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research report

Forensic Investigation
of Brick Paver Crosswalks
at Court Square in Charlottesville, Virginia

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Abstract

This report documents the findings of a forensic investigation to determine the causes of premature failures noted in brick paver crosswalks in the Court Square area in Charlottesville, Virginia. Brick paver crosswalks were installed in late November to early December 2004 and began to show signs of permanent deformation within approximately 18 months after installation.

This study sought to examine the cause(s) of this deterioration through a forensic investigation, to provide recommendations for corrective action, and to present suitable designs for long-term performance. The findings are expected to lead to significant cost savings for the City of Charlottesville by eliminating future premature failures and providing alternative designs suitable for moderate- to low-volume urban settings and criteria for evaluating future designs.

The findings suggest that minor changes to specifications and construction procedures could greatly increase the service life of brick crosswalk structures used in urban conditions. If an approximate construction cost is $25 per square foot, the construction of four crosswalks within each intersection (assuming two 10-foot travel lanes, two 8-foot parking lanes, and a length of 8 feet) costs nearly $30,000. Following the design recommendations provided herein should yield a service life of 15 to 20 years. If the same costs are incurred in constructing crosswalks with the current service life (approximately 3 years), the recommended design and specification changes could result in a savings of $120,000 to $170,000 per intersection by reducing the need to reconstruct on a frequent basis. This cost difference does not include the costs associated with delays to the traveling public or inconvenience to nearby businesses, which would increase the cost savings significantly.

FINAL REPORT

**FORENSIC INVESTIGATION OF BRICK PAVER CROSSWALKS
AT COURT SQUARE IN CHARLOTTESVILLE, VIRGINIA**

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DISCLAIMER

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

ABSTRACT

This report documents the findings of a forensic investigation to determine the causes of premature failures noted in brick paver crosswalks in the Court Square area in Charlottesville, Virginia. Brick paver crosswalks were installed in late November to early December 2004 and began to show signs of permanent deformation within approximately 18 months after installation.

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INTRODUCTION

As part of urban renewal projects, many municipalities consider or complete the installation of paver or paver-style crosswalks within their urban centers. Typically, the construction sequence of a retrofitted crosswalk consists of excavating an existing pavement and installing these crosswalks within a limited space. Most crosswalks have dimensions less than 15 feet (in the direction of vehicular traffic) by the width of the roadway. In late November to early December 2004, the City of Charlottesville, Virginia, excavated portions of the existing pavement and installed brick paver crosswalks at five intersections that form the boundary of the area known locally as Court Square, laid out in 1762 and surrounding the historic Albemarle County Courthouse (National Park Service, 2006). Within approximately 18 months after their installation, the paver crosswalks exhibited permanent deformation within the wheel paths and horizontal movement of the pavers under traffic.

Examples of typical materials used in paver and non-paver crosswalk construction for vehicular areas are pavers with an asphalt or concrete base over an aggregate subbase, pressed asphalt, and cast-in-place stamped concrete. Each system has particular benefits and detriments. Paver crosswalks may be composed of clay brick pavers or use concrete or asphalt pavers. Paver crosswalks are often used in areas to recreate a colonial style appearance. However, they almost always tend to feel rougher to passing motorists. This can be beneficial if they are used to produce a measure of traffic calming. Construction requires excavating a section of the existing pavement. Pressed asphalt and cast-in-place stamped concrete are examples of non-paver crosswalk construction. To create a pressed asphalt crosswalk, an area of existing HMA pavement is heated and a brick form is pressed into the warmed surface to create an imprint. Pressed asphalt crosswalks may require frequent repainting to maintain a brick-like appearance. Cast-in-place stamped concrete uses a brick form that is pressed into a recently cast concrete panel while the material is still plastic. The resultant crosswalk acts as a monolithic concrete pavement and can be either reinforced or non-reinforced.

PURPOSE AND SCOPE

The purpose of this study was to evaluate the existing condition of the paver crosswalks in the Court Square area in Charlottesville, Virginia, to determine the most probable causes for the premature failures.

This study sought to examine the cause(s) of this deterioration through a forensic investigation, to provide the city engineer's office with recommendations for corrective action, and to present suitable designs for long-term performance.

METHODOLOGY

This study was performed in four phases: (1) a literature review, (2) a field forensic investigation, (3) a laboratory analysis, and (4) a pavement design analysis conducted in accordance with the procedures of the Virginia Department of Transportation (VDOT). The literature review identified relevant literature concerning the structural design of paver crosswalks and examples of designs in use by other municipalities. The field forensic investigation and laboratory analysis involved the selection of two intersections where a visual survey and material sampling were performed to document the existing conditions and determine the in-place properties and conditions of the hot-mix asphalt (HMA), aggregate base layer, and subgrade materials.

Literature Review

A literature review was conducted to identify recent studies where other research was performed related to crosswalk pavements constructed using pavers and theoretical modeling of crosswalk pavement structures. The literature review was conducted by searching relevant electronic databases such as the TRIS online database (maintained by the Transportation Research Board).

Field Forensic Investigation

A field investigation at the intersections of 4th Street and High Street (4th and High) and 4th Street and Jefferson Street (4th and Jefferson) in Charlottesville, Virginia, was conducted on February 28, 2006, and March 15, 2006, respectively. The crosswalks at these locations were constructed in November 2004 by excavating the existing HMA pavement and replacing it with a system composed of clay brick pavers placed on top of a two-layer HMA system, which was placed on top of a compacted aggregate base. At both intersections, the pavers within the wheel path exhibited permanent deformation of up to approximately 1.5 inches compared to the surrounding pavement. The depressions were shallow and bowl shaped with no upheaval at the edges of the wheel path.

During the crosswalk field investigation, material samples from the selected crosswalks were collected from two areas: within a wheel path and outside a wheel path. These locations were chosen since the predominant mode of failure evident by visual inspection was permanent deformation within the wheel path. It was hypothesized that if a portion of the permanent deformation was caused by densification of the bound layers, the HMA cores collected within the wheel path would show a lower air-void than those cores collected outside the wheel path.

The as-designed crosswalk pavement structure consisted of a 3-inch-thick clay brick paver, a neoprene-modified asphalt adhesive, a 1-inch-thick layer of a surface HMA mixture (VDOT mix SM-1), a 6-inch-thick layer of a base HMA mixture (VDOT mix BM-3), and 10 inches of compacted aggregate (VDOT aggregate designated 21-A), all placed on top of the existing subgrade. Stone dust was used for joint sand (City of Charlottesville, 2002). The gradation specifications for the HMA materials and the aggregate base material, based on the 1993 edition of VDOT's *Road and Bridge Specifications* (VDOT, 1993), are presented in Appendix A, Tables A1 and A2, respectively. The brick pavers were surrounded by granite blocks that were bordered by concrete bulkheads to act as an edge restraint against lateral movement. An example is shown in Figure 1.

