Forensic Investigation of Cracking in a Portland Cement Stabilized Full Depth Recycled Pavement

Christopher L. Barnes, Assistant Professor
Dept. of Civil and Resource Engineering, Dalhousie University

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ABSTRACT

The Point Michaud Beach Road was the first Portland cement stabilized Full Depth Recycled (PC-FDR) asphalt pavement project conducted in Nova Scotia. Several modes of cracking have developed on the pavement surface over the past two years. Excessive transverse shrinkage cracks have developed across each stabilized mat. These cracks were expected, but not to the extent that have been observed, despite the use of a micro-cracking construction process that was used to mitigate shrinkage effects. Multiple parallel longitudinal cracks along the pavement centreline and shoulder edge cracking have appeared along the majority of the pavement section. An impulse-response survey and a coring investigation were conducted on the pavement section in 2009 in order to evaluate the effects of this distress on the structural properties of the road. The extent of damage across the section at various locations was determined to explore a rationale for the cause of the cracking. It was concluded that slab overloading during the micro-cracking process probably contributed to the development of the excessive structural cracking, while high cement content contributed to excessive shrinkage cracking. Stress analysis of loads applied during the micro-cracking process indicated that the maximum slab stress in two load configurations exceeded the PC-FDR modulus of rupture. Prevention of micro-cracking roller patterns from overlapping the PC-FDR and adjacent shoulder and/or unbound pulverized materials, and overlap between lanes on the pavement crown, were recommended to avoid rupturing the stabilized base layer.

Introduction

The Pt. Michaud Beach Road was the first Portland Cement stabilized Full Depth Recycled (PC-FDR) pavement project completed in Nova Scotia, in November of 2007. PC-FDR is a well established and viable rehabilitation method for heavily damaged asphalt concrete pavements that has been used for over 25 years in the United States (1). The full depth pulverization of the existing asphalt concrete pavement and upper region of the supporting granular base materials removes the potential for reflective cracking in overlays from pre-existing damage within the bound pavement structure and reuses quality aggregates from the existing pavement. As a general result, FDR methods incur significant savings with respect to sustainability and monetary cost by reducing the demand on virgin aggregate supplies and the trucking costs. PC-FDR has the added benefit of providing a much stiffer base layer than asphalt-based FDR methods, which spreads the applied out over the subgrade and reduces it's susceptibility to rutting. The stiff base reduces tensile strains in asphalt concrete overlays and improves the resulting fatigue life (2). This behaviour as a rigid slab-on-grade makes PC-FDR suitable for reconstructing a strong pavement system over weak subgrade materials. Many rural pavement failures in Nova Scotia are the result of subgrade and granular base failures that continue to propagate up through attempted overlay repairs, so PC-FDR can provide an appropriate solution to ongoing maintenance demands for such highways.

Contrary to expectations and the reported historical success of the method, the Pt. Michaud Beach Road, and some of the other PC-FDR projects that have been completed in Nova Scotia in 2008-2009, have exhibited moderate to severe centreline cracking, as shown in Figure 1, and both frequent transverse and shoulder edge cracking. Inspection of the cracked centreline areas revealed that the crown was actually depressed by 4-6 mm in many locations, as shown in Figure 2. As a result, the Nova Scotia Department of Transportation and Infrastructure Renewal are reviewing the completed projects in order to determine the cause of the cracking and the feasibility of continuing the program.

PC-FDR Methodology in Nova Scotia

The current practice in Nova Scotia for PC-FDR projects has been to conduct some pre-engineering materials sampling on candidate pavement sections in order to evaluate the thickness and gradation of the existing asphalt concrete and granular base materials. Ten inch diameter asphalt concrete samples are typically either saw-cut or drilled from the existing pavement structure, followed by extraction of granular base materials. The general design approach for PC-FDR has been to design the cement content based on equal proportions of these asphalt concrete and granular base field samples, after a 'pulverized' sample has been manufactured using a laboratory crusher. The optimum moisture content to reach maximum density for the blended manufactured materials is determined along with the optimum cement content to attain an unconfined compressive strength of 2 MPa after 7 days of moist curing.

During construction, the existing pavement is pulverized to a nominal depth of 200 mm, graded to achieve the desired 2% cross slope, and then compacted to carry local traffic until the materials are stabilized at a later time. Dry Portland cement is placed at the desired addition rate (according to the lab mix design) on the road surface using a drop spreader. The materials are then re-pulverized to combine the cement with sufficient moisture added to achieve the maximum density as determined by the lab mix design. The stabilized materials are then compacted from the bottom up using a padfoot roller, re-graded to achieve the design cross slope, and compacted to within 98% Proctor using a vibratory steel drum roller. After compaction, the PC-FDR mat is to be kept constantly moist for seven days, using water spray trucks.

