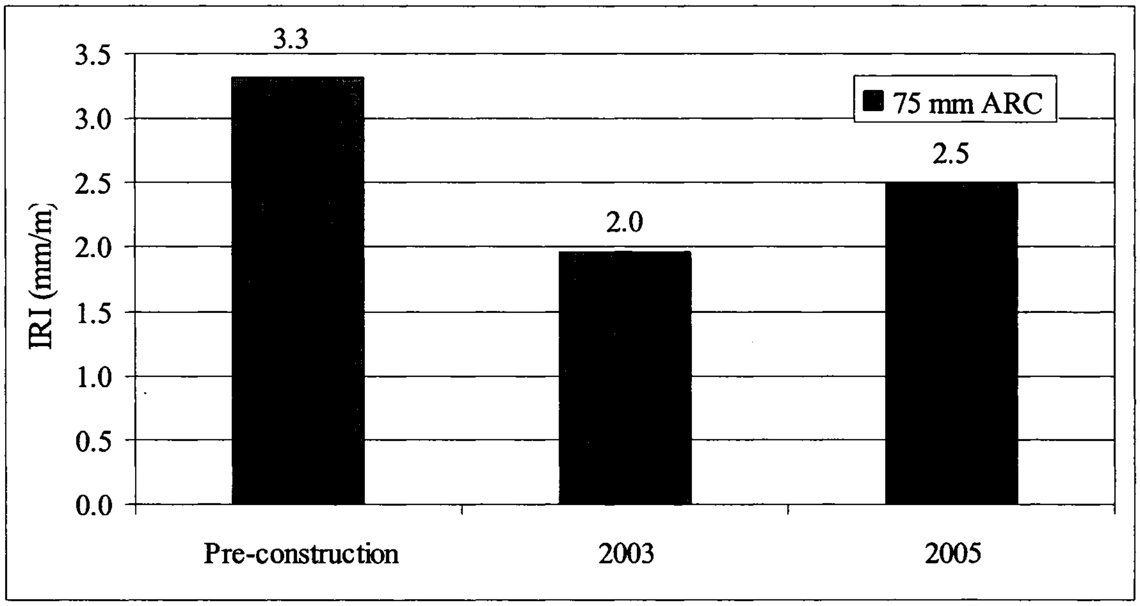

Figure 3.29 Summary of the IRI measurements at the Stony Plain Road

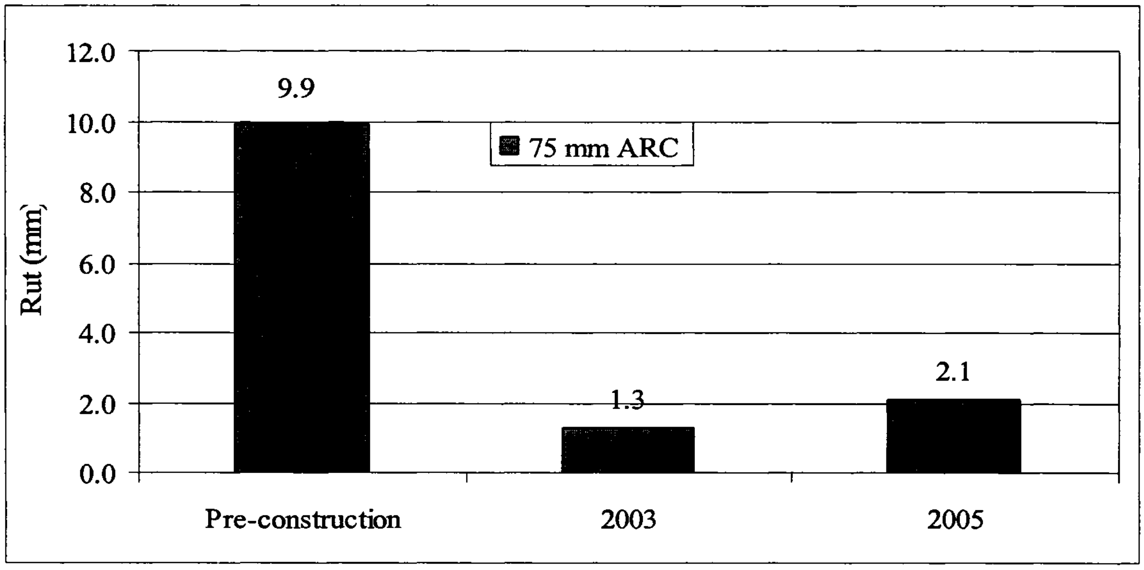

Figure 3.30 Summary of the rut measurements at the Stony Plain Road project location

Table 3.25 summarizes the crack measurements and other surface distresses conducted at this project location during the summer of 2005. The crack map for this project location is presented in Figure 3.31.

Table 3.25 Summary of the distress measurement at the Stony plain project location

Distress Type	Severity	Distress/100 lane m
Longitudinal crack (m)	Low	13.49
	Medium	2.56
	High	1.18
	Total	17.24
Transverse crack (m)	Low	9.75
	Medium	8.18
	High	0.00
	Total	17.93
Patch (m^2)		1.18
Moisture disintegration (m^2)		5.14
Raveling (m^2)		0.14
Fatigue cracking (m^2)		6.83

The ARM pavement section, at this project location, showed the highest occurrence of surface distresses compared to other pavement sections paved in 2003. Both transverse and longitudinal distresses were distributed uniformly along the length of the ARM section. Significant amounts of moisture damage were observed at this project location. A majority of the moisture damages were concentrated on the edge of the pavement and close to the intersections. As shown in Figure 3.31, a portion of the pavement close to the Edmonton Transit Services (ETS) bus terminal had failed shortly after paving because of fatigue cracking and showed substantial surface depression. High load and low speed clearly showed the limitation of ARM at highly stressed location.

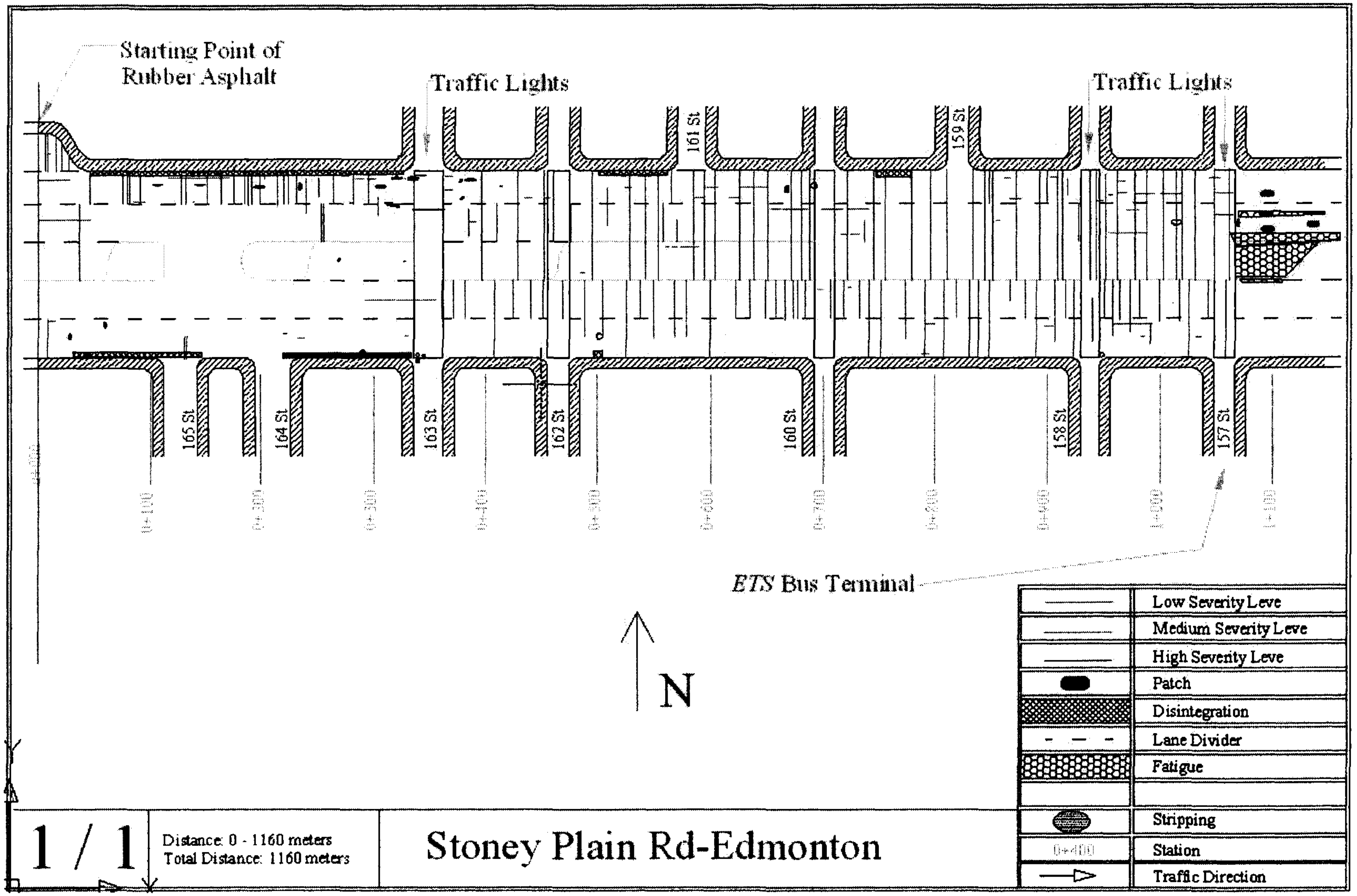

Figure 3.31 Crack measurements map at the Stony Plain Road

3.4.7 111 Avenue project (2003)

In the 111 Avenue project location, one pre-construction and two post-construction IRI and rut measurements were collected. The post-construction measurement was made on October 19, 2003. In addition to the IRI and rut measurements, the crack and surface distresses were measured manually at this project location during the summer of 2005.

The IRI measurement, at this project location, immediately after the construction was the highest compared to any other 2002 and 2003 ARM pavement sections.

Figures 3.32 and 3.33 present the summary of the IRI and rut measurements at this project location.

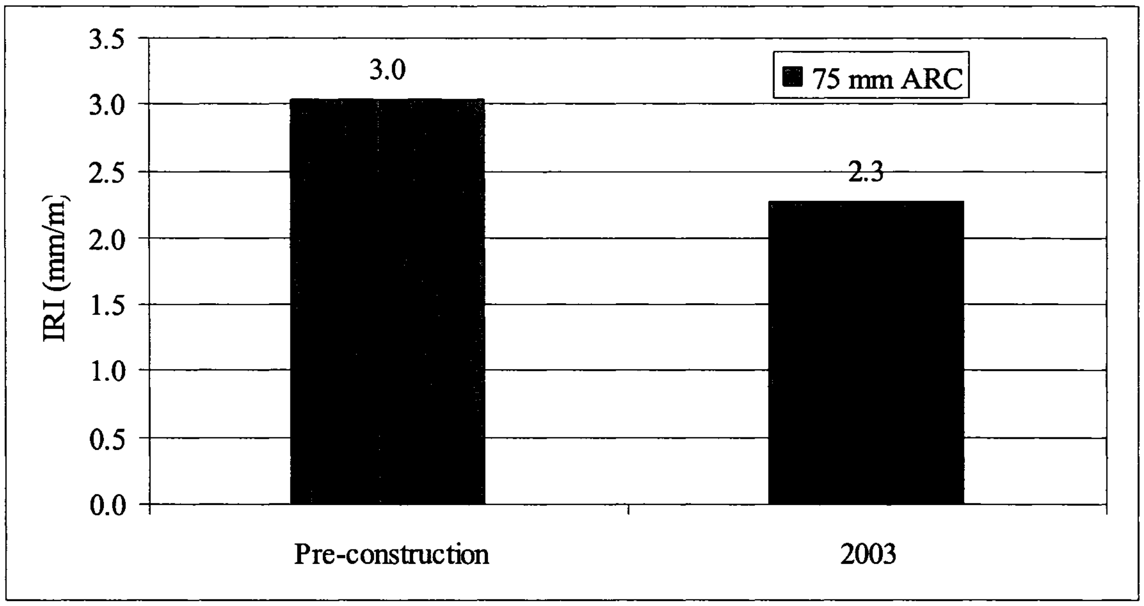

Figure 3.32 Summary of the IRI measurement at the 111 Avenue project location

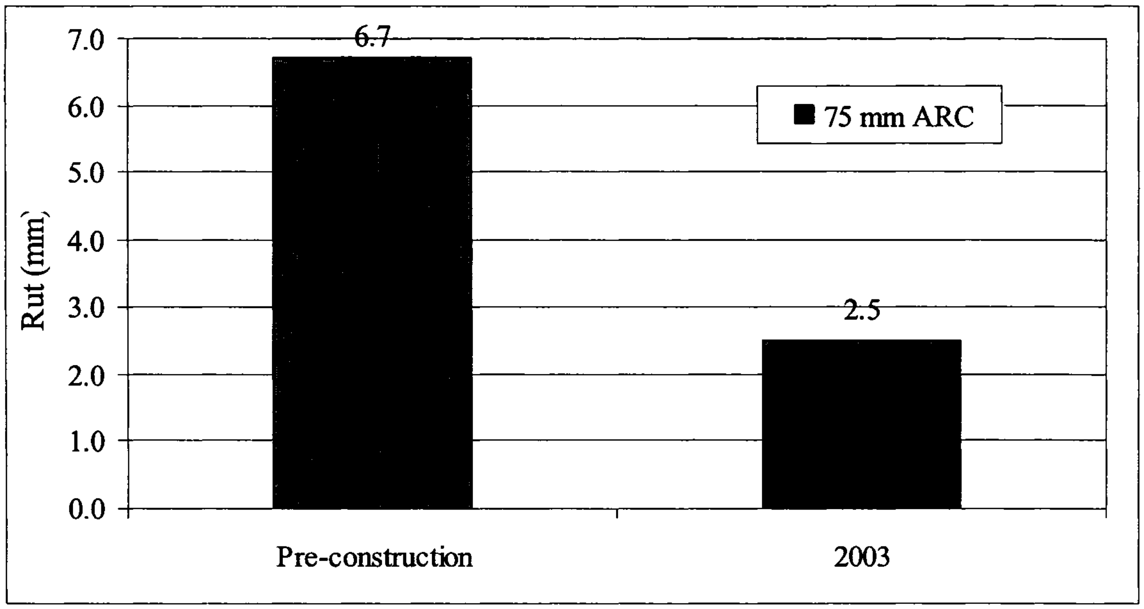

Figure 3.33 Summary of the rut measurement at the 111 Avenue project location

Table 3.26 summarizes the crack and surface distress measurements at this project location in the summer of 2005. The distress map for this project location is presented in Figure 3.34.

Table 3.26 Summary of the distress measurement at the 111 Avenue project location

Distress Type	Severity	Distress/100 lane m
Longitudinal crack (m)	Low	3.59
	Medium	11.49
	High	0.00
	Total	15.08
Transverse crack (m)	Low	16.19
	Medium	10.69
	High	0.37
	Total	27.25
Patch (m^2)		1.36
Moisture disintegration (m^2)		3.64
Raveling (m^2)		1.38

The distress measurement at this project revealed a high occurrence of moisture related surface distresses (moisture damage, raveling). Most of the moisture damage and raveling were concentrated close to signalized intersections or the pavement edge. It is to be noted that the moisture damage was not significant in the 2002 Asphalt Rubber project locations. Transverse and longitudinal cracking were also significantly present at this project location.

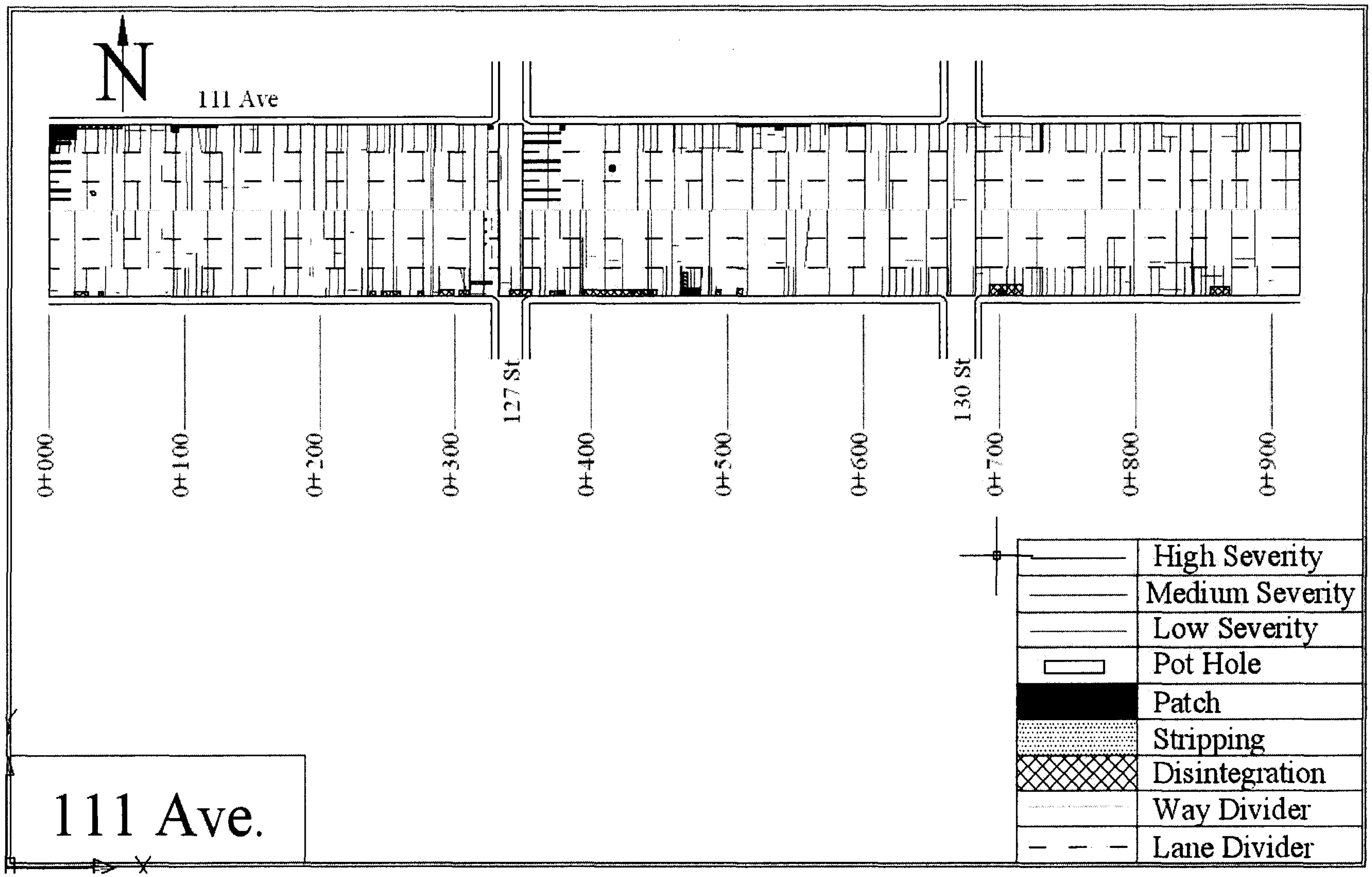

Figure 3.34 Crack measurements map at the 111 Avenue

3.4.8 Baseline Road – Chelsea Way to Highway 21 (2003)

In the Baseline Road project location, a pre-construction and two IRI and rut measurements were collected. The post-construction measurement was made on October 27, 2003 and October 24, 2005 respectively. In addition to the IRI and rut measurements, the crack and surface distresses were measured manually at this project location during the summer of 2005.

The post-construction IRI measurements, at this project location, show a slight increasing trend. The rutting measurements, however, showed a decreasing trend. Again, this could be attributed to the change from one measurement to another and the fact that the accuracy of rut measuring equipment is $\pm$1 mm.

Figures 3.35 and 3.36 present the summary of the IRI and rut measurements at this project location.

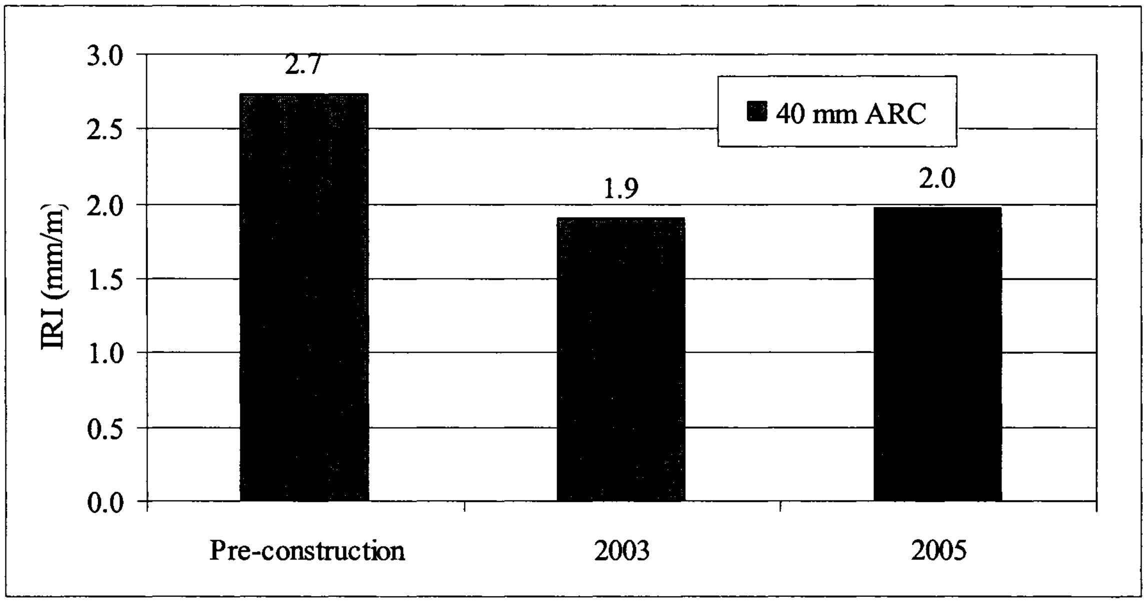

Figure 3.35 Summary of IRI measurement at the Baseline Road (2003) project location

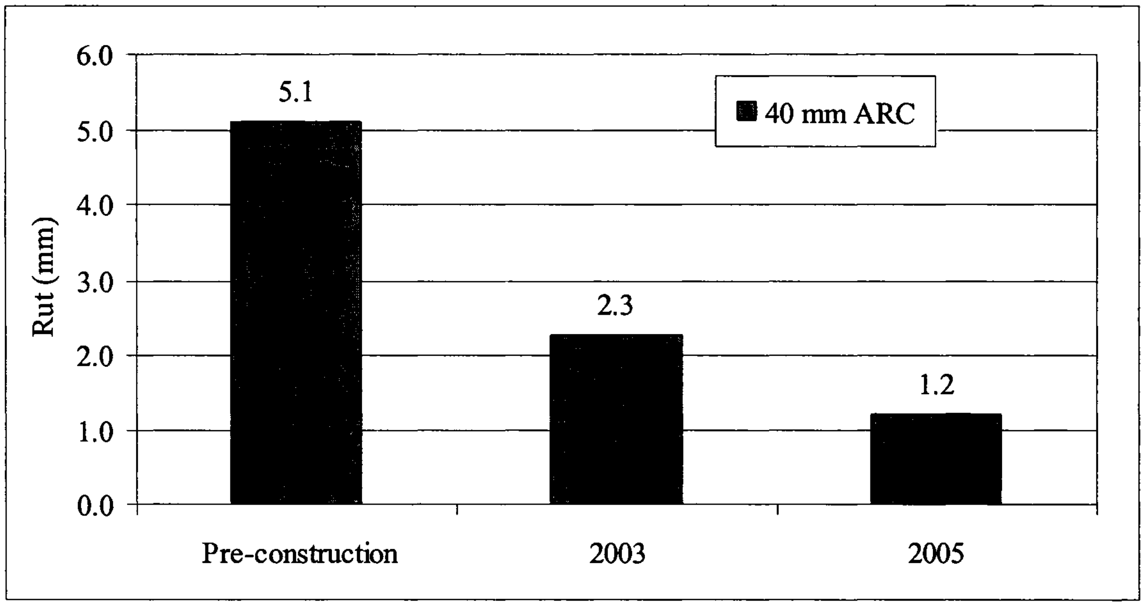

Figure 3.36 Summary of rut measurement at the Baseline Road (2003) project location

Table 3.27 summarizes the crack and distress measurement at this project location conducted during the summer of 2005. The crack map for this project location is presented in Figure 3.37.

Table 3.27 Summary of distress measurement at the Baseline (2003) project location

Distress Type	Severity	Normalized distress/100 lane m
Transverse Cracks (m)	High	0.00
	Medium	1.74
	Low	0.80
	Total	2.54
Longitudinal Cracks (m)	High	0.00
	Medium	0.09
	Low	1.75
	Total	1.84
Ravelling (m^2)		0.23
Moisture Disintegration (m^2)		0.25
Patching (m^2)		0.39

Based on the visual measurement conducted during the summer of 2005, cracking does not appear to be a significant problem at this project location. However, as shown in Figure 3.37, moisture related distresses were observed close to the signalized intersections (intersection with Cloverbar road). The moisture related surface distresses at this project location are not as significant as in the Stony Plain Road or the 111 Avenue project locations.