The field investigation focused on selecting one crosswalk in each of the two intersections, the choice of which was primarily driven by ease of accessibility considering existing traffic. The field investigation consisted of documenting the existing condition at the crosswalk and collecting specimens within and outside the wheel paths. HMA materials were collected by wet coring the brick paver and underlying HMA material. Samples of the underlying aggregate layer and subgrade were collected by using a pneumatic hammer to loosen the brick in an area of approximately 3 square feet. The hammer was then used to remove the underlying HMA material from this area. Removal of the brick and HMA by a dry process


Figure 1. Example of Paver Crosswalks at Charlottesville's Court Square

allowed for the aggregate base and subgrade material to be collected to determine the in-situ moisture content. At each intersection, two 6-inch-diameter cores containing the brick paver and the underlying HMA layers were collected from within a wheel path area and two from outside a wheel path area. An additional core was collected from the outside wheel path area at 4th and Jefferson. Samples of the aggregate and subgrade were collected from one additional wheel path location and one additional non–wheel path location at each intersection.

The cored HMA samples were tested to determine their bulk specific gravity, air void and asphalt binder content, and gradation. An example of the coring process at 4th and High is shown in Figure B1, Appendix B. The loose aggregate and subgrade samples were tested to determine their gradation, in-situ moisture content, and Atterburg limits. Knowing these properties is critical to establish a range of the moduli of the in-situ materials. The brick crosswalk responses (stresses, strains, and deflection) can be estimated and used to determine the failure mechanism(s). Samples from the aggregate base and subgrade were collected by a dry process and immediately placed into plastic specimen bags that were then sealed. An example of the collection of the aggregate base at 4th and High is shown in Figure B2, Appendix B.

The samples obtained from 4th and High were collected from the crosswalk on the west side of the intersection, an area trafficked by eastbound traffic on High Street. An overview of this intersection is presented in Figure 2a. The samples obtained from 4th and Jefferson were collected from the crosswalk on the south side of the intersection, an area trafficked by southbound traffic on 4th Street. An overview of this intersection is presented in Figure 2b. The traffic at both locations travels at low speeds (ranging from stopped to approximately 25 mph). Although no traffic records were available for 4th Street or Jefferson Street, High Street carries approximately 14,000 vehicles per day with approximately 3% trucks (VDOT, 2004) and is used as a major east-west thoroughfare by all vehicle types. An estimated equivalent single-axle load (ESAL) count of 153,720 per year was calculated. Although exact traffic counts were not available for 4th and Jefferson, based on visual observation, the daily traffic volume at 4th and High far exceeds the daily traffic volume at 4th and Jefferson.

(a)   (b)
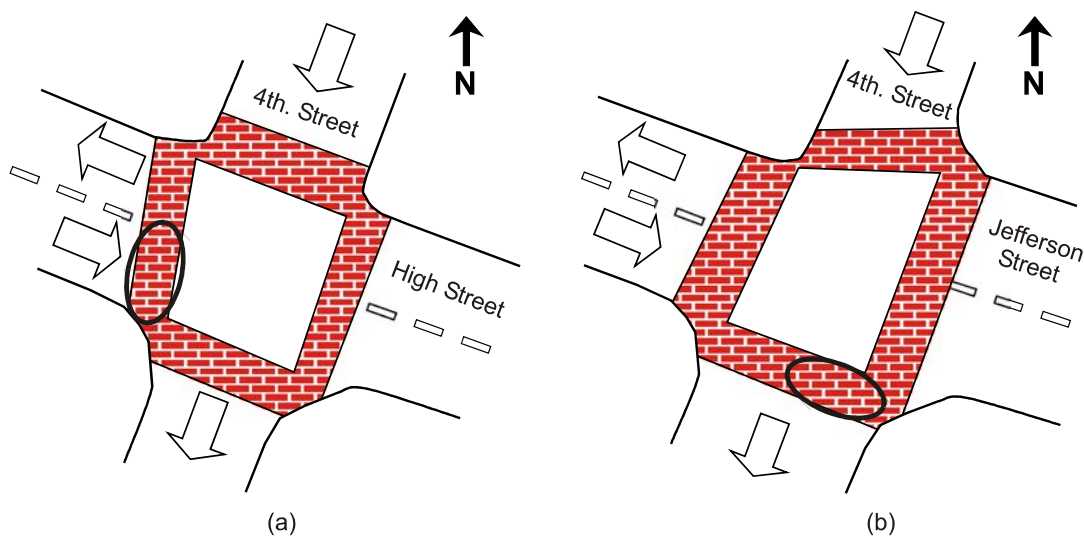
Figure 2. Location of Field Investigation at (a) 4th and High and (b) 4th and Jefferson. Areas from which specimens were collected are circled.

Laboratory Analysis

Laboratory tests performed on collected HMA samples included the Standard Method of Test for Determining the Asphalt Binder Content of Hot-Mix Asphalt by the Ignition Method (AASHTO T 308), Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates (ASTM C 136), and Standard Method of Test for Theoretical Maximum Specific Gravity and Density of Hot-Mix Asphalt Paving Mixtures (AASHTO T 209). Laboratory tests performed on the aggregate base and subgrade specimens included Determining the Liquid Limit of Soils (AASHTO T 89), Determining the Plastic Limit and Plasticity Index of Soils (AASHTO T 99), and Determining the Moisture Content of Soil—Conventional Oven Method (ASTM 2216).

Pavement Design Analysis

The pavement design analysis was completed using the DarWIN computer program following VDOT's procedures for pavement design (VDOT, 2000). Information from the literature review provided additional details concerning the designs of pavements using pavers in areas under vehicle traffic.

RESULTS AND DISCUSSION

Literature Review

Paver Crosswalks

A paver crosswalk used in areas of vehicular traffic must have the following design elements to ensure a long service life: joint sand to prevent vertical movement, edge restraint to prevent horizontal movement, bedding sand to facilitate load transfer, and solid foundational support (Interlocking Concrete Pavement Institute [ICPI], 1995a). Each component helps the pavers transfer loading to the foundation uniformly and as a system rather than as individual entities. Pavers with proper joint and bedding sand have been assumed to behave similarly to a flexible pavement (Rada et al., 1990; Rollings and Rollings, 1992; ICPI, 1995a). The purpose of the joint and bedding sand is to provide a mechanical interlock between the pavers. Despite their being rigid entities, the use of joint and bedding sand allows pavers to resist vertical displacement by frictional forces developed at each joint using sand and support from underlying layers. Thus, the pavement structure can be analyzed with the paver(s) and layer simulated as a typical flexible pavement using finite element, multi-layer elastic analysis, or other pavement design software.

