Control of Full Depth Pulverized Aggregate Production using Ground Penetrating Radar

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ABSTRACT

Many rural highways in Atlantic Canada are in poor condition due to limited maintenance funding available. The typical maintenance strategy has been to place an overlay on the damaged asphalt concrete layer to provide a new wearing surface, but this approach does not repair the damage embedded within the pavement structure. After a certain period of time, the original cracks reflect through the overlay, leading to its premature failure. A newer approach has been to repair these heavily damaged roads using a full depth pulverization technique which grinds and stabilizes the upper portion of the existing road to provide a new base layer that is free of defects. While this technique provides a more sustainable repair approach in re-using in-situ materials, the resulting base typically exhibits a high degree of variability. It is hypothesized that pulverizing the pavement to a constant depth, or using a retroactive control method to achieve a specific blend of asphalt concrete to granular base for the pulverized materials, may contribute to the observed variability in the recycled base layer. The objective of this research is to determine if ground penetrating radar thickness estimates can be used to improve variability in full depth pulverized aggregates by maintaining a constant blend ratio during pulverization. It is expected that improvements in the consistency of the full depth pulverized materials will lead to improvements in the compaction and consistency of the stabilized materials. Pulverized aggregate samples were obtained from two full depth recycled pavement rehabilitation projects utilizing retroactive depth control and GPR depth control, respectively. A wide degree of asphalt concrete/base blend ratio was observed for the retroactive control section, while greater consistency in the blend ratio was maintained in the GPR control section by subdividing the project according to various pulverization depths. The GPR control section exhibited lower variability in the gradation, optimum moisture content, optimum density, and California Bearing Ratio than the retroactively controlled section. It is expected that improvements in the consistency of the full depth pulverized materials will lead to improvements in the compaction and quality of the stabilized materials.

INTRODUCTION

Full depth reclamation (FDR) is a pavement rehabilitation method that has been used around the world for over 25 years. With this process, the entire depth of flexible pavement, along with a portion of the underlying layer is pulverized, typically stabilized, and recompacted to provide a strong, damage-free base layer. This strengthened base is then overlain with a chip seal, thin HMA, or other appropriate layer to act as a wearing surface (1). The FDR process has been the subject of extensive research as it provides an environmentally friendly, economical means of structurally rehabilitating damaged pavements.

Environmental benefits include eliminating waste materials by reusing the existing pavement materials on-site, reducing the requirement for virgin aggregates, and reducing emissions associated with transportation of materials (2). Cost benefits include the reduction in processing and trucking virgin aggregates to the site (3), as well as a fast production rate compared to most alternative rehabilitation methods, thus reducing both construction and user delay costs. By utilizing FDR, construction costs have been found to be reduced by as much as 25 to 50 percent, compared to conventional rehabilitation methods (4).

There are structural benefits to using FDR as well. The traditional maintenance approach in Canada has been to place a hot mix asphalt concrete overlay on the existing damaged asphalt concrete pavement. Regions of severe deterioration are generally recommended to be repaired by removal and replacement with new hot mix asphalt concrete prior to overlay placement, but this is rarely conducted on rural highways. The localized reduction in stiffness caused by the pre-existing cracking damage in the supporting materials can result in stress concentrations in the overlay, eventually resulting in reflective cracking that propagates to to the surface. This damage is removed from the pavement with the FDR process by crushing the bound materials back into an aggregate structure that can be stabilized or used as a granular base. Stabilizing the FDR aggregates improves the cohesiveness, strength, and modulus of the base, thereby reducing stresses on subgrades and requiring thinner wearing surfaces to withstand traffic loads.

A number of additives are available to stabilize the recycled material including bituminous (emulsified or expanded asphalt), chemical (lime, Portland cement, fly ash) and many others. Binding the FDR aggregate produced from the pulverization with expanded asphalt provides a similar inter-particulate frictional resistance to deformation, but with added cohesion and stiffness within the material. Since the particles are discontinuously bound, failures in expanded asphalt stabilized FDR bases tend to occur due to rutting instead of fatigue. Alternatively, the use of Portland cement as a binder tends to provide a more continuous bond throughout the layer, resulting in a relatively weak rigid base that generally fails under fatigue loading. The choice of stabilizer to be used depends on the changes to various physical properties that are required for each project. Properties that can be improved with additives include (5):

- Strength and durability;
- Plasticity index;
- Dust generation during construction;
- Moisture content; and,
- Moisture susceptibility

Agencies may opt to use full depth reclamation for a number of reasons. FDR may be the most appropriate option under the following conditions (5):

- Pavement requiring full depth patching over greater than 15-20% of the surface area;
- Pavement requiring increased structural capacity;
- Pavement in which existing distress is caused by base or sub base failure; and,
- Pavement condition so seriously damaged that traditional rehabilitation approaches will not correct the problem

Many rural Canadian highway pavements exhibit these characteristics. Restricted maintenance budgets, coupled with limited availability of alternate construction methods and materials, have typically resulted in a continuing cycle of pothole patching and overlay treatments to maintain serviceable pavements. The effect of restricted maintenance budgets for a deteriorating network of pavements usually means that such treatments are made too infrequently and after the rate of deterioration has progressed too far. As a result, the effective service life of the overlay is shortened and the pavement can quickly revert back to its deteriorated state until the next infrequent treatment is budgeted. The ability of FDR to remove cracking damage from the bound HMA layers removes these areas of localized flexibility, therefore increasing the service life of such overlays and stiffening the total pavement structure over the subgrade to reduce rutting effects. Accordingly, FDR offers a very attractive method for restoring rural Canadian pavements to a high level of service and a low rate of deterioration.

