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**ADEQUACY OF IN-PLACE QC/QA TECHNIQUES FOR EVALUATING
CONSTRUCTED AGGREGATE LAYERS OF WORKING PLATFORMS AND
FLEXIBLE PAVEMENTS**

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Adequacy of In-Place QC/QA Techniques for Evaluating Constructed Aggregate Layers of Working Platforms and Flexible Pavements

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ABSTRACT

Quality assurance is extremely important for satisfactory end performance of a constructed pavement. Traditional quality control and quality assurance (QC/QA) procedures based on volumetric and surface property checks are becoming outdated when constructing pavement foundation layers and ensuring pavement longevity. Recent emphasis in QC/QA procedures have shifted from a density based approach to stiffness and strength based approaches with newly adopted advanced technologies. However, the necessity of QC/QA is often overlooked when constructing low volume roads with unbound aggregate layers, which may be built using recycled or out of specification marginal quality materials, nowadays more common as sustainable practices. This paper summarizes key findings from QC/QA tests performed on full-scale pavement test sections in a recent Illinois Center for Transportation research study. The focus was to validate newly adopted Illinois DOT material specifications for large size unconventional aggregates, known as aggregate subgrade, through accelerated pavement testing. Seven representative aggregate types were used to construct test sections with aggregate subgrade and virgin and recycled capping and subbase layers. Density measurements from nuclear gauge were collected and routinely contrasted with modulus results of the lightweight deflectometer (LWD) and soil stiffness gauge (GeoGauge) from the constructed layers. Further, forensic strength assessment was carried out by dynamic cone penetrometer and variable energy PANDA penetration device. Geo-endoscopic imaging, coring and trenching were also conducted to identify depth of water table and as-constructed layer thicknesses. The PANDA penetrometer results in conjunction with geo-endoscopy proved to be effective in correlating rutting performances to QC/QA test results.

Key Words: Quality Control, Quality Assurance, Unbound Aggregate, Aggregate Subgrade, Lightweight Deflectometer, GeoGauge, Variable Energy PANDA Penetrometer, Geo-Endoscopy

1 INTRODUCTION

2 Low volume roads, often constructed by local agencies, constitute in the United States
3 approximately 68% and 72% of total lane miles in rural and urban highway systems, respectively.
4 Moreover, about 34% of total road network encompassing both rural and urban highway system
5 consists of unpaved roads (1, 2). Unbound aggregate layers are the integral structural components
6 of these pavements, and they are routinely used to build construction working platforms over weak
7 subgrade for placement and compaction of overlying bound layers consisting of hot mix asphalt
8 (HMA) or Portland cement concrete (PCC). Such layers are also introduced in pavement systems
9 to ensure adequate drainage or to prevent freeze-thaw damage.

10 Although proper characterization of granular materials is extremely important
11 considering the vast network of paved and unpaved roads, conventional recipe-based design
12 approaches often overlook the important mechanistic characterization and quality requirements of
13 unbound aggregate layers. Despite recent push for mechanistic-empirical (M-E) design
14 implementation, state of the art Pavement ME analysis and design software still does not address
15 issues like stress-dependency and stress induced anisotropy without which characterization of
16 unbound aggregates remains incomplete (3). Owing to budget constraints, sustainable design and
17 construction practices, transportation agencies, nowadays heavily emphasize on the enhanced use
18 of recycled and out of specifications marginal quality materials, especially for use in low volume
19 roads. Conversely, empirical damage models were developed based on performance of high quality
20 crushed stone materials. That's why, local calibrations of these damage models are imperative in
21 light of material source compositions.

22 Quality assurance programs aspire the monitoring agency and the contractor to build high
23 quality pavements with longer service lives with an incentive to gain financially on both sides (4).
24 Traditional in place quality control and quality assurance (QC/QA) procedures incorporate test
25 methods that determine volumetric and surface property checks for pavement materials. Besides
26 the above mentioned properties, dynamic cone penetrometer (DCP) is also frequently used to
27 estimate various correlated strength indices because of operational simplicity. Contrary to these,
28 recent advances in nondestructive testing (NDT) methodologies have prompted transportation
29 officials to utilize lasers, ground-penetrating radar, light and falling weight deflectometers (LWD
30 and FWD), automated penetrometers and seismic assessment technologies. Since layer modulus
31 is a key material property for M-E design procedures, NDT test procedures that can furnish
32 stiffness properties are slowly making their way to agency acceptance plans.

33 Apart from these technologies, continuous compaction control relating drum harmonics
34 to soil/aggregate compaction characteristics is also gaining momentum as a favorable QC/QA
35 program (5). Federal Highway Administration (FHWA) has been guiding a nationwide effort for
36 implementation of intelligent compaction (IC) technology through various research projects (6).
37 IC technology for continuous compaction control requires significant scientific expertise and fiscal
38 allocation. Note that adoption of such technology is still at the experimentation stage and quite
39 unlikely for low volume roads and unsurfaced pavements in near future.

40 This paper focuses on the effectiveness of traditional and emerging QC/QA techniques
41 for end performance characterization of unconventional granular materials and low cost pavement
42 alternatives. QC/QA results were obtained from a recent study at the University of Illinois
43 involving accelerated pavement testing of full scale test sections constructed with large size
44 unconventional aggregates from quarry primary crushers and recycled sources (7). Twelve
45 different working platforms and 12 different flexible pavement sections were constructed over
46 engineered subgrade with six types of aggregates varying in source compositions (7). As part of

1 the subgrade stability requirements, Illinois Department of Transportation (IDOT) primarily uses
2 unbound aggregates to build construction working platforms over weak subgrade. In a recent
3 Bureau of Design and Environment special provision, IDOT has introduced three new gradation
4 bands namely, CS01, CS02 and RR01, respectively to encourage the enhanced use of large-sized
5 and recycled materials for soft subgrade remediation. Details of these gradation bands can be found
6 elsewhere (8). These materials are often referred to as “aggregate subgrade.” As these gradations
7 allow particle sizes as large as 152 to 203 mm (6 to 8 in.), standardized laboratory test protocols
8 cannot characterize the material performance due to dimension specific requirements. To this end,
9 this research study investigated field performance trends of six aggregate subgrade materials from
10 accelerated pavement testing. Associated QC/QA properties are examined in this paper with
11 consideration given to rutting performance and post-construction forensic analyses.

12 REVIEW OF QC/QA SPECIFICATIONS

13 Compaction quality control of constructed subgrade and granular layers can either be density based
14 or strength/stiffness based. In a recent NCHRP Synthesis, Nazzal (2014) reported that only five
15 out of the fifty state transportation agencies have adopted stiffness/strength based compaction
16 control specifications (9). Among these five state agencies, Indiana and Minnesota Departments
17 of Transportation (DOTs) have deflection or stiffness based requirements for compaction control
18 of subgrade and granular layers. Many other DOTs still follow density based compaction
19 specifications.

