Pavement Assessment and Design Methodologies Used for the Rehabilitation of Urban Pavements in The City of Calgary, Alberta

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ABSTRACT

Like many agencies, the methods by which the pavement network is managed and maintained in The City of Calgary (The City) have evolved substantially over the past decades. At one time, the process was limited to visual windshield surveys, often by a construction superintendent or reaction to public complaints. Later, the development of more sophisticated pavement assessment tools enabled a more systematic Pavement Management System (PMS) approach.

Currently, The City utilizes the Municipal Pavement Management Application (MPMA), which was developed in the early 1980s by the province of Alberta. MPMA is used to assess the status of the pavement network, estimate the maintenance and rehabilitation needs of the network, and develop roadway rehabilitation programs based on user-defined budget constraints and Pavement Quality Index (PQI).

Around 2003, The City saw the need to expand upon their network-based assessment activities with a project level evaluation process. This resulted in the development of a three-tiered approach. Tier 1 comprises a review of PMS data, possibly a windshield survey, typically resulting in standard approaches being applied to lower profile projects. For Tier 2 assessments, this effort is supplemented by a detailed visual condition survey and review of other available information.

The subject of this paper is the Tier 3 Pavement Assessment program. In this case, available PMS information and a detailed visual condition surveys are supplemented by more in-depth evaluation. For a given project, this could include detailed traffic analysis, pavement thickness investigation using Road Radar™, Falling Weight Deflectometer (FWD) structural assessment, and potentially, geotechnical investigation. The overall approach is considered to represent the state-of-the-practice for pavement evaluation.

With the Tier 3 approach, the full spectrum of pavement strategies is considered, including maintenance only, mill and inlay, concrete pavement, concrete overlay, partial depth reconstruction, full depth reclamation, and pavement reconstruction. For some projects, more than one solution is deemed most appropriate for different segments. For higher trafficked roadways in urban municipalities, materials selection is particularly important and premium surfacing materials, such as Stone Matrix Asphalt (SMA), and mixes using asphalt rubber and modified asphalt binders have been used when warranted.

Over twenty Tier 3 evaluation projects have been completed to date. This paper will detail the assessment activities and process for development of rehabilitation strategies for two project-specific case studies where construction has been completed.

RÉSUMÉ

Abstracts provided in English will be translated to French and vice versa.

1.0 INTRODUCTION

The methods by which the pavement network is managed and maintained in The City of Calgary (The City) have evolved substantially over the past decades. Sophisticated pavement assessment tools were developed to enable a more systematic Pavement Management System (PMS) approach, compared to limited visual windshield surveys.

Around 2003, The City expanded upon their network-based assessment activities with a project level evaluation process. This resulted in the development of a three-tiered approach (Tier 1, Tier 2, and Tier 3).

The subject of this paper is the Tier 3 Pavement Assessment program. In this case, available PMS information and a detailed visual condition surveys are supplemented by more in-depth evaluation. For a given project this could include detailed traffic analysis, pavement thickness investigation using Road Radar™, Falling Weight Deflectometer (FWD) structural assessment, and potentially, geotechnical investigation. The overall approach is considered to represent the state-of-the-practice for pavement evaluation.

Over twenty Tier 3 evaluation projects have been completed to date. This paper will detail the assessment activities and process for development of rehabilitation strategies for two project-specific case studies.

2.0 THE CITY OF CALGARY PRACTICE FOR SELECTING PAVEMENT REHABILITATION PROJECTS

The City of Calgary currently utilizes the Municipal Pavement Management Application (MPMA), which was developed in the early 1980s by the province of Alberta. MPMA was designed to manage the provincial highway network and municipal pavement of major cities. MPMA is also used to assess the status of the pavement network, estimate the maintenance and rehabilitation needs of the network, and develop roadway rehabilitation programs based on user-defined budget constraints and Pavement Quality Index (PQI). The City of Calgary started to collect pavement condition data in 1985.

The purpose of the MPMA is to determine the most cost-effective use of public funds while maintaining safe, comfortable, and efficient roadways. An MPMA condition survey is performed every two years for major, collector, and industrial roads, and every six years for local roads. The MPMA condition survey, which is a walking survey, is conducted from April to November, where the following distresses are identified: alligator cracking, block cracking, longitudinal and transverse cracking, edge cracking, bleeding, distortion, rutting, ravelling, potholes, skin patches, and deep utility patches. The collected pavement data are loaded to the MPMA application to calculate a Visual Condition Index (VCI) and PQI. The PQI is used to analyze the condition of the road network, and ranges from 0.1 (very bad) to 10 (excellent).