Approximately 24-48 hours after initial placement of the stabilized materials, the vibratory steel drum roller is used to micro-crack the mat. Three to four complete passes of the roller are made at maximum amplitude in order to reduce the initial modulus by approximately forty percent, developing a network of micro-cracks which act to distribute shrinkage strains uniformly across the mat. This process is used to help mitigate against multiple large transverse cracks which may occur at higher cement contents as the material cures and dries. Following completion of the curing regime, an asphalt concrete layer is typically placed on the PC-FDR mat to provide a smooth wearing surface and additional structural capacity.

Structural Testing of Pt. Michaud Beach Road

The design cement content used for the Pt. Michaud Beach Road project was 6%. Current practice in Nova Scotia for both Portland cement and asphalt stabilized FDR projects is an initial pulverization depth of 200 mm, followed by a stabilized depth of approximately 150 mm. A ground penetrating radar survey of the pavement thickness indicated a mean asphalt concrete thickness of 132 mm with a standard deviation of 29 mm, while the mean granular base layer thickness was 297 mm with a standard deviation of 52 mm. This resulted in an average ratio of 0.65 for asphalt concrete to granular base within the pulverized materials, with a range of 0.50 to 0.80 based on the standard deviation of the asphalt concrete layer.

A falling weight deflectometer and ground penetrating radar system were used to conduct an outer wheel path deflection and thickness survey of the Pt. Michaud Beach Road. The measured deflections and layer thicknesses were used to compute the structural number, effective pavement moduli and subgrade moduli prior to and following the pavement reconstruction, based on the AASHTO 1993 Pavement Design Guide analysis procedure. The PC-FDR process resulted in a large immediate improvement of the mean effective pavement modulus, E_P, from 2,480 MPa to 45,720 MPa. The mean effective structural number, SN_{Eff}, of the pavement improved from 5.09 to 13.78. Full backcalculation of the layer moduli indicated that the mean elastic modulus of the PC-FDR was 7465 MPa with a standard deviation of 4684 MPa (3). Follow-up deflection and GPR testing were conducted in 2009 after the centerline and shoulder edge cracking appeared. This testing indicated an effective pavement modulus of 45070 MPa and an effective structural number of 13.67. It is apparent that the structural capacity of the wheel path regions of the pavement system remains largely unaffected by the damage processes which have lead to the observed centreline and shoulder edge cracking. However, the level of cracking that has occurred has created significant public outcry and without remediation will create strength issues due to moisture intrusion into the underlying pavement structure.

Evaluation of Damage Mechanisms

The transverse cracks observed resulted from drying shrinkage of the cement stabilized layer, reflecting up through the asphalt overlay. Consultation with the Portland Cement Association revealed that the 6% cement content appeared to be relatively high compared to other similar projects. It is likely that the higher cement content contributed to the excessive degree of shrinkage cracking observed on the Point Michaud Beach Road project. Subsequent PC-FDR projects in Nova Scotia have used approximately 4% Portland cement which has appeared to result in a lower number and severity of transverse cracking.

Various hypotheses were proposed as the probable cause of the observed centreline and shoulder cracking, including variable cement application rates, a potential overlap of cement application at the centreline, and inconsistent mixing of the cement and asphalt/granular material across the width of the pulverizer. However, the localized, dense, and severe nature of the centreline cracking seems to indicate damage due to focused mechanical loading or impact, making it unlikely that the cause is of a general materials-based issue. Impulse-response testing was conducted across the pavement section from shoulder to shoulder at four locations in order to assess the lateral distribution of damage.

Impulse response testing was developed from a vibration method of pile integrity testing (4) and has been used for testing plate-like concrete structures such as slabs, walls, and large cylindrical structures (5). Testing involves striking the surface of the slab with a handheld 1-kg hammer, instrumented with a load cell to record the time history of an impact force. A 4.5 Hz geophone can be used to measure the velocity response versus time of the pavement surface immediately adjacent to the impact point. Both time records are processed using a Fast Fourier Transform into the frequency domain. The velocity spectrum is divided by the force spectrum to obtain a transfer function indicating the mobility of the structural element.

The slope of the mobility spectrum below 50 to 100 Hz defines the compliance or flexibility of the system around the impact point. The inverse of this flexibility is the dynamic stiffness of the pavement system near the surface, which are a function of the slab thickness, modulus and the underlying support conditions. The mean and peak values of mobility between 100 to 800 Hz are generally used to evaluate the loss of support of the uppermost layer. Damage in the PC-FDR layer will decrease support conditions for the asphalt layer. For completely debonded asphalt layers, an increase in the measured mobility occurs, corresponding to the asphalt plate thickness and material properties instead of the composite asphalt and PC-FDR layer thickness and properties. Furthermore, the system becomes less damped due to the decreased structural rigidity, causing larger fluctuations in the mobility plot and a higher peak/mean mobility ratio.