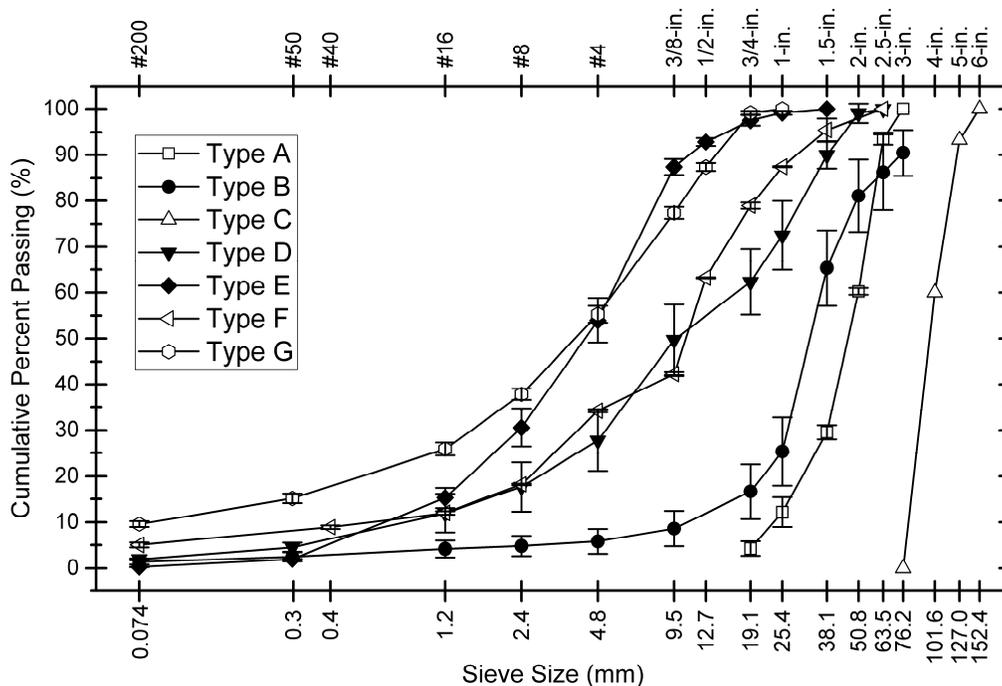
20 IDOT Subgrade Stability Manual (SSM) requires subgrade modification for working
21 platforms, if the strength index (unsoaked California Bearing Ratio [CBR], also known as Illinois
22 Bearing Value [IBV]) of the untreated soil is below 6% and accumulates more than 12.7 mm (1/2
23 in.) sinkage/rutting under construction traffic (10). Similarly, the compacted subgrade should have
24 a minimum 95% relative compaction with respect to standard Proctor maximum dry density
25 (ASTM D698). For aggregate base courses, IDOT has similar compaction requirements. In case
26 of the HMA layer, the field density should be in the range of 94-98% of the theoretical maximum
27 density (11). According to the Indiana Test Method (ITM) No. 508-12T, if a lightweight
28 deflectometer (LWD) exhibits 10% or higher change in deflection for any two consecutive drops,
29 the corresponding material shall require additional compaction (12). Moreover, ITM No. 514-15T
30 dictates that satisfactory compaction of soil or unbound layer can be assumed if the difference
31 between the average LWD test values from four and five roller passes is equal to or less than 0.02
32 mm. The same specification requires the in place average DCP blow count for a 152.4 mm (6-in.)
33 lift to be equal to that of laboratory established DCP target value (13).

34 Minnesota DOT specifications require 100% relative compaction in subgrade and
35 granular layer with reference to standard maximum dry density (14). In addition, the same
36 specification also restricts DCP penetration to a minimal value based on the gradation and moisture
37 condition (14, 15). For a granular base course, Missouri Standard Specifications for Highway
38 Construction require an average DCP penetration index (DPI [$= \frac{\text{Reading after 5 blows} - \text{Reading after 2 blows}}{3}$]) to be less than or equal to 10 mm (0.4 in.) per blow (16). This DPI value
39 corresponds to an equivalent of CBR of 10%. On a similar note, the maximum allowable DPI for
40 cohesionless soil is 35 mm (1.4 in.) according to Iowa DOT specifications (17). All these
41 specifications were developed using regular size base course type aggregates. Conversely, this
42 study involves the application of unconventional large rocks. To this end, conventional norms of
43 QC/QA as outlined above were not strictly followed during the construction.
44

1 In 2008, the European Union developed a compaction control specification based on
 2 lightweight-deflectometer (LWD) measured dynamic modulus and dynamic compactness rate (18).
 3 According to this specification, corresponding LWD testing involves six sequences of LWD drops
 4 on the loose, non-compacted material at the site. This sequential dynamic loading is assumed to
 5 replicate the modified Proctor effort. Similarly, United Kingdom specifications classify base
 6 course materials into four different categories with limiting modulus values (19). The same
 7 specification also requires a minimum subgrade modulus of 30 MPa (4.4 ksi).

8 **FIELD EXPERIMENT**

9 Figure 1 exhibits the particle size distributions of selected aggregate materials studied in the field
 10 experiment. Type A through F aggregates were selected as the aggregate subgrade; meanwhile
 11 Type G and E were used for capping and subbase layers. The empty and full symbols represent
 12 the virgin and recycled material compositions, respectively. Both Type A and Type C aggregates
 13 were uniformly graded loosely fitting RR01 and CS02 IDOT gradation bands, respectively. Type
 14 B aggregate subgrade material consisted of relatively well graded 100% recycled concrete
 15 aggregates (RCA). A 60%-40% blend of RCA and reclaimed asphalt pavement (RAP) materials
 16 was selected as the Type D aggregate subgrade. Although both Type B and D were expected to fall
 17 within the gradation specifications of CS01, their particle size distributions were similar to that of
 18 Type F virgin aggregates conforming to CA02 gradation envelope. Among the seven aggregate
 19 types, Type G dolomite was the densest with 10% of dry mass passing through No. 200 (0.074
 20 mm) sieve. Type E aggregate subgrade, capping and subbase materials were constituted of 100%
 21 RAP materials with negligible amount of fines content.
 22



23

24

Figure 1: Particle size distributions of aggregate subgrade and capping/subbase materials

1 Table 1 lists the design thicknesses for the proposed test sections in this study. A 105.2 m
 2 long (345 ft) test road encompassing 24 different test sections were constructed. On the north side
 3 of test road, twelve working platform sections were designed such that each of the test sections
 4 was 2.7 m (9 ft) wide and 4.6 m (15 ft) long. South side of the test road consisted of twelve different
 5 flexible pavement sections with similar dimensions. Prior to construction, moisture-density
 6 relationships of in situ soil were established in the laboratory. The subgrade was then engineered
 7 to two different controlled strengths through an iterative process of moisture addition and in place
 8 assessment of density and strength through nuclear gauge and DCP tests. A well-established
 9 correlation developed by Kleyn et al. was used to determine the CBR strength of the soil with
 10 respect to the penetration indices (20). Details of the subgrade engineering procedure can be found
 11 elsewhere (7). Notably, the same correlation was also used to determine the CBR profiles of
 12 granular layers. Two-thirds of existing subgrade in the test road were engineered to have a
 13 controlled strength of CBR = 1%; whereas, the remainder was modified to achieve a design
 14 strength of CBR = 3%.

15 Table 1: Design Thicknesses of the Pavement Test Sections

Working Platform			Flexible Pavement				Material Type ⁴	Subgrade CBR (%)	
Section ¹	Thickness (cm)		Section ¹	Thickness (cm)					
	Cap ²	AS ³		HMA	Subbase	AS ³			
<i>WP⁵-I</i>	7.6	53.3	<i>FP⁶-I</i>	10.2	15.2	53.3	A	1	
<i>WP-II</i>	7.6	53.3	<i>FP-II</i>	10.2	15.2	53.3			
<i>WP-III</i>	7.6	53.3	<i>FP-III</i>	10.2	15.2	53.3			
<i>WP-IV</i>	7.6	53.3	<i>FP-IV</i>	10.2	15.2	53.3	B		
<i>WP-V</i>	7.6	53.3	<i>FP-V</i>	10.2	15.2	53.3	C		
<i>WP-VI</i>	7.6	53.3	<i>FP-VI</i>	10.2	15.2	53.3			
<i>WP-VII</i>	7.6	53.3	<i>FP-VII</i>	10.2	15.2	53.3	D		
<i>WP-VIII</i>	7.6	53.3	<i>FP-VIII</i>	10.2	15.2	53.3			
<i>WP-IX</i>	7.6	22.9	<i>FP-IX</i>	10.2	15.2	22.9	E		3
<i>WP-X</i>	7.6	22.9	<i>FP-X</i>	10.2	15.2	22.9			
<i>WP-XI</i>	7.6	22.9	<i>FP-XI</i>	10.2	15.2	22.9	F		
<i>WP-XII</i>	7.6	22.9	<i>FP-XII</i>	10.2	15.2	22.9			

¹Odd numbered section: **Type G** (Dolomite) capping/ subbase;

Even numbered section: **Type E** (RAP) capping/ subbase;

²Cap = Capping aggregates; ³AS = Aggregate subgrade;

⁴Aggregate subgrade material type; ⁵WP = Working platform; ⁶FP = Flexible pavement.