Shackel (2000) stated that the load distribution depends on the paver shape, thickness, and placement pattern. The paver shape is often a simple rectangle the same size as a traditional masonry brick. However, masonry bricks may not provide the strength required for crosswalks in vehicular applications, and therefore a concrete paver is recommended (the concrete may be tinted to look similar to a traditional masonry brick). The paver shape may also help in resisting

rotation and lateral displacement if an interlocking paver is used. Figure 3 shows an example of an individual interlocking paver and its use as a pavement surface. The ICPI recommends a paver thickness of 3 1/8 inches (80 mm) for pavers exposed to vehicular traffic (ICPI, 1995b).

The number of placement patterns available is almost limitless; however, previous studies have concentrated on a few commonly used patterns. These patterns include the stretcher bond, basketweave, and herringbone (both 45 and 90 degrees). Figure 4 shows examples of each of these patterns with specific patterns suggested for pedestrian (stretcher bond and basketweave) and vehicular (herringbone) applications. ICPI (1995b) suggests that herringbone patterns are the most appropriate for street applications. Hassani and Jamshidi (2006) performed finite element analysis that showed the herringbone pattern to be superior to other patterns for performance in vehicular applications.

Interlocking Paver
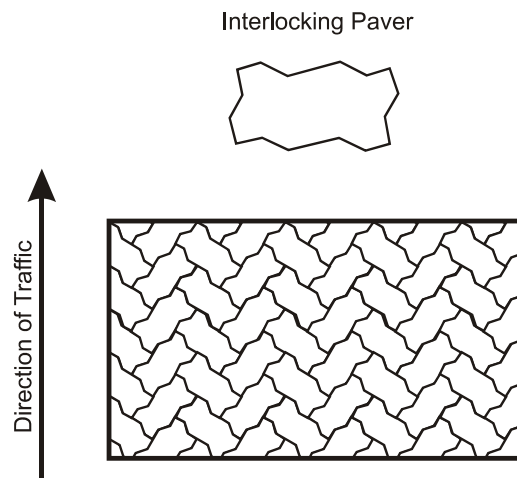
Figure 3. Example of an Individual Interlocking Paver and Its Use in a Pavement Surface

Pedestrian

Stretcher Bond

Basketweave

Vehicular
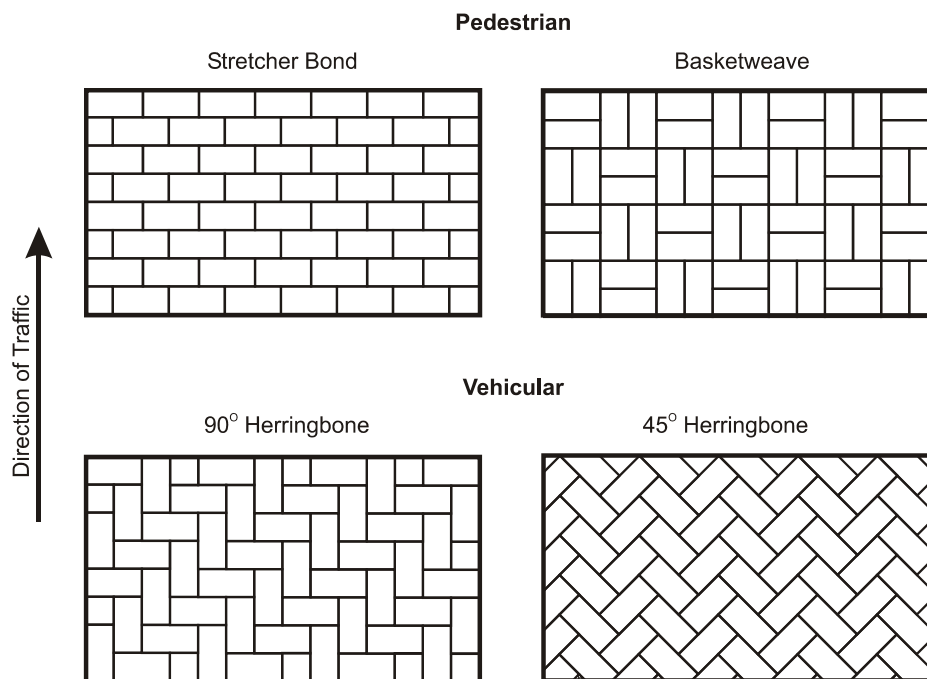
90° Herringbone

45° Herringbone

Figure 4. Placement Patterns for Paver Pavements (after ICPI, 2005)

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Hassani and Jamshidi (2006) studied the contribution of the sand bedding layer as a structural component and described its role in the overall system as vital. Through the use of finite element modeling, they showed that the use of bedding sand aides in distributing the load over a larger area of the pavement foundation and that maintaining the joint sand is essential to distribute the load throughout the pavement system. ICPI (1995b) recommends that the bedding sand be composed of a layer of concrete sand 1 to 1.5 inches thick complying with the requirements of ASTM C 33. Stone screenings or stone dust, as was specified in the contract for the Court Square crosswalks (City of Charlottesville, 2002), may tend to break down over time leading to loss of the jointing and bedding material and ultimately movement of the pavers under traffic (ICPI, 1995b; Yaginuna et al., 2000). Joint sand, used to provide frictional resistance to block movement, is recommended to be made up of masonry sand in accordance with ASTM C 144 rather than stone dust for similar reasons (ICPI, 1995b; Rollings and Rollings, 1992). Table A3, in Appendix A, presents the gradation requirements for each specification. It may also be necessary to reapply joint sand material periodically after the crosswalk is opened to traffic to ensure that any material that may have settled through the joint over time is replaced (Rollings and Rollings, 1992; Yaginuna et al., 2000).

