KIPDA INTERCHANGES STUDY

Final Report

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Kentucky Transportation Cabinet

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1.1 STUDY INTRODUCTION

The KIPDA Interchanges Study was initiated by the Kentuckiana Regional Planning and Development Agency (KIPDA) in conjunction with the Kentucky Transportation Cabinet, District (KYTC) 5. The purpose of the study is to explore solutions to traffic problems associated with eight interchanges in Jefferson and Bullitt Counties in Kentucky. The interchanges evaluated in this study are listed below.

1. I-265 / Preston Highway (KY 61)
2. I-265 / Bardstown Road (US 31E)
3. I-265 / Taylorsville Road (KY 155)
4. I-265 / Old Henry Road (KY 3084)
5. I-265 / LaGrange Road (KY 146)
6. I-64 / Blankenbaker Parkway (KY 913)
7. KY 841 / Stone Street Road
8. I-65 / Brooks Road (KY 1526)

Study Area

Of the eight interchanges, seven are located in Jefferson County, with the remaining one located in Bullitt County. The study area for each interchange varies, but primarily consists of the area immediately surrounding the interchange as well as intersections close to the interchange. For each interchange in this study, the limits of the study area are presented in the section for that interchange.

Study Objectives

The primary objectives for this study are based on input and initial direction provided by KIPDA and KYTC. The study objectives are as follows:

1. Define key issues and goals;
2. Examine the existing conditions for each interchange;
3. Determine where (or if) there are problems or deficiencies;
4. Develop a range of alternatives focusing on low cost / near-term improvements that satisfy project goals and address identified problems;
5. Evaluate the effectiveness and feasibility of the alternatives; and
6. Recommend an alternative or set of alternatives for implementation.

Study Process

The study process used for this project consists of the six steps listed above. Because the study addressed eight different interchanges, each interchange was examined separately. Therefore, the study process was repeated for each interchange with the resulting evaluation presented in separate chapters (one for each interchange).

To provide a better understanding of the study process, the next sections in this report provide the background information relative to the study process for all the interchanges. This includes a discussion of project goals for the study as a whole, the study methodology, and the evaluation process and criteria. The chapters for each interchange are presented following the discussion of the evaluation process and criteria. Each chapter contains a complete analysis of each interchange including the existing conditions analysis, the development of alternatives, the alternatives analysis and evaluation, a comparison of alternatives, and a recommendation.

1.2 PROJECT GOALS

The project goals were used to provide focus for the study, ensuring that the study remained on track. For this study, the project goals were developed through input from both KIPDA and KYTC. The primary goal of this study was to identify feasible, low-cost, alternatives that reduce delay, enhance safety, and address periods of critical traffic congestion at each of the eight interchanges.

2.0 STUDY METHODOLOGY AND EVALUATION CRITERIA

2.1 EXISTING CONDITIONS ANALYSIS

To determine if there are deficiencies or problems with the existing highway, a detailed analysis was completed examining traffic volumes and patterns including truck traffic, highway geometrics, land use (both current and future), historic traffic growth, levels of service, crash rates, and other key issues. The analysis focused on current traffic conditions, but also considered, where appropriate, changes in traffic conditions as a result of planned new development. In support of the analysis, highway and traffic data was collected from a variety of sources including:

- KYTC Highway Information System database
- KYTC District 5 data sources
- Recent Transportation / Planning Studies
- Peak period turning movement traffic counts
- Study area field views

2.1.1 Traffic Analysis Methodology

The KIPDA Interchanges Study focused on critical intersections for each interchange in the study area. For this analysis the Highway Capacity Software package (HCS 2000) was used to assess the peak period traffic operating conditions. This software package implements the Highway Capacity Manual (HCM) intersection analysis method. For each study intersection, average vehicle delays were calculated as well as the resulting levels of service (LOS).

Level of service (LOS) is a qualitative measure of expected traffic conflicts, delay, driver discomfort, and congestion. Levels of service are described according to a letter rating system ranging from LOS A (free flow, minimal or no delays – best conditions) to LOS F (stop and go conditions, very long delays – worst conditions). For intersections, the Highway Capacity Manual (2000) defines levels of service based on the average delay due to signal or STOP control as shown in Table 2-1.

Table 2-1: LOS Criteria for Intersections

LOS	Signalized Intersections Control Delay (seconds vehicle)	Unsignalized Intersections Control Delay (seconds/vehicle)
A	$\leq$ 10	$\leq$ 10
B	>10 – 20	>10 – 15
C	>20 – 35	>15 – 25
D	>35 – 55	>25 – 35
E	>55 – 80	>35 – 50
F	>80	>50

Source: Highway Capacity Manual (2000)

A facility is considered to have reached its physical capacity at LOS E. Generally, LOS B or C is considered the threshold for desirable traffic conditions. However, in heavily developed metropolitan areas, LOS D may be acceptable. In this study, LOS C is used as the desirable threshold with LOS D as acceptable in the heavily developed areas such as Bardstown and Blankenbaker. Operations below this threshold are noted as undesirable and warrant improvement. LOS C corresponds to $\leq$ 35 seconds of delay per vehicle at a signalized intersection and $\leq$ 25 seconds of delay at an unsignalized intersection. LOS D corresponds to $\leq$ 55 seconds of delay per vehicle at a signalized intersection and $\leq$ 35 seconds of delay at an unsignalized intersection (Refer to the HCM published by the Transportation Research Board for more specific information.)

In order to determine the turning movements at the key intersections, peak period traffic counts were performed during October and November 2004. Hourly traffic data for nearby count stations were examined to determine the peak traffic periods to be counted. The peak periods were 7:00 to 9:00 AM (AM peak) and 4:00 to 6:00 PM (PM peak) for most of the study intersections. Turning movement counts were conducted during both of these periods, and the highest peak hour for each was selected for use in the HCS analysis. Intersection geometry, signal timing and other necessary traffic operations data was also collected and used to evaluate the intersection operations. For several of the unsignalized intersections that were considered candidates for signalization, additional counts were conducted during the week of March 21, 2005 between 9:00 AM and 4:00 PM. This was to provide the necessary count data to evaluate signal warrants for these intersections.

2.1.2 Crash Analysis

The Kentucky Transportation Cabinet provided crash data for a three-year period from January 1, 2001 through December 31, 2003. Crash rates were computed for specific segments of each major roadway in the study area using the methodology provided in the crash analysis report periodically published by the Kentucky Transportation Center (KTC)[1]. The section crash rates are based on the number of crashes on a specified section, the average daily traffic on the roadway, the time frame of analysis, and the length of the section. They are expressed in terms of crashes per 100 million vehicle-miles. A section's crash rate was then compared to a statewide critical crash rate[2] derived from critical crash rate tables for highway sections in the KTC crash report (Appendix D of KTC crash report). This comparison is expressed as a ratio of the section crash rate to the critical crash rate and is referred to as the critical crash rate factor. Sections with a critical crash rate factor greater than one are considered high crash locations and are potential candidates for safety improvements.

The section crash rate is also compared directly to the statewide average crash rate presented in the KTC crash report. The statewide averages consider all crashes for a specified period that are listed in the Collision Report Analysis for Safer Highways (CRASH) database maintained by the Kentucky State Police and stratified by functional classification (Table A-1 in KTC crash report). Section rates that exceed the statewide

[1] Analysis of Traffic Crash Data in Kentucky (1999 – 2004), Kentucky Transportation Center Research Report KTC-04-25/KSP2-04-1F.

[2] The critical crash rate is the threshold above which an analyst can be statistically certain (at a 99.5% confidence level) that the section crash rate exceeds the average crash rate for a similar roadway and is not mistakenly shown as higher than the average due to randomly occurring crashes.

average crash rate but not the critical crash rate may be problem areas, but they are not statistically proven to be higher crash areas. Therefore, this second comparison is used to identify a second tier of highway sections that may have crash problems and could be considered for safety improvements if warranted based on further analysis.

2.2 ALTERNATIVE DEVELOPMENT AND ANALYSIS

Alternatives Development

Based on the existing conditions analysis, a range of improvement alternatives were proposed to address the identified problems and deficiencies. The focus of the alternatives development was on low-cost and short-term improvement alternatives, with some longer term alternatives proposed as well. The improvements generally fell into two categories: 1) Intersection improvements and 2) System improvements. Intersection improvements are limited to a specific intersection and include improvements such as:

- Traffic signal installation at unsignalized intersections
- Intersection capacity improvements (adding / extending turn lanes)

System improvements were developed to improve traffic operations and safety through the intersections in the study area. Some examples include:

- Traffic signal timing / coordination improvements
- Addition of through lanes
- Intelligent Transportation System (ITS) options including advance warning signs

Alternatives Analysis

The analysis procedure used to evaluate each alternative varied depending on the type of improvement under consideration. For the general categories of improvements listed, the evaluation procedures are described below.

