APPLIED RESEARCH ASSOCIATES, INC.

December 20, 2021

Conrad Moore, P.E.
County Engineer
Woodford County Highway Department
301 S. Main St.
Roanoke, IL 61561
Tel (309) 923-2891
cmoore@woodford-county.org

Subject: **Woodford County Pavement Evaluation.** ARA Project No. 004714

Dear Mr. Moore:

Applied Research Associates, Inc. (ARA) appreciates the opportunity to submit to Woodford County this report for an evaluation of selected county highways.

It has been a pleasure providing these services to Woodford County and we look forward to working with you in the future.

Sincerely,

Douglas A. Steele, P.E.	Michael J. Harrell, P.E.
Senior Engineer	Division Manager

Attachment

cc: Michael Bruner, TCRPC
Dr. Marshall Thompson

REPORT

Woodford County Pavement Evaluation

Prepared for:

Woodford County Highway Department
301 S. Main St.
Roanoke, IL 61561
Tel (309) 923-2891

Prepared by:

Applied Research Associates, Inc.
Transportation Sector
100 Trade Centre Dr., Suite 200
Champaign, IL 61820
Tel. (217) 356-4500

December 20, 2021

TABLE OF CONTENTS

BACKGROUND

The Woodford County Highway Department maintains approximately 150 centerline miles of county highways, primarily two-lane, rural roads. Many of these roads have year around restrictions on gross vehicle weights for tractor-trailer combinations and winter-spring load restrictions on axle weights. Figure 1 shows typical signage posting the load restrictions.

ARA performed a pavement evaluation on 60 centerline miles of selected county highways to assess their structural characteristics and to determine if the current load restrictions are necessary. Specific project objectives were:

- Assess existing pavement structural conditions (i.e., coring and deflection testing)
- Characterize materials and subgrade soils for use in engineering analysis
- Perform engineering analysis to assess current load restrictions
- Develop rehabilitation recommendations to support future road use

Figure 1. Typical load restrictions on a Woodford County highway.

METHODOLOGY

ARA performed a comprehensive pavement evaluation and analysis on eight county highways (figure 2) that included:

- Pavement coring, augering, and DCP testing
- Falling weight deflectometer (FWD) testing to evaluate structural uniformity and to determine in place pavement structural properties
- Geometric (i.e., crossfall and lane width) data collection for consideration during treatment development
- Traffic analysis
- Structural analysis for spring load conditions
- Remaining life analysis and overlay design for year around operations with an 80,000-lb load limit
- Feasibility and costs

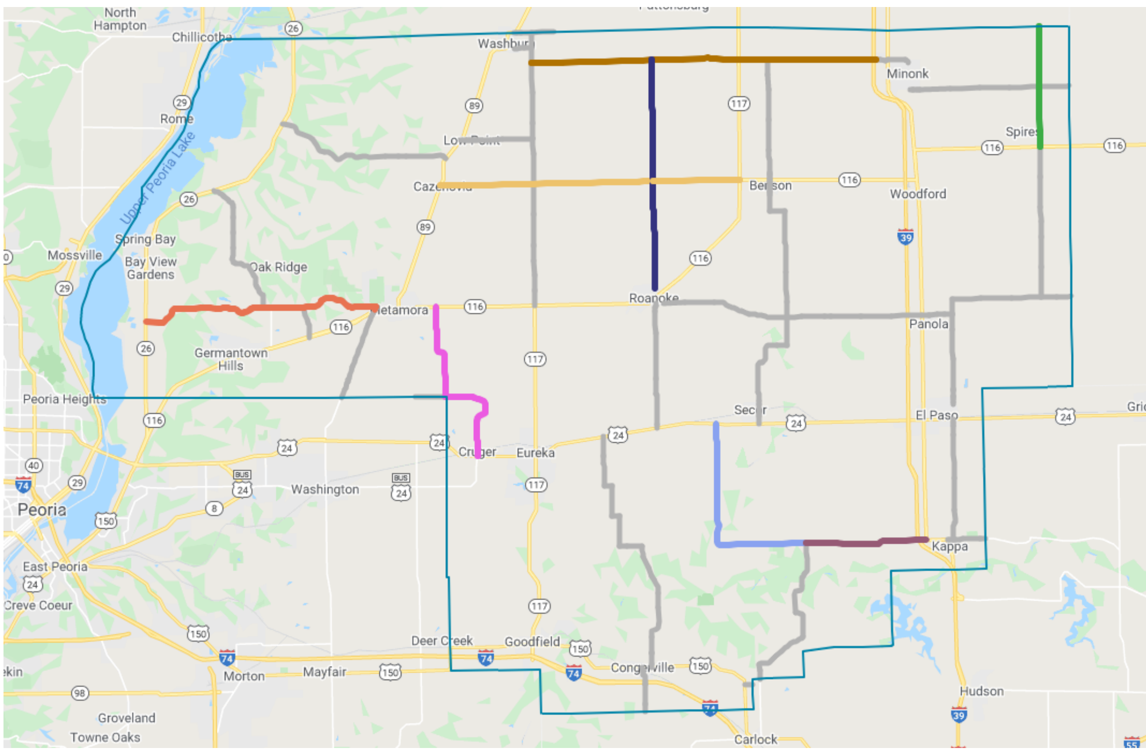

Figure 2. Woodford County highways – study roads shown in color.

County highways included in the study were:

- CH 2
- CH 3
- CH 7 (US 24 to CH 9)
- CH 9 (CH 3 to US 251)
- CH 13 (Roanoke to CH 2)
- CH 18
- CH 20 (US 116 to county line)
- CH 25 (Santa Fe Trail)

FIELD DATA COLLECTION AND ANALYSIS

ARA referenced field data using a linear measurement with the zero station located at either the west or south roadway end, with increasing distance in miles to the east and north.

Pavement Coring and Dynamic Cone Penetrometer (DCP) Testing

ARA performed pavement coring to determine pavement layer types and thicknesses. We cored at the rate of one core per centerline mile using a 6-inch diameter core barrel and measuring layer thicknesses in the core hole. We photographed each core and bagged representative samples of the aggregate base, prior to patching the core hole with asphalt cold patch. For each road, the field crew performed DCP testing at a single core location to determine the aggregate base thickness and the subgrade stiffness. ARA performs pavement coring and base augering with a trailer-mounted drill rig, shown in figure 3. Overall, we collected 58 cores and performed eight DCP tests.

Figure 3. Pavement coring and augering determined the layer types and thicknesses.

