D-CRACKING IN MANITOBA

by

GLENN J. HERMANSON ©

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presented to the University of Manitoba
in partial fulfillment of the
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in
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ABSTRACT

The occurrence and susceptibility of local concrete aggregates to a freeze-thaw deterioration known as D-cracking, was studied in Manitoba. The deterioration of the coarse aggregate component of concrete is the primary cause of D-cracking. The destructive force is excessive hydraulic pressure within the aggregate during freezing. Physically the deterioration manifests itself as cracks on the surface of pavements, with subsequent spalling reducing the service life. A field survey of 17 % of the exposed concrete highway pavements and the Winnipeg Airport showed D-cracking had occured on 66 % of these surfaces. Concrete made with Birdshill aggregate having a maximum size of 40 mm displayed severe D-cracking on pavements with 18 to 25 years of service. Pavements incorporating Birdshill aggregate with a 20 mm top size showed no sign of D-cracking on a pavement 11 years old and minor cracking on pavements 16 and 20 years old. Highways built with Poplar Point and Stonewall aggregate showed no D-cracking after 16 and 21 years of service respectively. Laboratory testing of three different aggregates and four top sizes using a modified ASTM C-666 B freeze-thaw test showed Birdshill 40 mm top size aggregate to be susceptible to D-cracking. Birdshill 20 mm and Ritcher 28 and 14 mm top size aggregates were more durable than the Birdshill 40 mm top size aggregate. Reduction in top size improved the performance of the Birdshill aggregate. No increase in durability was achieved through size reduction of the Ritcher aggregate. A Gull Lake 20 mm dominantly igneous aggregate was the most durable aggregate of the three tested for D-cracking.

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Chapter I

INTRODUCTION

The expansion of transportation systems in the United States and Canada, during the 1950's and 1960's resulted in an increase in the use of portland cement concrete (PCC) in highway pavements. Concentrated construction in urban areas has placed heavy demands on high quality aggregate supplies. The use of PCC pavements has become increasingly more dependent on laboratory testing to delineate high quality aggregates from low or marginal quality materials. This has resulted in many PCC pavements being constructed with materials that have little or no field performance history.

Exposure of some of these pavements to freezing and thawing environments has produced a type of distress known as D-cracking. D-cracking is defined as "fine, closely spaced cracks which occur parallel and adjacent to longitudinal and transverse joints, intermediate cracks and the free edges of pavement slabs" (Stark 1976). The cracking is the result of freeze-thaw deterioration of the coarse aggregate fraction in the concrete. Three conditions are necessary for D-cracking. They are:

1. a deleterious aggregate
2. freezing temperatures
3. a source of moisture.

The degree of damage associated with this problem varies widely. The damage ranges from minor cracking and spalling at joints to complete failure of the pavement slab. D-cracking decreases the service life and increases the cost of maintenance of PCC pavements.

The problem of D-cracking was first identified in the mid-western United States in the 1930's. Remedial measures consisted of eliminating the suspect aggregates from use in concrete pavements. Numerous field and laboratory studies on the freeze-thaw durability of concrete were initiated in the 1940's and 1950's. Emphasis was placed on the behaviour of the cement paste. Only after the wide spread use of air entrainment improved the quality of the paste, did the magnitude of the problem of aggregate deterioration become apparent. Efforts to eliminate deleterious aggregates through laboratory testing have been unsuccessful as indicated by the many kilometers of highways constructed in the last 20 years which display D-cracking. Standard aggregate quality control tests cannot detect D-cracking aggregate. Several non standard tests, such as the modified ASTM freeze-thaw test and porosity studies have shown good correlation between predicted and actual field performance. However, these tests are expensive and very time consuming.

In Manitoba D-cracking has been reported on pavements built in the last 20 years. Preventive measures have included removing one highly susceptible aggregate from use in concrete. As well, the Manitoba Department of Highways has reduced the aggregate top size used in pavements to 20 mm (0.75 inch). However recent deterioration of the apron at the Winnipeg Airport and of provincial highways has shown that a need exists for an investigation into the D-cracking potential of aggregates used in Manitoba for PCC pavements. A limited study was carried out by the writer and the results are presented herein.

1.1 OBJECTIVES

The objectives of this study were:

1. To undertake a review of the literature on D-cracking and related topics, particularly literature dealing with field studies and laboratory testing methods.
2. To assess the extent and severity of D-cracking in Manitoba through a field survey of provincial highways and the Winnipeg International Airport PCC pavements.
3. To determine the relative freeze-thaw durabilities of coarse aggregate from several Manitoba suppliers, using a modified ASTM C-666B freezing and thawing test on concrete.

Chapter II

LITERATURE SEARCH

2.1 INTRODUCTION

The majority of literature on D-cracking originated from studies of concrete pavements initiated by several mid western States. Figure 1 shows the areas in which D-cracking has been reported in the literature. A detailed overview of the literature may be found in Thompson et al (1980).

Several field studies were undertaken to determine which factors influenced the development of D-cracking. Girard et al (1982) identified twelve possible factors influencing D-cracking. Factors significantly influencing D-cracking were:

1. Coarse aggregate
2. Moisture and drainage
3. Climate

Other factors having little or no influence on D-cracking were:

4. Type of subgrade soil
5. Fine aggregate
6. Type of cement
7. Concrete component proportions (design)

Figure 1: Location of D-cracking studies

8. Quality of concrete

9. Pavement design

10. Method of curing

11. Weather conditions during constructions

12. Traffic and loading

Studies by Traylor (1982), Missouri State Highways Dept. (1977), Stark (1970 and 1973), and others support these findings.

2.2 AGGREGATE CHARACTERISTICS AND MECHANISMS OF DETERIORATION

2.2.1 Coarse Aggregate

Of the variables influencing D-cracking, coarse aggregate is the single most important. Stark (1976) found most aggregates associated with D-cracking to be of sedimentary origin, primarily fine-grained carbonates or silicious rocks. The primary factor governing the freeze-thaw behaviour of rocks are the porosity, permeability and pore size distribution of the material (Powers 1955).

Studies on the pore structure of D-cracking aggregate indicated that fine pores can become critically saturated, that is, greater than 91 % of the pore volume becomes filled with water. Mercury intrusion tests showed non-durable aggregates to have a narrow pore size range (Stark 1976), while rocks with wide pore size distributions tended to be associated with durable aggregate. Kaneuji (1978) in a study of coarse aggregate, correlated reduced freeze-thaw durability with decreasing pore size down to a limiting size of 45 A. Pores smaller than 45 A showed improved resistance to D-cracking. Aggregates with a coarse or very fine pore structure were found to be more durable than rock with an intermediate pore structure.

2.2.2 Mechanisms

The majority of research into the freeze-thaw durability of concrete has concentrated on the cement paste component. Typically entrained air has eliminated freezing problems in cement paste to the point of being a non issue (Thompson et al 1980). The coarse aggregate component is now the major source of problems in the freeze-thaw deterioration of concrete. Understanding of an aggregate's freeze-thaw durability has evolved from comparisons of paste properties to rock properties and extrapolating the mechanisms in paste to account for the behavior of rock in a freeze-thaw environment.

In general, the pore structure determines a materials reaction to freezing as it controls how water is absorbed into a material and the temperature at which the water freezes.

Water is drawn into a rock by two mechanisms, capillary tension and absorption. Both mechanisms favour smaller pores (Thompson et al 1980).

Ice formation occurs at depressed temperatures in porous material depending on the size of the pore cavity. Depressed freezing points result from:

1. capillary pressures,
2. structural differences between adsorbed and bulk water,

3. lack of a suitable seed to nucleate crystallization, and
4. temperature size limitation with reguards to the size at which ice crystals are stable. This has the effect of limiting ice growth in small pores until the temperature drops further.

All of the above processes can occur in any porous material. Typically, water in paste freezes well below 0 degrees celsius, while water in aggregate may or may not freeze depending on the pore size. Coarse pore systems tend to freeze near 0 degrees celsius while fine-grained rocks see significant freezing point depression.

D-cracking is caused by the excessive dilation of aggregate generated during freezing and thawing. Four mechanisms of dilation have been identified in paste:

1. Hydraulic pressure theory
2. Desorption theory
3. Diffusion of ice theory
4. Dual mechanism theory

Each mechanism is more prevalent under specific pore size and freezing conditions.

2.2.2.1 Hydraulic Pressure

Powers (1955) attributed freeze-thaw failure in rock to hydraulic pressure during freezing. Ice formation in the pores displaces unfrozen water in an aggregate which is in a saturated or near saturated condition. Ice first crystallizes in the larger pores and then progressively forms in smaller pores as the temperature drops. The displaced water is under pressure and seeks relief through the body of the rock to the nearest non-saturated pore. Given low permeability, frozen pore channels or a lack of unsaturated pores, pressure build up will occur.

The magnitude of the pressure is a function of the:

1. Rate of freezing,
2. Distance to an escape boundary (critical thickness),
3. Degree of saturation and permeability.

If the pressure is greater than the tensile strength of the rock, failure will occur. An aggregates durability depends on its pore structure and its ability to relieve the pressure. The absence of unfrozen unsaturated pores or the grouping of similar pore sizes,allowing freezing to occur at once, will reduce a rock's freeze-thaw durability.

The 'critical thickness' which is the minimum distance "escape voids" must be placed to dissipate hydraulic pressure, is a function of a material's permeability (Powers 1955). In paste, this distance varies from 0.0025 to .0000025 mm. In rock the critical distance varies over a greater range up to several tens of millimeters. This variation partially explains why rocks with similar lithology have very different freeze-thaw durability. This also helps explain why reduction in top size improves the durability of many aggregates, but not all, as some rocks critical thickness may be smaller than a usable size therefore making the material unusable for pavement concrete.

Saturation effects the amount of hydraulic pressure as partially saturated materials will have more empty pores to provide relief. Total saturation levels below 91 % do not necessarily mean the particle will be free from distress during freezing. Heterogeneity of the moisture distribution can cause localized dilation which may fail the aggregate. Low permeability provides a rock with protection against saturation as the time to critical saturation is relatively long. However once a particle with low permeability has become saturated it is more prone to damage than a more permeable particle.

Aggregates with coarse pore structures offer relief to pressure build up and very fine pore systems do not become critically saturated easily due to small pore volume. Rocks with intermediate permeability and porosity reach critical saturation but offer no pore pressure relief. Such rocks are susceptible to excessive dilations under hydraulic pressure during freezing.

2.2.2.2 Desorption Theory

A second explanation for dilation has been proposed by Litvan. According to his theory, pressure is caused by a decrease in relative humidity. The gradient occurs due to the difference in vapor pressure of the ice and the supercooled water. Essentially once freezing begins the vapor system can only exist at the vapor pressure of ice. Any vapor present at pressures greater than ice will condense as ice or desorbed water. This water migrates towards the nearest escape void causing hydraulic pressure. In the desorption theory the dilation mechanism is also hydraulic pressure with the magnitude of the pressure depending on the same variables as Power's theory.

2.2.2.3 Diffusion of Ice

If low temperatures are maintained or slow cooling rates encountered, dilation may be caused by diffusion. Differences in the free energies of the three major forms of water, absorbed water in the gel pores, capillary ice and bulk water in the larger pores, cause adsorbed water to migrate towards the ice and capillary water. This migration results in the development of pressure.

Differences in solute concentration may also cause diffusion of pore water. Freezing will leave pure ice and a solution of higher solute concentration in the unfrozen water. This results in a concentration differential between the gel water and the solution in the capillaries. Diffusion and accompanying osmotic pressures result.

Diffusion is most likely to occur in fine-grained rocks exhibiting depressed freezing points and in situations where long slow freezing conditions are prevalent.

2.2.2.4 Dual Mechanisms Theory

In this theory hydraulic pressure causes dilation during the initial freezing period. The remaining observed dilation occurs after freezing due to the ordering of unfrozen water molecules on to pore and ice surfaces. As the volume of water initially frozen approaches 100%, the amount of dilation after freezing is less, lending support to the theory. The amount of dilation is dependent on the severity of the freezing and the amount of pore volume able to freeze at the prevailing minimum temperature.

2.2.3 Summary of Mechanisms

The predominant mechanism for any aggregate will depend on the pore structure of the particle and the temperature during freezing. Power's theory depends on volumetric expansion seen during the phase change from water to ice while Litvan's theory involves the desorption and migration of water, however the destructive mechanism is still hydraulic pressure. Evidence to support Litvan's theory is found when samples are cooled very slowly. Up to thirty per cent of the total pore water can be expelled under such conditions. Power's and the Dual Mechanism theories cannot account for this. The same behaviour is seen in tests using absorbates, other than water, which decrease in volume upon freezing. Diffusion is not important with respect to aggregates as freezing periods are not long and solutes are not found in pore water except where d-icing salts are used. Thompson et al (1980) concluded that the mechanism for dilation is neither one nor the other but a combination of the theories. Clearly the mechanism responsible for D-cracking is not well understood.

2.3 TESTING METHODS

Several testing methods have been used to predict the freeze-thaw performance of coarse aggregate. In correlating test results with actual field performance, each test has shown varying success. The most successful test in terms of identifying susceptible aggregate was the modified ASTM C-666B freeze-thaw test. Williams et al (1974) Traylor (1982), Paxton and Feltz (1979) and Stark (1976) reported good correlation between predicted performance from laboratory tests and actual field performance.

Other coarse aggregate tests include the sodium/magnesium sulfate test, adsorption test, Iowa pore index test and aggregate pore structure analysis.

2.3.1 ASTM C-666 Freezing and Thawing Test

In this test concrete prims are subjected to freezing and thawing cycles. The measurement of the dynamic modulus (sonic) is used to monitor the deterioration of the concrete. Method A freezes and thaws in water, while Method B freezes in air and thaws in water. The dynamic modulus is not a sensitive indicator of D-cracking (Powers 1955, Verbeck 1972). A more sensitive indicator of D-cracking potential is the permanent dilation of the prism. The replacement of dynamic modulus with length measurement in Method B of the standard showed good correlation between predicted freeze-thaw behaviour in the laboratory, and field performance (Paxton 1982, Traylor 1982 and others).

The principal findings, established in the literature, on the correlation between laboratory results and field performance were;

1. Large elongation of the concrete prisms indicates a D-cracking susceptible aggregate. The lower boundary of "large" varied from 0.06 to 0.10 %. Aggregates producing concrete exhibiting elongations larger than 0.20 % were always associated with poor field performance (Traylor 1982). Paxton (1982) reported a lower boundary of 0.02 %, that is, D-cracking was not associated with concrete undergoing elongations of less than 0.02 % .
2. Decreasing elongation with decreasing top size, within one aggregate source, indicated that a reduction in top size improved field performance.
3. In comparisons between aggregate sources, lower elongation generally indicates a more durable aggregate.

The modified ASTM C-666 B test's most desirable feature, after the delineation of a nondurable aggregates, is that it can indicate if a top size reduction will improve the aggregates performance. Other benefits of the test are;

1. Consistent test result were achieved between different testing laboratories (Paxton 1982), and
2. The test result can be extrapolated to aggregates without field records, providing a failure criteria is determined from aggregates with field records.

Paxton (1982) reported that correlations between laboratory and field records were improved by comparing the area under the elongation versus freeze-thaw cycles curve, rather than the final elongation values. No other data analysis was reported in the literature.

2.3.2 Iowa Pore Index Test

This test is described in Myers and Dubberke (1980) and is a relatively simple, but inconclusive, method for determining the D-cracking potential of aggregate. The test measures the amount of water an aggregate particle will absorb in a short time under a small pressure. High absorption values are correlated to high D-cracking potential while low absorption values indicate more durable aggregate.

The Iowa Pore Index test was successful in identifying D-cracking aggregate in rocks with a substantial number of pores in the range of 0.04 to 0.02 micrometers. However tests of aggregates with non uniform pore systems were inconclusive (Marks and Dubberke 1982). Traylor (1982) reported substantial overkill using the test, that is, the test identified aggregates with satisfactory service records as being susceptible to D-cracking. Traylor also found that the test was not useful for gravels. The test cannot indicate if a reduction in top size would improve performance and was only recommended as a prescreening test for aggregate.

2.3.3 Pore Structure Studies

Studies on the pore size distribution of aggregate using mercury porosimetry tests were undertaken in Indiana and are reported in Kaneuji (1978), Kaneuji et al (1980) and Winslow et al (1982). The test measures the pore size distribution of the aggregate particle. This distribution is then compared to service records of coarse aggregates. Kaneuji (1978) developed an equation which predicted frost durability based on the aggregate's pore structure. The correlation was calculated for a limited number of aggregate sources and appeared to have significant potential as an approval type of test. However the technique is limited by the ability of the tester to obtain a representative sample of material.

2.3.4 Other Tests

Dilation occuring during the 'freezing of a particle' was not useful for determining D-cracking potential. (Verbeck et al 1972). Paxton and Feltz (1979) reported that the sodium/magnesium sulfate or the 10% calcium chloride tests did not delineate D-cracking susceptible aggregate from durable aggregate. The absorption-adsorption test was not useful in aggregate testing for D-cracking (Williams et al 1974).

2.4 FIELD STUDIES

D-cracking, being the result of a material property of the aggregate and not of concrete, cannot be eliminated by design procedures. Two alternatives exist for the control of D-cracking:

1. Elimination of adverse moisture and/or freezing conditions
2. Elimination of aggregates having D-cracking potential.

Clearly the elimination of freezing conditions is impossible, leaving moisture and aggregates as the two possible controls in pavement design.