Traffic Signal Installation – For alternatives involving installation of a traffic signal, three evaluation methods were used to determine if the intersection should be signalized. The evaluation criteria are: level of service analysis, queue lengths, and signal warrants. The purpose of the level of service analysis is to determine if the intersection is operating below the desirable LOS threshold based on the existing conditions analysis, and if it is, to determine if signalization of the intersection would improve traffic operations to a desirable LOS. The queue length analysis was conducted both for the current unsignalized condition as well as for the signalized condition. Both analyses are based on output from the Highway Capacity Software package. The 95th percentile queue is used to estimate the maximum number of vehicles waiting in a queue for a specific approach. If there are multiple lanes for an approach, the queue calculated by this method is assumed to be the maximum queue found in any lane of the lane group. To calculate the length of the queue, the number of vehicles is multiplied by 25 feet (the assumed front to front distance for two successive vehicles in a travel lane). If the calculated queue length exceeds the available storage, then a problem may exist, requiring additional improvements. Finally, a traffic signal warrant evaluation was performed to determine if the intersection meets or exceeds any of the signal warrants as outlined in the Manual of Uniform Traffic Control Devices (MUTCD). According to the MUTCD, there are eight warrants used to justify the installation of a traffic signal, four of which are most relevant to intersections analyzed as part of this study. These four warrants are listed below along with a brief definition.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day.
- Warrant 2: Four-Hour Vehicular Volume – For this warrant, traffic volumes for each of any 4 hours of an average day must be above the applicable curve in Figure 4C-1 or 4C-2 in the MUTCD manual.
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must be such that they exceed the given threshold curve as shown on either Figure 4C-3 or 4C-4 in the MUTCD.
- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection.

From these three evaluation measures (LOS, queue length, and signal warrants), a recommendation regarding signal installation was made.

Intersection Capacity Improvements – For other intersection improvements such as adding / lengthening turn lanes, a level of service analysis was performed using the HCS software package. Existing levels of service and delay were compared to values resulting from intersection improvements to determine the extent to which they improve intersection operations. If it was determined that a lane should be extended, queue lengths were evaluated from the HCS output to determine the length of the needed extension.

System and Other Improvements – System improvements such as added through lanes and signal coordination were evaluated using Synchro 6.0. Synchro allows for the evaluation of a traffic network and returns measures of effectiveness (MOEs) for the entire network. Therefore, an existing conditions network was set up as a base scenario, allowing for comparison between all of the subsequent improvement alternatives. Other system improvements such as ITS solutions are harder to evaluate quantitatively, and may be evaluated qualitatively.

2.3 EVALUATION CRITERIA

In order to select the preferred alternative(s), each alternative will be evaluated qualitatively in several key categories that reflect the project goals. These include:

- Congestion
- Operations
- Use
- Safety
- Impacts
- Costs

Quantitative values calculated during the alternatives evaluation process were used to aid in the qualitative analysis if possible. These include values of delay for congestion and level of service for operations. For other categories such as use, safety, and impacts, a qualitative assessment was made. For all alternatives, planning level cost estimates in year 2005 dollars were developed to compare benefits versus costs.

While it may not be possible to directly compare two different alternatives, such as signal coordination and extending a left turn lane, it is possible to compare the identified benefits and estimated costs. As a result, one or more alternatives may be recommended for implementation. Short and long term recommendations may also be made.

3.0 I-265 / PRESTON HIGHWAY (KY 61) INTERCHANGE

3.1 INTRODUCTION AND STUDY AREA

The study area for the I-265 / Preston Highway (KY 61) interchange consists of the intersections listed below. Refer to Figure 3-1 for the limits of the study area.

1. Preston Highway (KY 61) / Cooper Chapel Road (CR 1002) / Commerce Crossings Drive
2. Preston Highway (KY 61) / I-265 Eastbound Ramps
3. Preston Highway (KY 61) / I-265 Westbound Ramps

3.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns
The current average daily traffic volumes for I-265 came from the Highway Information System (HIS) database. For the ramps, ADT flows were determined based on turning movement counts performed during October 2004 using a K-factor of 11%. Listed below are some of the highest traffic volumes through the interchange.

- Approximately 49,300 ADT on Preston Highway south of I-265
- Approximately 27,500 ADT on Preston Highway north of I-265
- Approximately 11,000 ADT on the ramp from Northbound Preston Highway to I-265
- Approximately 14,000 ADT on the eastbound exit ramp

Based on these traffic volumes, the major traffic flows through the interchange are to and from the south and west.

Geometrics / Right-of-way
An evaluation of the existing interchange features revealed the following:

- Partial cloverleaf, with one loop ramp and one directional ramp
- The loop ramp radius is 400 feet, which meets 35 mph design guidelines
- Two-lane directional (flyover) ramp carries the heavy northbound to westbound volume
- Two lane eastbound off-ramp feeds a separate channelized right-turn lane designed to carry the heavy eastbound to southbound volume
- Distance from eastbound off-ramp to Cooper Chapel is approximately 1,900 feet

Land Use, Future Development, and Historic Traffic Growth
The area surrounding the interchange currently has significant retail, commercial, and residential development, and is continuing to grow with several new developments proposed for the area. Build-out of approved developments in the area including Preston Crossing is expected to add approximately 1,200 trips to the interchange in the PM peak. Growing residential development in the Preston Highway traffic shed is also expected to increase traffic volumes through the interchange in the future.

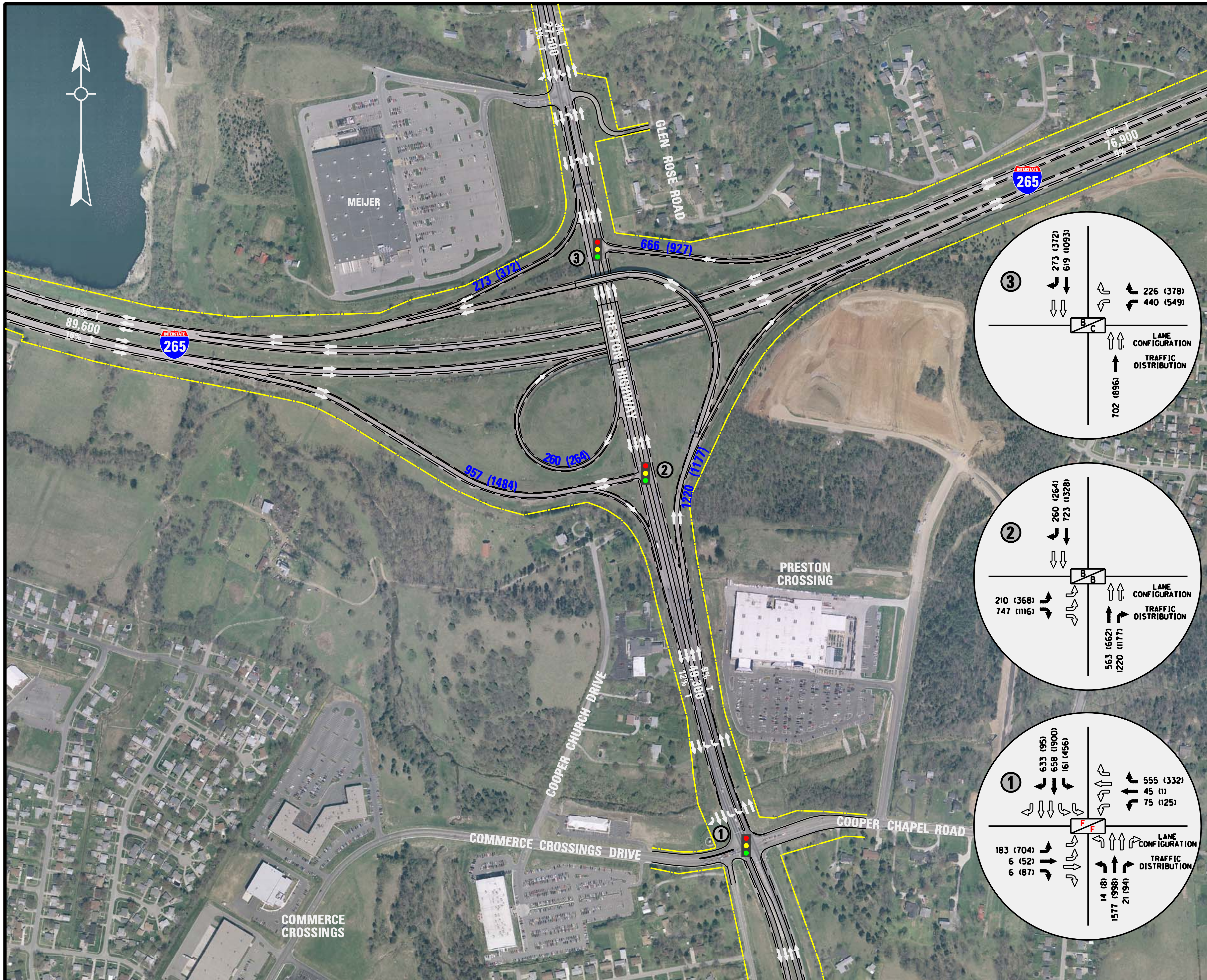


FIGURE 3-1: I-265 & PRESTON HIGHWAY (KY 61) INTERCHANGE

KEY ISSUES / DEFICIENCIES

- Poor LOS at Preston Highway / Cooper Chapel Road intersection
- Southbound merge/weave on Preston between EB Ramp and Cooper Chapel
- The poor operative conditions at Preston / Cooper Chapel cause significant upstream queues including congestion north past the westbound ramps.