Figures 4-6 show representative core and base samples and table 1 summarizes the typical pavement structures. Appendix A contains the detailed core log for all 58 core samples and selected base samples. Overall, the coring operation determined that seven of the eight roads consisted of several inches of accumulated chip seals over a thick aggregate base. The remaining road (CH 25) consisted of a chip seal surface over a combination of asphalt concrete (AC) and accumulated chip seal layers over a thick aggregate base. Several roads showed notable changes in layer thicknesses, as shown in the table.


Figure 4. Typical core sample showing accumulated chip seal layers – CH 2, mile 0.5.


Figure 5. Combination of chip seal and AC layers – CH 25, mile 2.56.

Figure 6. Sample crushed gravel with sand base – CH 3, mile 4.5.

Table 1. Coring and DCP results.

CH	**Miles**	**Surface Type**	**Mean Surface Thickness, in**	**Base Type**	**Base Thickness[c], in**
2	0 - 11.15	Chip	2.84	Sand w/gravel	12.2
	11.15 - 11.38	Chip/AC[a]	8.00		
3	0 - 3.35	Chip	4.50	Crushed gravel w/sand	14.3
	3.35 - 6.52	Chip	2.33		
7	0 - 7.01	Chip	2.89	Crushed gravel w/sand	16.0
9	0 - 4.05	Chip	2.44	Sand w/gravel	12.4
13	0.0 3.55	Chip	4.33	Crushed gravel w/sand	16.8
	3.55 - 7.62	Chip	2.88		
18	0 - 10.03	Chip	2.88	Crushed gravel w/ sand	16.9
20	0 - 4.02	Chip	3.31	Sand w/gravel	10.1
25	0 - 7.95	Chip+AC[b]	7.25	Crushed gravel w/sand	17.6
	7.95 - 8.39	Chip+AC[b]	7.50		

[a] Chip seal over AC.
[b] Combination of chip seal and AC layers.
[c] From DCP.

FWD Testing

The FWD is a nondestructive deflection testing device that delivers a dynamic load to the pavement, simulating the effects of a heavy moving wheel load. ARA performed FWD testing on October 18-19, 2021 to determine the pavement's structural response for use in pavement analysis. We tested at 0.2-mile intervals in both traffic directions, staggered between the two lanes, for a total of 595 test points. Figure 7 shows an example of the FWD during testing.

Figure 7. The FWD is a nondestructive testing measuring the pavement's structural response.

Figures 8 and 9 show example FWD maximum deflection (D0) profiles for a 9,000-lbf load normalized to a standard temperature of 68 °F. Higher deflections represent thinner or weaker pavements, while lower deflections indicate stiffer structural response.

ARA used the FWD deflections with a regression equation developed by Dr. Marshall Thompson that predicts the subgrade resilient modulus (E_{Ri}) based on the FWD deflection 36 inches away from the load center. We then correlated this to an equivalent unconfined compressive strength (Q_u) for use in the structural analysis, as well as an Immediate Bearing Value (IBV), which is used to characterize the subgrade for the IDOT BLR overlay design. Likewise, we calculated the average Area Under the Pavement Profile (AUPP) parameter for each of the three analysis sections for use in backcalculation of the pavement elastic modulus (Ep) using the ILLI-PAVE finite-element program developed by the University of Illinois. We used ILLI-PAVE in an iterative fashion until it generated theoretical deflection basins that approximately matched the average D0 and AUPP values measured by the FWD. Finally, we calculated the effective structural number (SNeff) of the existing pavement using the 1993 AASHTO backcalculation procedure. Table 2 summarizes the structural parameters obtained from the FWD and ILLI-PAVE. Appendix B contains the detailed data for each road.

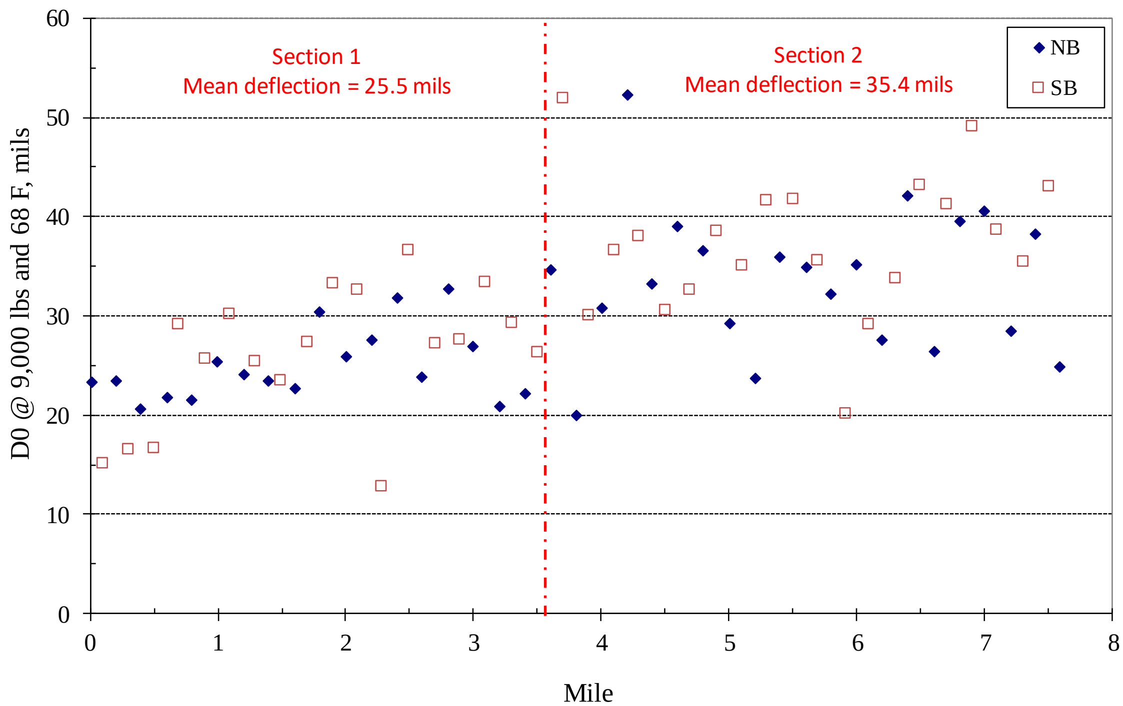

Figure 8. Typical deflections on the chip seal roads – CH 13.