FDR Design and Construction in Atlantic Canada

One of the most commonly used FDR design methods is provided in the Wirtgen Cold Recycling Technology manual (6). A pavement condition assessment is normally conducted first in conjunction with field sampling to determine the number, type and thickness of the pavement structure and the most appropriate maintenance treatment (7). If a FDR approach is warranted, asphalt concrete samples obtained from the pavement are crushed and blended with the underlying granular materials to develop an optimal blend ratio to meet certain gradation requirements. While the Wirtgen manual does not provide specific instruction on the materials preparation and blending methods, the practice in Atlantic Canada has been to reduce asphalt concrete specimens to a manufactured Recycled Asphalt Pavement (RAP) using a jaw crusher. The asphalt binder is extracted from a sample of the prepared RAP to determine the in-situ asphalt content. This permits post-stabilization quality assurance testing to ensure that the correct dosage of expanded asphalt has been added to the stabilized base materials. A design Job Mix Formula (JMF) blend of FDR aggregates is prepared, typically containing no more than 75 percent by mass of manufactured RAP, approximately 1 percent crusher dust to simulate fines production during the secondary stabilization phase of construction, approximately 0.5 percent Portland cement, and the remaining proportion by mass of the in-situ granular materials. The proportions of RAP and granular materials in the design FDR aggregate blend may be adjusted and/or a corrective aggregate may be required to obtain a gradation within the specified limits. Two percent hydrated lime may be used instead of 0.5-1.0 percent Portland cement if the plasticity index of the granular materials exceeds 10. The gradation limits used in the New Brunswick Department of Transportation (NBDOT) specification for expanded asphalt stabilized FDR provide for a range between 45-70% passing the 4.75 mm sieve opening and 5-20% passing the 0.075 mm sieve opening (8), whereas Wirtgen suggests the gradation limits listed in Table 1. It is not apparent from the Wirtgen manual if these gradation limits represent the unbound aggregate size distribution that is obtained after the asphalt concrete is extracted from the RAP component, or if it represents the particle size distribution considering the RAP as conglomerate individual particles. These limits are provided to presumably ensure adequate mechanical and strength properties of the stabilized base. The proportions by mass of crushed RAP and granular base required to meet the gradation specifications translate into the total pulverization depth required by virtue of the bulk densities of the two materials and the mean HMA thickness within the design section.

In the case of expanded asphalt FDR, the optimum water content required to obtain the foaming properties of the asphalt bitumen is determined as the average value from those yielding the minimum acceptable expansion ratio (≥ 8 times), which is the ratio of the maximum expanded volume to the original binder volume; and the minimum acceptable half-life (≥ 6 seconds), which is the time required for the expanded asphalt to collapse to half of its maximum volume. Various amounts of binder about the expected optimum value are then mixed with the JMF design FDR aggregate blend to prepare trial, followed by the addition of sufficient water to achieve optimum moisture content and density for the preparation of 100 mm diameter briquettes for tensile strength testing. Specimens are compacted using the Marshall method with 75 blows to each specimen face. The mean dry and soaked Indirect Tensile Strength (ITS) are measured for different trial batches to determine the optimum binder content to meet the specified ITS values and the indirect Tensile Strength Ratio (TSR) of soaked to dry ITS. The NBDOT specifications for expanded asphalt stabilized base materials were reduced in 2012 as listed in Table 2 as recommended in the 2010 Wirtgen Cold Recycling Manual (6) for secondary rural roads carrying up to 3 million ESAL.

A two-stage in-situ FDR process is typically used in Atlantic Canada where the deteriorated pavement is initially pulverized and compacted to carry local traffic until the stabilization work begins. Anecdotal evidence suggests that approximately one percent of unbound dust occurs in the gradation after the first pulverization, but this is highly dependent on the ratio of the total asphalt thickness to the full pulverization depth. Generally speaking, the larger the proportion of granular materials in the full pulverization depth, the greater will be the presence of dust in the blended material. The second stage involves using the same pulverizing equipment to mix the pulverized FDR aggregates with the binder. This provides further reduction of large HMA chunks which may not have been fully reduced in the initial stage. Anecdotal evidence has shown approximately two percent dust occurs in the gradation after the second pulverization. Optimum gradation generally seems to occur when sufficient pulverization depth is provided to cool the cutting tools with ambient moisture residing in the granular materials, yet shallow enough to avoid a vertical or slightly backwards upward cutting motion when the tools contact the asphalt. In such cases, the relative thickness of the HMA layer is small compared to the total pulverization depth and large chunks tend to break off, rather than be ground into FDR aggregate particles. Additional moisture also usually added to achieve optimum moisture content to maximize the compaction density. Alternatively, if the pavement or materials become too wet, the aeration and drying enabled by the pulverization process can permit the material to dry below optimum moisture content for the second stage of the process. For typical lane widths of 3.5 metres, two adjacent passes are usually required for complete coverage. The stabilized FDR aggregate materials are spread and shaped to approximately the desired final depth, grade and cross slope. Proper compaction for reclaimed material is key to developing adequate strength, modulus and pavement performance (7).

