APPENDIX A:

Existing Roadway Characteristics

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Roadway Characteristics

This document provides a detailed compilation of the existing roadway characteristics of the Wendell H. Ford Western Kentucky Parkway (WKP) and draws a comparison with AASHTO guidelines for interstate facilities. Specifically, the *2018 Green Book*, *A Policy on Design Standards – Interstate System*, and, where necessary, KYTC's *Highway Design Manual* were used to determine whether the existing conditions of the WKP meet all necessary interstate standards. The *2018 Green Book* references a recommended range of values for critical dimensions based on established practices and recent research while *A Policy on Design Standards – Interstate System* provide minimum criteria for a roadway to be regarded as an interstate facility. These references address the 10 controlling criteria for design outlined by FHWA as well as other design standards for interstate facilities. The evaluation of the existing roadway characteristics throughout this corridor was based on as-built plans, KYTC Highway Information System (HIS) data, statewide LiDAR data, and limited field review and observations.

1.1 Mainline Characteristics

The following sections describe the various roadway characteristics for the mainline excluding structures and interchanges, which will be described in **Section 1.2** and **1.3**, respectively.

Terrain

According to the *2018 Green Book,* the topography of the land traversed by a roadway determines its maximum grade and minimum design speed. The variation in topography is classified into three categories: level, rolling, and mountainous terrain. According to the HIS database the terrain for the WKP is flat. However, all as-built plans for the corridor utilize design criteria for rolling terrain. The *2018 Green Book* defines rolling terrain as natural slopes that consistently rise above and fall below the road grade, and occasional steep slopes offer some restriction to normal horizontal and vertical roadway alignment. It may be necessary for roadways that navigate through rolling terrain to have steeper vertical grades than those that navigate through level terrain, causing trucks to reduce speeds below those of passenger cars. To be consistent with the as-built plans for this corridor, rolling terrain was used to analyze the roadway geometry.

Design Speed

The AASHTO minimum design speed for a rural interstate through non-mountainous terrain is 70 mph. According to the as-built plans, there are several horizontal and vertical curves that do not meet the design criteria for a roadway with a 70 mph design speed according to the *2018 Green Book*. Those instances will be discussed in the **Horizontal Alignment** and **Vertical Alignment** paragraphs later in this section. The current posted speed limit throughout the corridor is 70 mph.

Typical Roadway Sections

There are three typical sections along the WKP in the study area (MP 38.326 to MP 77.143). The normal typical section shown in **Figure 1** occurs from MP 38.326 to MP 39.000, MP 40.700 to MP 75.100, and MP 76.400 to MP 77.143. In addition to the normal section, there are two locations through this corridor where the eastbound and westbound lanes are bifurcated. The first bifurcated section occurs from MP 39.000 to MP 40.700 around the Kentucky State Police (KSP) Post No. 2, while the second occurs from MP 75.100 to MP 76.400 around the Beaver Dam Rest Area (Huck's). The limits of each typical section throughout this corridor are shown in **Figure 2**.

Figure 1. Normal Typical Section

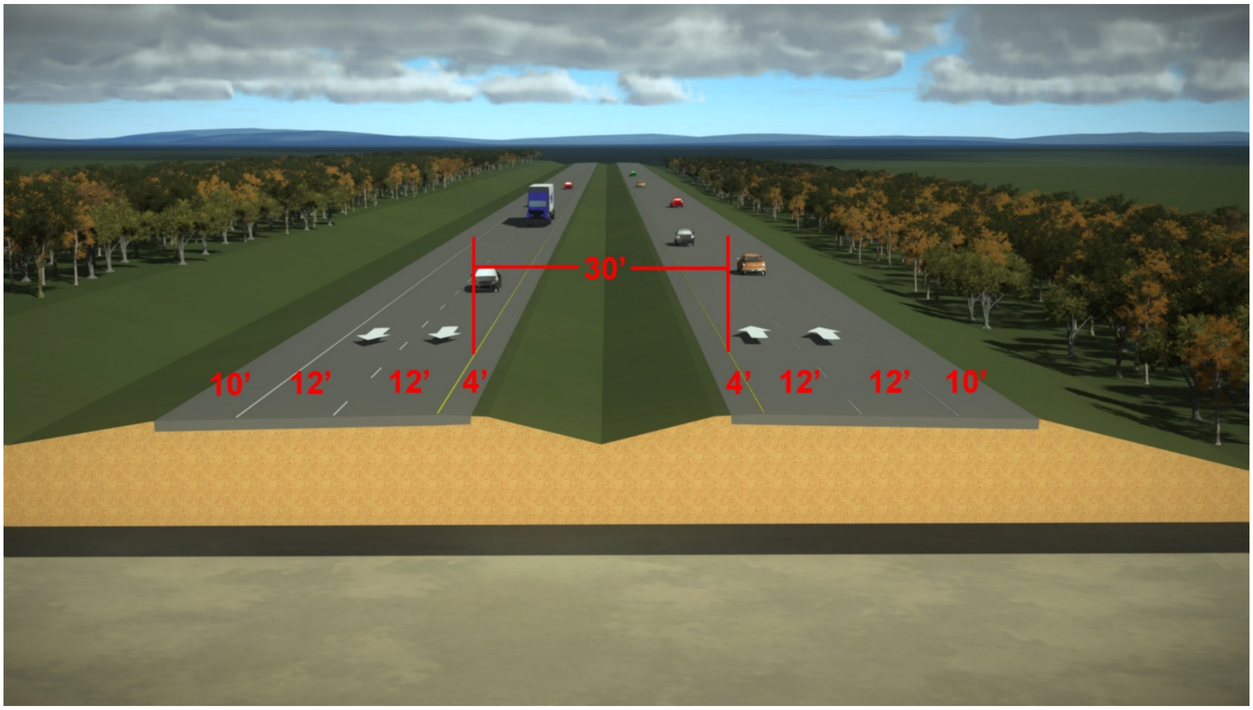
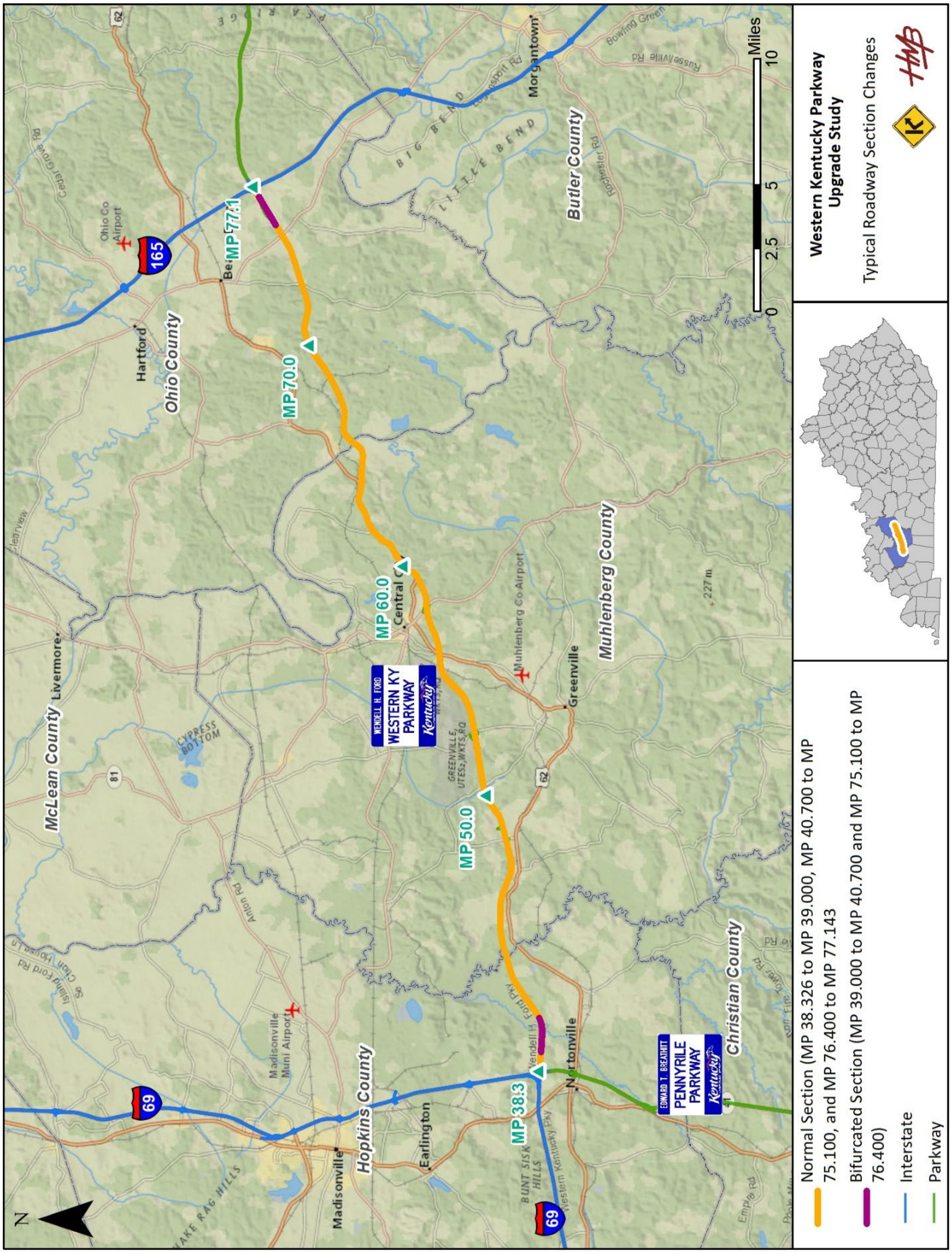


Figure 2. WKP Typical Section Overview



Lane Widths

The minimum lane width of an interstate facility is 12 feet. The HIS database shows that the lane widths from MP 38.326 to MP 43.424 are 11 feet, while the rest of the corridor is reported as having 12-foot lanes. It should be noted the HIS field measurements are usually taken in the non-Cardinal direction only; therefore, the database may not illustrate a complete representation.

All spot field measurements taken indicated that travel lanes were 12 feet wide, including locations where HIS noted 11 feet lanes. Additional field work may need to be completed in order to confirm that all travel lanes meet the minimum width throughout the study area. Based on overall roadway width, the roadway is wide enough to accommodate two 12-foot lanes in each direction, and any lane width less than 12 feet is most likely due to unbalanced restriping after roadway improvements such as a pavement rehabilitation project.

Shoulder Widths

The *A Policy on Design Standards – Interstate System* states, "Minimum paved shoulder widths in each direction of travel as a function of terrain and the number of through lanes shall be in accordance with the following table (**Table 1**)".

Table 1. Minimum Paved Shoulder Widths for Interstate Facility

One-Directional No. Through Lanes	Terrain	Left Shoulder (ft)	Right Shoulder (ft)
2-lane	Level or Rolling	4	10
3-lane or more	Level or Rolling	10	10
2 or 3-lane	Mountainous	4	8
4-lane or more	Mountainous	8	8

The minimum paved shoulder widths for a two-lane interstate facility through rolling terrain are 10 feet for the outside shoulder and 4 feet for the inside shoulder. Although the as-built typical section for the bifurcated section of roadway around Huck's shows a 3-foot inside paved shoulder, all field measurements in this area indicate that widening the inside paved shoulder to 4 feet was addressed on a previous 3R project. The shoulder widths are presented in **Table 2**.

Table 2. WKP Shoulder Widths

Rural/Urban	No. Lanes	Begin Milepoint	End Milepoint	Terrain	Left Shoulder (ft)	Right Shoulder (ft)	DDHV (Trucks)
Rural	2	38.326	39.000	Rolling	4	10	120
Rural	2	39.000	40.700	Rolling	10	10	120
Rural	2	40.700	75.100	Rolling	4	10	90 - 100
Rural	2	75.100	76.400	Rolling	4	10	140
Rural	2	76.400	77.143	Rolling	4	10	140

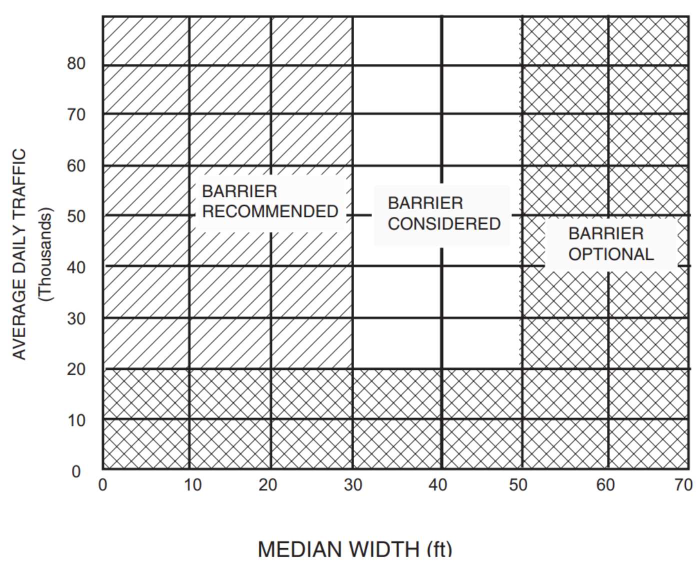
When the Directional Design Hour Volume (DDHV) for trucks for an interstate facility exceeds 250, *A Policy on Design Standards – Interstate System* suggests that additional shoulder width may be beneficial. As shown in **Table 2**, no Truck DDHV exceeds 250 for this section of the WKP. Therefore, the inside and outside paved shoulders of the WKP meet the design criteria for minimum widths according to *A Policy on Design Standards – Interstate System.*

Median Width and Type

The median of a roadway is provided for the purpose of driver comfort and safety. It is measured between the traveled lanes of opposing traffic and includes the width of the inside shoulders. According to *A Policy on Design Standards – Interstate System*, medians in rural areas in level or rolling topography should be at least 50 feet, and preferably 60 feet, wide. The WKP has a median width of 30 feet through the majority of this corridor, and therefore, does not meet minimum design criteria for an interstate facility.

When the minimum median width is not met, the *2018 Green Book* states that the *Roadside Design Guide* should be referenced in determining the use of median barriers. When referring to Figure 6-1 from the *Roadside Design Guide* pictured in **Figure 3**, median barriers are to be considered when the median width is greater than 30 feet and less than 50 feet and where the annual average daily traffic (AADT) is greater than 20,000 vehicles per day (vpd).

Figure 3. Guidelines for Median Barrier Installation



Source: Roadside Design Guide Figure 6-1

According to the traffic forecast that was completed as a part of this study and presented in the main body of the report and the Traffic Forecast Report, the AADT for the design year 2045 is less than 20,000 vpd throughout the corridor. The existing data on medians within the study area are shown in **Table 3** along with the ranges of AADT throughout each section.

Table 3. WKP Median Widths and Type

Rural/Urban	County	Begin Milepoint	End Milepoint	Length (Miles)	Median Type	Median Width	AADT, 2045 (vpd)
Rural	Hopkins	38.326	39.000	0.674	Depressed	30	14,500
Rural	Hopkins	39.000	40.700	1.700	Bifurcated	N/A	14,500
Rural	Hopkins/Muhlenburg/Ohio	40.700	75.100	34.400	Depressed	30	13,000 - 14,500
Rural	Ohio	75.100	76.400	1.300	Bifurcated	N/A	14,200
Rural	Ohio	76.400	77.143	0.743	Depressed	30	14,200

The crash analysis presented later in the main body of the report notes there were three recorded crossover crashes between January 1, 2014, and December 31, 2018, which would indicate there is not a significant history or concentration of crossover crashes within this corridor.

Median Turnarounds

The *2018 Green Book* states that median turnarounds may be provided where interchange spacing exceeds five miles to avoid excessive adverse travel for emergency and law enforcement vehicles. Median turnarounds may also be useful at one or both ends of interchange facilities for the purpose of snow removal and at other locations to facilitate maintenance operations. It is KYTC common practice to provide median turnarounds at county lines. They may be spaced at three to four-mile intervals, or as needed.

There are 23 median turnarounds located along this section of WKP as shown in **Table 4**. Twelve are gravel and do not appear to be maintained. It is KYTC common practice to eliminate these types of median turnarounds. The remaining 11 were evaluated for compliance/safety based on drainage, sight distance, the crash analysis, and AASHTO and KYTC guidelines.

Clear Zones / Foreslopes

The clear zone of a roadway is used for vehicle recovery, and it includes the area outside the edge of the travel lane that is free of obstructions. Table 3-1 in the *Roadside Design Guide* lays out guidelines for clear zone widths for roadways based on design speed, traffic volumes, fill / cut slopes, ditch slopes, and distances from fixed obstructions such as bridge piers, sign supports, culvert headwalls, trees, rock outcrops and drainage channels.

The *Roadside Design Guide* dictates the sideslopes within the clear zone be no steeper than 1V:4H, while 1V:6H sideslopes are preferable. As illustrated in **Table 5**, with a posted speed limit of 70 mph and an AADT greater than 6,000 vpd, to meet the minimum design criteria the WKP requires a clear zone ranging from 30-46 feet. This clear zone requirement is dependent on the sideslope of the roadway.