2.4.1 Moisture Studies

In the field moisture accumulation occurs rapidly and is difficult to stop. Stark (1970), from studies on concrete slabs, reported that moisture accumulations in field pavements were sufficient to initiate D-cracking within two freeze-thaw seasons. In field tests in Kansas, only 24 hours was required for moisture to penetrate 50 mm into a field core. Attempts to prevent moisture migration into the concrete were unsuccessful. In a long term study, the Missouri State Highways Department concluded that polyethylene sheets underlying the highway did not reduce the occurrence of D-cracking. Pavements underlain with a 4 mil polyethylene moisture barrier showed D-cracking earlier than pavements without the barrier (Girard et al 1982).

Bukovatz and Crumpton (1981) reported heat drying of the aggregate in the field prior to batch mixing was not beneficial in preventing D-cracking. Lankard et al (1974) reported predrying aggregate prior to testing delayed but did not eliminate D-cracking in laboratory testing. These studies would indicate that the predrying of aggregate is a very short term solution.

Increasing the surface area of the pavement by increasing the number of joints or uncontrolled cracks increased the development of D-cracking. Crumpton et al (1974) reported that pavements with map cracking showed increased occurrences of D-cracking. Continuously reinforced pavements also showed this correlation (Traylor 1982).

In field studies in Kansas (Best 1974) attempts were made to arrest D-cracking once it was initiated. The techniques used where:

1. Sealing the pavement with;
   a) linseed oil and mineral oil,
   b) commercial resin sealer, and
   c) commercial silicon.

2. Grouting the base with;
   i) linseed oil emulsion, and
   ii) commercial silicate based grout.

All methods were designed to seal the concrete from excess moisture. They were not successful.

Best (1974) noted that D-cracking does not occur uniformly along a stretch of pavement and was more developed on the lower side of super-elevated curves. Deterioration was reported to be greater where drainage was poor (Girard et al 1982). The type of subbase or drainage design had no significant effect on the development of D-cracking (Traylor 1982).

Drainage and moisture accumulation are important variables in the development of D-cracking. Presently, there are no effective drainage or sealing methods available to control moisture accumulation in D-cracking pavements.

2.4.2 Coarse Aggregate

Field studies have shown strong correlations between coarse aggregate sources and the rate and severity of D-cracking. Typically D-cracking will appear in pavements 5 to 6 years in age (Girard et al 1982) and is rarely noted in pavements younger than 5 years in age (Missouri State Highway Department 1977).

Limestone aggregates are the most susceptible to deterioration. Best (1974) reported that D-cracking occurred in concretes with 30 % or more deleterious limestone coarse aggregate. Bukovatz et al (1973) reporting a similar correlation of 35 %. Marks and Dubberke (1982) found that 15 % or more of a nondurable aggregate incorporated into a pavement will cause the concrete to exhibit D-cracking.

The reduction of top size was the only effective way of improving the performance of marginal aggregates (Missouri State Highway Department 1977, Verbeck et al 1972 and others). This reduction in size of the aggregate particle reduces the material to below its critical size and therefore eliminates the build up of destructive hydraulic pressures. The improvement in freeze-thaw durability can also be duplicated in the laboratory (Missouri State Highway Dept. 1977) allowing the prediction of the improvement an aggregate will experience through size reduction. Not all aggregates, due to different pore structures, will benefit from size reduction until the particle is below a usable size. In other cases, size reduction has been observed to delay but not eliminate the occurrence of D-cracking. Size reduction, while providing a practical technique in upgrading D-cracking aggregates, cannot be used indiscriminately as, the amount of improvement seen is unique to each aggregate.

Other field observations reported in the literature were:

1. Core drilling is an effective method for determining D-cracking before it manifests itself on the surface (Williams et al 1974).
2. Traffic loading promotes the unravelling of D-cracking surfaces but is not necessary for it's occurrence (Bukovatz and Crumpton 1981, Traylor 1982).
3. Gravels are difficult to sample and show much more inconsistent performance than crushed rock.
4. D-cracking has never been observed on bridge decks (Stark 1976) or structural members subject to periodic drying.

2.5 CONCLUSIONS

From the literature the following conclusions may be drawn:

1. D-cracking is a freeze-thaw type of deterioration generally seen only in Portland Cement Concrete pavements
2. Deterioration of the coarse aggregate particles is the primary cause of D-cracking.
3. Fine-grained sedimentary rocks, especially limestones are the most susceptible to the problem.
4. Fine aggregate, cement type, concrete design, pavement design, drainage design and end use (loading) have no influence on the development of D-cracking.
5. Hydraulic pressure, through the phase change from water to ice and/or the desorption of adsorbed water, is most likely responsible for the destruction of the aggregate during freezing.
6. The modified ASTM C-666 B freeze-thaw test can identify D-cracking susceptible aggregate. Other aggregate tests, with the exception of aggregate pore structure studies, cannot.
7. No method of preventing D-cracking by eliminating moisture from pavements is presently available.
8. Reduction in top size of a coarse aggregate is a practical method of improving the durability of the coarse aggregate. The improvement in performance can be duplicated in the laboratory, providing a method for predicting the improvement in a field application.
9. No long term solutions to the problem, other than removing the deleterious aggregate from service, have been found.

A complete Bibliography on D-cracking is compiled in the Reference section.

Chapter III

FIELD SURVEY

3.1 INTRODUCTION

D-cracking is a progressive, time dependent type of concrete pavement deterioration. The amount of distress exhibited by a pavement is primarily a function of the type of coarse aggregate used and the age of the concrete. In order to determine the susceptibility of local coarse aggregates to D-cracking a limited survey of Manitoba's PCC pavements was conducted. The description and results of the work are outlined in the following sections.

3.1.1 Objectives

The objectives of the field study were:

1. To evaluate the extent and severity of D-cracking on PCC pavements in Manitoba.
2. To compile the construction records of the above pavements.
3. To identify susceptible coarse aggregates and develop aggregate performance records to aid in the interpretation of laboratory testing results.

3.1.2 Location of Survey

The study was confined to the Red River valley where the majority of Manitoba's PCC pavements exist. The initial scope of the survey intended to systematically assess the provincial highways, several airports and undertake a statistical survey of the pavements in the City of Winnipeg. Difficulty in obtaining construction records limited the study to eleven Provincial Highway pavements and the taxi apron at the Winnipeg International Airport. The locations of the highway pavements studied are shown in Figure 2 and and are compiled along with the designations and coarse aggregate type used, in Table 1 . Four Winnipeg streets were surveyed before the difficulty in obtaining detailed construction records became apparent. These pavements are not included in the data base. The raw data is presented in the appendix.

Manitoba contains approximently 805 kilometers of PCC pavements of which 37 per cent are covered by an asphalt lift. The remaining 507 kilometers make up the total surveyable pavements. Three sections of provincial highways were surveyed without prior knowledge of their construction records. Later difficulty was encountered in compiling detailed construction records. In order to avoid this situation for the remainder of the study, only pavements with a known construction history were considered. The final highway survey consisted of 89.3 kilometers of equivalent two lane pavements. This represents 17 per cent of the exposed PCC pavements in Manitoba. The study included pavements from 11 to 37 years old with 98 per cent being in the 11 to 25 year old bracket. Table 2 summarizes the age breakdown for the provincial highways.

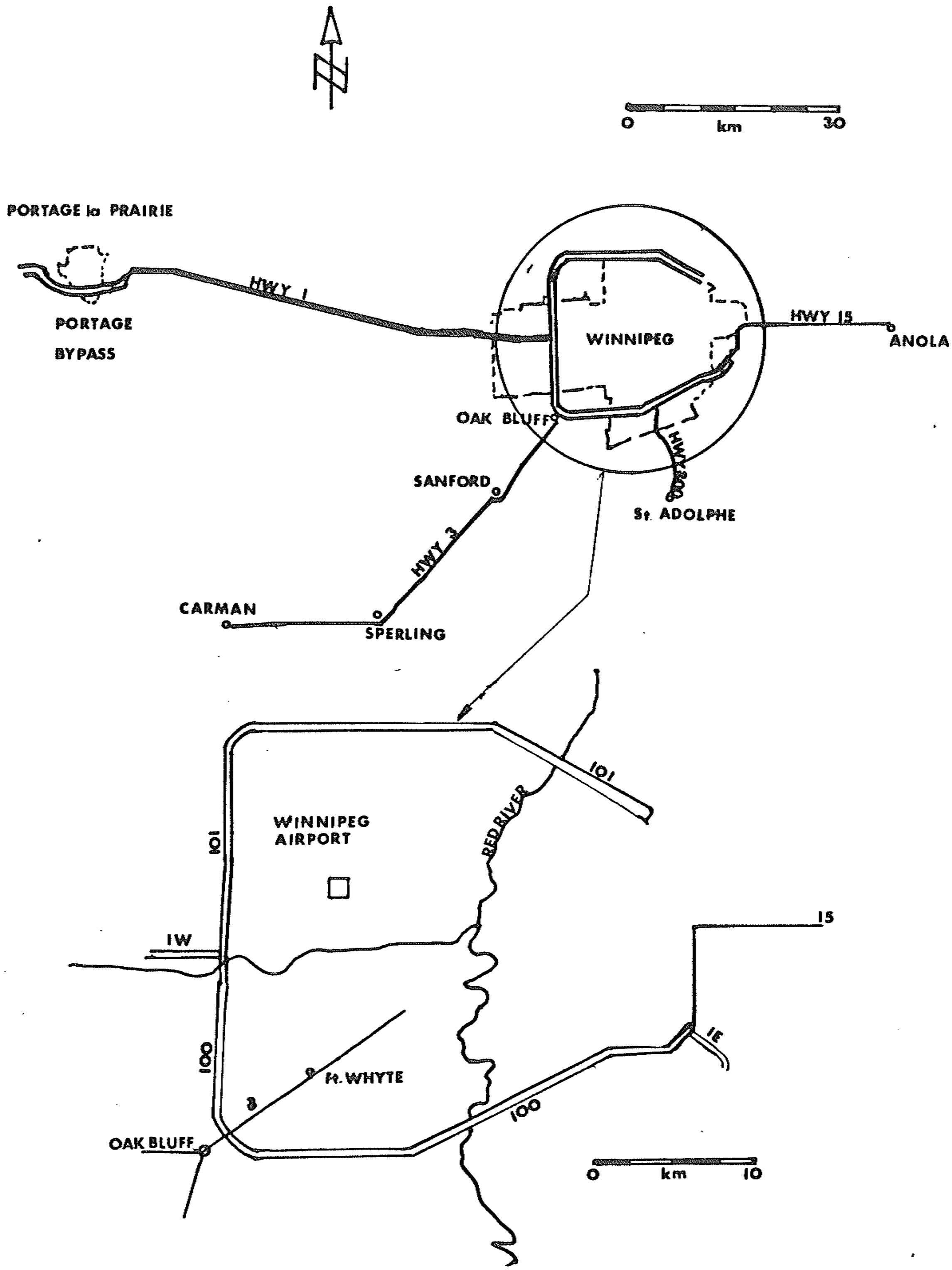

Figure 2: Location of study area

TABLE 1

Mileage, Location and Aggregate Data

Highway	Designation	Location	Coarse Aggregate	Length (km)
1	1BP	Portage Bypass	Ritcher	6.0
3	3FW	Fort Whyte	N.A.	1.5
3	3 F100	Ft. Whyte to Hwy 100	Stonewall	6.1
3	3 OB	South of Oak Bluff	N.A.	9.5
3	3 CS	Carman to Sperling	Birdshill	11.7
15	15	Hwy 101 to Anola	Birdshill	19.0
100E	100E	Hwy 3 to St. Annes Rd.	Birdshill	13.8
100W	100W StA	St Annes Rd. to St. Marys Rd.	N.A.	3.5
100W	100W 3RB	Hwy. 3 to Roblin Blvd.	N.A.	8.2
101	101	Hwy 1E to Hwy 15	Birdshill	5.3
200	200	North of St. Adolphe	Birdshill	4.7
				89.3

N.A. = Not Available

TABLE 2

Data Base of Surveyed Pavements

Age (years)	Length (km)	Fraction of Total (%)	Asphalt Overlay (km)	Asphalt Overlay (%)	Exposed Pavement (km)	Pavement Surveyed (km)	Pavement Surveyed (%)
<5	49.1	6.0	0	0	49.1	0	0
6-10	30.6	3.7	6.4	21	24.1	0	0
11-15	120.7	14.7	0	0	120.7	19.0	15.7
16-20	255.9	31.1	59.5	23	196.3	41.5	21.1
21-25	120.7	14.7	34.6	28.7	86.1	27.4	31.8
>25	244.6	29.8	201.2	82	43.5	1.5	3.3
TOTALS	821.6	100%	301.7	36.7	519.8	89.3	17

1. The information was obtained from a base map supplied by the Department of Highways.

3.2 FIELD SURVEY METHOD

3.2.1 General

In order to rate the pavement for D-cracking, a survey method had to be developed which met several conditions. First the survey had to provide a reliable determination of the pavement condition. Secondly, the method had to allow for a relatively rapid assessment of large networks of pavements. The former criterion dictated that the survey method be reproducible and free from personal bias. The latter was an important consideration as manpower constraints existed within the project. The literature was reviewed from which a simple visual rating scheme was synthesized.

The rating scheme chosen was similar to the one used in Illinois (Traylor 1982). The system assigned a number of 0, 1, 2 or 3 to a pavement surface using the following criteria:

0 No D-cracking evident on the surface of the pavement.

1 Minor D-cracking. Minor cracks and/or some staining visible.

2 Significant D-cracking. Cracks are well developed. Concrete shows the classic D-cracking pattern. The serviceability of the pavement has not been affected.

3 Severe D-cracking. Cracking very well developed. Concrete has started to fail. The serviceability of the pavement has been reduced.

An example of each classification is shown in Figure 3 . The criteria attempted to define boundaries between degrees of deterioration but avoided quantifying the damage too precisely. Detailed rating systems are susceptible to personal bias, are influenced by other types of deterioration and are very time consuming. The dividing line between a rating of 2 and 3 is an arbitrary one and varies between agencies, however it was felt that the criteria would adequately represent the condition of the concrete and allow comparisons to be made between

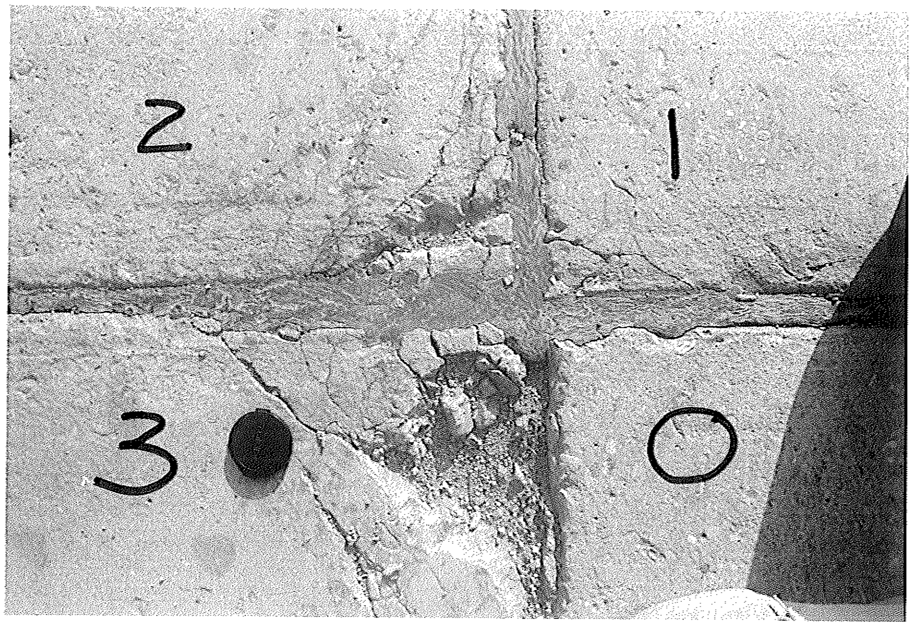

Figure 3: D-cracking rating

different pavements.

An initial trial survey was conducted in June 1983 and the field technique verified. A visit to the Portland Cement Association Laboratory at Skokie, Illinois, USA, afforded the author an opportunity to observe well documented pavements and to become familiar with the cracking patterns associated with D-cracking. The majority of highway pavements were surveyed from July to September 1983. Several minor sections were rated at a later date in order to fill gaps in the data base which occured as a result of unavailable construction records. In total 95.1 kilometers of pavements were rated on provincial highways and City of Winnipeg streets along with the Winnipeg International Airport. Over 17000 joints were inspected and their condition recorded. The survey took three months to complete.

3.2.2 Procedure

Field work consisted of rating the condition of the concrete surface, joints and free edges of the pavement. The survey was done by car and on foot. Initial surveillance revealed the deterioration of any given construction section to be fairly uniform. This allowed the averaging of large numbers of data points in the field. A diagrammatic illustration of the nomenclature used during the survey is given in Figure 4 . These conventions are used in the remainder of the text. The following procedure was used to rate and record the values:

1. Ratings were given to each joint section[1] of pavement. The values were averaged in the field for every 152 meters surveyed and recorded.
2. When both lanes of a standard undivided highway, or one direction of a divided highway, had the same degree of D-cracking they were rated together. If values differed significantly then 2 sets of ratings were recorded. A final value for the two lanes was computed to allow comparisons with highways whose lanes had been rated together.

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[1] A section was the area bounded by a pavement edge, longitudinal joint, and two transverse joints.