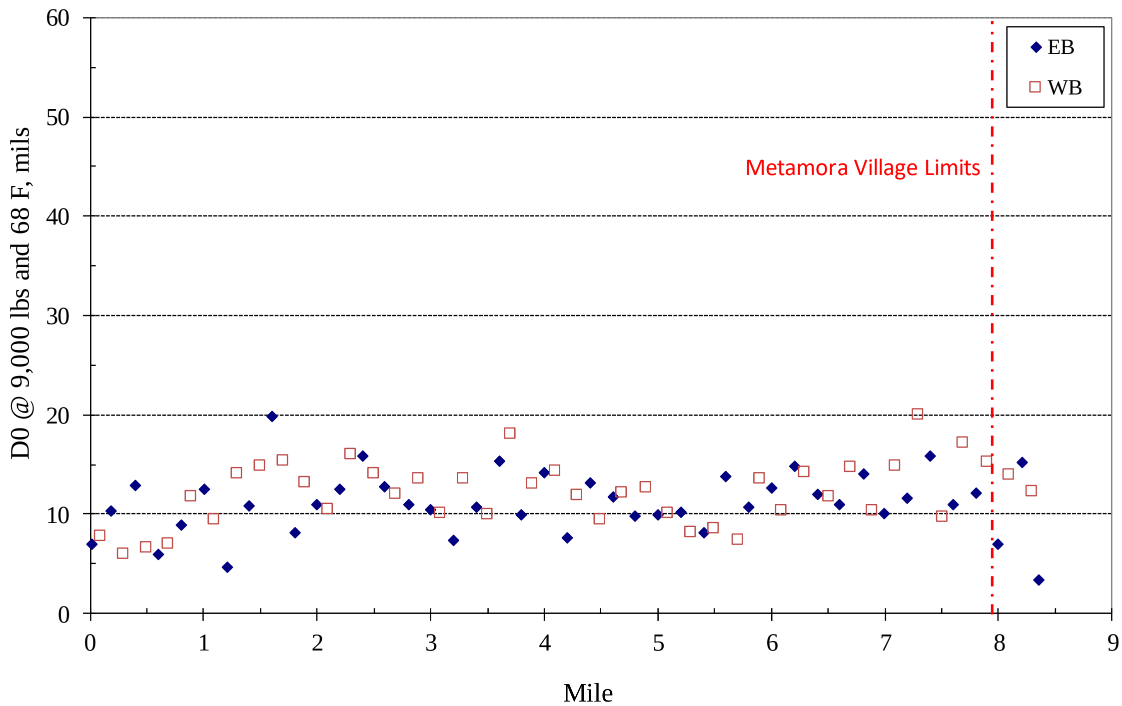

Figure 9. Lower deflections on CH 25 reflecting the thicker pavement, which includes AC.

Table 2. FWD results.

CH	Miles	D0, mils	AUPP, in-mils	E_{Ri}, psi	Qu, psi	IBV, percent	SNeff, in
2	0-11.38	27.86	42.3	6,669	18.9	4.2	2.39
3	0 - 3.35	27.98	42.5	6,311	17.8	3.9	2.79
	3.35 - 6.52	30.60	48.3	5,901	16.4	3.6	2.18
7	0 - 7.01	25.93	37.4	8,615	25.3	5.6	2.37
9	0 - 4.05	26.97	37.7	6,862	19.6	4.3	2.31
13	3.55	25.45	38.2	5,977	16.7	3.7	2.66
	3.55 - 7.62	35.37	53.3	3,062	7.2	1.6	2.19
18	0 - 10.03	29.17	48.2	6,577	18.6	4.1	2.27
20	0 - 4.02	30.24	40.2	6,106	17.1	3.8	2.31
25	0 - 7.95	11.67	17.7	12,769	38.8	8.6	4.12
	7.95 - 8.39	10.32	11.8	9,342	27.6	6.1	4.05

Geometric Data

One of the project requirements was to assess the existing pavement cross section geometry to determine if any improvements such as crown correction, pavement widening, and or paved shoulders are needed, taking into consideration IDOT's requirements for resurfacing, rehabilitation, and restoration (3R) projects. In the case of county highways, IDOT's 3R requirements are contained in Chapter 33 of the BLR manual, Geometric Design of Existing Highways. To address these criteria, ARA collected the following information at each FWD test point:

- Pavement crossfall
- Lane width
- Shoulder type
- Shoulder width

Figures 10 and 11 present sample crossfall results for two roads—CH 20 and 25. CH 20 is a straight road with no horizontal curves and the crossfall results are very uniform, averaging +1.3 percent (positive slope is to the shoulder). In comparison, CH 25 contains numerous horizontal curves, which introduce both positive and negative super elevations. The results range from -8.1 to +9.5 percent, with an average of 0.7 percent.

Table 3 summarizes the average values for all geometric attributes for each road. Appendix C contains the detailed results in spreadsheet format. Overall, lane width are typically 11-ft, except in the case of the eastern portion of CH 2 and the eastern end of CH 25, where it enters Metamora. Shoulders are typically loose chips 0.5 to 1 ft wide with a few exceptions. The eastern end of CH 2 is 4-ft wide AC at the approach to I-39, CH 13 is 1-ft wide chip seal in Roanoke, and CH 25 has concrete curb and gutter with no shoulder where it enters Metamora, although the lane widens, ranging from 12 to 18 ft in the village.