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION
67,800 2004 AVERAGE DAILY TRAFFIC
980 (1080) 2004 AM (PM) PEAK HOUR VOLUMES
3% T PERCENT TRUCKS
2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600
GRAPHIC SCALE IN FEET

From historic traffic data, traffic has grown at the following rates between 1985 and 2004:

KY 61 South of I-265	KY 61 North of I-265	I-265 West of Preston Hwy	I-265 East of Preston Hwy
4%	<1%	12%	4%

Based on the available information, traffic volumes (especially to and from the south and west) are likely to grow significantly over the next 10 years.

Traffic Operations / Level of Service Analysis
Peak period turning movement counts were conducted on 10/14/04 and 11/17/04. Additional data was acquired from the Preston Crossings Phase 3 Traffic Study (BTM, 2003). For each of the key intersections, AM and PM peak hour volumes are shown in Figure 3-1. Existing levels of service and delay are also shown in Table 3-1 below.

Table 3-1: 2004 Intersection Levels of Service for I-265 / Preston Highway

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Preston Highway / Cooper Chapel Road	Signalized	Eastbound	**74.7**	**E**	**124.9**	**F**
		Westbound	**66.1**	**E**	**66.5**	**E**
		Northbound	**169.1**	**F**	**55.3**	**E**
		Southbound	44.4	D	**125.8**	**F**
		Whole Int.	**108.7**	**F**	**104.7**	**F**
Preston Highway / I-265 EB Ramps	Signalized	**Whole Int.**	**15.1**	**B**	**18.3**	**B**
Preston Highway / I-265 WB Ramps	Signalized	**Whole Int.**	**18.4**	**B**	**21.4**	**C**

According to Table 3-1, both ramp intersections operate at an acceptable level of service (LOS C or better). The Preston / Cooper Chapel intersection currently operates at LOS F during both peak periods.

Field observations confirmed that the Preston / Cooper Chapel intersection does operate poorly and in fact causes upstream congestion during peak periods. Southbound queues were observed extending from the Preston / Cooper Chapel intersection back to the eastbound ramp intersection and there were even impacts to the westbound ramp intersection. The considerable congestion between the eastbound off-ramp and Cooper Chapel is likely due to three factors: poor LOS at Cooper Chapel, the merge with the eastbound off-ramp, and weaving between the ramp and the intersection.

Due to the poor intersection operations at the Preston / Cooper Chapel intersection, this intersection, unlike the ramp intersections, has a number of queuing problems. Specific movements with queues that exceed the available storage are listed in Table 3-2. This table is based on the Highway Capacity Manual method (95th percentile) and uses the existing signal timing. This method is sometimes conservative in estimating queues.

Table 3-2: Movements with Queues that Exceed the Available Storage at Preston Highway / Cooper Chapel

Int.	Approach / Movement	Design Hour	95th Percentile Queue (HCM)	Queue Length (ft)	Available Storage Length (ft)	Notes
Preston/ Cooper Chapel	EB Left	AM	11.5	288	270	**EXCEEDS** available storage
		PM	48.2	1205	270	**EXCEEDS** available storage
	EB Right	PM	5.2	130	50	**EXCEEDS** available storage
	WB Right	AM	23.4	585	310	**EXCEEDS** available storage
		PM	24.3	608	310	**EXCEEDS** available storage
	SB Left	PM	26.8	670	500	**EXCEEDS** available storage
	SB Right	AM	31.2	780	150	**EXCEEDS** available storage

Safety / Crash Analysis

The crash analysis did not show a crash rate problem (see Table 3-3). The majority of crashes on Preston were rear-end crashes. The fatal crash on Preston occurred north of the interchange in June 2002 during the middle of the day. It was an angle collision with one vehicle turning left.

Table 3-3: 2001 – 2003 Crash Analysis for I-265 / Preston Highway Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-265	**39**	**12**	**1**	**31**	**74**	**111**	**0.28**
KY 61	**125**	**47**	**1**	**215**	**332**	**339**	**0.63**

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies

Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor LOS at Preston Highway / Cooper Chapel Road intersection
- Southbound merge/weave on Preston between Eastbound Ramp and Cooper Chapel
- The poor operating conditions at Preston / Cooper Chapel cause significant upstream queues including congestion north past the westbound ramps.

3.3 RANGE OF ALTERNATIVES

Below is a list of possible improvement alternatives that address identified deficiencies in the existing conditions analysis. Refer to Figure 3-2 for conceptual drawings for these alternatives.

- **Alternative 1** – Add eastbound right turn lane on the I-265 eastbound off-ramp at the signal to accommodate ramp traffic turning right on Preston Highway and then left at Cooper Chapel (i.e. to reduce weaving).
- **Alternative 2** – Eliminate the free-flow right-turn movement at the I-265 eastbound off-ramp and allow right turns at the signal from new dual right turn lanes. This benefits pedestrians by eliminating a high-speed free-flow movement.
- **Alternative 3** – Widen the I-265 westbound off-ramp to provide dual left turn lanes to accommodate the heavy left turn traffic (>300 in the PM peak hour).
- **Alternative 4** – Add turn and/or through lanes at the Cooper Chapel intersection to improve intersection operations (four options).
- **Alternative 5** – Upgrade signal system and timing / phasing (in conjunction with other improvements).

3.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – New Eastbound Right-Turn Lane on I-265 Eastbound Off-Ramp at Signal

This alternative was proposed to separate the right turn traffic on the I-265 eastbound off-ramp by direction, by redirecting traffic ultimately making a left at Cooper Chapel to go through the traffic signal on Preston Highway at the eastbound ramp terminus. Field observations showed that during the PM peak traffic backs up from Preston Highway/Cooper Chapel to the ramp and on the ramp for the right turn movement. In order to more easily weave across Preston Highway to turn left at Cooper Chapel, vehicles were observed using the left turn lane at the signal to turn right to get in the leftmost lanes on southbound Preston Highway. This alternative allows for vehicles to make this maneuver by providing a separate right turn lane at the signal. Traffic headed south on Preston Highway or turning right into Commerce Crossings would continue to use the existing free-flow right turn movement. Vehicles getting off I-265 and wanting to make a left at Cooper Chapel would make a right through the ramp intersection. These vehicles would use the protected left-turn green time to pull directly into the leftmost lane of southbound Preston Highway, thereby reducing weaving across the short distance between the intersections.

Traffic and Safety – There are other locations that operate similarly to this proposed alternative including the Watterson (I-264) / Shelbyville Road interchange lanes leading to LaGrange Road. Separating these left-turn movements reduces weaving (thereby improving safety) between the ramp intersection and the next intersection downstream. A level of service analysis showed that adding the eastbound exit traffic destined for Cooper Chapel to the signalized ramp terminus had little effect on LOS or delay at the ramp intersection.

FIGURE 3-2: I-265 & PRESTON HIGHWAY (KY 61) INTERCHANGE

ALTERNATIVES

- Alt. 1 - New EB Right Turn Lane on I-265 EB Off-Ramp at Signal
- Alt. 2 - Eliminate Free-Flow Right Turn From I-265 EB Off-Ramp (Change to Dual Right Turn Lanes at Signal)
- Alt. 3 - Widen I-265 WB Off-Ramp to Provide Dual Left Turn Lanes
- Alt. 4 - Add Turn and / or Through Lanes at Cooper Chapel Intersection
  - Option 1 - Optimize Signal Timing (With Overlap Phases)
  - Option 2 - Optimize Signal Timing and Add WB Right Turn Lane
  - Option 3 - All of the Above Plus Add NB Thru Lane
  - Option 4 - All of the Above Plus Add SB Thru Lane
- Alt. 5 - Upgrade Signal System and Timing / Phasing

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

250 0 250 500 750
GRAPHIC SCALE IN FEET

Community / Environmental Impacts – The existing right-of-way appears to be sufficient to accommodate the addition of a right turn lane at the ramp intersection with Preston Highway. There are no known environmental or development impacts associated with implementation of this alternative.

Costs – The order of magnitude cost estimate for this alternative is $160,000 in 2005 dollars.

Alternative 2 – Eliminate Free-flow Right Turn from I-265 Eastbound Off-Ramp

For this alternative, all right turn traffic on the eastbound off-ramp would go through the signal. Separate right turn lanes would be constructed to accommodate this movement. Because the right turn volume is high, intersection operations were evaluated assuming dual right turn lanes.

Traffic and Safety – Levels of service and delay for the existing conditions and both lane configurations for this alternative are shown in Table 3-4.

Table 3-4: Alternative 2 Level of Service and Delay Comparison for Preston Highway / I-265 Eastbound Ramps

			AM		PM	
Intersection	**Scenario**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Preston Highway / I-265 EB Ramps	Existing	**Whole Int.**	**15.1**	**B**	**18.3**	**B**
	Alt. 2 (Dual Right Turn Lane)	**Whole Int.**	**18.3**	**B**	**31.8**	**C**

Based on this analysis, the intersection would operate acceptably with dual right turn lanes during both peak periods. One advantage of removing the free-flow right turn lane would be that it would increase crossing safety for pedestrians. Eliminating the free-flow right turn lane would also reduce weaving on the short section of Preston Highway between this intersection and the Preston / Cooper Chapel intersection.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative – all improvements should remain within the existing right-of-way.

Costs – The order of magnitude cost estimate for this alternative is $300,000 in year 2005 dollars.