Despite a well-established design process, as-built material properties have varied significantly in many projects constructed in New Brunswick in recent history. Table 3 lists the mean dry and soaked ITS, and the corresponding indirect TSR observed for expanded asphalt FDR projects conducted in New Brunswick since 2006. In many cases, either the dry or soaked ITS and/or TSR have not met the required specifications. A correlation has not been observed between the variability in the as-built strength parameters and specimens prepared from the job mix formula. It follows that some aspect of the actual pavement structure varies from the conditions assumed in the development of the design. Many pavements selected for rehabilitation with full depth reclamation may have previously been subject to a number of overlays, patchwork, or other measures meant to extend the life of the road. As a result, the thickness of asphalt can vary significantly along the length of a given road. It is hypothesized that the observed strength differences between the job mix formula and the as-built FDR base materials could result from thickness variability that is not accounted for by the relatively sparse number of test pits and cores typically used to investigate the pavement structures. One technology that can complement the site investigation to provide a much stronger statistical measure of the pavement layer thicknesses is Ground Penetrating Radar (GPR).

Ground Penetrating Radar

GPR is an electromagnetic non-destructive method of evaluating the various layer comprising a pavement structure and estimating their thickness. Typically an air-coupled antenna is used in pavement investigations to direct a brief pulse of microwave energy into the pavement structure. GPR waves travel through a medium at a velocity, V, that depends on its relative dielectric permittivity, e_r, and magnetic permeability, μ_r. Most construction materials are nonmagnetic, giving the magnetic permeability a value of 1 and making the velocity predominantly a function of the relative dielectric permittivity, as given in Equation 1. The velocity of GPR energy in air is generally assumed to equal the speed of light in a vacuum, c = 300 mm/ns. A portion of the transmitted pulse is reflected from boundaries between electrically dissimilar materials, for which the reflection coefficient, R, representing the proportion if the incident energy that is reflected, is given by Equation 2. The remaining energy is transmitted past the boundary into the underlying material and propagates at a velocity that depends on the relative dielectric permittivity of that layer. The variation in the reflected energy amplitude received by the antenna is recorded as a waveform over a range in time, typically measured in nanoseconds. Layer thickness is estimated as the product of the layer velocity and one half of the measured two-way travel time delay between reflections occurring at the surface and bottom of a given layer, as shown in Equation 3. GPR data is normally collected according to a spatial frequency that depends on the transmission frequency and the distance interval selected between adjacent waveforms. A 10 cm interval between waveforms provides 10,000 thickness estimates per kilometer, enabling a very strong statistical measure of the thickness variability along a pavement section.

$$V = c/\sqrt{e_r} \tag{1}$$

$$R = \frac{\sqrt{e_{r2}} - \sqrt{e_{r1}}}{\sqrt{e_{r2}} + \sqrt{e_{r1}}} = \frac{V_2 - V_1}{V_2 + V_1} \tag{2}$$

$$h = Vt/2 \tag{3}$$

Air-coupled antennae provide the advantage of producing a reflection from the asphalt concrete surface. Since the relative dielectric permittivity of air is equal to 1 by definition, the surface reflection amplitude can provide an estimate of the near surface dielectric permittivity and the layer velocity according to Equation 2. This estimate requires a measure of the total energy reflected from a metallic reflector placed at the same operating distance from the antenna as a measure of the incident energy projected onto the asphalt surface. The surface dielectric estimate may not provide an adequate estimate of the average velocity throughout the total asphalt layer on older deteriorated pavements which consist of multiple overlays exhibiting different thickness, levels of deterioration, moisture and de-icing salt accumulation. Variable lift thickness compounds the problem associated with the different dielectric material properties to result in a variable average velocity throughout the total layer. In such cases, calibration of the velocity relies on determining the average layer velocity from the measured two-way travel time between the reflections measured at the surface and bottom of the total composite asphalt layer and the known total thickness values obtained from drilled core specimens. The resulting thickness estimates should provide accurate results on average, but should also result in some significant errors at certain locations where the composite bound asphalt layer differs electrically from the sampled core locations.

RESEARCH APPROACH

It was hypothesized that asphalt concrete thickness variability along a pavement section could play a significant role in affecting the quality of FDR aggregates and the resulting stabilized base materials. Expanded asphalt FDR was used to rehabilitate two different pavement sections in New Brunswick, located on Route 335 near Caraquet and on Route 790 near Lepreau to compare the effect of two different production control methodologies on the quality of the FDR aggregate.

Route 335

The Route 335 section was constructed using conventional control methods, relying on test pits to establish the average pavement thickness and to provide samples of asphalt concrete and granular materials for developing the 0.67 RAP/total depth ratio developed in the Job Mix Formula (JMF) to meet the NBDOT specified gradation limits. The JMF for the Route 355 section is listed in Table 4, and consisted of proportions by mass of 65.33% RAP, 32.18% granular materials, and 2.50% of expanded PG 58-28 Asphalt Cement (AC). The pulverization depth was adjusted using a retroactive control method that relied on the thickness of the asphalt concrete at the edge of the pulverized material behind the process to estimate what the asphalt thickness might be in front of the process. Initially, the pulverization depth for the recycling machine was set to the design depth, but required immediate adjustment when it was noted that the asphalt concrete thickness did not match the average thickness as determined by the test pits. Subsequent adjustments occurred at approximately 10 m intervals over the next 200 m for asphalt concrete thickness that varied between 40 to 180 mm. The as-built blend ratio varied due to the changes from thin asphalt concrete behind the recycling machine to thick asphalt concrete in front of the recycling machine and vice versa. Six different locations were sampled to obtain a range of blend ratios occurring between 0.32 and 0.90.