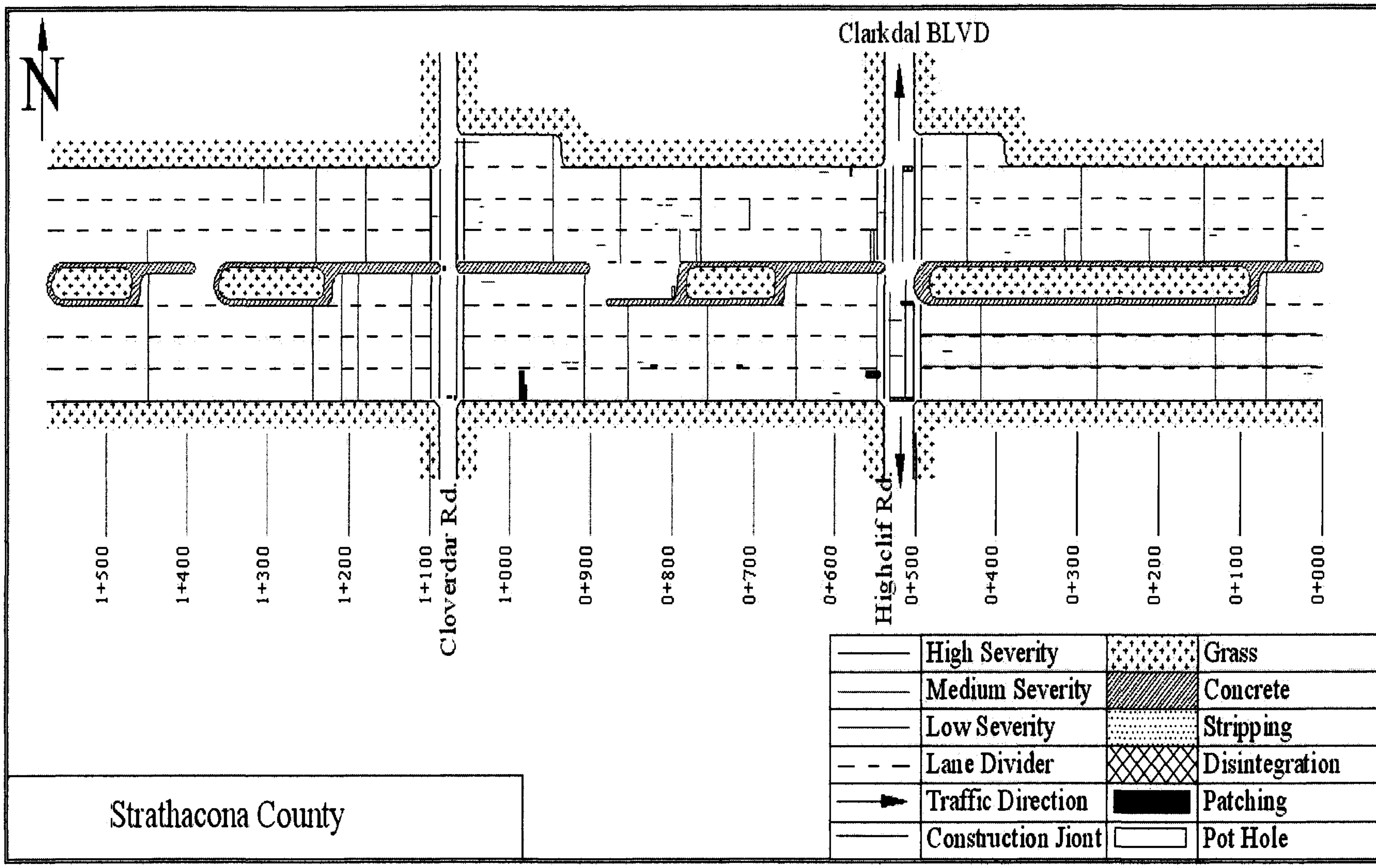

Figure 3.37 Crack measurements map at the Baseline Road (Chelsea Way to Highway 21)

3.4.9 Kaska Road (2003)

In the Kaska Road project location, one pre-construction and one post-construction IRI and rut measurements were collected. The post-construction measurement was made on October 27, 2003. In addition to the IRI and rut measurements, the crack and surface distresses were measured manually at this project location during the summer of 2005. Figures 3.38 and 3.39 present the summary of the IRI and rut measurements at this project location.

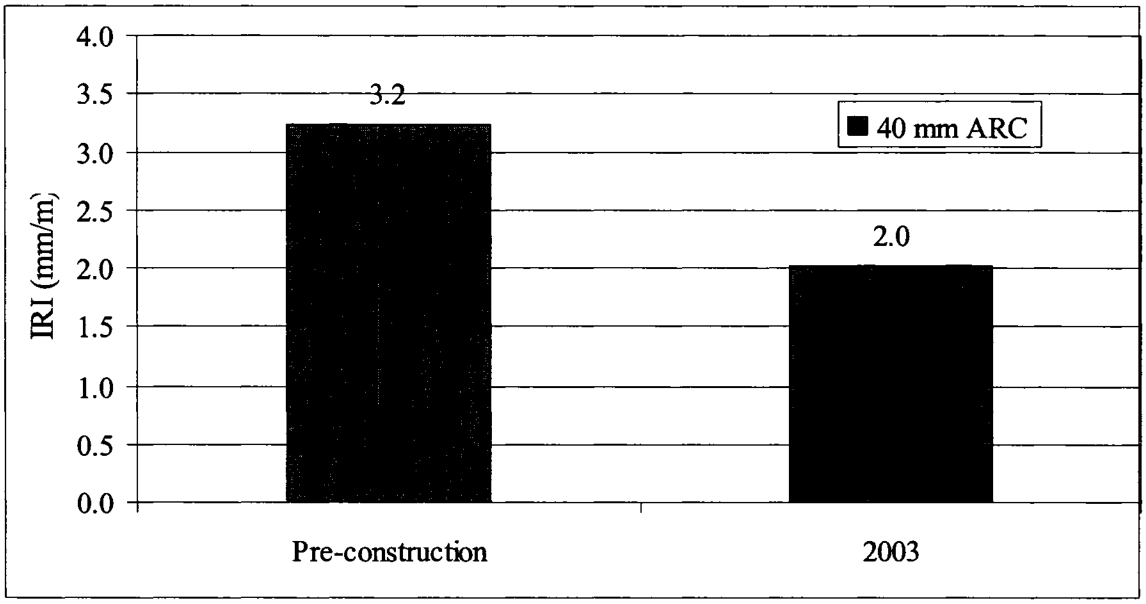

Figure 3.38 Summary of the IRI measurement at the Kaska Road project location

As illustrated the IRI measurement immediately after construction for this project location is almost close to 2 mm/m which can be considered slightly higher for a newly constructed pavement section.

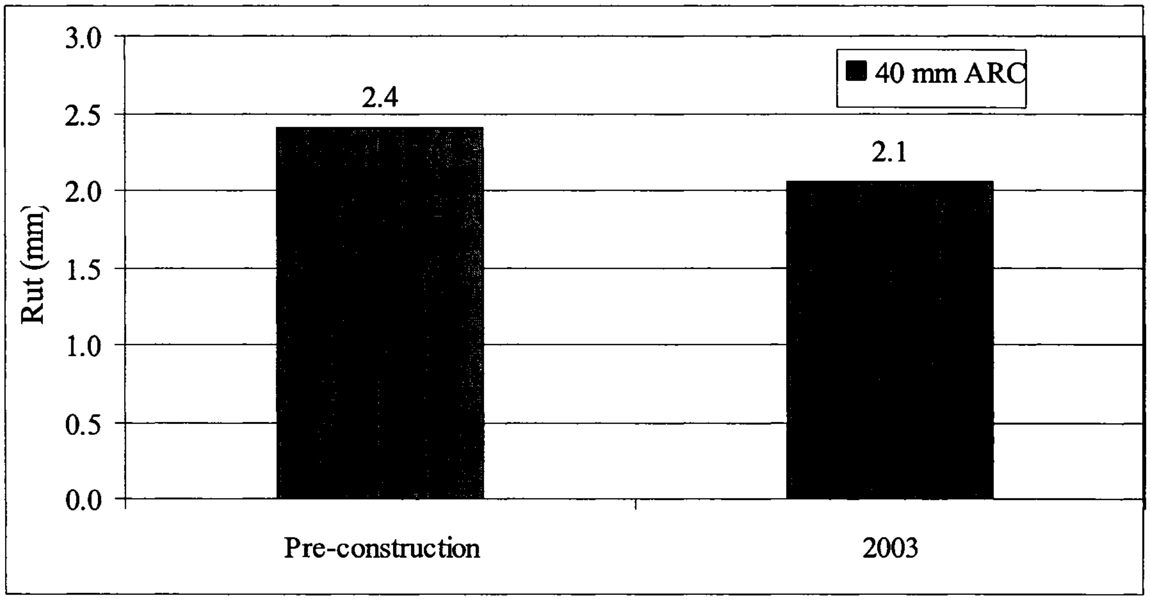

Figure 3.39 Summary of the rut measurement at the Kaska Road project location

Table 3.28 summarizes the crack measurements conducted at this project location during the summer of 2005. The crack map for this project location is presented in Figure 3.40.

Table3.28 Summary of the distress measurement at the Kaska Road project location

Severity	Normalized crack length (m)/100 lane m	
	Transverse Cracks	Longitudinal cracks
High	2.78	0
Medium	4.17	0.23
Low	5.58	6.13

Most of the cracks observed at this project location were of low severity. However, some medium and high severity transverse cracks were also observed. The other form of surface distresses (ravelling, pot holes, patches, stripping) were not observed at this project location.

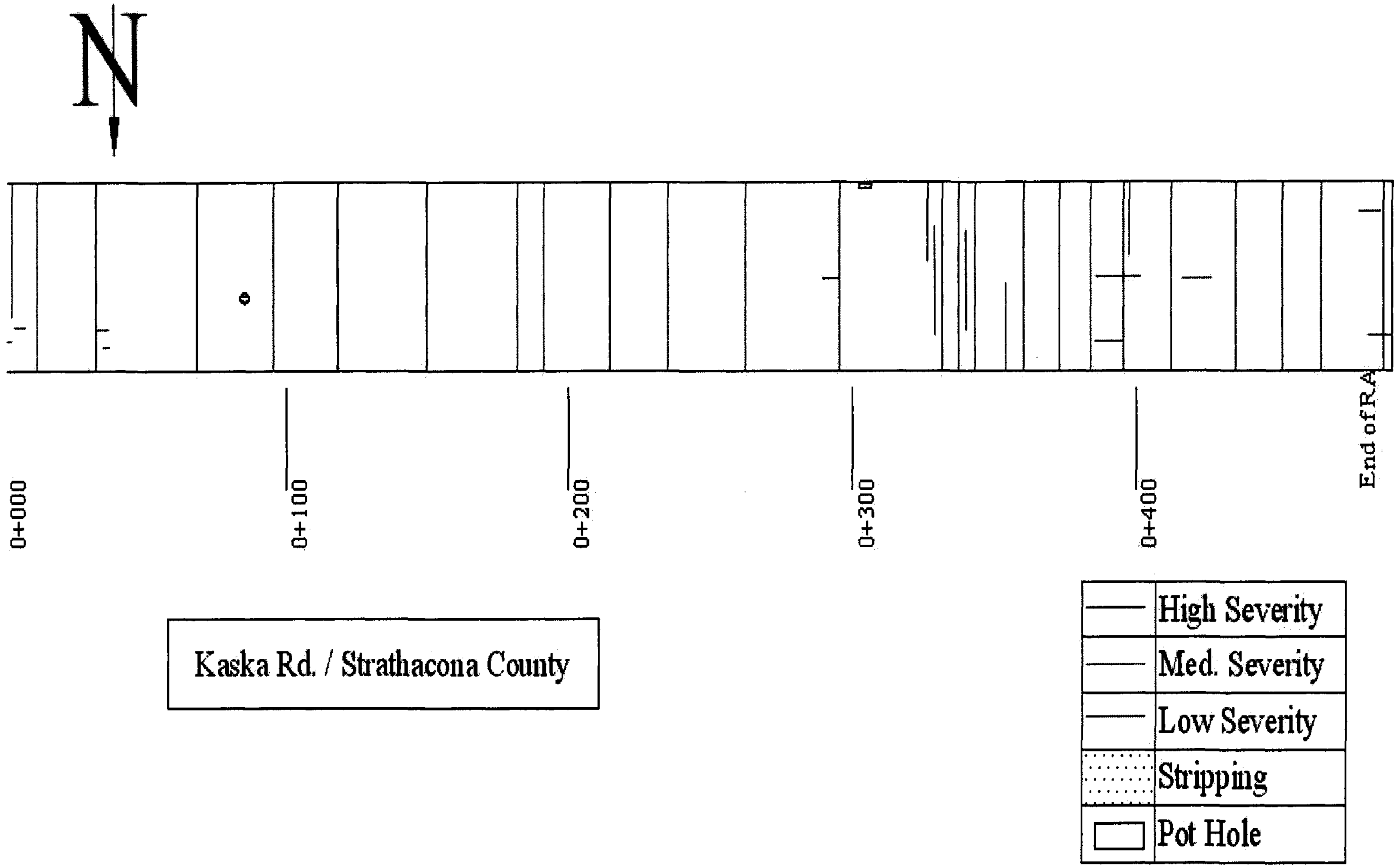

Figure 3.40 Crack measurements at Kaska Road project location

3.4.10 Highway 623 (2003)

In the Highway 623 project location, one pre-construction and two post-construction IRI and rut measurements were collected. The post-construction measurement was made on October 27, 2003 and November 9, 2005 respectively. Because of the location and length of this project, no crack measurements were made at this project location. Figures 3.41 and 3.42 present the summary of the IRI and rut measurements at this project location.

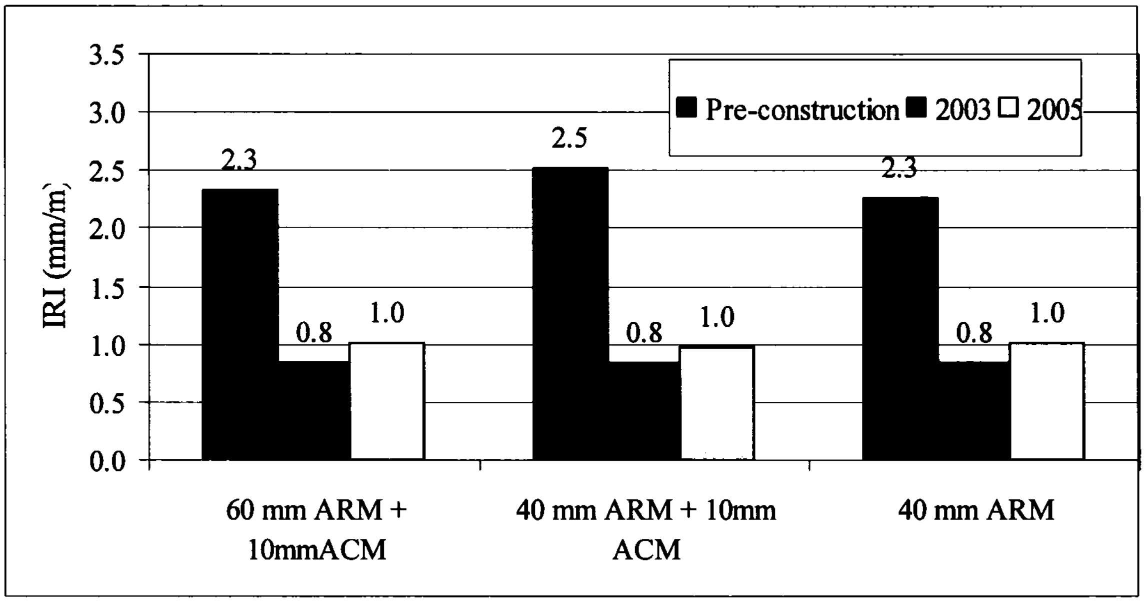

Figure 3.41 Summary of IRI measurement at the Highway 623 project location

The IRI measurement at this project location (for all pavement sections) is lower compared to the other project locations, which used the 2003 aggregate and the 2003 mix design. The increase in the IRI from 2003 to 2005 is also relatively small.

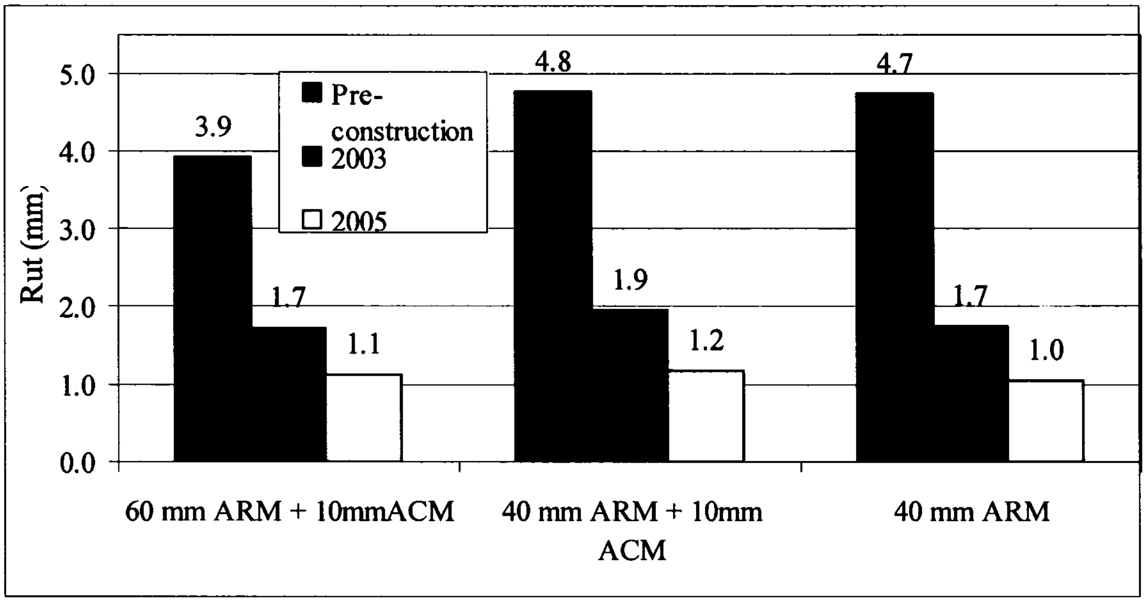

Figure 3.42 Summary of rut measurement at the Highway 623 project location

The rut measurement, however, is showing a decreasing trend like some of the other projects. This can be attributed to the fact that the accuracy of the measuring equipment is ±1 mm.

In the visual examination of the pavement sections at this project location, no significant distresses were observed. However, a low severity loss of aggregates was observed in some locations. Some low severity longitudinal cracks were also observed in some locations.

3.4.11 Summary of the performance of 2002 and 2003 Alberta Asphalt Rubber projects

It is definitely too early to comment on the performance of ARM overlays in Alberta's weather conditions. In the 2002 projects, both ARM and ACM sections were paved for comparing the performance of ARM sections with ACM sections. The comparison of IRI, rutting and crack measurements between the ARM and ACM section suggests the performance of ARM not to be superior to the ACM. Based on the measurements thus far, the half thickness ARM section does not appear to provide performance comparable to full thickness ACM sections, as were reported by some transportation agencies in the United States.

In the 2003 projects, only ARM overlays were paved and no ACM sections were paved for performance comparison. Some locations of the ARM sections, paved in 2003, have shown signs of severe distress and early failure. As previously mentioned the lower binder content might be responsible for the early signs of severe distress.

Although four different aggregate gradations were used in the 2002 and 2003 Alberta Asphalt Rubber project, it is very difficult to make a fair comparison of the performance of the pavement sections using these different aggregate gradations. This question cannot be answered from the field, as performance of the pavement sections is a function of many factors. In addition, measurements for the 2002 and 2003 project locations have been made at the different age of the pavement sections. For instance the measurement for the 2002 project locations are taken immediately, as well as one year and three years after construction while the measurement for the 2003 project locations are taken immediately and two years after construction.

A laboratory experiment was conducted at the University of Alberta to study the impact of aggregate gradations on the performance of ARM. The results of the study are presented in Chapter four of this thesis. The subsequent paragraphs present the summary of the pavement performance of different ARM pavement sections paved in 2002 and 2003.

3.4.11.1 IRI Comparison

Table 3.29 ranks the IRI measurement at the 2002 and 2003 project locations.

Table 3.29 Summary of IRI measurement of Alberta Asphalt Rubber 2002 and 2003 projects

Pavement performance based on IRI measurement	2002 Project locations	2003 project locations
Superior	80 mm ARM, Highway 630	40 mm ARM, Highway 623
Inferior	40 mm ARM, 112 Avenue	75 mm ARM, 111 Avenue

In the 2002 projects, the half thickness ARM overlays have performed inferiorly compared to the full thickness ARM overlays. Figure 3.43 presents the comparison of IRI measurements of pavement sections using different aggregate gradations. The comparison takes into account the age of the pavement sections.

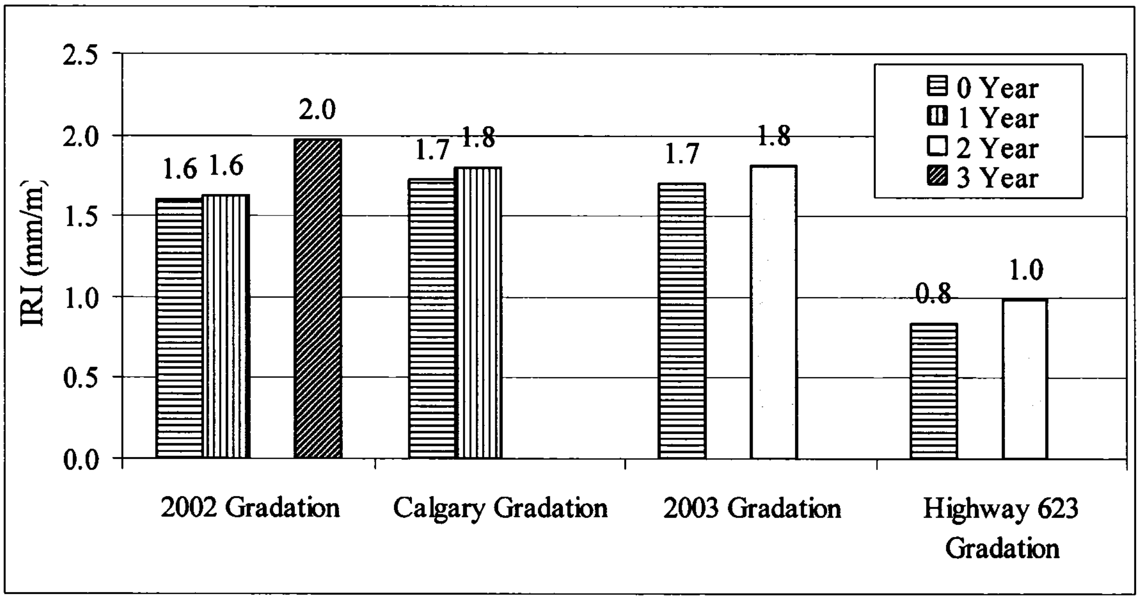

Figure 3.43 Comparison of the IRI for different aggregate gradation used in 2002 and 2003 Asphalt Rubber project

Based on the IRI comparison, the pavement sections in the Highway 633 project location have superior performance compared to pavement sections using other aggregate gradation. It is to be noted that the binder design for this project location was also different compared to other project locations. The performance of the project locations, using 2002 aggregate gradation (and mix design) and project locations using 2003 aggregate gradation (and mix design), is similar. However, other than the 50 Street project location the IRI performance of pavement sections using 2003 aggregate gradation and mix design was inferior compared to other project locations.

3.4.11.2 Rutting

The rutting measurement in some of the project locations (17 Street, 137 Avenue, 50 Street, Baseline Road [Chelsea Road-Highway 21] and Highway 623) showed decreasing rutting trends. Table 3.30 ranks the rut measurements on the 2002 and 2003 project locations.

Table 3.30 Summary of rut measurement of Alberta Asphalt Rubber 2002 and 2003 projects

Pavement performance based on Rut measurement	2002 Project locations	2003 project locations
Superior	40 mm ARM, 112 Avenue	75 mm ARM, Stony Plain Road
Inferior	30 mm ARM, 17 Street	75 mm ARM, 111 Avenue

The comparison was made based on the measurements from 2003. In the 2002 projects, except for the 112 Avenue project location, the half thickness ARM pavement have performed inferior compared to the full thickness ARM overlays from a rutting perspective.

Figure 3.44 compares the rut measurements of pavements sections using different aggregate gradation. The comparison takes into account the age of the pavement.

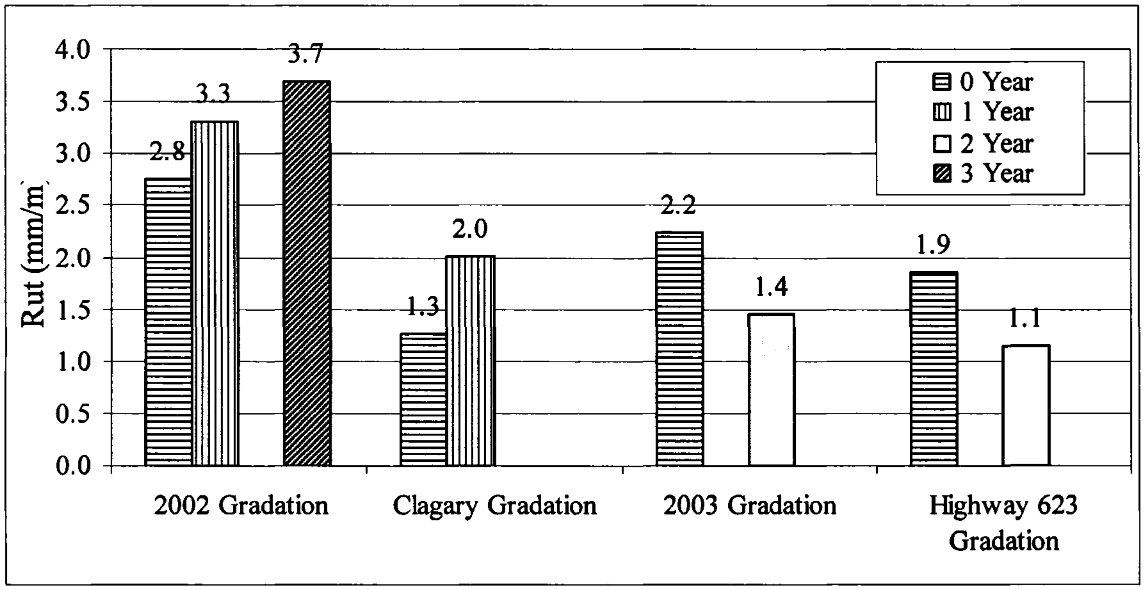

Figure 3.44 Comparison of the Rut for different aggregate gradation used in 2002 and 2003 AR project

3.4.11.3 Crack and surface distress measurements

The visual condition survey of the 2002 and the 2003 project location conducted in summer of 2004 and 2005 revealed alarming observations in some of the 2003 project locations. Some locations of 111 Avenue, Stony Plain Road and Baseline Road (Chelsea Way to Highway 21) were severely affected by moisture damage. A portion of Stony Plain Road close to ETS bus transit failed because of heavy rutting and fatigue cracking. Most of the moisture damage in these project locations were observed in pavement edge and signalized intersections. It is to be noted that all the above mentioned project locations utilized the same aggregate gradation and mix design. The recommended binder content for this project location was also less (6.9%) compared to other project locations. Considering the relatively young age of the pavement section, this premature moisture related surface distresses demand further investigation. Among the 2003 project locations, 111 Avenue and Stony Plain Road had the most occurrence of transverse and longitudinal cracking. Among the 2002 projects, the 137 Avenue project location was most severely affected by cracking. The uniformly spaced transverse cracks at the 137 Avenue project location indicated that the cracks might be a reflection of the cement treated joints underneath. The ARM sections in the 112 Avenue project location seem to be the least affected by cracking and other surface distresses.