The PQI trigger points vary depending on the type of the roadway, as shown in Table 1.

Table 1. PQI Trigger Points

Roadway Classification	PQI Trigger Point
Arterial	6.0
Collector	5.0
Industrial	5.0
Local	4.0

The pavement condition assessment for sections that reach the PQI trigger point and selected for rehabilitation strategies is generally a tiered approach. The tiered approach, which started in 2003, consists of three stages, as follows:

- Tier 1 comprises a review of PMS data, possibly a windshield survey, typically resulting in standard approaches being applied to lower profile projects.
- For Tier 2 assessments, this effort is supplemented by a detailed visual condition survey and review of other available information.
- Tier 3 comprises available MPMA information, detailed visual condition surveys, and more in-depth evaluation. For a given project, this could include detailed traffic analysis, pavement thickness investigation using Road Radar™, Falling Weight Deflectometer (FWD) structural assessment, and potentially, geotechnical investigation. The overall approach is considered to represent the state-of-the-practice for pavement evaluation.

With the Tier 3 approach, the full spectrum of pavement strategies is considered, including maintenance only, mill and inlay, concrete pavement, concrete overlay white-topping, partial depth reconstruction, full depth reclamation, and pavement reconstruction. For some projects, more than one solution is deemed most appropriate for different segments. For higher trafficked roadways in urban municipalities, materials selection is particularly important and premium surfacing materials, such Stone Matrix Asphalt (SMA), and mixes using asphalt rubber and modified asphalt binders have been used when warranted.

3.0 PAVEMENT CONDITION ASSESSMENT METHODOLOGY

The City of Calgary, Roads Division has commissioned EBA Engineering Consultants Ltd. (EBA) to undertake Tier 3 pavement evaluation and subsurface assessment program. The program includes:

- Visual Condition Survey.
- Road Radar™ to characterize pavement structure thickness and shallow subsurface anomalies, in addition to providing a video log record.
- FWD testing to assess structural adequacy.
- Subsurface investigation, if applicable.

The following sections describe the data collection and presentation.

3.1 Road Radar™ Survey

Subsurface profiling using the Road Radar™ System is a non-destructive measurement procedure. Although the system is generally able to resolve all significant subsurface layers, the automated classification of layer type based on material electrical properties can typically be improved with reliable as-built or historical information. Radar-detected layer interfaces are a function of the dielectric constant (or the electrical properties) of the material, which are measured by the Road Radar™ System using velocity determination.

The automatic classification of the different layer types (asphalt concrete, PCC, or granular base/sub-base, etc.) is based on the measured signal velocity and corresponding calculated dielectric values of each detected layer. Although the classification of the individual layer material type may be in error using this characteristic material dielectric approach, the measured layer thicknesses will be correct.

An example of the detailed continuous profile radar survey results for the surveys is provided as Figure 1.