16 For working platforms, layer thicknesses were selected in accordance with IDOT *SSM*.
 17 For example, a 61.0 cm (24 in.) thick aggregate cover is required for construction platforms in
 18 CBR=1% subgrade condition. Capping layer thickness was selected to be 7.6 cm (3 in.) such that
 19 the requirement for total aggregate cover could be met within the bounds of special provision for
 20 aggregate subgrade materials. This means that a 53.3 cm (21 in.) thick aggregate subgrade was
 21 placed over the CBR = 1% subgrade followed by a 7.6 cm (3 in.) capping layer. For flexible
 22 pavements on the south side, an additional 7.6 cm (3 in.) thick compacted lift of the same material
 23 was placed on top of capping layer. After the placement and compaction of 15.2 cm (6 in.) thick
 24

1 dense graded materials over the aggregate subgrade, a 10.2 cm (4 in.) thick HMA layer was
2 compacted in two equal lifts of binder and surface courses. Each of the aggregate subgrade types
3 had two different capping/subbase aggregates placed on top. The odd numbered sections had Type
4 G dolomite capping (/subbase) layer; whereas the even numbered sections were capped with Type
5 E 100% RAP materials. Aggregate subgrade materials with relatively large size particles (Type A
6 through D) were applied on top of CBR=1% subgrade; whereas, regular sized Type E and Type F
7 materials were placed over a relatively stronger CBR=3% subgrade.

8 The aforementioned sections were designed such a way that four consecutive test sections
9 could be subjected to accelerated pavement testing (APT) simultaneously. The Advanced
10 Transportation Loading Assembly (ATLAS) at the ICT full scale test facility was used for APT.
11 Adequate buffer zones were provided to maintain a constant moving wheel load of 44.5 kN (10
12 kips) at a speed of 8 km/h (5 mph) applying approximately 758 kPa (110 psi) of contact stress.
13 Quality control tests like nuclear gauge density, modulus measurements with LWD and soil
14 stiffness gauge (also known as GeoGauge) were conducted at the center of each test section.
15 Periodic rut measurements were carried out at two different locations each 1.5 m (5 ft.) apart from
16 transverse edges (west and east) of the corresponding test section. For brevity, the average of the
17 two measurements were reported in this paper. Rut measurements were conducted in working
18 platforms at an interval of 5.1 cm (2 in.) over a center span of 152.4 cm (5 ft). Contrary to that, rut
19 measurements in flexible pavements were taken every 2 mm over a center span of 81.0 cm (32-
20 in.). Details of the rut measurement equipment and APT facility can be found elsewhere (7, 21).

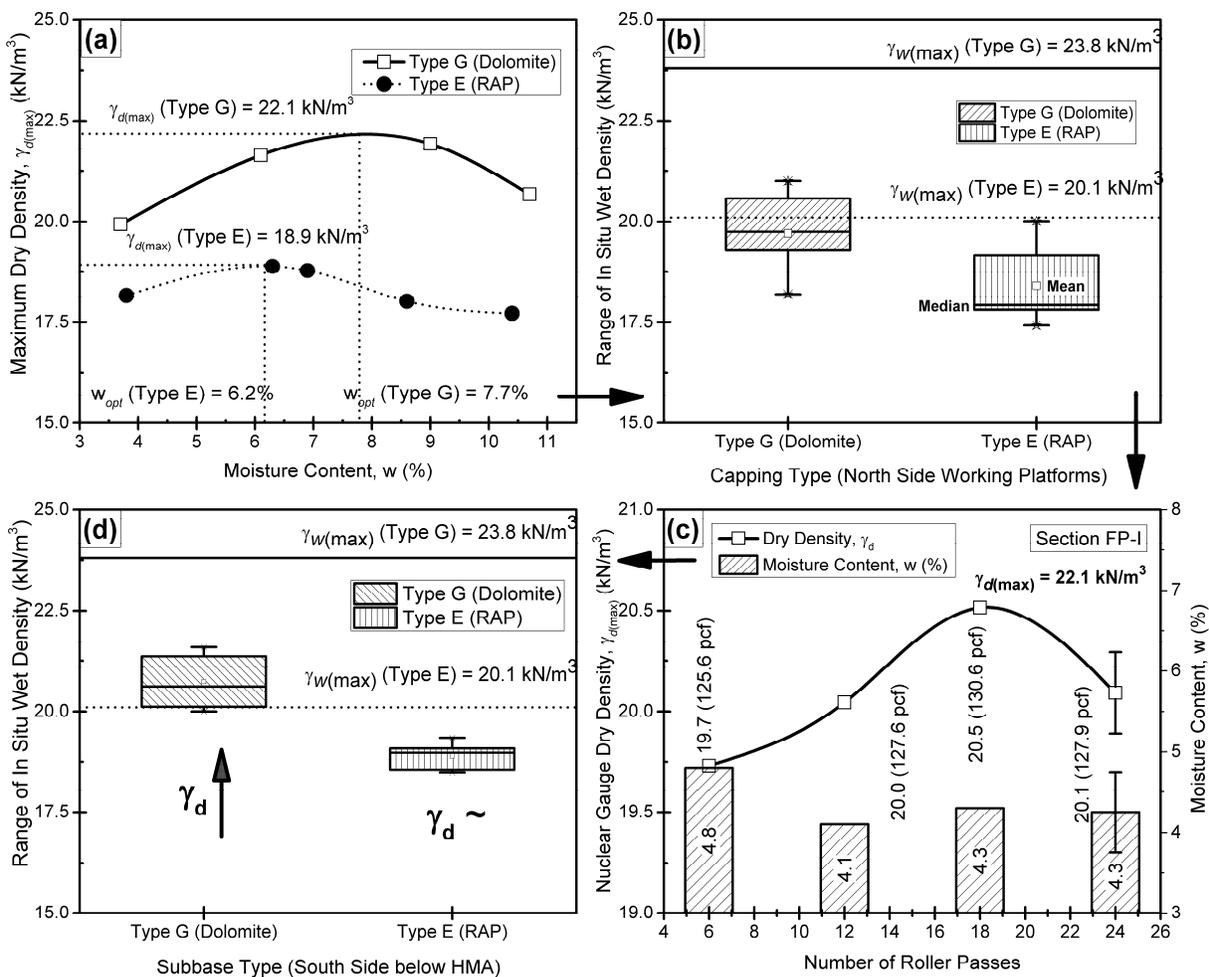
21 **QC/QA MEASUREMENTS DURING CONSTRUCTION**

22 According to the NCHRP Synthesis 456, LWD, GeoGauge and DCP are the most commonly used
23 non-nuclear devices for compaction control (9). Henceforth, all of these three devices were used
24 for compaction quality control of engineered subgrade and unbound granular layers. LWD and
25 GeoGauge were used for modulus assessment of compacted layers; whereas, DCP was used for
26 strength assessment of corresponding layers. Illinois DOT engineers performed the nuclear gauge
27 density tests for quality assurance of the constructed layers. In addition, in situ subgrade moisture
28 contents were evaluated with microwave in compliance with ASTM D4643. Considering the
29 limited scope of this paper, QC/QA tests on engineered subgrade and finished subbase are omitted
30 intentionally. QC/QA tests on HMA layer involved determination of existing pavement elevation
31 and nuclear gauge density. The following subsections highlights the key findings related to the
32 QC/QA testing techniques.

33 **Laboratory Compaction vs. In Place Density**

34 Figure 2 summarizes the sequences in the development of a compaction growth curve for Type G
35 virgin aggregates. According to Figure 2(a), the maximum dry density and optimum moisture
36 content for Type G dolomite were found to be about 17% and 24% higher than those of Type E
37 RAP. Type G dolomite capping was also found to be more sensitive to moisture variation than
38 Type E RAP. Initially, the aggregate subgrade capping layers on the north side were compacted
39 with 12 roller passes. Considering the thin lift of capping and subbase layers, all of the recorded
40 densities were measured with back-scatter method. The reported moisture contents in even
41 numbered sections were corrected for hydrogen-bound material influence and the details of the
42 correction procedure is discussed elsewhere (7).
43

1 Overall distribution of nuclear gauge densities for the two different capping types are
 2 shown in the box-plot of Figure 2(b). The solid black line in the same graph denotes the laboratory
 3 maximum wet density for dolomite; the dotted black line identifies the laboratory maximum wet
 4 densities for RAP aggregates. Overall, RAP capping exhibited better relative density compared to
 5 Type G dolomite. Oven dried loose samples collected during the placement of capping aggregates
 6 indicated in-place moisture contents in the range of 5.2% to 5.5% for both the virgin and RAP
 7 aggregate capping materials. Considering the minimal peak of moisture-density curve and closer
 8 proximity to optimum moisture content, the RAP capped working platforms ended up with lower
 9 densities but higher degrees of compaction. Note that the granular matrix of aggregate subgrade
 10 layer consisting large rocks was inherently unstable because of large voids. Moreover, both
 11 capping aggregate types had dry of optimum moisture conditions making them even harder to
 12 compact. The thin lifts of the compacted layers (both Type G and E) thus failed to reach the
 13 minimum required 95% of relative compaction.