A laboratory experiment was conducted at the University of Alberta to investigate the lower asphalt content used in the 2003 project. The results of laboratory study are presented in Chapter four.

CHAPTER 4 LABORATORY EXPERIMENTS FOR ALBERTA ASPHALT RUBBER MIX

Visual inspections during the summer of 2005 showed signs of moisture damage in several ARM sections such as a) Stony Plain Road b) 111 Avenue and c) Baseline Road (Chelsea Way to Highway 21). Figures 3.31, 3.32 and 3.33 reveal that most of these surface distresses are located close to locations such as intersections and pavement edges, which are more likely to accumulate water. It is to be noted that all three projects utilized the same aggregate gradations and mix designs which were explained in Chapter 3. The 2003 aggregate gradation and mix design appears to be different from other primarily in the following aspects; binder content and openness of the aggregate gradation.

a) Binder content: The binder content for these project locations (6.86%) was relatively less than the binder content of 8.9%, 7.6% and 7.9% used for the 2002, Calgary and Highway 623 projects respectively. The lower asphalt content could be responsible for the moisture damage at the above mentioned project locations.
b) Aggregate gradation: Although there is little difference in the aggregate gradation for all the 2002 and 2003 Alberta Asphalt Rubber projects, the 2003 project was more open compared to the other project location. It is safe to make a hypothesis that open-graded ARM, compared to dense-graded ARM, allows water to permeate, and as a result, remain inside for a prolonged time. This increases the possibility of severe water damage due to prolonged interaction with water and freeze thaw cycles.

A laboratory experiment was conducted at the University of Alberta to investigate the lower asphalt content used in the 2003 project and to study the impact of aggregate gradation on the moisture sensitivity of ARM sections.

This chapter presents an introduction to moisture damage of asphalt mixtures, the design of experiment and the results of the laboratory experiments.

4.1 Objectives of the laboratory experiment

The objective of the laboratory experiment was to reconfirm or reject the lower asphalt content used in the 2003 Alberta Asphalt Rubber project and to study the impact of aggregate gradations on the moisture damage. Specifically, the objective of the experiment is as follows:

- Investigate the 2003 mix design for the binder content.
- Compare the moisture sensitivity of open and dense-graded ARM subject to freeze and thaw cycles in order to study the effect of gradation on the moisture sensitivity of ARM.

4.2 Moisture damage of asphalt mixtures

The proneness of asphalt mixtures to moisture damage is often referred to as moisture sensitivity. Moisture damage can be defined as the loss of strength or durability due to effects of moisture on the asphalt mixture (Little and Jones, 2003). Moisture related damage can appear in the form of bleeding, cracking, rutting, localized failures, ravelling and structural strength reduction (Hicks et al., 2003). The following six possible mechanism of moisture damage have been reported in literatures: displacement, detachment, spontaneous emulsification, pore-pressure induced damage, hydraulic scour and the effects of environment on aggregate-asphalt system (Little and Jones, 2003).

Moisture related distresses in pavement have been reported as one of the prime problems faced by pavement agencies. Many test methods have been developed to predict the moisture sensitivity of hot mix asphalt. The test for predicting moisture sensitivity dates back to as early as the 1920s. Tests used to estimate the moisture sensitivity can be categorized into the following categories: test on loose mixtures and test on compacted mixtures. Tests on loose mixtures are best used for the comparison between different aggregate-asphalt mixtures in terms of compatibility, strength of adhesion and stripping. The test on compacted mixtures takes into account the physical and mechanical properties of the mix, traffic actions and pore pressure effects. Different test methods under each category are reported in literatures (Solaimanian et al., 2003). Two widely used methods of tests on compacted mixtures are reported briefly in the following paragraphs:

Original Lottman indirect tension test

Originally developed by Lottman in the late 1970s, this test requires one set of dry samples and one set of conditioned samples. Conditioning of the samples includes both vacuum saturation (under 26 inch mercury for 30 minutes) followed by saturation at atmospheric pressure for 30 minutes. The partially saturated samples are frozen at -18°C for 15 hours, followed by 24 hours at a water bath maintained at 60°C. This is considered too be accelerated freeze thaw cycles. Lottman proposed a thermal cyclic conditioning as an alternative. Each cycle consisting of 4 hours of freezing at -18°C followed by 4 hours at water bath maintained at 49°C. Conditioned and dry specimens are both tested for tensile resilient modulus and tensile strength using indirect tensile equipment. The loading rate is 0.065 in/min for testing at 13°C or 0.150 in./min for testing at 23°C. The severity of moisture sensitivity is judged on the basis of the ratio of test values for conditioned and dry specimens (Solaimanian et al., 2003).

AASHTO T283 (Modified Lottman Indirect Tension Test Procedure)

The AASHTO T283 test is one of the most commonly used procedures for determining HMA moisture susceptibility. The test is similar to the original Lottman with a few exceptions. One of the modifications is that the vacuum saturation is continued until a saturation level between 70% and 80% is achieved, compared with the original Lottman procedure that required a set time of 30 minute. Another change is in the test temperature and loading rate for the strength test. The modified procedure requires a loading rate of 2 in/min at 25°C rather than 0.065 in/min at 13°C (Solaimanian et al., 2003, AASHTO T283, 2005).

Some highway agencies believe that these methods (along with the other available methods) are not accurate in predicting the moisture sensitivity of the mixture and have developed their own methods for predicting the moisture sensitivity. Caltrans has adopted CTM 371 (modified T283) for assessing the moisture sensitivity in different climatic regions. In addition, Caltrans is actively involved in developing better methods for testing the moisture sensitivity. TxDOT specification requires the Hamburg Wheel-Tracking Device (HWTD) instead of a wet-dry retained TSR criterion similar to AASHTO T283. This departure from conventional moisture sensitivity tests is based on extensive research and field studies in Texas, which indicated that conventional tests are inadequate for performance prediction purposes while the HWTD is an effective tool to identify premature failures (Solaimanian et al., 2003).

The experiences of various states have indicated that an improved laboratory test or criterion to identify moisture sensitive HMA mixtures is urgently needed. Also, the larger states, in terms of HMA tonnage (California and Texas), are not satisfied with the repeatability and reproducibility of their selected version of AASHTO T283 (Solaimanian et al., 2003).

Along with the water, which is the primary cause in the moisture damage, anything that supports this might also be considered equally responsible. Dirty aggregates, presence of high dust content in aggregate, inadequate compaction and aggregate structure allowing water to remain for a longer period of time can be considered important factors influencing the moisture damage.

A high dust content in the aggregate gradation means that at the same void content the mixture will have less asphalt (thinner asphalt coating in the aggregate), thereby making the mix more susceptible to moisture damage. Inadequate compaction resulting in higher air voids then design air voids, or aggregate structure allowing water to remain for a prolonged time, increases the stripping potential of the mixture (Solaimanian et al., 2003).

Literatures suggest that there is no consensus on the influence of AR on the moisture sensitivity of the mix. Some literatures suggest that resistance to the moisture damage is improved by the addition of CR (Rebala and Estakhri, 1995, Harvey et al., 1995 and Palit et al., 2004), while others report AR not to increase the resistance to moisture damage (Shatnawi and Little, 2000 and Shatnawi et al., 1995).

Palit et al. (2004) found that the asphalt mix modified with 5%, 10% and 15% CR (passing ASTM 30 sieve) by weight of asphalt were found to have a better resistance to moisture damage than the conventional mix. The moisture resistance was judged based on the tensile strength and Marshall stability values of conditioned and unconditioned samples. The CR content of 10% gave the best result compared to other CR content.

Shatnawi et al. (1995) compared the TSR value of dense-graded and gap-graded ARM with the conventional dense-graded asphalt concrete. Two aggregate sources were considered in this study. The results of the study showed a lower TSR value for the gap-graded ARM compared to the conventional mix for both aggregate sources. However, the TSR value for the dense-graded ARM was better than that of conventional mix for one aggregate source. It is to be noted that the TSR value for both gap-graded and dense-graded ARM were below 80% for both aggregate sources.

In short, moisture damage has been accepted as one of the main reasons for poor performance of pavements. It appears the existing test procedures do not correlate well with the field performance, and transportation agencies are still looking for better tests to predict the moisture sensitivity of asphalt mixes. Although several researches are being conducted to develop a new method of testing for moisture sensitivity, T283 can be considered the most suitable test available.

A laboratory experiment was conducted to validate the design asphalt content of 2003 Alberta ARM and to evaluate the moisture resistance of it based on a change in its aggregate gradation.

4.3 Experiment design

In this experiment, five different aggregates a) 16 mm b) 12 mm c) 5 mm d) Manufactured fines and d) Washed manufactured fines, supplied by Inland, were used. Inland was also the aggregate supplier for the 2003 project locations in Edmonton and Strathcona County. The AR binder used for the laboratory experiment was provided by EBA Engineering Consultants and was the same as used in the 2003 project.

Figure 4.1 is a flow chart showing the important steps in this experiment design. In the subsequent paragraph, each element in the flowchart has been explained in detail.

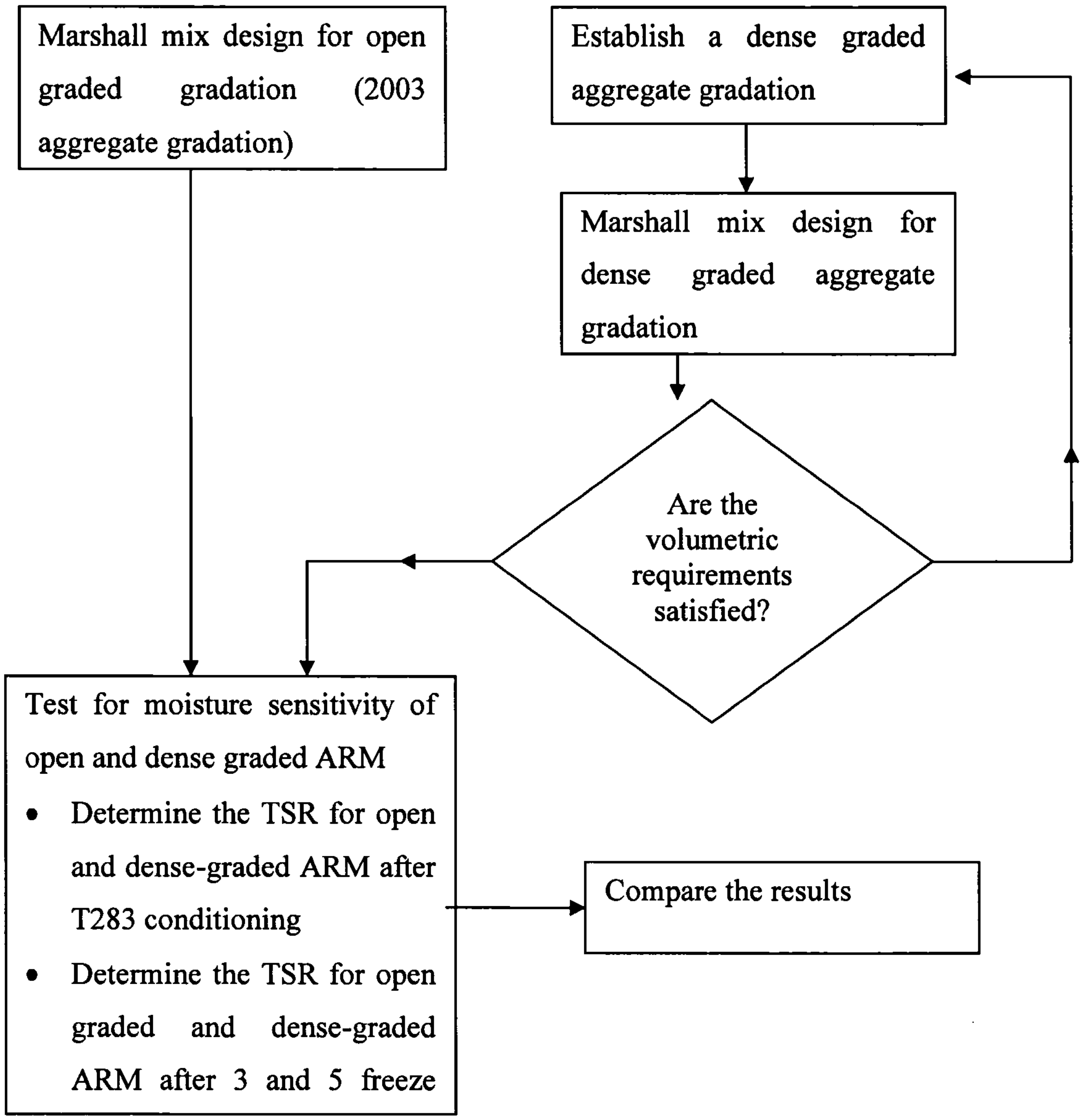

Figure 4.1 Flowchart showing experimental design

4.3.1 Marshall mix design for open-graded gradation (2003 aggregate gradation)

The first step in the experiment was to conduct a mix design to validate the 2003 mix design. Five different aggregate sizes were used to produce an open-graded aggregate gradation similar to the aggregate gradation used in the 2003 projects. Table 4.1 shows the aggregate gradation for each aggregate and their proportions to achieve the open-graded formulation similar to the one used in the 2003 project. Figure 4.2 compares the aggregate gradation for the 2003 aggregate gradation and the open-graded gradation formulated at the University of Alberta.

Table 4.1 Open-graded aggregate gradation formulated at the University of Alberta lab

Sieve Size (mm	Percent passing (%)					
	16 mm	12-10 mm	5 mm	Washed Fines	Manufactured Fines	Open-graded formulation
19.00	100.0	100.0	100.0	100.0	100.0	100.0
12.50	65.1	98.7	100.0	100.0	100.0	94.3
9.50	6.9	57.2	100.0	100.0	100.0	66.2
4.75	1.1	1.7	93.0	97.9	99.0	37.3
2.38	1.1	0.8	10.4	71.0	75.0	15.6
2.00	1.1	0.8	6.5	65.5	69.7	13.8
1.18	1.1	0.7	3.2	53.5	58.5	11.1
0.60	1.0	0.7	2.2	39.0	46.0	8.5
0.425	1.0	0.7	2.0	30.1	38.0	7.0
0.30	1.0	0.6	1.8	20.2	29.7	5.4
0.15	0.8	0.5	1.4	4.0	14.7	2.51
0.075	0.6	0.4	1.1	1.5	6.7	1.32
Proportion (%)	14.5	47.5	20.4	7.0	10.6	100.0

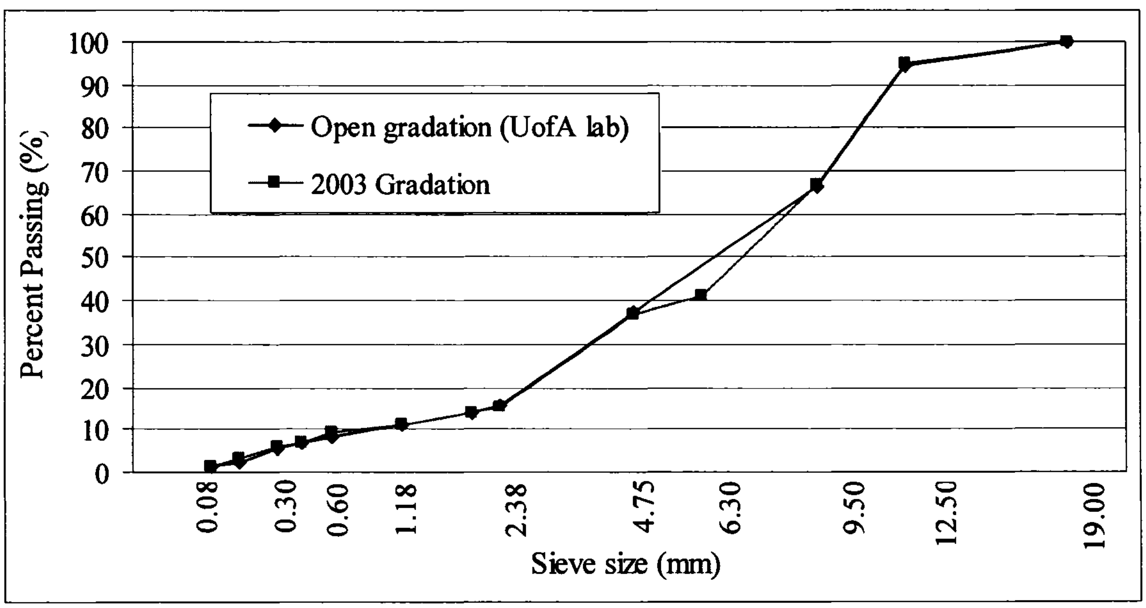

Figure 4.2 Comparison of the 2003 gradation and lab prepared open gradation

The AR binder used for the laboratory experiment was the same as the one used in the 2003 mix design.

Marshall specimens were prepared and tested in accordance with the AASHTO Designation T245-97 (75 blow). The mixing and compaction temperature were 173°C and 163°C respectively. The mixing and compaction temperature for the 2003 mix design was 163°C. Initially, for the laboratory experiment the mixing temperature of 163°C was selected. However, it was difficult to achieve proper mixing (the samples fell apart after 75 blows on one side during compaction) at this temperature and, hence, it was increased to 173°C to achieve proper mixing and compaction.

The specific gravity (G_{mb}) and maximum specific gravity (G_{mm}) of the ARM were calculated using the Corelok Device. The results from the mix design were compared with the data from the 2003 mix design and are presented in section 4.4.

4.3.2 Establish a dense-graded aggregate gradation

In order to study the effect of the aggregate gradation on the moisture sensitivity, it was essential to establish a gradation that is closer than the open-graded ARM described in Section 4.2.1. A 12.5 mm nominal size dense-graded aggregate gradation was formulated using three different aggregates. The aggregates used to produce the dense-graded gradation were supplied by Inland. Table 4.2 shows the gradation of the individual aggregate and their proportion used to achieve the dense-graded formulation. Figure 4.3 shows the dense-graded aggregate gradation used in this experiment. Some highway agencies believe that it is not possible to satisfy the required volumetric properties when CR is used in a dense-graded mix. The Bailey method (Vavrik et al., 2002) was used to analyze the dense-graded formulation in the lab.

Table 4.2 Aggregate gradation for the dense-graded formulation

Sieve size	Percent Passing (%)			
(mm)	12.5 mm	5 mm	Manufactured fines	Combined Blend
19.00	100.0	100.0	100.0	100.0
12.50	98.7	100.0	100.0	99.5
9.50	57.2	100.0	100.0	84.4
4.75	1.8	93.0	98.8	62.0
2.36	0.8	10.4	70.9	31.8
2.00	0.8	6.5	64.8	28.4
1.18	0.75	3.2	51.1	22.0
0.60	0.70	2.2	36.3	15.7
0.425	0.70	2.0	30.0	13.0
0.30	0.63	1.8	21.7	9.6
0.15	0.55	1.4	11.0	5.0
0.075	0.40	1.10	4.17	2.11
Proportions (%)	36.57	22.26	41.17	100.00

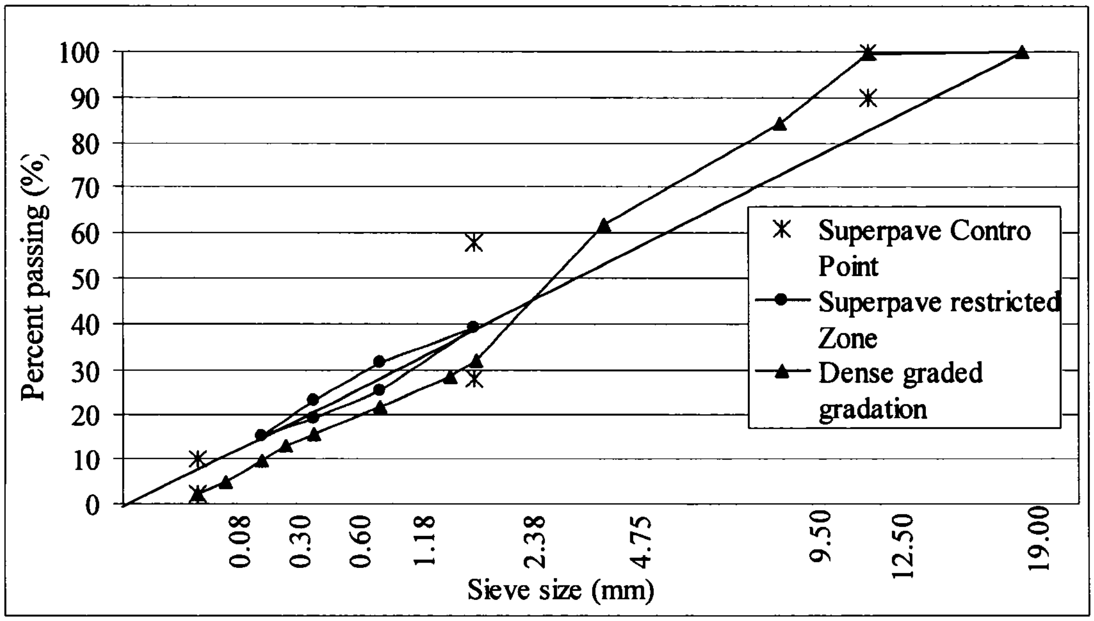

Figure 4.3 Dense-graded aggregate formulation

4.3.2.1 Bailey Method

The Bailey method for aggregate selection considers the aggregate packing characteristics. The parameters used in the method are directly related to the Voids in the Mineral aggregate (VMA), voids and compaction characteristics. The Bailey method uses various ratios to analyze and evaluate the combined blend (Vavrik et al., 2002). Before discussing these ratios it is important to define some terms.

Half size sieve: Half size sieve is the closest sized sieve given by Equation (4-1)

$$Half\ size\ sieve = 0.5\ NMPS \qquad (4\text{-}1)$$

where NMPS is the Nominal Maximum Particle Sieve, which is one sieve size larger than the first sieve to retain more than 10% aggregate.

Primary Control Sieve (PCS): PCS is the sieve which determines the coarse and fine aggregate in a combined blend. Aggregates retained above PCS are considered as coarse aggregate and the ones passing PCS are considered fine aggregate. PCS is defined as the closed sized sieve given by the formula in Equation (4-2).

$$PCS = 0.22\ NMPS \qquad (4\text{-}2)$$

Secondary Control Sieve (SCS): If the fine aggregate (passing the PCS) is considered a complete aggregate gradation and split further into two portions, the SCS determines which portion is considered coarse part and which one to be a fine part. SCS is defined as the closest sieve given by the formula in Equation (4-3).

$$SCS = 0.22\ PCS \qquad (4\text{-}3)$$

The aggregate passing SCS is considered the fine fraction of the fine aggregate and the one retained above SCS is considered the coarse fraction of the fine aggregate.

Tertiary Control Sieve (TCS): TCS allows further breaking of the fine fraction of fine aggregate and is defined as the closest sieve given by the formula in Equation (4-4).

$$TCS = 0.22\ SCS \qquad (4\text{-}4)$$

Briefly speaking the various control sieves allows breaking the entire aggregate blend into different categories based on the size of the aggregate (Vavrik et al., 2002). The following are essentially three ratios: a) Coarse Aggregate ratio (CA ratio) b) Fine Aggregate Coarse ratio (FA_C) and c) Fine Aggregate Coarse ratio (FA_F) that are used to analyze and evaluate the blend.

CA ratio: The CA ratio is used to evaluate packing of the coarse portion of the aggregate gradation and to analyze the resulting void structure. This ratio describes how the coarse aggregate particles pack together and, consequently, how these particles compact the fine aggregate portion of the aggregate blend that fills the voids created by the coarse aggregate. Equation (4-5) gives the coarse aggregate ratio.

$$CA \text{ ratio} = \frac{\% \text{ passing half sieve - } \% \text{ passing PCS}}{100\% - \% \text{ passing PCS}} \tag{4-5}$$

The packing of the coarse aggregate fraction, observed through the CA ratio, is a primary factor in the constructability of the mixture. As the CA ratio decreases (below ~1.0), compactability of the fine aggregate fraction increases because there are fewer interceptors (aggregates passing half sieve and retained above PCS) to limit compaction of the larger coarse aggregate particles. As the CA ratio increases towards 1.0, VMA will increase. However, as this value approaches 1.0, the coarse aggregate fraction becomes "unbalanced" because the interceptor size aggregates are attempting to control the coarse aggregate skeleton.

As the CA ratio exceeds a value of 1.0, the interceptor-sized particles begin to dominate the formation of the coarse aggregate skeleton. The coarse portion of the coarse aggregate is then considered "pluggers," as these aggregates do not control the aggregate skeleton, but rather float in a matrix of finer coarse aggregate particles (Vavrik et al., 2002).

FA_C ratio: This ratio describes how the coarse portion of the fine aggregate packs together and, consequently, how these particles compact the material that fills the voids it creates. The equation that describes the fine aggregate coarse ratio (FA_c) is given in Equation (4-6).

$$FA_C \text{ ratio} = \frac{\% \text{ passing } SCS}{\% \text{ passing } PCS} \tag{4-6}$$

As this ratio increases, the fine aggregate (i.e., below the PCS) packs together tighter. This increase in packing is due to the increase in volume of the fine portion of fine aggregate. It is generally desirable to have this ratio less than 0.50, as higher values generally indicate an excessive amount of the fine portion of the fine aggregate that is included in the mixture. A FA_c ratio higher than 0.50, which is created by an excessive amount of natural sand and/or excessively fine natural sand, should be avoided. This ratio has a considerable impact on the VMA of a mixture due to the blending of sands and the creation of voids in the fine aggregate. The VMA in the mixture will increase with a decrease in this ratio (Vavrik et al., 2002).