Permanent Deformation in Flexible Pavements

Examples of typical permanent deformation types including unbound material (subgrade) rut failure are shown in Figure 5. White et al. (2002) investigated in-service pavements and summarized work on the contribution of various layers in a flexible pavement to permanent deformation. The authors investigated many rutted flexible pavements and sought to determine if it was possible to relate the transverse surface profile with permanent deformation in specific layers. They found that permanent deformation is caused by a combination of material densification and shear-related deformation. These mechanisms may occur in any of the
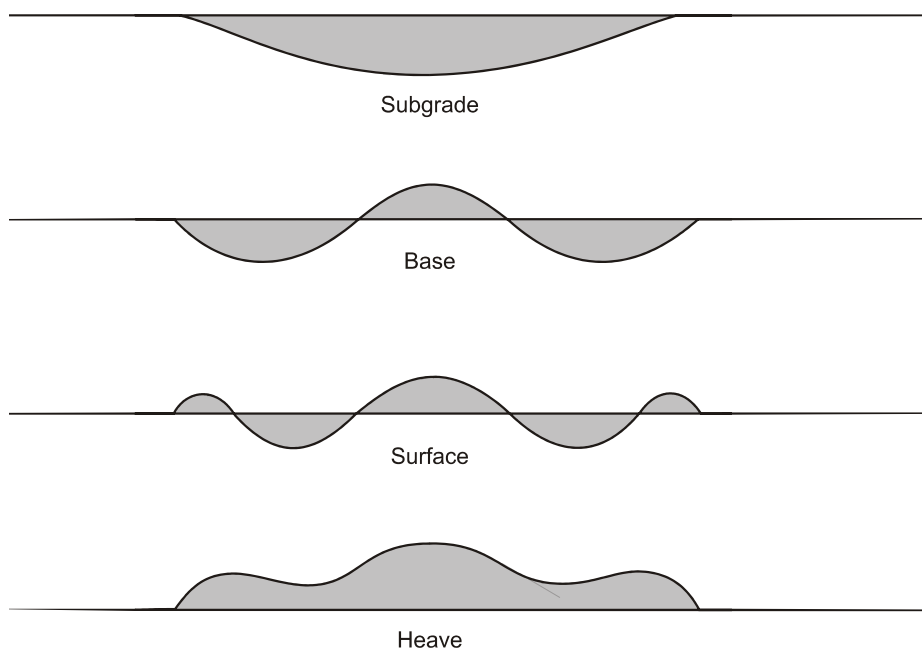

Figure 5. Transverse Profiles Showing Four Types of Permanent Deformation (after White et al., 2002)

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pavement structural layers. The investigators cited many studies reporting that when permanent deformation attributable to shear deformation of the surface HMA occurs, a depression within the wheel path is often accompanied by some measure of upheaval adjacent to the wheel path. This was found by several researchers investigating HMA pavements with very stiff base layers. However, when permanent deformation occurs in the supporting layers, it manifests as a wide depression with no uplift along the sides of the wheel paths. Researchers have also suggested that permanent deformation can be attributed to the vertical compressive strain at the top of the subgrade. It has been reported that this strain should be less than approximately 200 microstrains to prevent permanent deformation within the subgrade (Monismith and Long, 1999; Newcomb et al., 2001).

Laboratory Analysis

HMA Specimens

As discussed in the "Methods" section, eight cored HMA specimens, four each from two crosswalk locations, were processed in the laboratory to determine their thickness, bulk specific gravity, air-void, asphalt binder content, and gradation. The cores were sawn into two pieces, an upper and a lower portion, with the upper portion containing the SM-1 HMA surface mix and the lower portion containing the BM-3 HMA base mix. At 4th and High, Specimens 1 and 2 were collected from outside the wheel path and Specimens 3 and 4 from within the wheel path areas. At 4th and Jefferson, Specimens 5 and 7 were collected from outside the wheel path and Specimens 8 and 9 from within the wheel path.

Tables 1 and 2 present the results of the laboratory testing of the HMA core specimens (surface mix and base mix, respectively). The in-place air-void content is high, ranging from approximately 10% to 19%. Although permeability will vary with the same air-void content for various mixes, Brown et al. (2004) and Choubane et al. (1998) suggested that air voids should be less than approximately 6% to 7% to avoid excessive permeability.

Specimens 1 through 4 were collected from 4th and High. The average air-void content for the HMA surface mix from Specimens 1 and 2 (outside the wheel path) is 18.6%. The average air-void content for the HMA surface mix from Specimens 3 and 4 (within the wheel path) is 12.2%. Densification at the wheel path location for the HMA surface mix can be seen at this location. However, the distinction is not as significant for the HMA surface mix specimens collected at 4th and Jefferson. The average air-void content for the HMA surface mix from Specimens 5 and 7 (outside the wheel path) is 16.1%. The average air-void for the HMA surface mix from Specimens 8 and 9 (within the wheel path) is 15.0%. As stated previously, the traffic volume is much less at 4th and Jefferson and, thus, sufficient traffic loading may not have been applied to cause the same differentiation as noted at 4th and High. The distinction in air-void content for HMA base layer specimens collected either within or outside the wheel path is not as great as for the surface layer specimens. This would be expected as the effects of traffic loading dissipate with depth from the surface.

Table 1. Results of Laboratory Testing on HMA Surface Mix Specimens

Core ID[1]	Intersection Location	Bulk Specific Gravity	Air-Void Content, %	Asphalt Binder Content, %[2]	Layer Thickness, inch	% by Weight Passing No. 4	% by Weight Passing No. 200
1-OWP[3]	4th and High	2.122	17.9	7.64	2.5	62.2	9.8
2-OWP		2.089	19.2	5.83	2.4	60.3	9.2
3-WP[4]		2.272	12.1	6.00	2.5	59.8	10.0
4-WP		2.271	12.2	6.21	2.2	59.8	9.0
5-OWP	4th and Jefferson	2.210	14.5	6.90	1.8	73.2	6.5
7-OWP		2.132	17.6	6.72	1.9	75.7	6.3
8-WP		2.211	14.5	6.76	2.8	84.6	6.2
9-WP		2.187	15.4	6.75	2.4	81.5	6.4

[1]Specimen 6 removed during testing.
[2]Binder content by ignition method, AASHTO T 308.
[3]OWP = outside wheel path location.
[4]WP = within wheel path location.

Table 2. Results of Laboratory Testing on HMA Base Mix Specimens

Core ID[1]	Intersection Location	Bulk Specific Gravity	Air-Void Content, %	Asphalt Binder Content, %[2]	Layer Thickness, inch	% by Weight Passing No. 4	% by Weight Passing No. 200
1-OWP[3]	4th and High	2.261	15.2	5.60	4.1	55.7	5.9
2-OWP		2.282	14.4	5.56	4.0	56.0	5.6
3-WP[4]		2.282	14.4	5.30	4.1	54.6	5.0
4-WP		2.315	13.2	5.34	4.3	55.0	5.5
5-OWP	4th and Jefferson	2.305	13.6	5.53	5.6	60.9	7.1
7-OWP		2.407	9.7	5.26	5.7	56.3	6.8
8-WP		2.184	18.1	5.33	3.7	61.7	6.0
9-WP		2.233	16.2	5.18	3.5	58.1	6.2

[1]Specimen 6 removed during testing.
[2]Binder content by ignition method, AASHTO T 308.
[3]OWP = outside wheel path location.
[4]WP = within wheel path location.