14
 15 Figure 2: (a) Laboratory compaction characteristics of capping aggregates; (b) Range of achieved
 16 densities in capping layer; (c) Compaction growth curve developed in Section FP-I; (d) Range of
 17 achieved densities in subbase layer

1 Accordingly, a compaction growth curve was developed for the aggregates to be overlain
2 as subbase layers in Section FP-I on the south side of test road. As shown in Figure 2(c), the highest
3 achievable in-place density was recorded at 18 roller passes and found to be 20.5 kN/m^3 , resulting
4 in approximately 92% relative compaction. Throughout the compaction process on subbase layer,
5 18 vibratory roller passes were used. Despite the increasing number of roller passes, Figure 2(d)
6 clearly shows that the resulting densities for Type G virgin aggregate capping were significantly
7 low compared with the laboratory maximum dry density. Similar to the trend in capping layer,
8 Type E RAP also exhibited lower densities in comparison with the crushed dolomite. However,
9 Type G dolomite achieved higher densities compared with those recorded for capping layers. In
10 contrast, RAP subbase sections did not show any improvement in terms of density achieved. This
11 finding indicates that in situ densities of RAP were insensitive to the increase in compactive effort.
12 Presence of visco-elastic asphalt mastic in RAP might have contributed to the absorption of some
13 of the compaction energy exerted by the higher number of vibratory roller passes.

14 **Stiffness Measurements**

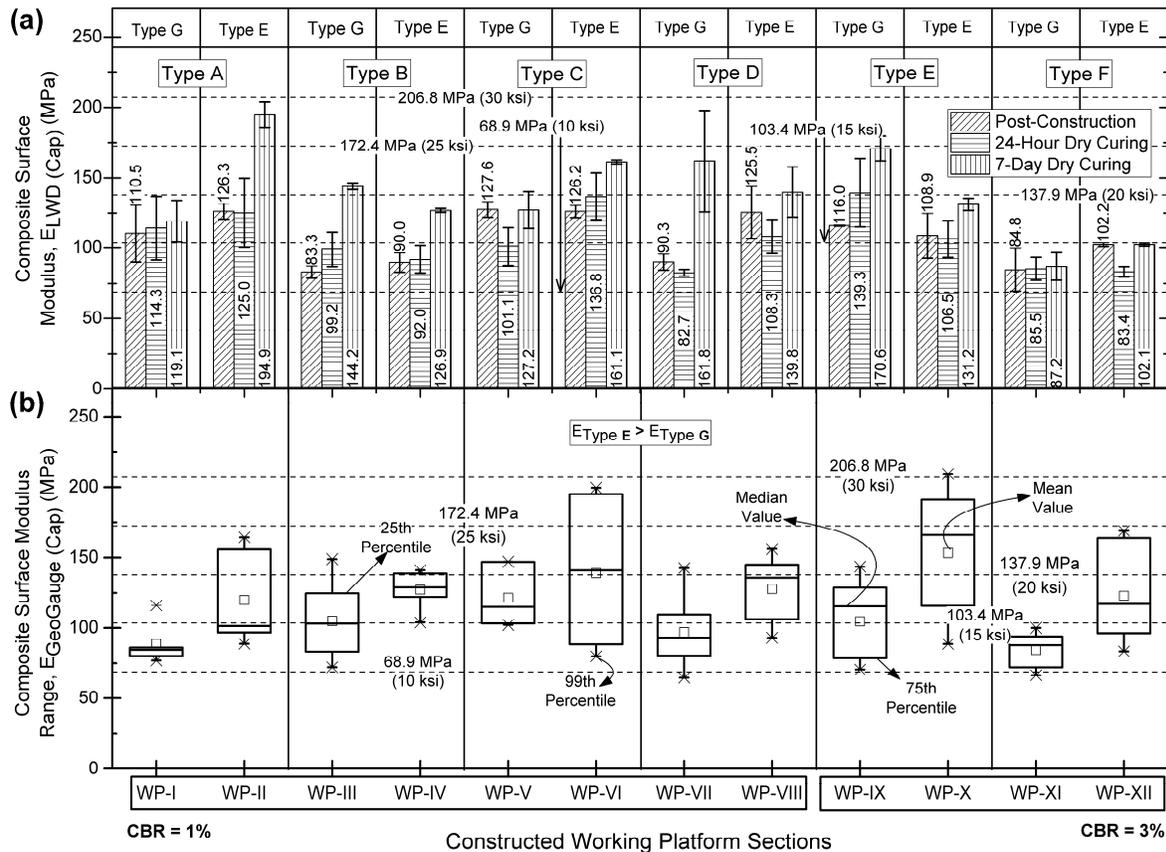
15 In place surface moduli were measured with two different approaches. For the deflection based
16 method, an initial seating load was applied followed by nine drops of dynamic loads on finished
17 surface of the working platforms. The sequential loading scheme involved three drops for each of
18 three different drop-heights. For a simple interpretation, Boussinesq's half-space equation was
19 used to calculate the composite layer moduli. Drop heights and plate radii were selected such that
20 the resulting stress level was within the expected range of stress state of a granular layer in an
21 actual pavement. In case of steady state vibratory method, the GeoGauge measures the impedance
22 at the surface of soil or unbound layer imposing small displacements and stresses through the use
23 of 25 steady-state frequencies between 100 to 196 Hz. For stiffness properties, at least four sets of
24 measurement were taken by placing the GeoGauge device over a thin sand bed and rotating it 90°
25 from its previous position.
26

27 ***Deflection Based Method***

28 Figure 3 (a) shows the LWD measured moduli in the working platforms at different stages of
29 construction. All of the sections exhibited stiffness gain after one week of dry curing. Section WP-
30 VII with recycled blend of aggregate subgrade and dolomite capping indicated the highest increase
31 in modulus values. On the other hand, that section also exhibited significant variation in modulus
32 values during the sequential drops. Next to Section WP-VII, Section WP-II also exhibited 54%
33 stiffness gain over the seven-day period. Sections with Type F (virgin limestone) aggregate
34 subgrade exhibited the lowest moduli values among the six different aggregate subgrade types.
35 This might be an indication of poor compaction in the corner sections or an effect of lower
36 thickness compared to the design depth. Mooney and Miller (16) reported that the depth of
37 influence is 0.9-1.1 times the plate diameter. Henceforth, the reported moduli are likely to be
38 affected by the aggregate subgrade. Similarly, if the total aggregate covers in Section WP-XI and
39 WP-XII are lower than 30.5 cm (12-in.), the resulting composite modulus values are expected to
40 be lower due to subgrade condition.

41 Conversely, higher moduli in the working platform sections especially for thicker granular
42 cover can be attributed to three possible scenarios: (I) the pavement layers were well compacted
43 and there was negligible amount of voids in the granular matrix for particle reorientation under
44 moving traffic; (ii) since the moisture level in capping materials and aggregate subgrade were on

1 the dry side of optimum, matric suction might have contributed to the stiffness gain; or (iii) higher
 2 moduli might also have resulted from pore water pressure buildup during transient LWD loading
 3 caused by shallow water table. Apeageyi and Hossain (17) observed misleadingly high moduli
 4 values in weak saturated soils with LWD and made similar remarks. Apart from these reasons,
 5 stiffness gain in dolomite capping layers might also be an effect of carbonate cementation through
 6 dissolution-precipitation of fines fraction as reported in a previous study (22).