Alternative 3 – Widen I-265 Westbound Off-Ramp to Provide Dual Left Turn Lanes

This alternative was proposed as a result of high left-turn volumes at the westbound ramp intersection. For this area and type of intersection, KYTC typically considers dual left-turn lanes when the peak hour volume exceeds 300 vehicles. Based on turning movement counts conducted at this intersection, the peak hour turn volume in the AM is 440 and in the PM is 549, both of which exceed 300 vehicles.

Traffic and Safety – To determine how the addition of a left turn lane affects intersection operations, levels of service and delay were calculated for this alternative. They are shown in Table 3-5 compared to the existing values.

Table 3-5: Alternative 3 Level of Service and Delay Comparison for Preston Highway / I-265 Westbound Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Preston Highway / I-265 WB Ramps	Existing	**Whole Int.**	**18.4**	**B**	**21.4**	**C**
	Alt. 3	**Whole Int.**	**16.8**	**B**	**18.4**	**B**

The existing ramp configuration operates at or above an acceptable level of service. With the addition of the left turn lane, intersection operations are improved one LOS further in the PM peak hour. The queue lengths for the existing conditions as well as Alternative 3 do not exceed the available storage even with the high left turn volumes. Therefore, according to this analysis, this intersection operates acceptably for either scenario. While there is currently not any level of service or queuing issues, it is possible that intersection operations could decline in the future if traffic volumes continue to increase. With the expected future traffic growth and the already high left turn volumes, it is recommended that KYTC consider widening this ramp to construct dual left turn lanes. To accommodate the maximum calculated queue based on the existing traffic volumes, the turn lanes should extend back from the intersection for a minimum distance of 350 feet.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The order of magnitude cost estimate for this alternative is $290,000 in year 2005 dollars.

Alternative 4 – Add Turn and/or Through Lanes at the Cooper Chapel Intersection

Traffic and Safety – Of the three intersections analyzed for this interchange, this is the only intersection that operates below the desirable Level of Service C threshold. In fact, congestion as this location causes other intersections along Preston Highway to operate poorly during peak traffic periods. Currently, Louisville Metro has proposed improvements to Cooper Chapel Road, but the project will likely not affect conditions at Preston Highway since the project does not include improvements to the Preston Highway / Cooper Chapel Road intersection. Therefore, in order to improve intersection operations, several different improvements were evaluated that target the movements with the highest values of delay. Each of these improvement options were evaluated using the HCS methods. The results are presented in Table 3-6. A discussion of each improvement follows the table.

Table 3-6: Alternative 4 Level of Service and Delay Comparison for Preston Highway / Cooper Chapel Road

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Preston Highway / Cooper Chapel Road	Existing	**Whole Int.**	**108.7**	**F**	**104.7**	**F**
	Option 1 - Optimize Signal Timing (with overlap phases)	**Whole Int.**	**34.5**	**C**	**46.3**	**D**
	Option 2 - Optimize Signal Timing, Add WB Right Turn Lane	**Whole Int.**	**32.8**	**C**	**45.9**	**D**
	Option 3 - Optimize Signal Timing, Add WB Right Turn Lane, Add NB Thru Lane	**Whole Int.**	**27.9**	**C**	**45.2**	**D**
	Option 4 - Optimize Signal Timing, Add WB Right Turn Lane, Add NB / SB Thru Lanes	**Whole Int.**	**28.9**	**C**	**34.2**	**C**

Note: For Options 1, 2, and 3 the eastbound and westbound approach levels of service are actually LOS E. Only Option 4 has acceptable levels of service for all movements.

Optimizing the signal timing for this intersection significantly reduces the overall delay. The whole intersection operates at either a level of service C or D, which in this case is acceptable, but the eastbound and westbound movements still operate below the desirable level of service threshold at LOS E. To achieve these levels of service, the signal optimization also includes right turn overlap phases during the EB-WB left turn phase, the NB-SB left turn phase, and the EB and SB phases. The previous signal timing plan only allowed a right turn overlap phase for the WB right during the NB-SB left turn phase and the southbound phase. While it appears that good overall levels of service can be achieved through signal optimization, given daily fluctuations of traffic and the fact that this timing plan is based on one day of traffic counts, these levels of service may not be achievable at all times. In addition, signal optimization will not fix all of the queuing issues identified at this intersection in the existing conditions analysis. Therefore, while it appears that signal optimization would improve intersection operations, it is only the first step to solving the operational issues at this intersection.

A second option is to construct an additional right turn lane to provide dual right turns from westbound Cooper Chapel to Preston. This improvement was proposed by Birch, Trautwein, and Mims, Inc. (BTM) in a study completed in 2003 for the Preston Crossings development. It was primarily proposed to provide adequate storage for the right-turn movement. The addition of the westbound right turn lane reduces average delay by a few seconds, but, it reduces the queue length such that vehicles do not back up to Preston Crossing Boulevard / Cooper Chapel Road intersection to the east.

Even with these improvements, the eastbound and westbound left and through and the northbound and southbound left turns operate at a poor level of service. All of these movements except the eastbound left and the southbound left have low volumes (125 or less vehicles during the peak hour). Adding turn lanes for these low volumes may not be the most cost-effective measure of reducing delay at this intersection. The other two movements (eastbound and southbound left) have higher volumes, but already have dual left turn lanes. Triple turn lanes are only used in special circumstances where high turning movement volumes warrant the use. Overall, these volumes are not high enough to warrant improvements at this time. Therefore, the last two options that were explored include the addition of a northbound through lane and the addition of northbound and southbound through lanes.

The addition of a northbound through lane was considered first because it could be constructed more easily with the existing geometrics. Preston Highway would be widened to three through lanes from just south of Cooper Chapel to the I-265 on-ramp. As shown in Table 3-6, the third northbound through lane only moderately improves intersection operations, reducing delay by a few seconds in the AM peak. To determine how constructing a short six-lane section on Preston Highway in this area would affect traffic operations, the intersection was also evaluated with the addition of a southbound through lane. This led to the best intersection operations with Level of Service C for the intersection overall. In addition, all queuing issues on the Preston Highway and westbound Cooper Chapel Road approaches were resolved through implementation of these improvements. Queuing issues still remained for the eastbound approach on Commerce Crossings Drive, but as these are on the side street they should have a limited impact on traffic operations on Preston Highway and through the interchange.

Therefore, to improve intersection operations at Preston Highway / Cooper Chapel Road / Commerce Crossings, it is recommended that the westbound right turn lane on Cooper Chapel be constructed in addition to a new southbound and northbound through lane for a total of three through lanes in each direction (Option 4). In conjunction with these improvements, the signal timing should be optimized allowing for right turn overlap phases where feasible. The additional southbound and northbound through lanes should begin approximately 1,000 – 2,000 feet south of Cooper Chapel Road or near the intersection of Preston Highway / Maple Spring Drive (the next intersection south). They should then continue north to the I-265 eastbound ramps.

Community / Environmental Impacts – It is likely that even with the additional westbound turn lane and through lanes, these improvements can be constructed within the existing right-of-way.

Costs – The order of magnitude cost estimate for the full improvement of this intersection (Option 4) is $1.8 million in year 2005 dollars. The estimated cost of the westbound right-turn lane proposed in the BTM study is $250,000 in year 2005 dollars. This could be deducted from the total cost if completed by the developer.

Alternative 5 – Upgrade Signal System and Timing / Phasing

Traffic and Safety – Currently the two signalized ramp intersections operate acceptably, with only the Preston / Cooper Chapel intersection operating below the desirable LOS threshold. To further improve operations at Preston / Cooper Chapel and reduce congestion between this intersection and the I-265 Eastbound Ramp intersection, the timing plans for all three signals could be coordinated. According to the FHWA Signalized Intersections Informational Guide (FHWA-RT-04-09, August 2004), signal coordination is recommended if signals are within 0.5 miles of each other on a major route. The distance between each of these three signals is less than 0.5 miles, therefore signal coordination may be of benefit. Using Synchro 6.0 to analyze the impact of signal coordination on the network, the overall reduction in delay was found to be moderate (<10%). The intersection LOS and delay also are only improved slightly, with the intersection of Preston / Cooper Chapel still operating poorly. Based on this analysis, the three signals could be coordinated, but this should be completed in conjunction with other more critical improvement projects at the interchange.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The order of magnitude cost estimate for this alternative is between $40,000 and $90,000 in year 2005 dollars.

3.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed I-265 / Preston Highway Interchange alternatives is provided in Table 3-7.

Table 3-7: I-265 / Preston Highway Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	New EB Right Turn Lane on I-265 EB Off-Ramp at Signal	◗	●	◗	●	●	●	**YES**
2	Eliminate Free-flow Right Turn from I-265 EB Off-Ramp - Dual Right Turn Lanes	◗	◗	●	●	●	◗	**NO**
3	Widen I-265 WB Off-Ramp to Provide Dual Left Turn Lanes	◗	◗	◗	◗	●	◗	**YES** (low priority)
4	Option 4: Optimize Signal Timing, Add WB Right Turn Lane, Add NB and SB Thru Lanes at Preston Highway / Cooper Chapel Intersection	●	●	●	◗	●	○	**YES**
5	Upgrade Signal System and Timing / Phasing	◗	◗	●	○	●	●	**Possibly with other projects**

Ratings Guide: ○ **= Poor** ◗ **= Fair** ● **= Good**

3.6 RECOMMENDATION AND PHASING

Several improvement projects are recommended to reduce identified congestion, queuing problems, and safety issues. These include the following:

- Construct new eastbound right turn lane on the I-265 eastbound off-ramp at the signal.