The thickness of each layer (surface and base) was measured in the laboratory from the collected core specimens. Table 1 presents the as-measured thickness of the HMA surface mix. The thickness at 4th and High ranges from 2.2 to 2.5 inches, with an average of 2.4 inches. The thickness at 4th and Jefferson ranges from 1.8 to 2.8 inches, with an average of 2.2 inches. In all cases, the thickness was greater than the as-designed thickness of 1.0 inch. Table 2 presents the as-measured thickness of the HMA base mix. The thickness at 4th and High ranges from 4.0 to 4.3 inches, with an average of 4.1 inches. The thickness at 4th and Jefferson ranges from 3.5 to 5.7 inches, with an average of 4.6 inches. In all cases, the thickness was less than the as-designed thickness of 6.0 inches. The total thickness (both layers) at 4th and High ranged from 6.4 to 6.6 inches, with an average of 6.5 inches. In all cases from 4th and High, the total thickness (both layers) was less than the as-designed thickness of 7.0 inches. The total thickness (both layers) at 4th and Jefferson ranged from 5.9 to 7.5 inches, with an average of 6.8 inches. In two of the four cases, the total thickness (both layers) was less than the as-designed thickness of 7.0 inches.

When comparing the gradation analysis results of Table 1 with VDOT's specifications from Table A1, Appendix A, it can be seen that the HMA surface mix from Specimens 1 through 4 did not comply with the criteria for percentage by weight passing the No. 4 and the No. 200 sieve. The specification requires 65% to 85% by weight passing the No. 4 sieve and a maximum of 8% by weight passing the No. 200 sieve. The results for Specimens 1 through 4 range from 59.8% to 62.2% by weight passing the No. 4 sieve. The results for Specimens 1 though 4 range from 9.0% to 10.0% by weight passing the No. 200 sieve. The results for Specimens 5, 7, 8, and 9 complied with the specifications for percent passing the No. 4 and the No. 200 sieves for the HMA surface mix.

When comparing the gradation analysis results of Table 2 with VDOT's specifications from Table A1, Appendix A, it can be seen that the HMA base mix from all specimens did not comply with the criteria for percentage by weight passing the No. 4 sieve and all specimens except for Specimen 3 did not comply with the criteria for percentage by weight passing the No. 200 sieve. The specification requires 26% to 44% by weight passing the No. 4 sieve and a maximum of 5% by weight passing the No. 200 sieve. The results of all specimens range from 54.6% to 61.7% by weight passing the No. 4 sieve and from 5.0% to 7.1% by weight passing the No. 200 sieve. Higher fine contents (higher percentage passing the No. 200 sieve) result in an increased asphalt mastic (binder plus fines) with the potential for reduced asphalt stiffness that can also make the HMA mixture prone to failure by permanent deformation.

Aggregate and Subgrade Specimens

As discussed in the "Methods" section, aggregate and subgrade specimens were stored in sealed plastic bags in the laboratory until they were tested. Enough material was collected in the field to allow the specimens to be split so that duplicate specimens could be tested for moisture content and gradation. Table 3 presents the results for moisture content, Atterburg limit, and gradation testing. The specimens are presented by a numerical and an alphabetical ID. The numerical portion indicates the number of the excavation and the alphabetical portion represents the strata within each excavation. For example, Specimens 1A through 1D represent the four strata collected at Excavation Site 1. The approximate depths from the surface for each collected specimen are also given in Table 3. The 21-A aggregate base was divided into approximately three equal sections and comprise specimens A, B, and C from each excavation site; the subgrade comprises specimen D from each excavation site.

The moisture content of the 21-A aggregate base layer specimens range from 4.1% to 13.7% by weight. When compared to the plastic limit for these specimens, it can be seen that the aggregate base layer remains within the semi-solid region. The moisture content of the subgrade specimens range from 11.0% to 18.5% by weight. When compared to the plastic limit for these specimens, it can be seen that the subgrade specimens are also still within the semi-solid region. Each of these materials should be expected to behave elastically when small vertical strains are applied.

When the gradation analysis results in Table 3 are compared with VDOT's specifications from Table A2, Appendix A, it can be seen that the percent by weight passing the No. 200 sieve

Table 3. Results of Laboratory Testing on Aggregate and Subgrade Specimens

Excavation ID		Intersection Location	Material Description	Approximate Depth from Surface, inch	Moisture Content, %	Plastic Limit	Liquid Limit	% by Weight Passing No. 200
1-OWP[1]	A	4th and High	21-A	10-13	4.1	17.9	26.7	13.9
	B			13-16.5	4.6	21.4	29.0	22.9
	C			16.5-18.5	5.1	21.5	36.2	14.3
	D		Subgrade	> 18.5	17.2	27.4	44.4	30.4
2-WP[2]	A		21-A	10-12.5	3.0	18.7	22.9	9.4
	B			12.5-15.5	3.5	21.4	26.8	11.6
	C			15.5-18.0	5.6	19.1	28.5	12.5
	D		Subgrade	> 18.0	18.5	31.4	49.6	30.0
3-WP	A	4th and Jefferson	21-A	10.5-13	7.7	22.5	33.3	17.4
	B			13-16.5	11.9	27.1	43.4	23.8
	C			16.5-18.5	13.7	25.9	44.3	26.6
	D		Subgrade	> 18.5	11.9	25.5	38.0	23.4
4-OWP	A		21-A	10.5-13.5	4.6	18.8	23.3	15.2
	B			13.5-15.5	5.0	18.0	22.8	16.2
	C			15.5-18	5.8	19.1	26.1	18.2
	D		Subgrade	> 18.0	11.0	23.6	35.3	27.1

[1]OWP = outside wheel path location.
[2]WP = within wheel path location.

for the 21-A aggregate base layer material exceeds the maximum value of 12% for all cases except Specimens 2A and 2B. The percent by weight passing the No. 200 sieve for the collected 21-A aggregate base layer specimens ranges from 9.4% to 26.6%. Table 3 also shows that in three of the four tested locations, the percent by weight passing the No. 200 sieve is greater for Specimen C than for Specimens A and B. This indicates that the bottom of the 21A aggregate base material has a higher fines content than the top or the middle. According to Huang (2004), mixing of fines from the subgrade with the lower portions of an aggregate base is possible when a pavement is subjected to heavy traffic loading. This phenomenon may be the cause for the higher fines content at the bottom of the 21-A aggregate base layer. Migration of fines can be a great concern in terms of pavement performance since the bottom of the aggregate base can become weaker with the intrusion of excessive fines (Huang, 2004). Thus, instead of one thicker aggregate layer resting on top of the subgrade, the resultant pavement consists of several thinner and weaker layers. The percent by weight passing the No. 200 sieve for the collected subgrade specimens ranges from 23.4% to 30.4%. Thus, a high percentage of fines is available in the subgrade.