Table 4. Median Turnarounds

Location Milepoint	Pavement or Gravel	Drainage	Sideslopes	Nearest Distance to Interchange Ramp (miles)	Ramp Measured To:
38.775	Pavement	No Pipe; No DBI	8.5 E , 7.3	0.446	Exit 38
42.160	Pavement	No Pipe; No DBI	8.1 E , Flat W	3.828	Exit 38
42.890	Gravel	No Pipe; No DBI		4.533	Exit 38
43.350	Gravel	No Pipe; No DBI		4.993	Exit 38
44.950	Pavement	No pipe; DBI in Sag	5.2 , flat	3.070	Exit 48
48.775	Pavement	No pipe; DBI present		0.705	Exit 48
50.390	Gravel	None		2.323	Exit 48
54.790	Pavement	No Pipe	3.8 , 0.3	2.238	Exit 53
55.960	Gravel	None		1.996	Exit 58
57.200	Gravel	None		0.757	Exit 58
59.140	Gravel	None		1.170	Exit 58
61.440	Pavement	No pipe, DBI present	Flat E , 3.6% W	3.480	Exit 58
64.780	Pavement	No pipe, DBI present	5.5 E , 3.0 W	6.815	Exit 58
65.350	Gravel	None		7.395	Exit 58
65.810	Gravel	None		7.846	Exit 58
67.340	Gravel	None		7.290	Exit 75
67.370	Gravel	None	at end of bridge	7.253	Exit 75
68.200	Pavement	Perf. Pipe Slope Box outlets, both sides (northern side of median)	Flat , 13.3%	6.141	Exit 75
71.200	Pavement	Perf. Pipe Slope Box outlets, both sides (northern side of median)	7.3 , 6.8%	3.430	Exit 75
74.140	Pavement	No pipe, DBI present	7.8 , 7.6%	0.493	Exit 75
74.940	Pavement	Perf. Pipe Slope Box outlets, both sides (southern side of median)	3.3% W , 11.1%	0.359	Exit 75
76.700	Gravel	None		0.071	Exit 77
77.100	Gravel	None		0.334	Exit 77

Table 5. Minimum Clear Zone Requirements

Design Speed (mph)	Design AADT (vpd)	1V:6H or flatter (feet)	1V:5H to 1V:4H (feet)
65-70	UNDER 750	18 - 20	20 - 26
	750 - 1,500	24 - 26	28 - 36
	1,500 - 6,000	28 - 32	34 - 42
	OVER 6,000	30 - 34	38 - 46

Based on spot field measurements and visual inspection, the median slopes on the WKP are 1V:6H or flatter except for the areas where the eastbound and westbound lanes are bifurcated, while the sideslopes outside the roadway that are not protected by guardrail vary from 1V:6H to 1V:4H. Additional observations were made using aerial imagery and design software to determine that 113 locations outside of the roadway along this corridor do not meet minimum clear zone requirements. These locations are shown in **Table 6**. Additional field measurements may be necessary to determine every instance in which minimum clear zone is not met. If slopes / clear zone criteria are not met, then they should be analyzed to see if barrier protection is warranted.

Guardrail Placement and Condition

Guardrail is a barrier that runs parallel to the roadway for the purpose of shielding motorists from natural or manmade obstacles located on either side of the traveled way within the clear zone. It prevents vehicles from potentially leaving the roadway by absorbing the vehicle's energy and protecting it from roadside hazards. Chapter 5 of the *Roadside Design Guide* provides guidance on the application and situation of guardrail placement.

Aerial imagery and a limited field review provided enough information to determine the location and a sampling of heights for all guardrail and guardrail end treatments along the WKP. Based on this information, all guardrail end treatments throughout this corridor appear adequate for an interstate facility. A more in-depth field review may be required in order to verify the findings of this study.

According to the *KYTC Standard Drawings and Active Sepias*, any new guardrail shall be installed at a height of 31 inches. This height was revised from a previous installation height of 29 inches with an active sepia after the *KYTC 2016 Standard Drawings* were released. On previous 3R projects, all guardrail with a height of 27 inches has been deemed adequate and left in place. Several field measurements were taken to assess whether the guardrail height was appropriate throughout this corridor. Of those limited field measurements, 44% were measured less than 27 inches, 56% were measured 27 inches or greater, and all of the field measurements were less than the most recent KYTC standard of 31 inches. Additional field measurements are necessary to determine how much guardrail needs to be replaced on the WKP.

Table 6. Clear Zone Deficiencies

County	Direction	Side of Road	Beginning Milepoint	Ending Milepoint	Existing Clear Zone (ft)	Length (miles)	Remarks
Hopkins	EB	RT	38.600	38.900	20	0.300	
Hopkins	WB	RT	38.700	38.900	22	0.200	
Hopkins	EB	RT	39.000	39.100	20	0.100	
Hopkins	EB	LT	39.300	39.400	18	0.100	
Hopkins	EB	RT	39.300	39.400	21	0.100	
Hopkins	EB	RT	40.600	40.700	20	0.100	
Hopkins	WB	RT	40.600	40.700	19	0.100	
Hopkins	EB	RT	41.100	41.300	20	0.200	
Hopkins	EB	RT	41.300	41.500	19	0.200	
Hopkins	EB	RT	41.700	41.800	19	0.100	
Hopkins	WB	RT	41.800	41.900	19	0.100	
Hopkins	EB	RT	41.900	42.000	19	0.100	
Hopkins	EB	RT	42.100	42.300	20	0.200	
Hopkins	EB	RT	42.400	42.500	18	0.100	
Hopkins	WB	RT	42.400	42.500	20	0.100	
Muhlenburg	EB	RT	43.800	44.000	20	0.200	
Muhlenburg	EB	RT	44.000	44.300	24	0.300	
Muhlenburg	WB	RT	43.800	44.000	20	0.200	
Muhlenburg	WB	RT	44.200	44.300	20	0.100	
Muhlenburg	EB	RT	44.800	44.900	19	0.100	
Muhlenburg	WB	RT	44.800	44.900	18	0.100	
Muhlenburg	EB	RT	45.000	45.100	24	0.100	
Muhlenburg	WB	RT	45.200	45.300	20	0.100	
Muhlenburg	EB	RT	45.200	45.300	18	0.100	
Muhlenburg	EB	RT	45.400	45.700	20	0.300	
Muhlenburg	WB	RT	45.500	45.800	24	0.300	
Muhlenburg	WB	RT	46.000	46.000	22	0.050	
Muhlenburg	EB	RT	46.100	46.400	22	0.300	
Muhlenburg	WB	RT	46.100	46.400	22	0.300	
Muhlenburg	EB	RT	46.700	47.200	20	0.500	
Muhlenburg	WB	RT	46.800	47.000	19	0.200	
Muhlenburg	EB	RT	47.400	47.500	19	0.100	
Muhlenburg	WB	RT	47.400	47.600	20	0.200	
Muhlenburg	WB	RT	47.700	47.800	22	0.100	
Muhlenburg	WB	RT	48.900	49.100	27	0.200	
Muhlenburg	EB	RT	49.300	49.500	21	0.200	
Muhlenburg	WB	RT	49.300	49.500	19	0.200	
Muhlenburg	EB	RT	49.800	50.100	15	0.300	
Muhlenburg	EB	RT	50.400	50.700	20	0.300	
Muhlenburg	WB	RT	50.400	50.500	22	0.100	
Muhlenburg	WB	RT	51.000	51.400	16	0.400	Rock
Muhlenburg	EB	RT	53.000	53.200	20	0.200	
Muhlenburg	WB	RT	53.000	53.200	17	0.200	Rock
Muhlenburg	WB	RT	53.200	53.300	20	0.100	
Muhlenburg	WB	RT	53.400	53.900	22	0.500	
Muhlenburg	WB	RT	54.500	55.000	22	0.500	
Muhlenburg	EB	RT	54.900	55.000	25	0.100	

Table 6. Clear Zone Deficiencies (Cont.)

County	Direction	Side of Road	Beginning Milepoint	Ending Milepoint	Existing Clear Zone (ft)	Length (miles)	Remarks
Muhlenburg	EB	RT	55.700	55.900	16	0.200	Rock
Muhlenburg	WB	RT	55.800	55.900	18	0.100	Rock
Muhlenburg	EB	RT	56.700	56.800	20	0.100	
Muhlenburg	WB	RT	56.800	57.000	20	0.200	Rock
Muhlenburg	EB	RT	56.800	57.000	19	0.200	Rock
Muhlenburg	EB	RT	57.100	57.200	20	0.100	
Muhlenburg	WB	RT	57.100	57.200	17	0.100	
Muhlenburg	EB	RT	57.200	57.400	20	0.200	
Muhlenburg	WB	RT	57.300	57.500	20	0.200	
Muhlenburg	EB	RT	57.700	57.800	21	0.100	
Muhlenburg	WB	RT	57.700	57.800	20	0.100	
Muhlenburg	EB	RT	58.200	58.400	19	0.200	
Muhlenburg	WB	RT	58.200	58.300	16	0.100	
Muhlenburg	EB	RT	58.300	58.400	20	0.100	
Muhlenburg	WB	RT	58.400	58.700	18	0.300	Rock
Muhlenburg	EB	RT	58.600	58.700	18	0.100	
Muhlenburg	EB	RT	58.900	59.100	19	0.200	
Muhlenburg	WB	RT	58.900	59.100	17	0.200	Rock
Muhlenburg	EB	RT	59.300	59.400	20	0.100	
Muhlenburg	WB	RT	59.400	59.500	14	0.100	
Muhlenburg	EB	RT	59.700	59.800	14	0.100	
Muhlenburg	WB	RT	59.700	59.800	15	0.100	
Muhlenburg	EB	RT	60.000	60.100	22	0.100	
Muhlenburg	WB	RT	60.000	60.100	16	0.100	
Muhlenburg	WB	RT	60.200	60.300	16	0.100	
Muhlenburg	EB	RT	60.200	60.300	16	0.100	
Muhlenburg	WB	RT	60.500	60.600	16	0.100	
Muhlenburg	EB	RT	60.600	60.900	16	0.300	
Muhlenburg	WB	RT	60.700	60.900	16	0.200	Rock
Muhlenburg	WB	RT	61.000	61.100	16	0.100	Rock
Muhlenburg	WB	RT	61.500	61.600	22	0.100	
Muhlenburg	EB	RT	61.700	61.800	20	0.100	
Muhlenburg	WB	RT	61.800	61.900	15	0.100	
Muhlenburg	EB	RT	61.900	62.000	17	0.100	
Muhlenburg	WB	RT	61.900	62.100	22	0.200	
Muhlenburg	EB	RT	62.500	62.700	17	0.200	
Muhlenburg	WB	RT	62.500	62.700	17	0.200	
Muhlenburg	WB	RT	62.900	63.000	26	0.100	
Muhlenburg	EB	RT	63.100	63.200	20	0.100	
Muhlenburg	WB	RT	63.300	63.400	23	0.100	
Muhlenburg	EB	RT	63.500	63.600	17	0.100	Rock
Muhlenburg	WB	RT	63.500	63.600	18	0.100	Rock
Muhlenburg	WB	RT	63.900	64.100	18	0.200	
Muhlenburg	EB	RT	63.900	64.000	18	0.100	
Ohio	EB	RT	65.800	66.000	19	0.200	
Ohio	WB	RT	65.900	66.000	21	0.100	
Ohio	WB	RT	66.900	67.000	16	0.100	

Table 6. Clear Zone Deficiencies (Cont.)

County	Direction	Side of Road	Beginning Milepoint	Ending Milepoint	Existing Clear Zone (ft)	Length (miles)	Remarks
Ohio	WB	RT	67.200	67.300	13	0.100	Rock
Ohio	EB	RT	67.300	67.400	20	0.100	
Ohio	EB	RT	67.500	67.600	18	0.100	
Ohio	EB	RT	67.700	67.900	17	0.200	
Ohio	WB	RT	67.800	67.900	18	0.100	
Ohio	EB	RT	68.000	68.100	16	0.100	
Ohio	WB	RT	68.000	68.100	17	0.100	Rock
Ohio	EB	RT	68.600	68.700	16	0.100	Rock
Ohio	WB	RT	68.600	68.700	14	0.100	Rock
Ohio	EB	RT	68.800	69.000	24	0.200	
Ohio	WB	RT	68.900	69.000	21	0.100	
Ohio	EB	RT	69.200	69.300	17	0.100	
Ohio	EB	RT	70.300	70.400	19	0.100	
Ohio	WB	RT	70.300	70.500	16	0.200	
Ohio	WB	RT	70.700	70.800	17	0.100	
Ohio	EB	RT	72.000	72.100	17	0.100	
Ohio	WB	RT	72.000	72.100	19	0.100	
Ohio	WB	RT	73.500	73.600	19	0.100	
Ohio	EB	RT	76.100	76.200	20	0.100	

Horizontal Alignment

According to the *2018 Green Book*, the design of roadway curves should be based on an appropriate relationship between design speed and curvature as well as their joint relationships with superelevation and side friction. Information was extracted from the as-built plans of the WKP in order to assess whether these relationships meet minimum standards for a 70 mph design speed.

Superelevation Rate

Superelevation is the physical tilting of the roadway to help counteract the centripetal forces developed as a vehicle travels around a curve. Superelevation represents the ratio of the vertical difference between each edge of the roadway to the roadway width. This value is often expressed as a percentage.

The *2018 Green Book* allows for the use of maximum superelevation rates from 4% to 12%. The maximum rate of superelevation used on a particular roadway is controlled by climate conditions, terrain conditions, type of area, and frequency of slow-moving vehicles that may be affected by high superelevation rates. According to *A Policy on Design Standards – Interstate System,* curvature and superelevation shall be designed in accordance with the *2018 Green Book.* AASHTO, however, does not recommend a maximum superelevation for an interstate facility. The decision lies with user agencies to make specific policies concerning maximum allowable rates of superelevation. KYTC policy for a rural interstate facility is to use the $e_{max} = 8\%$ superelevation table in the *2018 Green Book*; however, when this portion of the WKP was designed, it appears that a maximum superelevation of 10% was utilized. It is currently KYTC common practice to use the $e_{max} = 8\%$ superelevation table found in the *2018 Green Book* in when determining the design speed of existing curves.

Horizontal curve data was obtained from the as-built plans of the WKP, and a design speed was extracted from the $e_{max} = 8\%$ superelevation table using each curve's radius and superelevation rate. Several curves throughout this corridor did not meet the design criteria for a 70 mph design speed based on this table. The side friction factor for each curve was then calculated given the desired design speed (V), the radius of each curve (R), and the superelevation of each curve (e) using the following formula:

$$\boldsymbol{f = \frac{V^2}{15R} - 0.01e}$$

The maximum allowable side friction factor using a 70 mph design speed for both a maximum superelevation of 8% and 10% is 0.10 according to Table 3-7 in the *2018 Green Book*. As shown in **Table 7**, none of the horizontal curves throughout this corridor exceeded the maximum allowable side friction factor for a 70 mph design speed. Based on previous 3R projects, if the maximum allowable side friction factor was not exceeded for the desired design speed, then the horizontal curve in question was deemed adequate. A high crash spot analysis using critical rate factors (CRF) is be presented in Section 6.1 of the main body of the report. A more in-depth field review may be required for curves that fall within a high crash spot considering the horizontal curve data was determined from the as-built plans alone. These locations are noted on **Table 7**. Any improvements to this corridor such as pavement rehabilitation projects may have resulted in changes to information that may not be represented in this study.

Table 7. Mainline Horizontal Curve Data

County	Direction	Begin Milepoint	End Milepoint	Radius (ft)	Existing Superelevation (As-Built Plans)	Current AASHTO Design Speed 8% Table	Side Friction Factor (f)	Superelevation Needed to Meet 70 mph 8% Table	High CRF Spot
Hopkins	EB	39.000	39.250	4583.66	3.50%	60	**0.036**	4.40%	YES
Hopkins	EB	39.350	39.600	4583.66	3.50%	60	**0.036**	4.40%	YES
Hopkins	EB	39.800	40.700	7639.44	2.00%	55	**0.023**	2.80%	NO
Hopkins	WB	40.032	40.734	5729.58	2.80%	55	**0.029**	3.40%	YES
Hopkins	EB/WB	41.937	42.626	11459.16	NC	55	**0.049**	RC	NO
Muhlenberg	EB/WB	43.575	44.535	17188.74	NC	75	**0.039**	N/A	YES
Muhlenberg	EB/WB	45.210	46.076	11459.16	NC	55	**0.049**	RC	YES
Muhlenberg	EB/WB	46.548	47.240	5729.58	2.80%	55	**0.029**	3.40%	YES
Muhlenberg	EB/WB	48.762	49.180	5729.58	2.80%	55	**0.029**	3.40%	NO
Muhlenberg	EB/WB	49.677	50.345	7639.44	RC	55	**0.023**	2.80%	YES
Muhlenberg	EB/WB	53.237	54.245	8594.37	1.85%	60	**0.020**	2.60%	YES
Muhlenberg	EB/WB	54.908	55.674	5729.58	2.80%	55	**0.029**	3.40%	YES
Muhlenberg	EB/WB	59.257	59.518	1909.86	8.30%	70	**0.088**	N/A	YES
Muhlenberg	EB/WB	59.810	60.040	1909.86	8.30%	70	**0.088**	N/A	NO
Muhlenberg	EB/WB	60.870	61.094	1909.86	8.30%	70	**0.088**	N/A	NO
Muhlenberg	EB/WB	56.107	56.764	11459.16	NC	55	**0.049**	RC	NO
Muhlenberg	EB/WB	57.322	58.086	11459.16	NC	55	**0.049**	RC	YES
Muhlenberg	EB/WB	58.217	58.558	4583.66	3.50%	60	**0.036**	4.40%	YES
Muhlenberg	EB/WB	61.268	61.403	1909.86	8.30%	70	**0.088**	N/A	YES
Muhlenberg	EB/WB	61.835	62.235	4583.66	3.50%	60	**0.036**	4.40%	NO
Muhlenberg	EB/WB	62.487	62.811	5729.58	2.80%	55	**0.029**	3.60%	YES
Muhlenberg	EB/WB	63.008	63.300	1909.86	8.30%	70	**0.088**	N/A	NO
Muhlenberg	EB/WB	63.583	64.033	11459.16	NC	55	**0.049**	RC	YES
Muhlenberg	EB/WB	64.238	64.431	22918.32	NC	80	**0.034**	N/A	YES
Muhlenberg	EB/WB	65.000	65.338	1909.86	8.30%	70	**0.088**	N/A	YES
Ohio	EB/WB	65.878	66.406	2864.79	5.50%	60	**0.059**	6.60%	YES
Ohio	EB/WB	66.814	67.392	5729.58	2.80%	55	**0.029**	3.60%	YES
Ohio	EB/WB	68.309	68.713	11459.16	NC	55	**0.049**	RC	NO
Ohio	EB/WB	69.294	69.418	22918.31	NC	80	**0.034**	N/A	NO
Ohio	EB/WB	69.907	71.087	7639.44	2.00%	55	**0.023**	2.80%	NO
Ohio	EB/WB	71.339	71.823	4583.66	3.30%	55	**0.038**	4.40%	YES
Ohio	EB/WB	73.773	74.306	11459.16	NC	55	**0.049**	RC	NO
Ohio	WB	75.000	75.200	5729.58	2.80%	55	**0.029**	3.60%	YES
Ohio	WB	75.400	76.000	5729.58	2.80%	55	**0.029**	3.60%	YES
Ohio	WB	76.100	76.300	5729.58	2.80%	55	**0.029**	3.60%	YES

NC – Normal Crown

RC – Reverse Crown

N/A – Not Applicable; horizontal curve meets design speed

Degree of Horizontal Curvature

The minimum horizontal curvature radius for a 70 mph design speed on a rural interstate is 1,810 feet according to the $e_{max} = 8\%$ superelevation table found in the *2018 Green Book*, which equates to approximately 3°10’ of curvature. All mainline horizontal curves throughout this corridor meet these minimum criteria.