8

9 Figure 3: Ranges of (a) LWD moduli recorded on capping layer at different phases of
 10 construction and (b) Composite moduli measured with GeoGauge on capping layer

11 **Steady State Vibratory Method**

12 Unlike the LWD, GeoGauge reported composite moduli were more representative of the capping
 13 aggregates owing to the lower influence depth. Figure 3 (b) shows the distribution of GeoGauge
 14 reported moduli values in box-plot formation. To this end, the RAP capped working platforms
 15 consistently exhibited higher modulus and variability in successive measurements. Although
 16 several previous studies had reported significantly low variability in consecutive GeoGauge
 17 measurements over subgrade (9), the highest coefficient of variation in the current study was found
 18 to be about 34%. Notably, GeoGauge is extremely sensitive to the stiffness of top 5.1 cm (2 in.) of
 19 tested soil layer. Therefore, the repeatability of this device was significantly diminished owing to
 20 the thin lift of dense graded aggregates underlain by variety of aggregate subgrade layers used in
 21 this study. Such high variability also reflects the sensitivity of this device to the sand-bed seating
 22 over dense graded capping with large size aggregate subgrade. The highest GeoGauge modulus

1 was reported for Section WP-X which had the same Type E 100% RAP capping materials and
 2 aggregate subgrade. The trend of higher modulus in RAP capping is consistent to the findings of
 3 previous laboratory studies on resilient behavior of RAP. These studies have reported an increase
 4 in resilient modulus with the increase in RAP percentage of certain aggregate blend (23–27).

5 **RUTTING ACCUMULATION UNDER ACCELERATED PAVEMENT TESTING**

6 Figure 4 presents the rutting progression in full scale test sections with increasing number of
 7 passes. The top two graphs summarize the rutting trends in working platforms; whereas, the bottom
 8 two graphs show the rutting accumulation in flexible pavement sections. The horizontal dash line
 9 designates the failure criterion for rutting. The empty symbols in Figure 4 designate dolomite
 10 capping or subbase; meanwhile, the solid black symbols represent Type E RAP capping or subbase.
 11 Rutting trends in working platforms and flexible pavements were further subdivided on the basis
 12 of engineered subgrade strength. As indicated in Figure 4, the rutting criteria for construction
 13 platforms and flexible pavements were selected to be 7.6 cm and 1.27 cm (3 in. and ½ in.),
 14 respectively.

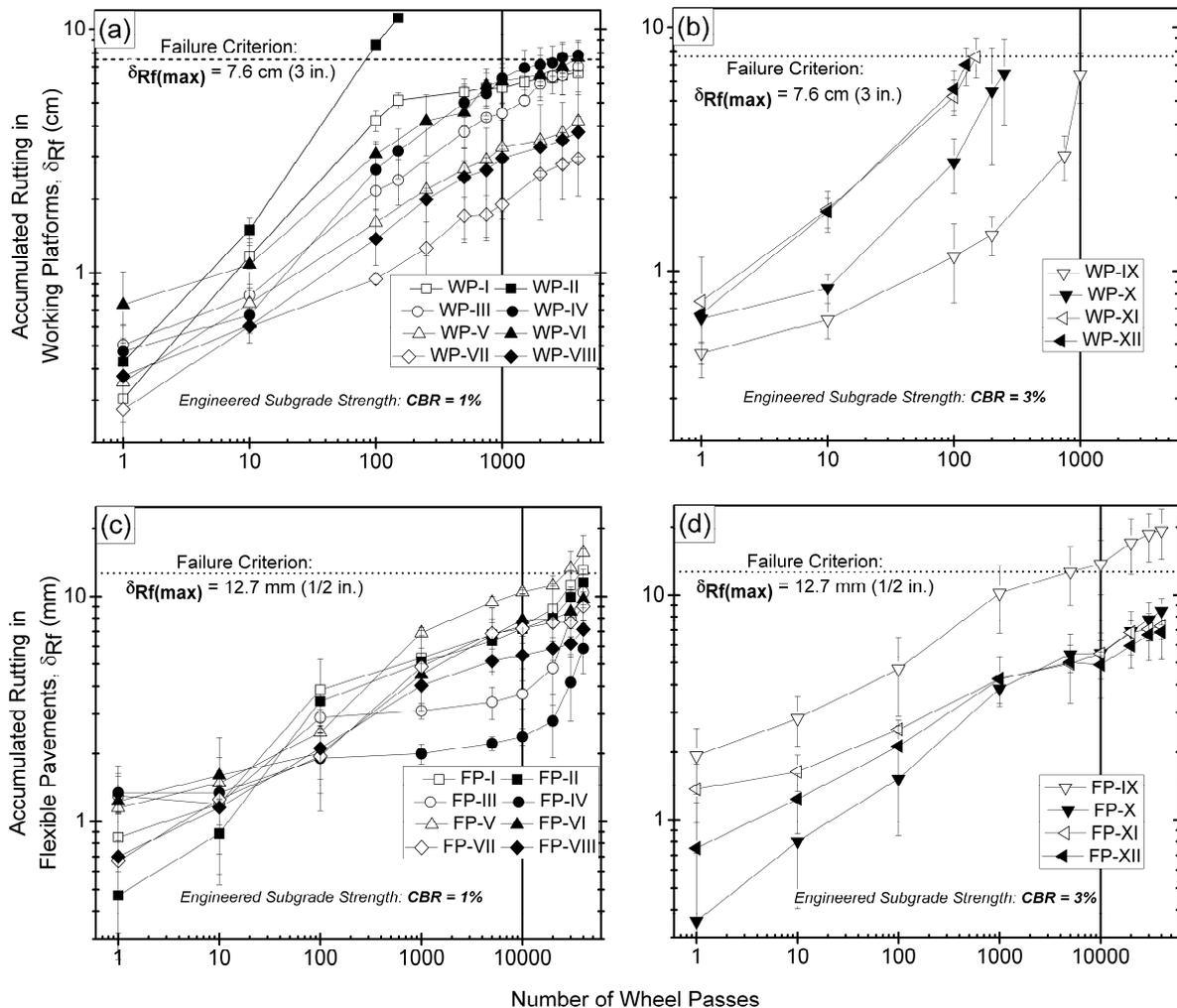


Figure 4: Rutting progression trends in constructed test sections

15
 16
 17

1 According to Figure 4 (a) and (b), large size aggregates over weaker subgrade superseded
2 the regular sized aggregates over CBR=3% subgrade owing to the thicker granular cover. Except
3 Section WP-II, all of the sections over CBR=1% subgrade survived 4,000 passes. Section WP-VII
4 accumulated the least amount of rutting. This section also exhibited significantly high modulus
5 value (7-day dry curing) during the LWD tests. Similarly, poor rutting performance in Sections
6 WP-XI and WP-XII can also be substantiated by the low LWD moduli on construction platform
7 surface. However, Section WP-II exhibited premature failure despite having the highest modulus
8 response with LWD. This might be an indication for shallow water table leading to excessive pore
9 water pressure. On a similar note, Minnesota DOT restricts the use of LWD on unbound layers if
10 the water table exists within the depth of 1 m (3 ft.) (14).

11 The dolomite capped working platforms consistently exhibited less rutting compared with
12 RAP counterparts. However, flexible pavement sections showed exactly opposite trend in terms of
13 rutting progression (see Figure 4 (c) and (d)). Section FP-IX with Type G dolomite subbase and
14 Type E aggregate subgrade failed prematurely within 10,000 passes accumulating the highest
15 amount of rutting in flexible pavement sections. Up to 40,000 passes, the effect of thicker granular
16 cover was subsided by the presence of bound HMA layer unlike the construction platforms.
17 Another important observation was that both Sections FP-III and FP-IV exhibited a gradual slope
18 of rutting progression prior to completion of 10,000 passes. Beyond 10,000 passes, those two
19 sections showed incremental damage accumulations under APT. Among all the sections built over
20 the target CBR=1% subgrade, Section FP-V reached failure at approximately 30,000 passes.