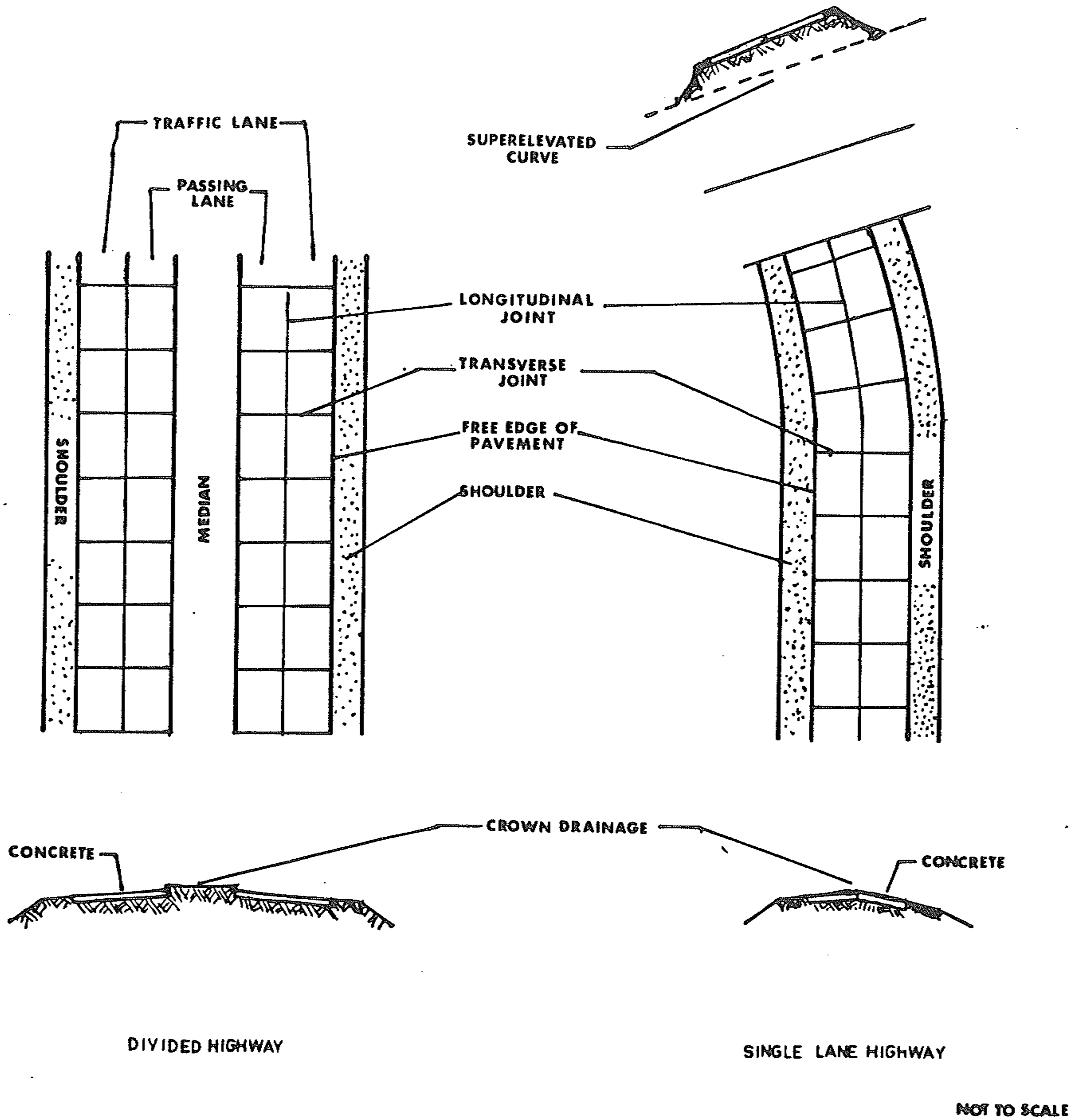

Figure 4: Survey nomenclature

3. Anomalous sections were identified and described in detail.

4. Photographs were taken of typical conditions on each construction section.

5. The interval ratings were tabulated and a final pavement rating calculated.

A data sheet is shown in Figure 5 .If possible the construction contract boundaries were identified before the pavement was assessed. This eliminated the error of averaging a 152 meter interval over a boundary between two different construction sections. Other types of deterioration were described and recorded using the ACI terminology as set out in Lauer (1968). These observations were for supplementary information and did not affect the rating of the pavement. They are included in the appendix. Where joints were repaired or replaced and there was no obvious evidence of D-cracking at the repaired joint no value was assigned to the concrete. The repaired joint was simply eliminated from the survey to avoid assuming a value of 3 to a joint that may have been damaged by another type of deterioration. Large numbers of repairs where noted to aid the analysis.

Deviations from the survey methodology were necessary while undertaking the field work. They occurred when the 152 meter averaging interval was not sensitive enough to properly describe the pavement. For example in a 152 meter interval, if 14 joints rated 2, 9 joints rated 1 and 2 rated 0, a single rating of 1 or 2 would not describe the interval. The rating would then be adjusted by averaging the readings as follows:

$$(14 \times 2)+(9 \times 1)+(2 \times 0) = 37$$

$$37 / 25 \text{ readings} = 1.48 \text{ which was rounded to } 1.5$$

LOCATION

DATE

AIR ENTRAINED

PAVEMENT DESIGN

AGGREGATE

PIT LOCATION

PROPERTIES

D-CRACKING SURVEY

STATION	RATING 0 1 2 3	SCALING	POPOUTS	COMMENTS

Figure 5: Data Sheet

Calculations of this type were rounded to the nearest 0.5 of a rating. More precise averaging would have over extended the accuracy of the empirical rating scheme. The above method was used when necessary, however for the large majority of readings a single rating was adequate to accurately describe a 152 meter interval.

The Winnipeg International Airport (WIA) apron was rated in January, 1985. The apron was divided into a grid using the existing construction joints as boundaries. Each rectangle defined by four joints was assigned a D-cracking rating according to the aforementioned criteria. Areas of repair were identified and considered separately. Using these values an average D-cracking value was then calculated for the entire apron pavement.

3.3 RESULTS

The final Provincial highway and Winnipeg Airport pavement ratings are compiled along with the pavement age and coarse aggregate sources in Table 3 and Figure 6 . The pavements were divided into the following groups according to aggregate source and top size:

1. Pavements built with a 40 mm aggregate from the Birdshill area.
2. Pavements built with a 20 mm aggregate from the Birdshill area.
3. Pavements built with either 20 mm Poplar Point or Stonewall aggregate.
4. Pavements built with aggregates of unknown source.

The Manitoba Department of Highways classifies the 40 mm top aggregate as Type 1 and the 20 mm aggregate as Type 2. The gradations of the two classes are shown in Table 4 .

These groups were examined for:

1. The D-cracking severity of the pavements.
2. Time deterioration relationships and any external variables such as moisture or traffic loading conditions which could influence the rate of deterioration.
3. Relative susceptibility to D-cracking.

Each group is discussed separately in the subsequent section.

TABLE 3

D-cracking rating of Manitoba pavements.

Highway	Age (years)	Coarse Aggregate	Top Size	Length (km)	Rating
100E	18	Birdshill	40	13.8	2.35
200	16	Birdshill	40	4.7	0.62
WIA	21	Birdshill	40	-	2.2
100W 3RB	24	N.A.	40	8.2	2.17
100W StA	25	N.A.	40	3.5	2.10
3 CS	20	Birdshill	20	11.7	0.37
101	16	Birdshill	20	5.3	1.10
15	11	Birdshill	20	19.0	0.0
1E PB	16	Pop.Point	20	6.0	0.0
3 F100	21	Stonewall	20	6.1	0.0
3 FW	37	N.A	-	1.5	1.81
3 OB	21	N.A.	-	9.5	1.01
				89.3	

N.A.-Aggregate source was unknown.
Note: Aggregates with 40 mm top size are classed Type 1 and aggregates with 20 mm top size are classed Type 2 by the Manitoba Department of Highways.

TABLE 4

Size specifications for Type 1 and 2 aggregate

Size (mm)	Type 1 (% passing)	Type 2 (% passing)
51	100	--
40	90-100	--
25	70-95	100
20	--	90-100
13	10-30	--
9.5	--	20-55
No.4	0-5	0-10

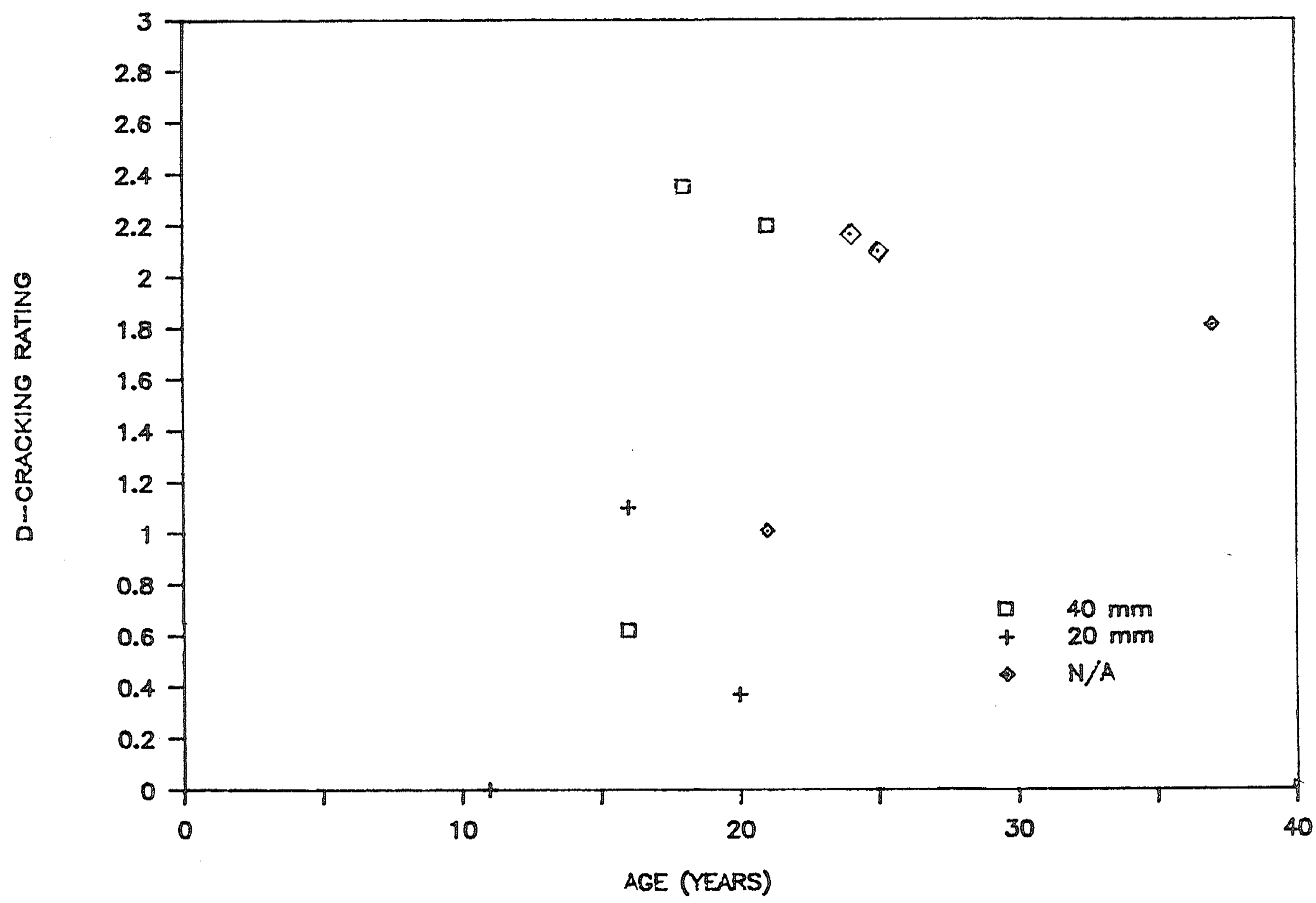

Figure 6: D-cracking rating of all pavements

3.3.1 Birdshill Type I

The pavements included in this group were Highways 100E, 200, and the apron at the airport. Two pavement sections, 100W 3RB and 100W STA were rated under the assumption that they were built with Birdshill Type 1 aggregate. Although the source of aggregate could not be verified for these locations both pavements had a top size of 40 mm and visually similar deterioration as the other pavements in this group. It was also very likely that these pavement sections were constructed with the Birdshill aggregate due to their close proximity to the Birdshill pit.

All the divided highways, along with the airport apron had severe D-cracking with pavements rating greater than 2. Highway 200 had a lower incidence of D-cracking than the others and in general the pavement was in better shape. On the divided highways the traffic and passing lanes showed different degrees of D-cracking. The traffic lanes were more severely D-cracked as shown in the single lanes aver-

TABLE 5

Single lane ratings of divided highways

Highway	Traffic	Passing	Combined
100E*	2.44	1.47	1.96
100W 3RB	2.62	1.72	2.17
100W StA	2.35	1.82	2.10

*4.75 of 13.8 kilometers of 100E had the two lanes rated separately.

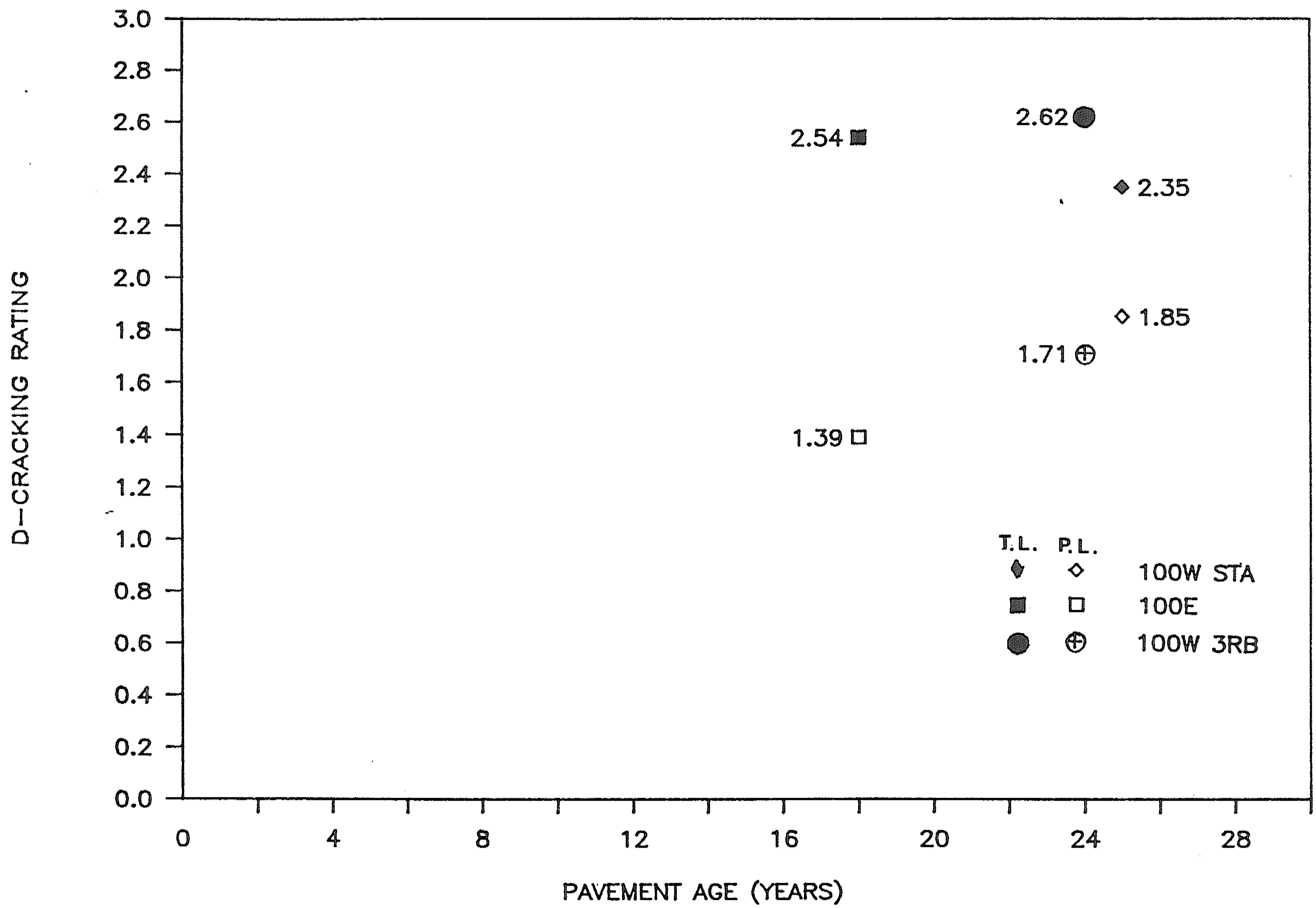

Figure 7: Rating of passing and traffic lanes

ages listed in Table 5 and plotted in Figure 7 . Initially traffic loading was thought to explain the increased rate of deterioration, however the higher D-cracking rating was not always observed in the traffic lanes.

A change in D-cracking rating, from higher ratings in the traffic lane to higher ratings in the passing lane, was observed on several curves inspected during the survey. On highway 100W 3RB at station 13+70 meters the D-cracking rating between the traffic and passing lane reversed. The ratings then returned to the precurve pattern at the end of the curve. This situation is shown in Figure 8 which plots D-cracking rating versus survey station. A similar situation was also seen on highway 100E at station 0+75 meters, south of Hwy 3. Here the passing lane had suffered more deterioration than the traffic lane along the curve. This was the only section on 100E which showed more severe deterioration in the passing lane.