FA_F ratio: This ratio describes how the fine portion of the fine aggregate packs together. It also influences the voids that will remain in the overall fine aggregate portion of the blend because it represents the particles that fill the smallest voids created. The equation for the $FA_{f\ ratio}$ is given in Equation (4-7).

$$FA_F \text{ ratio} = \frac{\% \text{ passing } TCS}{\% \text{ passing } SCS} \tag{4-7}$$

Similar to the FA_c ratio, the value of the FA_f ratio should be less than 0.50 for typical dense-graded mixtures. VMA in the mixture will increase with a decrease in this ratio (Vavrik et al., 2002).

These ratios can be used to evaluate and adjust the VMA of the asphalt mixtures. For a 12.5 mm NMPS aggregate blend the suggested range of the ratios are given in Table 4.3 (Vavrik et al., 2002). The values of these ratios for the aggregate blend formulated in the lab were well within the suggested ranges.

Table 4.3 Bailey method ratios for the dense-graded aggregate gradation

Ratio	Suggested range	Measured value
CA	0.50 – 0.65	0.55
FA_C	0.35 – 0.50	0.44
FA_F	0.35 – 0.50	0.36

The ranges provide a starting point where no prior experience exists for a given set of aggregates. It is desirable to have the FA_C and FA_F ratio in opposite end of the limits. Past experience with these mixes has shown that field compactability can be very difficult, due to the fact that the fine fraction (minus PCS) of the combined blend doesn't want to pack and /or lock up (Vavrik et al., 2002).

The idea behind using the Bailey method for analyzing the blend was to ensure that volumetric properties will be satisfied or, if not satisfied forms the basis for correction in the second trial.

As it can be seen from Figure 4.3, the dense aggregate gradation formulated in the University of Alberta laboratory also satisfied the superpave requirements on aggregate gradation.

4.3.3 Marshall mix design for the dense-graded aggregate gradation

The next step in the experiment design was to determine the optimum AR content for this gradation. Marshall specimens were prepared and tested in accordance with the AASHTO Designation T245-97 (75 blow). The mixing and compaction temperature were 173°C and 163°C respectively.

The specific gravity (G_{mb}) and maximum specific gravity (G_{mm}) of the ARM was calculated using the Corelok Device. The volumetric calculations are done to see if the volumetric requirements at the optimum binder content are satisfied. If the requirements were not satisfied, the gradation would be adjusted based on recommendations from the Bailey method and the mix design would be conducted again.

4.3.4 Test for moisture sensitivity of the open graded and dense-graded mixtures

Once the optimum AR content for both open and dense-graded gradation was obtained, tests for moisture sensitivity for both these gradations were conducted. The test for moisture sensitivity consisted of comparing the tensile strength of the unconditioned Marshall samples with the conditioned Marshall samples.

Eight Marshall samples (75 blows) each for open-graded and dense-graded formulation were prepared in the lab in accordance with the AASHTO Designation T245-97 at the optimum AR content determined from step 4.3.1 and 4.3.3. Out of the eight samples, two samples were left unconditioned and the other six samples were conditioned as illustrated in Table 4.4.

Table 4.4 Moisture conditioning of the open-graded and dense-graded aggregate

Moisture Conditioning	Number of samples for	
	Open-graded gradation	Dense-graded gradation
No conditioning	2	2
T283 Conditioning	2	2
Three freeze and thaw cycles	2	2
Five freeze and thaw cycles	2	2

4.3.4.1 AASHTO T283 Conditioning

The samples were saturated using vacuum pressure until 70 – 80% saturation was achieved. The saturation was measured using the procedure outlined in the AASHTO T283 specification. The saturated samples were then covered tightly with a plastic film. The wrapped samples were then placed and sealed in plastic bag containing 10±0.5 mL of water. After freezing, the plastic bags were placed in a freezer at -18 ±3°C for 16 hours. Next the samples were placed in a water bath at 60±1°C for 24±1 hours. After 24 hours the samples were placed in a water bath at 25°C for 30 minutes before testing for the tensile strength (AASHTO T 283).

4.3.4.2 Freeze and Thaw cycles

The samples that undergo 3 or 5 freeze and thaw cycles were initially saturated by submerging the samples in water for 12 hours. The samples were then covered tightly with a plastic film. The wrapped samples were then subjected to freeze and thaw cycles. Each freeze and thaw cycle consisted of 8 hours of freezing at -18 ± 3°C for 8 hours followed by 8 hours of thawing at 60 ± 1°C for 8 hours. After the required numbers of freeze and thaw cycles were completed the samples were placed in a water bath at 25°C for 30 minutes before testing for the tensile strength. The reason for using the freeze and thaw cycle was to simulate the numerous freezes and thaw cycles that the pavements undergo in Alberta weather condition. It is anticipated that different number of cycles will enable researchers to assess the impact of the number of freezing and thawing cycles on the tensile strength of the ARM.

The moisture sensitivity of the open-graded and dense-graded ARM was assessed by comparing the tensile strength of the unconditioned samples with the tensile strength of the conditioned samples.

It is anticipated that comparing the tensile strength ratio (ratio of tensile strength of conditioned and unconditioned samples) for open-graded and dense-graded aggregate gradations will enable researchers to rank these gradations for Alberta conditions based on the moisture sensitivity test mentioned above.

4.4 Experiment results

4.4.1 Marshall mix design for open-graded aggregate gradation

The Marshall mix design for the open-graded aggregate gradation recommended a binder content of 8.8% by total weight of the mix. Table 4.5 shows the mix properties at this binder content. The values in Table 4.5 have been interpolated from Figures 4.4 to 4.9.

Table 4.5 Properties of open-graded mix at recommended AR content

Mix design properties	Value
AR Content (by total weight of mix, %)	8.82
Bulk Specific gravity (G_{mb})	2.209
Maximum Specific Gravity (G_{mm})	2.301
Air Void (%)	4.00
Stability (lbs)	13.08
Flow (0.254 mm)	15.00
Effective Specific Gravity (G_{se})	2.601
Voids in Mineral Aggregate (VMA, %)	22.60

The recommended value of 8.8% is slightly higher than the value recommended by the 2003 mix design for a similar gradation. The design air void content for 2003 was 5.5%. A targeted air void of 5.5% results in optimum binder content of 8.4% which is still higher than the value from the 2003 mix design.

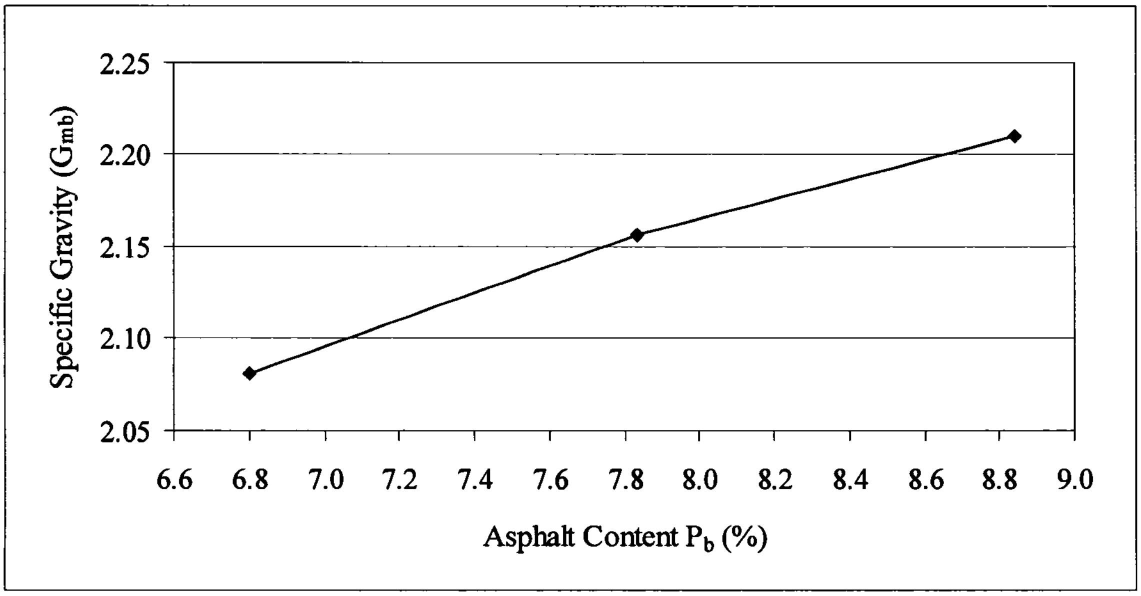

Figure 4.4 Relationship between asphalt content and specific gravity for open-graded ARM

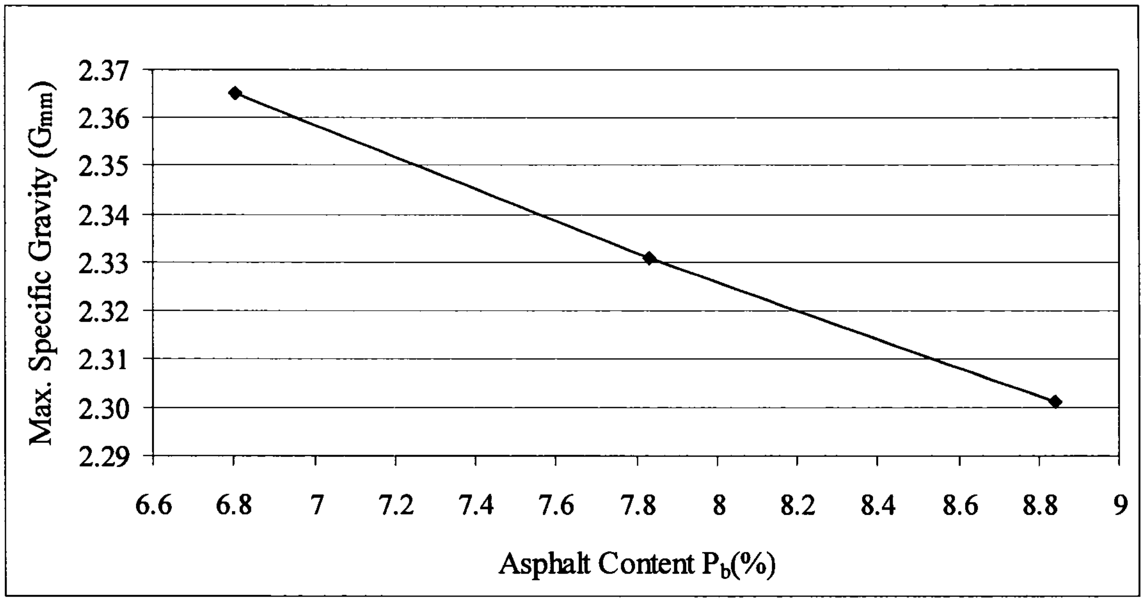

Figure 4.5 Relationship between asphalt content and maximum specific gravity for open-graded ARM

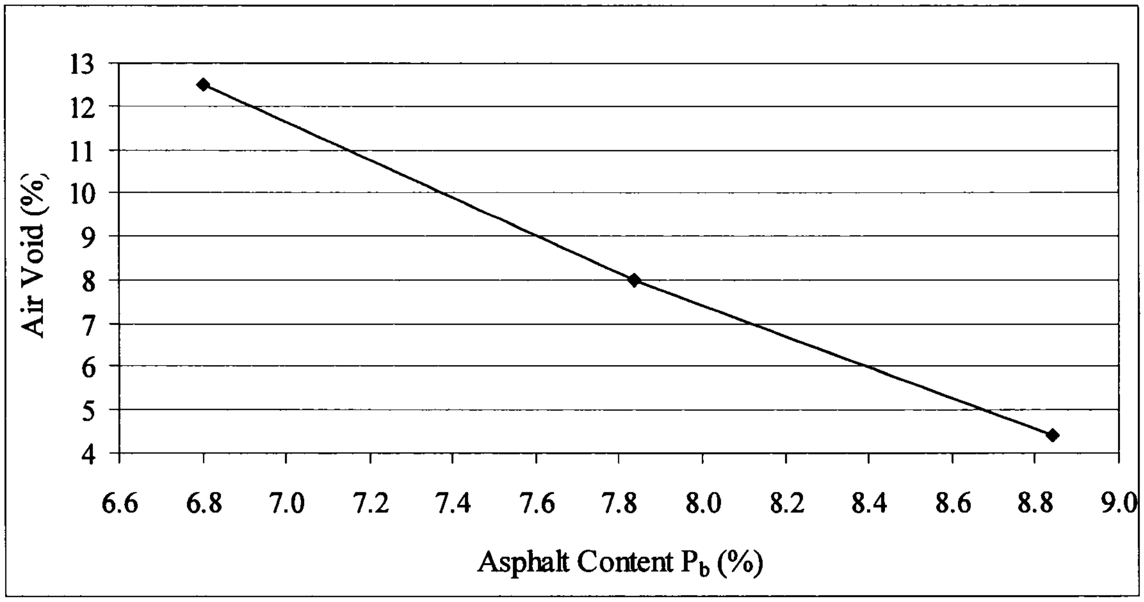

Figure 4.6 Relationship between asphalt content and air void for open-graded ARM

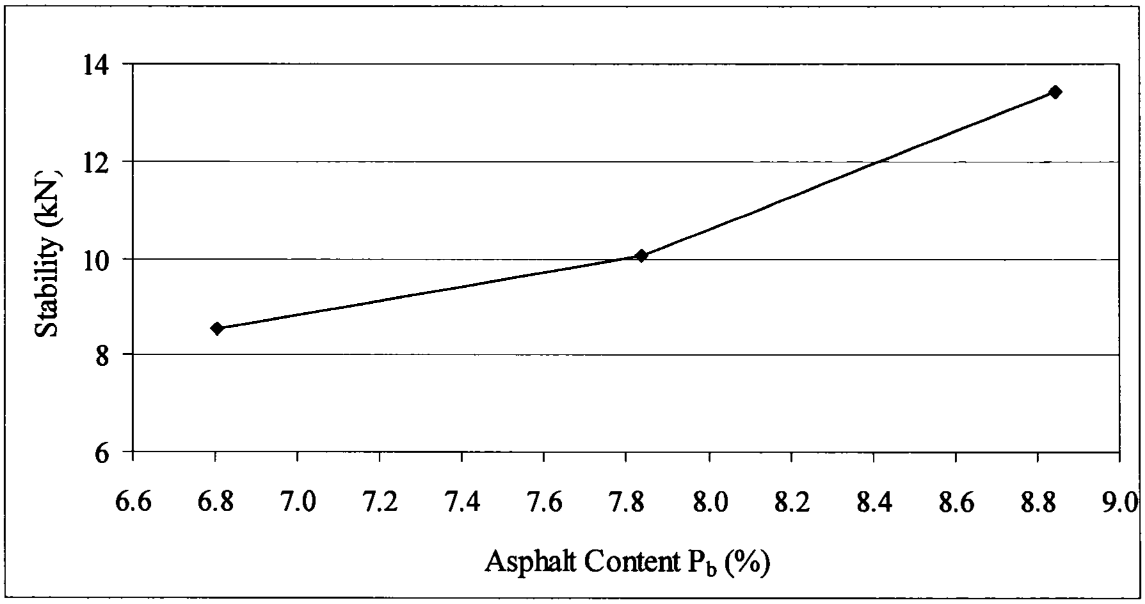

Figure 4.7 Relationship between asphalt content and stability for open-graded ARM

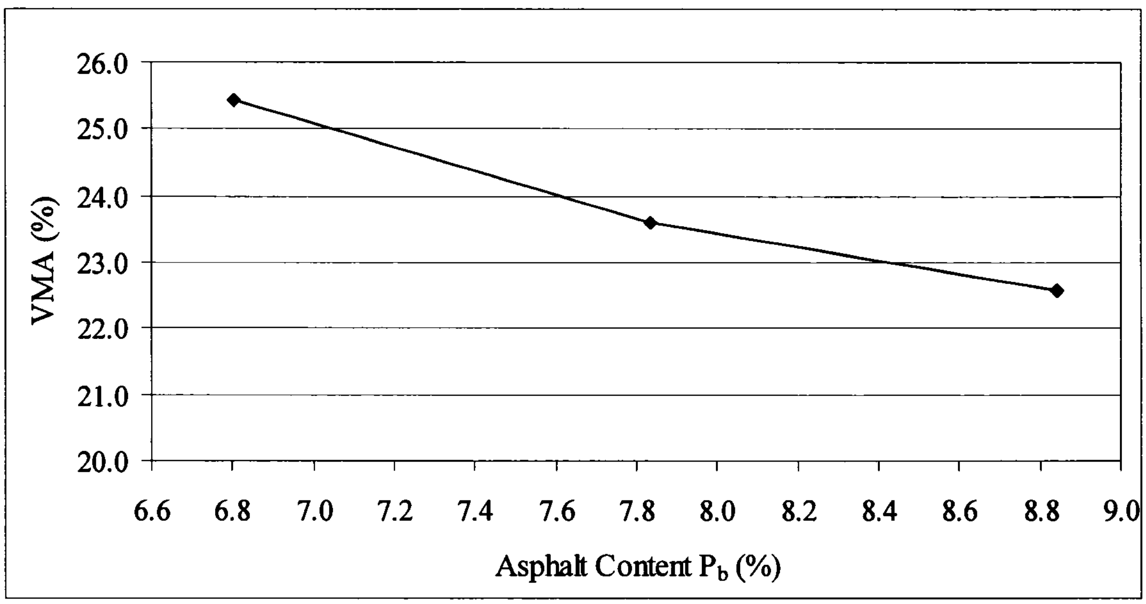

Figure 4.8 Relationship between asphalt content and VMA for open-graded ARM

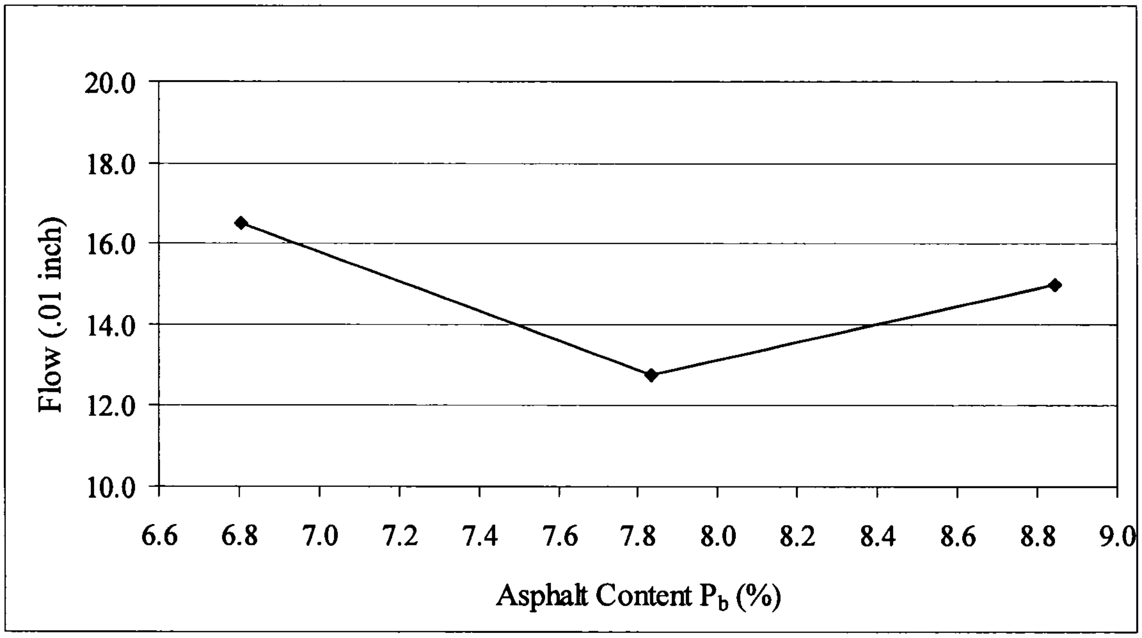

Figure 4.9 Relationship between asphalt content and flow for open-graded ARM

Careful comparison of the 2003 mix design and the open-graded mix design at the University of Alberta reveals a slight difference in the value of specific gravity of the mix (G_{mb}). Figure 4.10 shows the comparison of the G_{mb} value as reported from 2003 mix design and the G_{mb} value calculated for the open-graded mix measured at the University of Alberta. It is to be noted that Corelok Device was used to calculate the G_{mb} at the University of Alberta, asphalt lab.

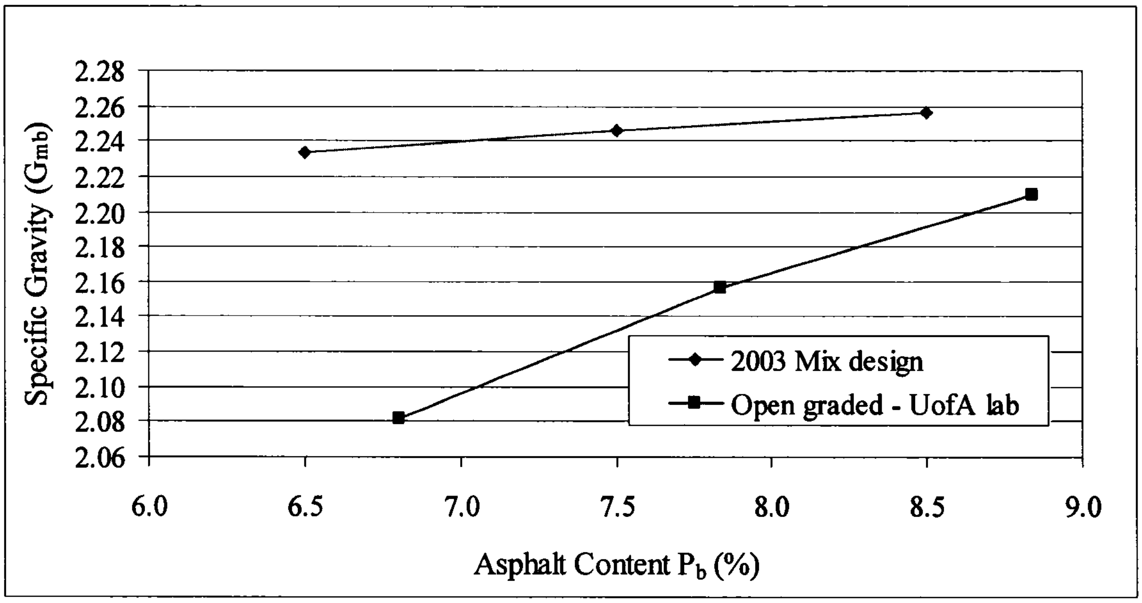

Figure 4.10 Comparison of G_{mb} measurement with Corelok with G_{mb} for 2003 mix design

It is clear from Figure 4.10 that G_{mb} values for the 2003 mix design were higher than the G_{mb} values for the mix design conducted at the University of Alberta. This difference may be attributed to the inability of the conventional method (AASHTO T166) to accurately determine the bulk specific gravity of samples with higher air voids due to excess water absorption. The use of Corelok Device, however, seals the sample with a plastic bag and does not allow any water absorption by the sample and, hence, can be considered more accurate than the conventional method for open-graded samples with excess air void.

These differences in the value of the G_{mb} result in the different air void calculations and, hence, the optimum binder content.

4.4.2 Marshall mix design for dense-graded aggregate gradation

The Marshall mix design for the dense-graded ARM recommended a binder content of 8.8% by total weight of the mix. Table 4.6 shows the mix properties at this binder content. The values in Table 4.6 have been interpolated from Figures 4.11 to 4.16.