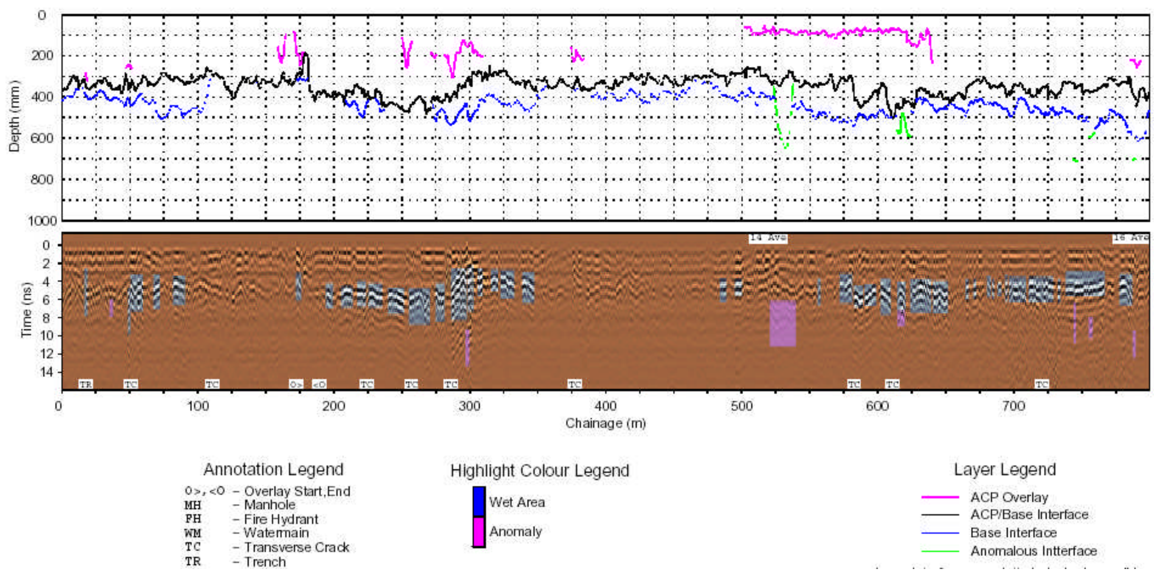

Figure 1. Typical Road Radar™ Profile Presentation

3.2 Falling Weight Deflectometer Survey

Pavement load/deflection testing is one of the main components of the pavement assessment process. FWD is an impulse type testing device that imparts a transient load upon the pavement surface. The magnitude and duration of the load closely approximates that of a single axle moving at moderate speed. Nine radial geophone sensors provide data used to determine surface deflection at known distances from the location of load application. At each test location, three load levels are used to determine the deflection response of the pavement.

FWD analysis following the American Association of State Highway and Transportation Official (AASHTO) design methodology is then carried out using the data from the FWD survey and the pavement layer thickness information using DAPA (Design of Asphalt Pavements Using AASHTO), which is a computer program developed by EBA. This program (DAPA) provides profile plots of back-calculated pavement layer and subgrade moduli and overlay thickness requirements for each individual FWD test location (point-by-point analysis). The structural assessment is normally undertaken to identify if any significant pavement structure deficiency exists (generally or localized) that may influence the selection of an appropriate treatment. In few scenarios, FWD test data is analyzed using ELMOD (Evaluation of Layer Modulii and Overlay Design), which is software that evaluates individual layer moduli and determines the overlay requirements.

4.0 PAVEMENT CONDITION ASSESSMENT CASE STUDIES

4.1 Case Study 1: Barlow Trail – 61 Avenue SE to Peigan Trail

4.1.1 Project Overview

The evaluation for Barlow Trail between 61 Avenue SE (km 0.0) and Peigan Trail (km 2.4) was part of The City of Calgary 2006 Pavement Rehabilitation Program.

The objective of this project was to assess the pavement condition, estimate traffic loading conditions, and provide rehabilitation strategies for consideration. The scope of work, considered consistent with a Tier 3 review, included a review of the available pavement condition and traffic information. This information review was supplemented by a detailed visual condition survey, pavement structure thickness survey (using Road Radar™), and deflection survey (utilizing FWD methods).

The section of Barlow Trail, subject of this review, is a four-lane, two-way, divided arterial roadway. The section is approximately 2,400 linear metres, with a total pavement area of approximately 40,000 m^2. The City's objectives for this road were the following:

- Provide a design solution to a specified serviceability for a 15-year period, before major rehabilitation may be required.
- The existing roadway footprint is to be maintained without widening.

The roadway is located in an industrial district. The section was generally a cut section, with curb and gutter and storm sewer drainage. Along much of the length, the areas adjacent to the roadway were landscaped. The majority of the centre median was landscaped, with some hard surface median in the vicinity of intersections. Signalized intersections exist at Peigan Trail, 50 Avenue, 58 Avenue, and 61 Avenue. In addition, there is a railway level crossing south of 50 Avenue. The posted speed limit is 70 km/hr, although congestion due to railway traffic and/or traffic volumes frequently results in slow traffic movement or standing traffic conditions. The section accommodates relatively high traffic volumes, with a very high percentage of truck traffic (up to 30%). The section is also a transit bus route. Generally, the primary performance issues with the subject pavement included wheel-path rutting, and in some areas, frequent, localized load-related failures that typically required ongoing maintenance to maintain serviceability.

4.1.2 Visual Condition Assessment

A detailed visual condition survey was undertaken for both lanes in each direction of travel. The survey was generally done in accordance with ASTM D 6433, "Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys". Based on the survey data, the most prevalent pavement distresses, and those likely to influence the rehabilitation, are summarized as:

- Segment A (km 0.0 to km 1.65), northbound lanes, from 61 Avenue to 50 Avenue: Medium to high severity wheel-path rutting (more severe at intersections), edge cracking, and moderate severity ravelling at 61 Avenue and 50 Avenue intersections.