Values for the dry ITS, soaked ITS, and TSR for the Route 335 JMF mixture were 590.8 kPa, 512.7 kPa, and 0.868, respectively. These values exceeded the corresponding NBDOT minimum specified values of 300 kPa, 150 kPa, and 0.5.

Route 790 near Lepreau

The Route 790 section was surveyed with GPR using a 2 GHz horn antenna prior to test pit sampling in order to select calibration locations where the total asphalt concrete layer appeared to vary in thickness. The asphalt thickness obtained from the various test pits was used to determine the average GPR signal velocity in the asphalt concrete layer, which was used to estimate the thickness throughout the pavement section at 10 cm intervals. Figure 1 shows the variation in the GPR based total asphalt concrete thickness estimated along the northbound lane of the Route 790 test section and the suggested pulverization depths for various subsections, while Figure 2 shows similar results for the southbound lane.

Data from this extensive survey were used to delineate subsections of relatively consistent thickness and a corresponding pulverization depth required to achieve the blend ratio of RAP and granular material as designed for the JMF, listed in Table 5, which consisted of proportions by mass of 72.96% RAP, 24.35% granular materials, 0.49% Portland cement, and 2.20% of expanded PG 58-28 AC. Samples of the FDR aggregate materials were obtained from six different locations after initial pulverization where the estimated asphalt concrete thickness differed, but the pulverization depth selected in the subsection provided the nominal 0.746 RAP/total depth ratio developed in the JMF. If the GPR estimates were accurate, the similar blend ratios obtained by the proactive control method utilizing the combinations of different asphalt concrete thickness estimates and corresponding different pulverization depth were expected to provide improved consistency in the quality of the FDR aggregate in comparison to the variable blend ratios that resulted from the retroactive control method on the Route 335 project.

Values for the dry ITS, soaked ITS, and TSR for the Route 790 JMF mixture were 332.4 kPa, 290.6 kPa, and 0.874, respectively. These values exceeded the corresponding NBDOT minimum specified values of 300 kPa, 150 kPa, and 0.5.

LAB TESTING PROGRAM

A series of laboratory tests were conducted on all of the Route 335 and Route 790 FDR aggregate samples to evaluate the variability of the results as a function of the HMA/granular blend ratio. The tests included particle gradation, California Bearing Ratio, optimum moisture content and optimum density as indicators of the expected consistency and performance of the materials for use either as an un-stabilized granular base or as an expanded asphalt stabilized base. Variability observed in the test results for the Route 335 specimens would indicate the influence of the blend ratio on these factors, while the lack of variability observed for the Route 790 specimens would indicate the feasibility of a GPR-based proactive control method for providing consistency of the FDR aggregate materials.

Gradation

The gradation of the FDR aggregate materials can have a large effect on the properties of the recycled base. Density, modulus, strength, and moisture susceptibility are all affected by the gradation of the material so it is important to properly control, and optimize the size distribution of the pulverized material. Fuller and Thompson (1907) developed a grading curve shown in Equation 4, which was later modified by the Federal Highways Administration in the 1960s to describe the optimum size distribution to minimize voids in the mineral aggregate (VMA) of hot mix asphalt concrete (10). Gradations closely matching this theoretical curve should maximize particle packing and minimize void space among the aggregate particles, thereby maximizing compaction and density. A higher density improves inter-particulate contact and load transfer through the base, providing a higher modulus, reductions in the vertical stresses exerted on the subgrade, and a reduction in the wearing surface thickness needed to protect the base layer itself.

$$P = 100(d/D)^n \tag{4}$$

where:

P	= Percentage by mass of material passing sieve opening size ‘d’ ;
d	= Selected sieve size;
D	= Maximum particle size; and,
n	= grading coefficient, originally equal to 0.5, now usually taken as 0.45 as per FWHA adapation.

The goal in blending pulverized materials during the FDR process should therefore be to match this size distribution as closely as possible. This can be complicated by several factors including (11):

- degree of oxidation of the reclaimed material;
- thickness of existing pavement;
- original asphalt mix;
- geometry and amount of cracking;
- conditions of bonding between any overlays;
- equipment; and,
- temperature of asphalt during recycling process

Optimum Density and Moisture Content

The optimum moisture content to provide maximum dry density under a standard Proctor compaction effort was determined for samples prepared at various moisture contents according to ASTM D698 (12). All samples were dried to constant mass over a period of several days at a temperature of 40 °C in order to prevent melting the asphalt binder contained within the RAP and the loss of fines which would become bound within the fluid asphalt cement. Field compaction to values approaching the optimum density will result in the best performance for the stabilized base layer. If the material quality varies along the section, different levels of compaction effort may be required to maximize the density, and the maximum dry density may vary along the section. This implies that different levels of modulus, strength, and performance might be observed along the section and could be responsible for the variable as-built strength results observed in Table 3.

Theoretical Maximum Density

The Theoretical Maximum Density (TMD) differs from the optimum dry density because this is the density of material containing zero air voids between the aggregate particles (10). While it is not possible to achieve this density during field compaction, this value provides the basis for determining the volume of air voids contained within the compacted mix. The maximum theoretical densities of the FDR aggregate samples were determined according to ASTM D2041/D2041M-11 (13), again using a procedure of drying the FDR aggregates to constant mass over a period of several days at 40 °C.