There are six curves that have a radius of 1,910 feet, which is the smallest radius found within this corridor. The radii of these curves exceed the minimum radius for a 70 mph design speed.

Cross Slopes

According to *A Policy on Design Standards – Interstate System*, the normal cross slope of the traveled way is 2.0% and shall not be less than 1.5%, and paved shoulders should have a cross slope in the range of 2.0% to 6.0% but shall not have a cross slope less than the cross slope of the adjacent traveled way.

A review of the as-built plans shows that driving lane cross slopes on the WKP are 1.56%, while the shoulder cross slopes are 4.17%. In reality, the cross slopes of both the travel lanes and shoulders likely vary from the as-built plans throughout the corridor due to previous resurfacing and rehabilitation projects. A limited field review revealed several instances in which the driving lane cross slope was less than the normal cross slope on the as-built plans:

- Driving Lane Cross Slope < 1.5%: MP 38.68 and MP 38.84
- Driving Lane Cross Slope < 1.0%: MP 44.11, MP 44.95, MP 51.25, MP 73.82, MP 74.12

Additional field work is required to determine all instances in which the cross slopes of the WKP do not meet the minimum cross slope according to *A Policy on Design Standards – Interstate System.*

Vertical Alignment

The topography has some effect on horizontal alignment, but its effect on a roadway’s vertical alignment is even more substantial. As stated previously, the topography of the land traversed by a roadway determines its maximum grade. It has been established that the WKP was designed and constructed utilizing design criteria for rolling terrain. The vertical alignment inventory is included in **Table 8.**

Vertical Grade

Vertical grades up to 4% can be used in rolling terrain for a 70 mph design speed according to *A Policy on Design Standards – Interstate System*. According to the as-built plans, the maximum grade utilized on WKP is 4%, which meets the design criteria for rural sections through rolling terrain.

Table 8. Mainline Vertical Curve Data

Direction	Milepoint of VPI	Grade 1 (%)	Grade 2 (%)	A (%)	Length of Curve (ft)	Sag / Crest	Minimum L for SSD	Minimum L for HLSD	Meets AASHTO
Both	38.570	-2.50	-0.50	2.00	600	Sag		INFINITE	YES
Westbound	39.120	-0.50	0.98	1.48	800	Sag		266.9008	YES
Eastbound	39.120	-0.50	2.56	3.06	800	Sag		551.8355	YES
Westbound	39.350	0.98	-0.75	1.73	1000	Crest	427.209		YES
Eastbound	39.350	2.56	-1.53	4.09	1750	Crest	1009.497		YES
Eastbound	39.900	-1.53	0.00	1.53	800	Sag		275.5571	YES
Westbound	40.000	-0.75	0.00	0.75	800	Sag		135.2538	YES
Westbound	40.100	0.00	0.40	0.40	600	Sag		INFINITE	YES
Eastbound	40.150	0.00	0.54	0.54	400	Sag		INFINITE	YES
Westbound	40.200	0.40	-0.30	0.70	600	Crest	INFINITE		YES
Eastbound	40.200	0.54	-0.30	0.84	400	Crest	INFINITE		YES
Westbound	40.400	-0.30	0.75	1.05	650	Sag		INFINITE	YES
Eastbound	40.500	-0.30	1.00	1.30	1500	Sag		234.4399	YES
Westbound	40.600	0.75	-0.78	1.53	800	Crest	378.8084		YES
Eastbound	40.600	1.00	-0.78	1.78	800	Crest	439.5561		YES
Both	40.800	-0.78	1.00	1.78	800	Sag		321.0024	YES
Both	41.500	1.00	-0.80	1.80	800	Crest	444.4949		YES
Both	41.700	-0.80	0.75	1.55	600	Sag		INFINITE	YES
Both	42.200	0.75	-1.00	1.75	800	Crest	432.1478		YES
Both	42.700	-1.00	0.30	1.30	1500	Sag		234.4399	YES
Both	42.900	0.30	0.49	0.19	400	Sag		INFINITE	YES
Both	43.000	-0.49	0.00	0.49	400	Sag		INFINITE	YES
Both	43.200	0.00	0.30	0.30	400	Sag		INFINITE	YES
Both	43.500	0.30	-0.30	0.60	600	Crest	INFINITE		YES
Both	43.700	-0.30	0.95	1.25	400	Sag		INFINITE	YES
Both	44.000	0.95	-0.80	1.75	800	Crest	432.6417		YES
Both	44.300	-0.80	0.00	0.80	400	Sag		INFINITE	YES
Both	44.500	0.00	0.50	0.50	400	Sag		INFINITE	YES
Both	45.000	0.50	1.00	0.50	400	Sag		INFINITE	YES
Both	45.600	1.00	2.00	1.00	400	Sag		INFINITE	YES
Both	46.100	2.00	1.98	0.02	300	Crest	INFINITE		YES
Both	46.400	1.98	-1.40	3.38	1500	Crest	834.4157		YES
Both	47.100	-1.40	0.83	2.23	400	Sag		136.0753	YES
Both	47.300	0.83	-0.83	1.66	800	Crest	411.0096		YES
Both	47.600	-0.83	0.60	1.43	400	Sag		INFINITE	YES
Both	48.000	0.60	-2.00	2.60	1150	Crest	642.0482		YES
Both	48.200	0.20	-0.59	0.79	400	Crest	INFINITE		YES
Both	48.400	-0.59	0.50	1.09	400	Sag		INFINITE	YES
Both	48.600	0.50	-0.50	1.00	500	Crest	INFINITE		YES
Both	48.900	-0.50	0.67	1.17	400	Sag		INFINITE	YES
Both	49.200	0.67	2.70	2.03	400	Sag		6.48303	YES
Both	49.500	2.70	0.82	1.88	800	Crest	464.2502		YES
Both	49.700	0.82	1.59	0.77	400	Sag		INFINITE	YES
Both	50.000	1.59	-3.10	4.69	2000	Crest	1158.156		YES
Both	50.300	-3.10	1.40	4.50	800	Sag		811.5228	NO
Both	50.600	1.40	4.00	2.60	600	Sag		323.4615	YES
Both	51.200	4.00	-4.00	8.00	3400	Crest	1975.533		YES
Both	51.300	-4.00	-0.75	3.25	600	Sag		550.7692	YES
Both	51.600	-0.75	1.70	2.45	400	Sag		253.8776	YES

Table 8. Mainline Vertical Curve Data (cont.)

Direction	Milepoint of VPI	Grade 1 (%)	Grade 2 (%)	A (%)	Length of Curve (ft)	Sag / Crest	Minimum L for SSD	Minimum L for HLSD	Meets AASHTO
Both	51.900	1.70	-2.50	4.20	1500	Crest	1037.155		YES
Both	52.300	-2.50	0.50	3.00	600	Sag		475	YES
Both	52.500	0.50	-0.50	1.00	600	Crest	INFINITE		YES
Both	53.100	-0.50	-1.00	0.50	400	Crest	INFINITE		YES
Both	53.600	-1.00	0.32	1.32	400	Sag		INFINITE	YES
Both	53.800	0.32	-0.33	0.65	400	Crest	INFINITE		YES
Both	54.400	-0.33	0.50	0.83	400	Sag		INFINITE	YES
Both	54.800	0.50	-0.50	1.00	600	Crest	INFINITE		YES
Both	55.500	-0.50	2.20	2.70	800	Sag		486.9137	YES
Both	55.600	2.20	-0.60	2.80	1700	Crest	691.4365		YES
Both	55.900	-0.60	-0.62	0.02	400	Crest	INFINITE		YES
Both	56.200	-0.62	-1.00	0.38	700	Crest	INFINITE		YES
Both	56.500	-1.00	2.30	3.30	800	Sag		595.1168	YES
Both	57.200	2.30	-1.83	4.13	1800	Crest	1018.634		YES
Both	57.800	-1.83	0.80	2.63	500	Sag		334.2857	YES
Both	58.500	0.80	-1.61	2.41	1200	Crest	595.3762		YES
Both	58.700	-1.61	0.65	2.26	400	Sag		153.0562	YES
Both	59.300	0.65	2.75	2.10	400	Sag		52.85714	YES
Both	59.500	2.75	-1.00	3.75	1600	Crest	926.031		YES
Both	59.800	-1.00	2.00	3.00	500	Sag		475	YES
Both	60.200	2.00	-1.50	3.50	1500	Crest	864.2956		YES
Both	60.300	-1.50	2.50	4.00	800	Sag		721.3536	YES
Both	60.900	2.50	-3.50	6.00	2000	Crest	1481.65		YES
Both	61.100	-3.50	1.70	5.20	800	Sag		937.7597	NO
Both	61.400	1.70	3.00	1.30	2000	Sag		234.4399	YES
Both	61.700	3.00	1.50	1.50	800	Crest	370.4124		YES
Both	61.900	1.50	-2.00	3.50	1500	Crest	864.2956		YES
Both	62.300	-2.00	4.00	6.00	1200	Sag		1082.03	YES
Both	62.700	4.00	-1.00	5.00	2200	Crest	1234.708		YES
Both	63.300	-1.00	-3.50	2.50	1100	Crest	617.354		YES
Both	63.600	-3.50	2.25	5.75	1200	Sag		1036.946	YES
Both	64.100	2.25	-3.75	6.00	2600	Crest	1481.65		YES
Both	64.300	-3.75	-0.30	3.45	600	Sag		603.4783	NO
Both	64.500	-0.30	0.00	0.30	400	Sag		INFINITE	YES
Both	64.900	0.00	0.50	0.50	400	Sag		INFINITE	YES
Both	65.000	0.50	-0.34	0.84	400	Crest	INFINITE		YES
Both	65.300	-0.34	3.26	3.60	700	Sag		639.1667	YES
Both	65.500	3.26	0.50	2.76	1200	Crest	681.5589		YES
Both	66.100	0.50	-3.31	3.81	1600	Crest	941.5884		YES
Both	66.300	-3.31	0.30	3.61	800	Sag		651.5627	YES
Both	66.600	0.30	-0.60	0.90	400	Crest	INFINITE		YES
Both	66.800	-0.60	4.00	4.60	800	Sag		829.5567	NO
Both	67.500	4.00	-3.84	7.84	3500	Crest	1935.034		YES
Both	68.100	-3.84	1.50	5.34	1000	Sag		962.2858	YES
Both	68.400	1.50	-1.01	2.51	1100	Crest	619.0826		YES
Both	68.800	-1.01	0.50	1.51	400	Sag		INFINITE	YES
Both	69.000	0.50	-0.59	1.09	400	Crest	INFINITE		YES
Both	69.100	-0.59	0.50	1.09	400	Sag		INFINITE	YES
Both	69.300	0.50	-0.39	0.89	400	Crest	INFINITE		YES

Table 8. Mainline Vertical Curve Data (cont.)

Direction	Milepoint of VPI	Grade 1 (%)	Grade 2 (%)	A (%)	Length of Curve (ft)	Sag / Crest	Minimum L for SSD	Minimum L for HLSD	Meets AASHTO
Both	69.600	-0.36	0.00	0.36	400	Sag		INFINITE	YES
Both	70.200	0.00	-0.38	0.38	400	Crest	INFINITE		YES
Both	70.800	-0.38	0.70	1.08	400	Sag		INFINITE	YES
Both	71.000	0.70	-0.50	1.20	400	Crest	INFINITE		YES
Both	71.200	-0.50	0.50	1.00	400	Sag		INFINITE	YES
Both	71.500	0.50	0.25	0.25	400	Crest	INFINITE		YES
Both	71.700	0.25	0.50	0.25	400	Sag		INFINITE	YES
Both	72.100	0.50	2.16	1.66	400	Sag		INFINITE	YES
Both	72.300	2.16	-2.53	4.69	2150	Crest	1156.921		YES
Both	72.500	-2.53	0.00	2.53	1000	Sag		455.3545	YES
Both	72.700	0.00	0.80	0.80	400	Sag		INFINITE	YES
Both	72.800	0.80	0.50	0.30	400	Crest	INFINITE		YES
Both	72.900	0.50	1.22	0.72	400	Sag		INFINITE	YES
Both	73.000	1.22	0.50	0.72	400	Crest	INFINITE		YES
Both	73.700	0.50	0.95	0.45	400	Sag		INFINITE	YES
Both	74.300	0.95	2.43	1.48	400	Sag		INFINITE	YES
Both	74.700	2.43	-3.00	5.43	2300	Crest	1340.893		YES
Both	75.000	-3.00	-1.00	2.00	400	Sag		INFINITE	YES
Both	75.400	-1.00	-0.50	0.50	400	Sag		INFINITE	YES
Both	75.800	-0.50	0.50	1.00	400	Sag		INFINITE	YES
Both	76.300	0.50	3.00	2.50	400	Sag		278	YES
Both	76.700	3.00	-3.94	6.94	3100	Crest	1713.281		YES
Both	76.800	-3.94	0.00	3.94	900	Sag		710.1727	YES
Both	76.900	0.00	3.00	3.00	600	Sag		475	YES
Both	77.100	3.00	-3.94	6.94	3100	Crest	1713.775		YES

Vertical Curves

According to the *2018 Green Book,* vertical curves allow gradual changes between two tangent grades and may be either crest or sag types. They should be simple in application and result in a design that enables the driver to see the road ahead, enhances vehicle control, is pleasing in appearance, and is adequate for drainage.

Crest Vertical Curves – Stopping Sight Distance

A crest vertical curve is introduced when two vertical tangents come to a peak. According to the *2018 Green Book,* the major design control for crest vertical curves is the provision of ample stopping sight distance for the design speed. While research has shown that vertical curves with limited stopping sight distance do not necessarily experience frequent crashes, it is recommended that all vertical curves be designed to provide at least stopping sight distance adequate for the roadway's design speed. The stopping sight distance required for a 70 mph design speed is 730 feet.

Minimum lengths of crest vertical curves based on stopping sight distance criteria are generally satisfactory from the standpoint of safety, comfort, and appearance. There are exceptions, however, such as ramp exit gores, where longer sight distances should be provided.

Using vertical grade and length of vertical curve data obtained from the as-built plans, the following formulas were used to determine the minimum length of vertical curve needed to achieve adequate stopping sight distance given the algebraic difference in grades (A as a percent) and the stopping sight distance for a 70 mph design speed (S, 730 feet).

When S < L

$$L = \frac{AS}{2158}$$

When S > L

$$L = 2S - \frac{2158}{A}$$

When comparing the results of these calculations to the length of vertical curves on the as-built plans, all crest vertical curves on the WKP meet stopping sight distance design criteria for 70 mph.

Sag Vertical Curves – Headlight Sight Distance

A sag vertical curve is introduced when two vertical tangents make a valley. According to the *2018 Green Book*, the four different criteria used for establishing the length of sag vertical curves are headlight sight distance, passenger comfort, drainage control, and general appearance. Headlight sight distance has been used directly by some agencies and for the most part is the basis for determining the desirable length of sag vertical curves. When a vehicle traverses a sag vertical curve at night, the portion of highway lighted ahead is dependent on the position of headlights and the direction of the light beam. The vertical curve should be long enough that the light beam distance is approximately the same as stopping sight distance.

Using vertical grade and length of vertical curve data obtained from the as-built plans, the following formulas were used to determine the minimum length of vertical curve needed to achieve adequate headlight sight distance given the algebraic difference in grades (A, percent) and the stopping sight distance for a 70 mph design speed (S, 730 feet).