Figure 3 shows a photograph of a section on the Pt. Michaud Beach Road where no cracking was observed on the pavement surface. Impulse-response tests were conducted at 0.2 m intervals to determine the variation in dynamic stiffness and mean mobility across the width of the paved surface. Reduced values of dynamic stiffness, shown as 'hotter' red-yellow colour, and higher values of mobility, shown as brighter blue-white colour, can be observed near the outer edges of the pavement surface. Since no visible evidence of damage was observed, it was assumed that these reductions corresponded to reductions in structural rigidity associated with the less supported free edge of a rigid pavement slab. Furthermore, the dynamic stiffness and mobility appeared to be consistent throughout the interior of the pavement width, including the region of the crown where cracking damage was observed elsewhere. This might provide some evidence against the idea that the pavement was cemented twice near the centreline during the construction process. This also appears to indicate that the softened behaviour of the free edge seems to converge to that of an interior loaded plate within 35-40 cm of the plate edge for this pavement.

Figure 4 shows a photograph of a typical damaged section where only heavy centreline cracking was observed. Similar reductions in stiffness and increases in mobility can be observed near the pavement edges. However, a significant decrease in the stiffness can be observed near the 0.25 m distance from the pavement edge (foreground) and near the centreline of the pavement. A slight increase in mobility can be observed at the 0.25 m distance from the pavement edge, while a large increase in mobility can be observed near the centreline. The changes in stiffness and mobility are readily associated with the heavy cracking observed near the centreline, but indicate that the damage likely extends beyond the asphalt layer into the PC-FDR. If only the asphalt layer were affected, a subtle change in the measured stiffness might be expected due to the higher PC-FDR modulus and thickness. Furthermore, the increase in mobility indicates that the PC-FDR layer below the asphalt layer is likely deteriorated, providing reduced support conditions. The testing also revealed the likely presence of a crack in the PC-FDR at the 0.25 m distance from the shoulder which has not yet propagated up through the asphalt concrete, resulting in decreased stiffness and a slight increase in mobility.

Figure 5 shows a photograph of a non-typical damaged section with both centreline and shoulder edge cracking, plus a wet, porous region that can be observed in the asphalt concrete. Shoulder edge cracking can be observed in the first 10-20 cm of asphalt concrete in the forefront of the photograph, but decreased stiffness and increased mobility can be clearly observed extending 1.10 and 0.90 m into the lane. It is likely that the shoulder area of the PC-FDR has become damaged but has not yet resulted in reflective cracking into the asphalt layer. Similar reductions in stiffness and increases in mobility can be observed near the observed centreline cracking in the asphalt concrete. The majority of the westbound lane exhibits decreases in stiffness and increases in mobility. It is likely that these effects are the result of moisture damage in both the asphalt concrete and the PC-FDR. A gravel road exits on to the Pt. Michaud Beach Road at this location and water was observed trickling down onto and the pavement.

One of the impulse-response test sections was cored to investigate the condition of the PC-FDR materials in the damaged centreline area and the apparently undamaged mid-lane area. The filled core locations can be observed to the right of the chalked test locations in Figure 4. Figure 6 shows a photograph of an intact PC-FDR core extracted from the outer wheel path near the impulse-response tests. A delamination near the interface between the asphalt concrete and the PC-FDR base can be observed which may have developed during the drilling operation. An intact sample of the PC-FDR base materials could not be obtained from the centreline region. The bit was able to core through the asphalt concrete layer, but refused to progress farther than 50-60 mm into the stabilized base. Upon extraction of the bit and the asphalt concrete core, it was observed that the stabilized materials were heavily damaged, resulting in loss of the matrix binding the coarse aggregate particle and fragmentation during drilling. Figure 7 shows the intact asphalt core and a sample of the PC-FDR fragments retrieved from the drilled hole. A second attempt at drilling the pavement in this location yielded the same result.

It was observed from the coring and impulse-response test results that the damage in the pavement system appeared to be contained to the shoulder edge and centreline regions. Shoulder cracking could be identified as a larger decrease in stiffness than the decreases observed due to a free slab edge. The interior regions of each lane appeared to be in sound condition and did not exhibit changes in dynamic stiffness or mobility as was observed in the damaged regions of the pavement. It may be assumed from the condition of these interior regions of the lanes that the damage mechanism was restricted to focused mechanical effort in the affected areas and not due to general material deficiencies.