- Segment B (km 1.65 to km 2.25), northbound lanes, from 50 Avenue to south of Peigan Trail: Fatigue (alligator) cracking primarily in the outer wheelpaths along curb lines with patching and shoving, medium to high severity wheel-path rutting (low severity where milled), frequent high severity transverse and medium severity longitudinal cracking.

- Segment C (km 2.25 to km 2.40), northbound lanes, from south of Peigan Trail to Peigan Trail: Few distresses, low to medium severity wheel-path rutting.

- Segment D (km 0.0 to km 1.65), southbound lanes, from 61 Avenue to 50 Avenue: Frequent low to medium severity transverse and longitudinal cracking, some localized fatigue cracking, low severity wheel-path rutting (medium at intersections), edge cracking, moderate severity ravelling at 61 Avenue and 50 Avenue intersections.

- Segment E (km 1.65 to km 2.40), southbound lanes, from 50 Avenue to Peigan Trail: Fatigue (alligator) cracking primarily in the outer wheel-path along curb lines with patching and shoving, medium severity wheel-path rutting (low severity where milled), frequent high severity transverse and medium severity longitudinal cracking, edge cracking.

Figures 2 and 3 represent some of the typical distresses for the subject section of Barlow Trail.

Figure 2. Barlow Trail, northbound lanes, between 61 Avenue and 50 Avenue, high severity wheel-path rutting.

Figure 3. Barlow Trail, southbound lanes, between 50 Avenue and Peigan Trail, fatigue cracking in the outer wheel-path along curb lines, medium severity wheel-path rutting

4.1.3 Traffic Loading Analysis

The traffic loading, in terms of the Equivalent Single Axle Loadings (ESALs), was estimated based on the Average Annual Daily Traffic (AADT) and percentage of truck traffic information for 2005 provided by The City. For this section, the AADT was 28,000, with 29% truck traffic. Based on a growth rate of 2% and 15-year design period, and based on a 60% lane split for the outside lane, the 15-year design ESALs were 28 million. The design ESALs for the inside lanes were 18 million.

4.1.4 Road Radar™ Investigation

The existing road structure thickness was assessed using the Road Radar™ System in June 2005. Data was collected continuously throughout the section (inside and outside lanes) in both the northbound and southbound lanes. Data was collected at the posted traffic speed, thereby eliminating the need for lane closures and the traffic disruption associated with closures.

Generally, the data indicates that the pavement thickness layers vary substantially within the section. A summary of the Road Radar™ investigation results is provided in Table 2.

Table 2. Road Radar™ Pavement Thickness Measurements (mm)

Lanes	NB	SB	NB	SB
Km to km	**0.0 – 1.7**	**0.0 – 1.7**	**1.7 – 2.4**	**1.7 – 2.4**
Asphalt Concrete Average Thickness	335	250	260	265
Granular Base/Sub-base Average Thickness	270	270	300	240
Total Pavement Average Thickness	605	540	565	505

*km 0.0 corresponds to the intersection of Barlow Trail and 61 Avenue. Stations increase northerly.

4.1.5 FWD Survey and Analysis

FWD survey of the portion of Barlow Trail under consideration was also conducted in June 2005. With the data from the FWD survey and the pavement layer thickness information from the Road Radar™ investigation, an analysis of the FWD test data following the AASHTO design methodology was carried out. A design reliability of 85% was used in the analysis. The initial and terminal serviceability were 4.2 and 2.5, respectively. The overall standard deviation was taken as 0.45, and the resilient modulus correction factor was taken as 0.36. The design input values required by the AASHTO method were as outlined in Alberta Transportation's (AT's) Pavement Design Manual. The overlay requirement with the back-calculated modulus of subgrade is presented in Table 3.

Table 3. FWD Survey and Overlay Thickness Analysis

Lanes	NB	SB	NB	SB
Km to km	**0.0 – 1.7**	**0.0 – 1.7**	**1.7 – 2.4**	**1.7 – 2.4**
Average Subgrade Modulus (MPa)	56	42	34	31
Subgrade Modulus Standard Deviation (MPa)	20	13	9	5
85th Percentile Overlay Thickness (mm)	0	121	166	208

FWD analyses indicated that, with the exception of the northbound lanes between km 0.0 and km 1.7, significant strengthening was required for this section of roadway. It should be noted that the analysis used a traffic loading of 12 million Design ESALs, combined with a relatively high reliability, 85%. Although less than the design 15-year traffic loading estimated, this value was selected to provide a legitimate structural assessment, given that the AASHTO method is considered conservative for very high traffic loading conditions. Of note is that no analysis was available for the southbound outside lane, north of km 1.5 (50 Avenue), due to data collection problems.