When S < L

$$L = \frac{AS^2}{400 + 3.5S}$$

When S > L

$$L = 2S - \frac{400 + 3.5S}{A}$$

When comparing the results of these calculations to the length of vertical curves on the as-built plans, four sag vertical curves on the WKP did not meet headlight sight distance design criteria for 70 mph and are as follows:

- MP 50.3 – Actual Length 800 feet, Length needed 812 feet, HLSD 721 feet
- MP 61.1 – Actual Length 800 feet, Length needed 938 feet, HLSD 635 feet
- MP 64.3 – Actual Length 600 feet, Length needed 603 feet, HLSD 726 feet
- MP 66.8 – Actual Length 800 feet, Length needed 830 feet, HLSD 707 feet

1.2 Bridges and Overpasses

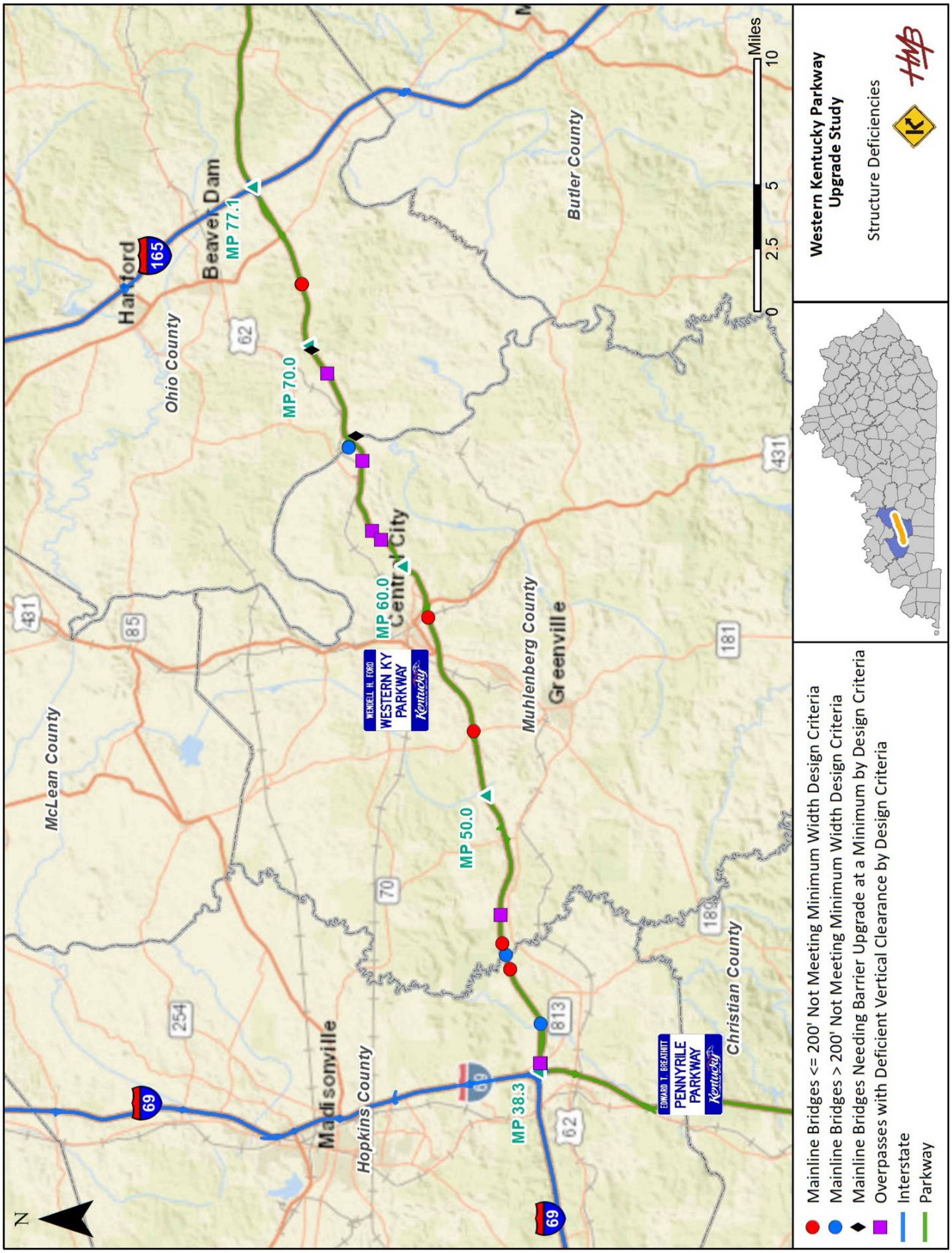
The structural and functional condition of each bridge on the WKP was compiled utilizing the KYTC Bridge Inspection Report and the National Bridge Inventory and Appraisal Report. The WKP has 28 mainline bridges, 12 overpass bridges, and four box culverts. Structures that carry WKP traffic over another roadway or terrain feature are considered mainline bridges, while structures that carry crossroad traffic over the WKP are considered overpass bridges.

According to *A Policy on Design Standards – Interstate System,* mainline bridges on the Interstate system and bridges on routes to be incorporated into the system may remain in place if, at a minimum, they meet all of the following criteria:

- For bridges less than or equal to 200 feet in length, the bridge cross section consists of at least 12-foot lanes, 10-foot shoulder on the right and 3.5-foot shoulder on the left,
- For longer bridges, shoulder width on both left and right is at least 3.5 feet measured from the edge of the nearest travel lane, and
- Bridge railing meets or will be upgraded to current standards.

Overpass bridges are evaluated by vertical clearance, which is the minimum vertical distance measured from any point on the roadway to the bottom of the overpass structure. According to *A Policy on Design Standards – Interstate System*, in rural areas, the vertical clearance to structures shall not be less than 16 feet over the entire roadway width, including auxiliary lanes and shoulders, as well as to ramps and collector-distributor roadways. The locations of all structures and identified deficiencies are shown on **Figure 4**.

Figure 4. Structure Deficiencies



Bridges Less Than or Equal to 200 Feet in Length

The minimum bridge width is 37.5 feet for a bridge less than or equal to 200 feet in length on the interstate system according to *A Policy on Design Standards – Interstate System*. That width includes two 12-foot travel lanes, a 10-foot outside shoulder, and a 3.5-foot inside shoulder. **Table 9** includes all bridges less than or equal to 200 feet in length along the WKP that do not meet this minimum width.

Table 9. Bridges ≤ 200 Feet that Do Not Meet Minimum Width

County	Milepoint	Bridge No.	Direction	Features Intersected	Length (Ft)	Curb to Curb Width (Ft)
Hopkins	42.800	054B00136L	WB	Pond River Relief	165	30
Hopkins	42.800	054B00136R	EB	Pond River Relief	165	30
Muhlenburg	43.600	089B00090L	WB	Pond River Relief	165	30
Muhlenburg	43.600	089B00090R	EB	Pond River Relief	165	30
Muhlenburg	52.500	089B00091L	WB	KY 181	179	32
Muhlenburg	57.600	089B00096L	WB	Railroad Crossing	169	32
Muhlenburg	57.600	089B00096R	EB	Railroad Crossing	169	32
Ohio	72.500	092B00133L	WB	KY 369	186	30
Ohio	72.500	092B00133R	EB	KY 369	186	30

Bridges Greater Than 200 Feet in Length

The minimum bridge width for bridges greater than 200 feet in length according to *A Policy on Design Standards – Interstate System* is 31.0 feet. That width includes two 12-foot travel lanes and a 3.5-foot shoulder on either side. **Table 10** includes all bridges longer than 200 feet in length along the WKP that do not meet this minimum width.

Table 10. Bridges > 200 Feet that Do Not Meet Minimum Width

County	Milepoint	Bridge No.	Direction	Features Intersected	Length (Ft)	Curb to Curb Width (Ft)
Hopkins	40.300	054B00146L	WB	Drakes Creek	420	30
Hopkins/Muhlenburg	43.400	054B00137R	EB	Pond River	205	30
Muhlenburg/Ohio	65.700	089B00093L	WB	Green River	1813	30
Muhlenburg/Ohio	65.700	089B00093R	EB	Green River	1813	30

Structures with Curbs

In addition to bridge width requirements, *A Policy on Design Standards – Interstate System* dictate that any existing mainline structure to remain in place must have railings / barriers that meets or will be upgraded to current interstate standards. As shown in **Table 11**, there are four bridges along the WKP that have railings / barriers that will need to be modified in order to meet interstate standards. The Green River bridges were constructed with brush block curb, while the Lewis Creek bridges were constructed with a railing on top of the barrier that will need to be removed.

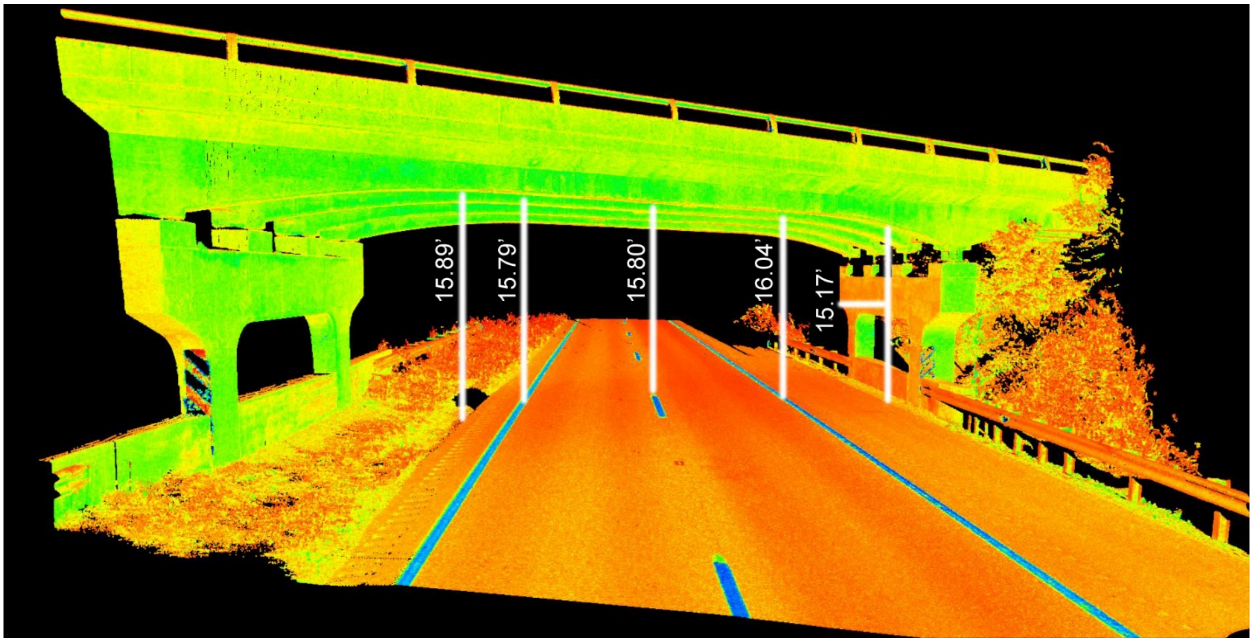
Table 11. Bridges without Crash Worthy Railing / Barrier

County	Milepoint	Bridge No.	Direction	Features Intersected	Length (Ft)	Curb to Curb Width (Ft)
Muhlenburg/Ohio	65.700	089B00093L	WB	Green River	1813	30
Muhlenburg/Ohio	65.700	089B00093R	EB	Green River	1813	30
Ohio	69.800	092B00134L	WB	Lewis Creek	120	38
Ohio	69.800	092B00134R	EB	Lewis Creek	120	38

Vertical Clearance of Overpasses

The minimum vertical clearance for a rural interstate according to *A Policy on Design Standards – Interstate System* should be at least 16 feet across the entire roadway including auxiliary lanes and shoulders. The vertical clearance measurements for the overpass structures on the WKP were obtained by a field survey crew using a terrestrial LiDAR scanner in order to produce the most accurate results for this study. After the field operations were complete, the LiDAR data was processed to produce measurements across the entire width of the roadway. An example of an overpass scan is shown in **Figure 5**.

Figure 5. Overpass Bridge Clearance Scan



There are nine locations where overpass structures along the WKP do not meet minimum vertical clearance, and they are listed in **Table 12**.

Table 12. Deficient Overpass Vertical Clearance

Milepoint	Direction	Crossroad	Inside Shoulder (ft)	Left EP (ft)	Center (ft)	Right EP (ft)	Outside Shoulder (ft)
38.700	EB	White Plains Rd	15.83	16.46	17.54	18.94	18.68
38.700	WB	White Plains Rd	15.65	16.06	15.99	15.65	15.47
44.980	EB	Henry Oats Rd	15.58	15.77	16.00	16.41	15.52
44.980	WB	Henry Oats Rd	15.89	15.79	15.80	16.04	15.17
61.390	WB	Monsanto Haul Rd	17.08	16.90	16.27	15.94	16.18
61.880	WB	Howerton Rd	16.30	16.24	16.03	15.87	15.85
64.860	WB	Rockport Paradise Rd	15.85	15.92	16.29	16.72	17.14
68.570	EB	Abandoned Rail Crossing	15.57	16.13	16.43	16.62	15.52
68.570	WB	Abandoned Rail Crossing	15.39	15.85	15.82	15.86	14.58

There are four locations where vertical clearance of overpass structures need to be considered when constructing any rehabilitation and resurfacing projects in these areas. In these locations, milling may be required before additional pavement can be added to the existing surface. They are listed in **Table 13**.

Table 13. Vertical Clearance Close to Minimum Interstate Standards

Milepoint	Direction	Crossroad	Inside Shoulder (ft)	Left EP (ft)	Center (ft)	Right EP (ft)	Outside Shoulder (ft)
57.180	EB	Stringtown Rd	16.36	16.35	16.51	16.74	17.00
61.390	EB	Monsanto Haul Rd	16.35	16.35	16.49	16.86	16.99
61.880	EB	Howerton Rd	16.12	16.09	16.24	16.41	16.62
64.860	EB	Rockport Paradise Rd	16.45	16.14	16.07	16.15	16.23

Crash Worthy Pier Protection

According to the *Roadside Design Guide* a vehicle impacting the untreated end of a roadside barrier can result in a vehicle being stopped abruptly, barrier elements penetrating the passenger compartment, or the vehicle rolling over. To be considered "crashworthy," an end treatment must not spear, vault, or roll a vehicle for head-on or angled impacts. Overpass bridge piers within the clear zone must have adequate protection so that they are not a hazard to oncoming traffic.

A limited field review was conducted and found that all overpass structures along the WKP have sufficient pier protection according to KYTC standards. More field measurements may be required to confirm the findings of this study.

Bridge Conditions

The structural and functional condition of each bridge was compiled utilizing the KYTC Bridge Inspection Report and the National Bridge Inventory and Appraisal Report. KYTC is responsible for assessing bridge conditions and determining where replacements and repairs are made. Bridge condition is determined by the lowest National Bridge Inventory (NBI) rating per KYTC Bridge Inspection Reports of the deck, superstructure, and substructure. Bridges are classified as Poor, Fair, or Good, and the correlation of the lowest NBI rating to these bridge conditions are as follows:

- Poor – NBI rating less than or equal to 4
- Fair – NBI rating of 5 or 6
- Good – NBI rating greater than or equal to 7

NBI ratings are provided for mainline bridges in **Table 14** and overpass bridges in **Table 15**.

According to the *Kentucky Transportation Cabinet Transportation Asset Management Plan* (KYTC TAMP) published in June 2019, the bridge life cycle planning strategy is a balanced approach of bridge preservation through regular maintenance and bridge replacement. The four different types of projects that KYTC utilizes as a part of its bridge management practice are as follows:

- Maintenance of bridges in “Fair” and “Good” condition
- Rehabilitation of bridges in “Fair” condition
- Major rehabilitation of bridges in “Poor” condition
- Bridge functional improvements at the network level

The *KYTC TAMP* outlines a method to calculate the remaining life of each bridge that can be used for asset management purposes. The estimated remaining life is based on an assumed life of 75 years for a new bridge and is determined using three bridge components: deck, superstructure, and substructure. Each bridge component is weighted and combined with the NBI rating per KYTC Bridge Inspection Reports to determine how much the bridge asset has depreciated. The bridge component weights are:

- Deck: 50% of total asset value
- Superstructure: 25% of total asset value
- Substructure: 25% of total asset value

The depreciated value for each component corresponding to its NBI rating can be found in **Figure 6**.