California Bearing Ratio

The California Bearing Ratio (CBR) as determined according to ASTM D1883 (14) is a quick method of evaluating the relative strength of a material. Though the results obtained are empirical, correlations between CBR values and performance are commonly used in engineering design practice. A comparison of CBR values between the Route 335 and Route 790 projects provides some indication of the differences in the materials and a measure of the suitability of the aggregates for use as an un-stabilized base, while variability among specimens within each project is a measure of the consistency and the effect of any blend ratio differences. The CBR values might also be considered to imply the quality of the stabilized base as some consideration of the inter-particulate contact and ability to transfer load.

RESULTS

Samples from six locations exhibiting different combinations of measured total asphalt concrete and pulverization depth that resulted in variable thickness blend ratios were obtained at different location along the Route 335 project. The samples were tested to evaluate the effect of the variable thickness blend ratio on the consistency of the material properties. Samples from six locations exhibiting different combinations of total asphalt concrete thickness and total pulverization depth to maintain a nominally constant thickness blend ratio were obtained at different locations along the Route 790 project. These samples were tested to evaluate the feasibility of using the GPR based thickness estimates for establishing an appropriate pulverization depth in order to achieve a consistent blend ratio and consistent material properties. The mean value and standard deviation was calculated for each material test in order to compare variability of the materials at these different locations.

Sieve analyses were performed on the FDR aggregates obtained after the first pulverization stage from both projects to evaluate the consistency of the size distribution of the different samples. Table 6 lists the results of each sieve analysis, fineness modulus, and the predicted RAP to total pulverization depth ratio for the six samples obtained from Route 335. A high degree of variability is observed among the Route 335 samples with increasing contributions of fines occurring with higher contributions of granular materials in the FDR aggregate blend. The as-built FDR aggregates produced from the Route 335 section generally fell between the specification limits specified by NBDOT and recommended by Wirtgen (6) for the intermediate sizes between 0.600 to 5 mm, but exhibited excessive amounts of coarse particles and insufficient fines to provide the desired aggregate structure according to the optimal 0.45-power grading curve.

Table 7 lists the results of each sieve analysis, fineness modulus, and the predicted RAP to total pulverization depth ratio for the six samples obtained from Route 790. Very little variability is observed among the Route 790 samples, indicating a highly consistent material where the blend ratio was maintained. However, some discrepancy is observed between the as-built FDR aggregates the design JMF size distributions, but this can be attributed to the lack of fines that are expected to be generated during the second pulverization stage of the project. The as-built Route 790 FDR aggregates appear to significantly lack in fines content for sizes below the 4.75 mm sieve and exhibits excessive coarse materials above the 9.5 mm size, falling outside of the NBDOT specifications (8) and those recommended by Wirtgen (6). This discrepancy may be due to the difference between the post-extracted residual aggregate size distribution used in evaluating the JMF design blend against the specification limits, whereas the asphalt cement was not extracted from the field samples. The RAP particles were considered as conglomerate particles for which the cold recycling process would be unlikely to liberate fines bound by the in-situ asphalt cement. However, since the field specimens were collected after the first pulverization stage, less fines might be expected in the measured gradations than what should occur after the stabilization process is completed. It should be noted that with an expected increase of 2-3 percent fines during the second stage pulverization stage, the materials are likely to meet the gradation limits recommended by Wirtgen (6). Future research should examine the generation of fines in successive pulverization stages of un-stabilized materials in order to better match the JMF design blend to the as-built FDR aggregate gradation. The ideal gradation should be developed to target optimum particle packing according to the 0.45-power curve and could require the use of crusher dust corrective aggregate to better approximate the shape of the ideal gradation curve. Such a gradation would be expected to provide a stronger, more dense, aggregate structure that should improve the strength of the stabilized base.

A summary of the other aggregate test results are included in Tables 8 and 9, for Route 335 and Route 790 FDR aggregate specimens, respectively. Tests performed include California Bearing Ratio (CBR), Maximum Relative Theoretical Density (MTD), and Standard Proctor tests to determine maximum dry density and optimum moisture content. As with the sieve analysis, the results from this set of tests showed variability in the Route 335 materials and consistency in the Route 790 materials.

A high quality base material is one with CBR in the range of 70 to 90, and a suitable subbase material as one with a CBR of around 20 (15). Results from the Route 790 tests indicated that the material is unsuitable for use as a base material without stabilization with CBR values below 5. Similarly poor CBR strength results were also observed for the Route 335 materials, but exhibited a larger range of CBR values between 7 – 18. It is likely that the lack of fines observed for both the Route 790 and the Route 335 specimens can explain the low strengths resulting from the CBR tests. The Route 335 specimens better approached the recommended gradation limits provided by Wirtgen than did the Route 790 materials, and a similar trend is reflected in the CBR values. It is expected that corrective aggregate added to the as-built FDR aggregate blend to approach the 0.45 power Fuller Thompson curve would greatly improve the CBR results and better enabling this material for use as an unstabilized base. An additional two percent of fines generated during stabilization may also improve the strength of the aggregate structure, but the addition of a corrective aggregate might be warranted to provide a gradation within the midrange of the specification limits.