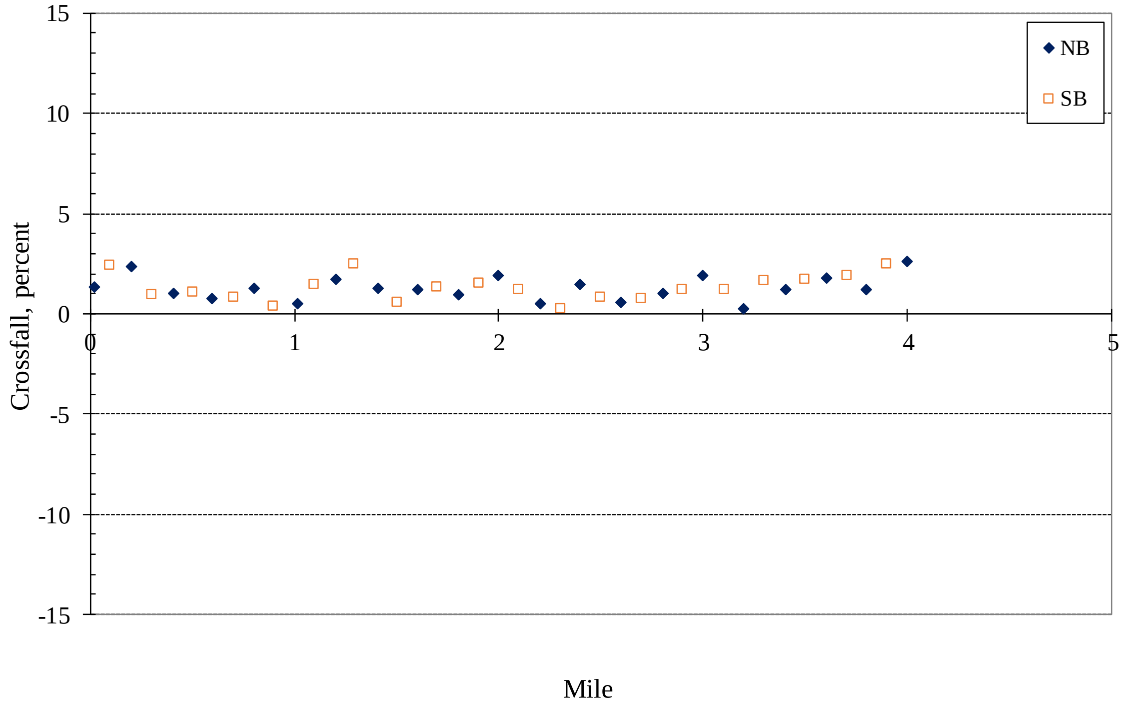

Figure 10. Crossfall results on CH 20 – no horizontal curves.

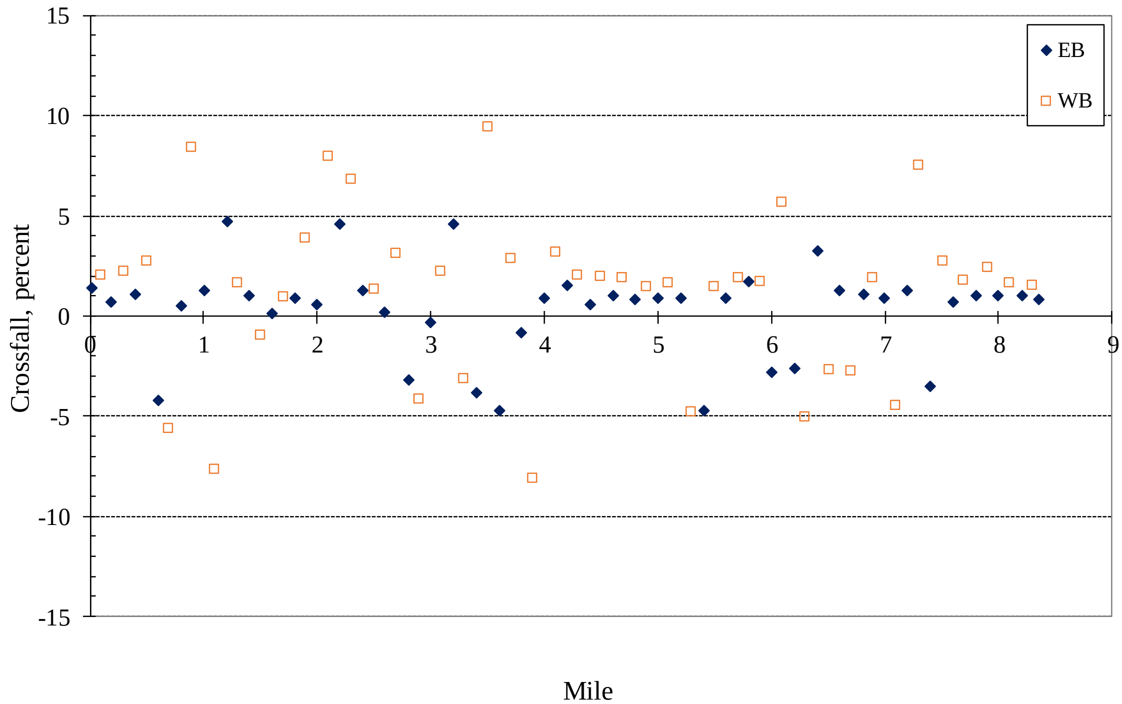

Figure 11. Crossfall results on CH 25 – numerous horizontal curves.

Table 3. Geometric attributes.

CH	Miles	Mean Crossfall, percent	Typical Lane Width, ft	Shoulder Type	Typical Shoulder Width, ft
2	0 – 6.95	1.4	11	Stone	1.0
	6.95 – 11.15	1.8	12	Stone	1.0
	11.15 – 11.38	1.1	12	AC	4.0
3	0 – 6.52	0.5	11	Stone	1.0
7	0 – 7.01	0.9	11	Stone	1.0
9	0 – 4.05	1.2	11	Stone	1.0
13	0 – 0.65	1.4	11	Chip seal	1.0
	0.65 – 7.62	1.0	11	Stone	0.5
18	0 – 10.03	1.4	11	Stone	1.0
20	0 – 4.02	1.3	11	Stone	1.0
25	0 – 7.95	0.8	11	Stone	0.5
	7.95 – 8.39	1.2	12-18	PCC C&G	0.0

TRAFFIC ANALYSIS

ARA determined design traffic using the IDOT average daily traffic (ADT) map and the Traffic Factor (TF) equations presented in the IDOT BLR manual, Chapter 46. The TF equations determine the design equivalent single axle loadings (ESALs) based on ADT, traffic class, percent trucks, and design period. For this analysis, ARA assumed a 20-year design life for the overlay design. Traffic class is based on ADT was typically Class IV (less than 400 vehicles per day) and Class III (between 400 to 2000 vehicles per day). The easternmost section of CH 25 entering Metamora qualified as a Class II road (greater than 2000 vehicles per day). We adopted IDOT default values for single- and multiple-unit trucks based on traffic class—9 and 3 percent for Class IV and 7 and 5 percent for Class III. In cases where the ADT varied for a road, ARA selected a value equal to the mean plus one standard deviation. Table 4 summarizes the design traffic for each road.

Table 4. Design traffic.

County Highway	Miles	ADT Range, vpd	Design ADT, vpd	IDOT Traffic Class	TF, MESALs[a]
2	0 – 11.38	375 – 750	675	III	0.222
3	0 – 3.35	200 – 300	300	IV	0.079
	3.35 – 6.52	175 – 225	237	IV	0.062
7	0 – 7.01	350 – 525	515	III	0.169
9	0 – 4.05	450 – 550	571	III	0.188
13	0 – 3.55	275 – 450	446	III	0.147
	3.55 – 7.62	150 – 200	210	IV	0.055
18	0 – 10.03	250 – 325	338	IV	0.088
20	0 – 4.02	150 – 200	210	IV	0.055
25	0 – 7.95	675 – 1400	1305	III	0.429
	7.95 – 8.39	1900 - 3650	3597	II	1.195

[a] Millions of ESALs.