Table 14. Mainline Bridge Conditions Per KYTC Inspection Reports

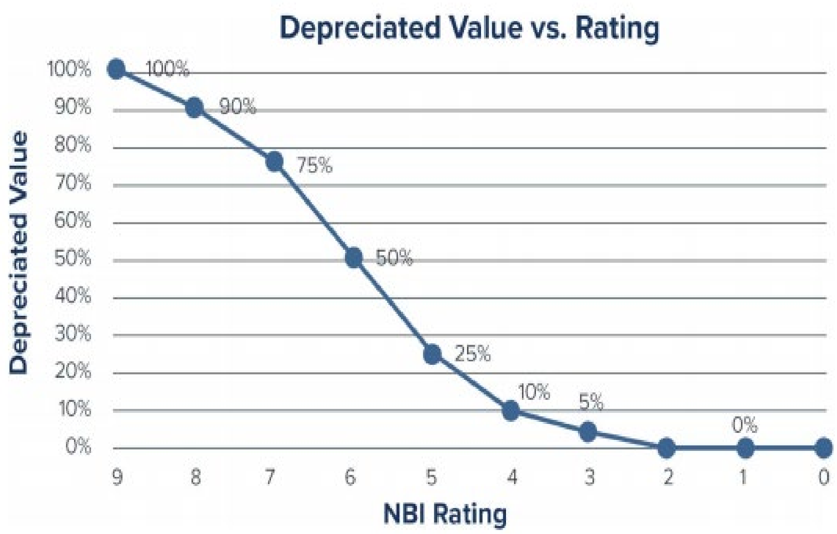
Bridge Crossing	Milepoint	Bridge ID	Sufficiency Rating	Deck Condition	Supstr. Condition	Substr. Condition	Bridge Condition	Remaining Life (Yrs)
Drakes Creek	40.300	054B00146L	71.6	6	6	5	Fair	**33**
Drakes Creek	40.300	054B00146R	81.4	6	6	5	Fair	**33**
Pond River Relief	42.800	054B00136L	70.3	5	6	5	Fair	**23**
Pond River Relief	42.800	054B00136R	71.3	6	6	5	Fair	**33**
Pond River	43.400	054B00137L	92.5	7	6	6	Fair	**47**
Pond River	43.400	054B00137R	82.4	7	6	6	Fair	**47**
Pond River Relief	43.600	089B00090L	82.0	7	7	6	Fair	**52**
Pond River Relief	43.600	089B00090R	70.0	7	7	6	Fair	**52**
KY 175 Interchange	48.100	089B00089L	93.0	7	6	6	Fair	**47**
KY 175 Interchange	48.100	089B00089R	82.0	7	7	6	Fair	**52**
KY 181 Interchange	52.500	089B00091L	81.0	6	6	6	Fair	**38**
KY 181 Interchange	52.500	089B00091R	98.0	6	7	6	Fair	**42**
Muhlenberg Co Rail Trail	55.600	089B00094L	93.0	7	6	6	Fair	**47**
Muhlenberg Co Rail Trail	55.600	089B00094R	89.0	7	6	6	Fair	**47**
US 62	56.000	089B00109L	100.0	7	7	7	Good	**56**
US 62	56.000	089B00109R	100.0	7	7	7	Good	**56**
Railroad Crossing	57.600	089B00096L	78.0	7	7	7	Good	**56**
Railroad Crossing	57.600	089B00096R	89.0	7	7	6	Fair	**52**
Cleaton Green River Road	59.200	089B00092L	84.4	7	7	6	Fair	**52**
Cleaton Green River Road	59.200	089B00092R	95.5	7	7	6	Fair	**52**
Green River	65.700	089B00093L	68.5	6	5	6	Fair	**33**
Green River	65.700	089B00093R	68.5	6	5	6	Fair	**33**
Lewis Creek	69.800	092B00134L	89.0	6	6	5	Fair	**33**
Lewis Creek	69.800	092B00134R	89.0	6	7	5	Fair	**38**
KY 369	72.500	092B00133L	71.0	5	6	5	Fair	**23**
KY 369	72.500	092B00133R	82.0	5	6	6	Fair	**28**
US 231 Interchange	74.600	092B00132L	New					
US 231 Interchange	74.600	092B00132R	New					

Table 15. Overpass Bridge Conditions Per KYTC Inspection Reports

Overpass Bridge Route	Milepoint	Bridge ID	Sufficiency Rating	Deck Condition	Supstr. Condition	Substr. Condition	Bridge Condition	Remaining Life (Yrs)
White Plains Rd	38.700	054B00131N	59.1	6	5	5	Fair	**28**
Henry Oats Rd	44.980	089B00085N	69.1	6	7	7	Fair	**47**
KY 601	50.370	089B00130N	90.1	8	8	8	Good	**68**
Stringtown Rd	57.180	089B00131N	91.7	8	7	7	Good	**62**
US 431	57.960	089B00132L	82.5	7	7	7	Good	**56**
US 431	57.960	089B00132R	93.6	7	7	7	Good	**56**
Youngstown Rd	58.330	089B00133N	93.4	8	8	7	Good	**65**
Monsanto Haul Rd*	61.390	089X00905N	97.0	6	6	7	Fair	**42**
Howerton Rd	61.880	089B00058N	83.5	6	7	7	Fair	**47**
Rockport Paradise Rd	64.860	089B00059N	80.3	6	6	7	Fair	**42**
KY 1245	67.360	092B00108N	78.6	5	6	6	Fair	**28**
Abandoned Rail Crossing	68.570	092B00135N	-2.0	N/A	4	6	Poor	**N/A**
KY 1245	68.700	092B00112N	80.6	5	7	7	Fair	**38**

*Last inspection in 2012

Figure 6. Depreciation versus NBI rating of Bridge Components



Source: KYTC Transportation Asset Management Plan Figure 3-20

The remaining life of each mainline and overpass bridge along the WKP was determined using this method provided by the *KYTC TAMP.* For example, the Green River bridges have a deck rating of 6, a superstructure rating of 5, and a substructure rating of 6. The following formula was used to arrive at an estimated remaining life of 33 years.

$$\begin{aligned}\boldsymbol{Remaining\ Life} &= \boldsymbol{75\ years} \times ((\boldsymbol{0.50} \times \boldsymbol{Deck\ Dep.Value}) \\ &+ (\boldsymbol{0.25} \times \boldsymbol{Superstruture\ Dep.Value}) \\ &+ (\boldsymbol{0.25} \times \boldsymbol{Substructure\ Dep.Value}))\end{aligned}$$

The resulting remaining life for each bridge can be found in **Table 14** for mainline bridges and **Table 15** for overpass bridges. The condition and estimated remaining life of each bridge along the WKP can be used as a tool in developing future asset management plans specific to this corridor or statewide.

Overhead Signs

According to the *2009 MUTCD,* overhead signs shall provide a vertical clearance of not less than 17 feet to the sign, light fixture, or sign bridge over the entire width of the pavement and shoulders except where the structure on which overhead signs are to be mounted or other structures along the roadway near the sign structure have lesser vertical clearance. If the vertical clearance of other structures along the roadway near the sign structure is less than 16 feet, the vertical clearance to an overhead sign structure or support may be as low as 1-foot higher than the vertical clearance of the other structures in order to improve the visibility of the overhead signs.

The vertical clearances of overhead signs were measured using the same methods as the overpass structures along the WKP. An example of an overhead sign scan is pictured in **Figure 7**. One bridge mounted sign at Exit 58 over the westbound lanes of the WKP has a vertical clearance of less than 17 feet. All overhead sign vertical clearance measurements can be found in **Table 16**.

Figure 7. Overhead Sign Clearance Scan

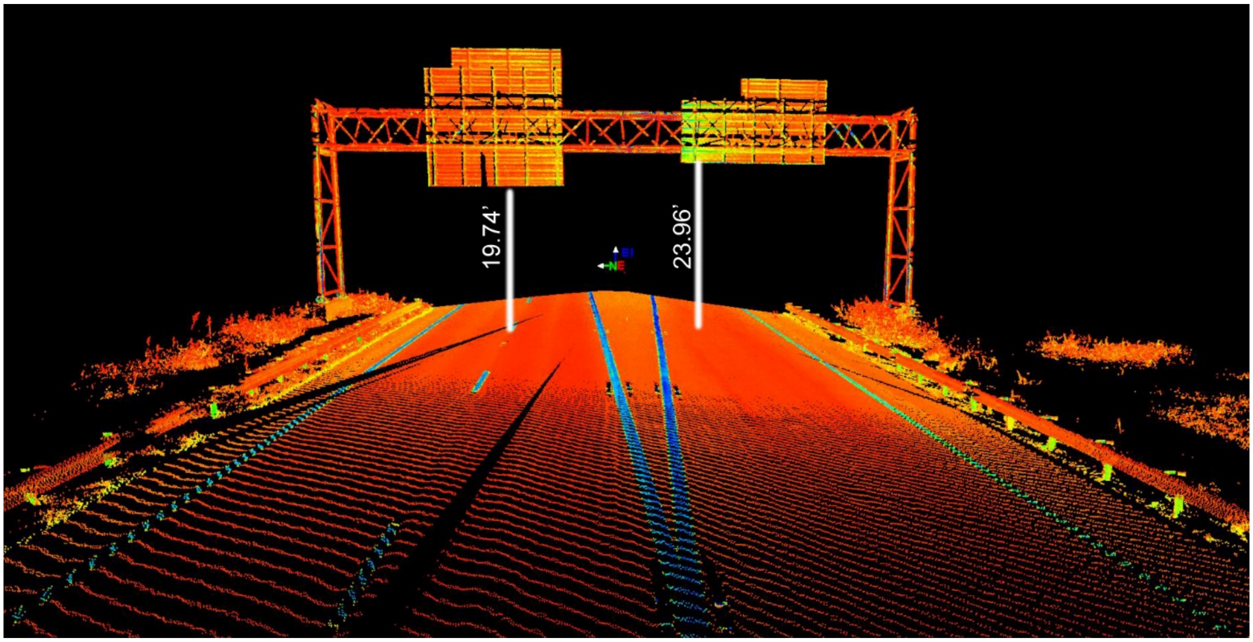


Table 16. Overhead Sign Vertical Clearance

County	Direction	Milepoint	Description	Type	Vertical Clearance (ft)
Hopkins	EB	38.620	Exits 38B-A I-69 S Hopkinsville/Fulton, Exit 38C I-69 N Henderson	Overhead Truss	19.74
Muhlenburg	EB	58.000	Exit 58 Drakesboro/Central City	Bridge Mounted Sign	17.89
Muhlenburg	WB	58.000	Exit 58 Central City/Drakesboro	Bridge Mounted Sign	16.83
Ohio	EB	76.480	Exit 77A I-165 S Bowling Green,Exit 77B I-165 N Owensboro	Overhead Truss	21.12
Ohio	WB	76.770	Exit 77 A I-165 S Bowling Green	Cantilever	18.45

1.3 Interchanges and Ramps

Similar to the mainline geometry guidelines, the *2018 Green Book* provides recommended design criteria for interchanges and ramps that includes: design speed, typical section, and horizontal and vertical alignment. This chapter addresses those design criteria compared to the existing conditions of the WKP as well as speed change lanes, weaving characteristics, interchange configuration, interchange spacing, and crossroad control of access. **Table 17** summarizes the interchanges that are present along the WKP. The existing characteristics of these interchanges were obtained through as-built plans if they were available, a limited field review, and statewide LiDAR data when necessary.

Table 17. WKP Interchange Summary

County	Exit No.	Milepoint	Intersecting Route	Lighted (Yes or No)
Hopkins	N/A	39.670	KSP Post	No
Muhlenburg	48	48.049	KY 175	Yes
Muhlenburg	53	52.518	KY 181	Yes
Muhlenburg	58	57.959	US 431	Yes
Ohio	75	74.583	US 231	Yes
Ohio	N/A	75.650	Beaver Dam (Huck's) Rest Stop	Yes

Design Speed

In Section 10.9.6 – Ramps, Table 10-1 of the *2018 Green Book*, values for ramp design speed as related to highway design speed are provided. They apply to the sharpest, or controlling ramp curve, usually on the ramp proper. These speeds do not pertain to the ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway design speed involved.

From that table the ramp design speeds for a 70 mph highway design speed range from 35 mph to 60 mph. Upper range values of design speed generally are not attainable on loop ramps. For highway design speeds above 50 mph, the loop design speed should be no less than 20 mph. If less restrictive conditions exist, the loop design speed and the radius may be increased.

All ramps on the WKP meet the minimum criteria for design speed with the exception of the westbound on-ramp at Exit 58. Reconstructing this interchange was investigated as part of this study and presented in **Section 8.3** in the main body of the report. Some ramps, however, have deficient acceleration and deceleration lengths, which is discussed later in this section.

Typical Section

Just as AASHTO has minimum guidelines for mainline travel lane and shoulder widths, it also provides similar guidelines for a ramp typical section. The existing conditions of the ramps along the WKP were compared to those guidelines found in the *2018 Green Book.*

According to KYTC's *Highway Design Manual,* single lane ramps shall have a minimum pavement width of 15 feet, with a 6-foot usable right shoulder and a 4-foot usable left shoulder. The *2018 Green* Book states that the combined paved shoulder width of a ramp with one-way traffic operation should be 10 to 14 feet, with shoulders ranging from 6 to 10 feet on the outside and 2 to 4 feet on the inside. As shown in **Table 18**, all of the WKP ramps meet these guidelines/criteria for minimum widths with the exception of the ramps at Huck's. **Figure 8** illustrates a typical section consistent with the other five interchanges throughout this corridor.

Table 18. Interchange Typical Sections

County	Exit No.	Milepoint	Intersecting Route	Lane Width	Outside Shoulder Width	Inside Shoulder Width
Hopkins	N/A	39.670	KSP Post	15	6	4
Muhlenburg	48	48.049	KY 175	15	6	4
Muhlenburg	53	52.518	KY 181	15	6	4
Muhlenburg	58	57.959	US 431	15	6	4
Ohio	75	74.583	US 231	15	6	4
Ohio	N/A	75.650	Beaver Dam (Huck's) Rest Stop	18	4	2

Figure 8. KY 175 Interchange Ramp Typical Section

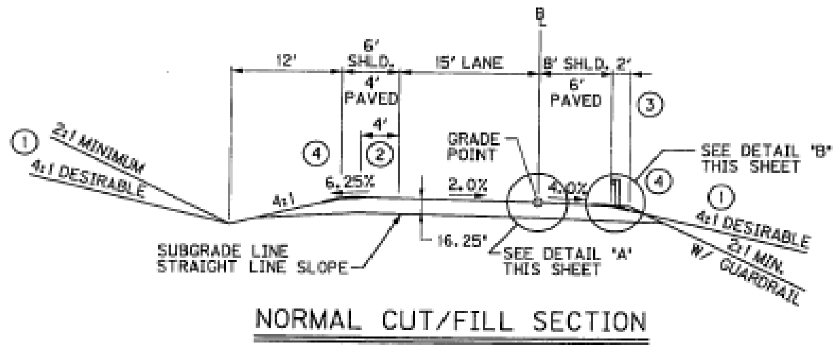


Table 3-27 within Section 10.9.6 – Ramps of the *2018 Green Book* provides recommended design widths of ramp traveled ways given various traffic conditions. Based on truck volumes it was assumed that ramps along the WKP should be designed for Traffic Condition B – sufficient single unit vehicles to govern design, but some consideration for semitrailer vehicles. Ramp travel lane widths for Case I (One-lane, one-way operation with no provision for passing and stalled vehicles) and Traffic Condition B can be reduced to 14 feet if the radii of the ramp are greater than 600 feet. All ramps at Huck's have a radius greater than 600 feet. The existing pavement is wide enough to accommodate a 14-foot travel lane, 6-foot usable right shoulder, and 4-foot usable left shoulder. These issues on the ramps can be addressed through restriping. Additional field work may need to be done to verify these findings.

Horizontal Alignment

The horizontal alignment data for all interchange ramps along the WKP were reviewed through as-built plans, a limited field review, and statewide LiDAR data. Each horizontal curve's radius and the superelevation of the ramps through each curve determine its design speed. It is currently KYTC common practice to use the $e_{max} = 8\%$ superelevation table found in the *2018 Green Book* when determining the design speed of existing curves. All ramps throughout this corridor meet minimum design speed when using this design criteria, and; therefore, meet criteria for minimum horizontal radius, with the exception of the westbound on-ramp of the interchange between the WKP and US 431 (Exit 58).

The minimum design speed criteria for loop ramps according to the *2018 Green Book* is 25 mph. Since the superelevation data for Exit 58 was not available on as-built plans, it was measured based on statewide LiDAR data. With a measured superelevation rate of 4%, the westbound on-ramp's current design speed is less than 15 mph, however, it would meet minimum criteria for design speed if its minimum horizontal radius were coupled with a superelevation rate of 8%. Reconfiguring the WKP and US 431 interchange to a traditional diamond was investigated as a part of this study. Additional field review may be required to verify the findings of this study.

Superelevation Rate

From the review of the as-built plans and, in some instances, using statewide LiDAR data, it appears that the interchanges along the WKP were initially designed based on $e_{max} = 10\%$ table found in the *2018 Green Book.* Since it is currently KYTC common practice to use $e_{max} = 8\%$ table, the existing design speeds of the ramps along the WKP were determined using that assumption. All ramp superelevation data can be found in **Table 19**.

Cross Slope

According to the *2018 Green Book*, the cross slope on portions of ramps on tangent should be sloped in one direction at a practical rate ranging from 1.5% to 2% for high type pavements. From the review of the as-built plans, all ramps along the WKP meet this minimum criterion.

Divergence Angle

According to the *2018 Green Book,* the taper type exit terminal provides a clear indication of the point of departure from the through lane, and the angle between the mainline traveled way and the traveled way of the exiting ramp is called the divergence angle. The divergence angle is usually between 2 and 5 degrees.

The divergence angle for exiting ramps along the WKP was estimated through the use of statewide aerial imagery. All ramps meet design criteria for divergence angle. This information can be found in **Table 19**.