Table 4.6 Properties of the dense-graded mix at the recommended AR content

Property	Value
AR Content (by total weight of mix, %)	8.8
Bulk Specific gravity (G_{mb})	2.229
Maximum Specific Gravity (G_{mm})	2.321
Air Void (%)	4.0
Stability (kN)	17.8
Flow (0.254 mm)	18.0
Effective Specific Gravity (G_{se})	2.628
Voids in Mineral Aggregate (VMA, %)	22.80

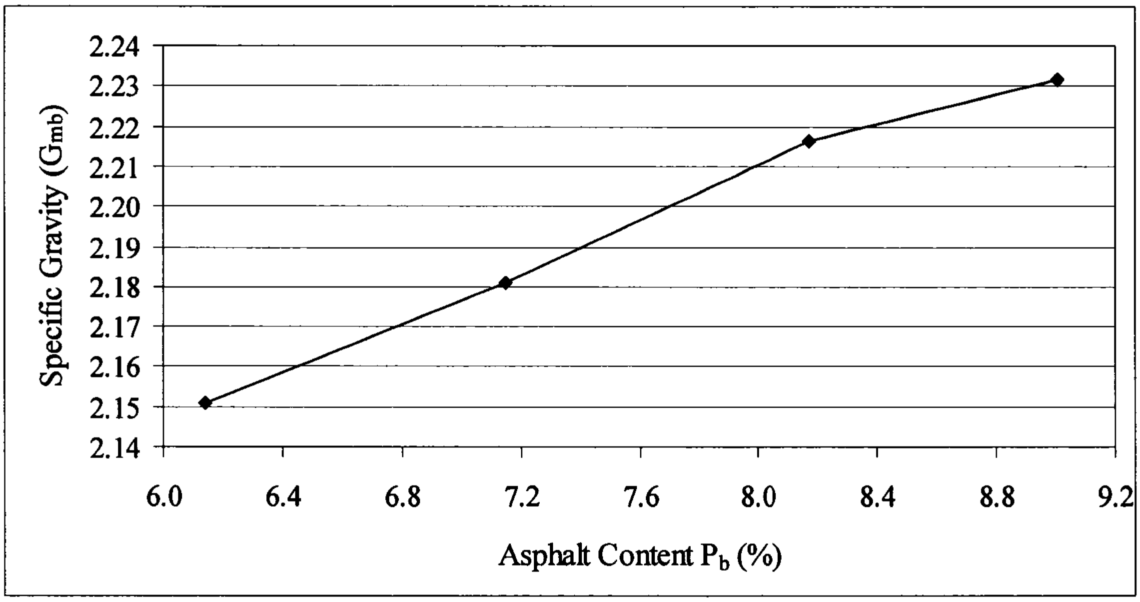

Figure 4.11 Relationship between asphalt content and specific gravity for dense gradation ARM

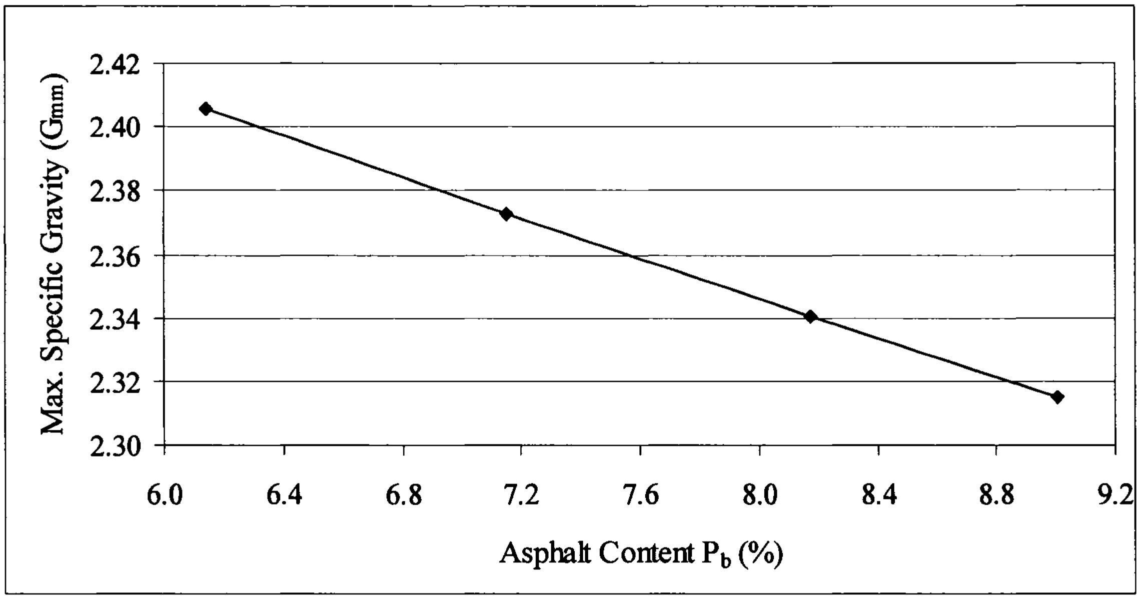

Figure 4.12 Relationship between asphalt content and maximum specific gravity for dense-graded ARM

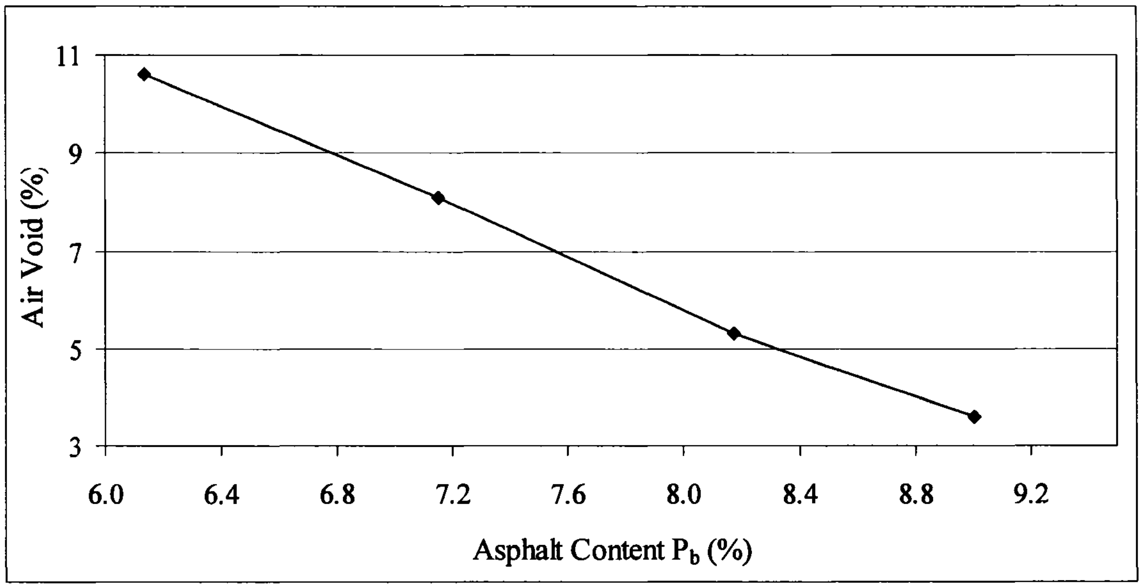

Figure 4.13 Relationship between asphalt content and air void for dense-graded ARM

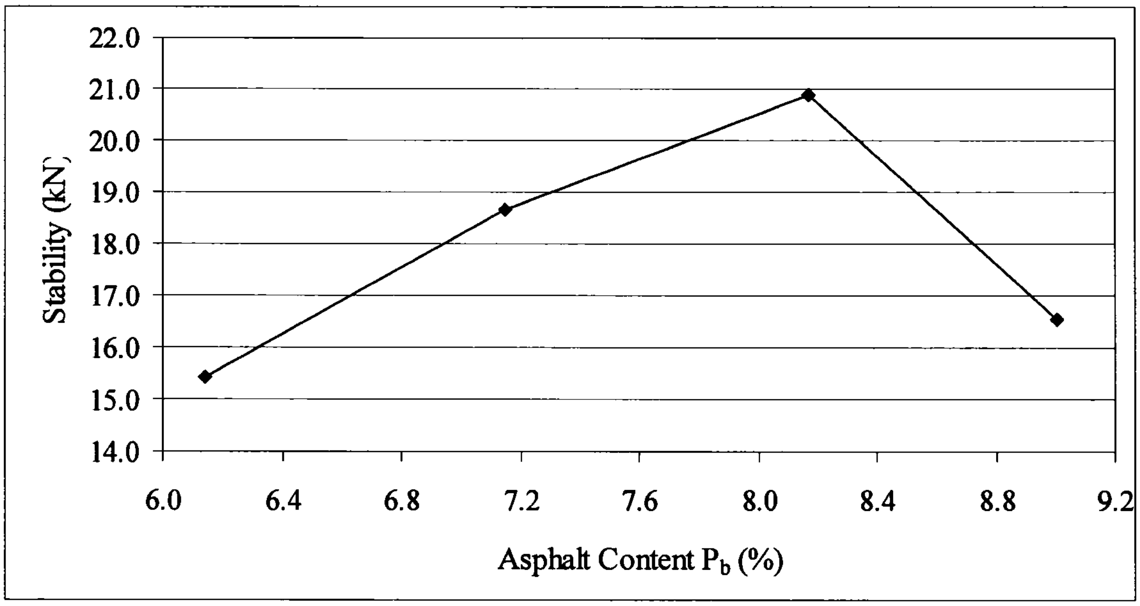

Figure 4.14 Relationship between asphalt content and stability for dense-graded ARM

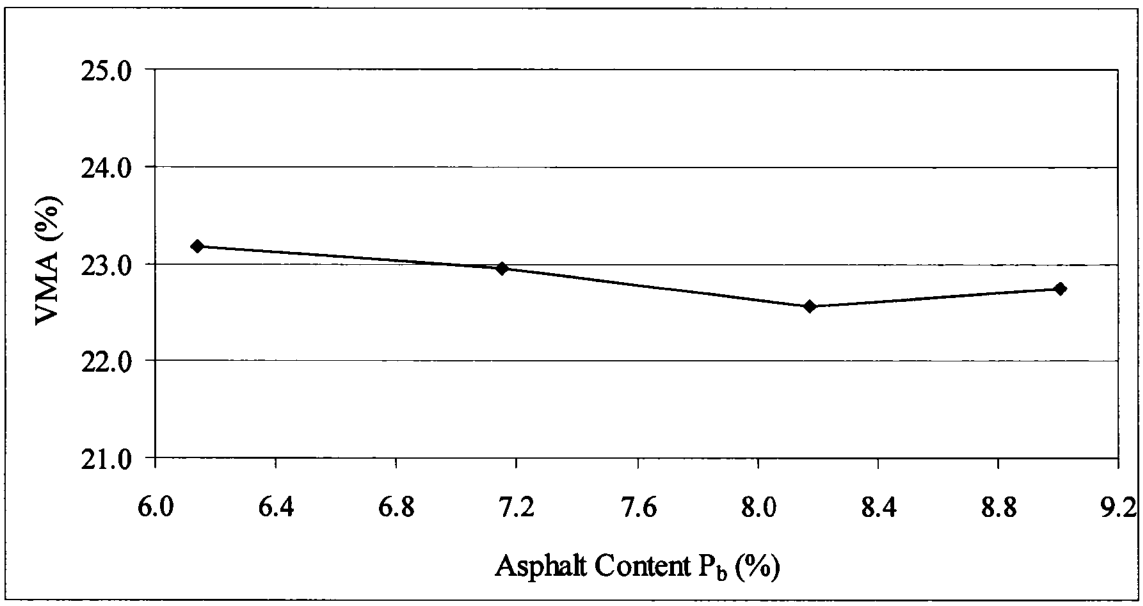

Figure 4.15 Relationship between asphalt content and VMA for dense-graded ARM

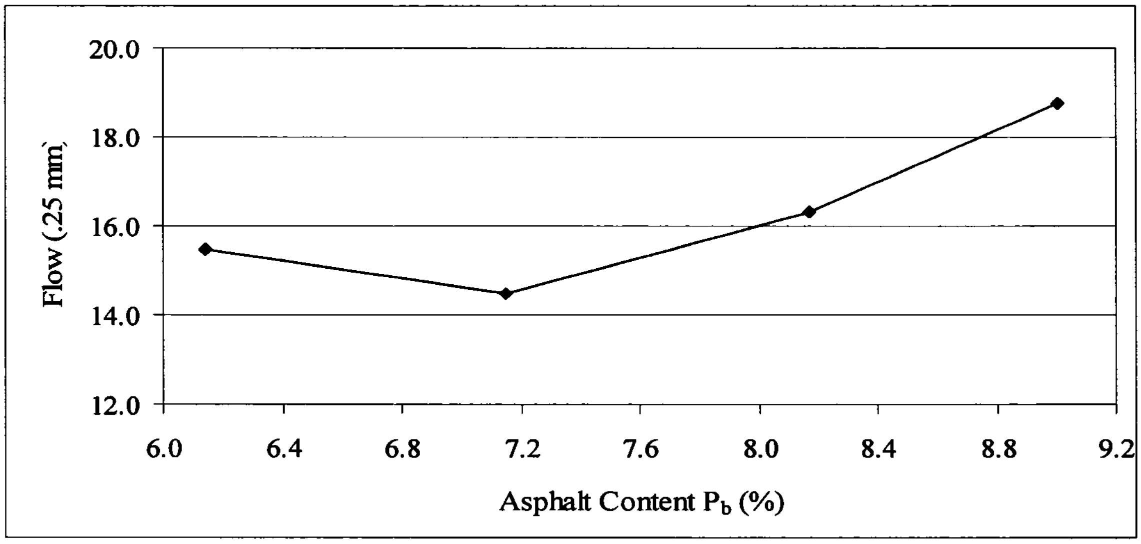

Figure 4.16 Relationship between asphalt content and flow for dense-graded ARM

Since the volumetric requirement at the recommended binder content was satisfied, no further change in the aggregate gradation was required.

4.4.3 Test on the moisture sensitivity

As explained under section 4.3.4, a total of sixteen ARM samples were prepared and tested to assess moisture sensitivity. For convenience, the different samples have been indexed as shown in Table 4.7.

Table 4.7 Index of the samples for moisture sensitivity test

Conditioning	Sample Index			
	Open-graded		Dense-graded	
No Conditioning	O1	O2	D1	D2
T283	O3	O4	D3	D4
3 freeze and thaw cycles	O5	O6	D5	D6
5 freeze and thaw cycles	O7	O8	D7	D8

The physical measurements including height, weight, diameter and specific gravity are presented in Table 4.8. The specific gravity (G_{mb}) was measured using the Corelok Device.

Table 4.8 Physical measurement of Marshall samples for moisture sensitivity test

Gradation	Sample ID	Weight (gm)	Height (mm)	Diameter (mm)	Specific Gravity G_{mb}
Open	O1	1147.8	68.62	100.94	2.198
Open	O2	1102.5	66.33	100.69	2.200
Dense	D1	1127.5	65.23	100.87	2.223
Dense	D2	1122.9	67.61	100.69	2.205
Open	O3	1121.5	67.97	100.84	2.196
Open	O4	1112.4	68.30	100.74	2.189
Dense	D3	1072.0	61.00	100.85	2.216
Dense	D4	1139.3	66.20	100.91	2.221
Open	O5	1152.0	70.12	100.62	2.183
Open	O6	1107.9	65.59	100.59	2.211
Dense	D5	1080.3	61.47	101.27	2.230
Dense	D6	1196.5	68.62	100.87	2.220
Open	O7	1068.9	66.95	100.37	2.175
Open	O8	1099.5	67.50	100.60	2.184
Dense	D7	1125.3	65.20	100.94	2.233
Dense	D8	1015.4	60.10	100.15	2.171

Tables 4.9 through 4.11 present the results of the tensile strength test conducted on the Marshall samples. The tensile strength was calculated in accordance with section 11 of the standard method of test for "Resistance of Compacted Asphalt Mixture to Moisture-Induced Damage (AASHTO Designation T283-03)." Appendix C presents the results of tensile strength measurement at the laboratory. The percentage saturation of all the T283 conditioned Marshall samples were within the specified limit (0.7–0.8). The tensile strength is calculated using the formula given by Equation (4-8).

$$S_t = \frac{2000\,P}{\pi t d} \tag{4-8}$$

Where,

S_t = Tensile Strength kPa

P = maximum load, N

t = sample thickness, mm

d = specimen diameter, mm

Table 4.9 Tensile strength of the unconditioned Marshall samples

Sample ID	Tensile Strength (kPa)	Average Tensile Strength (kPa)
O1	960.49	880.86
O2	801.23	
D1	945.17	790.20
D2	635.22	

Table 4.10 Tensile strength of the T283 conditioned Marshall samples

Sample ID	Tensile Strength (kPa)	Average Tensile Strength (kPa)	Tensile Strength Ratio
O3	666.79	636.08	0.722
O4	605.37		
D3	775.81	691.01	0.874
D4	606.20		

Table 4.11 Tensile strength of freeze and thaw conditioned Marshall samples

Sample ID	**Tensile Strength (kPa)**	**Average Tensile Strength (kPa)**	**Tensile Strength Ratio**	**No of Freeze Thaw Cycles**
O5	471.47	559.06	0.635	3
O6	646.67			
D5	789.93	739.73	0.936	
D6	689.53			
O7	344.36	385.17	0.437	5
O8	425.97			
D7	505.44	373.67	0.473	
D8	241.91			

Figure 4.17 shows the results of the moisture sensitivity tests

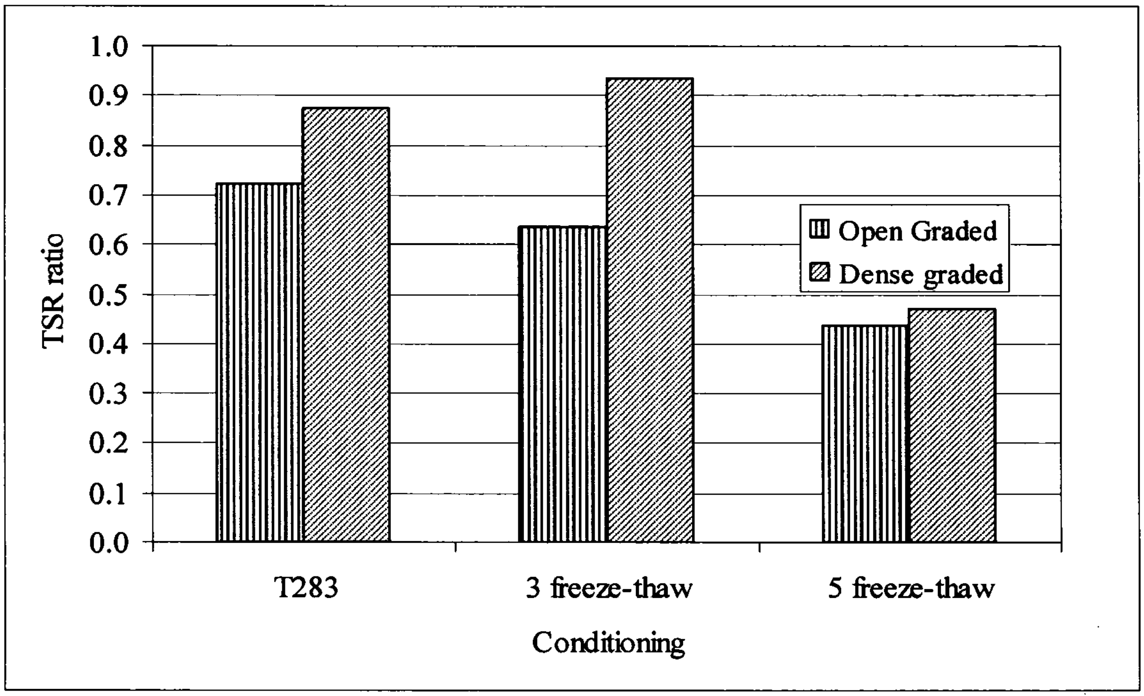

Figure 4.17 Results of moisture sensitivity test on open and dense-graded aggregate gradation

It is clearly seen from the results of the tensile strength test that the dense-graded gradation outperformed the open-graded sample for all moisture conditioning. The comparison of TSR values for sample conditioned for three freeze-thaw cycle shows a substantial difference between the open-graded and dense-graded ARM, thereby indicating dense-graded ARM to be favorable for harsh weather condition like the one in Alberta. It is to be noted that while conditioning the sample D8, the sealing was not effective and that in the last freeze-thaw cycle the entire sample came in contact with the water while thawing at 60°C. This might have influenced the tensile strength of the sample. If we discard the sample D8 from analysis, the difference in TSR ratio between open-graded (0.437) and dense-graded (0.639) ARM is substantial.

4.5 Summary

This chapter focused on the laboratory experiments conducted at the University of Alberta to investigate the lower binder content (6.9%) used in the 2003 Asphalt Rubber project and to study the effect of gradation on the moisture sensitivity of ARM.

The Marshall mix design using almost similar aggregate gradation and the same AR binder used in 2003 Asphalt Rubber project, resulted in optimum binder content of 8.8%. A careful analysis of the 2003 mix design and the mix design at the University of Alberta laboratory experiment revealed that the G_{mb} measurements in the 2003 mix design were higher compared to G_{mb} measurement at the University of Alberta. The G_{mb} measurements at the University of Alberta were conducted using the Corelok Device. However, we cannot ignore the possibility of this difference in optimum AR content because of the following reasons:

1. Although the aggregate supplier for laboratory experiment and the 2003 project were the same, the aggregates used were slightly different than the ones used in 2003 project.
2. The mixing temperature used in the laboratory experiments (173°C) was slightly higher.

In order to investigate the effect of aggregate gradation on the moisture sensitivity of ARM, the Bailey method was used to establish a dense-graded mix. The moisture sensitivity of dense-graded and open-graded ARM were compared using TSR value. In addition to the T283 test for moisture sensitivity freeze and thaw cycles were also used to simulate the freeze and thaw cycles that pavements in Alberta undergo during winter. The relatively higher TSR value for the dense-graded ARM compared to open-graded ARM for all three conditioning (T283, three and five freeze-thaw cycles) justified the hypothesis that open gradation allows water to remain for a prolonged time, thereby increasing the possibility of severe water damage due to prolonged interaction with water and freeze thaw cycles.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATION

This thesis provided background information on the 2002 and 2003 Alberta Asphalt Rubber projects. The materials used in the 2002 and 2003 projects were also briefly discussed. A significant portion of the report focused on the short-term performance evaluation of the 2002 and 2003 projects. The findings from the performance evaluation can be summarized as follows:

- The performance of half thickness ARM section is inferior compared to the performance of full thickness ARM and conventional ACM sections.
- The performance of ARM sections do not show any additional advantages in terms of pavement performance compared to conventional asphalt mixture.
- Some of the 2003 pavement sections showed signs of surface distresses mainly due to moisture damage. Interestingly, all these pavement sections utilized the same aggregate gradation and mix design. The binder content for these pavement sections was low at 6.9% compared to the 2002 Alberta Asphalt Rubber project locations.

The relatively low asphalt content for the 2003 mix design prompted a need for further investigation. A laboratory experiment was conducted at the University of Alberta to investigate lower asphalt content. Furthermore, although the aggregate gradations used in all the 2002 and 2003 projects were fairly similar, the gradation for the 2003 project was slightly open compared to other projects locations. It was felt that open gradation allows water to permeate inside the ARM sections more easily, allowing the water to stay for a prolonged time. As a result, this makes the ARM section more susceptible to moisture damage. A laboratory experiment was conducted to study the effect of aggregate gradation on the moisture sensitivity of ARM. The experiment consisted of comparing the moisture sensitivity of an open-graded and dense-graded aggregate mix. The Bailey method was used to establish the dense-graded aggregate gradation.

The laboratory mix design using open-graded ARM similar to the 2003 aggregate gradation and the same AR binder, resulted in an optimum binder content of 8.82%. This value is higher than the binder content of 6.9% used in the 2003 projects. A careful analysis of these two mix design results revealed the G_{mb} values in the 2003 mix design were higher than the ones obtained at the University of Alberta. It is to be noted that the G_{mb} of ARM were measured using the Corelok Device, while the G_{mb} measurements for the 2003 mix design were measured using the AASHTO T166 method. NCHRP research by Brown et al. (2004), found that the AASHTO T166 method, although is accurate to measure bulk specific gravity at lower levels of water absorption, may not be suitable for mixes with higher absorption (greater than 1%) levels.

We cannot, however, ignore the possibility of this higher optimum AR content because of the following reasons:

- The mixing temperature used at the University of Alberta was higher than the one used in 2003 mix design.
- Although the aggregate supplier for laboratory experiment and the 2003 mix design were the same, the aggregates used were different. Even a small variation among the aggregates can affect the results of the mix design.

The moisture sensitivity test on the open-graded and dense-graded ARM samples conditioned using AASHTO T283 procedures and freeze and thaw cycles showed dense-graded ARM to perform better than the open-graded mix. The results from the moisture sensitivity tests forms a basis for justifying the hypothesis that open-graded gradations allows water to permeate easily, thereby making it more prone to moisture damage.

5.1 Recommendations

Before making any recommendations based on the short-term performance, it is important to clarify at the outset that a correct judgment on the performance and effectiveness of the ARM sections is possible only after a long-term performance evaluation. The recommendation made in the thesis are based on short-term performance and, hence, should only form the basis for further investigation, and no final decision should be made solely based on the recommendation made herein.

Based on the observations from the short-term performance evaluation and the laboratory experiment results, the following recommendations are made:

- Considering the inferior short-term performance of half thickness ARM compared to full thickness ARM and conventional ACM sections, it is suggested not to pave any further half-thickness ARM sections.
- The IRI and rutting measurements were not measured at the same interval for all the pavement sections. This made it difficult to compare the performance of different ARM sections. It is recommended to do the IRI and rutting measurements for all the ARM sections at the same age, so that it is possible to make a fair comparison of the ARM sections.
- The performance of ARM sections do not show any additional benefit in terms of pavement performance compared to conventional ACM sections. However the performance of ACM sections is also not clearly superior to the ARM sections. Considering the ARM success story in various states of the USA and the benefits associated, it is reasonable to expect ARM will perform better in Alberta condition as well. It is, however, important to find an aggregate gradation and CR size that suits Alberta conditions rather than adopting what has been in use in other jurisdictions. It is recommended to pave other experimental sections with different aggregate gradations and CR sizes, rather than stop paving ARM sections or continue paving with the same combination of aggregate gradation and AR.
- The G_{mb} value of the 2003 mix design were found to be slightly higher than the ones of samples made at the University of Alberta and measured using the Corelok Device. Measuring the G_{mb} value of samples cored from the 2003 pavement sections, using the Corelok Device, might provide a better insight on the 2003 mix design and, hence, the lower recommended asphalt content.
- It is recommended to use an automatic vacuum sealing method (e.g. Corelok Device) to measure G_{mb} in future mix designs of ARM. Unlike the conventional method (T166), the vacuum sealing method of determining G_{mb} is independent of the water absorption.
- The results of the moisture sensitivity test at the University of Alberta showed dense-graded ARM to perform better than the open-graded ARM. It might be worth to pave some experimental ARM sections using dense-graded aggregate gradation.

- The Bailey method was used in the research to establish a dense-graded aggregate gradation and was very useful in producing a mix that potentially satisfies volumetric requirements. It is recommended to use the Bailey method as a tool while selecting dense-graded gradation in future experiments.

5.2 Research Contributions

As already mentioned, a significant portion of the research study was focused on short-term pavement performance evaluation of the ARM sections paved during the summer of 2002 and 2003 in Alberta. The short-term performance evaluation has enabled researchers to achieve a better understanding of ARM performance in Alberta weather condition. It is felt that the performance analysis at the University of Alberta will form a basis for future paving of experimental ARM sections in Alberta, and also in other similar weather conditions prevalent in Canada. Based on the short-term performance of ARM pavement, the issues requiring immediate attention were analyzed by conducting a laboratory experiment at the University of Alberta. It is hoped that the experiment provided justification for further investigation into the 2003 mix design. The research also triggered the need to experiment with ARM aggregate gradation other than open-graded or gap-graded gradations. To be specific, the research contribution can be summarized as follows:

- Better understanding of ARM performance in Alberta weather condition.
- Forms basis for future experiments with ARM sections in Alberta and other part of Canada.
- Justifies the need for further investigation into the 2003 mix design and the pavement sections paved using the 2003 mix design results.
- Triggers the need for paving experimental sections using different gradation to determine an aggregate gradation that is suitable for Alberta's conditions.
- Provides a better understanding of the effect of aggregate gradation on the moisture sensitivity, and justifies the use of dense-graded aggregate gradation in future ARM experimental sections.