21 **POST-CONSTRUCTION/TESTING FORENSIC ANALYSES**

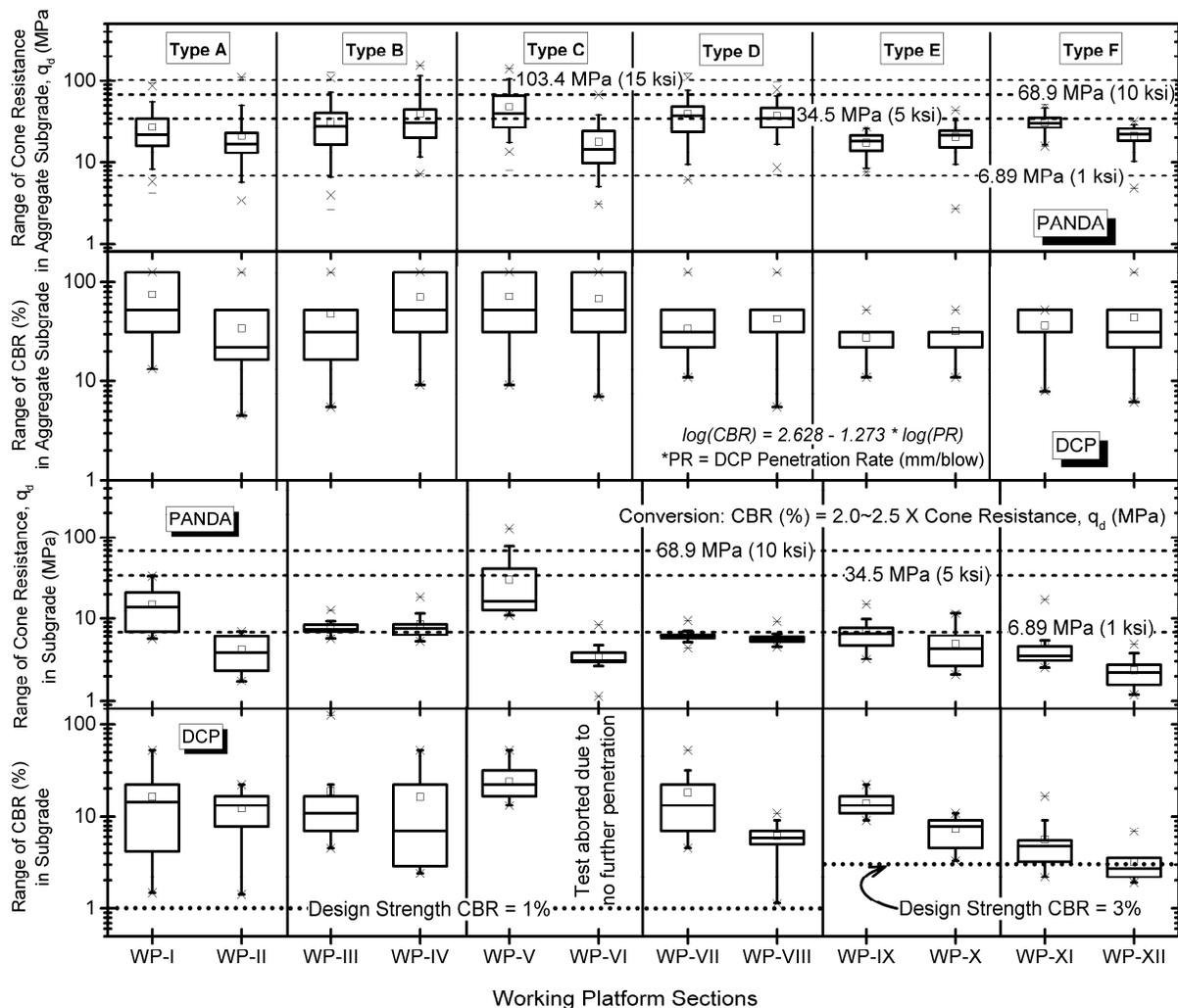
22 To investigate the differences between rutting trends of working platforms and flexible pavements,
23 a detailed forensic investigation scheme was undertaken. A variable energy PANDA penetration
24 device was used in working platforms for strength assessment followed by the completion of APT.
25 A geo-endoscopic imaging technique was also implemented in conjunction with PANDA
26 soundings to identify real time depth of water table in the construction platforms. Details of the
27 automated variable energy penetrometer and geo-endoscopy can be found elsewhere (28). In HMA
28 sections, representative cores were taken from the bound layer at the center, north and south ends
29 of the rut measurement line. Thicknesses of these cores and layer interfaces were measured
30 carefully and then the samples were tested for bulk specific gravity. Also, a laser guided level was
31 used to estimate the true elevations of test section HMA surfaces. These volumetric properties
32 were scrutinized against the nuclear gauge reported densities. Finally, trenching was done in all of
33 the sections to determine the as constructed layer thicknesses. The ensuing sub-sections present
34 the key findings from forensic analyses.

35 **Penetration based Strength Indices**

36 Figure 5 presents the penetration based strength indices from the constructed test sections. Note
37 that the cone resistance values were measured with PANDA penetration device; whereas, the
38 reported CBR values were obtained from dynamic cone penetration (DCP) tests. According to the
39 box plots, cone resistance values obtained in aggregate subgrade had lower variability than the
40 CBR values from the DCP tests. Despite exhibiting higher modulus with GeoGauge tests, both
41 PANDA and DCP measured strength indices indicated the lowest strength for the Type E RAP
42 among all the other aggregate subgrade types. Compared to the uniformly graded (Type A and C)
43 aggregate types, relatively well graded materials (Type B and D) exhibited less variability in cone

1 resistance results in consecutive sections with alternative capping layers. Such subtlety was not
 2 demonstrated by the DCP reported CBR values. Sections WP-II and WP-III showed lower strength
 3 than the other sections over engineered CBR=1% subgrade.

4 Similar to aggregate subgrade, DCP obtained CBR values varied over a wide range in the
 5 subgrade compared to PANDA cone resistance values. Although the existing subgrade was
 6 engineered to match controlled strength indices, significant strength gain was noticed in the
 7 aggregate subgrade sections with large rocks. Overall, these sections exhibited 10-fold increase in
 8 subgrade strength in terms of CBR. Such strength gain was successfully achieved by large rock
 9 mobilization into the weak subgrade under compaction effort. CBR strength in Section WP-II was
 10 found to be misleading since this construction platform failed prematurely. To this end, trends in
 11 cone resistance was more in line with the rutting performance trends. Regular sized aggregates in
 12 Sections WP-IX through WP-XII did not exhibit significant strength gain and this was reflected in
 13 the poor rutting performance. Subgrade strength was found to be lower in the RAP capped sections
 14 especially with uniformly graded aggregate subgrade materials.
 15