4.1.6 Salvage Feasibility Assessment

A second method of pavement structural analysis was used for the assessment. This analysis could be considered a salvage feasibility assessment. Based on the pavement structure identified and the estimated traffic loading, a remaining service life was estimated. An assumed subgrade modulus was assigned for each roadway section that corresponded to the average back-calculated value, less one standard deviation.

Based on AASHTO Table 5.2 in Part III, layer coefficients of 0.35 and 0.10 were assumed for the asphalt concrete and granular base, respectively, for the northbound and southbound sections between km 0.0 and 1.7 (61 Avenue to 50 Avenue). For the northbound and southbound section from km 1.7 to 2.4 (50 Avenue to Peigan Trail), layer coefficients of 0.10 for asphalt concrete and 0.00 for granular base were assigned. These layer coefficients were selected based on the existing pavement condition, and consideration of granular base quality (e.g., likely pumping condition as a result of full depth failure). Drainage coefficients were also assigned to reflect the quality of drainage. Based on these values, an Effective Structural Number (SN_{eff}) was determined for each pavement section. A design reliability of 75% was used for the analysis.

Based on this method, the analysis results were consistent with that using the deflection results (i.e., Table 3). The potential to accommodate the assumed loading condition for a 15- to 20-year service period was generally indicated for the northbound lanes from km 0.0 to 1.7, and marginally indicated for the southbound lanes from km 0.0 to 1.7.

Conversely, for the northbound and southbound lanes between km 1.7 and 2.4, the analysis indicated a very low reliability of satisfying a legitimate service period without significant strengthening. It should be noted that this analysis was theoretical in nature and that much of the pavement in this area has reached the terminal serviceability level.

4.1.7 Design Considerations

A primary consideration with regards to this project was the potential impact of traffic disruption. Any construction activity on this roadway, particularly during peak hours, would result in significant delays to commercial traffic. This warranted consideration in selecting a strategy that will potentially minimize the disruption and/or selecting a design concept that would minimize the need for future delays associated with maintenance and major rehabilitation works.

This roadway is one of the highest traffic loading routes within the city. The percentage of truck traffic is significant and as such was considered not only in the selection of an appropriate rehabilitation (or reconstruction) strategy, but also in selecting the proper materials for the construction.

In considering rehabilitation alternatives for this project, it was assumed that the existing curb lines will be maintained. This would eliminate the potential for overlay rehabilitation.

4.2 Recommendations and Considerations

4.2.1 61 Avenue to 50 Avenue – Northbound Lanes

Based on this assessment, this section was considered a good candidate for a mill and hot mix asphalt (HMA) inlay approach. This section included the 50 Avenue and 61 Avenue intersections. In addition, the following strategy was also considered appropriate for the northbound lanes for approximately 150 m south of the Peigan Trail intersection. Given that minimal load-related distress was noted within this section, very little partial or full depth repairs were warranted.

In response to the existing wheel-path rutting and the anticipated future traffic condition, a 100 mm replacement depth was recommended. The HMA inlay was recommended to be of the highest quality material to mitigate future instability rutting.

This rehabilitation strategy was expected to provide an adequate level of serviceability, with only routine maintenance, for a 15-year or more service period. As the work could be completed in stages and/or outside of peak traffic volume periods, the resulting traffic disruption would be minimal.

4.2.2 61 Avenue to 50 Avenue – Southbound Lanes

Based on this assessment, the condition of this section did not warrant reconstruction. Although the structural assessment identified a need for strengthening, limited pre-rehabilitation treatment would aid in attaining an appropriate service period and serviceability. This section included the 50 Avenue and the 61 Avenue intersections. Although this section was considered a legitimate candidate for a mill and HMA inlay approach, the probability of fulfilling a 15-year or more service period was not as high as for the northbound lanes. There was the requirement to undertake full depth repair of localized fatigue failure. In addition, the relatively high frequency of transverse cracking would reflect through the new surfacing, resulting in a potential for water infiltration.

The severity of rutting in this section was significantly less than the northbound lanes. Based on the existing wheel-path rutting condition, a 50 mm replacement depth was considered appropriate for the majority of the length. A replacement depth of 100 mm was recommended in intersection areas, extending a minimum 150 m back from the intersection stop-line. The HMA inlay was recommended to be of the highest quality material to mitigate future instability rutting. This rehabilitation strategy was expected to provide an adequate level of serviceability, with ongoing crack treatment and routine maintenance, for a 12- to 15-year period.