Increased D-cracking on curves was observed on highway 200. This pavement is an undivided highway which roughly parallels the Red River. The geometrics consisted of tight superelevated curves and straight sections with crown drainage. There was only minor D-cracking and staining on the straight sections joining the curves. At the curves the severity of the cracking increased. Figure 9 shows this pattern clearly. Ratings as high as 2 were observed on the downslope side of the curve with minor cracking present on the high side of the curve.

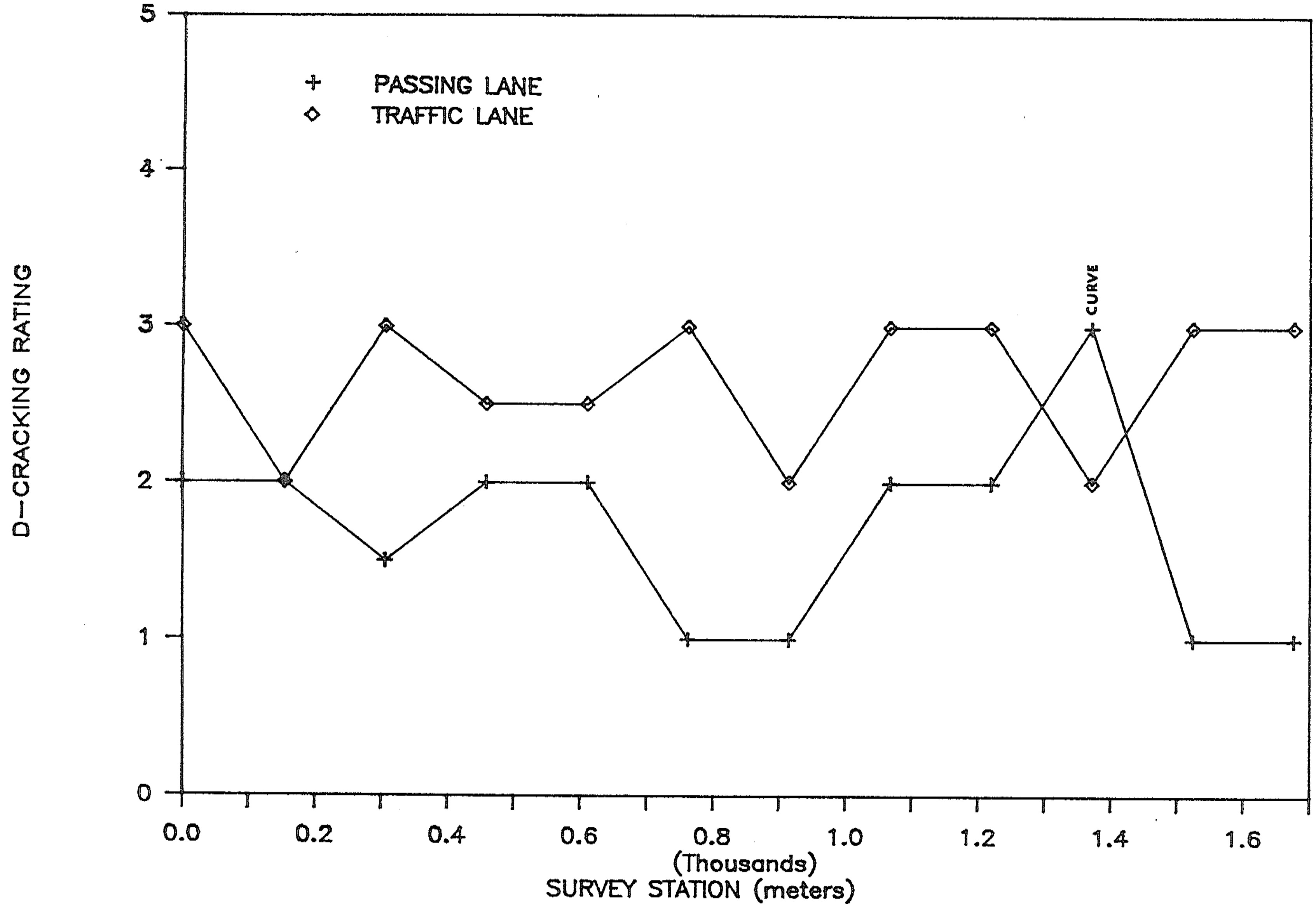

Figure 8: Highway 100 3RB Increased D-cracking at curves

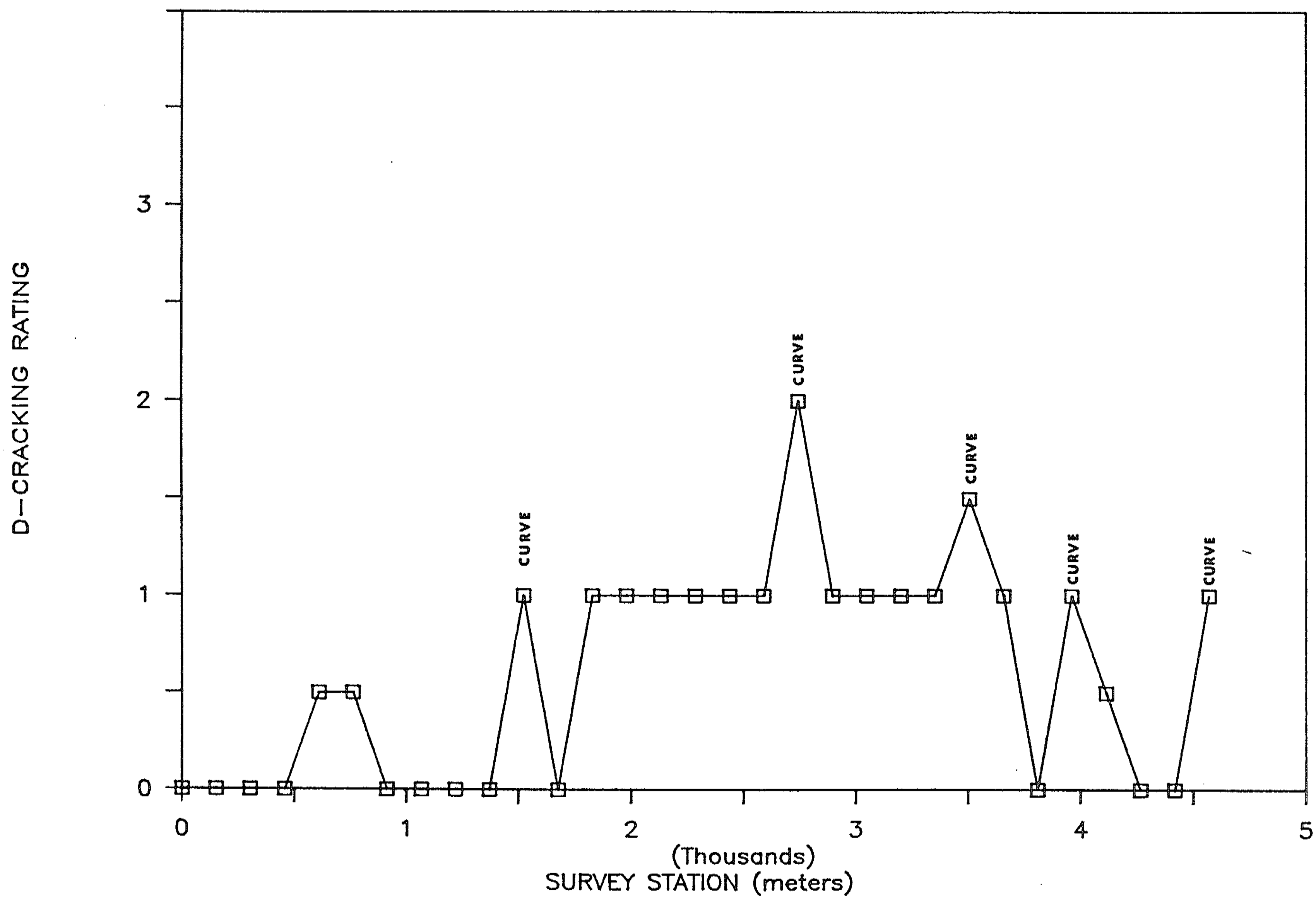

Figure 9: Highway 200 D-cracking at superelevated curves

At all curves where D-cracking was seen to increase, a change in drainage was observed. A possible explanation for the change in rating may be attributed to the accumulation of moisture under the lower lane due to different drainage conditions. Normal drainage conditions (crown) have the downslope towards the outside of the pavement, that is the traffic lane is lower. On superelevated curves, to the driver's left, the drainage changes to the inside or the passing lane. It is curves of these types that showed the change in D-cracking rating. If traffic loading was the cause of the increased D-cracking seen in the traffic lanes on divided highways, it would be resonable to assume that this behaviour should continue through curves. A similar argument could be made for the differences in D-cracking rating between the straight and curved sections of highway 200. This was not the case. On divided highways the damage to the passing lane on curves was as severe as the traffic lane in the pre and post curve areas. On highway 200, the D-cracking was worse on the curve than in the straight sections. Although not entirely conclusive there appeared to be a correlation between drainage and the severity of D-cracking on pavements built with Birdshill Type 1 aggregate.

In summary pavements incorporating Birdshill Type 1 coarse aggregate exhibited extensive D-cracking. Drainage conditions rather than traffic loading appeared to be the most influential factor in the rate of deterioration. Pavements with crown drainage displayed improved durability while the downslope side of divided highways showed the highest D-cracking ratings. Figures 10 to 14 show typical sections of the pavements in this group.

Figure 10: Severe D-cracking on highway 100W 3RB

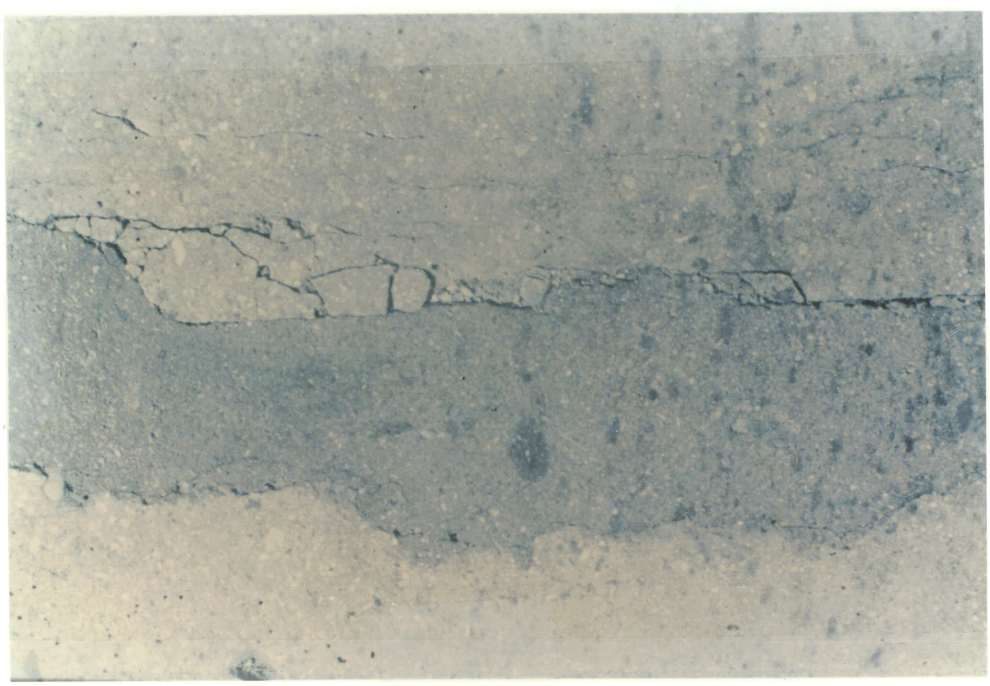

Figure 11: D-cracking on highway 100W

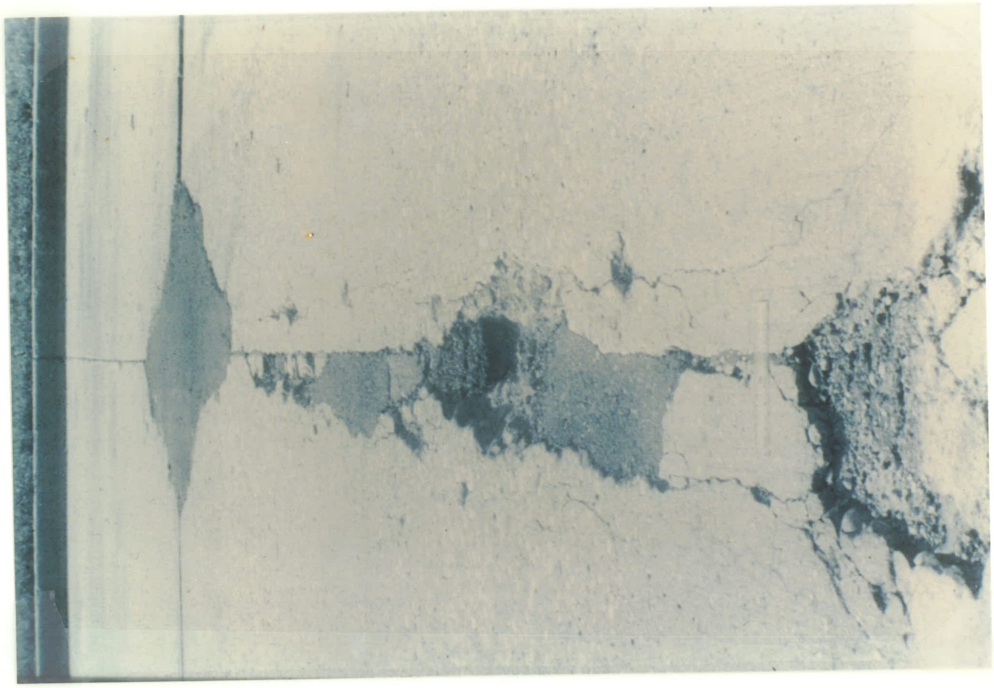

Figure 12: D-cracking on highway 100E

Figure 13: Deterioration at the Winnipeg Airport

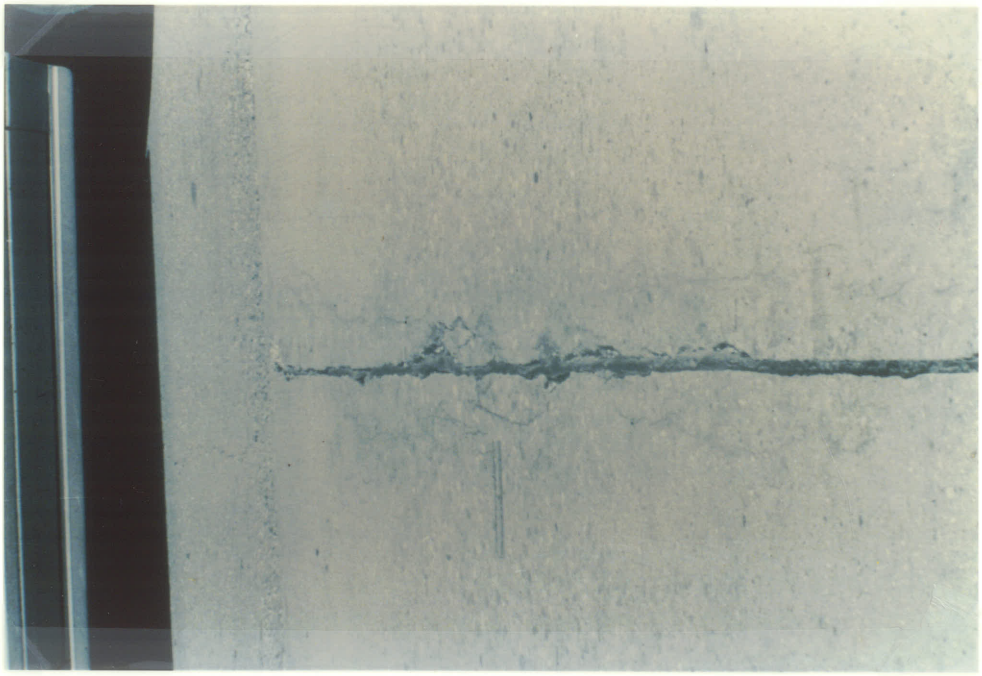

Figure 14: D-cracking on highway 200

3.3.2 Birdshill Type 2

Birdshill Type 2 aggregate had a top size of 20 mm. The highways surveyed containing this aggregate were 3CS, 15 and 101. The pavement ratings are plotted in Figure 15 and typical pavement conditions are shown in the photographs, Figures 16 through 18 . Minor D-cracking was present on highways 3CS and 101. These pavements exhibited staining and minor D-cracking related cracks along the transverse joint at the edge of the pavement. There was minimal damage to the pavement surface.

Highway 15 was free from D-cracking after 11 years of service. Figure 17 shows the typical deterioration seen on highway 15. Often this type of distress is mistaken for D-cracking. The deterioration was a joint spall displaying distinct angular sharp breaks. These types of spalls were common on Birdshill Type 2 pavements.

In summary pavements constructed with Birdshill Type 2 aggregate displayed a low incidence of D-cracking. Cracking and staining was generally restricted to the transverse joint at the free edge of pavement. A joint spalling deterioration was identified as being non D-cracking due to the dissimilarity in the two crack patterns.

3.3.3 Poplar Point - Stonewall

The Poplar Point Type 2 coarse aggregate was used for the paving of the Portage Bypass and a Stonewall Type 2 aggregate was incorporated into Highway 3 between Fort Whyte and Highway 100. No D-cracking was observed on these pavements. Several joint repairs were noted but the source of the distress was not apparent. The highway ratings were 0.0 and are plotted in Figure 15 .