The comparatively high variability observed in the test results for the Route 335 samples indicates that the retroactive approach of pulverization depth control may cause inconsistency in the material properties of the FDR aggregate. In cases where the thickness changes drastically over the short distance spanned by the length of the recycling machine, the thickness blend ratio was observed to vary significantly with a standard deviation of 0.23. The GPR based thickness estimates obtained from the Route 790 project were used as the basis of establishing the pulverization depth for various subsections. The low variability observed in the test results for the Route 790 samples indicated that the GPR based thickness estimates provided sufficiently accurate estimates of the in-situ thickness to enable a proactive method of controlling the pulverization depth and maintaining consistency in the material properties for a given thickness blend ratio. Compared to applying a single pulverization depth for the entire project, sub-sectioning the project into regions of different pulverization depths based on the average subsection asphalt concrete thickness provided an overall decrease in the standard deviation of the thickness blend ratio. However, given the thickness variability that occurs within the subsections, the best approach to achieve a consistent blend ratio throughout the project might be to collect the geospatial location of the GPR data and use this information to control a variable pulverization depth on the fly via a machine guidance system. High accuracy real time kinematic GPS data collected during the GPR survey could be used to establish the pulverization depth over short distances of 1-2 meters. This data should be easily incorporated into a machine control system to set the pulverization depth on the fly, greatly simplifying the construction process and achieving consistent material properties throughout the pavement section. The authors believe that this material consistency would greatly reduce the occurrence of variable strength parameters along the section by providing uniform gradation, optimum moisture content and density, compaction effort required, and optimum binder content required.

CONCLUSIONS & RECOMMENDATIONS

The pulverization depth was controlled on two expanded asphalt FDR pavement test sections using either a retroactive method or a proactive method. The retroactive method used on Route 335 tests section entailed measuring the total in-situ asphalt concrete thickness adjacent to pulverized materials behind the recycling machine as it progressed. These thickness values were used to estimate the unknown asphalt concrete thickness ahead of the recycling machine in order to set an appropriate pulverization depth targeting the blend ratio developed in the JMF. The proactive approach used on the Route 790 test section entailed conducting a GPR survey to estimate the pavement thickness versus distance along the test section. Ideally, the GPR based thicknesses estimates would provide foreknowledge of the expected asphalt concrete thickness in establishing appropriate pulverization depths for each specific location. The Route 790 section was sub-sectioned into regions of approximately similar average thickness with corresponding pulverization depths to meet the blend ratio developed in the JMF.

Large variations in the retroactively controlled Route 335 blend ratio that were observed during construction resulted in significant variability in the gradation of the FDR aggregates that were produced; in the aggregate strength as measured by the CBR; and in the optimum moisture content and density. A retroactive control approach and the resulting variability is likely to have been a contributing factor in the observed variability in the dry and soaked ITS and the TSR values in this and previous projects. Much less sample variability was observed for the Route

790 specimens, which were specifically selected at locations where the estimated asphalt thickness and the prescribed pulverization depth provided approximately the same thickness blend ratio. This substantial decrease in variability may indicate that the development of this proactive control method into a continuous on-the-fly variable pulverization depth technique may provide extremely consistent material properties and substantially decrease variability of the stabilized strength parameters within a given section. Such a method might rely on the collection of spatial data with the GPR thickness survey to provide inputs for a machine control system of controlling the recycling machine pulverization depth.

It is recommended that future JMF design approaches utilize the conglomerate particles of manufactured RAP in developing the design gradation to maximize particle packing and material density. In the case of the Route 335 and Route 790 projects, with a two percent expected increase in dust content due to the secondary stabilization stage, this approach may require the use of corrective crusher dust aggregates to approach the midrange of the specified gradation limits.

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Table 1 – Recommended gradation limits for expanded asphalt FDR materials (6).

Sieve Size (mm)	Percent Passing (%)	
	Ideal Range	Less Suitable Range
50	100	
37.5	87 – 100	
26.5	77 – 100	100
19.0	66 – 99	99 – 100
13.2	67 – 87	87 – 100
9.6	49 – 74	74 – 100
6.7	40 – 62	62 – 100
4.75	35 – 56	56 – 95
2.36	25 – 42	42 – 78
1.18	18 – 33	33 – 65
0.600	14 – 28	28 – 54
0.425	12 – 26	26 – 50
0.300	10 – 24	24 – 43
0.150	7 – 17	17 – 30
0.075	4 – 10	10 – 20

Table 2 – Strength Specifications for Expanded Asphalt Stabilized FDR Base.

Year	Minimum Dry ITS (kPa)	Minimum Soaked ITS (kPa)	Tensile Strength Ratio
2011	300	150	0.50
2012	225	100	0.50

Table 3 – Sample of strength parameters from previous FDR projects in New Brunswick.

Location	Dry ITS (kPa)	Soaked ITS (kPa)	TSR
Churchland	427-463	392-422	0.92-0.91
Route 313	324-398	182-302	0.56-0.76
Route 114	77-99	205-237	2.70-2.40
Route 335	320-626	127-270	0.40-0.43
Route 790	168-274	113-194	0.67-0.71
Route 320	153-186	226-278	1.48-1.50
Hillcrest	280-368	158-193	0.56-0.52
Lakeside	460	332	0.72
St. Marys	300	103	0.34

Table 4 – Material gradations used to develop Route 335 section JMF.

Sieve Size (mm)	Granular Material	RAP	Portland Cement	Blended Job Mix Formula	NBDOT Minimum	NBDOT Maximum
50.0	100.0	100.0		100.0	100	100
25.0	96.5	100.0		98.9		
19.0	92.5	100.0		97.5		
16.0	88.7	100.0		96.3		
13.2	85.2	97.8		93.7		
9.50	76.5	91.1		86.4		
4.75	65.8	68.6		68.0	45	70
2.36	57.5	49.8		52.8		
1.18	50.9	35.8		41.4		
0.630	43.0	24.3		31.2		
0.300	19.0	16.8		18.3		
0.150	10.1	11.2		11.7		
0.075	7.6	8.2		8.9	5	20
% AC		5.43		6.14		

Table 5 – Material gradations used to develop Lepreau section JMF.