Proposed Source of Cracking Damage

During construction of the PC-FDR mat, it was observed during the initial attempts at micro-cracking that the roller drum was not making full contact with the surface of the materials, but was focusing the load application near the extremities of the drum, due to some surface ruts. These ruts may have occurred due to a combination of poor nozzle spray patterns from the water trucks and the construction and local traffic driving on the compacted, but uncured, materials. The spray patterns of the curing water trucks were causing some erosion and loosening of the uncured PC-FDR surface which may have been tracked away by the traffic. Figure 8 shows the poor contact pattern of the initial micro-cracking effort on the PC-FDR mat, leaving the rutted areas uncracked. As a result, the rolling pattern was changed from a single pass being centred in the middle of each lane to adjacent passes spanning over the shoulder and the centreline edges of each lane.

It is hypothesized that the centerline cracking that is observed on many of the Nova Scotia PC-FDR projects is the direct result of excessive micro-cracking effort. The two different lanes of the highways tend to be reconstructed at different times, causing relative differences in their stiffness at early ages. The overlap of the roller drum onto the unstabilized shoulder and the unstabilized second mat can cause excessive force to be applied to the much stiffer stabilized mat. A typical vibratory steel drum roller used on these projects imparts 150,000 kN to the pavement at maximum amplitude used during the micro-cracking process. If half of the span of the roller is resting on the stabilized mat, then the relative stiffness of the PC-FDR compared to the shoulder and unstabilized pulverized materials would attract the majority of the applied load. Similarly, during initial compaction of the second stabilized mat, if the roller spans on to the first cured mat, then a significant portion of the applied load will be carried by the much stiffer mat and not by the material being compacted. Finally, if the roller spans onto the first mat during micro-cracking of the second mat, the roller would be directing the majority of the load onto the crown of the road, and subsequently to the stiffer first mat, if the crown fracture were to fracture.

The above hypothesis can be evaluated using Ioannides' modification of Westergaard's equations for maximum stress in loaded rigid plates supported on a Winkler foundation (6). The maximum stress in the plate under a circular loaded area adjacent to the free edge of the plate is given by Equation 1 in imperial units.

$$\sigma_e = \frac{3P(1+\nu)}{\pi h^2(3+\nu)}\left[\ln\left(\frac{Eh^3}{100ka^4}\right)+1.84-\frac{4\nu}{3}+\frac{1-\nu}{2}+1.18(1+2\nu)\left(\frac{a}{l}\right)\right] \quad (in\ psi) \qquad [1]$$

Where:

ν = Poisson's ratio of the plate;
E = Young's modulus of the plate (psi);
P = the applied load (lbs);
h = plate thickness (in);

k = modulus of subgrade reaction (lb/in^3);
a = radius of the circular contact area (in); and,
l = radius of relative stiffness (in)

Similarly, the maximum stress in the plate under a circular loaded area within the interior region of the plate is given by Equation 2 in imperial units.

$$\sigma_e = \frac{3P(1+v)}{2\pi h^2}\left[\ln(l/b) + 0.6159\right] \quad (in\ psi) \qquad [2]$$

Where:
b = a when a ≥ 1.724 h; or
b = $(1.6a^2 + h^2)^{0.5}$ – 0.765h when a < 1.724 h

The modulus of rupture of the PC-FDR can be estimated based on the unconfined compressive strength (7), as estimated by the elastic modulus (2) by Equation 3, also given in imperial units:

$$M_R = 7.24\sqrt{E/1200} \quad (in\ psi) \qquad [3]$$

If the maximum amplitude load of 150,000 kN was applied across a 150 mm wide strip over the 1.950 m length of the drum, this would result in a contact pressure of 513 kPa. Conversely, if this contact pressure was applied over a circular area with a radius of 75 mm on a subgrade with a conservative reaction modulus of 54 MN/m^3, the resulting applied force would be 9062 N (2036 lb) resulting in a maximum stress, σ_e = 1054 kPa, using Equation 1. Comparing this stress to the modulus of rupture, M_R = 1500 kPa, obtained using an average modulus of 7465 MPa and Equation 3, it is evident that the shoulder edge should not crack. However, with the roller spanning onto the relatively softer granular shoulder during micro-cracking, it might be assumed that the load experienced by the stabilized base is approximately doubled. This higher load results in a maximum stress, σ_e = 2107 kPa, which exceeds the modulus of rupture and would result in shoulder edge cracking. A similar evaluation might apply at the centreline during micro-cracking of the first mat when the roller spans onto the unstabilized second mat, resulting in edge cracking of the first mat. The most extreme case occurs when the second mat is being micro-cracked and the roller spans both mats. As a worst case, if the roller were positioned directly over the crown and the PC-FDR was completely rigid, the majority of the 150,000 kN applied load might be applied directly on the crown of the road. A more conservative approach might consider a quarter of the maximum load applied to this position (P = 37500 kN), resulting in a maximum stress for an interior loaded plate, σ_e = 1834 kPa, which exceeds the modulus of rupture and would crack the slab. It should be noted here that micro-cracking typically occurs 24 to 48 hours after placement, whereas the computed modulus of rupture is based on the 7-day unconfined compressive strength. As a result, the computed strengths likely underestimate the true value significantly.