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Appendix A

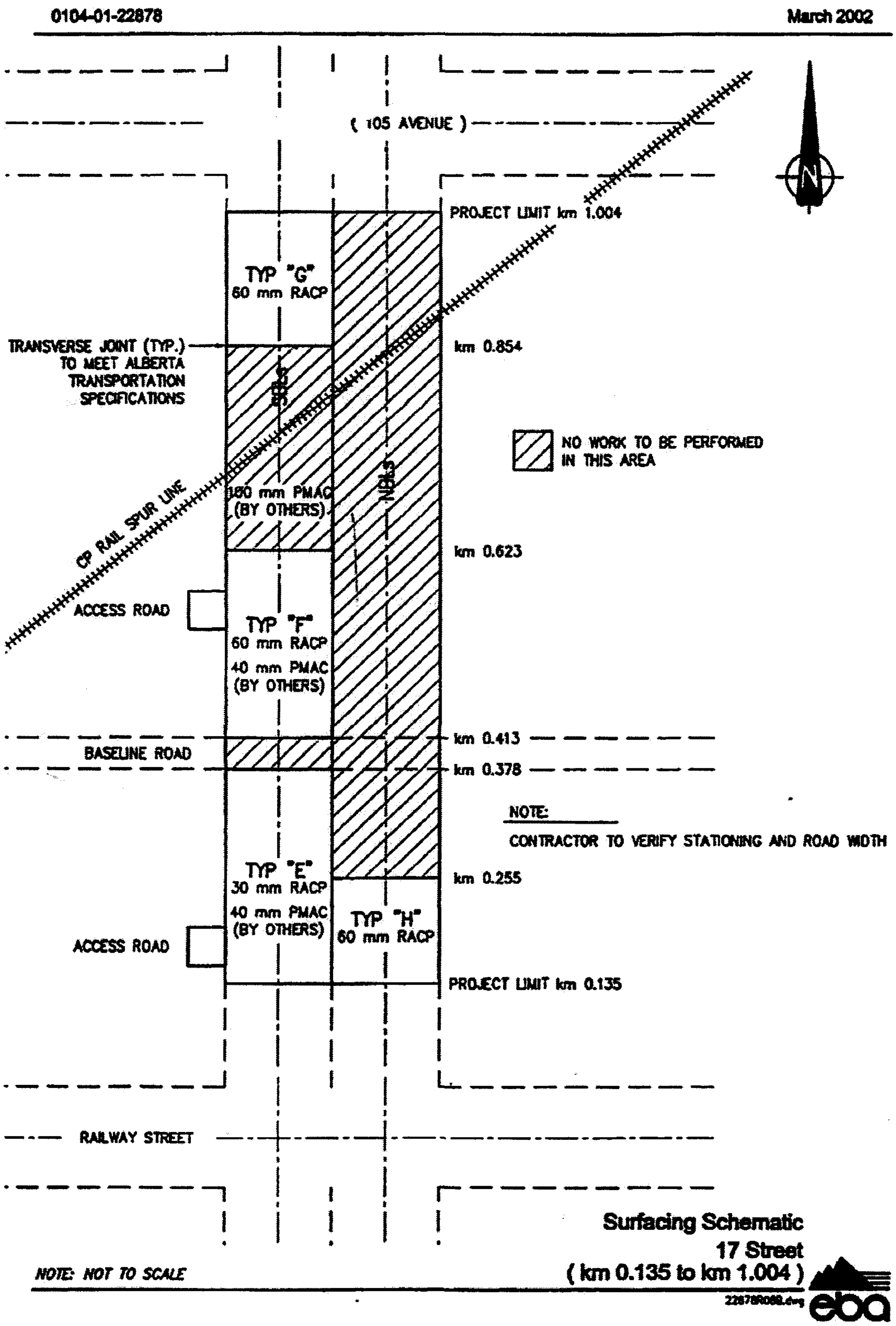


Figure A-1 Layout of the 17 Street

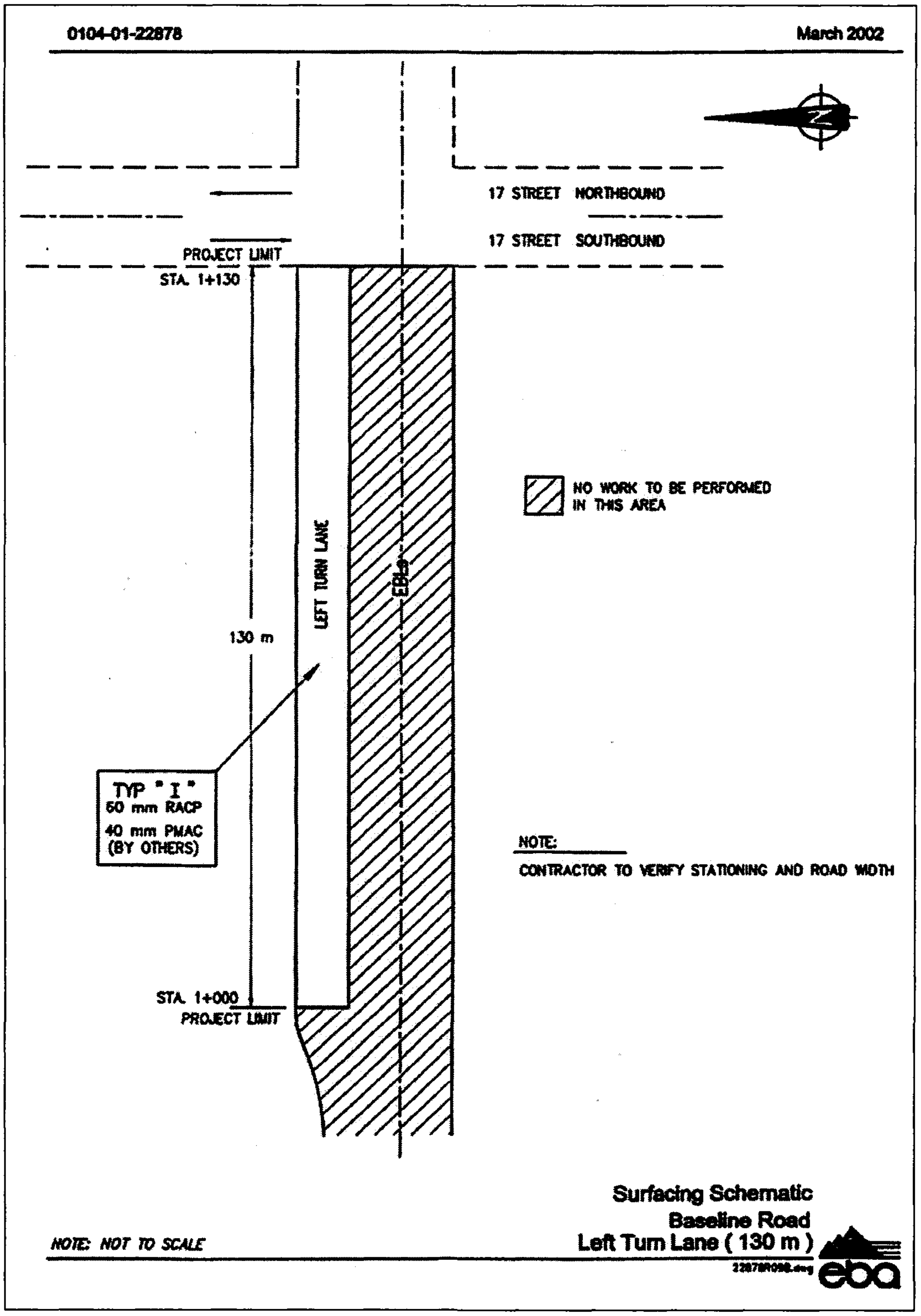


Figure A-2 Layout of Baseline Road Left Turn Lane

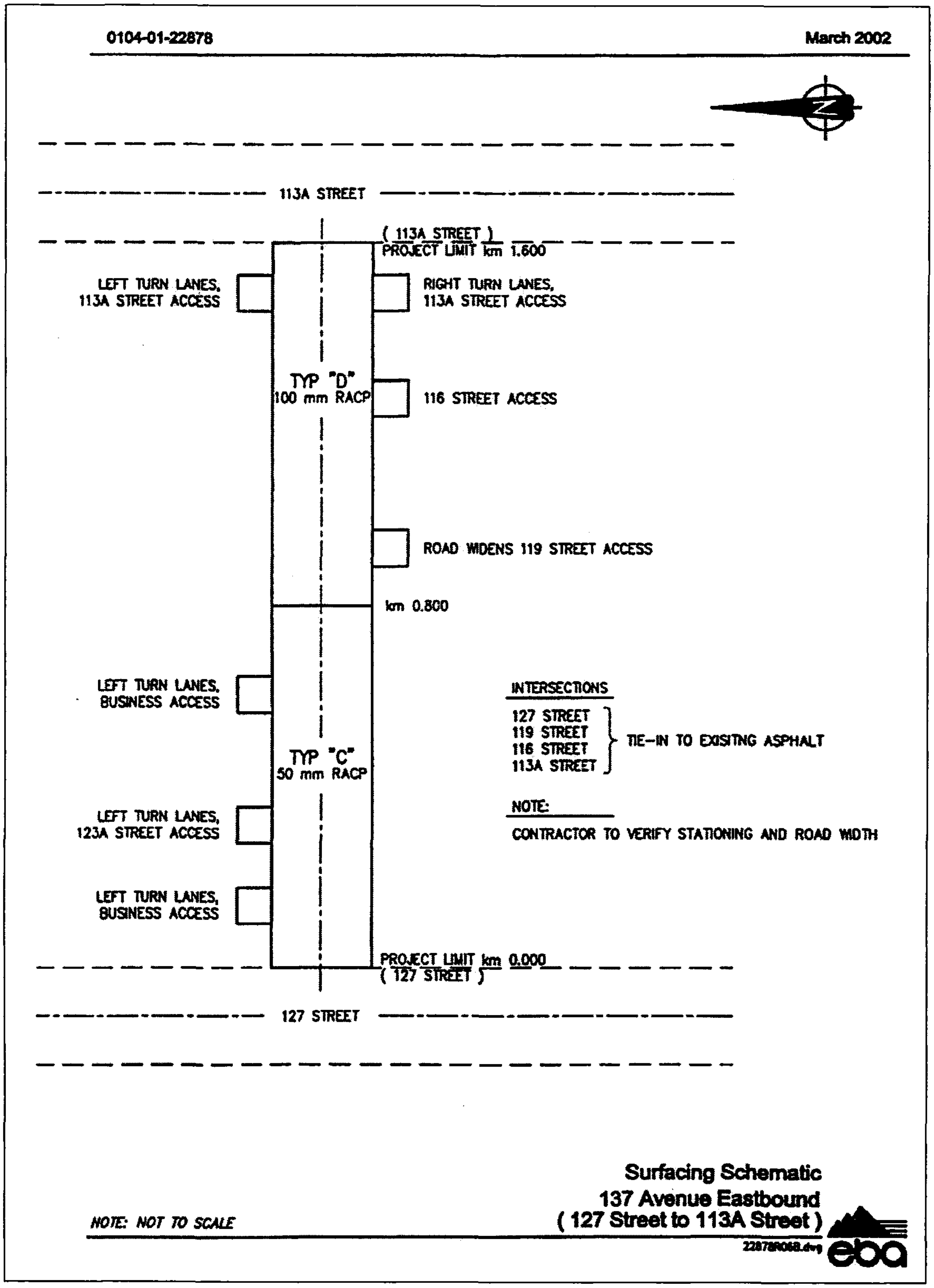


Figure A-3 Layout of the 137 Avenue Project

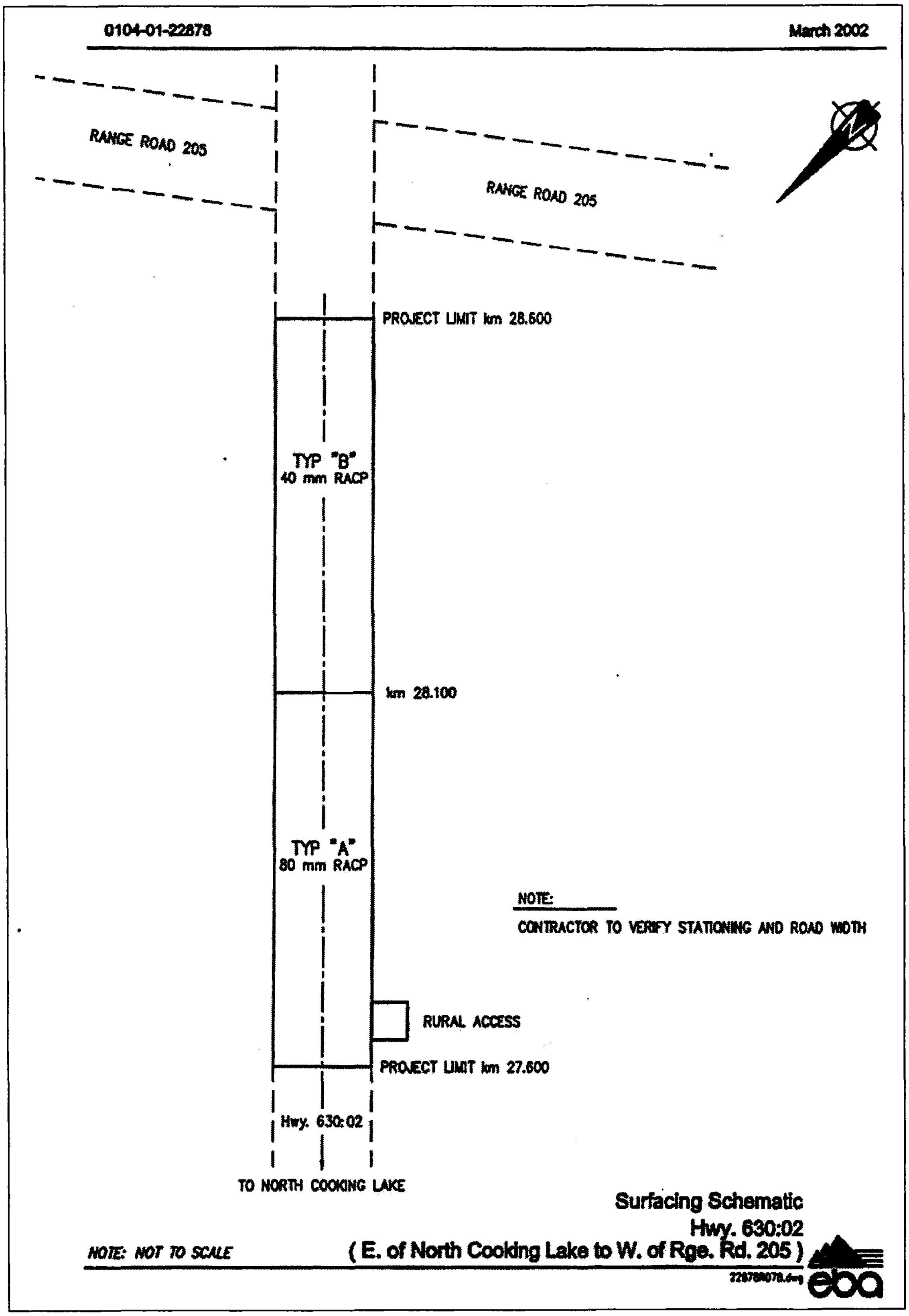


Figure A-4 Layout of Highway 630:02 Project

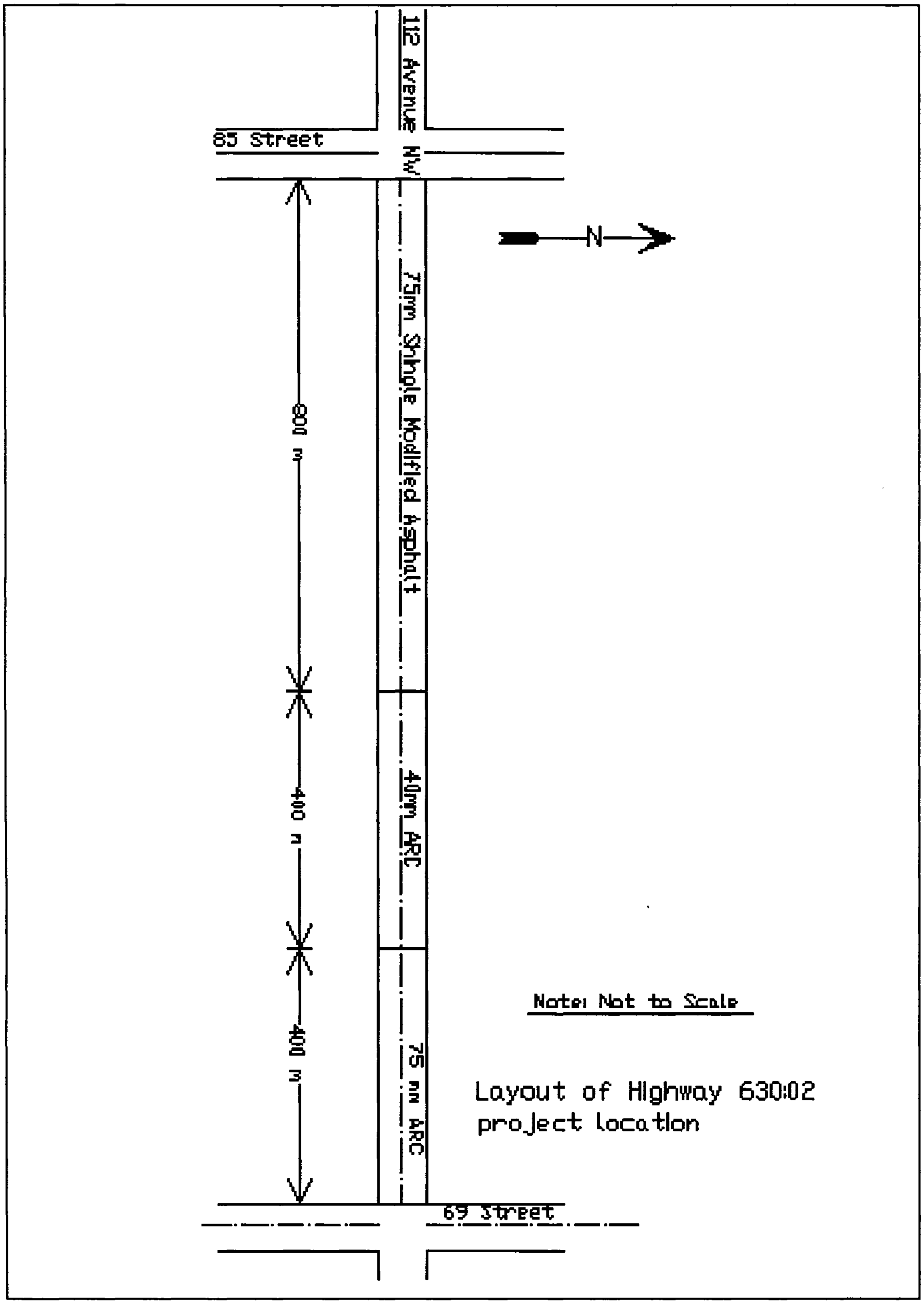


Figure A-5 Layout of 112 Avenue NW Project (Calgary)