- Construct a second westbound left turn lane on the I-265 westbound off ramp to provide dual left turn lanes. The turn lane lengths should extend back from the intersection for a minimum of 350 feet. (This is a lower priority item.)

- Optimize signal timing and construct westbound left turn lane and northbound and southbound through lanes at the intersection of Preston Highway / Cooper Chapel Road (Alternative 4, Option 4). Based on the BTM Study, the westbound right-turn lane is the developer's responsibility.

- Upgrade the signal system and timing / phasing to allow for coordination between all three signals. This should be completed as part of the other future improvement projects at this interchange.

It is recommended that the new eastbound right turn lane on the I-265 eastbound off-ramp be constructed in conjunction with the improvements at the Preston Highway / Cooper Chapel Road intersection which includes widening Preston Highway from approximately 1000 – 2000 feet south of Cooper Chapel Road to the I-265 eastbound ramps. The other option of eliminating the eastbound free-flow right turn at the eastbound intersection and constructing dual right turn lanes at the signal was considered instead of recommending the single right turn lane addition. This option would reduce weaving between the eastbound ramps and Cooper Chapel and may be better from a safety standpoint. However, it was determined that while this option may improve safety, there are no currently apparent crash rate problems with the existing configuration and the option of constructing a single right turn lane at the signal and leaving the free-flow right turn lane is the preferred option since it would likely yield better utilization of the new southbound through lane.

4.0 I-265 / BARDSTOWN ROAD (US 31E) INTERCHANGE

4.1 INTRODUCTION AND STUDY AREA

The study area for the I-265 / Bardstown Road (US 31E) interchange consists of the four signalized intersections listed below (refer to Figure 4-1).

1. Bardstown Road (US 31E) / I-265 Eastbound Ramps
2. Bardstown Road (US 31E) / I-265 Westbound Ramps
3. Bardstown Road (US 31E) / Wal-Mart Driveway
4. Bardstown Road (US 31E) / Kroger Driveway

4.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns

The current average daily traffic volumes for I-265 came from the Highway Information System (HIS) database. For the ramps, ADT flows were determined based on turning movement counts performed during October 2004 using a K-factor of 11%. Listed below are some of the highest traffic volumes through the interchange.

- Approximately 24,400 ADT on Bardstown Road south of I-265
- Approximately 37,700 ADT on Bardstown Road north of I-265 (3% trucks)
- Eastbound exit ramp has highest ramp volume (approx. 1,200 vehicles) in the PM peak

Traffic flow through the interchange is generally balanced with slightly more traffic coming to and from the south and west. Of the two intersections north of the interchange, peak hour volumes are higher for the Wal-Mart intersection compared to the Kroger intersection.

Geometrics / Right-of-way

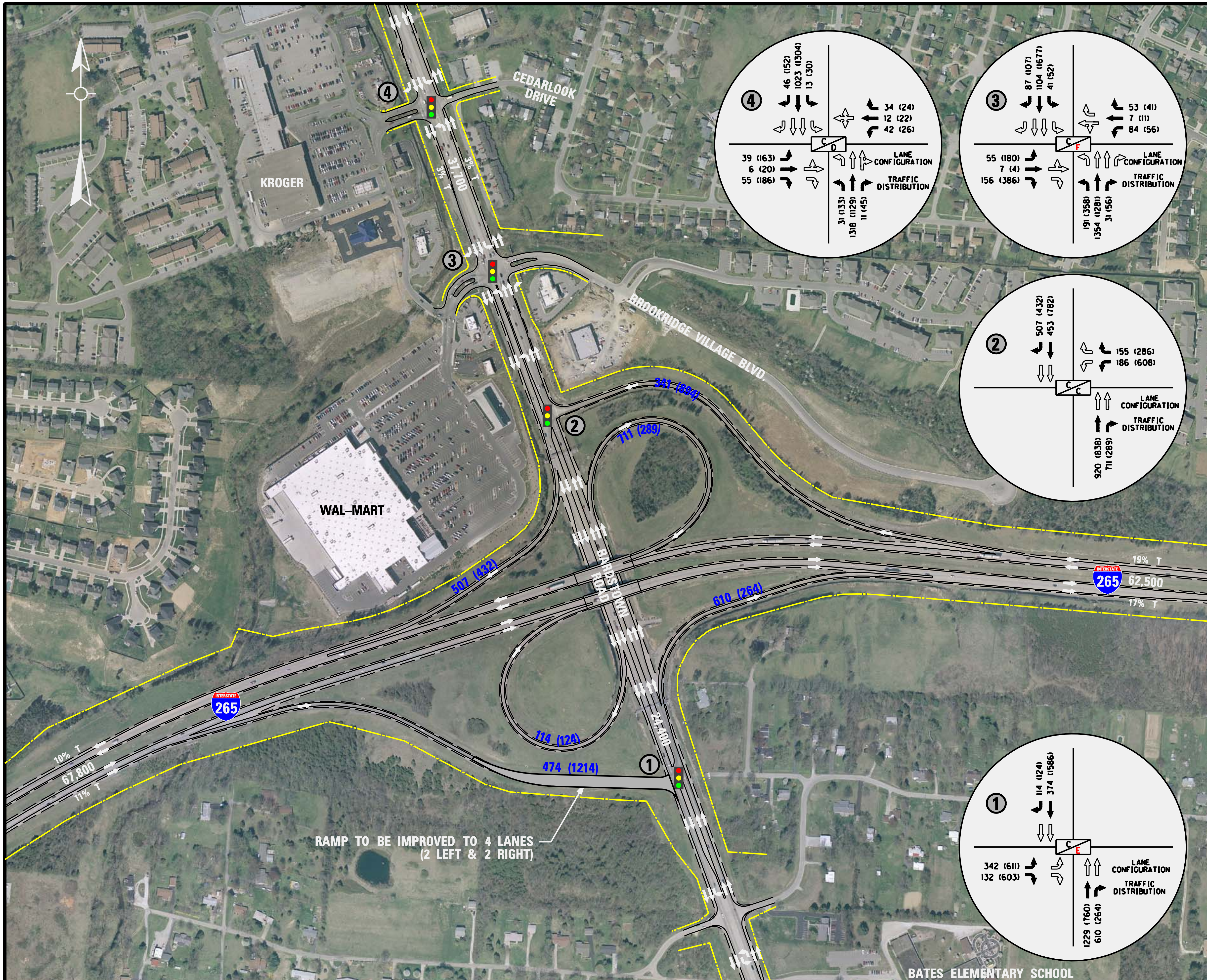
An evaluation of the existing interchange revealed the following:

- Partial cloverleaf, with single-lane free flow on-ramps to I-265 in all directions
- Single lane off-ramps, flaring to two lanes at the ramp intersections
- Bardstown Road is a four-lane divided highway, with turn lanes at major intersections
- Distance from the westbound off-ramp to the Wal-Mart drive is 630 feet
- Distance from Wal-Mart drive to Kroger driveway is 720 feet
- No signals are located for some distance to the south
- Eastbound Ramp terminus currently being widened to 4 lanes (2 left-turn and 2 right-turn lanes)

Land Use, Future Development, and Historic Traffic Growth

The area surrounding the interchange, particularly north of the interchange, is heavily developed with a mix of commercial, retail and residential land uses including Wal-Mart and Kroger shopping centers. Southeast of the interchange is Bates Elementary

FIGURE 4-1: I-265 & BARDSTOWN ROAD (US 31E) INTERCHANGE



KEY ISSUES / DEFICIENCIES

- Heavy traffic flows with congestion on Bardstown Road
- Poor levels of service on ramps from I-265 to Bardstown Road
- Poor levels of service at Bardstown Road / Wal-Mart Driveway intersection

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP-CONTROLLED INTERSECTION
- 67,800 — 2004 AVERAGE DAILY TRAFFIC
- 980 (1080) — 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T — PERCENT TRUCKS
- 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600

GRAPHIC SCALE IN FEET

School. Also, south of the interchange, residential and commercial development continues thereby adding to the already high traffic volumes. An examination of historic traffic data showed that traffic in the vicinity of the interchange has been growing at approximately 4% per year since 1992, with slightly higher growth south of the interchange on Bardstown at 6% per year since 1989. Substantial background traffic growth is expected to continue due to growth and development in Bullitt, Spencer, and Nelson counties.

Traffic Operations / Level of Service Analysis

Peak period turning movement counts were conducted on 10/13/04. Turning movement count data for the Bardstown Road / Wal-Mart Driveway intersection was provided by KYTC. For each of the key intersections, AM and PM peak hour volumes are shown in Figure 4-1. Existing levels of service and delay are shown on Table 4-1.