A second issue which might compound the problems caused by the micro-cracking effort spanning adjacent slabs and the road crown is the pulverization of 200 mm of the pre-existing pavement and stabilization of only 150 mm. This might result in a disturbed and less compacted layer between the rigid PC-FDR plate and the underlying undisturbed granular materials. The effect of this layer would be a reduction in the level of support under the slab, via a reduced modulus of subgrade reaction, and a corresponding increase in the maximum stress under loading. It appears that this weaker supporting layer may have permitted the ruptured crown to become depressed under the subsequent paving operations and traffic loading and may contribute to additional edge fracturing in this area by providing poor support compounded with moisture intrusion through the surface opening cracks.

Conclusions and Recommendations

As a result of the field investigation, it was determined that the observed damage in the area surrounding the centreline crown of the road was severe, but was limited to the area where cracking was observed. Decreases in dynamic stiffness and increases in mobility were observed where cracking was observed in the centreline area and the shoulder edges. The investigation also indicated that the mid-lane regions of the pavement appeared to be in structurally sound condition. A simplified numerical analysis indicates that excessive applied load was probably imparted to the PC-FDR base layer during the micro-cracking effort. This excessive load may result from two different aspects of the micro-cracking process used in Nova Scotia:

a) the bulk of the applied load being imparted to the hardened stabilized mat when the roller spans onto the adjacent granular shoulder materials or unstabilized pulverized materials during micro-cracking of the first stabilized mat; and,

b) the roller spanning both lanes over the crown of the road during micro-cracking of the second stabilized mat, imparting the bulk of the applied to the crown.

As a result of this study, the following recommendations are made:

1) Ensure that the spray nozzles on moist curing water supply trucks are functioning properly to avoid erosion and subsequent loosening and loss of the uncured stabilized mat surface under local traffic. An alternative curing approach is the application of a sprayed on asphalt sealant to prevent the loss of internal mixing water. This approach also avoids the possibility of alternating wetting and drying cycles of the mat at early ages.

2) The width of future PC-FDR reconstruction projects should be increased by approximately 35-40 cm per lane into the existing granular shoulders, beyond the lane width to be paved using asphalt concrete. This will increase the distance from the free edge of the plate relative to the point of traffic loading and provide reasonable margin to avoid overlapping the roller onto the granular shoulder during micro-cracking.

3) Micro-cracking efforts should be confined to the centre of each traffic lane only. Roller patterns used for micro-cracking should avoid the slab edges and crown area by at least 35-40 cm to avoid cracking the shoulder edges and rupturing the crown of the road. Rollers should be prevented from spanning both lanes across the crown.

4) Future full depth recycling projects should take advantage of available materials by stabilizing the full pulverized depth. Full depth stabilization not only improves the modulus of subgrade reaction that might be expected by minimizing disturbed, loose materials under the stabilized slab, but contributes greatly to the stiffness of the pavement system. By Equation 1, the stiffness of a pavement is a function of the thickness cubed, making it much more sensitive to an increase in thickness than in modulus.