It should be noted that due to inconsistencies with the Class IV TF equation, and following discussion with IDOT, we decided to use the Class III equation for both Class III and IV roads, which provided a more realistic prediction.

STRUCTURAL ANALYSIS AND DESIGN

Spring Load Limits

ARA analyzed the spring thaw condition using the subgrade stress ratio concept, where the goal is to limit the stress placed on the subgrade in its weakest state. This is different from typical pavement design procedures, which look at the accumulated effects of multiple load repetitions. ARA used the subgrade stress ratio (SSR), defined as the ratio of the deviator stress placed on the subgrade due to a heavy truck load divided by the subgrade's unconfined compressive strength (Q_u) determined through correlation to FWD results, times 100. Typically, it is desirable to have a SSR less than 70 percent for low-volume roads and less than 50 percent for high-volume roads to prevent the likelihood of subgrade failure.

ARA modeled the chip seal roads (i.e., all sections except CH 25) as a 3-inch thick, low-modulus AC layer (i.e., 200,000 psi) with a 12-inch aggregate base and a very weak subgrade (E_{Ri} = 3,000 psi), using the ILLI-PAVE finite element program to determine the deviator stress on the top of the subgrade. We used ILLI-PAVE to calculate stresses in the pavement due to a single tire, single axle load of 6, 7, 8, 9, and 10 kips per wheel at 100 psi tire pressure to evaluate the sensitivity of results with load. 10 kips represents the maximum legal load limit for a single axle and 6 kips is just above the current posted spring load limit of 5 tons per axle (i.e., 5 kips per wheel). Table 5 summarizes the results.

Table 5. Subgrade stress ratio – chip seal roads.

Wheel Load, kips	**ILLI-PAVE Subgrade Deviator Stress, psi**	**Subgrade Unconfined Compressive Strength, psi**	**SSR, percent**
6	5.3	13	41
7	5.5	13	42
8	5.9	13	45
9	6.0	13	46
10	6.1	13	47

The results showed SSR values ranging from 41 to 47 percent, all below the desired threshold of 70 percent. In addition, we repeated the analysis using an 8-inch aggregate base to evaluate the results in areas where the base may be thinner than expected. This produced SSR values ranging from 44 to 54 percent, within an acceptable range.

The field data showed that CH 25 consists of a thicker and stiffer AC layer than the chip seal roads, therefore, we analyzed this pavement separately from the others. In the case of CH 25, we modeled a 7.25-inch AC layer over 8- and 12-inch aggregate bases and a very weak subgrade (ERi = 3,000 psi). We modeled the AC layer with two moduli values—550 and 250 ksi—to represent the AC layer at the field temperatures present during data collection, as well as lower a lower modulus due to higher temperatures at other times of the year. Table 6 presents the results.

Table 6. Subgrade stress ratio – CH 25.

Wheel Load, kips	Aggregate Base Thickness, in	AC Modulus, ksi	ILLI-PAVE Subgrade Deviator Stress, psi	Subgrade Unconfined Compressive Strength, psi	SSR, percent
10	12	550	3.5	13	27
10	8	550	3.6	13	28
10	12	250	4.3	13	33
10	8	250	4.6	13	35

The CH 25 results more acceptable and even more favorable than the chip seal road SSR values, as expected, due to its thicker, higher-modulus AC layer.

The SSR results indicate that no excessive damage to the subgrade and pavement is expected due to heavy loads during weakened soil conditions and, therefore, the current restriction may not be necessary. However, the SSR analysis does have limitations, mainly it does not consider rutting of the AC and base layers, just the subgrade. Modeling of base failures requires further characterization of the aggregate base material beyond the scope of this study, and currently there are not rutting models for accumulated chip seal layers, such as these roads.

Remaining Life Analysis

ARA performed a remaining life analysis based on the IDOT BLR nomograph for the Modified AASHTO Design - Chapter 46, Figure 46-4E and the FWD-determined SNeff, IBV, and TF values determined above. Figure 12 shows an example of this prediction. By using FWD-determined SNeff and IBV values as inputs, we predicted the number of MESALs until rehabilitation is needed, which are then compared to the 20-year TF determined previously to estimate the years until rehabilitation. Table 7 summarizes the remaining life estimates for each analysis section.

Table 7. Remaining life estimates.

CH	Miles	SNeff, in	IBV, percent	Remaining MESALs	TF_20 Years, MESALs	Remaining Life, years
2	0-11.38	2.39	4.2	0.050	0.222	4.5
3	0 - 3.35	2.46	3.9	0.050	0.079	12.7
	3.35 - 6.52	2.18	3.6	0.020	0.062	6.4
7	0 - 7.01	2.37	5.6	0.075	0.169	8.9
9	0 - 4.05	2.31	4.3	0.040	0.188	4.3
13	3.55	2.66	3.7	0.070	0.147	9.5
	3.55 - 7.62	2.19	1.6	0.005	0.055	1.8
18	0 - 10.03	2.27	4.1	0.033	0.088	7.4
20	0 - 4.02	2.31	3.8	0.035	0.055	12.7
25	0 - 7.95	4.12	8.6	5.000	0.429	>20[a]
	7.95 - 8.39	4.05	6.1	2.500	1.195	>20[b]

[a] Although this section is structurally sound, a functional improvement would be required within this time frame.
[b] Although this section does not need structural improvement, the surface condition is deteriorated to a level where a functional mill and overlay are required.