3.3.4 Other Pavements

The only construction data available for two sections of highway 3, at Ft. Whyte and south of Oak Bluff, was the year of construction. The D-cracking ratings are listed in Table 3 .

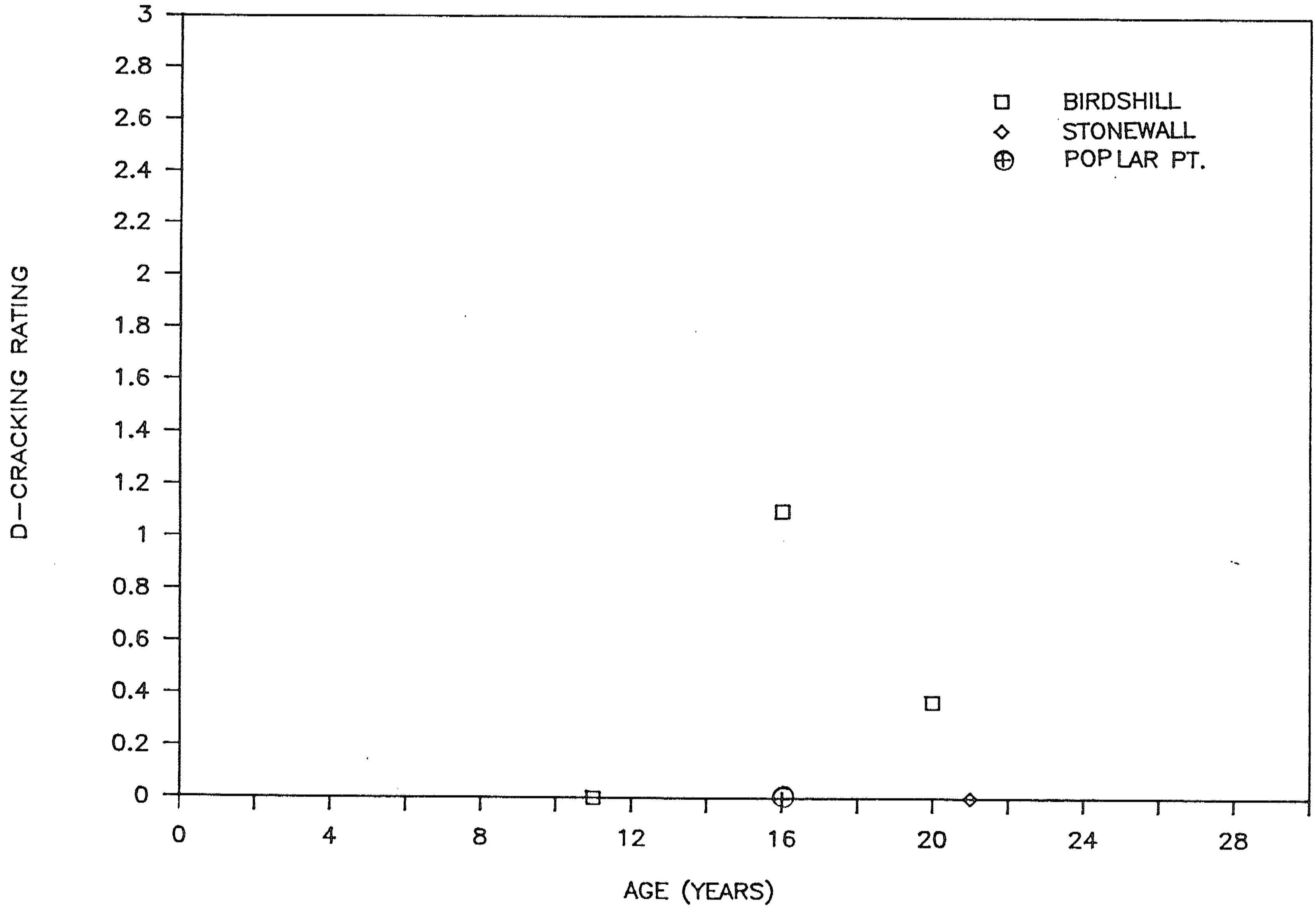

Figure 15: D-cracking rating of pavements with 20 mm aggregate

Figure 16: Pavement condition on highway 3

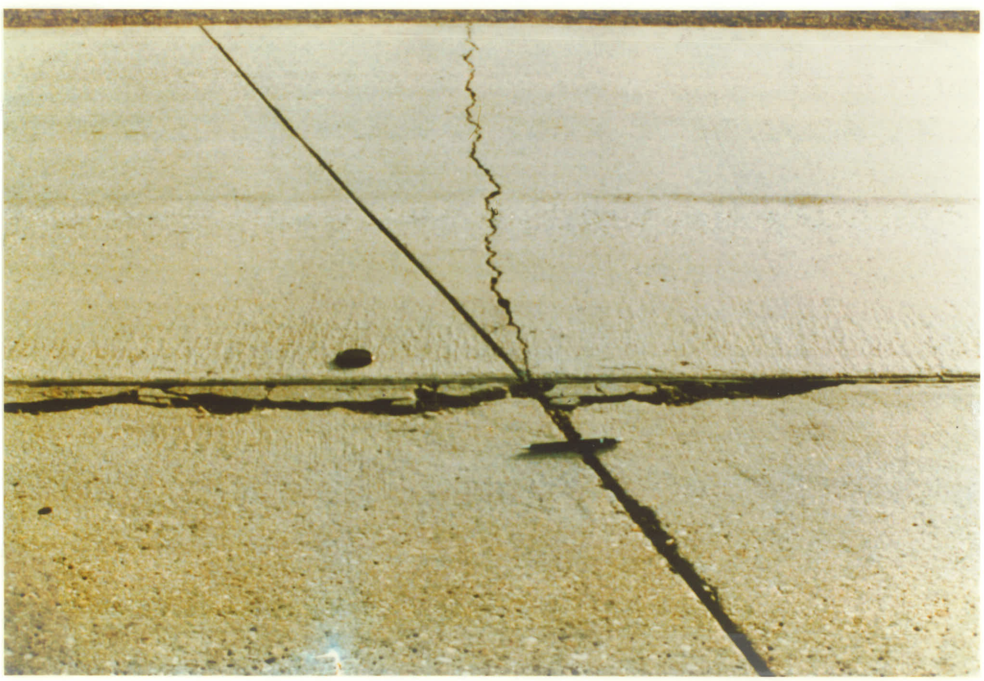

Figure 17: Highway 15 showing spalling deterioration

Figure 18: D-cracking on highway 101

3.3.5 Comparison of Birdshill Type 1 and 2 Coarse Aggregate.

The literature has identified the top size of a coarse aggregate as an important variable in determining an aggregates susceptibility to D-cracking. The pavement study allowed two top sizes of the Birdshill area to be examined. The pavement ratings are shown in Figure 19. Both groups of pavements showed increased D-cracking with increasing age. However the extent of the cracking varied significantly. The larger 40 mm top size pavements displayed very severe cracking on pavements 18 years old and moderate D-cracking on a 16 year old highway. The smaller 20 mm top size pavements exhibited only minor D-cracking on pavements 16 and 20 years old and no D-cracking on a 11 year old highway. Reduction in top size apparently decreased the rate of deterioration but did not eliminate it. The data does not indicate whether the improvement in performance though a reduction in top size of aggregate, will continue after D-cracking is started. It may be that the smaller top size has simply delayed the onset of cracking. Future periodic examination of the pavements with 20 mm top size aggregate would help determine if the improved performance is of a permanent nature.

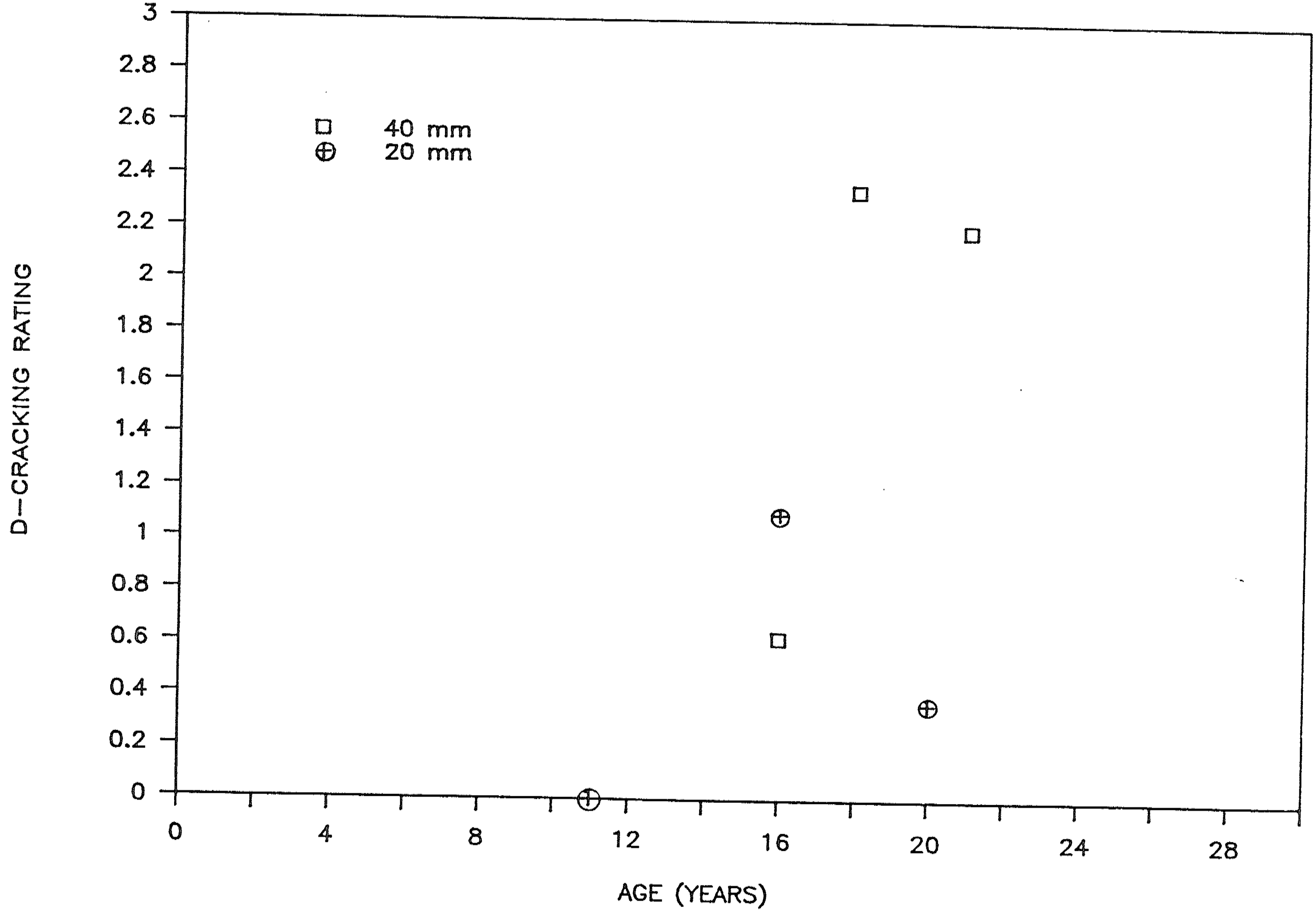

Figure 19: Comparison of Birdshill Type 1 and 2 coarse aggregate

Chapter IV

LABORATORY FREEZE-THAW TESTING PROGRAM

4.1 INTRODUCTION

Freeze-thaw deterioration of the coarse aggregate in Portland Cement concrete pavements is common in the Mid-Western United States and Manitoba. Routine tests have been unable to detect D-cracking susceptible aggregates. The absorption test on aggregate and the standard freeze-thaw test on concrete gave poor correlations between predicted quality and actual field performance (Verbeck et al 1972, Williams et al 1974). Research has shown that a modified freeze-thaw test can distinguish between D-cracking and non D-cracking susceptible aggregate (Traylor 1982, Paxton 1982). A modified freeze-thaw testing program was undertaken to test three local aggregates for D-cracking potential.

4.1.1 Objectives

The objective of the test was to investigate the D-cracking susceptibility of coarse aggregates from three different sources by subjecting concrete prisms to 300 freeze-thaw cycles using a modified ASTM C-666 B freeze-thaw test. Three variables were examined:

1. The relative durability of the aggregates
2. The effect of top size on the durability of aggregates from the same source.

3. A comparison of the durability of a dominantly carbonate aggregate with an igneous aggregate of the same top size.

4.2 METHODS

4.2.1 Aggregate

Five aggregates were supplied by two local contractors. Table 6 gives the supplier, location of the source pit, top size and dominant mineralogy of the coarse aggregate. The same fine aggregate was used in all concrete mixes and was supplied by Building Products from their Birdshill CN pit. Standard tests were preformed on the aggregates and are summarized in Table 7 .

Preparation of the aggregate for use in concrete consisted of:

1. Drying the aggregate for 24 hours at 110 degrees celsius.
2. Weighing out the required batch quantities.
3. Soaking the aggregate for 24 hours prior to batching.
4. Draining the water and drying to an approximate saturated surface dry condition. The aggregate was left slightly wet and was weighed before mixing in order to determine the amount of free water.

The aggregates were incorporated into the concrete as received by the university. Sieve analysis was done to determine the amount of material that passed the No. 4 sieve. The results of the analyses are listed in Table 8 . All testing on aggregate followed CSA standards.

TABLE 6

Aggregate Sources

Supplier	Designation	Source	Top Size	Mineralogy
Building Products	BP 2	Birdshill CN pit	40 mm	Carbonate
Building Products	BP 1	Birdshill CN pit	20 mm	Carbonate
Building Products	BP 3	Gull Lake	20 mm	Igneous
Supercrete	S 1	Ritcher	28 mm	Carbonate
Supercrete	S 2	Ritcher	14 mm	Carbonate

TABLE 7

Results of Standard Tests on Aggregates

Aggregate	Size (mm)	Dry Rodded un.wt. (kg/m3)	Bulk sp.gr.	Bulk (SSD) sp.gr.	Apparent sp.gr.	Absorption (%)	*Mineralogy (%)		
							C	Ig	O
BP 1	20	1710	2.621	2.674	2.767	2.02	75	23	2
BP 2	40	1635	2.643	2.683	2.752	1.50	62	31	7
BP 3	20	1640	2.614	2.642	2.689	1.70	28	67	5
S 1	28	1600	2.620	2.661	2.733	1.58	70	27	3
S 2	14	1640	2.566	2.627	2.743	2.40	59	35	6

* From pebble counts
C-carbonate
Ig-igneous
O-others including cherts,volcanics and metasediments

TABLE 8

Results of sieve analysis

Screen Size	Per Cent Passing				
	BP1	BP2	BP3	S1	S2
No.4	2.2	0	0	0	6.3

4.3 MIX DESIGN

The mix design was prepared using the following Transport Canada specifications; a water/cement ratio = 0.45, air entrainment = 5 ± 1 %, slump = 50 ± 10 mm and 28 day strength = 25 MPA . An initial design was calculated using the ACI absolute volume method. Two trial batches were mixed in order to determine the correct amounts of air entraining

TABLE 9

Mix Design (kg/m3)

Aggregate	Concrete	Water	Coarse Aggregate	Fine Aggregate	W/C	C/F
BP 1	304	137	1116	680	0.45	1.64
BP 2	275	124	1228	635	0.45	1.93
BP 3	282	127	1061	718	0.45	1.48
S 1	233	105	1208	575	0.45	2.10
S 2	298	134	973	758	0.45	1.28

agent and mix water. The final mix design is shown in Table 9 .

The concrete was batched over two days. The slump, density, air content and yield of the fresh concrete were measured and are listed in Table 10. Three 76 x 76 x 356 mm prisms and five 152 mm cylinders were cast from each mix. The prisms had studs placed in each end which were used for measuring the change in specimen length during the freeze-thaw test. Several additional prisms were made from batches containing aggregates BP 1 and BP 3. These prisms contained temperature monitoring equipment.

Several prisms had the studs, used for length measurements, eccentrically placed at the end of the prisms. This resulted in incorrect initial lengths being recorded for two prisms. These prisms were disqualified from the test and the Ritcher 14 mm and Gull Lake 20 mm concrete was tested using two, instead of the planned three, prisms. Molds were removed after 24 hours and the specimens were placed in a moisture room. The prisms were cured for 28 days. Cylinders were cured and broken at 7 and 28 days. The compressive strength test results are shown in Table 11. All procedures followed CSA standards.

TABLE 10

Concrete Test Results

Aggregate	Slump (mm)	Density (kg/m3)	Air Content (%)	Yield (m3)
BP 1	80	2350	5.9	0.057
BP 2	50	2390	4.7	0.057
BP 3	80	2300	6.3	0.057
S 1	40	2350	5.9	0.054
S 2	60	2390	4.0	0.054

TABLE 11

Compressive Strength of Concrete Cylinders

Aggregate	Sample Number	7 Day Strength (MPa)	28 Day Strength (MPa)	Average 28 Day Strength (MPa)
BP 1	1	27.9	34.0	
	2	30.1	35.3	34.8
	3		35.1	
BP 2	1	21.5	26.7	
	2	23.1	28.4	27.7
	3		28.6	
BP 3	1	21.5	31.3	
	2	23.0	30.5	30.1
	3		28.6	
S 1	1	22.1	27.4	
	2	19.7	25.9	26.2
	3		25.2	
S 2	1	41.0	43.1	
	2	35.5	44.1	43.8
	3		44.3	

4.3.1 Apparatus

The freeze-thaw tests were performed in an E&L automatic freezing and thawing cabinet. Although built to ASTM C-666 specifications, the unit was not suitable for the Method B of the standard procedure, which requires freezing in air and thawing in water, without modification. A pumping system had to be installed to fill and drain the cabinet. The cabinet was instrumented with thermocouples to monitor temperature gradients within the unit.