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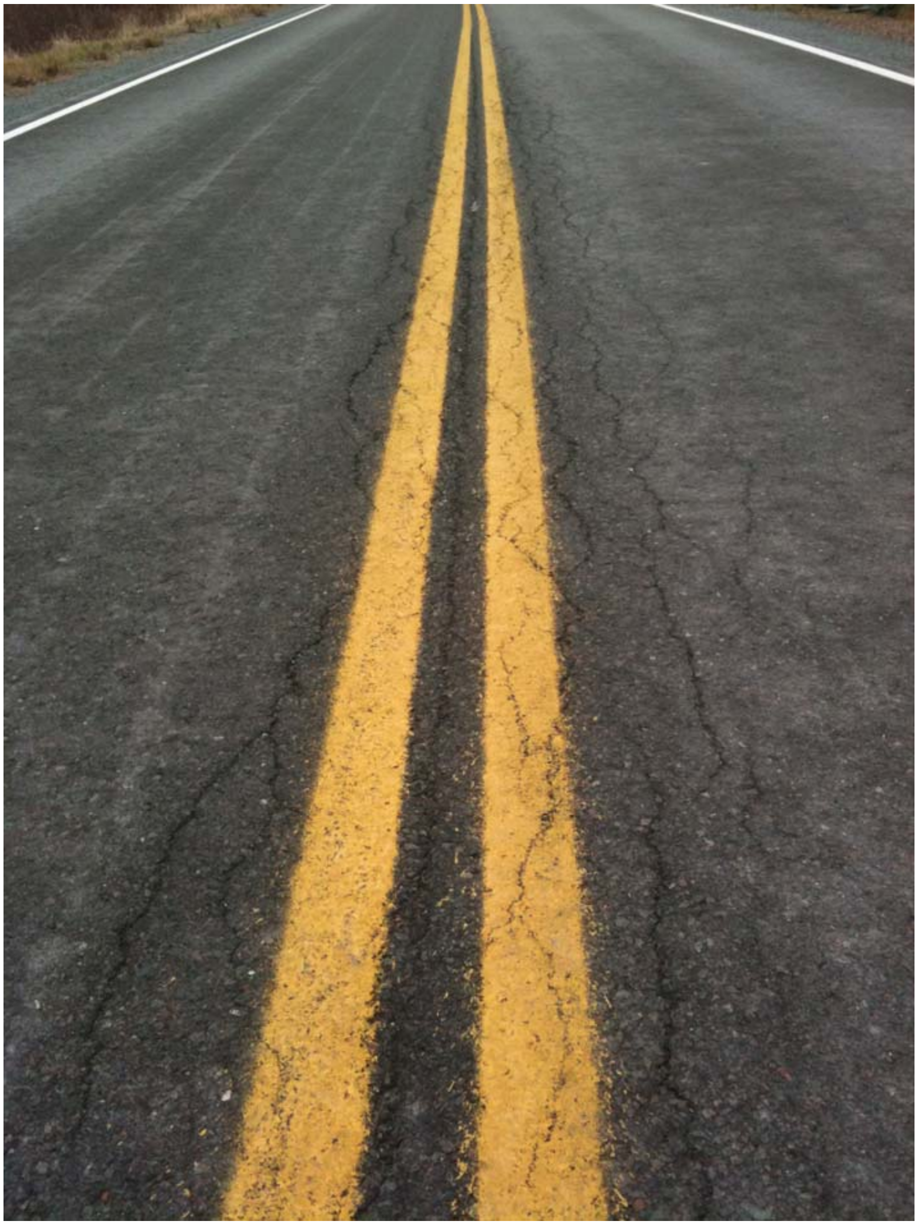

Figure 1 – Severe centreline cracking observed on Pt. Michaud beach Road PC-FDR pavement.

Figure 2 – Depressed crown in a region of moderate centreline cracking.