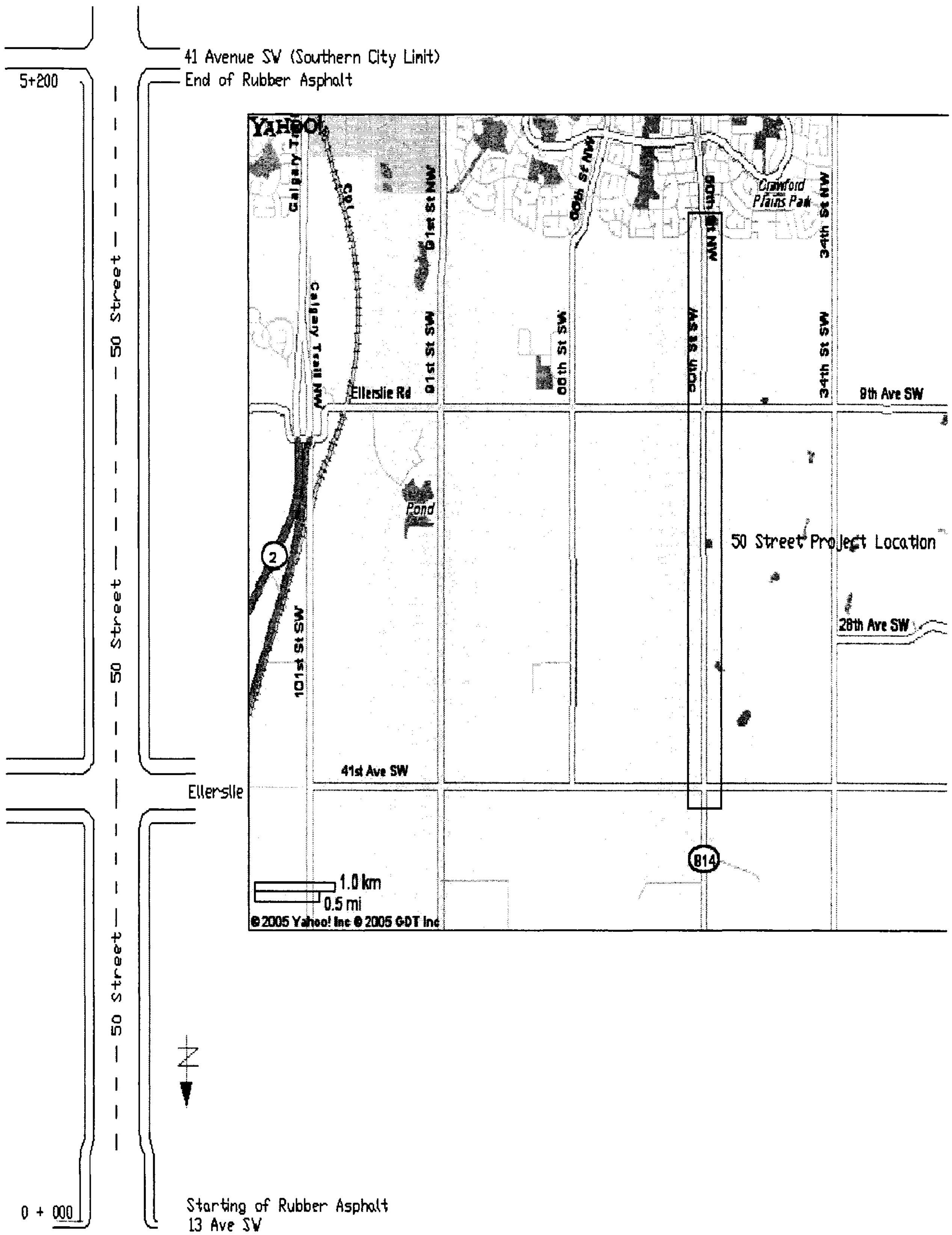


Figure A-6 Layout of the 50 Street project location

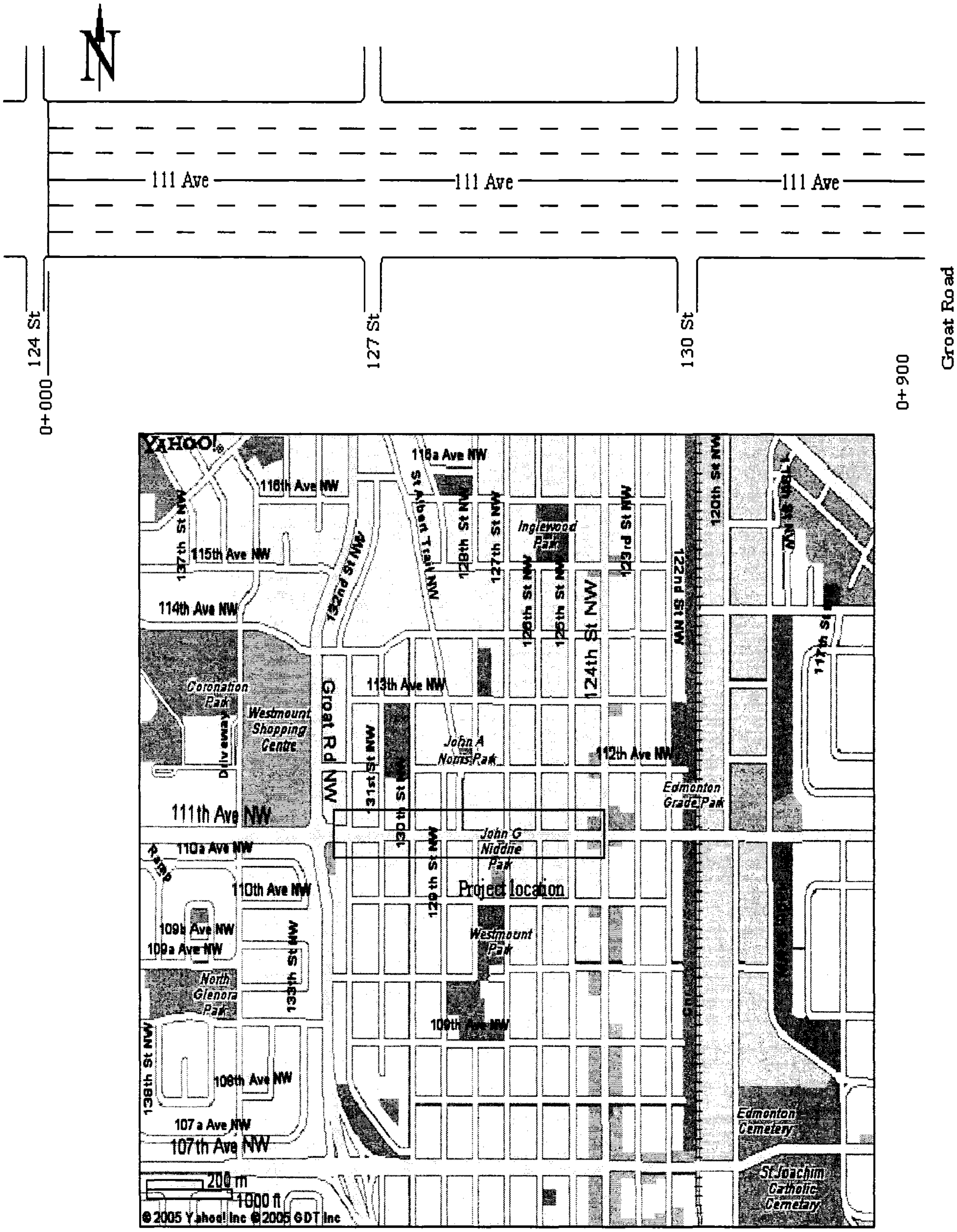


Figure A-7 Layout of the 111 Avenue project location

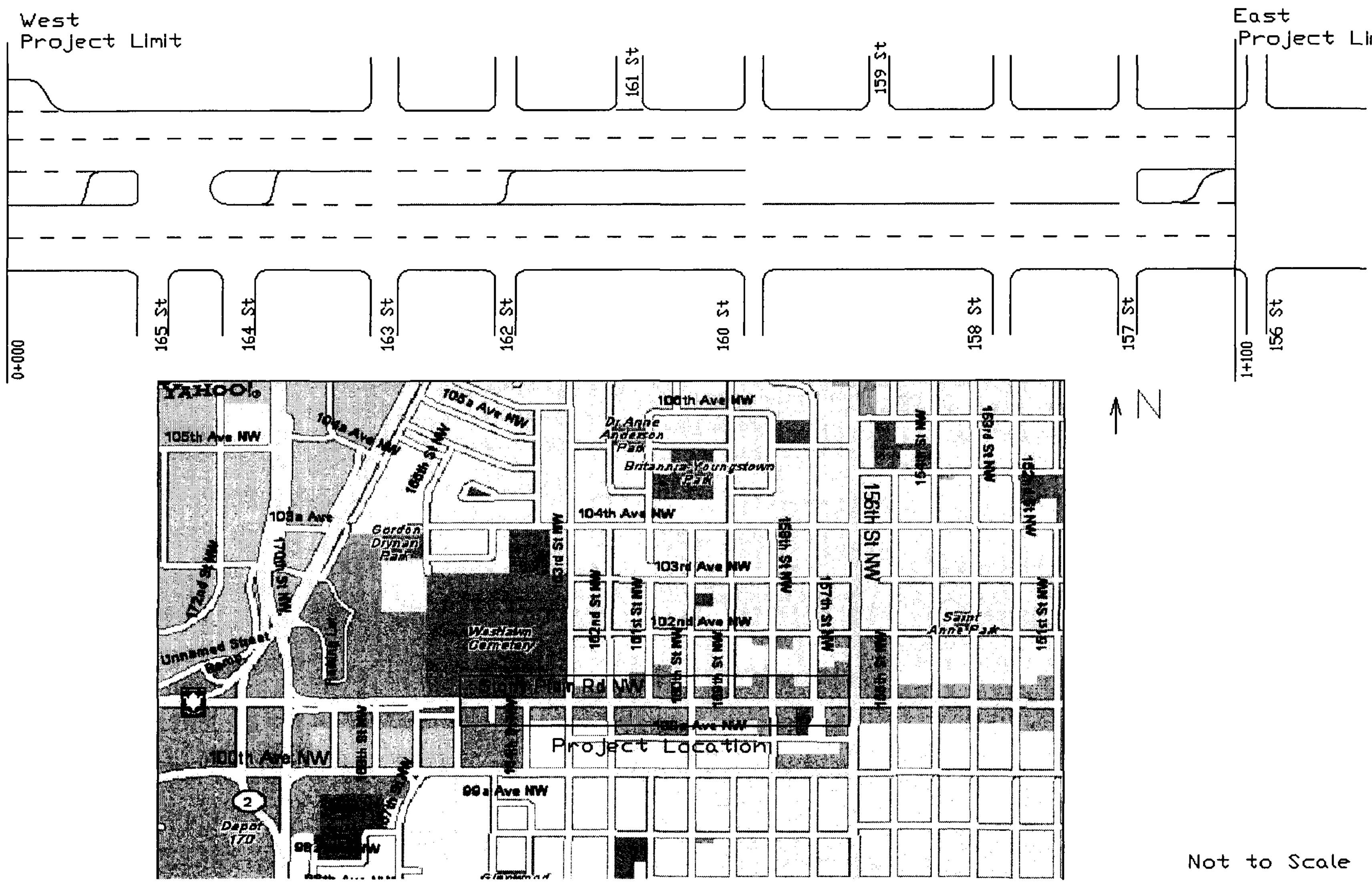


Figure A-8 Layout of the Stony Plain project location

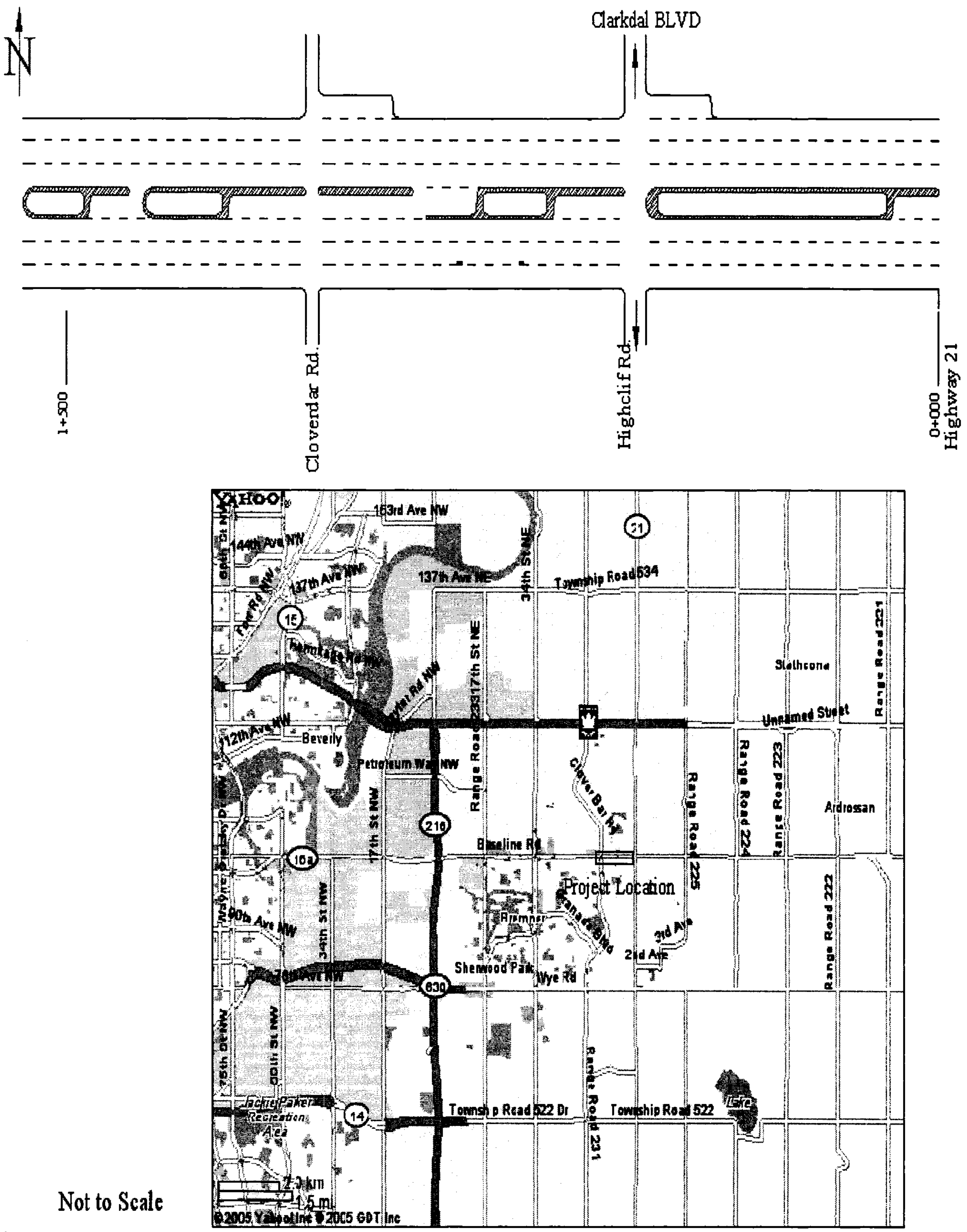
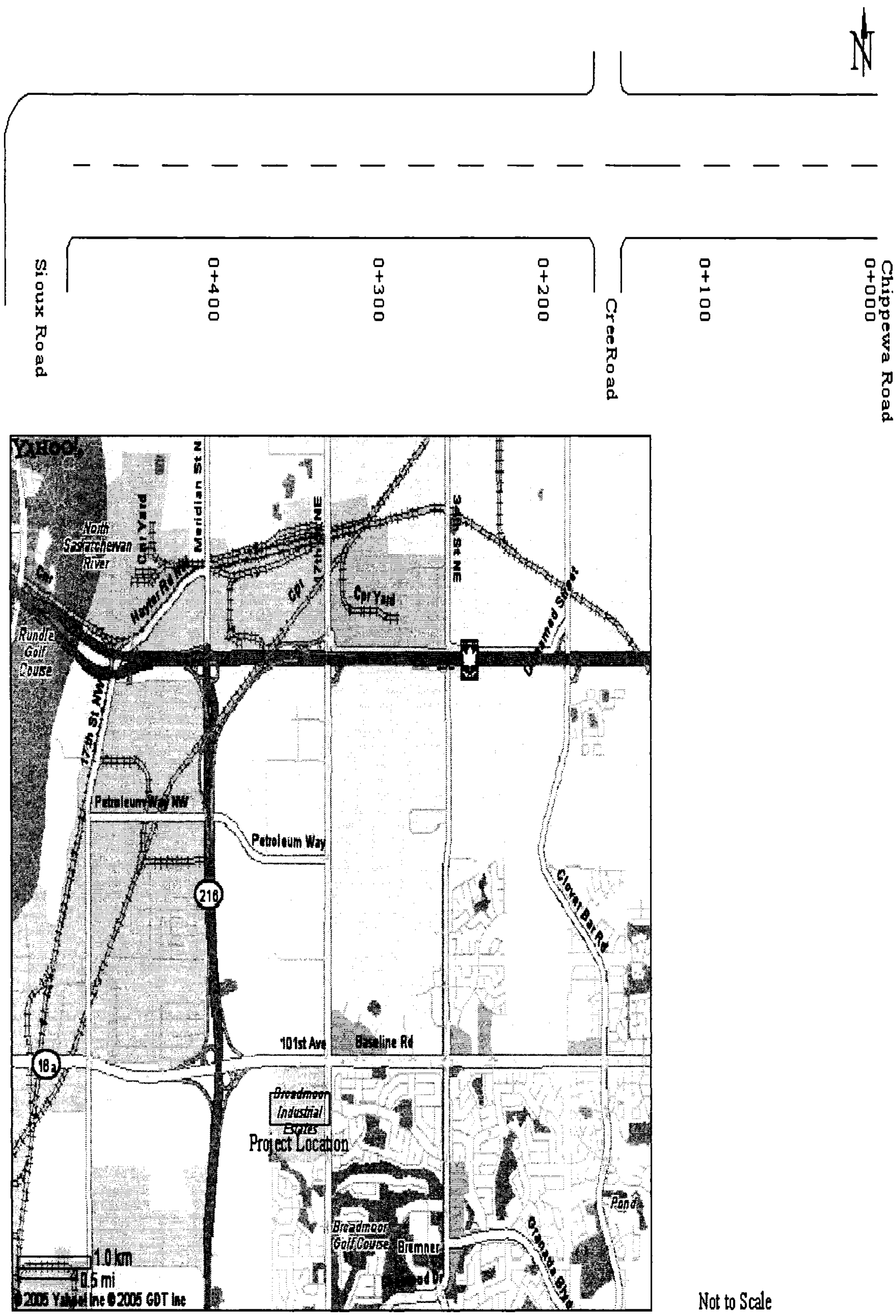


Figure A-9 Layout of the Baseline Road project location



Not to Scale

Figure A-9 Layout of the Kaska Road project location

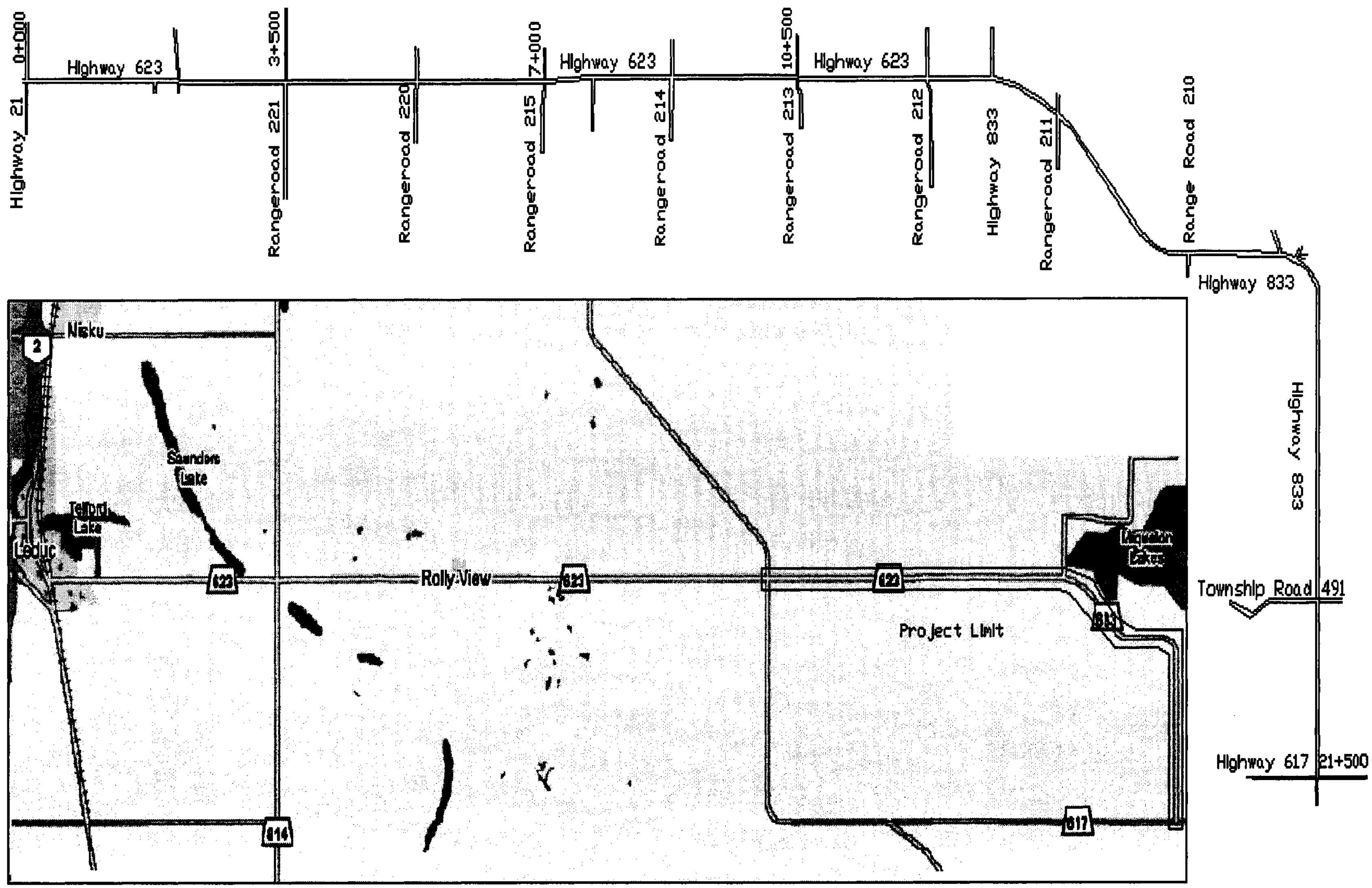


Figure A-10 Layout of the Highway 623:04 project location

Appendix B

Figure B-1 Transverse crack at Highway 630 project location

Figure B-2 Longitudinal joint crack at Highway 630 project location

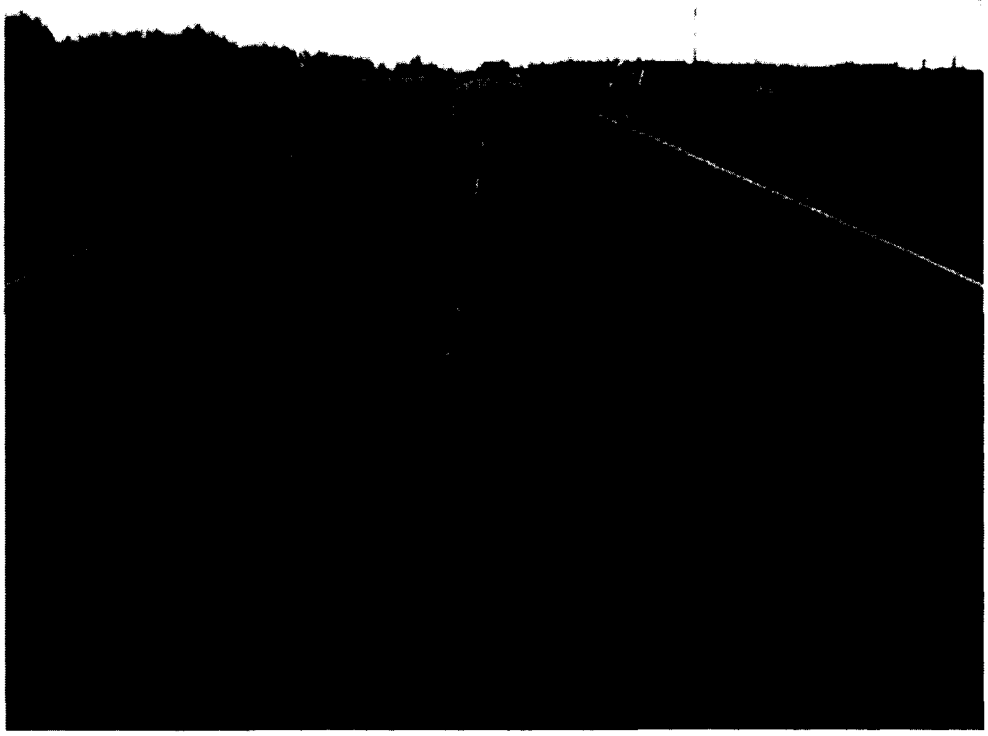


Figure B-3 112 Avenue project location at Calgary was least affected by surface distresses

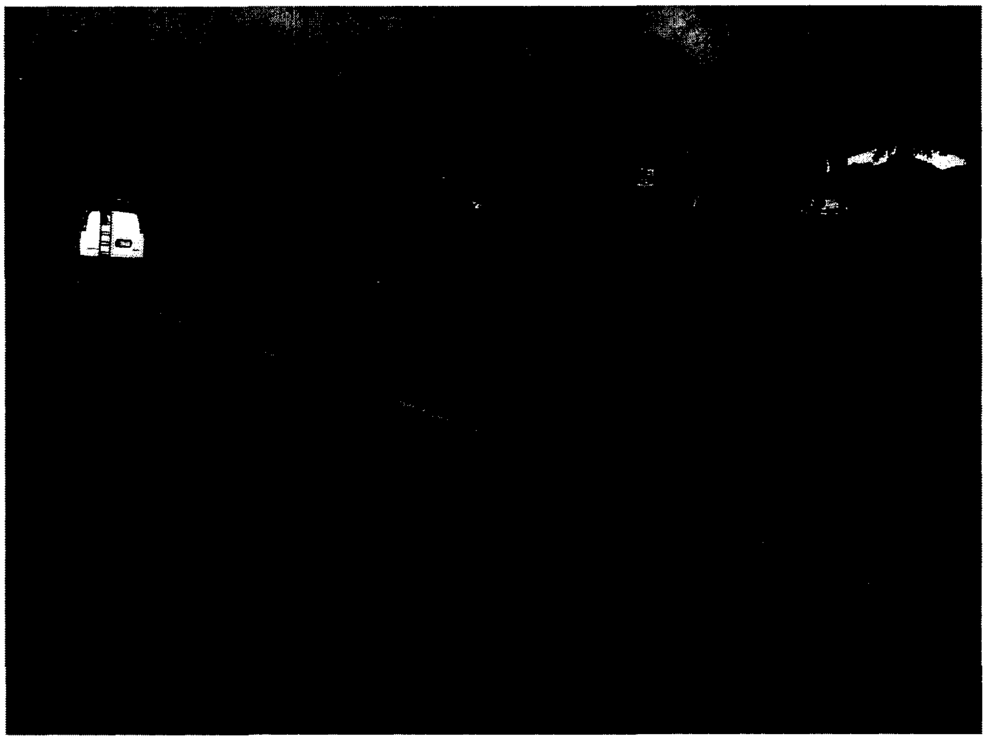


Figure B-4 Uniformly spaced transverse cracks at 137 Avenue project location

Figure B-5 Moisture damage at the pavement edge of 111 Avenue project location

Figure B-6 Surface distresses located at road intersection at 111 Avenue project location

Figure B-7 Aggregate disintegration at Stony Plain Road project location

Figure B-8 Fatigue crack close to ETS bus terminal at Stony Plain Road project location

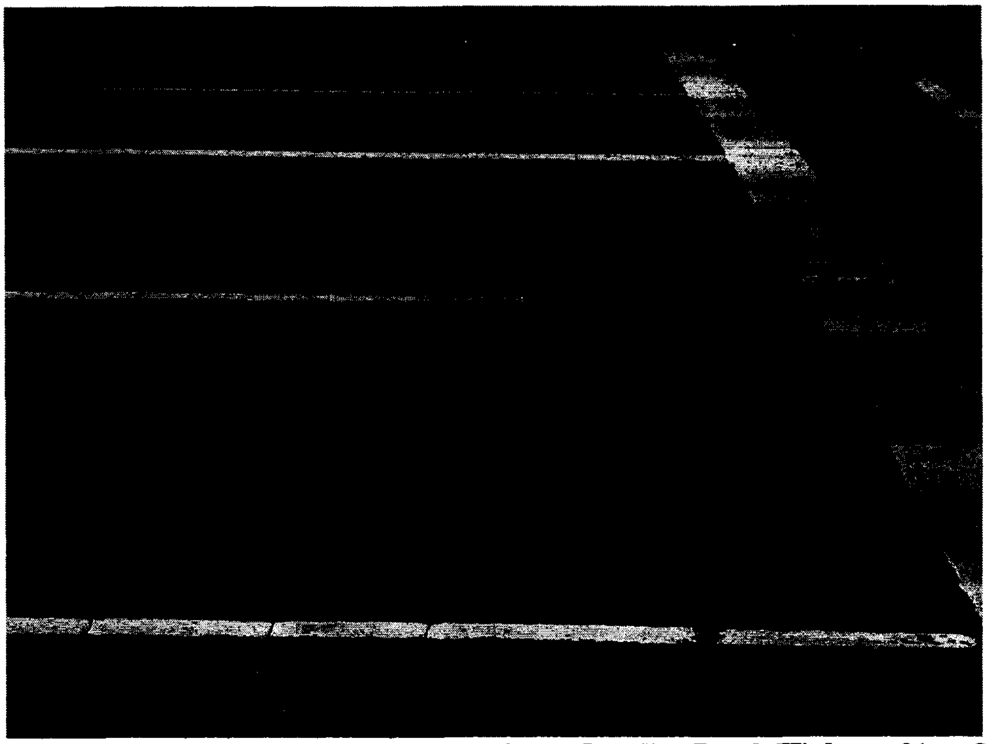


Figure B-9 Surface distresses at road intersection at Baseline Road (Highway 21 to Chelsea Way)