4.2.3 50 Avenue to Peigan Trail – Northbound and Southbound Lanes

In developing alternative strategies for this segment of the project, consideration was given to a range of treatments, from that which provides a high performance (e.g., concrete overlay) to what might be termed a minimalist approach (e.g., "standard" mill and inlay treatments, which have a limited service period). Other considerations included:

- The existing condition was poor with frequent areas of failure. This, and the ongoing maintenance required indicated this section, exhausted the service life associated with a minimal acceptable level of serviceability.
- The structural assessment indicated that the pavement structure was inadequate for the anticipated traffic loading. Therefore, a pavement reconstruction approach (flexible or rigid) was considered.
- Even when the poor condition was considered, based on the existing structure thicknesses, the pavement should probably be providing a higher level of serviceability and structural adequacy to what is currently the case. This would suggest that foundation issues, such as poor subgrade materials and/or subsurface drainage, were major contributing factors.

Several alternatives were considered for this section. The options of assumed rehabilitation are as follows:

- Option 1: Rehabilitation consisting of Full Depth Repairs, with 100 mm Mill and Inlay. It was assumed that approximately 30% of the area would require full depth repair.

- Option 2: Conventional Flexible Pavement Reconstruction consisting of 300 mm HMA, 200 mm Crushed Granular Base, and 250 mm Granular Sub-base. This option would include subgrade preparation and subsurface drainage, which would result in improved subgrade support characteristics, with a 20-year or more service period.

- Option 3: Flexible Pavement Reconstruction consisting of 250 mm of Milling, 200 mm depth of Full Depth Reclamation (FDR) and Stabilization, and 250 mm HMA. Because the subgrade would not be improved, the design would provide a lower design service period of 15 years.

- Option 4: Rigid Pavement Partial Depth Reconstruction, consisting of 230 mm of Milling with a 230 mm Portland Cement Concrete Inlay. The design assumed a minimum structure below the rigid pavement. This option would provide a 30-year design life.

4.2.4 Life Cycle Cost Analysis and Preferred Strategy

A simplified Life Cycle Cost Analysis (LCCA) was carried out to evaluate the four alternatives identified for Barlow Trail between 50 Avenue and Peigan Trail. The baseline analysis was carried out using unit costs for various construction materials and operations provided by The City.

Basic inputs were an analysis period of 30 years, a discount rate of 4%, and a salvage value proportional to the estimated remaining service life at the end of the analysis period. Based on the service life and the associated estimated costs, LCCA was used to calculate the net present value of each option. User costs, including delay costs associated with construction and/or major rehabilitation, were not included in the analysis. Table 4 provides a comparison of the initial cost and net present value (complete with ranking) for the four options.

Table 4. Life Cycle Cost Analysis – Barlow Trail – 50 Avenue to Peigan Trail

Option	Initial Capital Cost ($)	Net Present Value (NPV) ($)	Rank	Net Present Value (NPV) Comparison (Based on Lowest NPV)
1	536,500	726,000	2	1.17
2	663,000	620,000	1	1.00
3	708,000	774,000	3	1.25
4	1,200,00	931,000	4	1.50

As shown, the number one ranked alternative based on LCCA is Option 2, Conventional Reconstruction, which has a net present worth (NPV) that is 17% less than the next lowest ranked treatment. It is noted that generally, differences in NPV of less than 5% are not considered significant.

4.2.5 Reconstruction Alternative Assessment

An additional cost analysis was undertaken to assess three reconstruction alternatives:

- Option A, the conventional construction described in the previous section.

- Option B, an alternative using FDR base as a granular base substitute.

- Option C, a design incorporating foamed asphalt stabilized base.

For each of the FDR alternatives, the material would be pulverized, stockpiled on site, and placed after completion of granular sub-base construction. Each alternative design was based on a 20-year design traffic or 37 million ESALs, a design reliability of 90%, and a design subgrade modulus of 32 MPa. The required structural number to satisfy these inputs was 175 mm. The alternative designs are described in Table 5.