Table 4-1: 2004 Intersection Levels of Service for I-265 / Bardstown Road

			AM		PM	
Intersection	**Type**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Bardstown Road / I-265 EB Ramps	Signalized	Eastbound	34.0	C	**165.1**	**F**
		Northbound	21.2	C	18.9	B
		Southbound	14.7	B	30.9	C
		Whole Int.	**22.9**	**C**	**78.4**	**E**
Bardstown Road / I-265 EB Ramps	Signalized with Imp.	**Whole Int.**	**22.0**	**C**	**31.4**	**C**
Bardstown Road / I-265 WB Ramps	Signalized	Westbound	28.5	C	**60.3**	**E**
		Northbound	20.6	C	21.3	C
		Southbound	16.6	B	20.7	C
		Whole Int.	**20.8**	**C**	**33.7**	**C**
Bardstown Road / Wal-Mart Driveway	Signalized	Eastbound	**89.2**	**F**	**218.6**	**F**
		Westbound	54.9	D	53.9	D
		Northbound	29.2	C	**102.4**	**F**
		Southbound	24.1	C	**230.6**	**F**
		Whole Int.	**32.5**	**C**	**172.8**	**F**
Bardstown Road / Kroger Driveway	Signalized	**Whole Int.**	**32.6**	**C**	**40.4**	**D**

Both the Bardstown Road / I-265 Eastbound Ramp intersection and the Bardstown Road / Wal-Mart Driveway intersection have poor overall levels of service. However, the widening scheduled for the Eastbound Ramp should improve the ramp operations to an acceptable LOS (as shown in Table 4-1). Generally, field observations showed that there is heavy congestion through the entire interchange vicinity as a result of high traffic volumes, particularly in the north part of the study area.

An analysis of queue lengths for these intersections showed that there are several locations where queue lengths exceed the available storage. Table 4-2 lists all of the movements with queues that exceed the available storage. Queue lengths for the eastbound ramp intersection currently exceed the available storage, but when the ramp widening is completed, these queuing issues will be resolved.

Table 4-2: Existing Queuing Issues for I-265 / Bardstown Road Interchange

Int.	Approach / Movement	Design Hour	95th Percentile Queue	Queue Length (ft)	Available Storage Length (ft)	Notes
Bardstown Road / I-265 WB Ramps	WB Left	AM	11.8	295	190	**EXCEEDS** available storage
		PM	51.3	1283	190	**EXCEEDS** available storage
	WB Right	PM	10.9	273	190	**EXCEEDS** available storage
Bardstown Road / Wal-Mart Driveway	EB Left	PM	15.2	380	160	**EXCEEDS** available storage
	EB Right	AM	11.3	283	160	**EXCEEDS** available storage
		PM	37.9	948	160	**EXCEEDS** available storage
	NB Left	PM	28.5	713	330	**EXCEEDS** available storage
Bardstown Road / Kroger Driveway	EB Left	PM	13.0	325	150	**EXCEEDS** available storage
	EB Right	PM	9.4	235	150	**EXCEEDS** available storage
	WB Left and Right	AM	6.8	170	120	**EXCEEDS** available storage
		PM	6.2	155	120	**EXCEEDS** available storage

Note: The available storage on the westbound ramp was assumed to be the distance from the intersection to where the ramp narrows to below 18 feet or where two cars can no longer pass. The actual length of the ramp is 1,990 feet.

Safety / Crash Analysis

The crash analysis did not show a crash rate problem (see Table 4-3).

Table 4-3: 2001 – 2003 Crash Analysis for I-265 / Bardstown Road Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-265	**43**	**13**	**0**	**50**	**74**	**115**	**0.44**
US 31E	**16**	**6**	**0**	**37**	**332**	**550**	**0.07**

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies

Based on the existing conditions analysis, the key issues / deficiencies are:

- Heavy traffic flows with congestion on Bardstown Road
- Poor levels of service on westbound ramp from I-265 to Bardstown Road
- Poor levels of service at Bardstown Road / Wal-Mart Driveway intersection

4.3 RANGE OF ALTERNATIVES

- **Alternative 1** – Improve westbound ramp terminus (added / longer turn lanes)
- **Alternative 2** – Intersection improvements at Bardstown Road / Wal-Mart Driveway and Kroger Driveway
- **Alternative 3** – Southbound through lane to westbound on-ramp
- **Alternative 4** – Install signal coordination system hardware and upgrade signal timing/phasing

4.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Improve Westbound Ramp Terminus

Traffic and Safety – According to the existing conditions level of service analysis, the Bardstown Road / I-265 Westbound off-ramp intersection operates acceptably in the AM peak period. The intersection also operates acceptably in the PM peak period as a whole, but the westbound movement operates at LOS E. A review of the HCS output file showed that the westbound left turn is the movement that is causing the westbound movement to operate at LOS E. An option to improve traffic flow for this movement would be to widen the ramp to allow for dual left turn lanes. The left turn volume is low in the AM (186 vehicles per hour), but is much higher during the PM peak period (608 vehicles per hour). With the PM peak hour turning volume exceeding 300 vehicles per hour (a general guidance threshold); the volumes may warrant dual left turn lanes. To evaluate how the intersection would operate with this improvement, levels of service and delay values were calculated (refer to Table 4-4).

Table 4-4: Alternative 1 Level of Service and Delay Comparison for Bardstown Road / I-265 Westbound Ramps

<table>
<tr><th rowspan="2">Intersection</th><th rowspan="2">Scenario</th><th rowspan="2">Approach</th><th colspan="2">AM</th><th colspan="2">PM</th></tr>
<tr><th>Avg. Delay</th><th>LOS</th><th>Avg. Delay</th><th>LOS</th></tr>
<tr><td rowspan="6">Bardstown Road / I-265 WB Ramps</td><td rowspan="2">Existing</td><td>WB Left</td><td>29.1</td><td>C</td><td>69.0</td><td>E</td></tr>
<tr><td>Whole Int.</td><td>20.8</td><td>C</td><td>33.7</td><td>C</td></tr>
<tr><td rowspan="2">Dual Left Turn Lanes</td><td>WB Left</td><td>27.0</td><td>C</td><td>33.0</td><td>C</td></tr>
<tr><td>Whole Int.</td><td>20.6</td><td>C</td><td>24.7</td><td>C</td></tr>
<tr><td rowspan="2">Conversion of Right Turn Lane to Shared Left and Right Turn</td><td>WB Left</td><td>29.1</td><td>C</td><td>32.2</td><td>C</td></tr>
<tr><td>Whole Int.</td><td>20.8</td><td>C</td><td>26.9</td><td>C</td></tr>
</table>

According to Table 4-4, the addition of a left turn lane does improve the level of service and reduces delay for the westbound left turn as well as for the entire intersection. To provide adequate storage, the left turn lanes should extend back from the intersection for a minimum distance of 500 feet.