Appendix C

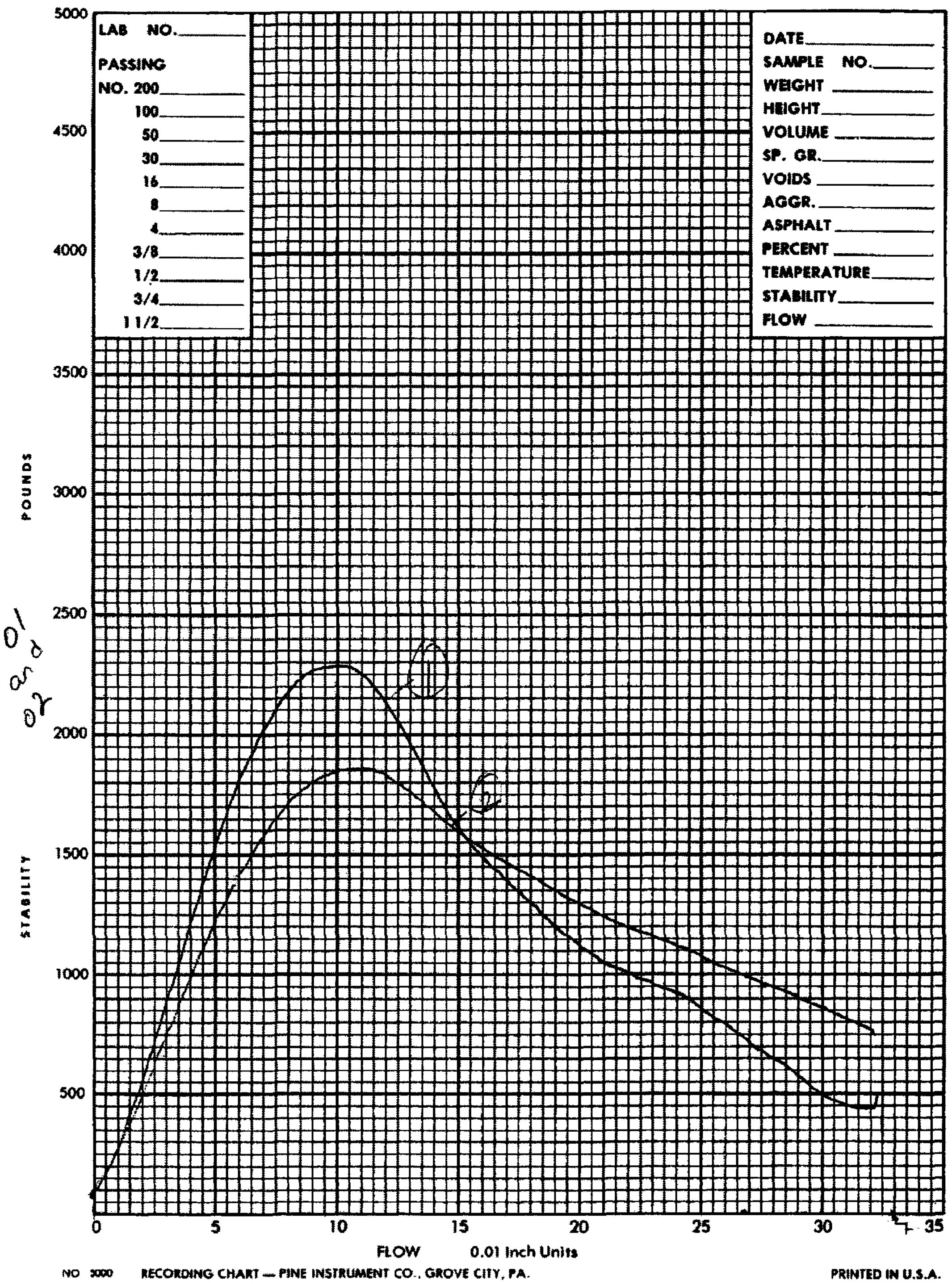


Figure C-1 Tensile strength test for samples O1 and O2

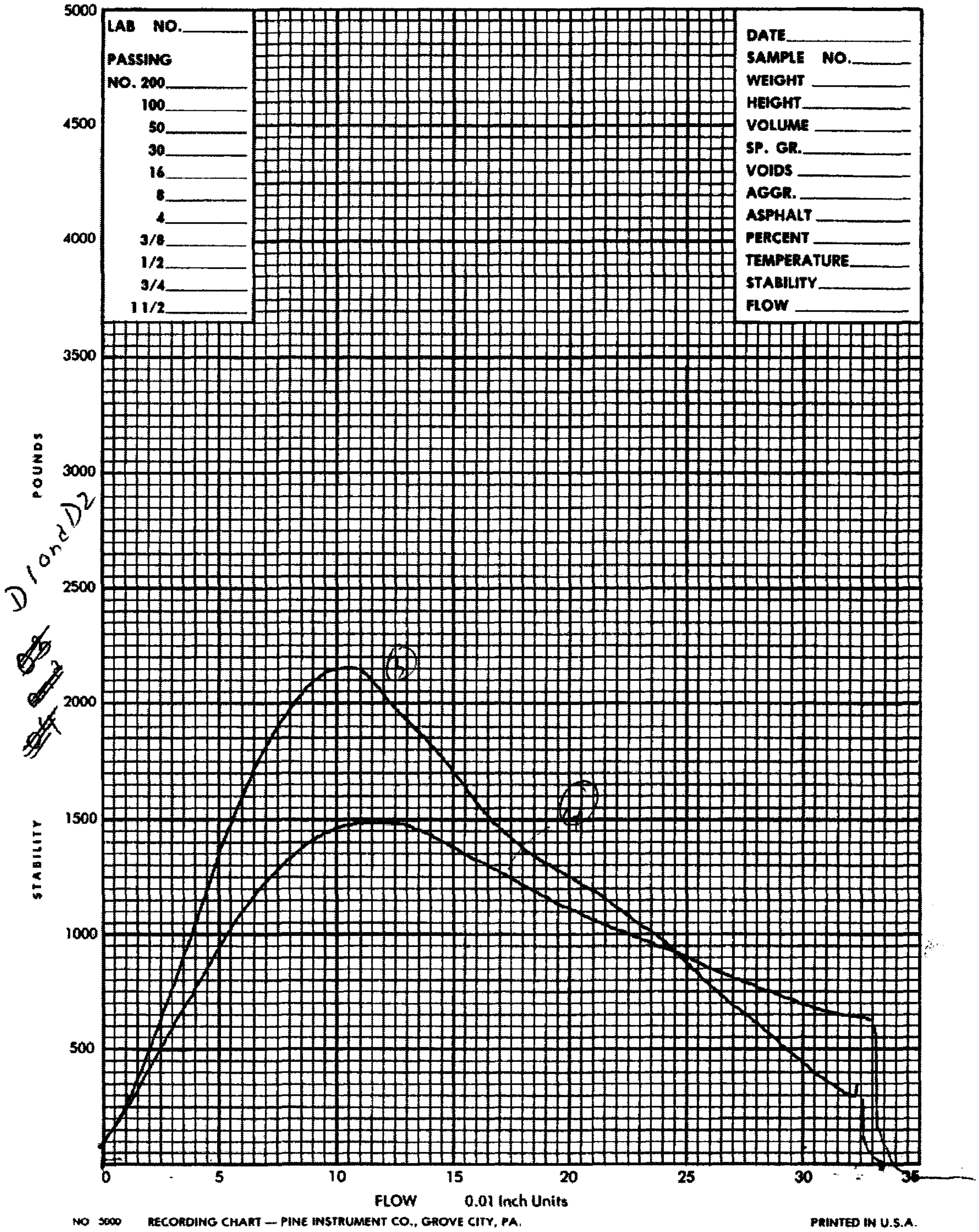


Figure C-2 Tensile strength test for samples D1 and D2

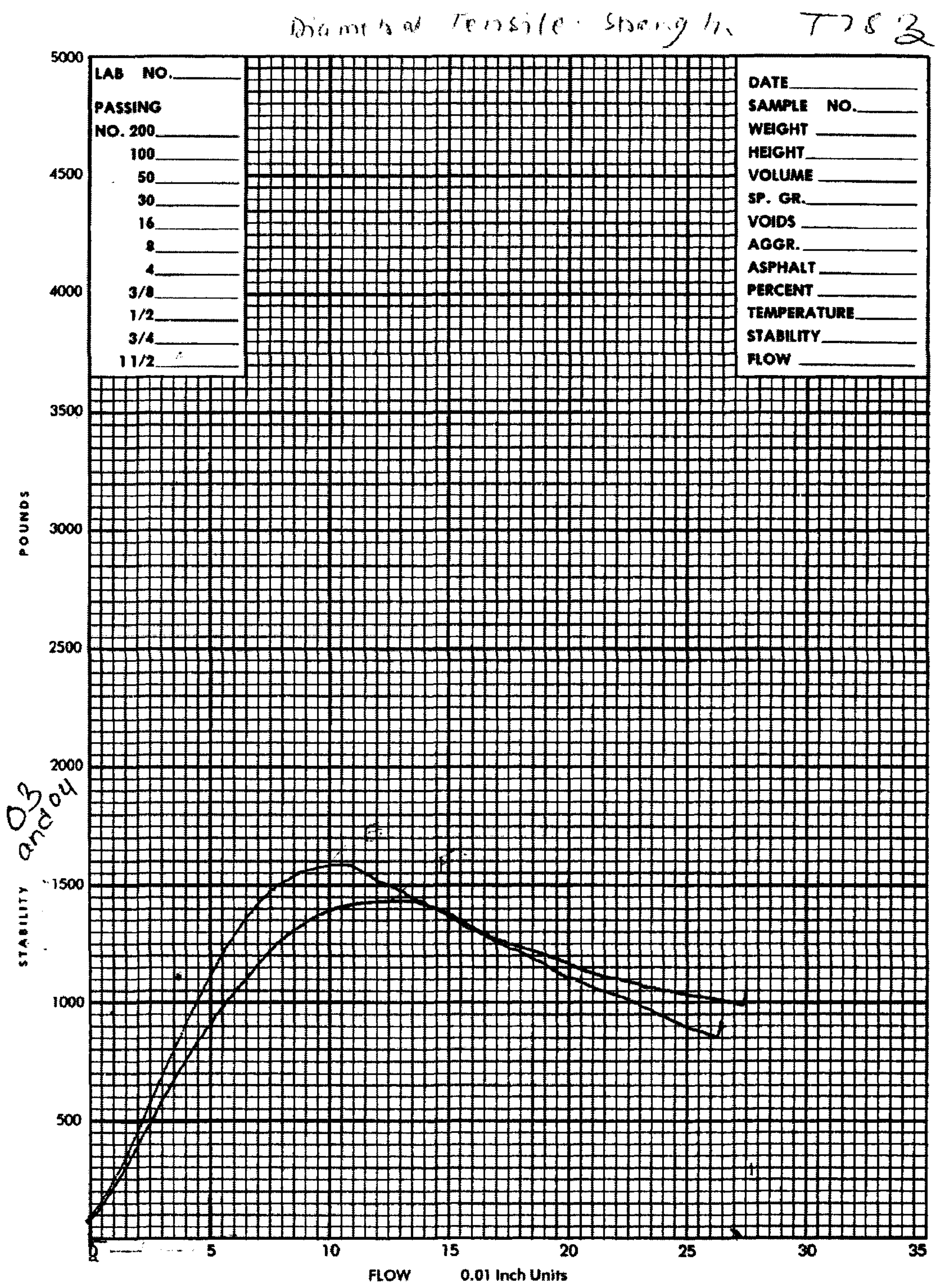


Figure C-3 Tensile strength test for samples O3 and O4

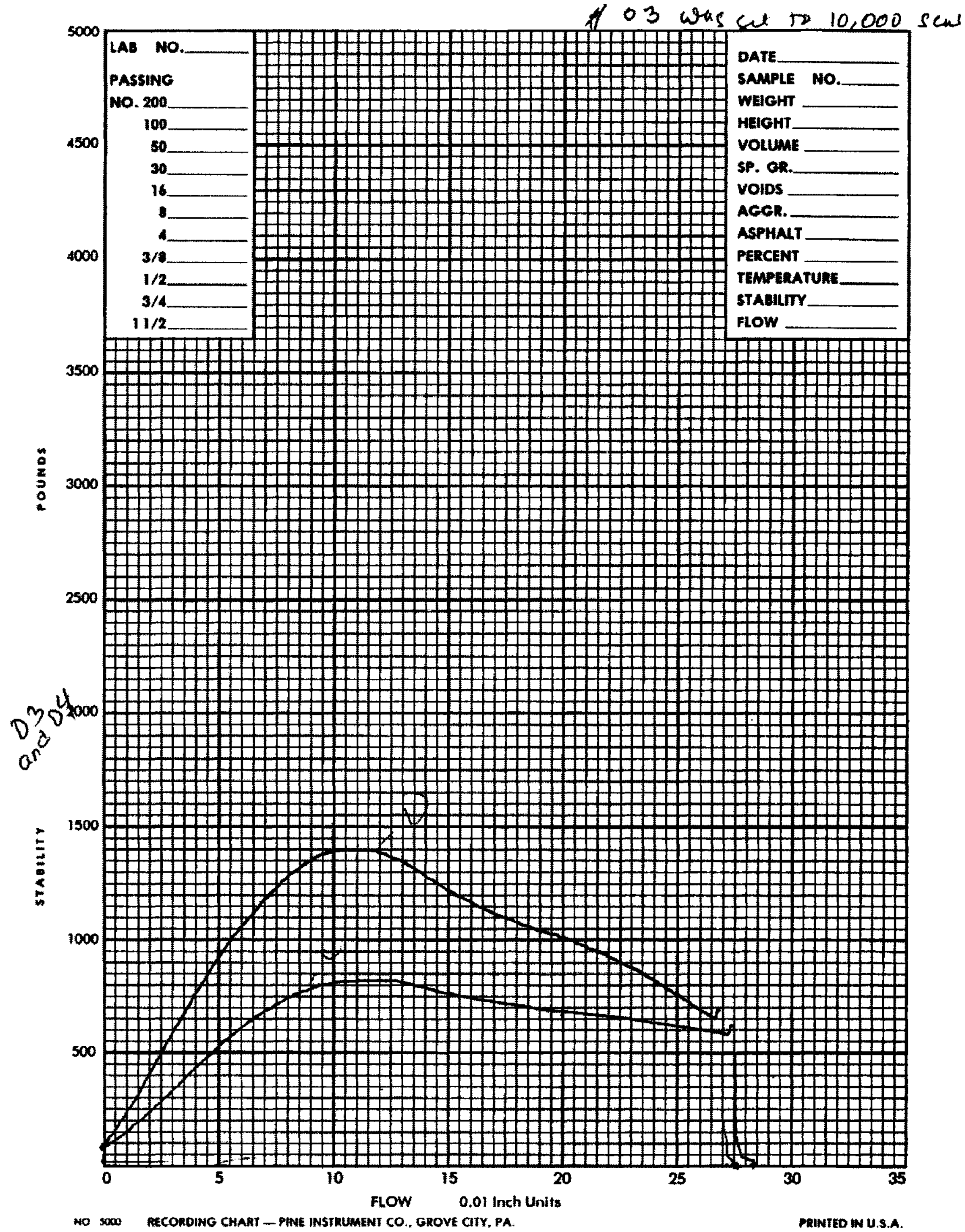


Figure C-4 Tensile strength test for samples D3 and D4

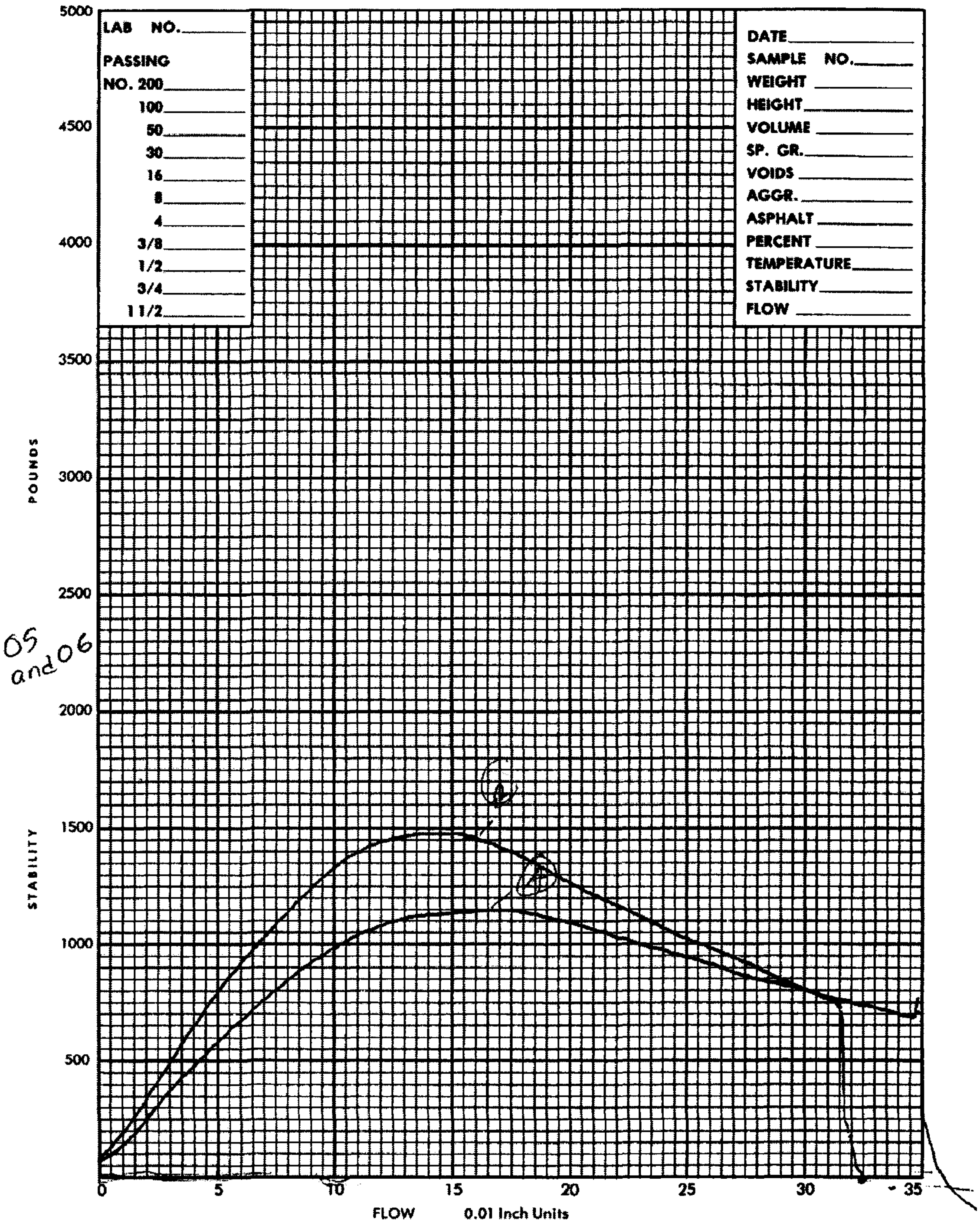


Figure C-5 Tensile strength test for samples O5 and O6

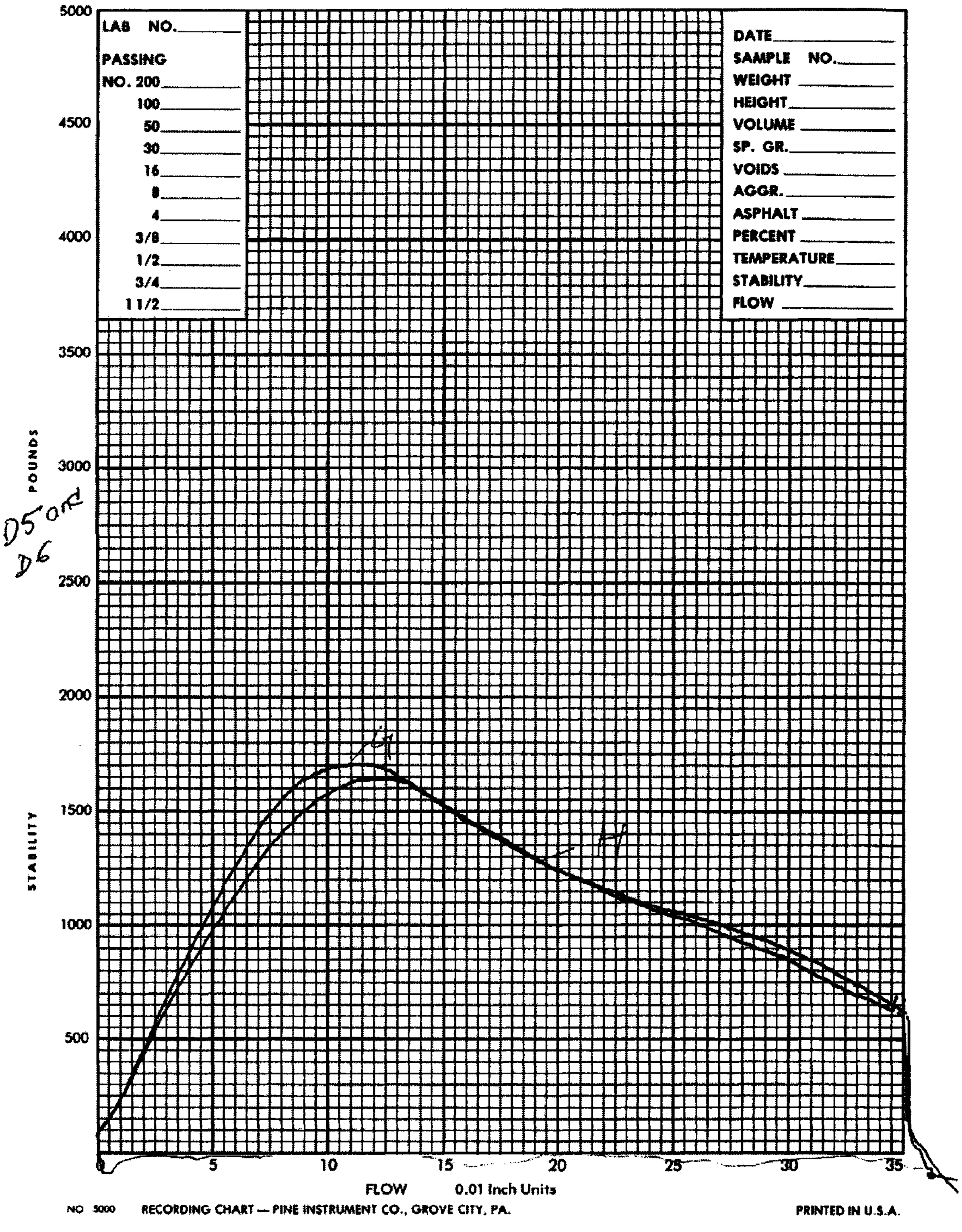


Figure C-6 Tensile strength test for samples D5 and D6

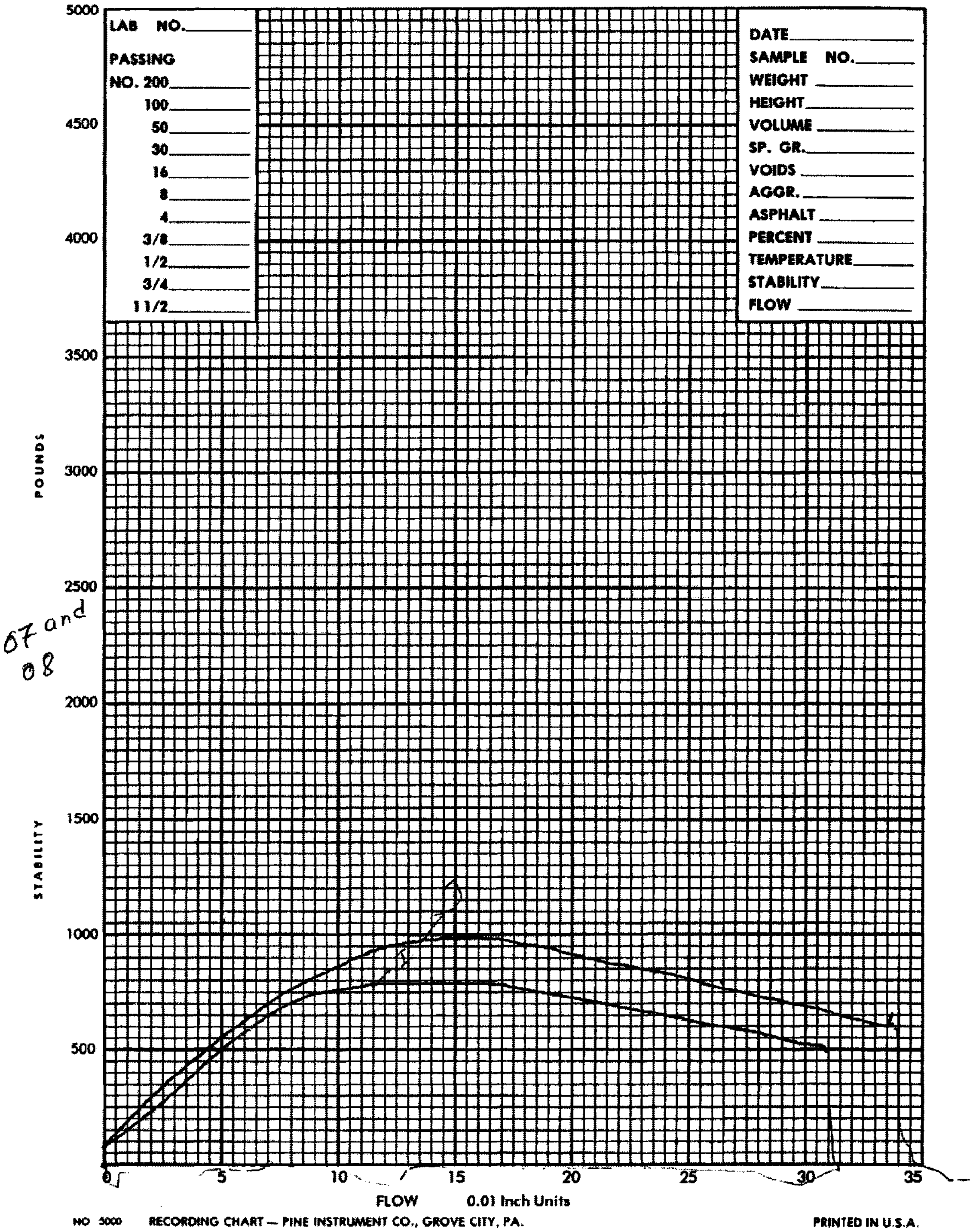


Figure C-7 Tensile strength test for samples O7 and O8

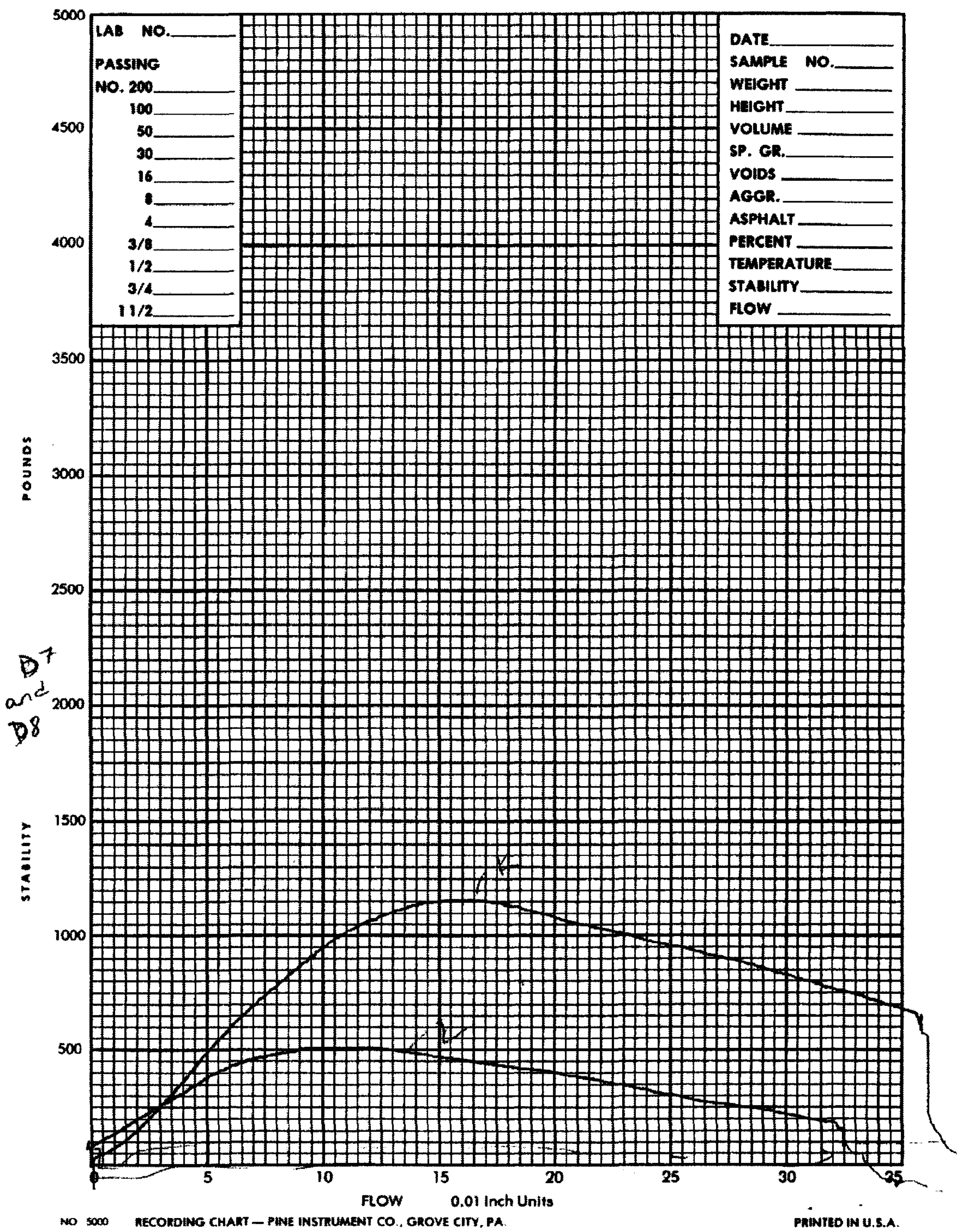


Figure C-8 Tensile strength test for samples D7 and D8