Table 19. Horizontal Alignment - Interchanges

Ramp	Entering/Exiting Curve Radius	Superelevation Rate	Design Speed AASHTO 8% Table	Curve Radius	Superelevation Rate	Design Speed AASHTO 8% Table	Divergence Angle
KSP Post No. 2							
EB Off Ramp	N/A	N/A	N/A	N/A	N/A	N/A	2
EB On Ramp	N/A	N/A	N/A	N/A	N/A	N/A	N/A
WB Off Ramp	N/A	N/A	N/A	N/A	N/A	N/A	4.5
WB On Ramp	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Exit 48 - KY 175							
EB Off Ramp	850.00	6%	35	N/A	N/A	N/A	4
EB On Ramp	850.00	6%	35	N/A	N/A	N/A	N/A
WB Off Ramp	850.00	6%	35	N/A	N/A	N/A	5
WB On Ramp	850.00	6%	35	N/A	N/A	N/A	N/A
Exit 53 - KY 181							
EB Off Ramp	716.20	8%	45	396.00	8%	35	4
EB On Ramp	325.00	8%	35	N/A	N/A	N/A	N/A
WB Off Ramp	230.00	8%	30	N/A	N/A	N/A	3.5
WB On Ramp	716.20	8%	45	230.00	8%	30	N/A
Exit 58 - US 431							
EB Off Ramp	1432.40	4%**	35	180.00	10%**	25	Weave
EB On Ramp	230.00	6%**	20	N/A	N/A	N/A	Weave
WB Off Ramp	1298.80	4%**	30	230.00	10%**	30	Weave
WB On Ramp	135.00	4%**	N/A	230.00	10%**	30	Weave
Exit 75 - US 231							
EB Off Ramp	954.93	10%	50	716.20	10%	50	4
EB On Ramp	954.93	10%	50	N/A	N/A	N/A	N/A
WB Off Ramp	954.93	10%	50	716.20	10%	50	4
WB On Ramp	954.93	10%	50	N/A	N/A	N/A	N/A
Huck's							
EB Off Ramp	3000*	4%**	50	N/A	N/A	N/A	4
EB On Ramp	1200*	7%**	50	N/A	N/A	N/A	N/A
WB Off Ramp	1600*	7%**	55	N/A	N/A	N/A	2
WB On Ramp	1200*	8%**	60	N/A	N/A	N/A	N/A

*Radius estimated using statewide aerial imagery

**Superelevation not available on as-built plans; derived from statewide LiDAR data

Vertical Alignment

Vertical Grade

According to the *2018 Green Book*, ramp grades should be as flat as practical to minimize the driving effort from one road to another, but for any one ramp, the gradient to be used is dependent on a number of factors unique to that site and quadrant. In general, adequate sight distance is more important than a specific gradient control and should be favored in design. The guidelines for maximum ramp grade based on design speed are as follows:

- 45 mph or greater – 3% to 5%
- 35-40 mph – 4% to 6%
- 25-30 mph – 5% to 7%
- 15-20 mph – 6% to 8%

However, with proper ramp terminal facilities, short upgrades of 7% to 8% permit good operation without unduly slowing passenger cars. Desirably, downgrades should be limited to 3% to 4% on-ramps with sharp horizontal curvature and significant heavy truck traffic, but gradients of up to 8% do not cause undesirable operation due to excessive acceleration of passenger vehicles. All ramps throughout this corridor meet the guidelines for maximum vertical grade outlined in the *2018 Green Book.*

Vertical Curves

According to the *2018 Green Book,* vertical curves allow gradual changes between two tangent grades and may be either crest or sag types. They should be simple in application and result in a design that enables the driver to see the road ahead, enhances vehicle control, is pleasing in appearance, and is adequate for drainage. Just as on mainline, stopping sight distance is used to determine the desirable length of a crest vertical curve, while headlight sight distance is used to determine the desirable length of a sag vertical curve.

The vertical alignment data for each ramp was obtained from the as-built plans when available, but at the KSP Post No. 2, Exit 58, Exit 75, and Huck's, statewide LiDAR data was used to approximate existing vertical grades and vertical curve lengths on each ramp. Since there were no entering or exiting curves for of the ramps at KSP Post No. 2, a design speed of 50 mph, 70% of the posted speed limit on the WKP, was assumed to analyze the vertical alignment data at that interchange.

Of all ramps within the study area, only two vertical curves did not meet sight distance requirements. Both the eastbound and westbound on-ramps at Exit 75 have sag vertical curves near the approach that do not meet headlight sight distance requirements for a 50 mph design speed. However, according to *Mitigation Strategies for Design Exceptions* published by the FHWA in 2007, if lighting is provided at sag vertical curves, a design to the driver comfort criteria may be adequate. Head light sight distance is mitigated as all sag vertical curves have adequate stopping sight distance under lit conditions. The interchange between the WKP and US 231 (Exit 75) has high mast lighting. It is also likely that vehicles will not be traveling at high speeds through these vertical curves since they occur just as vehicles are entering the ramp near the approach. The vertical alignment data for interchanges on the WKP can be found in **Table 20**.

Speed Change Lanes

According to the *2018 Green Book*, drivers leaving a highway at an interchange are required to reduce speed as they exit onto a ramp. Drivers entering a highway from a turning roadway accelerate until the desired highway speed is reached. Because the change in speed is usually substantial, provision should be made for acceleration and deceleration to be accomplished on auxiliary lanes to minimize interference with through traffic on the interstate and to reduce crash potential.

The two general forms of speed change lanes are taper and parallel. The taper type provides a direct entry or exit at a flat angle, while the parallel type has an added lane for changing speed. Regardless of the type of speed change lane, it must be of adequate length for the vehicles entering and exiting the highway to accelerate and decelerate.

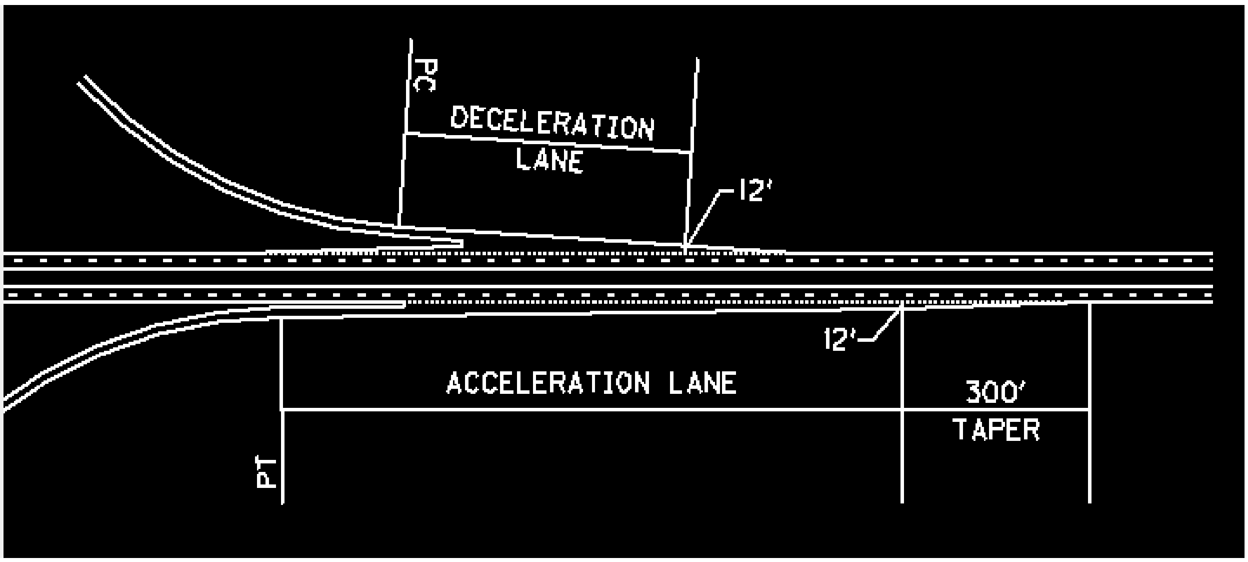
Table 20. Vertical Alignment - Interchanges

Ramp	Grade 1	Vertical Curve	Sight Distance	Grade 2	Vertical Curve	Sight Distance	Grade 3	Ramp Design Speed	Sight Distance Needed
KSP Post No. 2									
EB Off Ramp	-2.00	N/A	N/A	N/A	N/A	N/A	N/A	50	425
EB On Ramp	0.50	200	639.58	-1.50	N/A	N/A	N/A		
WB Off Ramp	0.50	150	440.75	2.00	N/A	N/A	N/A		
WB On Ramp	1.00	250	2283.30	0.50	N/A	N/A	N/A		
Exit 48 - KY 175									
EB Off Ramp	-1.09	600	847.79	-3.06	200	562.96	-0.50	35	250
EB On Ramp	0.51	500	1989.80	1.44	165	1742.73	0.79		
WB Off Ramp	0.57	400	1492.29	1.58	200	2163.13	1.05		
WB On Ramp	-1.10	320	1989.80	-3.00	375	409.40	1.13		
Exit 53 - KY 181									
EB Off Ramp	-1.27	250	647.45	0.32	700	981.08	-1.39	35	250
EB On Ramp	-0.81	700	884.54	2.46	300	621.24	0.17		
WB Off Ramp	-0.90	450	925.75	-2.44	400	347.18	2.92	30	200
WB On Ramp	-1.42	300	514.17	1.6	250	1923.58	1.00		
Exit 58 - US 431									
EB Off Ramp	0.42	250	349.97	4.04	250	696.77	2.15	25	155
EB On Ramp	-2.02	150	568.75	-4.21	350	277.84	2.02	20	115
WB Off Ramp	-0.96	150	1068.98	-2.04	300	386.47	1.66	30	200
WB On Ramp	1.42	200	1365.29	0.63	N/A	N/A	N/A		
Exit 75 - US 231									
EB Off Ramp	1.48	500	745.22	-0.70	200	503.88	1.00	50	425
EB On Ramp	-0.85	200	315.84	2.64	800	578.10	-2.53		
WB Off Ramp	2.55	500	498.61	-1.79	200	799.80	0.50		
WB On Ramp	-0.94	250	361.98	2.58	500	485.83	-1.99		
Huck's									
EB Off Ramp	-1.21	200	999.44	-0.43	N/A	N/A	N/A	50	425
EB On Ramp	-0.77	250	802.58	-2.36	350	546.09	0.76		
WB Off Ramp	-0.31	350	3121.10	2.24	350	734.49	0.31	55	495
WB On Ramp	0.22	350	1075.80	1.48	N/A	N/A	N/A	60	570

Acceleration Lengths

The speed change lanes entering a highway must be long enough to allow a vehicle to accelerate to the speed of the roadway it is entering prior to merging. This length is measured from point of tangency (PT) of the ramp's entering curve and the point where the width between the normal mainline traveled way and the edge of traveled way for the speed change lane is equal to 12 feet. **Figure 9** further illustrates how these lengths are measured.

Figure 9. Acceleration and Deceleration Lane Detail



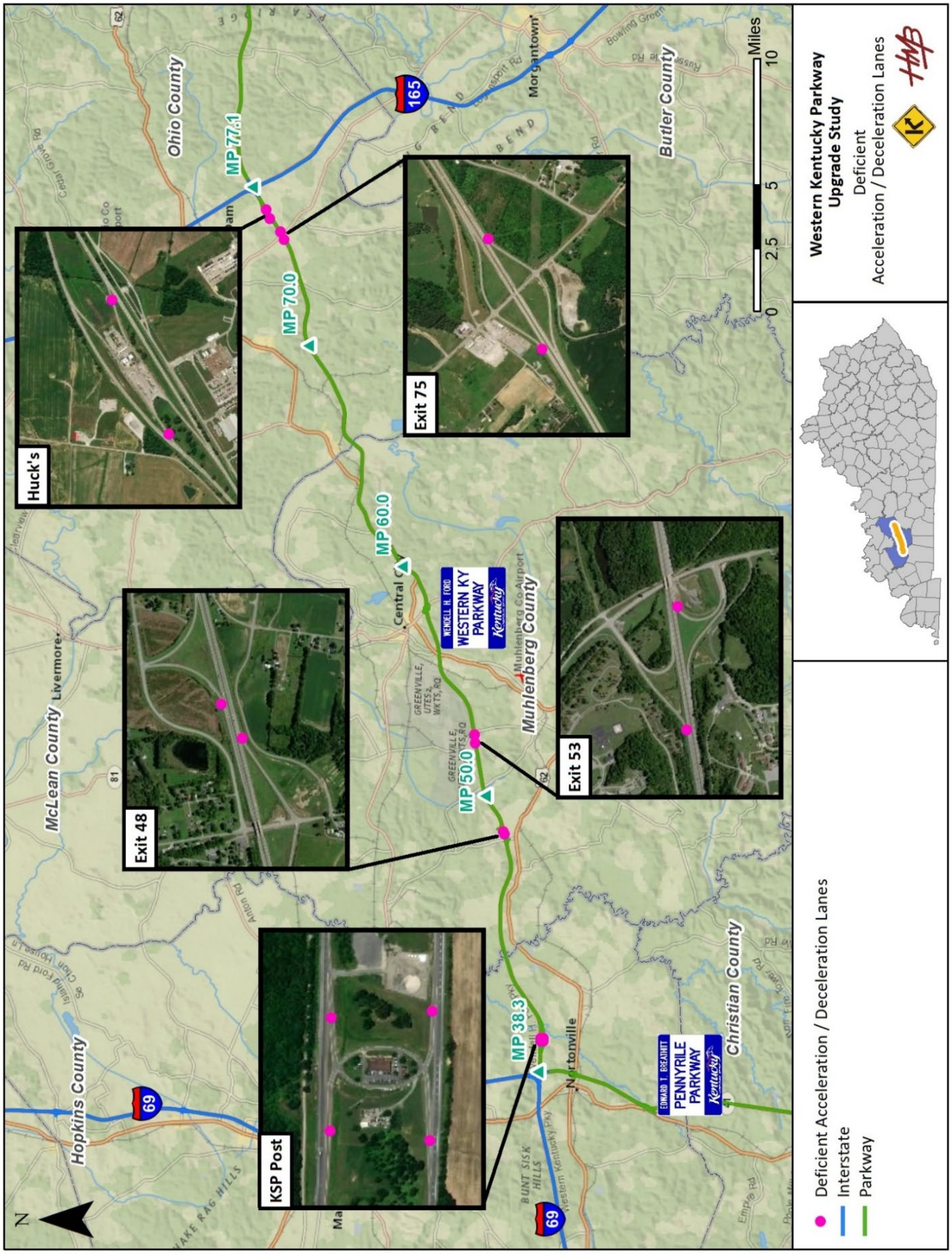
The length of a speed change lane entering a highway is governed by the design speed of the entering curve, located on the ramp proper, and the design speed of the facility being entered. The acceleration lengths for interchanges along the WKP were compared to Table 10-4 in the *2018 Green book*, which provides minimum acceleration lengths given the design speed of the ramp's entering curve and the design speed of the highway being entered.

The existing acceleration lengths provided along the WKP were determined using limited field review and statewide aerial imagery. Additional field review may be required to verify the information in this study. Ramps that do not meet minimum requirements for acceleration lengths can be found in **Table 21** and illustrated on **Figure 10**.

Table 21. Interchange Speed Change Lanes

Interchange	Curve Radius (ft)	Superelevation (%)	Curve Design Speed Used (mph)	Divergence Angle (°)	Accel/Decel Lane Grade (%)	Measured Length (ft)	AASHTO Minimum Length (ft)	Difference (ft)	Meets AASHTO Criteria	KYTC Common Practice Length	Difference	Meets KYTC Common Practices
KSP Post No. 2 - MP 39.7												
EB off ramp	266	2	Stop Condition		1.9%	592	615	-23	No	561.2	-	-
EB on ramp	209	4	15		1.4%	1362	1560	-198	No	1000	-	-
WB off Ramp	266	2	Stop Condition	4	0.5%	476	615	-139	No	561.2	-	-
WB on Ramp	209	4	15		0.8%	743	1560	-817	No	1000	-	-
Exit 48 - KY 175/KY 2693												
EB off ramp	850	6	35	4.1	1.0%	551	490	61	Yes	561.2	-	-
EB on ramp	850	6	35		0.4%	919	1230	-311	No	1000	-	-
WB off ramp	850	6	35	5	0.5%	555	490	65	Yes	561.2	-	-
WB on ramp	850	6	35		0.8%	903	1230	-327	No	1000	-	-
Exit 53 - KY 181												
EB off ramp	716	8	45	4.2	1.3%	504	390	114	Yes	561.2	-	-
EB on ramp	325	8	35		0.4%	980	1230	-250	No	1000	-	-
WB off ramp	230	8	30	4.2	2.0%	563	520	43	Yes	561.2	-	-
WB on ramp	716	8	45		1.2%	687	820	-133	No	1000	-	-
Exit 58 - US 431												
EB off ramp	1116	8	55		0.8%	598	340	258	Yes	561.2	36.8	Yes
EB on ramp	230	8	30		0.8%	393	1350	-957	No	1000	-	-
WB off ramp	1432	8	60		0.8%	484	340	144	Yes	561.2	-77.2	No
WB on ramp	230	8	30		0.8%	435	1350	-915	No	1000	-	-
Exit 75 - US 231												
EB off ramp	955	8	50	5.4	1.7%	438	< 340	98	Yes	561.2	-123.2	No
EB on ramp	955	8	50		1.9%	373	< 580	-207	No	1000	-627	No
WB off ramp	955	8	50	4	2.1%	441	< 340	101	Yes	561.2	-120.2	No
WB on ramp	955	8	50		1.9%	479	< 580	-101	No	1000	-521	No
Huck's - MP 75.7												
EB off ramp	2355	8	50	4.3	1.5%	353	<340	13	Yes	561.2	-208.2	No
EB on ramp	725	8	50		1.3%	329	820	-491	No	1000	-671	-
WB off ramp	1272	8	55		0.2%	740	< 340	0	Yes	561.2	-221.2	No
WB on ramp	760	8	60		1.6%	478	580	-102	No	1000	-522	No

Figure 10. Deficient Acceleration / Deceleration Lanes



Deceleration Lengths

The speed change lanes exiting a highway must be long enough to allow a vehicle, most likely traveling at a relatively high speed, to decelerate to a speed that will allow them to travel safely through the exiting curve located on the ramp proper. As illustrated in **Figure 9**, this length is measured from the point where the width between the normal mainline traveled way and the edge of traveled way for the speed change lane is equal to 12 feet and the point of curvature (PC) of the ramp's exiting curve.