As seen in Table 3, the subgrade soils for the two intersections are typical of those soils classified as A-7-6 and A-6. The moduli estimates for these types of soils range from as low as 3,000 psi to as high as 12,000 psi (California bearing ratio values from 2% to 8%). These types of soils are often weak. Proper compaction at optimum moisture content or chemical treatment is highly recommended to increase the load-carrying capacity and reduce potential densification under repeated traffic loading. In addition, these soils may become even weaker when subjected to saturated conditions caused by poor drainage.

When considering the summation of all effects shown in Table 3, it should be expected that these materials will not have their optimum load-carrying capability. This effect will be compounded by the fact that the materials are placed in a retrofitted section of pavement of limited size. Therefore, the strength of the pavement in the crosswalk is likely to be lower than that of the surrounding original pavement, given the same total thickness. In addition, since the crosswalk pavement structure represents a discontinuity in the existing pavement, there may be no opportunity for the pavement structure to distribute the applied vehicular loading over a large area (beyond the bounds of the crosswalk). Thus, vehicle loading near the edges of the crosswalk pavement may cause a stress concentration within the pavement structure, causing higher stresses in the supporting layers below. To counteract these effects, the load-carrying capacity of the replaced pavement should be greater than that of the original pavement section.

Uniformity

For a pavement to uniformly perform as designed during its service life, it is important that the pavement structure be constructed in a uniform manner. The concept of uniformity can extend across many facets to include properties such as void and binder content, percent passing particular critical sieve sizes, and thickness. Often, when a pavement is not constructed uniformly, a portion(s) of the pavement is likely to exhibit premature deterioration. During the course of the field sampling several instances where the pavement structure was not uniform were noted. Examples of the lack of uniformity in air-void and binder content, gradation, and thickness were previously discussed.

A visual example of a non-uniform pavement layer thickness is shown in Figure B3, Appendix B. This photograph shows the excavation process at 4th and High. The thickness of the 21-A aggregate base material is shown to be approximately 5 inches at this location. In the other locations for sampling, the layer thickness was approximately 8 to 9 inches. Another example includes two cores collected at 4th and Jefferson. Figure B4, Appendix B, shows Specimens 6 (top of the figure) and 8 (bottom of the figure), which were collected approximately 24 to 30 inches apart. It can be seen that Specimen 6 contains some larger diameter aggregate particles in the bottom half of the core (left-hand side of the core). However, Specimen 8 appears to contain only smaller diameter particles. In addition, the HMA material in Specimen 6 is nearly 1.5 inches longer than in Specimen 8. These two observations would indicate that the pavement performance and the ability to which compaction was achieved could be vastly different for these two locations. The difference in thickness of the HMA portions of the core may also indicate poor grade control at the surface of the 21-A aggregate base layer (although this could be the result of underground obstructions, such as utilities, that are unknown to the authors).

Proposed Replacement Crosswalk Pavement Designs and Alternatives

In this section, two designs are proposed that could be used to replace the existing crosswalk pavement structure. Following the typical VDOT procedures using the AASHTO 1993 Design Guide (VDOT, 2000), a set of pavement designs using brick-style pavers placed on top of HMA or PCC was created for the anticipated conditions studied in this project. The

proposed designs are based on traffic levels for the expected traffic conditions at this location and a design life of 20 years. A schematic of these designs is shown in Figure 6.

The design with an HMA base consists of a 3-inch brick-style paver, a 1-inch layer of bedding sand, 9 inches of HMA, and 8 inches of crushed aggregate (may be treated with cement or asphalt). If desired, the upper 12 inches of the subgrade may be treated with either lime or cement if the subgrade is in poor condition. The design with a PCC base consists of a 3-inch brick-style paver, a 1-inch layer of bedding sand, an 8-inch PCC slab, and 6 inches of crushed aggregate. Both designs include placing a layer of non-woven geosynthetic fabric between the crushed aggregate and the subgrade.

The bedding sand should be placed at a 1-inch thickness and vibrated in place using a plate-type vibrator. If necessary, additional sand should be placed following vibration such that the final thickness of the bedding sand is 1 inch. If confinement of the bedding sand is a concern, a non-woven geosynthetic material can be placed on top either the HMA surface layer or the PCC slab and wrapped up the sides of the excavated area to help ensure that the bedding sand remains confined throughout the life of the crosswalk. The apparent opening size of the geosynthetic must be specified to ensure that loss of the bedding sand does not occur.

Choosing the proper HMA materials will depend on the ability to place the material in successive lifts and the ability of the contractor to compact the mixture. Given a total thickness of 9 inches, the HMA materials will likely have to be placed with a minimum of three lifts. The actual number of lifts will depend on the nominal maximum aggregate size of the mixture specified. Each lift must be allowed to cool sufficiently to allow for the next layer to be placed and compacted. Proper compaction in an area expected to receive heavy vehicular traffic can be achieved only by using a heavy roller. It is advisable to use a full-sized roller (8 to 10 tons) to ensure that the desired compaction (resulting in approximately 6% to 8% air voids in place) is achieved. The authors realize that a roller of this size may not be able to compact the edges of the pavement fully; in these areas, a smaller roller may be necessary. However, compaction within the anticipated wheel path areas is critical to achieving the design performance of the crosswalk.
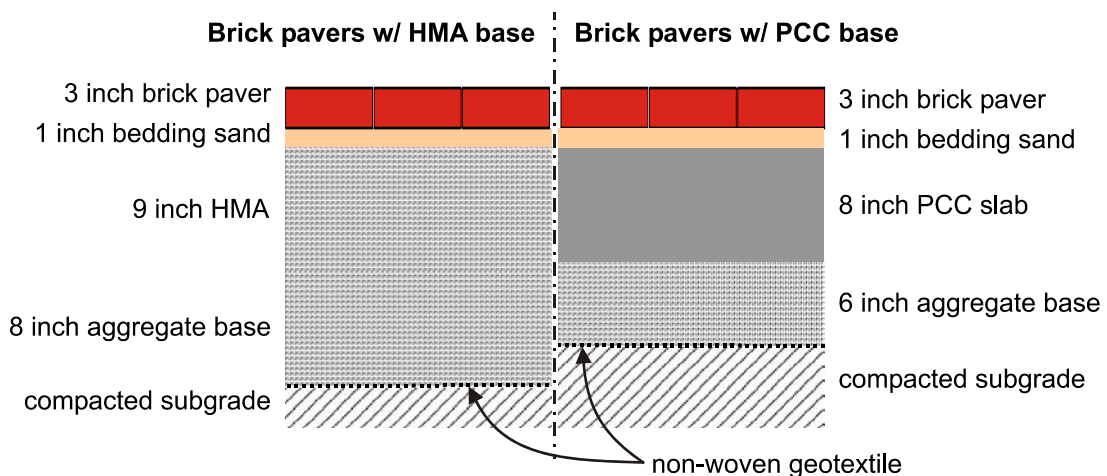
Figure 6. Schematic of Proposed Replacement Crosswalk Design Using Both an HMA and a PCC Base