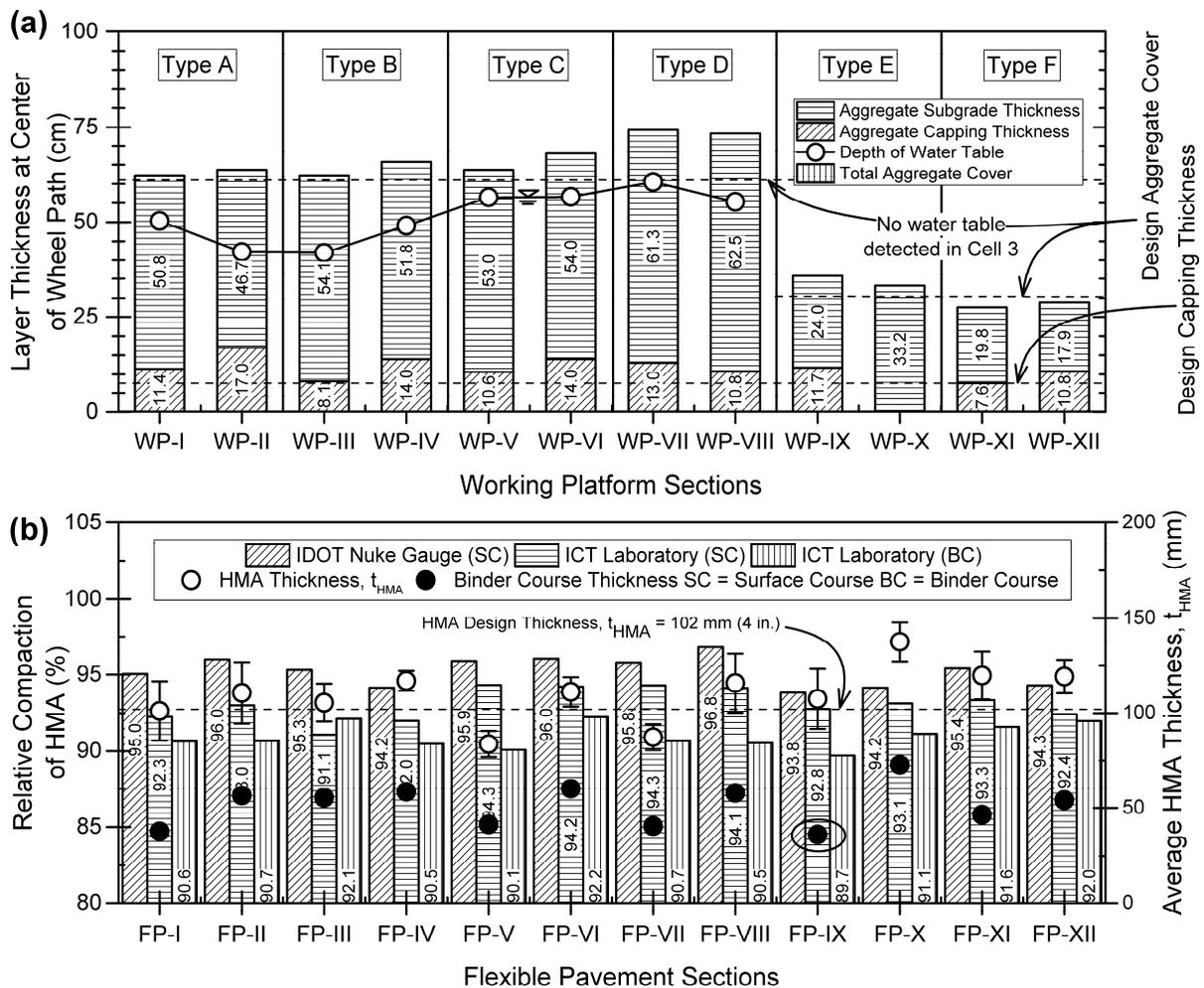


16
 17 Figure 5: Penetration based strength indices recorded in aggregate subgrade and subgrade layers
 18 of unsurfaced pavement sections
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Constructed Layer Thicknesses and Depth of Water Table

Figure 6 (a) indicates that the total aggregate cover thickness was the highest in Section WP-VII and this was also reflected in the measured modulus results and the extremely low accumulation of rutting trends observed in this section. The influence of thickness becomes more evident when aggregate cover thicknesses for Section WP-IX through Section WP-XII are taken into consideration. As the granular layer thickness decreased along those sections, so did the subgrade support. Therefore, rutting accumulation increased significantly, and ultimately, resulted in failure. Even though Section WP-IX through WP-XII were built over relatively stronger subgrade, the RAP layer performance was in general poor and the aggregate cover thickness for a stable working platform was insufficient at the field applied compaction levels.



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Figure 6: (a) As constructed layer thicknesses and depth of water table observed in working platform sections; (b) Average as constructed HMA thicknesses and relative compaction achieved in HMA

1 Notably, Type E RAP in Section WP-X was the only aggregate layer that exhibited
2 significant heaving along the edge of wheel path indicating internal shear failure. This observation
3 was consistent with the fact that the RAP-capped working platforms always showed higher
4 permanent deformation or earlier failure than their dolomite-capped counterparts. This finding
5 further substantiates RAP's susceptibility to rutting. According to Figure 6 (a), Sections WP-II and
6 WP-VII had the shallowest and deepest water table among the sections over CBR=1% subgrade,
7 respectively. Accordingly, Sections WP-II and WP-VII accumulated the highest and lowest
8 magnitude of rutting among the test sections.

9 **Relative Compaction and as Constructed Thicknesses of HMA Layer**

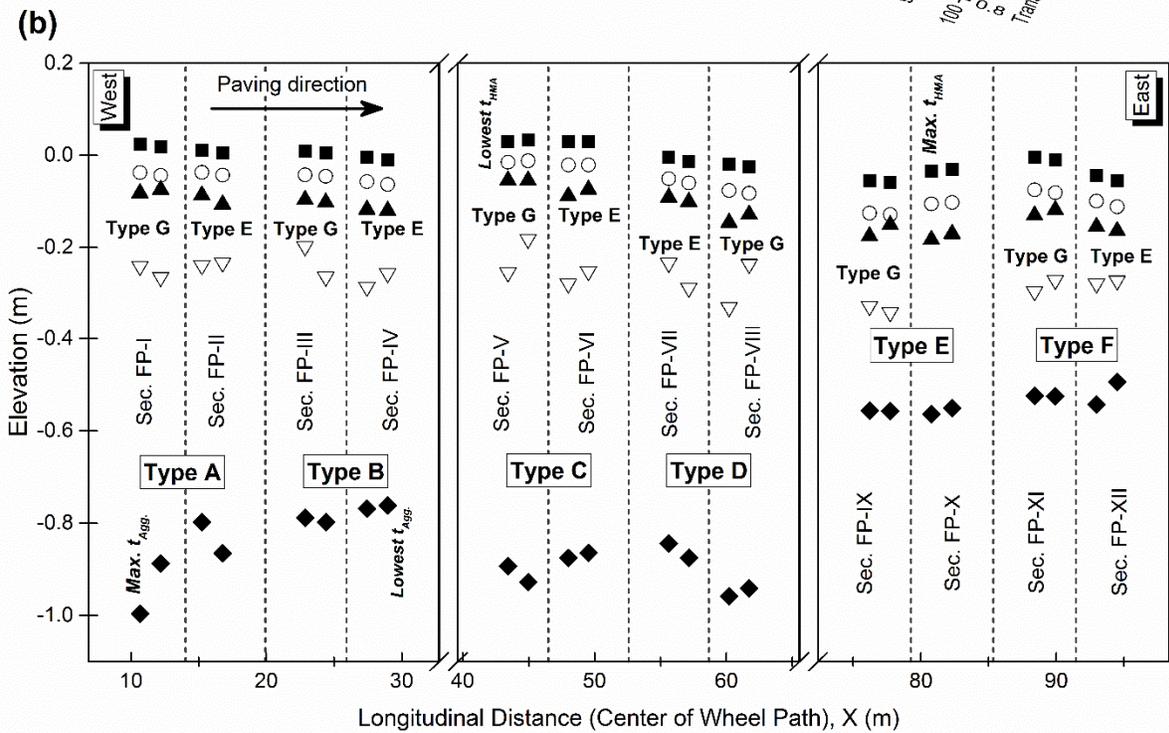
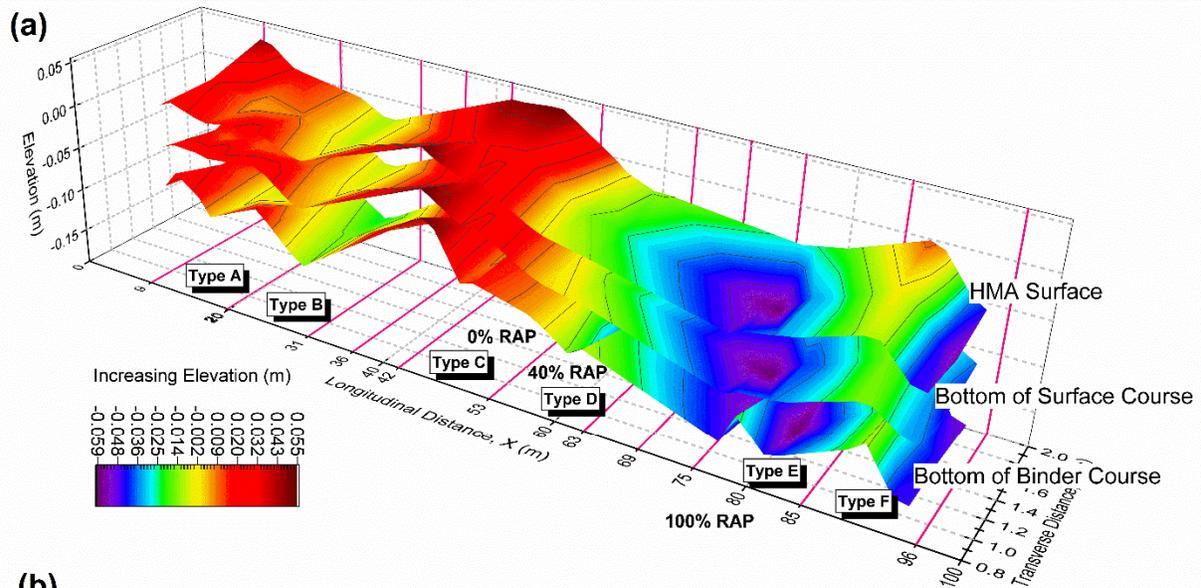
10 To further investigate the rutting trends in flexible pavements, HMA core thicknesses, as well as
11 relative compaction percentages achieved in the field and in the laboratory in reference to
12 theoretical maximum specific gravity, were closely examined. Figure 6 (b) summarizes the
13 corresponding test results. Furthermore, thicknesses of binder course and total bound (HMA) layer
14 are presented with solid black and empty circles, respectively. The black dash line in the same
15 figure denotes the design thickness for HMA layer. Figure 6 (b) clearly shows that the relative
16 compaction values measured with nuclear gauge were consistently higher than the laboratory
17 measured relative densities. Moreover, nuclear gauge densities satisfactorily met the IDOT QC/QA
18 criteria. According to the nuclear gauge test results, Section FP-IX exhibited the lowest relative
19 compaction. The same section also accumulated the highest rutting among the flexible pavements.
20 Laboratory test on binder course specific gravity also indicated the lowest relative density in this
21 section.