Table 5. Reconstruction Alternatives

Pavement Component	Option A	Option B	Option C
Hot Mix Asphalt [1]	300 mm	300 mm	220 mm
Crushed Granular Base	200 mm	-	-
Full Depth Reclamation Base (Water Bound)	-	200 mm	-
Full Depth Reclamation Base (Bituminous Stabilized)	-	-	200 mm
Granular Sub-base	250 mm	250 mm	250 mm
Resulting Structural Number (SN)	178 mm	178 mm	178 mm
Total Pavement Structure Thickness	750 mm	750 mm	670 mm
Notes: [1] All options include 40 mm SMA, 100 mm Superpave Intermediate Course, and 80 mm to 160 mm Superpave Base Course			

An initial cost estimate was developed for each alternative. Table 6 provides the results of the life cycle cost analysis for the three options.

Table 6. Life Cycle Cost Analysis – 50 Avenue to Peigan Trail – Reconstruction Alternatives

Option	Initial Capital Cost ($)	Net Present Value (NPV) ($)	Rank	Net Present Value (NPV) Comparison (Based on Lowest NPV)
A	676,350	631,942	3	1.07
B	648,550	608,427	2	1.03
C	625,890	589,261	1	1.00

Based on the LCCA, the two FDR alternatives indicate a 3% to 7% lower NPV compared to conventional reconstruction. This suggests that these alternatives may provide significant savings both in terms of initial cost and LCC. In addition, these options would reduce track traffic to the site (less excavation removal and less imported granular). The NPV cost savings (3%) between the stabilized (Option C) and unstabilized FDR (Option B) was not considered significant. This, in addition to the fact that the construction period would likely be lengthened with the stabilization process, suggested that stabilization may not be cost effective for this project. Based on this assessment, FDR was worthy of consideration, but no clear superiority existed between stabilized and non-stabilized alternatives. Given that pulverization and, if required, stabilization equipment were as yet not common, their mandated use, when no clear cost advantage existed, may be inappropriate. For this reason, the best solution was considered offering the 200 mm base layer as the contractor's option to use either crushed granular base course or FDR material.

4.2.6 Hot Mix Asphalt Mix Types

The HMA mixtures for the project were specified in accordance with current Superpave and SMA protocols. Table 7 provides the specific recommendations for this project.

Table 7. HMA Mix Types – 50 Avenue to Peigan Trail – Reconstruction Alternatives

Mix Type	Binder Grade	Aggregate Gradation Classification	Aggregate Nominal Maximum Size (NMS)	Aggregate Consensus Property Requirements	HMA Design Requirements
Premium Asphalt Concrete Surfacing	PG 76-31	Stone Matrix Asphalt (SMA)	12.5mm	NOTE 1	NOTE 1
Intermediate Course Asphalt Concrete	PG 76-31	Superpave Fine or Course Graded	20mm	<30 Million ESALs (Depth from Surface <100mm)	3 – <30 Million ESALs
Base Course Asphalt Concrete	PG 58-31	Superpave Fine or Course Graded	20mm or 25mm	10 – <30 Million ESALs (Depth from Surface >100mm)	3 – <30 Million ESALs
NOTE 1: In accordance with AASHTO Provisional Standard PP 41-01 (2003) Designing Stone Matrix Asphalt (SMA)					

4.2.7 Construction

The 50 Avenue to 61 Avenue section of Barlow Trail paving was undertaken on August 26 and 27 on the northbound lanes and September 2, 2006, and September 3, 2006, on the southbound lanes (Figure 4). The paving was undertaken during two weekends to minimize traffic disruption. The rehabilitation included milling of the existing pavement to a typical depth of 60 mm in the southbound lanes and 100 mm in the northbound lanes. In the southbound lanes, the milling and replacement depth was increased to 100 mm within intersection areas. Based on the monitoring, the level of organization, dedication to quality and workmanship associated with the SMA installation exceeded industry standards. This resulted in to date high performance of the rehabilitation for this section. The 50 Avenue to Peigan Trail section was reconstructed in 2008/2009 in conjunction with an AT upgrading of the Barlow/Peigan/ Deerfoot Trail (Highway 2) Interchange. The construction was in accordance with the recommendations provided to The City, including Superpave specified mixes with an SMA surfacing. To date, the early performance of this section is very good (Figure 5).

Figure 4. Barlow Trail Paving, northbound lanes, south of 61 Avenue.

Figure 5. Barlow Trail Reconstruction (2009), between 50 Avenue and Peigan Trail.

4.3 Case Study 2: Spruce Meadows Drive (Highway 22X) – 85 Street to 8 Street

4.3.1 Project Overview

Highway 22X (Spruce Meadows Drive) from 85 Street (Station 0+000) to 8 Street (Station 9+500), was part of The City of Calgary 2006 Pavement Rehabilitation Program.