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Selection of the binder grade for the HMA material should reflect the availability of having a heavy roller (8 to 10 tons) to compact it. If a heavy roller is used, a 70-22 or a 76-22 binder performance grade would provide increased stiffness to the mix but might make the mix more difficult to compact. If a heavy roller cannot be used, a 64-22 binder performance grade should be used such that the desired compaction is achieved. An effective means for fitting a heavy roller into the crosswalk area may be to increase the size of the crosswalk. A heavy roller will be able to achieve a higher density (lower air-void content) more easily, which will offer a significant improvement in pavement performance.

VDOT Class A3 or A4 concrete would be suitable for the crosswalk design using the PCC base. The PCC option may be suitable if underground utilities are located close to the surface where a heavy roller to compact the HMA might pose concerns. Another design option, the use of cast-in-place stamped concrete, may be considered for use in areas of high traffic volumes. A typical design may consist of 8 to 10 inches of tinted concrete using concrete (having a 28-day compressive strength between 3,000 and 4,000 psi) that may be reinforced if desired. Details of a similar crosswalk are described in the following section.

The supporting structure beneath the pavers may be designed similarly to a flexible pavement in accordance with AASHTO design procedures (AASHTO, 1993). This is because the paver, the joint sand, and the bedding sand act as an elastic system under normal traffic loading. As with any flexible pavement, the structural design must be sufficient to distribute the effects of loading to the subgrade over a large area to prevent permanent deformation within the subgrade. However, paver crosswalks are often placed as a retrofitted application where the existing pavement is excavated at the crossing location. Thus, the replaced crosswalk structure is not homogeneous with the surrounding pavement and may not transfer the load over an area greater than the lateral extent of the retrofitted crosswalk. Therefore, it is possible that stress concentrations will occur at the edge of the crosswalk structure causing the effects of loading to be magnified. It is imperative that the structural design consider this effect and account for the response to loading such that the crosswalk structure will not deteriorate prior to reaching the end of the intended service life. In addition, inspection or quality verification during construction of the crosswalk is probably more critical than for a traditional pavement that is placed over a much larger area.

The final selection of a crosswalk design to replace the existing crosswalks should be made based on a combination of the information presented here, material prices, expected service life, and the experience of available contractors.

Examples of Alternative Crosswalk Designs Currently in Use by Other Municipalities

Brick crosswalks were installed in 2003 at particular intersections in downtown Hagerstown, Maryland. These crosswalks consist of a U-shaped reinforced concrete cradle with brick pavers placed in the open end of the "U." The total thickness of the cradle plus brick is designed to be 8 inches resting on top of 4 inches of No. 57 aggregate base. The bricks are placed on a ¼-inch-thick mortar bed. The upturned ends of the U-shaped cradle are 12 inches thick and act as lateral edge restraints. Recent performance shows that the bricks have

undergone significant movement in two of the seven crosswalks installed; a forensic study is forthcoming (personal communication, R. Tissue, City Engineer, Department of Public Works, Hagerstown, Maryland, April 3, 2006). This style of crosswalk replaced an earlier design consisting of brick placed on a sand bed constructed in 1999 that experienced rapid failures. The design of the pavement structure beneath the pavers and sand bed was not obtained.

In downtown Savannah, Georgia, stamped concrete crosswalks were installed in nine intersections along Bay Street. They replaced an earlier brick design that experienced rapid failures (Bay Street carries approximately 25,000 vehicles per day with numerous heavy trucks). The design consists of a 10-foot-wide cast-in-place PCC slab that is 12 inches thick. The concrete material was tinted to achieve the desired color. The crosswalk is bordered by a 15-inch-wide concrete band. Number 4 dowel bars, 18 inches long and placed every 24 inches, are used to transfer loading from the concrete band to the crosswalk structure. These crosswalks are performing well in a high-traffic location (personal communication, C. Purvis, Construction Inspector, Street Maintenance Department, City of Savannah, Georgia; April 10, 2006).

CONCLUSIONS

This study found evidence of permanent deformation within the wheel paths at the crosswalks investigated. The authors attribute the observed permanent deformation to the following (listed in order of influence from greatest to least):

1. *The potential for high vertical compressive strains at the top of the subgrade aggravated by high moisture and fines contents in the subgrade material.* The potential for high vertical compressive strains may be caused where the total in-place HMA thickness was found to be less than designed for six of eight locations.

2. *Additional densification of the HMA surface mix under repeated traffic loading attributable to a high air-void content.*

3. *The potential for formation of a thin and weak layer as a result of mixing of subgrade and base layer (21-A) fines.*

4. *Potential deterioration of stone dust as joint sand that contributes to movement of the brick pavers.* This may result in areas of high stress concentration from poor stress distribution through the crosswalk pavement cross section.

RECOMMENDATIONS

1. *Where the permanent deformation is deemed unacceptable at current in-place crosswalks around the Court Square area, the brick pavers should be removed and the entire crosswalk*

structure rebuilt using one of the two aforementioned designs, i.e., placing pavers on top of either HMA or PCC. In addition, the following are recommended:

- Treatment of weak subgrade is highly recommended to improve the load-carrying capabilities of the subgrade. Lime treatment is recommended for fine-grained clay or silty clay soils; cement treatment may be used in areas having sandy or coarse grained soils. Five percent by volume for lime treatment and 10% by volume for cement treatment are typical in Virginia. The authors recognize that subgrade treatment may not be realistic in an enclosed trench-like location such as that seen with crosswalk construction.

- A layer of non-woven geosynthetic fabric should be placed on top of the subgrade prior to placing the aggregate layer to prevent intermixing.