The prisms were measured using a comparator caliper accurate to 0.000025 mm. A calibrated standard bar was used to eliminate any error caused by temperature fluctuations of the caliper.

4.3.2 Procedure

The standard procedure for determining deterioration of test specimens during the ASTM C-666 test, consists of measuring the change in dynamic modulus of the concrete prism. This type of measurement is not sensitive to freeze-thaw deterioration of coarse aggregate (Williams et al 1974). The test was modified, by measuring the change in length of the specimens rather than the dynamic modulus (Willians et al 1974, Paxton 1982). All other specifications of ASTM C-666 B were followed.

The test conditions consisted of:

1. A freeze-thaw cycle ranging between -17.7 and 4.4 degrees celsius.
2. Six freeze-thaw cycles per day.

3. A test period of 300 cycles (50 days).

4. Speciman length measurements every 30 cycles.

The original length of each specimen was measured at the start of the test after the specimens had cooled to 4.4 degrees celsius. Length measurements were taken at the end of a thaw cycle with the prisms at a temperature of 4.4 ± 1 degrees celsius. The prisms remained immersed in water while waiting to be measured in order to maintain a constant temperature. After each measurement the specimens were rotated 90 degrees and their positions were shifted to minimize the effect of temperature gradients within the cabinet.

Elongation, E of each specimen was calculated by dividing the elongation measured during the test by the original length. Elongation was the only variable measured during the test.

4.4 RESULTS

The aggregates tested were divided into three coarse aggregate sources and four top sizes as shown in Table 12 . In accordance with ASTM C-666 Method B the concrete prisms were subjected to 300 freeze-thaw cycles and the elongation of each prism measured. The average elongation values are listed in Table 13 and plotted against the number of freeze thaw cycles in Figure 20

Correlations between freeze-thaw test behavior and field performance have been established in the literature and were discussed in Chapter 2. As no correlation between field performance and freeze-thaw testing results were available for local aggregates, the follow-

TABLE 12

Coarse aggregate groups

Source	Top Size mm	Elongation at 300 cycles %
Birdshill	40	0.594
Birdshill	20	0.026
Ritcher	28	0.026
Ritcher	14	0.026
Gull Lake	20	0.015

TABLE 13

Change in length of Test Prisms

Cycle Number	BP1	BP2	BP3	S1	S2
0	0	0	0	0	0
30	0.0129	0.0421	0.0086	0.0115	0.0118
57	0.0114	0.0326	0.0129	0.0111	0.0133
90	0.0210	0.0571	0.0144	0.0172	0.0193
127	0.0181	0.0837	0.0133	0.0167	0.0211
149	0.0193	0.0971	0.0108	0.0160	0.0211
184	0.0232	0.182	0.0165	0.0200	0.0247
213	0.0202	0.324	0.0094	0.0172	0.0204
253	0.0234	0.594	0.0111	0.0241	0.0254
280	0.0317	***	0.0147	0.0286	0.0267
300	0.0263		0.0147	0.0261	0.0263

*** All BP2 prisms were destroyed by 253 cycles.

ing relationships reported in the literature were used to analyize the test data:

a) Prisms undergoing elongations larger than 0.06 % were susceptible to D-cracking. In Illinois, 0.06 % was used as a design guideline in that concrete prisms exhibiting elongations greater than 0.06 % had the coarse aggregate incorporated in it disqualified from pavement concretes.

b) Concrete exhibiting elongations of less than 0.02 % were not associated with D-cracking (Paxton 1982). In the region of 0.02 % there could still be some overlap between durable and D-cracking susceptible aggregate, that is, both could show elongations in this range (0.02 %). In Ohio, further freeze-thaw testing to 450 cycles was generally undertaken when concrete exhibited elongations of 0.02 % by 300 cycles.

Using the above criteria the results of the testing program were as follows:

1. Birdshill 40 mm aggregate

Concrete prisms produced from Birdshill 40 mm coarse aggregate had an average elongation of 0.59 % which was 10 times larger than the Illinois limit of 0.06 % . All prisms in this group were destroyed by 253 cycles. Figure 21 shows severely cracked coarse aggregate particles, which were the source of distress within the concrete. This 40 mm aggregate would be classified as a D-cracking susceptible aggregate according to the Illinois failure criteria.

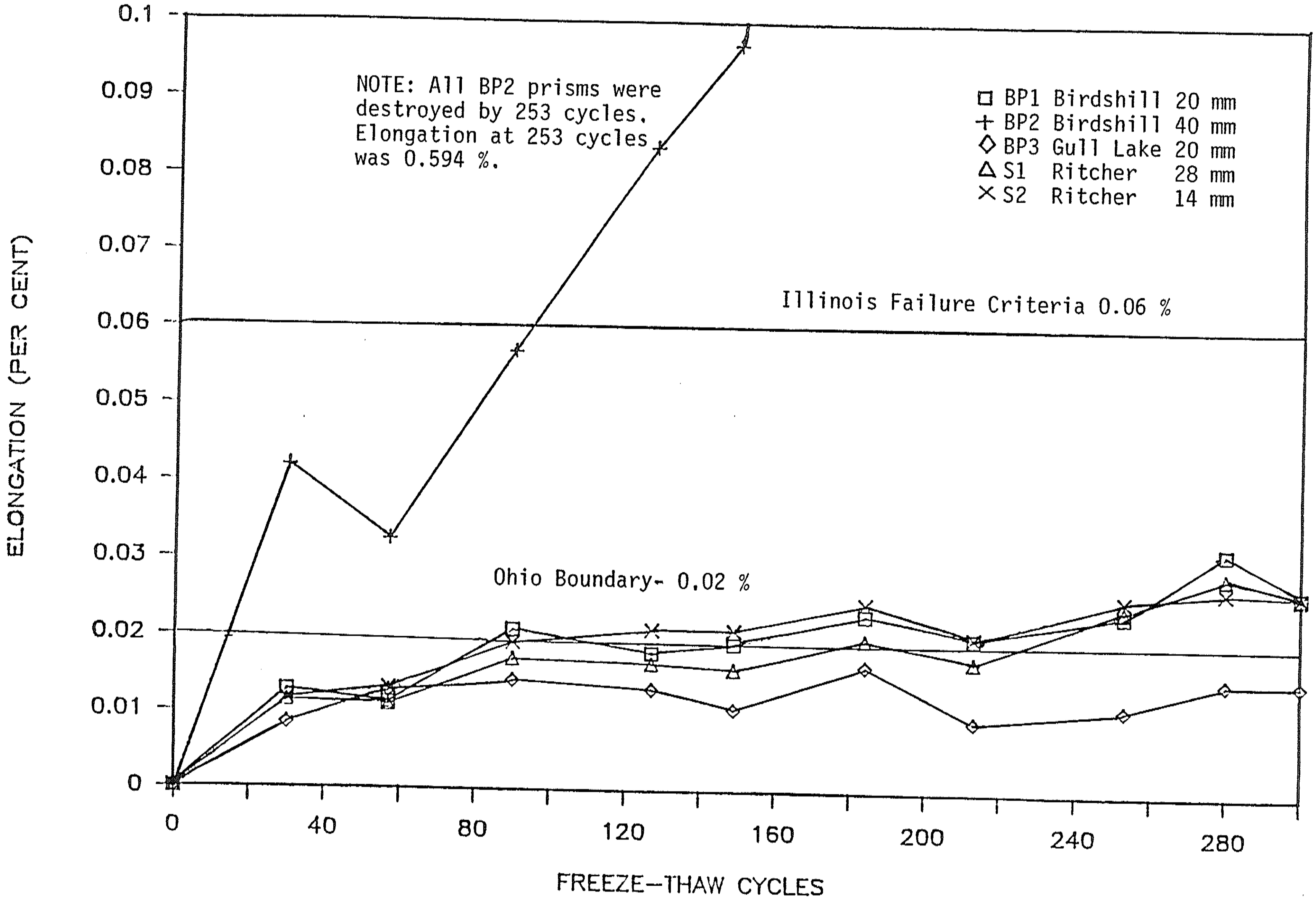

Figure 20: Elongation of test prisms

Figure 21: Deterioration of Birdshill 40 mm coarse aggregate

Concrete prisms produced from Birdshill 20 mm aggregate showed an average elongation of 0.026 % . The elongations were below the the Illinois failure criteria and in the range of the Ohio boundary of 0.02 % . The aggregate would be classed as non D-cracking using the Illinois failure criteria. Field studies reported in Chapter 3 concluded that pavements constructed with Birdshill 20 mm aggregate displayed minor D-cracking in 16 and 21 years of service, indicating that the aggregate is suscepti- ble to minor D-cracking. The freeze thaw test results were

21 years of service, indicating that the aggregate is susceptible to minor D-cracking. The freeze thaw test results were not conclusive as the elongations were lower than the Illinois failure criteria but still the aggregate was associated with minor D-cracking. The results fall into the 0.02 % overlap range reported in Ohio (Paxton 1982) which indicates that the aggregate may require further testing to 450 cycles in order to assess the material's durability.

The reduction in top size from 40 to 20 mm for the Birdshill aggregate resulted in a reduction in elongations from 0.59 to 0.02 % . This indicates that a significant improvement in durability was realized with a reduction in top size.

3. Ritcher 28 mm aggregate

Concrete produced with this coarse aggregate displayed elongations of 0.026 % which was under the Illinois criteria and in the range of the Ohio boundary. According to the Illinois failure criteria the aggregate would be classed as non D-cracking. As the results were in the range of Ohio boundary, where the elongation of non D-cracking and D-cracking concrete may overlap and no field performance records were available, no firm conclusions may be drawn regarding the susceptibility of this aggregate to deterioration.

4. Ritcher 14 mm aggregate

The average elongation for this set of concrete prisms was 0.026 % at 300 cycles and was identical to the Ritcher 28 mm prisms. The concrete produced with the 14 mm aggregate would also be classed as non-cracking using the Illinois criterion and borderline using the Ohio criterion.

Reduction in the top size of the Ritcher aggregate did not result in a change in elongation, indicating that a top size reduction would not improve the field performance of this aggregate source.

5. Gull Lake 20 mm aggregate

Concrete prisms made with the Gull Lake 20 mm aggregate had an average elongation of 0.015 % after 300 cycles. No significant elongation occurred after 60 cycles. The concrete would be classed as non D-cracking using both the Illinois and Ohio failure criteria. The apparent high durability of the dominently igneous Gull Lake aggregate is in agreement with findings by Stark (1976) which indicated that igneous aggregates were not associated with D-cracking.

4.5 SUMMARY

The testing program delineated one aggregate, Birdshill with a 40 mm top size, as clearly susceptible to D-cracking. and one aggregate, Gull Lake with a 20 mm top size, as nonsusceptible to D-cracking. The ASTM C-666 B test was unable to delineate the relative durability of the Birdshill 20 mm and Ritcher 28 and 14 mm top size coarse aggregates. Further testing to 450 cycles may further define the durability of these aggregates.

Top size reduction from 40 mm to 20 mm significantly improved the durability of the Birdshill aggregate. Top size reduction of the Ritcher aggregate from 28 to 14 mm did not improve the durability of the aggregate.

Chapter V

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

5.1 CONCLUSIONS

The following conclusions were drawn from the results of the study:

1. D-cracking was found to be present on Manitoba highways and the Winnipeg International Airport. The severity of the deterioration ranged from severe on 30 % of the highways, to no D-cracking on 34 % of the provincial highways surveyed.
2. Deterioration of pavements was found to vary with different coarse aggregate.
   a) Pavements constructed with Birdshill aggregate, having a top size of 40 mm, exhibited moderate to extensive D-cracking on all surfaces surveyed.
   b) Pavements constructed using a 20 mm top size Birdshill exhibited minor D-cracking.
   c) Pavements constructed with Poplar Point and Stonewall 20 mm top size aggregates showed no D-cracking.
3. Poor drainage conditions showed an increase in the rate of deterioration, whereas traffic loading did not.
4. Concrete made with Birdshill 40 mm top size coarse aggregate displayed elongations consistent with D-cracking susceptible aggregate when tested using a modified ASTM C-666 method B freeze thaw test.

5. In the same test , concrete made with Birdshill 20 mm and Ritcher 28 and 14 mm top size coarse aggregates exhibited smaller elongations relative to the Birdshill 40 mm aggregate indicating a reduced susceptibility to D-cracking. The degree of D-cracking, if any, that would be present in a field pavement, could not be inferred from the results of the ASTM C-666 B freeze-thaw test performed to 300 freeze-thaw cycles.
6. Concrete constructed with a Gull Lake 20 mm top size coarse aggregate exhibited small elongations and no visual damage, when tested using a modified ASTM C-666 method B freeze-thaw test, indicating a durable aggregate.
7. Top size reduction of two coarse aggregate sources incorporated into concrete and tested using a modified ASTM C-666 method B freeze-thaw test showed:
   a) Significant improvement in performance of the Birdshill coarse aggregate when the top size was reduced from 40 mm to 20 mm.
   b) No improvement in freeze-thaw performance of the Ritcher aggregate when the top size was reduced from 28 mm to 14 mm.

5.2 RECOMMENDATIONS FOR FURTHER STUDY

The results of the study indicate that there are still several important issues remaining to be studied in regards to D-cracking:

1. A better understanding of the destructive mechanism in D-cracking is required. Porosity studies reported in the literature could reveal the primary mechanism of deterioation as these

procedures deal with fundamental properties of the aggregate. Studies of this type are necessary in order to understand the factors governing an aggregate's resistance to freezing and thawing.

2. Detailed construction records should be obtained for all pavements in Manitoba. This will provide a larger data base for interpretation of testing results and would also provide deterioration time relationships on which to base economic decisions on aggregate performance.
3. Periodic surveys of Highways 3, 15 and 101, are required to determine the amount of improvement realized from aggregate top size reduction.
4. Further research into the effect of surface and subsurface drainage on the rate of D-cracking should be conducted. While drainage has not been correlated as a cause or a prevention of D-cracking, poor drainage can accelerate the problem. This issue is especially critical in the Red River Valley since few other areas reporting D-cracking had the same conditions for drainage, that is subgrade soils of low permeability, and poor natural surface drainage.
5. Any new or suspected aggregates should be tested for D-cracking susceptibility using the ASTM C-666 method B freezing and thawing test, in which permanent dilation of the concrete prism is measured. Although the testing program takes three months to complete the method has been shown to identify deleterious aggregate.

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Appendix A

D-CRACKING SURVEY DATA

Note, the survey was initially done using imperial units and later converted to meters. This is the reason for the uneven station numbers.

ALL STATIONS AND REFERENCE NUMBERS ARE IN METERS

Location: Highway 100 from McGillivary to Pembina
Date Built: 1965
Aggregate: Birdshill Type I

Station	Rating	Station	Rating	Station	Rating
0+00	3	35+05	3	68+58	3
1+52	3	36+58	3	70+10	3
3+05	3	38+10	3	71+63	2
4+57	3	39+62	3	73+15	2.5
6+10	3	41+15	3	74+68	2.5
7+62	3	42+67	3	76+20	2
9+14	3	44+20	3		NL. SL.
10+69	3	45+72	2.5	77+72	2.5 1.5
12+19	3	47+24	2	79+25	2.5 1.5
13+72	3	48+77	2.5	80+77	2 1
15+24	3	50+29	3	82+30	2 2
16+76	3	51+82	3	83+82	2 2
18+29	3	53+34	2.5	85+34	2 2
18+29	3	54+86	3	86+87	2 2
21+34	3	55+63	asph	88+39	2 2
22+86	3	56+39	3	89+92	2
24+39	3	57+91	n.r.	91+44	2
25+91	3	59+44	3	92+96	2
27+43	3	60+96	3	94+49	2
28+96	3	62+48	2.5	96+01	1.5
30+48	3	64+01	3	97+54	2.1
32+00	3	65+53	2.5	99+06	1.5
33+53	3	67+06	3	100+58	1.5
				102+11	3

Comments

Reference station 0+00: 152 meters south of the intersection of highway 3 and highway 100.

0+00-0+76: Severe D-cracking of pavement. North lane is in the worst condition. Concrete shows D-cracking patterns. Difficult to see fine cracks as the pavement is very wet. Cracking is at the edge of pavement and center line intersection of joints.
30+48: D-cracking is seen along most longitudinal joints.
4+57: Approximently 200 feet of pavement has totally failed due to D-cracking along the joints. No concrete remains at the joint.
28+96: Concrete is in better condition. Some joints rate 2 but on average the section is 3
27+43- 30+48 Several joints show collapse at the center line.
45+72: D-cracking shows classic patterns at the joints but there is less damage than 0+00 to 27+43, where the pattern has been destroyed along with the pavement.
50+29 Some very large holes in the center intersection of joints. Many sections also show D-cracking on the lateral edges of concrete.
62+48: The damage to pavement is less than previous sections but is still apparent.
65+53: Rating of 2 along most joints in this section.
76+20: Occasional joints display rating of 3 on south lane. North lane remains approx. the same as 76+20.
82+30: West side of joints is more badly damaged than the east edge. West edge is leading edge for traffic loading. The increase in damage reflects the loading conditions.
85+34- 94+49 Pavement is in relatively good condition. D-cracking is isolated to the center intersection of the joints and free edge of pavement.
100+58: Badly D-cracked section between 99+06 and 100+58 - 30 meters in length.
10+211 Last 150 meters is badly D-cracked along both the longitudinal and transverse joints.