22 Significant deviation from design thickness was observed in as constructed HMA layer
23 thicknesses. Even numbered sections with RAP subbase had always higher HMA thickness than
24 the odd numbered sections with dolomite subbase. Since HMA layer is significantly stronger and
25 stiffer than the underlain granular materials, higher as constructed HMA thickness was the primary
26 reason for discrepancy in rutting trends of working platforms and flexible pavement sections.
27 Similar to findings of specific gravity, Section FP-IX had almost a non-existent binder course. A
28 thinner binder course coupled with the lowest relative density resulted in the maximum rut
29 accumulation, even though Section FP-V had a thinner HMA layer. As the percentage of RAP
30 increased from Type C through Type E (0%, 40% and 100% for Type C, D and E, respectively)
31 aggregate subgrade sections, variation in HMA thickness in the alternating subbase sections
32 escalated. Besides these observations, no apparent correlation was found among bulk specific
33 gravity, HMA thickness and rutting progression. This implies that the dissimilarity in HMA
34 thicknesses might be related to certain constructability issues.

35 **Surface Elevation and Sinkage Potential of RAP**

36 Because of substantial disparity in rutting performances of working platforms and flexible
37 pavements, special attention was paid to ascertain the construction issues. Any string lining
38 technique or automated screed control with mobile reference beam was not feasible during the
39 paving operation due to limited pavement width. To evaluate whether those issues had any
40 significant bearing on the as constructed HMA thicknesses and APT performance, grade elevations
41 of the flexible pavement sections were closely examined throughout the transverse span of rut
42 measurement along the test road. Estimated elevations of the pavement surface, binder course
43 interface, and subbase interface are presented with a three-dimensional contour plot in Figure 7

- 1 (a). Surface elevation of the flexible pavement varied widely and those discrepancies originated
- 2 from aggregate subgrade surface.
- 3



HMA Surface
 Binder Course Interface
 Subbase Interface
 Aggregate Subgrade Interface
 Subgrade Interface

4
5 Figure 7: Variations in (a) HMA surface elevation of the test road and (b) as constructed layer
6 thicknesses in low volume road sections

7 Sections with aggregate subgrade Types A and B exhibited similar elevations. Beyond that,
8 the grade peaked over Type C aggregate subgrade as if the primary crusher virgin materials resisted

1 the downward force of the paver. With inclusion of higher percentages of RAP in the pavement
2 system, surface elevation gradually decreased reaching the lowest point in Section FP-X. Surface
3 elevation again increased with change in aggregate subgrade interface followed by a dip in Section
4 FP-XII with RAP subbase. To this end, the lowest HMA thickness was obtained at the highest
5 surface elevation. In contrast, the highest HMA thickness was recorded at the lowest elevation.
6 One minor anomaly from this pattern was the subbase elevation in Section FP-IX, which was
7 higher than that in Section FP-X. For this, Section FP-IX had minimal thickness of binder course
8 as discussed in the preceding section.

9 For further investigation of rutting trends, as constructed layer thicknesses of the flexible
10 pavement sections were plotted alongside surface elevation in Figure 7 (b). The effect of aggregate
11 cover became evident during the later stage of accelerated pavement testing. Because Sections FP-
12 III and IV had the thinnest aggregate cover among the CBR=1% flexible pavement sections, those
13 two sections exhibited incremental rutting after 10,000 passes. A closer look at the subbase
14 thicknesses at the west and east measurement lines in the RAP subbase sections reveals that
15 subbase layer always got thinner along the direction of paving (east end). Considering the fact that
16 the HMA paver was much heavier than the vibratory roller used for granular layer compaction,
17 RAP subbase layers were either consolidating under higher compactive effort or sinking into the
18 soft subgrade. Based on the forensic analyses of working platforms and flexible pavements, 100%
19 RAP materials were definitely prone to rutting and exhibited significant sinkage potential.

20 CONCLUSIONS

21 To validate the newly introduced Illinois DOT (IDOT) material specifications for the large sized
22 and unconventional aggregate subgrade, a research study was undertaken involving construction
23 of twenty-four full scale pavement sections with seven different aggregate types. Prior to
24 construction, the existing subgrade strength over two segments of the test road was modified to a
25 design CBR of 1% and 3%, respectively. Upon compaction of capping and subbase layers, QC/QA
26 tests like nuclear gauge density, and lightweight deflectometer and GeoGauge for modulus were
27 conducted. Followed by the construction, those test sections were subjected to accelerated
28 pavement testing and monitored for rutting progression periodically. To establish linkages between
29 the QC/QA results and rutting performance trends, detailed forensic analyses were also conducted.
30 In light of the presented test results and corresponding rutting performances, following conclusions
31 can be drawn:

- 32
- 33 (a) Although none of the capping layers met the IDOT required compaction level at or above
34 95% of the standard Proctor maximum dry density; working platforms constructed with
35 large rocks performed considerably well surviving well over 4,000 passes. The nuclear
36 gauge showed high sensitivity to hydrogen bound materials leading to erroneously high
37 moisture contents. To this end, density based compaction control on unbound granular
38 layers with large rocks was proven to be insufficient for prediction of rutting trends.
 - 39 (b) The highest recorded coefficients of variation (COV) were 22% for the lightweight
40 deflectometer (LWD) modulus measurements and 34% for GeoGauge measurements,
41 respectively. As opposed to the GeoGauge measurements, LWD moduli could be better
42 correlated to the rutting performances of layered construction platform systems with
43 thinner aggregate cover over CBR=3% subgrade.
 - 44 (c) Especially in cases of shallow water table, a strength based approach involving cone
45 penetration testing was found to be more effective. The automated variable energy PANDA

1 device exhibited less variability than the commonly used dynamic cone penetrometer
2 (DCP). Strength indices measured by both devices showed excellent correlations with
3 rutting performances in working platforms.

- 4 (d) Significant variability was observed in constructed hot mix asphalt (HMA) thicknesses
5 over the reclaimed asphalt pavement (RAP) aggregate subgrade and subbase sections. In
6 light of the rutting trends and sinkage behavior, use of RAP in unbound granular layers
7 should be considered with caution. If certain agency permits significant percentage of RAP
8 to be included in these layers, strict grade control with string lining or automated self-
9 adjusting screed may be needed to track the elevation during paving process. Also, reported
10 HMA nuclear gauge densities in this paper were consistently higher than laboratory
11 measured values for surface and binder courses.

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