Sieve Size (mm)	Granular Material	RAP	Portland Cement	Blended Job Mix Formula	NBDOT Minimum	NBDOT Maximum
50.0	100.0	100.0	100.0	100.0	100	100
25.0	90.4	100.0	100.0	97.6		
19.0	85.0	100.0	100.0	96.3		
16.0	79.9	100.0	100.0	95.0		
13.2	74.7	92.3	100.0	88.0		
9.50	66.0	83.2	100.0	79.2		
4.75	51.6	57.1	100.0	56.4	45	70
2.36	41.0	41.1	100.0	42.0		
1.18	30.5	27.4	100.0	29.3		
0.630	20.8	20.3	100.0	21.6		
0.300	12.5	14.1	100.0	15.0		
0.150	8.1	9.2	100.0	10.2		
0.075	6.4	6.0	100.0	7.5	5	20
%AC		5.68		6.44		

Table 6 – Gradation Analysis of Route 335 Specimens

Sieve Opening (mm)	#1	#2	#3	#4	#5	#6	Mean	Standard Deviation
50	100	100	100	100	100	100	100	0.0
20	94.63	97.02	99.88	97.95	98.82	98.00	97.71	1.79
14	87.81	93.83	94.31	94.73	95.34	94.29	93.39	2.78
10	75.23	88.12	84.37	88.83	87.80	88.28	85.44	5.25
5	47.43	74.88	57.36	71.14	67.27	71.66	64.96	10.51
2.5	27.16	59.49	36.87	50.36	45.70	55.66	45.87	12.11
1.25	15.97	46.86	21.05	32.57	29.01	41.29	31.12	11.74
0.63	9.87	33.58	11.39	19.04	16.74	26.62	19.54	9.11
0.315	4.00	15.13	4.00	7.16	6.13	10.70	7.85	4.3
0.16	1.18	5.04	1.12	2.25	2.06	3.56	2.54	1.51
0.075	0.053	0.22	0.073	0.14	0.13	0.21	0.14	0.07
FM	7.37	5.86	6.90	6.36	6.51	6.10	6.51	0.55
RAP/total depth	0.90	0.31	0.80	0.43	0.50	0.57	0.59	0.23

Table 7 – Gradation Analysis of Route 790 Specimens

Sieve Opening (mm)	#1	#2	#3	#4	#5	#6	Mean	Standard Deviation
50	100	100	100	100	100	100	100	0.0
25	100	98.3	99.2	100	100	100	99.6	0.7
19	96.5	95.6	95.8	98.3	99.3	97.4	97.1	1.4
15.9	92.8	92.7	93.1	94.1	95.7	93.4	93.6	1.1
12.7	84.7	83.5	84.9	85.3	90.1	84.9	85.5	2.3
9.5	74.8	74.8	75.8	75.4	81.0	72.5	75.7	2.8
4.75	48.9	50.7	51.6	50.9	54.0	43.3	49.9	3.6
2.5	27.4	33.0	31.7	31.0	31.2	25.5	30.0	2.9
1.25	14.3	19.2	16.9	17.8	16.8	12.8	16.3	2.4
0.63	7.2	9.9	8.0	10.0	8.8	6.2	8.3	1.5
0.315	3.3	4.2	3.3	5.7	4.5	3.0	4.0	1.0
0.16	1.4	1.5	1.2	3.4	2.4	1.5	1.9	0.8
0.075	0.8	0.8	0.7	2.4	1.5	1.0	1.2	0.6
FM	5.26	5.11	5.16	5.08	5.02	5.38	5.17	0.13
RAP/total depth	0.74	0.74	0.75	0.76	0.76	0.76	0.75	0.01

Table 8 – Summary of Route 335 FDR aggregate strength and density results.

Location	Blend Ratio	CBR	MTD	Optimum M/C	Optimum Density (kg/m^3)
7+725	0.90	7.0	2.476	8.25%	1712
7+790	0.80	7.0	2.511	7.63%	1775
7+075	0.57	11.0	2.532	9.75%	1761
7+750	0.50	18.0	2.492	9.75%	1818
7+772	0.43	11.0	2.512	9.63%	1778
7+703	0.31	12.0	2.514	10.75%	1955
Mean	0.59	11.0	2.506	9.29%	1800
Standard Deviation	0.22	4.1	0.019	1.14%	83.3

Table 9 – Summary of Route 790 FDR aggregate strength and density results.