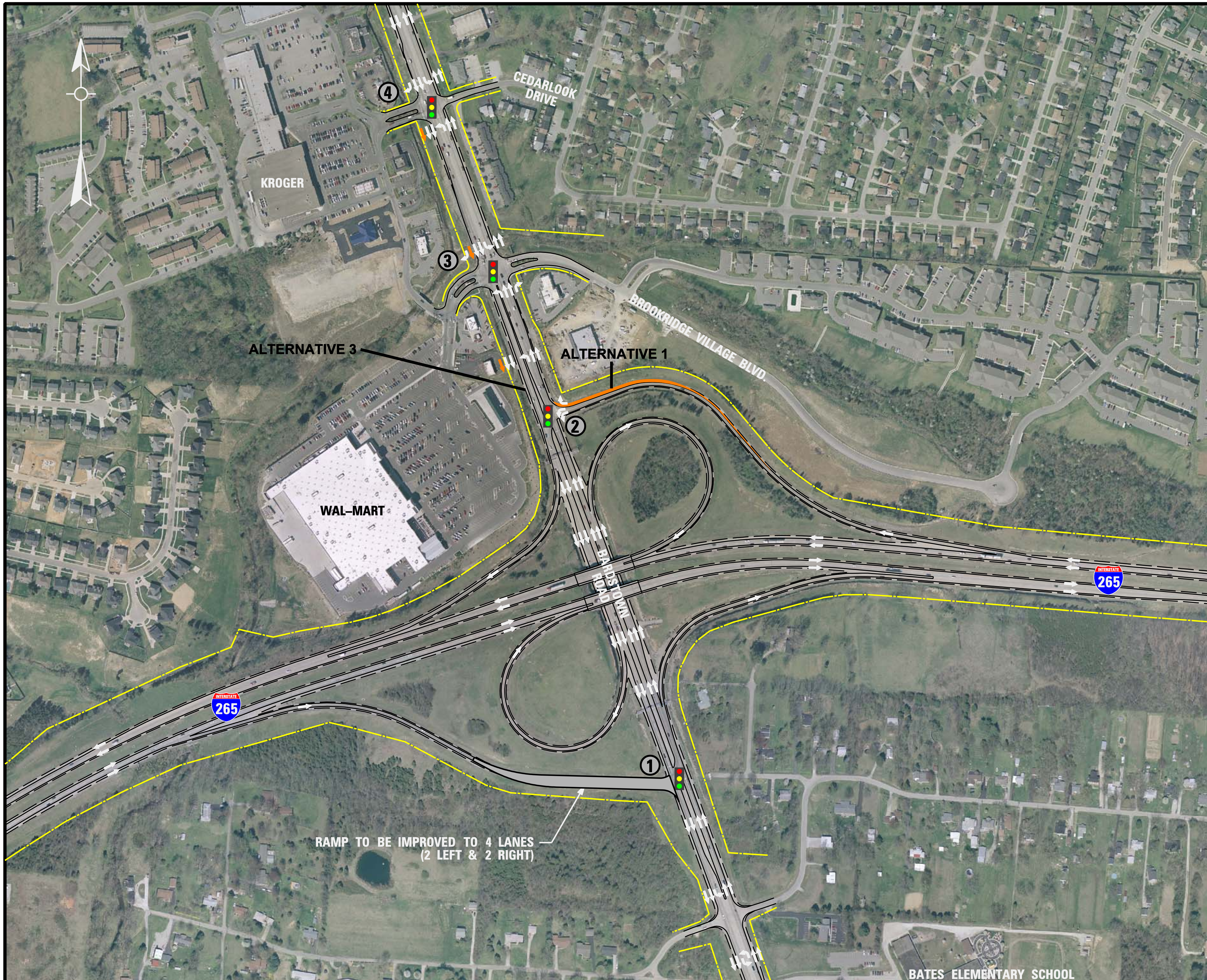


FIGURE 4-2: I-265 & BARDSTOWN ROAD (US 31E) INTERCHANGE

ALTERNATIVES

- Alt. 1 - Improve WB Ramp Terminus by Constructing Dual Left Turn Lanes
- Alt. 2 - Intersection Improvements at Bardstown Road / Wal-Mart Driveway and Kroger Driveway Through Signal Timing Modification
- Alt. 3 - SB Through Lane to WB On-Ramp
- Alt. 4 - Install Signal Coordination System Hardware and Upgrade Signal Timing / Phasing

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

Because of the topography and potential right-of-way constraints imposed by nearby development, construction of an additional left turn lane may be difficult. One option to limit the amount of new construction would be to convert the existing right turn lane to a left turn lane and add a new right turn lane that would preferably be long enough to store the maximum right-turn queue which is 280 feet. However, left turn queues may block access to the new right turn pocket during peak periods. The level of service would still be acceptable and would be similar to the option with the full new lane.

If it is determined that any new construction is not feasible, intersection operations could still be improved by converting the existing right turn lane to a shared right and left. The left turn should operate acceptably as would the whole intersection as shown in Table 4-4. However, this is the least desirable of the three options.

Community / Environmental Impacts – Construction of an additional turn lane on this ramp may be somewhat limited due to development and physical restraints. North of the ramp is a Cracker Barrel restaurant and to the south is a rock cut. Both of these features limit the available land for expansion of the roadway.

Costs – The initial order of magnitude cost estimate for the addition of a second left turn lane is approximately $250,000 in year 2005 dollars. To convert the existing right-turn lane to a left turn lane and add a right-turn pocket, the approximate cost is $100,000 in year 2005 dollars. The estimated cost of converting the right-turn lane to a shared left-right lane is $30,000 in 2005 dollars.

Alternative 2 – Intersection improvements at Bardstown Road / Wal-Mart Driveway and Kroger Driveway

Traffic and Safety – The existing conditions analysis showed that the intersection of Bardstown Road / Wal-Mart Driveway has a poor level of service for the eastbound movement during the AM peak period and poor levels of service for all movements except the westbound approach during the PM peak period. Modifying the signal phasing and adjusting the signal timing does improve intersection operations and levels of service to at or near the acceptable LOS D threshold. However, the queuing issues for the eastbound right and left movements and the northbound left turn movements will not likely be fixed through modification to the signal operations. The turn lanes for these movements could be increased to try to provide adequate storage, but for these movements this is not recommended. The eastbound approach is the Wal-Mart driveway and is not KYTC's responsibility. The northbound left turn lane is already extended back to the I-265 westbound ramp intersection and constructing a second left turn lane to reduce queue lengths is not practical given that there is not an adequate two lane roadway to turn into and there is little room on Bardstown Road for adding a second turn lane.

The Bardstown Road / Kroger Driveway intersection operates acceptably with the existing configuration; however there are queuing issues for several of the movements. In particular, existing queue lengths exceed the available storage for the eastbound left turn, eastbound right turn, and the westbound left and right turn. As was the case with the Wal-Mart Driveway intersection, it is not recommended that turn lengths be increased for these movements. These eastbound queues cause internal site circulation issues, not level of service issues and extension of these turn lanes is not necessarily KYTC's responsibility. The westbound volumes are very low (the maximum is 42 vehicles per hour) therefore lengthening these turn lanes is not necessary.

Community / Environmental Impacts – There are no adverse impacts associated with this alternative.

Costs – The estimated cost for this alternative is minimal.

Alternative 3 – Southbound Through Lane to Westbound On-Ramp

Traffic and Safety – This alternative was proposed to reduce congestion between the Bardstown Road / Wal-Mart Driveway intersection and the Bardstown Road / I-265 Westbound ramps intersection. It would actually begin at the Kroger Driveway intersection and continue through the Wal-Mart Driveway intersection where it would continue to become the I-265 Westbound on-ramp. Using Synchro 6.0 to evaluate the system operations, this improvement primarily benefits operations at the Bardstown Road / Wal-Mart Driveway and Bardstown Road / I-265 Westbound ramp intersections. With this improvement, both intersections operate at or above the desirable LOS C threshold, greatly reducing delay and congestion on this portion of Bardstown Road. Therefore, this project is recommended to improve the traffic operations at this location. To ensure maximum benefit, the signal timing for the Wal-Mart Driveway intersection should be adjusted to reflect the new lane addition.

Community / Environmental Impacts – It is not expected that additional new right-of-way will be required for this alternative; however an urban section with appropriate drainage may be required.

Costs – The estimated order of magnitude cost for this alternative is $600,000 in year 2005 dollars.

Alternative 4 – Install Signal Coordination System Hardware and Upgrade Signal Timing / Phasing

Traffic and Safety – In order to improve traffic flow through the interchange and the intersections immediately to the north, upgraded coordination of the four intersections was proposed as a possible alternative. The intersections are currently part of a time-based coordination system. An evaluation comparing the current coordination plan to a new "optimized" plan showed a moderate system wide delay reduction (<5% in the AM peak and 5-10% in the PM peak). Overall, the current coordination plan appears to be working fairly well. However, it is important to continue to maintain and update any signal system; therefore, it may be beneficial to pursue a true interconnected signal system, with remote operating capabilities. A demand responsive system with multiple peak and off-peak plans may prove even more effective as traffic continues to grow in the corridor. This could include the installation of additional (through lane) loops, new communications to interconnect the signals and communicate with a central system probably at District 5, new signal software and upgraded controllers. Overall, an upgrade of the current signal equipment is recommended to provide the best traffic control for this high demand corridor. However, given that the existing coordination is working well at present, it is reasonable to pursue this project in conjunction with other future improvement projects at this interchange.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $50,000 – $100,000 in year 2005 dollars. The cost would depend on factors such as the sophistication of the new system, how much of the existing hardware is useable, and how the signals were interconnected.

4.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed I-265 / Bardstown Road Interchange alternatives is provided in Table 4-5.

Table 4-5: I-265 / Bardstown Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Improve WB Ramp Terminus – Dual Left Turn Lanes	●	●	●	◗	○	◗	**YES**
2	Improve Bardstown Road / Wal-Mart Driveway and Kroger Driveway Intersections	◗	○	○	◗	◗	●	**NO**
3	New SB Through Lane to WB On-Ramp	●	●	●	◗	◗	○	**YES**
4	Install Signal Coordination System Hardware and Upgrade Signal Timing / Phasing	◗	◗	●	○	●	●	**YES** (as part of another project)

Ratings Guide: ○= Poor ◗= Fair ● = Good

4.6 RECOMMENDATION AND PHASING

The following projects are recommended projects to improve traffic flow and safety though the I-265 / Bardstown Road interchange.

- Construct a second westbound left turn lane on the I-265 Westbound Off-Ramp / Bardstown Road intersection. Both turn lanes should extend back from the intersection a minimum distance of 500 feet. If this is not feasible, then the right turn lane should be converted to a left turn lane and a right turn pocket should be constructed as far back as possible to accommodate peak period queues. If no new construction is determined to be feasible in the near term at this location, then the last recommendation would be to convert the right turn lane to a shared right and left turn lane. While this would not solve the queuing issue, it would improve the level of service for this movement to a desirable level.

- Construct a new southbound through lane beginning at the Bardstown Road / Kroger Driveway intersection to the I-265 westbound on-ramp, diverging from Bardstown Road to become the on-ramp. This will improve intersection operations for the Bardstown Road / Wal-Mart Driveway intersection to a desirable LOS threshold and should in general improve congestion in this area. The traffic signal at the Bardstown Road / Wal-Mart Driveway intersection should be retimed to account for the added through lane.

- Upgrade the current signal coordination from time-based coordination to a full interconnected signal system (dynamic if possible). This is not an immediate need and could be completed as part of another improvement project on Bardstown Road.

5.0 I-265 / TAYLORSVILLE ROAD (KY 155) INTERCHANGE

5.1 INTRODUCTION AND STUDY AREA

The study area for the I-265 / Taylorsville Road (KY 155) interchange consists of the intersections listed below. Refer to Figure 5-1 for the limits of the study area.

1. Taylorsville Road (KY 155) / I-265 Northbound Ramps
2. Taylorsville Road (KY 155) / I-265 Southbound Ramps
3. Taylorsville Road (KY 155) / Stone Lakes Drive / St. Michael's Church Drive

5.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns

The current average daily traffic volumes for I-265 came from the Highway Information System (HIS) database, and are listed below.