The scope of work, considered consistent with a Tier 3 review, included a review of the available pavement condition and traffic information. This information review was supplemented by a visual condition survey and deflection survey (utilizing FWD methods).

The section of Highway 22X, subject of this review is a four-lane, two-way, divided arterial roadway. At the time of the investigation, there was an AT inventory that provided the existing pavement structure for the subject roadway. The inventory indicated that the eastbound lanes were the original carriageway and were constructed in 1961 as a gravel-surfaced roadway. In 1967, the roadway was upgraded to a cold-mix asphalt surface, and in 1970, it received an asphalt concrete surface. Subsequently, the roadway was overlaid in 1988.

The roadway was then twinned in 1987, with the new construction representing the westbound lanes. This section was paved in 1988 and chip-sealed in 1992.

4.3.2 Visual Condition Assessment

A detailed visual condition survey was undertaken for both lanes in each direction of travel. Based on the visual condition survey, the primary distresses in the eastbound lanes were longitudinal and transverse cracking, which were medium to high severity and extensive (50% density and higher). Other less extensive distresses were high severity alligator cracking and low to medium severity rutting.

The most predominant distresses in westbound lanes were medium severity longitudinal cracking and low severity rutting throughout the section. Transverse cracking of varying severity effected about 7% of the section.

Photographs illustrating the condition of the eastbound and westbound lanes are provided in Figures 6 and 7, respectively.

Figure 6. Typical Pavement Condition, Spruce Meadows Boulevard Eastbound.

Figure 7. Typical Pavement Condition, Spruce Meadows Boulevard Westbound.

4.3.3 Traffic Loading Analysis

The traffic loading, in terms of the ESALs, was estimated based on the AADT and percentage of truck traffic information by data provided by The City and the existing data by AT. Based on AT PMS data, the AADT was 8560 with 7.0% commercial traffic, which would result in 401.9 ESALs/Direction/Day. Using 15 years, 2% Growth, and 70% in Design Lane, the design ESALs were 1.8 million. However, The City's data indicated an AADT of 12,000 with 11% Truck Traffic. Using 15 years, 2% Growth, and 70% in Design Lane, the design ESALs were 2.6 million.

4.3.4 Existing Pavement Structure from AT Records

The existing pavement structure in the westbound lanes of this roadway section consisted of 110 mm asphalt concrete and 350 mm of granular base. The eastbound lanes consisted of 110 to 175 mm asphalt concrete and 350 mm of granular base. The westbound lanes were chip sealed.

4.3.5 FWD Survey and Analysis

FWD survey of the portion of Highway 22X under consideration was conducted. With the data from the FWD survey and the pavement layer thickness from the existing AT&T PMS data, an analysis of the FWD test data following the AASHTO design methodology was carried out. The overlay requirement was 97 mm (80th Percentile: 125 mm) for the westbound lanes and 60 mm for the eastbound lanes (80th Percentile: 78 mm). The overlay requirements were based on the 1.8 million design ESALs.

4.3.6 Recommendations and Considerations

Due to the requirement to increase pavement width and recognizing the poor pavement condition, a FDR Design Alternative was selected for the eastbound lanes, with the following design structure:

- 125 mm HMA (2 lifts, 65 mm lower/60mm upper).
- 200 mm Foamed Asphalt Stabilized FDR (comprised of the upper 350 mm of the existing pavement structure 175 mm ACP/175 mm GBC).
- 200-250 mm Granular Sub-base (in-place beneath the existing roadway footprint).

For the westbound lanes, a 100 mm, two lift overlay was selected.

4.3.7 Construction

Construction was undertaken in 2006 for both the eastbound and westbound lanes. Figure 8 shows the FDR construction in the eastbound lanes.

Figure 8. Spruce Meadows Boulevard Eastbound Lanes – FDR Construction

To date, this section of roadway has no major distress issues and high performance is expected over the design service period.

5.0 SUMMARY

The intent of this paper was to illustrate how "state-of-the-practice" pavement assessment techniques and design methodologies can be applied to urban pavement infrastructure. The objective of these efforts is to provide an optimum pavement strategy or strategies for each project based on the site-specific conditions.

The case study projects presented show how a wide range of alternatives can be considered, enabling the selection of a pavement strategy that will provide the best long-term solution in terms of both performance and cost effectiveness. Ultimately, it is The City of Calgary taxpayers who benefit from this level of pavement asset management.