- In-place cement treatment of the base layer is highly recommended to reduce the vertical compressive strains on top of the subgrade and if grade restrictions preclude placing the additional pavement thickness as detailed in the recommended designs.

- A layer of manufactured concrete bedding sand compacted to a 1-inch thickness should be used beneath the pavers. This layer has a desirable confining effect under repeated traffic loading.

- Manufactured masonry sand should be used as joint sand between the pavers with additional material added periodically.

2. *All other crosswalks in the Court Square area should be monitored to determine if unacceptable deterioration develops.* Areas with low traffic or material conditions different from those tested in this study may undergo different rates of permanent deformation.

3. *The City of Charlottesville should implement the following specification revisions for future construction of crosswalks:*

- Require subgrade lime treatment or cement treatment of weak subgrade soils in areas with high traffic volumes.

- Require proof rolling of the subgrade where crosswalks are to be placed, following excavation of the existing pavement, to identify weak areas of subgrade needing replacement or improvement.

- Require a layer of non-woven geosynthetic fabric to act as a separator between the subgrade and an aggregate base layer. This layer will also enhance the ability to resist tensile strain at the interface between layers when used with weak soils.

- If HMA materials are placed as part of the crosswalk construction, require and monitor density measurements for quality control. At a minimum, the measurements should be taken in areas where the expected wheel paths are located.

- Discontinue use of stone dust for joint sand during paver crosswalk construction.

- Discontinue the use of a neoprene asphalt adhesive beneath the pavers.

- Require the use of manufactured concrete sand for bedding sand and manufactured masonry sand for joint sand during paver crosswalk construction.

- Require the contractor to reapply joint sand approximately 9 to 12 months following initial construction.

COSTS AND BENEFITS ASSESSMENT

This study showed that minor changes to design procedures and specifications could potentially increase the service life of brick crosswalk structures used in urban conditions. If an approximate construction cost is $25 per square foot, the construction of four crosswalks within each intersection (assuming two 10-foot travel lanes, two 8-foot parking lanes, and a length of 8 feet) costs nearly $30,000. It is anticipated that the design recommendations given herein should yield a service life of 15 years. If the same costs are incurred in constructing crosswalks with the current service life (approximately 3 years), the recommended design and specification changes could result in a savings of $120,000 per intersection over 15 years from reducing the need to reconstruct on a frequent basis. This cost difference does not include costs associated with delays to the traveling public or inconvenience to nearby businesses.

ACKNOWLEDGMENTS

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APPENDIX A
MATERIAL GRADATIONS

Table A1. VDOT Gradation for SM-2A and BM-3 (VDOT, 1993)

Mix Type	Percentage by Weight Passing Square Mesh Sieves										
	2 in	1½ in	1 in	¾ in	½ in	3/8 in	No. 4	No. 8	No. 30	No. 50	No. 200
SM-1					100	94-100	65-85		20-40	9-25	4-8
BM-3	100	97-100	70-86		46-64		26-44		8-18		2-5

Table A2. VDOT Gradation for 21-A Aggregate Base Material (VDOT, 1993)

Size No.	Amounts Finer Than Each Laboratory Sieve (Square Openings), Percentage by Weight					
	2 in	1 in	3/8 in	No. 10	No. 40	No. 200
21-A	100	94-100	63-72	32-41	14-24	6-12

**Table A3. Gradation Specifications for Bedding and Joint Sands**

Sieve Size	Percentage Passing by Weight	
	Bedding Sand (ASTM C33)	Jointing Sand (ASTM C144)
3/8 inch	100	100
No. 4	95-100	100
No. 8	80-100	95-100
No. 16	50-85	70-100
No. 30	25-60	40-75
No. 50	10-30	10-35
No. 100	2-10	2-15
No. 200	0-3*	-**

*Allowed to increase to 5% if no clay or shale is present.
**Allowed to increase to 0%-10% for manufactured sand.
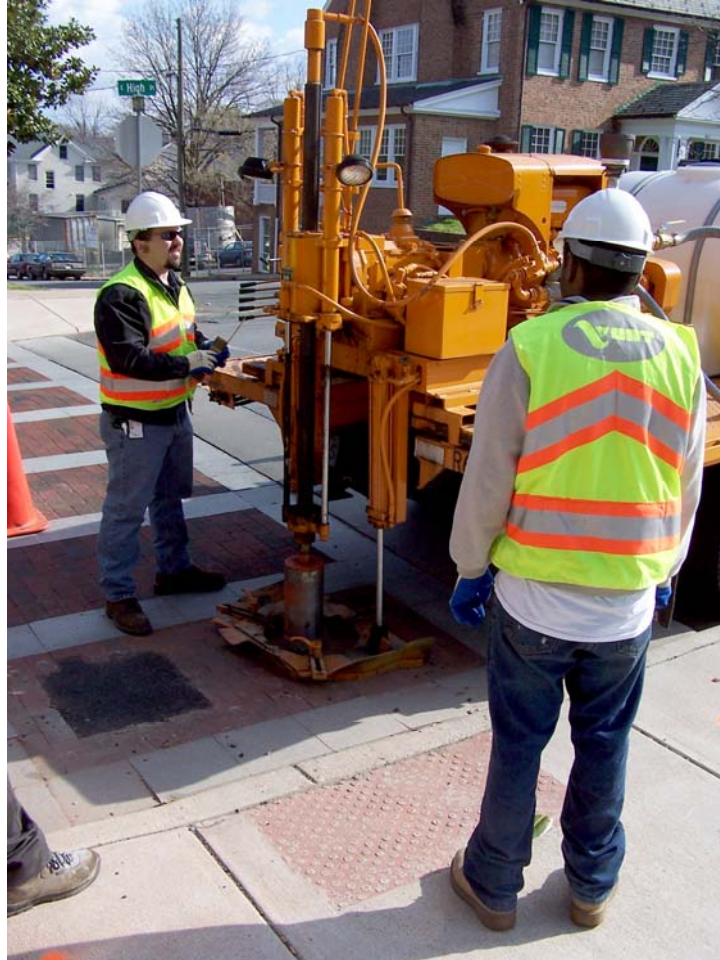
Figure B1. Specimen Collection by Coring at Intersection of 4th Street and High Street

Figure B2. Specimen Collection of Aggregate Base at Intersection of 4th Street and High Street

Figure B3. Measurement of HMA and 21-A Aggregate Base Layer at Intersection of 4th Street and High Street (approximately 7 and 5 inches, respectively)

Figure B4. Example of Non-Uniformity in Particle Size Distribution and HMA Layer Thickness (specimens collected at intersection of 4th Street and Jefferson Street)