The length of a speed change lane exiting a highway is governed by the design speed of the highway and the design speed of the exiting curve of the ramp. The deceleration lengths at interchanges along the WKP were compared to Table 10-6 in the *2018 Green Book*, which provides minimum deceleration lengths given the design speed of the highway being exited and the design speed of the ramp's exiting curve.

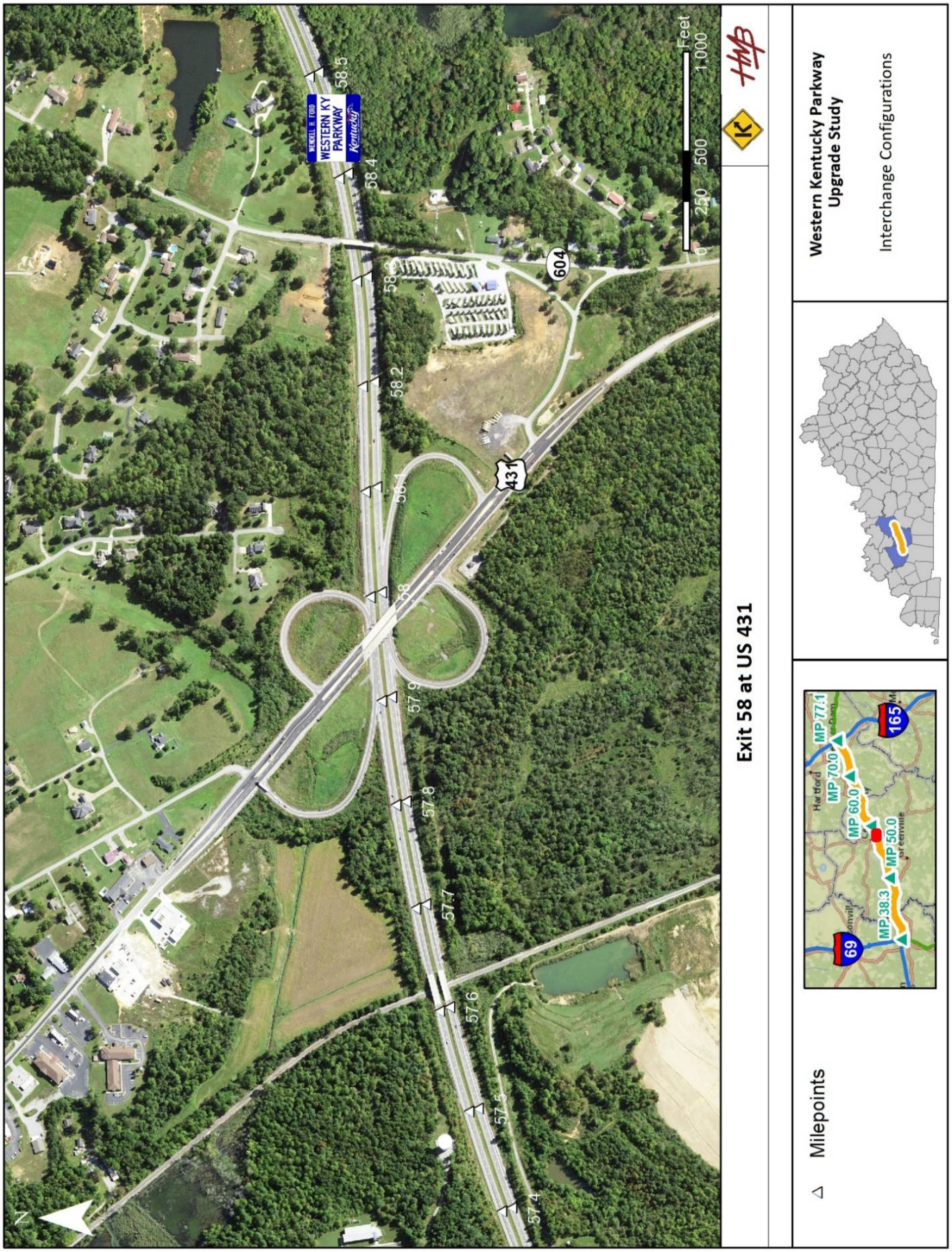
The existing deceleration lengths provided along the WKP were determined using limited field review and statewide aerial imagery. Additional field review may be required to verify the information in this study. Ramps that do not meet minimum requirements for deceleration lengths can be found in **Table 21** and illustrated in **Figure 10**.

Weaving Characteristics

According to Figure 10-70: Recommended Ramp Terminal Spacing in the *2018 Green Book*, the minimum distance required from a service interchange entrance terminal to a service interchange exit terminal is 1000 feet. The WKP / US 431 interchange at Exit 58 **(Figure 11)** has a distance of 430 feet between the eastbound entrance and exit ramp terminals and a distance of 390 feet between the westbound entrance and exit ramp terminals, and; therefore, does not meet minimum design criteria. When this interchange was constructed motorists exiting the roadway stopped to pay a toll. It has been standard practice for KYTC to reconstruct these interchanges. A traditional diamond interchange has been investigated as a part of this study.

When evaluating the functionality of this interchange the design year (2045) traffic volumes and existing weave distance resulted in a LOS A. However, according to the crash analysis presented in Chapter 6 of the main body of the report, a high volume of crashes have occurred between vehicles entering and exiting the WKP at this interchange.

Figure 11. Exit 58 at US 431 Interchange Configuration



Interchange Configuration

Five interchanges along the WKP are not consistent with common practice for interstate interchange configuration. Each are described in the following sections.

Kentucky State Police (KSP) Post No. 2 – Service Interchange

As shown on **Figure 12**, the ramps at KSP Post No. 2 enter and exit the WKP from the left. The eastbound and westbound lanes are bifurcated, and KSP Post No. 2 as well as a KYTC Maintenance Facility are located in the median. There is an entrance for the maintenance facility located on the westbound off-ramp and an exit located on the eastbound on-ramp. According to the crash analysis that is a part of this study, there have been no reported crashes at these locations attributed to the placement of the entrance and exit based on reviewing detailed crash reports. However, improvement concepts have been explored as a part of this study to mitigate any potential issues.

KY 175 – Service Interchange

As shown on **Figure 13**, the KY 175 interchange consists of two ramps serving the eastbound lanes in a traditional diamond configuration with terminals on KY 175 and two ramps serving the westbound lanes with terminals on KY 2693. This split interchange configuration is likely due to right-of-way constraints north of the WKP on KY 175.

KY 181 – Service Interchange

The KY 181 interchange is configured as a folded diamond. Instead of the eastbound on-ramp and westbound off-ramp being located in the southeast and northeast quadrants, they are instead located in the southwest and northwest quadrants as loop ramps. The eastbound off-ramp and westbound on-ramp are located in the same quadrants. The interchange is illustrated on **Figure 14**.

US 431 – Service Interchange

As shown previously in **Figure 11**, the US 431 interchange is made up of loop ramps in all four quadrants. The entrance and exit ramps are connected on the WKP with an auxiliary lane to form a short weaving section parallel to both the eastbound and westbound lanes. As discussed previously, when this interchange was constructed, it housed a toll booth for both directions, but the toll booth has since been removed. It has been standard practice for KYTC to reconstruct these interchanges. A potential reconfiguration of this interchange to a traditional diamond has been investigated as a part of this study.

Beaver Dam Rest Area (Huck’s) – Service Interchange

As illustrated on **Figure 15**, Huck’s is located in the median of the WKP where the eastbound and westbound lanes are bifurcated. The ramps at Huck’s enter and exit the WKP from the left.