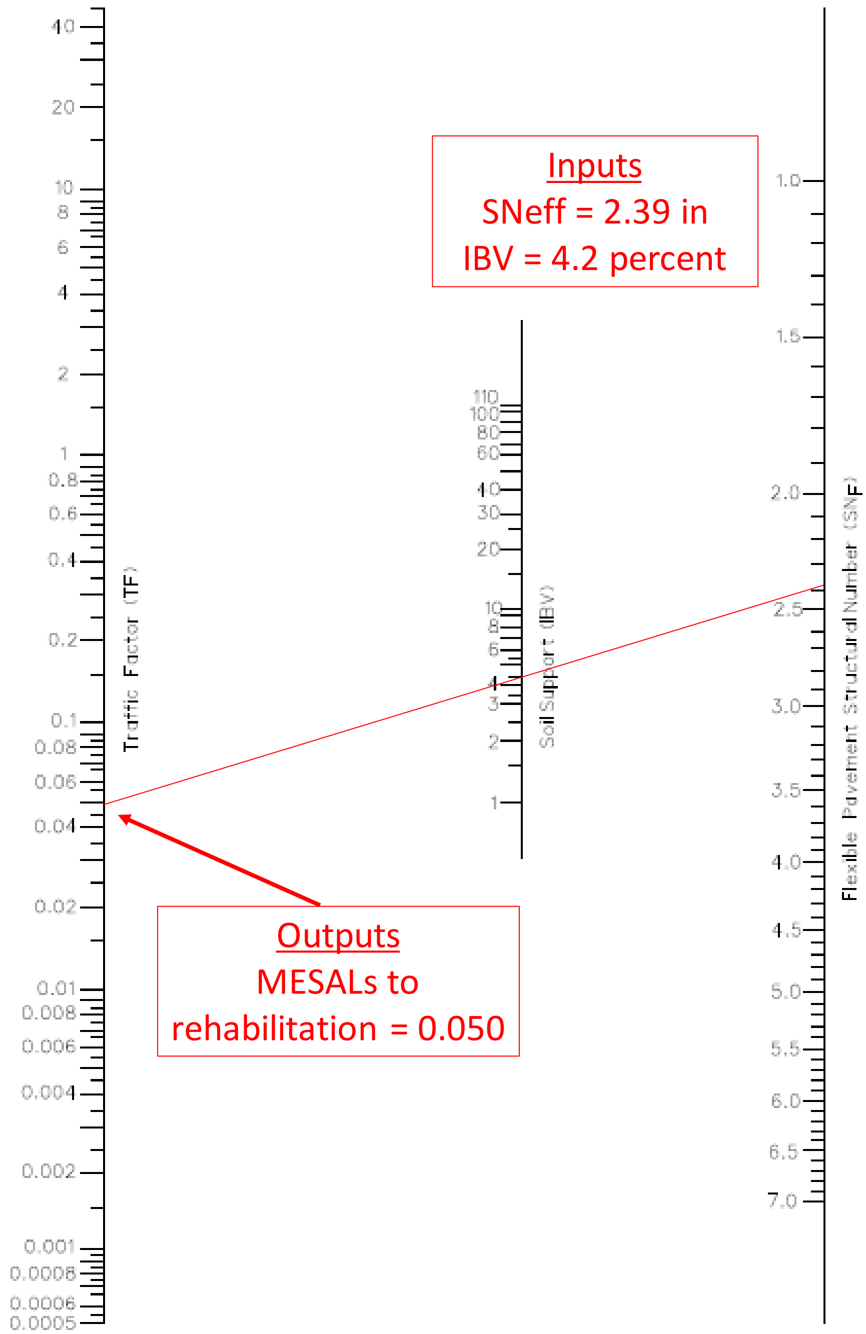

Figure 12. Sample remaining life prediction using the IDOT Modified AASHTO design nomograph, BLR manual, figure 46-4E – CH 2.

Overlay Design

ARA performed overlay designs based on the IDOT BLR manual, Chapter 46 Pavement Rehabilitation, using the Modified AASHTO design procedure. This method uses the TF and subgrade IBV to determine the required SN to carry the design traffic. Figure 13 shows an example of using the design nomograph to determine the required SN. The overlay thickness is determined as the difference between the required and existing structural numbers, converted to an equivalent thickness of new AC. We determined the existing SN through two methods—FWD testing and structural coefficients. The structural coefficients were selected using recommendations from the BLR manual as guidelines. Once a required overlay thickness was determined for each method and averaged the two results. The BLR manual requires a minimum overlay thickness of 3 inches AC for required SNs from 2.50 to 3.49 inches. Therefore, some of the recommended overlay thicknesses have been adjusted to meet the required minimum.

Table 8 summarizes the overlay thickness results for each analysis section.

Table 8. Overlay design.

					FWD Method		Coefficients		Final
CH	**Miles**	**TF, MESALs**	**IBV, percent**	**SNf, in**	**SNeff, in**	**Hac, in**	**SNeff, in**	**Hac, in**	**Hac, in**
2	0-11.38	0.222	4.2	3.05	2.39	1.8	1.80	3.5	3
3	0 - 3.35	0.079	3.9	2.70	2.46	0.7	2.10	1.7	3
	3.35 - 6.52	0.062	3.6	2.60	2.18	1.2	1.67	2.6	3
7	0 - 7.01	0.169	5.6	2.7	2.37	0.9	1.78	2.6	3
9	0 - 4.05	0.188	4.3	3.00	2.31	1.9	1.69	3.6	3
13	3.55	0.147	3.7	2.95	2.66	0.8	2.07	2.5	3
	3.55 - 7.62	0.055	1.6	3.10	2.19	2.5	1.78	3.7	3
18	0 - 10.03	0.088	4.1	2.7	2.27	1.2	1.78	2.6	3
20	0 - 4.02	0.055	3.8	2.55	2.31	0.7	1.86	1.9	3
25	0 - 7.95	0.429	8.6	2.80	4.12	0	3.38	0	3[a]
	7.95 - 8.39	1.195	6.1	3.50	4.05	0	3.45	0	3[b]

[a] Although this section is structurally sound, a 3-inch overlay is required to meet the minimum thickness
[b] Although this section does not need structural improvement, the surface condition is deteriorated to a level where a functional mill and overlay are required.