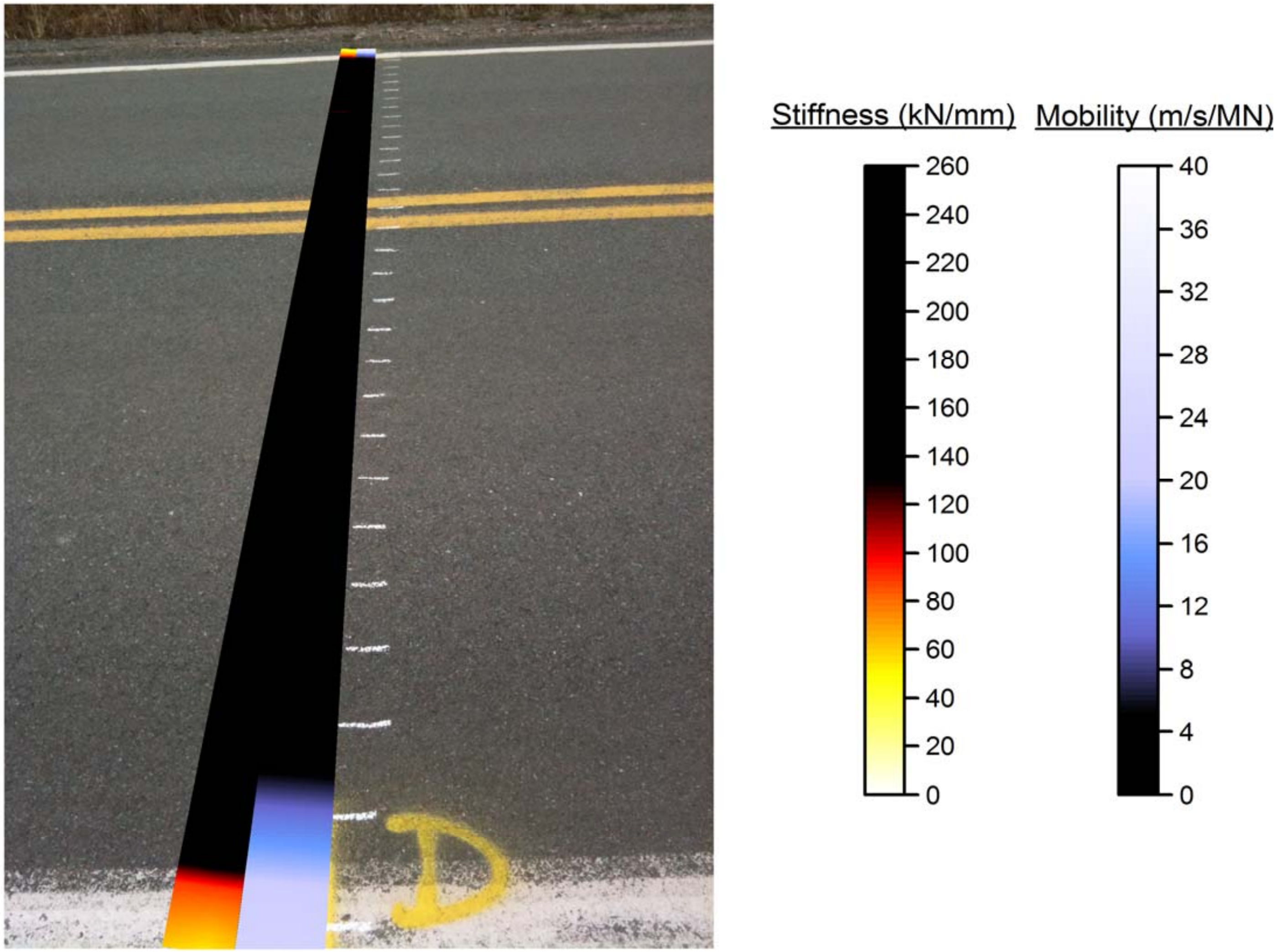

Figure 3 – Variation in dynamic stiffness and mobility across an undamaged pavement section.

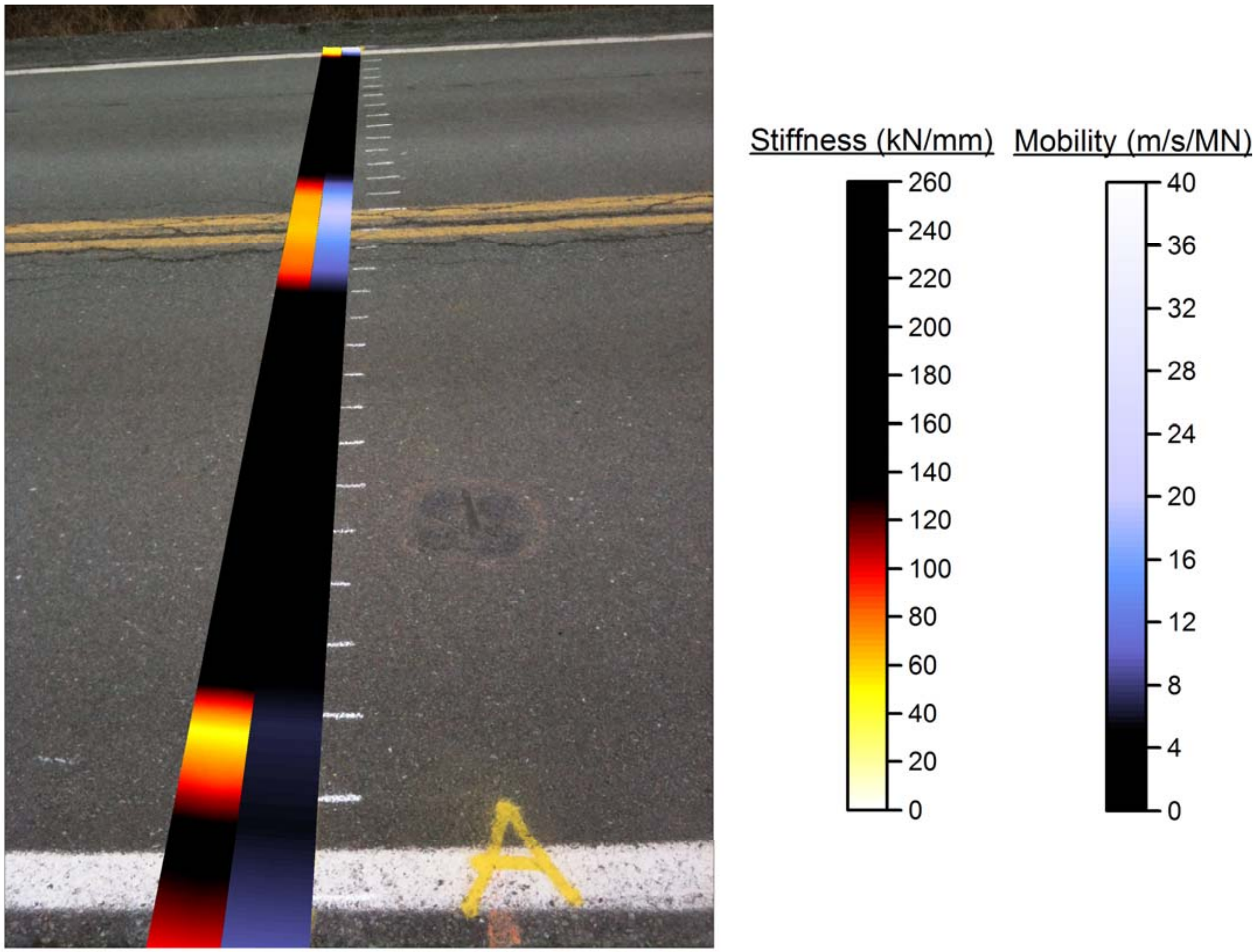

Figure 4 – Variation in dynamic stiffness and mobility across a typical damaged pavement section.