Location	Blend Ratio	CBR	MTD	Optimum M/C	Optimum Density (kg/m^3)
1+769	0.74	3.6	2.491	5.59%	1760
2+185	0.74	3.2	2.504	5.85%	1796
2+550	0.75	3.2	2.509	5.30%	1802
3+128	0.76	4.2	2.505	4.58%	1859
3+446	0.76	2.8	2.486	6.70%	1818
4+040	0.76	2.9	2.487	5.37%	1768
Mean	0.75	3.3	2.497	5.57%	1800
Standard Deviation	0.01	0.5	0.010	0.70%	35.9

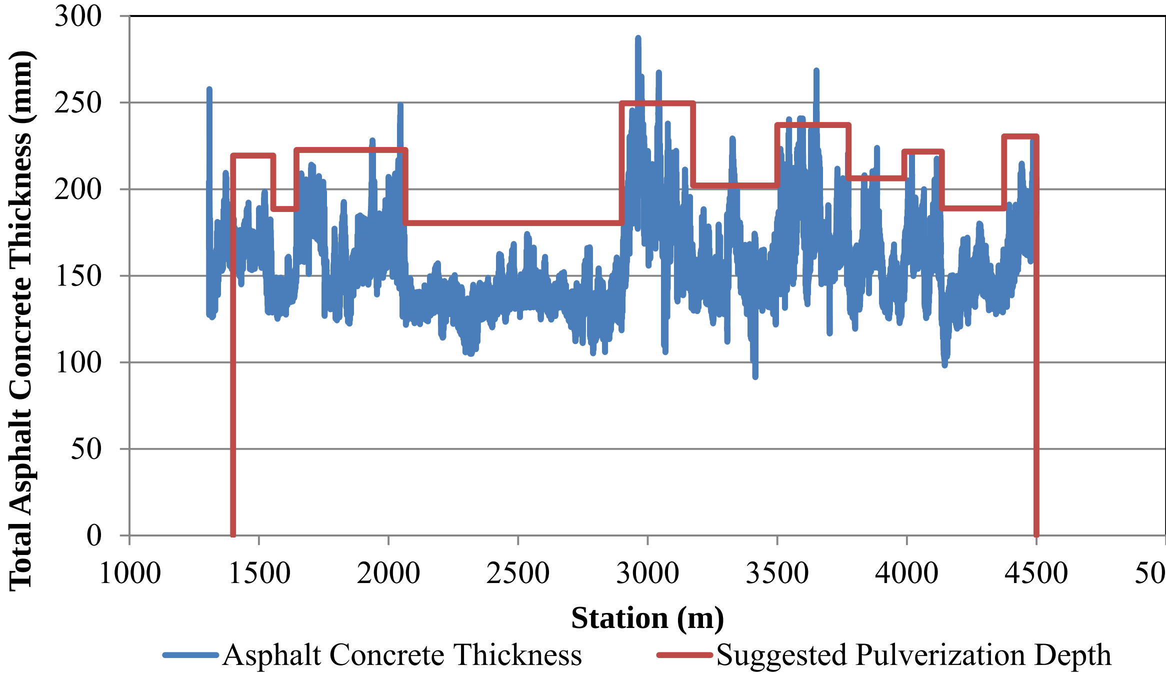

Figure 1 – GPR based total asphalt concrete thickness and suggested pulverization depth for Route 790 northbound lane.

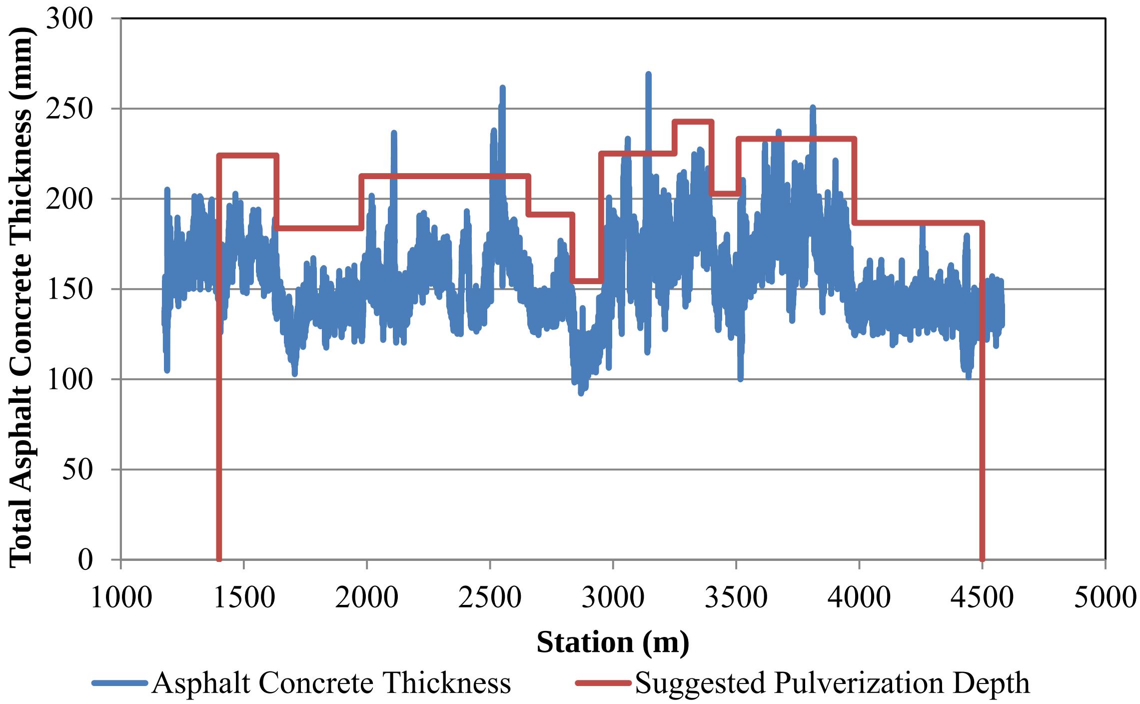

Figure 2 – GPR based total asphalt concrete thickness and suggested pulverization depth for Route 790 southbound lane.