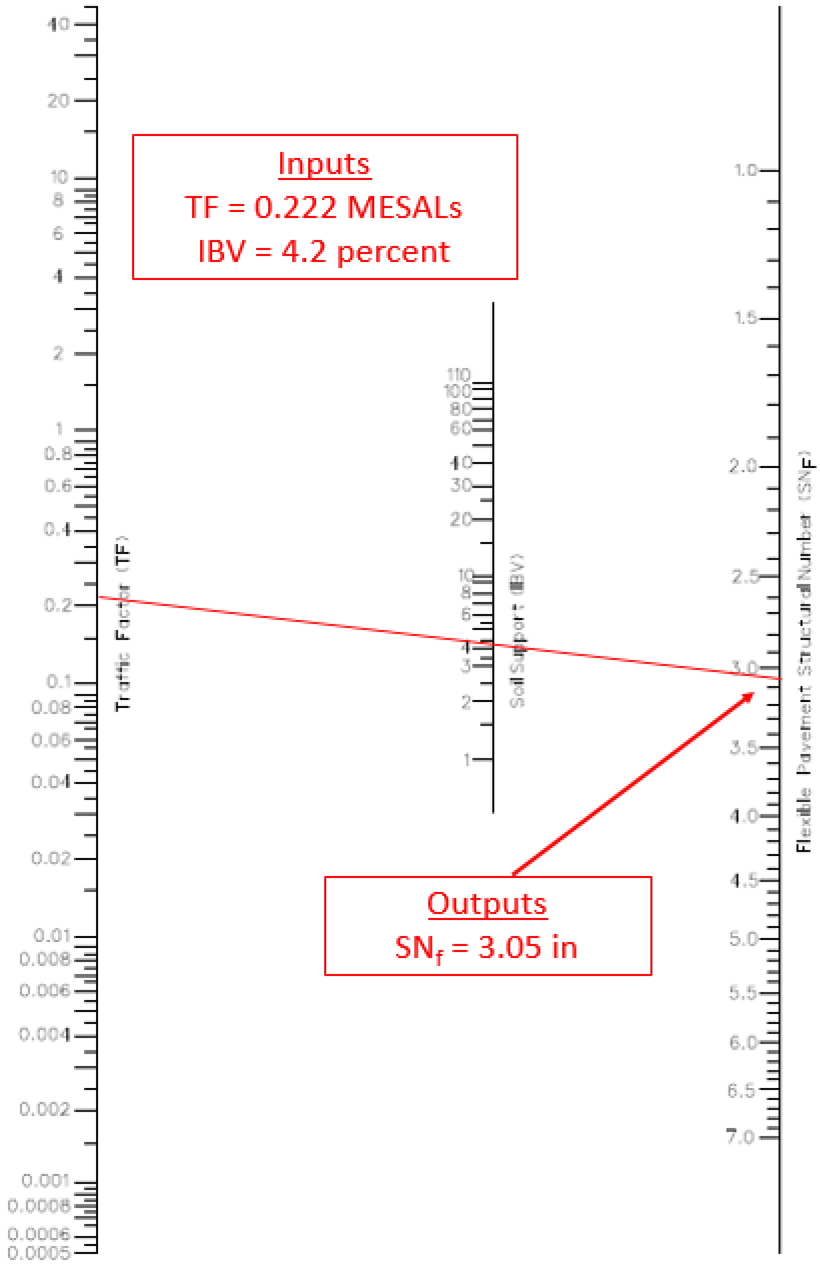

Figure 13. Sample overlay design using the IDOT Modified AASHTO design nomograph, BLR manual, figure 46-4E – CH 2.

FEASIBLE TREATMENTS AND COSTS

Feasible Treatments

There are several feasible options for Woodford County highways, depending on the County's strategic goals. They are:

- Continue routine surface treatments
- Perform base widening and continue surface treatments
- Defer future year rehabilitation
- Perform rehabilitation at the present

The seven chip seal roads included in the study consist of 2-4 inches of accumulated chip seals over a granular base of approximately 12 inches or more. The eight road (CH 25), is approximately 7-8 inches of combined chip seal and AC over a similar granular base as the other roads. The spring load analysis shows that this is sufficient pavement structure to protect the subgrade from ultimate load failure during the critical spring period. However, there are limitations to this analysis, for example, it does not account for potential failure of the aggregate base layer due to causes such as strength loss when saturated or loss of confinement. It was reported by the County Engineer that several of the chip seal roads were constructed with base thicknesses the same width as the chip seal surface, meaning the granular base ends at the pavement edge. IDOT design standards require the granular base to be 2 ft wider than the riding surface in each traffic direction, in part to prevent loss of confinement (and therefore failure) of the granular base when heavy loads pass near the pavement edge. Assuming that the remaining chip seal roads are constructed the same way, this is a potential drawback of removing spring load restrictions. CH 25 was reported to be originally constructed of AC and with a base that was wider than the riding surface. If the County decides to maintain the roads as chip seal pavements, we recommend performing chip sealing every 5 years to maintain a safe driving surface and extend the existing pavement longevity.

One alternative is to widen the granular base on each shoulder and continue to maintain the roads as chip seal pavements. This would reduce the possibility of edge of pavement failure due to heavy loads traveling near the pavement edge when the subgrade is weak. This existing right-of-way and ditches would have to be evaluated to confirm that there is sufficient width to perform this option.

Either of the above two options can be used to defer the time of rehabilitation to a later date, at which time an AC overlay would be placed to accommodate 80,000-lb truck loads without year around restrictions. The remaining life analysis showed that the chip seal roads can carry 80,000-lb maximum truck loads on a year around basis; however, they will require rehabilitation with an AC overlay within approximately 5 to 10 years. The overlay thickness would be designed at the time of rehabilitation to account for damage that accumulates since the present survey; however, as per IDOT standards, it would likely be a minimum of 3 inches of AC.

The final option is to perform AC overlay within the next couple of years, using a 3-inch AC overlay, as shown in table 8. This is the most reliable strategy for removing spring and year around load restrictions, and also the most expensive. The overlays would be placed directly over the existing chip seal surface without milling and maintain the existing 11-ft minimum traffic lane. The only exception is CH 25 in Metamora, where the pavement would need to be milled to match the current grade due to the existing concrete curb and gutter. In addition to providing adequate structural capacity for 80,000-lb trucks, the AC overlay provides the opportunity to achieve a 1.5 to 2 percent cross slope, as required by IDOT geometric standards. AC overlay would also require construction of a 4-ft wide turf or aggregate wedge shoulder. The existing right-of-way, side slopes, and ditches would need to be surveyed to determine if sufficient space exists to construct 4-ft wide shoulders, without needing performing earthwork.

Given the high cost of rehabilitating these roads through widening and AC overlay, ARA recommends giving consideration to full-depth reclamation (FDR) as a rehabilitation alternative. We did not have cost data available to include in the following tables; however, FDR has been used extensively on Illinois local roads and is likely cost competitive and will provide longer pavement life than traditional overlays. FDR pulverizes the upper pavement layers, in this case the chip seals and aggregate base, which is then mixed with a stabilizing agent to serve as the new base. It typically reclaims to a depth of 8 to 12 inches and can also be used for base widening at the same time. FDR bases can either be overlaid with AC or a surface treatment, such as a Cape seal.

Cost Estimates

ARA prepared preliminary cost estimates for each treatment option based on 2021 construction prices reported from Bidtabs.net. We used the average unit costs for Illinois, as of November, 2021. Table 9 presents the unit costs, quantities, and the cost per centerline-mile to perform each activity associated with the four treatment options considered in this analysis. The costs are based on two 11-ft wide lanes per mile.

Table 9. Unit cost and quantity data.