Location: Highway 100 Eastbound St. Mary to St. Annes
Date Built: 1965
Aggregate: Birdshill Type I

Station	Rating	Station	Rating	Station	Rating
	SL.NL.		SL.NL.		SL.NL.
1+52	3 2	15+24	3 1	24+39	3 1.5
3+05	2 2	16+76	3 1	25+91	3 1
4+57	2 1.5	17+53	3 1	27+43	3 1
6+10	2 2	17+68	3 3	28+96	3 1
7+62	2 1	18+29	3 1	30+48	2.5 1
9+14	2 1	19+81	2 1	32+00	3 1
10+69	3 1.5	21+34	3 1	33+53	3 1
12+19	2 1	22+86	3 1	35+05	3 1
13+72	2.5 1				

Location: Highway 100 West, Highway 3 to Wilkes Overpass
Date Built: 1959
Aggregate: Unknown

Station	Rating	Station	Rating	Station	Rating
	WL.EL.		WL.EL.		WL.EL.
0+00	asph	21+34	1.5 3	44+20	2 2
1+52	3 2	22+86	nr.	45+72	1.5 2
3+05	1.5 3	24+39	2 3	47+24	2 2
4+57	1.5 3	25+91	1 3	48+77	2 2
6+10	2 3	27+43	1 2	50+29	2.5 1
7+62	2 3	28+96	1 2.5	51+82	2 2
9+14	1 3	30+48	2 3	53+34	2 2
10+69	1 3	32+00	2 3	54+86	2 2.5
12+19	2 3	33+53	2 3	56+39	1.5 2
13+72	2.5 3	35+05	1 2.5	57+91	1.5 3
15+24	1 3	36+58	1.5 2	59+44	1.5 3
16+76	1 3	38+10	1.5 2	60+96	1.5 3
18+29	2 3	39+62	2 2	62+48	2 3
19+81	2 3	41+15	1.5 2	64+01	2 3
		42+67	2 2.5		

Comments

Reference station 0+00: Intersection of highway 3 and highway 100.
1+52: East lane (traffic lane) shows severe D-cracking that has required patching. West lane is not as severely D-cracked . The D-cracking occurs mainly along the transverse joints.
7+62 D-racking is severe along the longitudinal joint as well as the transverse joints.
16+76-19+81: Many longitudinal joints show D-cracking.
25+91: Deterioration not as severe as previous 2440 meters.
39+62 Both passing and traffic lanes have approximently the same D-cracking.
36+58- 64+01 Longitudinal joints show little signs of D-cracking.

Location: Highway 100 West: North of Wilkes Overpass to Roblin Blvd.
Date Built: 1959
Aggregate: Unknown

Station	Rating	Station	Rating	Station	Rating
	WL.EL.		WL.EL.		WL.EL.
0+00	2 3	6+10	2 2.5	12+19	2 3
1+52	2 2	7+62	1 3	13+72	3 2
3+05	1.5 3	9+14	1 2	15+24	1 3
4+57	2 2.5	10+69	2 3	16+76	1 3

Comments

Reference station 0+00: North end of overpass approach.
13+72: West lane (passing lane) is in worst condition than the east lane (traffic lane). Left hand curve changes drainage to the west.

Location: Highway 100 West-St. Annes to St. Marys.
Date Built: 1959
Aggregate: Unknown

Station	Rating	Station	Rating	Station	Rating
	NL.SL.		NL.SL.		NL.SL.
0+00	2.5 2	12+19	3 1	24+39	2 2
1+52	2.5 2	13+72	3 2	25+91	2 2
3+05	2 2	15+24	2 2	27+43	2 2
4+57	3 2	16+76	2.5 2	28+96	2 1.5
6+10	3 1	18+29	2 2	30+48	2 2
7+62	2 2	19+81	1.5 1.5	32+00	2 2
9+14	3 1.5	21+34	2.5 2	33+53	2 1.5
10+69	3 2	22+86	2 2.5	35+05	3 2

Comments

Reference station 0+00: Intersection of Highway 100 and St Annes.
1+52: Many joints have been replaced on the traffic lane. As joint replacement has removed deteriorated concrete it is difficult to establish if D-cracking was responsible. Note; the repaired joints were not included in the rating.
6+10-12+19: The road is in poor condition even though many joints, up to 30 % per 150 meters, rated 2.
16+76 Very well developed D-cracking patterns but serviceability has not been affected.

Location: Highway 15 Highway 101 to 2.3 miles west of Anola.
Date Built: 1972
Aggregate: Birdshill Type 2

Station	Rating	Station	Rating	Station	Rating
*From 0+00 to Highway 101 the pavement rated zero. Minor occurrences of D-cracking were noted see Comments.					

Comments Carson Creek to Highway 101

Reference station 0+00: Carson creek
6+10 Pavement is in good shape, minor spalling along some transverse joints. Occasional uncontrolled transverse cracks.
Spalling
0+00-18+29:Concrete is in good shape, some scaling and popouts.
30+48: Some scaling and transverse cracks but in good shape.
73+15: Two joints show minor D-cracking at the intersection of longitudinal and transverse joints.
77+72 Spalling appears at almost every joint.
88+39 Approx. 91 meters of pavement shows spalling along longitudinal joints and at the intersection of the transverse and longitudinal joint.
99+06 Uncontrolled transverse tension cracks run 90 degrees from the edge of pavement. The sawn joints have been cut at an angle to the length of the pavements. Some staining similar to those associated with D-cracking appear in SMALL tension cracks.
112+78 Transverse cracks and some and scaling in tire ruts.
118+87 Some spalling.

Comments Cooks Creek to Carson Creek

Reference station 0+00:Cooks Creek
0+00- 4+57: Spalling, approx. 10 spalls per 150 meters.
7+62: Two joints were D-cracked with a rating of 1. The D-cracking appeared to have started after a spall.
21+34:0+00- 4+57 In general the pavement is in good shape.

Comments Anola to Cooks Creek

Reference station 0+00: 3.78 kilometers west of Anola at the contact between the asphalt and concrete.
1+52: Typical highway 15 deterioration. Spalling, a non D-cracking type of deterioration.
4+57: Some very minor D-cracking at the edge of pavement. Only seen between 0+00 and 4+57.
10+69: Spalling at approx. 1-5 joint intersections per 150 meters.
18+29: Spalling, appears to be the same type as elsewhere on highway. Usually within the spall their are 3 to 4 small tension cracks to 7 to 20 cm apart and 90 degrees to the joint.
22+86 Large popout.

Location: Highway 3: South from Oak Bluff
Date Built: 1962
Aggregate: unknown

Station	Rating	Station	Rating	Station	Rating
0+00	asph				
1+52	asph			70+10	1.5
3+05	asph			71+63	1
4+57	asph	36+58	1	73+15	1
6+10	asph	38+10	1.5	74+68	1
7+62	asph	39+62	1	76+20	0.5
9+14	asph	41+15	1	77+72	1
10+69	1	42+67	1	79+25	1
12+19	1	44+20	1	80+77	0.5
13+72	1.5	45+72	1	82+30	1
15+24	1.5	47+24	1	83+82	1
16+76	1	48+77	1	83+82	1
18+29	1.5	50+29	1	85+34	1
19+81	1.5	51+82	1	86+87	1
21+34	1	53+34	1	88+39	1
22+86	1	54+86	1	89+92	1
24+39	1.5	56+39	1	91+44	1
25+91	1	57+91	1	92+96	1
27+43	1	59+44	1	94+49	1
28+96	1	60+96	1	96+01	1
30+48	1	62+48	1	97+54	0.5
32+00	1	64+01	1	99+06	0.5
33+53	1	65+53	1	100+58	0.5
		67+06	1	102+11	0.5
35+05	1	68+58	1.5	103+63	0.5

Comments

Reference station 0+00: Intersection of highway 100 and 3
0+00- 9+14: Asphalt cover
37+50: D-cracking observed at the edge of pavement. No cracking seen at the center line intersection of the longitudinal and transverse joints.
38+00: D-cracking at several joints, worst at edge.
13+72: Several joints display well developed D-cracking, rating of 2. Very fine cracking, generally joints show staining. Northbound is in better condition than southbound.
21+34 D-cracking lines are better developed at the edge of pavement than in the center. This applies to all pavement surveyed to date along highway 3.
65+53: Well developed D-cracking on several joints, cracking runs through exposed aggregate.
68+58- 76+20: Minor D-cracking, many joints have rating of zero.
82+30- 85+34: Some joints rate 2 and several have been replaced possibly indicating a rating of 3.
88+39: Cracking is most prevalent at the edge of pavement.
97+54: Joints show D-cracking, rating 2 and 3 occasionally, but the majority rate 1 and zero. Joints with no D-cracking can easily be iden-

tified by the lack of staining. In general if there is staining there is minor cracking present.

Location: Highway 3: Route 90 to Highway 100
Date Built: Station 53+00 - 250+50 1962
0+00 to 53+00 unknown
Aggregate: 53+00 to 250+50 Stonewall Type 2
0+00 to 53+00 unknown

Station	Rating	Station	Rating	Station	Rating
0+00	3	28+35	0	54+25	0
0+91	1	29+87	0	55+78	0
2+43	0.5	31+39	0.5	57+30	0
3+96	0.5	32+92	0	58+83	0
5+49	0.5	34+44	0	60+35	0
7+01	1	35+97	0	61+87	0
8+53	2.5	37+49	0	63+40	0
10+06	3	39+01	0	64+92	0
11+58	2.5	40+54	0	66+45	0
13+11	3	42+06	0.5	67+97	0
14+63	3	43+59	0	69+49	0
16+15	0	45+11	0	71+02	0
17+16	0	46+63	0	72+54	0
19+20	0	48+16	0	74+07	0
20+73	0	49+68	0	75+59	0
22+25	0	51+21	0	76+35	asph
23+77	0	52+73	0		
25+30	0				

Comments

Reference station 0+00: Intersection of Route 90 and 3 (McGillivary)
0+45: Some joints show well developed D-cracking while others show no deterioration. These joints are in close proximity. There is no apparent physical reason why there has been selective deterioration.
3+96-5+49: D-cracking on transverse joints.
7+01: D-cracking well developed in some joint sections.
8+53: Several joints exhibit a rating of 3.
14+94: The concrete is in much better condition and appears to be younger.
14+78-16+15: No evidence of concrete deterioration. Slight scaling is evident om east bound lane.
22+25: Occasional intersection between longitudinal and transverse joints show very minor cracking.
28+35: Deterioration of joint, does not appear to be related to D-cracking
42+82: Isolated joints show early signs of D-cracking.
60+81: A 30 meter section shows definite D-cracking.
76+20: End of concrete start of asphalt.

Location: Highway 1 West-Portage Bypass
Date Built: 1968
Aggregate: Poplar Point

Station	Rating	Station	Rating	Station	Rating
From 0+00 to 28+96 the pavement rated zero. For further details see Comments.					

Location: Highway 1 East-Portage Bypass.
Date Built: 1968
Aggregate: Poplar Point

Station	Rating	Station	Rating	Station	Rating
From 0+00 to 30+48 the pavement rated zero. For further details see Comments.					

Comments Portage Bypass West

Reference station 0+00: 2.4 km west of end of asphalt on 1W.
0+00-2+29: Concrete in good condition. Traffic lane shows more wear.
9+14: Joints in passing lane display some deterioration, not D-cracking.
10+69: Similar deterioration to 9+14. A type of joint spall accompanied with a loss surface mortar.
19+05: Joint deterioration similar to spalling seen on highway 15.
29+72: end of section. (traffic lights)

Comments Portage Bypass East

Reference station 0+00: Traffic lights on Bypass.
4+69: Transverse joint shows deterioration similar to a joint spall but with a triangular shape.
5+33- 6+10: Joint repairs but no evidence of D-cracking.
16+00: Pavement shows serious deterioration due to scaling caused by segregation of top lift.
18+29: Several uncontrolled transverse cracks.
25+91-27+43: Joint repairs increasing, approximently one repair per 30 meters.
0+00-end: Concrete is in good condition. Traffic lane shows more wear.

Location: Highway 3 Carman to Sperling
Date Built: 1964
Aggregate: Birdshill type 2

Station	Rating	Station	Rating	Station	Rating
0+00	0	39+62	1	79+25	0
1+52	0	41+15	1	80+77	0
3+05	0	42+67	1	82+30	0
4+57	0	44+20	1	83+82	0
6+10	0	45+72	1	85+34	0
7+62	0	47+24	1	86+87	0
9+14	0	48+77	1	88+39	0
10+69	0	50+29	1	89+92	0
12+19	0	51+82	0.5	91+44	0
13+72	0	53+34	0.5	92+96	0.5
15+24	0	54+86	0.5	94+49	0
16+76	0	56+39	0	96+01	0
18+29	0	57+91	0	97+54	0
19+81	0	59+44	0.5	99+06	1
21+34	0	60+96	1	100+58	1
22+86	0	62+48	1	10+211	0
24+38	0	64+01	1	103+63	0
81+52	0	65+53	1	105+16	0
27+43	0.5	67+06	0.5	106+68	1
91+52	1	68+58	0.5	108+20	1
30+48	0.5	70+10	0.5	109+73	1
32+00	1	71+63	0.5	111+25	0.5
33+53	1	73+15	0	112+78	0.5
35+05	1	74+68	0	114+30	0
36+58	1	76+20	0	115+82	0
38+10	0.5	77+72	0	117+35	0

Comments

Reference station 0+00: Carman town limits.
0+00-15+24: No D-cracking evident, some minor staining. Pavement in good condition.
81+52: One joint in this interval rated 2. The remaining joints rated zero.
27+43: One joint in ten shows fine cracks.
33+53: Intersection of edge of pavement and transverse joint show staining and minor cracking.
39+62: One zero rating per 10 joints.
42+67: Staining prominent but few visible cracks.
45+72- 48+77: Little cracking but staining visible.
59+44 D-cracking line at free edges of pavement and joint.
65+53 Double spaced joints. Uncontrolled transverse cracks between cut joints. The uncontrolled joint shows prominent staining.
74+68- 83+82: Occassional staining no cracking.
89+92- 92+96: Staining minor no cracking.
106+68-108+20: Staining with very little cracking.
109+73: Staining, one joint in ten shows cracking.
112+78: Four joints in ten rate zero.

Location: Highway 200: North of St. Adolphe
Date Built: 1968
Aggregate: Birdshill Type 1

Station	Rating	Station	Rating	Station	Rating
0+00	0	16+76	0	33+53	1
1+52	0	18+29	1	35+05	1.5
3+05	0	19+81	1	36+58	1
4+57	0	21+34	1	38+10	0
6+10	0.5	22+86	1	39+62	1
7+62	0.5	24+38	1	41+15	0.5
9+14	0	81+52	2	42+67	0
10+69	0	27+43	1	44+20	0
12+19	0	91+52	1	45+72	1
13+72	0	30+48	1		
15+24	1	32+00	1		

Comments

Reference station 0+00: Intersection of highway 200 and PR 429.
0+00-2+29: No deterioration, staining or cracking. The pavement is in excellent condition.
5+33: Joint spalling of a type similar to Hwy 15. No evidence of D-cracking at these joints.
5+33-6+86 Free edges of pavement show staining and very fine cracks. Cracks are filled with a grayish oblique material.
10+69: Curve east. East lane has slightly more D-cracking than previous pavements. Staining is also more pronounced.
9+75(start)-12+95(end of curve): Joints show detectable staining
15+24: Well developed D-cracking along an uncontrolled longitudinal joints.
18+29: Curve west, D-cracking increasing.
20+57: Bridge and asphalt overlay.
21+95: Cracking common, staining more pronounced.
25+90: Curve west-superelevated.
26+21: Well developed D-cracking.
35+05: Curve West-superelevated.
39+62: Curve West-superelevated.
46+33: Curve West-superelevated.

Location: Highway 101 from Highway E to 15
Date Built: 1967
Aggregate: Birdshill Type II

Station	Rating	Station	Rating	Station	Rating
0+00	1	12+19	1.5	24+38	1
1+52	1	13+72	1.5	81+52	1
3+05	2	15+24	1	27+43	1
4+57	2	16+76	1	91+52	2
6+10	1	18+29	1	30+48	1
7+62	0	19+81	0	32+00	1
9+14	1	21+34	1	33+53	1
10+69	1	22+86	1	35+05	1

Comments

0+00-3+05: Joints show well developed D-cracking. Staining evident on all transverse joints. Longitudinal joints show little or no D-cracking.
3+05- 4+57: West lane shows more D-cracking than east lane.
8+38: Asphalt lift.
9+75: East lane shows more D-cracking.
13+72: Spalling with well developed D-cracking.
21+34-22+86: Spalling at the intersection of transverse and longitudinal joints. Adjacent concrete shows D-cracking. The spall does not display the D-cracking pattern.
81+52: Uncontrolled transverse joint has a rating of 3.
35+05: Asphalt.