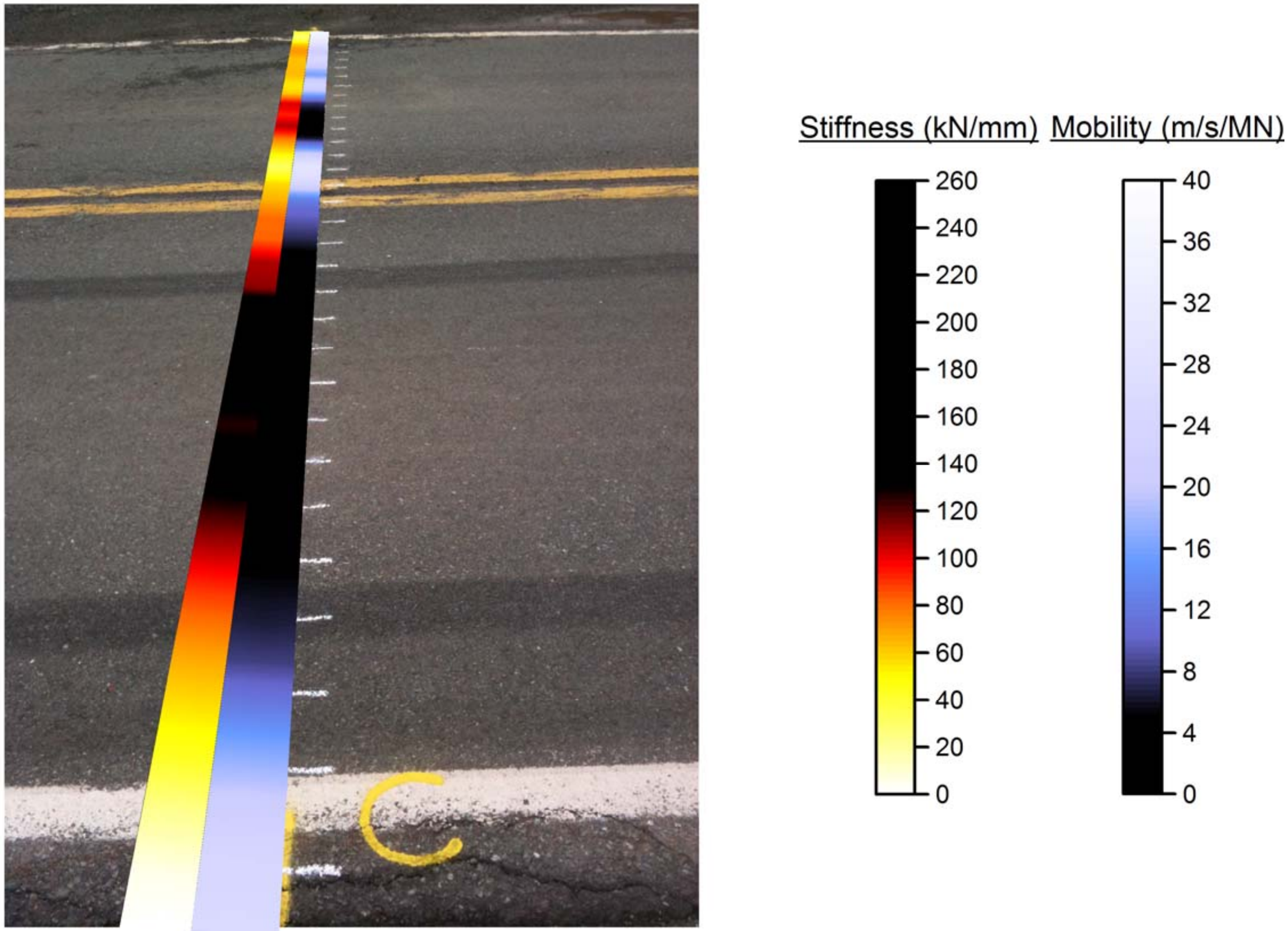

Figure 5 – Variation in dynamic stiffness and mobility across a non-typical damaged pavement section.

Figure 6 – Photograph of intact asphalt concrete and stabilized base core extracted from outer wheel path area.

Figure 7 – Photograph of core location in heavily damaged PC-FDR base near the pavement centreline.

Figure 8 – Poor contact of micro-cracking effort on rutted surface of uncured PC-FDR mat.