Item	Unit Cost, $/Unit	Units	Quantity per CL-Mile	Cost, $/CL-Mile
Place a single layer chip seal	2.00	SY	12,907	25,813
Excavate a 2’ base widening	56.17	CY	782	43,937
2’ Base widening – place aggregate	19.18	SY	2,347	45,009
4’ Aggregate wedge shoulder	39.19	CY	196	7,664
3” Pavement milling	2.79	SY	12,907	36,010
Place tack coat	0.54	SY	12,907	6,970
Place a 3” AC overlay	227.50	CY	1,076	244,689

Table 10 shows the activities and costs to perform each of the four treatment options on a per centerline-mile basis.

Table 10. Treatment unit cost data.

Item	Treatment Cost, $/CL-Mile			
	Chip Seal	Chip Seal w/Widening	AC Overlay w/Widening	Mill w/AC Overlay and Widening
Place a single layer chip seal	25,813	25,813		
Excavate a 2' base widening		43,937	43,937	43,937
2' Base widening – place aggregate		45,009	45,009	45,009
4' Aggregate wedge shoulder			7,664	7,664
3" Pavement milling				36,010
Place tack coat			6,970	6,970
Place a 3" AC overlay			244,689	244,689
Total, $/CL-Mile	25,813	114,760	348,269	384,278
Total, $/SY	2.00	8.89	26.98	29.77

The total cost for each feasible treatment per road is shown in table 11, based on each road's centerline length.

Table 11. Treatment costs per road.

CH	Length, CL-Miles	Treatment Cost per Road, $			
		Chip Seal	Chip Seal w/Widening	AC Overlay w/Widening	Mill w/AC Overlay and Widening
2	11.38	293,756	1,305,967	3,963,299	
3	6.52	168,303	748,234	2,270,713	
7	7.01	180,951	804,466	2,441,364	
9	4.05	104,544	464,777	1,410,489	
13	7.62	196,698	874,470	2,653,808	
18	10.03	258,908	1,151,041	3,493,136	
20	4.02	103,770	461,334	1,400,041	
25	7.95	205,216	912,341	2,768,737	
25[a]	0.44	11,358			169,082

[a] Within Metamora village limits, where concrete curb and gutter is present.

CONCLUSIONS AND RECOMMENDATIONS

The following are ARA's findings and conclusions regarding the structural evaluation of Woodford County highways, the assessment of existing load limits, and future treatments.

- Seven of the eight roads (CH 2, 3, 7, 9, 13, 18, and 20) are very similar in their pavement structure and structural response. They consist of 2-5 inches of accumulated chip seals over an aggregate base of at least 12 inches over a weak to fair fine-grained soil. The remaining road (CH 25) is a combination of AC and chip seals, totaling more than 7 inches, over a 12-inch aggregate base, and it has a higher structural capacity than the chip seal roads. The roads are typically 11-ft wide travel lanes with narrow shoulder, primarily loose stone over turf. The current practice of periodic chip sealing has maintained the roads in good condition and provided a safe and reliable driving
- The spring load limit analysis showed that all roads are expected to be capable of supporting a 10-ton single axle without exceeding the subgrade stress ratio. This analysis does not account for other possible failure modes, such as base failure or surface rutting. Also, it was reported that the aggregate base layers do not extend beyond the width of the traveled lane, which could potentially lead to pavement damage for heavy wheel loads operating near the pavement edge during periods of high moisture or weak base/subgrade strength. Despite these limitations and potential concerns, ARA believes the roads are sufficiently structurally sound that the current 5-ton January to April load limits may be removed. The roads should be monitored for pavement damage, especially after the spring thaw, and restrictions should be reconsidered, if unacceptable damage levels occur.
- The remaining life analysis based on the IDOT BLR Modified AASHTO approach showed the roads have expected remaining service lives ranging from 2 to 13 years, with the exception of CH 25, which has higher structural capacity than the chip seal roads. The short section of CH 25 in Metamora is structurally sound, but currently needs a functional overlay to remove surface distress and restore rideability. Traffic levels on the chip seal roads are low, ranging from 55,000 to 222,000 design ESALs over the next 20 years. CH 25 design ESALs range from 430,000 in the rural areas to 1,200,000 in Metamora. This analysis is based on removal of the current year around load limits and increasing the maximum gross truck load to 80,000 lbs. It was developed for AC pavements, not chip seal roads, but due to the significant accumulation of chip seal layers, the current pavements are behaving more like thin, low-modulus AC pavements, rather than surface-treated, aggregate roads. Despite this limitation, ARA believes the current year around load limits of 21 and 33 tons for three- and five-axle tractor trailer combinations, respectively, can be removed. The roads should be monitored for pavement damage, especially after periods of significant heavy vehicle use (i.e., fall harvest), and restrictions should be reconsidered, if unacceptable damage levels occur.
- The overlay design was based on the IDOT BLR Modified AASHTO approach, using two methods for characterizing the pavement existing SN—FWD-based and structural coefficients. Overall, the structural numbers from the FWD-based method ranged from 2.2 to 2.5 inches, typical of thin flexible pavements. The structural numbers using IDOT-recommended structural coefficients for the chip seal and aggregate layers produced slightly lower SN values, and therefore, slightly higher required overlay thicknesses. ARA averaged the overlay thicknesses from both methods and compared to the IDOT-required minimum overlay thickness of 3 inches of AC, based on required future SN's ranging from 2.5 to 3.49 inches. Therefore, the recommended overlay thickness for all roads is 3 inches of AC.

- ARA considered four treatment options for the roads and estimated construction costs based on 2021 average unit cost data for Illinois. The four treatment options are—continued use of routine chip sealing (e.g., every 5 years), chip sealing combined with a 2-ft base widening to provide additional edge support, continued treatment with chip seals, deferring AC overlay until a later date, and placement of AC overlays within the next few years. We developed a per centerline-mile unit cost for the different options, as well as the total cost to perform each for each road. Given the significant investment required to perform base widening and AC overlays, as well as the low traffic volumes on the majority of roads, ARA recommends removing the spring and year-around load restrictions and the continued use of periodic chip seals. The roads should be monitored for any damage occurring due to the removal of load restrictions, and that satisfactory performance is achieved. If this is not the case, the County has the option to either reconsider load restrictions, or proceed with structural improvements, such as base widening and AC overlays.
- It is highly recommended to perform a limited amount of FWD deflection testing during the spring, timed to measure deflections at the subgrade's weakest state, to verify the assumptions and results of the spring load analysis and to inform the decision of year-around load postings. The FWD tests can be optimized by selecting representative locations, basAed on the data collected for this study.

APPENDIX A

CORING LOG

(attached electronically)

APPENDIX B

FWD RESULTS

(attached electronically)

APPENDIX C

GEOMETRIC DATA

(attached electronically)