Figure 12. KSP Post No. 2 Interchange Configuration

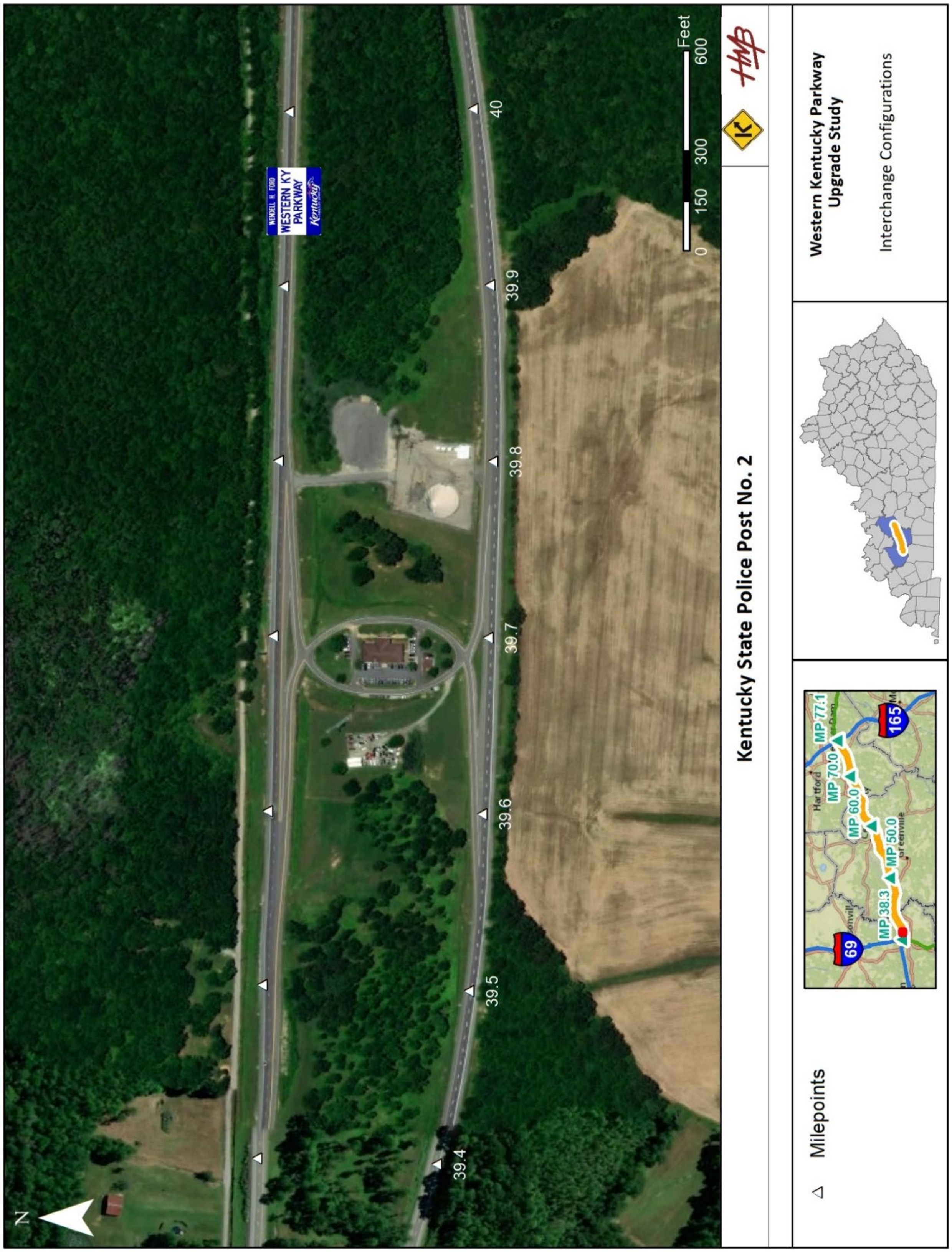


Figure 13. Exit 48 at KY 175 Interchange Configuration

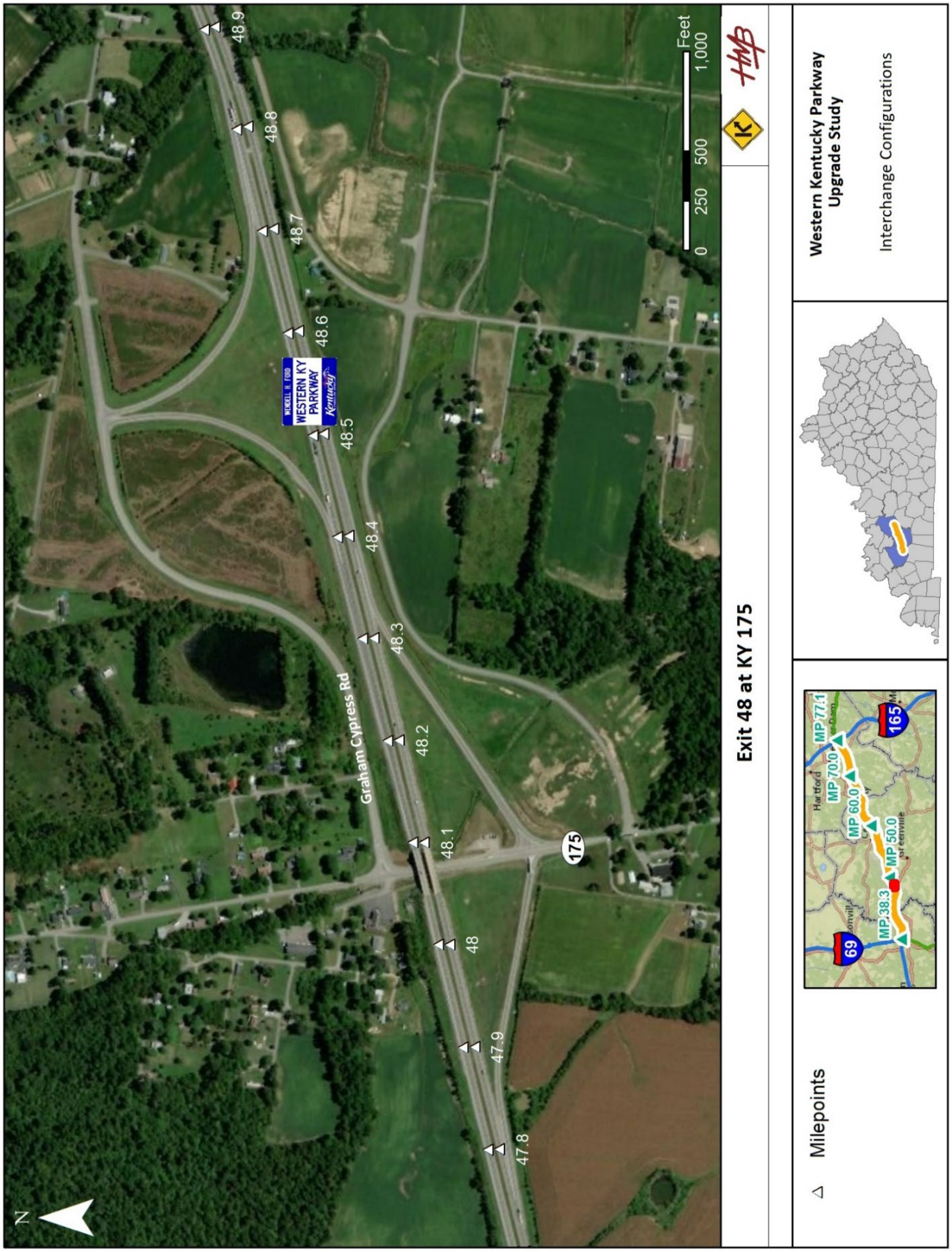


Figure 14. Exit 53 at KY 181 Interchange Configuration

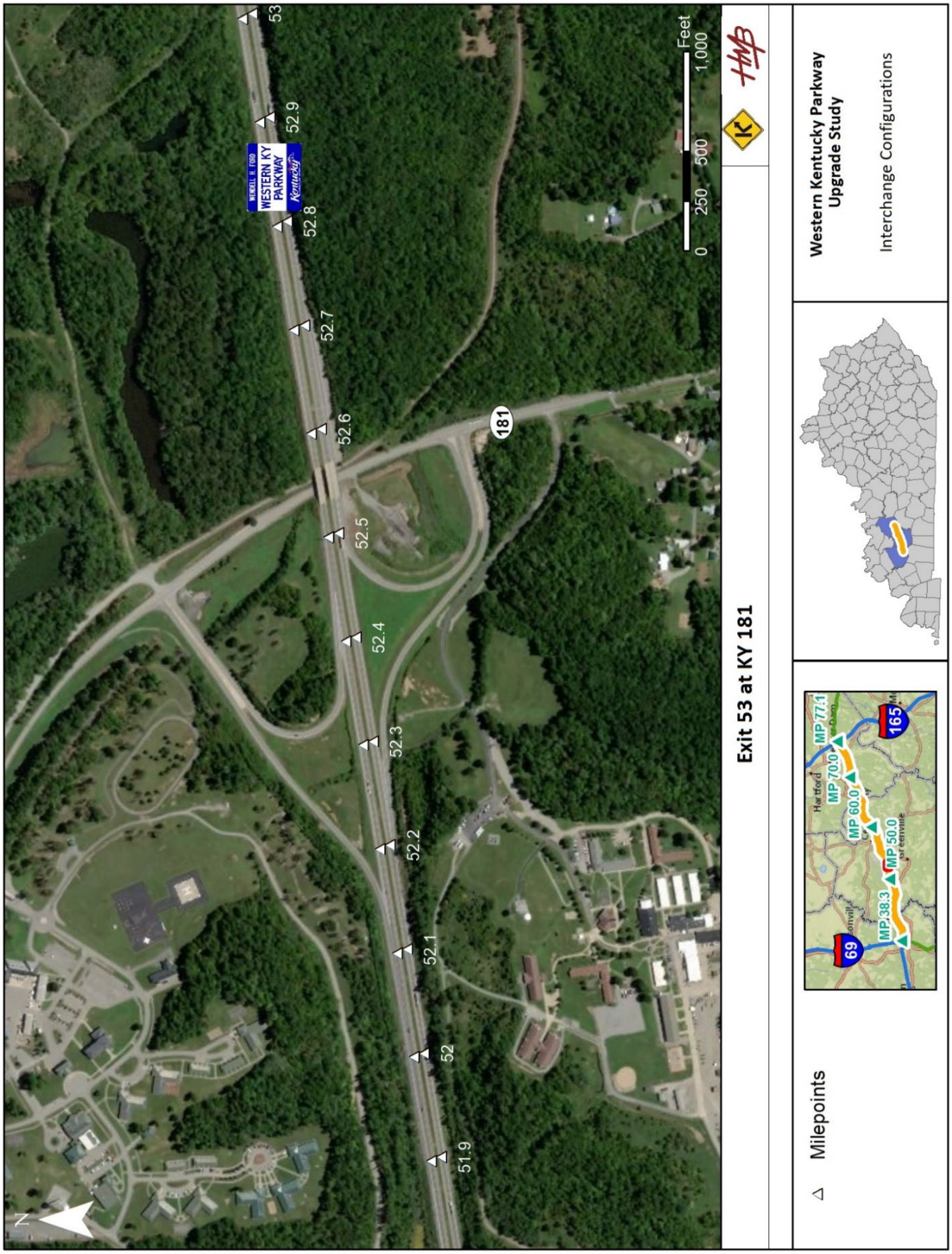
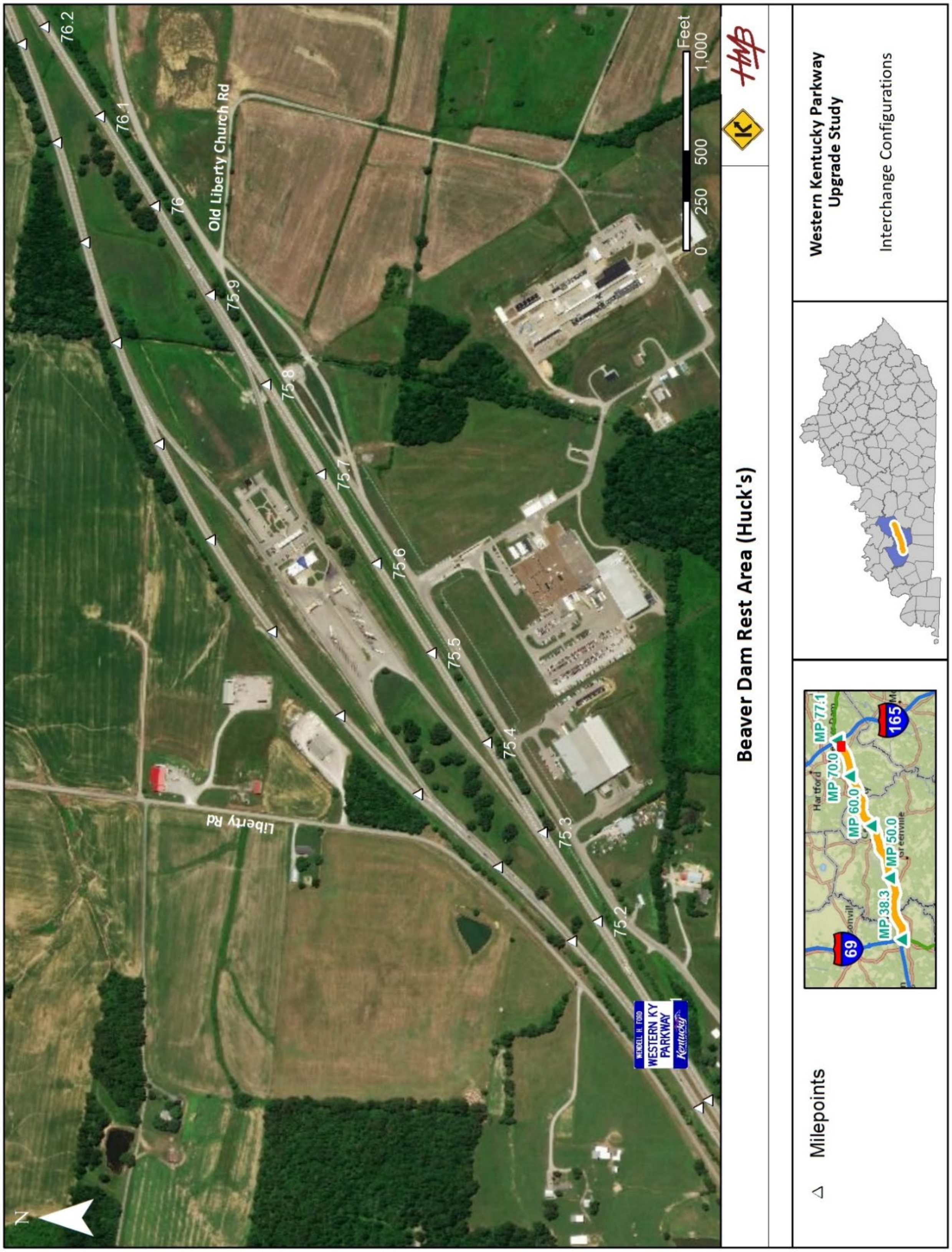


Figure 15. Huck's Interchange Configuration



Interchange Spacing

According to the *2018 Green Book,* a general rule of thumb for minimum interchange spacing is one mile in urban areas and two miles in rural areas. However, the *KYTC Highway Design Manual* recommends referring to *A Policy on Design Standards – Interstate System* when determining minimum criteria for interchange spacing on interstate facilities. Those standards state that the spacing of interchanges and between ramps has a significant effect on the operation of interstate highways. The spacing needed between interchanges will depend upon the combined effects of geometric design, traffic operations, safety performance, and signing. In rural areas interchanges should be spaced by no less than three miles.

Interchange spacing is measured from crossroad to crossroad. The following interchanges on the WKP do not meet the minimum criteria for interchange spacing and are illustrated on **Figure 16**:

- I-69 (Exit 38) to KSP Post No. 2 – 1.35 miles
- US 231 (Exit 75) to Huck's – 1.01 miles

Figure 10-70 in the *2018 Green Book* presents recommended minimum ramp terminal spacing for the various ramp pair combinations as they are applicable to interchange classifications. The minimum length between a successive entrance and exit ramp terminal between service interchanges on a full freeway is 1600 feet. Although the interchanges listed above do not meet interchange spacing requirements from crossroad to crossroad, all entrance and exit ramp terminals at these interchanges meet minimum ramp spacing criteria. The interchange and ramp terminal spacing for these locations are illustrated in **Figure 16**.

Interchange Control of Access

The *A Policy on Design Standards – Interstate System* state that access to the interstate system, including ramps, shall be fully controlled. Access control shall extend the full length of ramps and ramp terminals at the crossroad or frontage road. Controlling access on crossroads in the vicinity of interchanges can provide significant benefits to traffic operations and safety performance through the interchange area. A break in access control on the crossroad or frontage road should be no closer to the ramp terminal than 300 feet in rural areas.

According to the *KYTC Highway Design Manual,* access control should be measured from the radius return or the end of a taper on an entrance or exit ramp to the edge of the traveled way of the nearest access point. The farthest point from the entrance and exit ramps should determine the limits for access at an interchange, and the control of access should terminate at the same point on either side of the crossroad. Exhibit 1100-01 from the *KYTC Highway Design Manual* illustrates how control of access is measured. This is shown in **Figure 17**, and **Table 22** displays access control spacing for each interchange.

Figure 16. Deficient Interchange Spacing

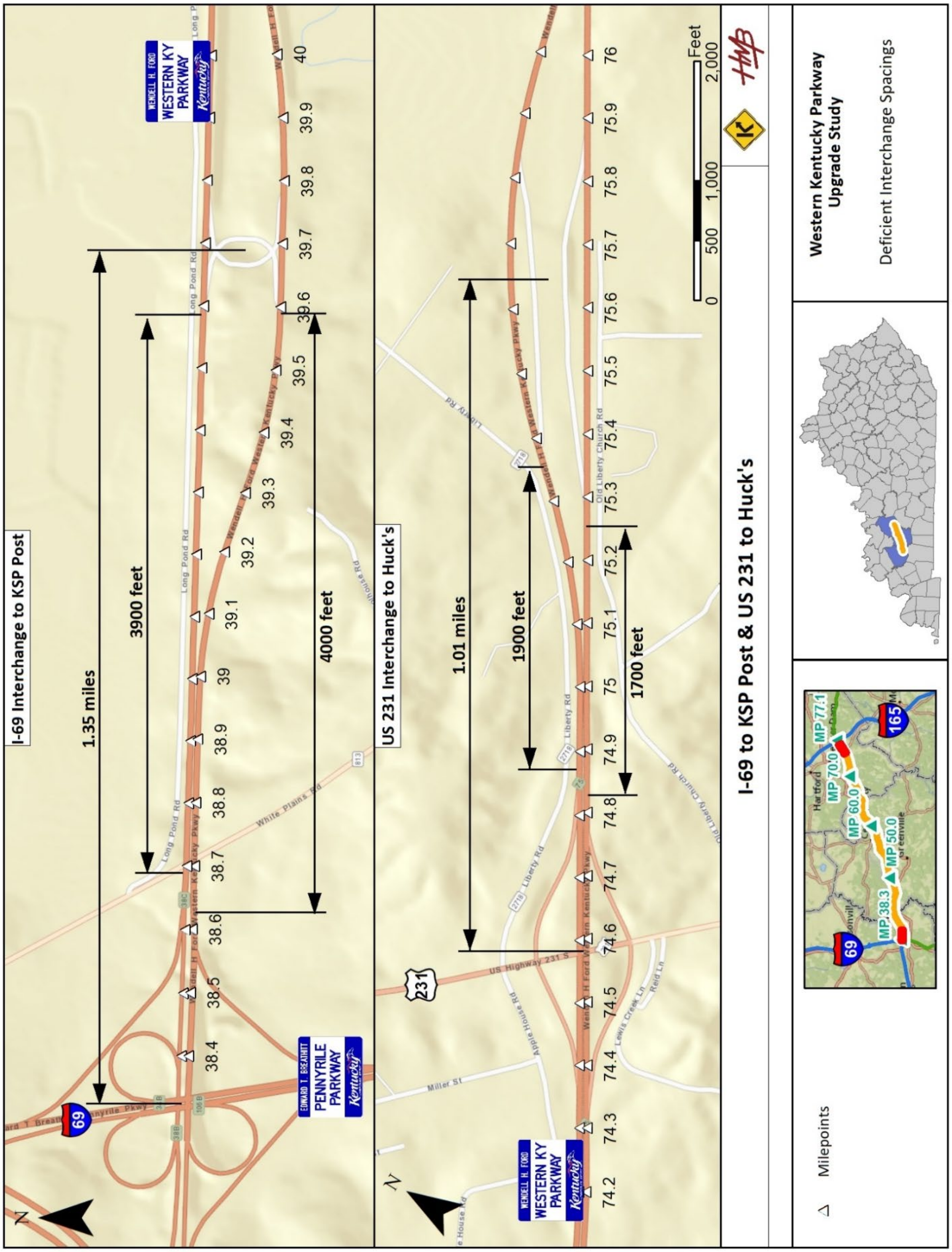
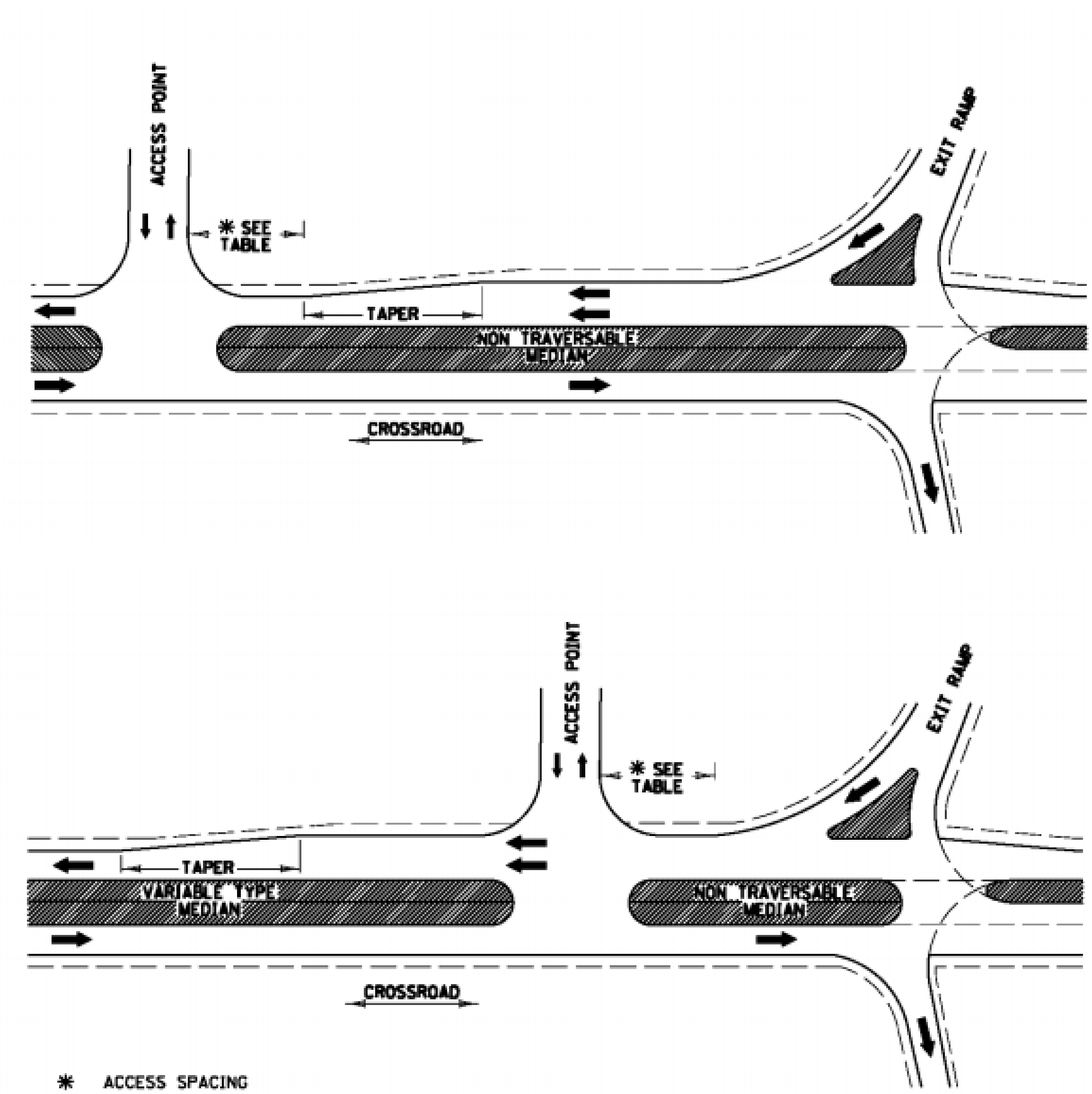


Figure 17. Control of Access Detail



* ACCESS SPACING

	MINIMUM	DESIRED
URBAN	100'	300'
RURAL	300'	600'

Source: KYTC Highway Design Manual Exhibit 1100-01

Table 22. WKP Existing Control of Access

Interchange	Measured from Ramp Termial (AASHTO, ft)	Measured From Ramp Taper (KYTC, ft)
Exit 48		
EB off Ramp	363	333
EB on Ramp	450	150
WB off Ramp	545	485
WB on Ramp	684	631
Exit 53		
EB off Ramp/EB on Ramp	389	337
WB off Ramp/WB on Ramp	0	0
Exit 58		
EB off Ramp	368	306
EB on Ramp	N/A	N/A
WB off Ramp	0	0
WB on Ramp	528	491
Exit 75		
EB off Ramp	118	81
EB on Ramp	267	210
WB off Ramp	200	153
WB on Ramp	211	168

Access control along the WKP was measured using statewide aerial imagery. Minimum criteria for interchange control of access was not met at the following locations along this corridor:

- KY 175 – 53 feet between the eastbound on-ramp and the nearest access point to the south, 30 feet between the eastbound on-ramp and the park and ride to the north
- KY 181 – Access point to a frontage road is directly across from the westbound on and off-ramps
- US 431 – Access point to a frontage road is directly across from the westbound off-ramp
- US 231 – 47 feet between the westbound off-ramp and the nearest access point to the north, 116 feet between the eastbound on-ramp and the nearest access point to the south

These four interchanges are illustrated on **Figure 18**. The new interchange configuration at US 431 that has been investigated as a part of this study corrects the existing control of access issue at this location.

Figure 18. Locations with Deficient Control of Access