Location: Winnipeg International Airport
Date Built: 1963
Aggregate: Birdshill Type 1

West Joint #	Rating 0	1	2	3	R	West Joint #	Rating 0	1	2	3	R
1	0	17	3	0	0	24	3	4	0	0	27
2	0	16	4	0	0	25	0	7	3	0	0
3	0	17	3	0	0	26	0	7	3	0	62
4	1	7	3	0	0	27	0	7	12	0	53
5	0	2	15	0	0	28	0	6	13	0	53
6	0	8	8	0	0	29	0	6	13	0	53
7	0	4	8	0	4	30	1	6	12	0	53
8	0	0	9	0	34	31	1	6	12	0	53
9	0	2	5	0	34	32	1	5	13	0	53
10	0	2	5	0	36	33	0	6	13	0	53
11	0	5	2	0	36	34	0	7	12	0	53
12	0	4	3	0	36	35	1	9	17	2	43
13	0	4	3	0	36	36	4	9	15	1	43
14	0	4	3	0	38	37	0	21	43	0	0
15	1	2	4	0	38	38	0	22	42	0	0
16	0	3	4	0	38	39	0	31	33	0	0
17	1	4	2	0	38	40	0	33	31	0	0
18	2	3	2	0	27	41	2	47	15	0	0
19	0	3	3	0	27	42	1	50	14	0	0
20	2	4	1	0	27	43	3	72	2	0	0
21	0	5	2	0	27	44	5	71	1	0	0
22	0	5	2	0	27	45	5	49	5	0	0
23	1	5	1	0	27						

TOTALS	RATING 0	1	2	3
	35	605	428	3

Comments

Total data points 3734.
Data Points under terminal building = 767
Data points not rated = 767
Joint intersections 3734 - 767 - 767 = 2200
Joints inferred to be rated 3 due to repairs = 1129
Remaining joints were rated visually 2200 - 1129 = 1071
Statistical breakdown 3734 - 767 = 2967 Total Number of Joints Exposed
Joints not rated (767/2967) x100%= 25.9%
Joints repaired (1129/2967) x100%= 38.1%
Rating of: 3 (3/2967) x100%= 0.1%
2 (428/2967) x100%= 14.4%
1 (605/2967) x100%= 20.4%
0 (35/2967) x100%= 1.1%
100.0%

D-cracking survey of the City of Winnipeg

Several pavement were rated in the City of Winnipeg as part of a statistical survey of the city streets. Difficulties in obtaining construction records of aggregate top size and source and pavement age resulted in the cancellation of the project. The ratings of the three streets surveyed are listed in the following section.

Location: McGillivary Blvd.; Waverly to Kenaston Blvd
Date Built: ?
Aggregate: Unknown

Station	Rating	Station	Rating	Station	Rating
0+00	3	6+10	2	12+19	2
1+52	2	7+62	2	13+11	3
3+05	2	9+14	2	15+24	2
4+57	2	10+69	2		

Comments

Reference station 0+00: Start of single lane west of Waverly.
1+00: D-cracking is severe. West bound lane shows significantly more D-cracking than east bound lane.
2+10; Severe deterioration at several joints. Joints before and after show much less (Rating of 3 vs. 2).

Location: Kenaston Blvd: McGillivary to Wilks Ave.
Date Built: unknown
Aggregate: unknown

Station	Rating	Station	Rating	Station	Rating
0+00	3	13+72	2		
1+52	1.5	15+24	2	24+39	2
3+05	3	16+76	2	25+91	1
4+57	1	17+53	3	27+43	1
6+10	1	17+68	3	28+96	1
7+62	2	18+29	3	30+08	2
9+14	1	19+81	3		
10+69	1	21+34	Asph.		
12+19	1	22+86	3		

Comments

Reference Station 0+00: Intersection of Kenaston and McGillivary.
9+14: South bound lane shows much less D-cracking than north bound lane.
17+60: Transverse joints rate 3, longitudinal joints rate 1.

Location: Grant Ave: Kenaston to Kelvin
Date Built: unknown
Aggregate: unknown

Station	Rating	Station	Rating	Station	Rating
0+00	0	4+57	1	9+14	2
1+52	0	6+10	1.5	10+69	2
3+05	1	7+62	1.5	12+19	1

Comments

Reference Station 0+00: Intersection of Kenaston and Grant
0+27: Wearing surface showing the coarse aggregate component to be primarily igneous.
5+30: D-cracking present. The coarse aggregate visible on the wearing surface has significantly more carbonates than the previous pavement.
7+50: Well developed D-cracking present.

Appendix B

LABORATORY TESTING

Mix Design

Example: S-2 14 mm aggregate

Transport Canada Specifications:

W/C Ratio	0.45
Air Entrainment	5+1%
28 Day Strength	25 MPa

Materials

Cement:Type 10, Normal Portland.

Admixture: Darex Air Entraining Agent

Aggregate:

Fine-Building Products Birdshill

Gradation meets CSA A23.1	
Fineness Modulus	2.6
Specific Gravity	2.71
Absorption	--

Coarse-Supercrete 14mm Birdshill

Material passing No.4 sieve	1.75
Specific Gravity	2.627
Dry Rodded Weight	1640 kg/m
Absorption	2.40 %

Trial Mix Design

Absolute Volume Method

Portland Cement Association Design Handbook

Concrete Mindess and Young

1. Estimate water; 14 mm coarse aggregate-air entrained concrete. From table 7-6 (PCA handbook) water = 175 kg/m3

2. Cement W/C 0.45 Cement = 175/0.45 = 388.8 = 389 kg/m3

3. Coarse Aggregate: Volume of coarse aggregate per unit of volume of concrete from table 7-7 PCA handbook. For a fine aggregate with a F.M. = 2.6 the volume = 0.58.

4. Total dry mass of coarse aggregate is equal to the volume times the aggregates dry rodded weight. from table the volume = 0.58

0.58 X 1640 = 951 kg/m3

5. Total Saturated Surface Dry (SSD) mass of coarse aggregate.

951 + 951(2.4) = 973 kg/m3

6. Calculate absolute volume of known materials.

Cement	389 kg/(3.15 X 1000 kg/m3)	= 0.123 m3
Water	175 kg/1000 kg/m3)	= 0.175 m3
Air	5% of m3	= 0.050 m3
Coarse agg	973 kg/(2.62 X 1000 kg/m3)	= 0.371 m3
Total		0.719 m3

7. Absolute volume of sand required for mix.

1.00 - 0.719 = 0.281 m3

8. SSD mass of sand required for 1 m3 of concrete

$$0.281(\ 2.70 \times 1000\ kg/m3\ ) = 759\ kg/m3$$

9. Total Design Mass

Cement	389
Water	175
Coarse	973
Fine	759 SSD

2296 kg/m3 at 5 % air and aggregate at SSD.

10. Theoretical air free density

$$2296/(\ 1.0 - 0.05\ ) = 2416\ kg/m3$$

11. Sand content as a % of total aggregate by volume.

$$\{\ 0.281/(\ 0.281 + 0.0371)\ \} \times 100\ \% = 43.1\ \%$$

12. Admixture quantities-air entraining agent (AEA)

Darex AEA: Manufacture's suggested dosage 0.75 fluid oz. (US) per sack cement

0.75 oz. = 22.2 ml/oz.

22.2 ml/40 kg = 0.55 ml/kg (1 sack = 40 kg)

Note: In a previous trial mix undertaken to become familiar with proper laboratory techniques it was determined that the entrained air was to low using a ratio of 0.55 ml/kg. In the next 5 trial mixes the quantity of AEA was varied until a suitable relationship between cement, type of coarse aggregate and % of entrained air was established.

For S-2 AEA was 0.65 ml/kg

13. Calculation of actual batch weights.

Batch Volume 0.02 m3

	Design Mass
Cement	389 kg/m3 X 0.02 = 7.78 kg
Water	175 kg/m3 X 0.02 = 3.50 kg
Coarse	973 kg/m3 X 0.02 = 19.46 kg SSD
Fine	759 kg/m3 X 0.02 = 15.18 kg SSD

14. Correction for oversize and undersize in aggregates.

Sand:

Per cent retained on No. 4 = 2.6 %

15.18 / 0.974 = 15.59 kg - amount needed to provide 15.18 kg of fines to the mix assuming no undersize in the coarse fraction.

Coarse

Per Cent passing No. 4 = 6.3 %

19.46 / 0.937 = 20.76 kg - amounted needed to provide 19.46 kg of coarse to the mix assuming no oversize in the fine fraction.

Adjustment for over/undersized.

Fine 15.59 - 1.31 = 14.28 kg

Coarse 20.76 - 0.41 = 20.35 kg and the final design values become:

Cement	7.78 kg
Water	3.50 kg
Fine	14.28 kg
Coarse	20.35 kg

Dry weights for measuring out of the aggregates are:

Fine 14.28(1 - 0.02) = 14.00 kg

Coarse 20.35(1- 0.024) = 19.86 kg

This was the final calculation before preparing the components for mixing.

Preparation for mixing

The coarse aggregate was dry overnight at 110 degrees C. The material was then weighed and place in water and allowed to stand overnight. Prior to mixing the water was drained and the amount of in excess of SSD was calculated.

Weight of coarse SSD = 16.95 - 16.77 = 0.18 kg - free water

Reduce water to be added. 2.88 - 0.18 = 2.70 kg

Fresh Concrete Test Calculations

Density: Concrete was places in a bucket of known volume and weighted. The bucket was filled with concrete and the density calculated using:

(wt. concrete - wt. tare)/ volume of tare

(19.71 - 3.36 kg)/ 0.007 m3 = 2312 = 2310 kg/m3

Appendix C

VISUAL DESCRIPTION OF THE TEST PRISMS

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
BP 1-1	280	Corner failed. Two aggregate pieces show deterioration. 1) 13 mm white chert badly fractured. 2) 6 mm carbonate mottled shows one large fracture.
	300	In the middle of the specimen two hairline D-cracks have appeared at 300 cycles.
	Popouts: 10 small popouts approximate size 6 mm maximum size, 10 mm on sides.	
BP 1-2	168	Failed edge - large piece fell off of edge- Two > 13 mm aggregate caused distress Both brown homogeneous fine-grained carbonate.
	170	Pop-out > 13 mm aggregate brown carbonate - large crack.
	212	D-cracks - network of hairline cracks forming at end. Small failure; 2 carbonate aggregates showing large crack near stud. One white, one brown.
	260	Failed edge and 13 mm aggregate, brown fine-grained aggregate showing numerous cracks, badly shattered.
	Popouts: 12 small approx. 3 - 6 mm except for popout noted at cycle 170.	

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
BP 1-3	210	D-crack - large hairline crack appearing on corner and end.
	213	Failed Corner - large piece of corner failed all (3) > 13 mm aggregate show cracks and include 1) very white homogeneous fine-grained carbonate badly shattered. >13 mm 2) mottled greenish brown >19 mm badly shattered 3) cream coloured fine-grained carbonate smallest of 3. Shows 2 large fractures, along possible bedding planes as alignment of cracks is preferred along a similar orientation.
	280	Failed edge - small failure no large aggregate visible. Well developed network of D-cracks.
	300	Failed edge 280 cycles - further deterioration reveals >13 mm as source of distress. Brown homogeneous fine grained; further a red brown very light >10 mm is cut in half. Appears to be deteriorated as result of the brown homogeneous aggregate expansion. Failed edge (new at 300) - >6 mm chert; violet somewhat opaque. Badly shattered. Popouts: 11 large size range 5- 6 mm 5-13 mm 1-10 mm

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
BP 2-1	30	Large D-cracks - developed in the middle of the specimens parallel to the end.
	57	D-cracks very large, threatening to fail prism completely.
	130	Complete failure - prism has broken into two pieces. Coarse aggregate varied in degree of deterioration 1) >25 mm white 'chalky' limestone badly shattered 2) brown grey fine-grained carbonate badly shattered. A large crack developed between this aggregate piece and the one described previously. the crack ran through both the mortar and other coarse aggregate. The other aggregates generally show no other signs of distress. 3) Other coarse (>25 mm) aggregate showed deterioration; generally not as severe as point 1 or 2. Often, the form being exfoliation of the particles surface. 4) Several coarse aggregates show no indication of freeze-thaw deterioration.
BP 2-2	168	Large D-cracks. Well developed pattern of cracking at end A.* Some minor cracking along sides.
	220	Failure of end A. Progressive deterioration occurred between 168 and 220 cycles. At 220 cycles End A separated from the prism, revealing many badly shattered carbonate coarse aggregate. Description 1) chalky white limestone; completely shattered almost "friable". >19 mm 2) several brown homogeneous carbonate range in size from 19 to 38 mm show severe cracking. All appear similar. Note: This type of carbonate does not appear to have benefitted from size reduction as the 19 mm is as badly shattered as the 38 mm. 3) Orange brown >25 mm aggregate shows deterioration, texture appears coarser than aggregate in point 2. 4) Intacted carbonate coarse aggregates are darker brown than aggregates above.

		aggregate. No change in porosity is obvious through visual inspection although the aggregate appears to have a coarser macro texture.
	260	Well developed D-cracks are developing along edge and sides of prism.

*Note: END A and END B refer to the ends of the prisms that are LABELLED "A" and "B".

BP 2-3	200	Progressive failure of the specimen. Many badly fractured carbonates. Again there are some durable carbonates. 1) Evidence of paste separating from aggregate, possibly due to hydraulic pressure, from the aggregate particle. End A shows examples of; a) badly overstressed paste b) deterioration of small coarse aggregate c) freeze-thaw resistant carbonate aggregate +13 mm. 2) End B shows one large brown aggregate with dark brown inclusions. - inclusion - pseudo dendritic pattern possibly pyrite. This aggregate is badly shattered. Dark brown inclusions have been noted in other deteriorated aggregate. Cracking is well developed throughout the prism.

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
BP 3-1	300	Fine D-crack or popout developing Popouts: 3, very small, 3 to 6 mm BP 3-1 had a bad stud, no expansion data is available on this prism.
BP 3-2	300	No deterioration present Popout: 10 - 3 mm (small) one chert, mainly carbonate
BP 3-3	300	Popout: large +13 mm appear to be carbonate off white may possibly be chert; on end.
BP 3-4	300	No deterioration present.

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
S 1-1	270	Failed corner End A. Well developed D-crack around a shattered >25 mm aggregate. Brown homogeneous fine grained badly shattered. D-crack hairline crack pattern seen at other end on corner and side.
	300	End B; minor failed corner. D-crack lines have widened
		Popouts: 5 very minor 6 - 10 mm
S 1-2	300	No serious deterioration; one set of fine D-cracks along one edge Popouts: 10 minor 6 mm. End B shows two brown carbonate popouts 6 mm.
S 1-3	290	Large popout +19 mm grayish aggregate appears heterogeneous, black flecks.
	300	D-cracks - End A; shows fine cracking developing on the end and on the side adjacent to the end.

PRISM	CYCLE	DESCRIPTION OF DETERIORATION
S 2-1	56	Large popouts at +13 mm and many at 10 mm. Surface was poor due to a very dry mix used during casting.
	300	Popouts: approx. 10-10 to 13 mm 10 at sizes less than 10 mm
	300	No D-cracking evident
S 2-2	56	Large popouts; 10 to 13 mm also some smaller. Three of the aggregate are chert.
	101	Large popout plus D-cracks. Yellow brown mottled carbonate; brown fleck, possibly pyrite. This piece of aggregate has caused distress in the surrounding area resulting in hairline D-crack appearing.
	282	Popout are frequent, generally carbonates. The larger fraction (10 to 13 mm) of the coarse aggregate appears to be deteriorating faster.
	300	Popouts: Size; 10 to 13 mm approx. 15 major popouts. Less than 10 mm is difficult to estimate due to poor finish as many were exposed and have not necessarily deteriorated.
S 2-3	General	- better finish than the other S 2 prisms.
	160	Large popout. End A; caused a large amount of damage for size of disrupting particle approx. 10 mm; possibly Chert.
	300	Large popout End B. Brown homogeneous fine grained carbonate; Note: there is a difference in the degree of damage between End B and End A. The mortar around End B is not effected where as there is a large amount of disruption around End A. Popouts: 9 - 6 to 13 mm.

Appendix D

ASTM C-666B RAW DATA

TABLE 14

Elongation of test prisms

(meters x 10-4)

CYCLE	BP1			BP2			BP3		S1			S2	
	1	2	3	1	2	3	3	4	1	2	3	1	2
0	0	0	0	0	0	0	0	0	0	0	0	0	0
30	.457	.457	----	2.46	.508	----	9	.381	.381	.406	.432	.356	.483
57	.406	.406	.406	2.03	.584	.838	----	.457	----	.381	.406	.432	.508
90	.711	.787	.737	3.18	1.09	1.78	.432	.584	.711	.432	.686	.66	.711
127	.584	.686	.66	4.90	1.32	2.64	.356	.584	.584	.508	.686	.66	.838
149	.635	.686	.737	*	1.83	5.03	.356	.406	.610	.508	.584	.711	.787
184	.762	.787	.914		2.90	9.93	.559	.610	.838	.66	.635	.787	.965
213	.66	.737	.762		3.86	19.1	.279	.381	.762	.533	.559	.635	.813
253	.762	.838	.889		21.0	*	.356	.432	1.04	.737	.787	.889	.914
280	.991	1.12	1.27		*		.432	.610	1.29	.940	.813	.864	1.09
300	.965	1.02	1.22				.483	.559	1.22	.762	.787	.762	1.09

Note: ----- No reading
* Specimen destroyed

Several prisms had the studs, used for length measurements, eccentrically placed on the end of the prism. This resulted in different lengths being measured depending on the orientation of the prism in the caliper. As this problem was discovered after the start of the test several prisms had to be disqualified as the original length (lo) was not known. The problem was seen with prisms BP3-1,BP3-2 and S2-3. During the remainder of the test these prisms were left in the freeze-thaw cabinet to act as dummy specimens, ensuring constant test conditions.

TABLE 15

Length of prisms at zero cycles

(meters)

Prism	lo (m)	Prism	lo (m)	Prism	lo (m)
BP1-1	0.35438	BP3-3	0.35093	S1-1	0.35356
BP1-2	0.35510	BP3-4	0.35420	S1-2	0.35474
BP1-3	0.35416			S1-3	0.35487
BP2-1	0.35248			S2-1	0.35449
BP2-2	0.35389			S2-2	0.35458
BP2-3	0.35290				