- Approximately 12,300 ADT on Taylorsville Road west of I-265 (2% trucks)
- Approximately 17,500 ADT on Taylorsville Road east of I-265

Major traffic flows through the interchange are from east to west in the AM peak period and west to east during the PM peak period. Major traffic flows on the ramps are between the north and west. It should also be noted that in the morning a police officer is assigned to control traffic for the St. Michael Catholic School, just west of the interchange at the Taylorsville Road / Stone Lakes Drive intersection.

Geometrics / Right-of-way

An evaluation of the existing interchange features revealed the following:

- Diamond interchange with single lane entrance and exit ramps
- Both ramp junctions are signalized; no other signals exist in the immediate area
- Exit ramps provide separate left and right turn lanes at Taylorsville Road
- Taylorsville Road is a four-lane divided highway in the vicinity of I-265, narrowing to two through lanes east and west of the interchange

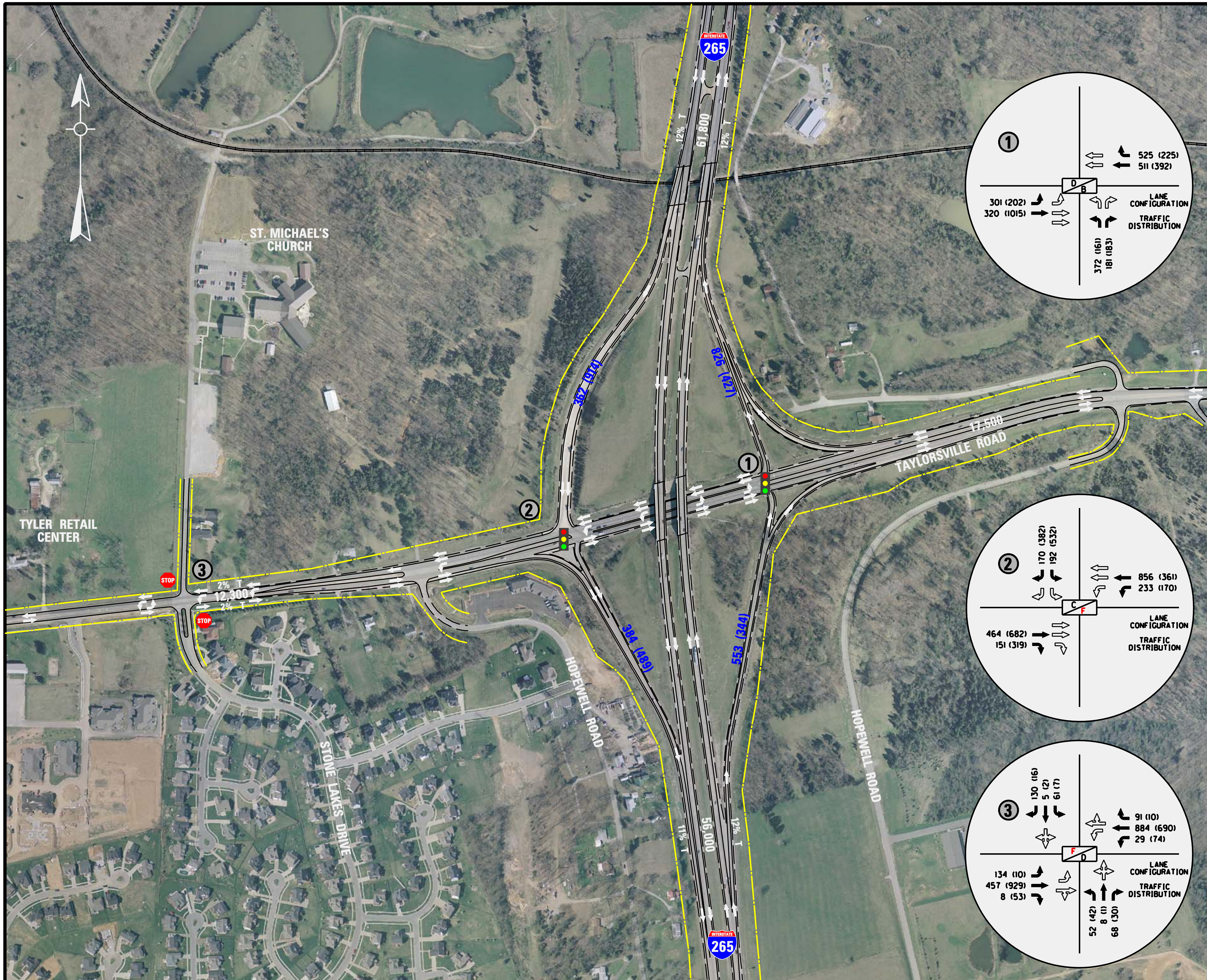
Land Use, Future Development, and Historic Traffic Growth

Land use in the vicinity of the interchange is predominately residential, with some commercial / institutional uses as well. A new development (Tyler Retail Center) is proposed north of Taylorsville Road, near Stone Lakes Drive. Projected uses for the development include a supermarket, bank, restaurants, and a gas station. As a result, approximately 400 trips may be added near the interchange. A review of historic traffic data showed that traffic has been growing at the following rates since 1985:

KY 155 East of I-265	KY 155 West of I-265	I-265 South of Taylorsville Road	I-265 North of Taylorsville Road
6%	3%	5%	5%

As shown above, traffic in the vicinity of this interchange has been growing at a steady rate and will likely continue to increase, especially with the continued residential development to the east (such as in Spencer County).

FIGURE 5-1: I-265 & TAYLORSVILLE ROAD (KY 155) INTERCHANGE



KEY ISSUES / DEFICIENCIES

- Poor levels of service at ramp intersections could be related to signal timing
- Poor levels of service at Taylorsville Road / Stone Lakes Drive intersection
- Proposed new Retail Development west of interchange and strong traffic growth in the area

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP STOP-CONTROLLED INTERSECTION
- 67,800 2004 AVERAGE DAILY TRAFFIC
- 980 (1080) 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T PERCENT TRUCKS
- B E 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600

GRAPHIC SCALE IN FEET

Traffic Operations / Level of Service Analysis

AM peak period turning movement counts were conducted in October and November 2004. PM peak period turning movement counts were provided in the Tyler Retail Center Study (BTM, 2004). For each of the key intersections, AM and PM peak hour volumes are shown on Figure 5-1. Existing levels of service (LOS) and delay based on the Highway Capacity Manual method are shown in Table 5-1 below.

Table 5-1: 2004 Intersection LOS for I-265 / Taylorsville Road Interchange

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Taylorsville Road / I-265 NB Ramps	Signalized	Eastbound	8.4	A	9.2	A
		Westbound	26.1	C	25.1	C
		Northbound	**116.3**	**F**	47.8	D
		Whole Int.	**41.3**	**D**	**16.3**	**B**
Taylorsville Road / I-265 SB Ramps	Signalized	Eastbound	25.9	C	28.2	C
		Westbound	8.7	A	8.2	A
		Southbound	53.3	D	**228.0**	**F**
		Whole Int.	**21.5**	**C**	**102.4**	**F**
Taylorsville Road / Stone Lakes Drive	STOP Controlled on Stone Lakes Drive	Eastbound Left	12.4	B	9.0	A
		Westbound Left	8.9	A	11.2	B
		Northbound	*****[1]	**F**[1]	33.7	D
		Southbound	**578.6**[1]	**F**[1]	21.4	C

[1]The LOS and delay shown here are those computed by HCS based on the AM peak volumes. The actual LOS for the side streets is better during the AM peak period because traffic is controlled by a police office during the peak periods for school traffic.

As shown in Table 5-1, the Taylorsville Road / Stone Lakes Drive intersection operates poorly in the morning when school is in session. This is the primary reason a police officer is there to control traffic. Other intersections that were shown to operate poorly include the northbound exit ramp during the AM peak period and the southbound exit ramp during the PM peak period.

An analysis of queue lengths in HCS showed that during the AM peak period, queues extended past the available storage for the northbound left-turn at the northbound I-265 ramp intersection and the southbound movement for the Taylorsville Road / Stone Lakes Drive intersection. During the PM peak period, queues extended back beyond the available storage for the southbound left turn movement at the southbound I-265 ramp intersection. For purposes of this analysis, the available storage for each turn lane was assumed to extend back on the ramp to the point where the width was less than 18 feet or where two cars could not pass.

Field observations at the Taylorsville Road / Stone Lakes Drive intersection showed that while a police officer was at the intersection controlling traffic, traffic headed westbound on Taylorsville Road generally backed up from this intersection toward Hopewell Road. Some queuing was observed at the ramp intersections, but not to the extent indicated by HCS. The HCS 95th percentile queue lengths are sometimes conservative estimates.

Safety / Crash Analysis
The crash analysis did not show a crash rate problem in the study area (see Table 5-2 below).

Table 5-2: 2001 – 2003 Crash Analysis for I-265 / Taylorsville Road Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-265	16	4	0	25	74	118	0.22
KY 155	28	10	0	124	332	369	0.34

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies
Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor levels of service and queuing issues at the I-265 Northbound Ramps / Taylorsville Road intersection during the AM peak period
- Poor levels of service and queuing issues at the I-265 Southbound Ramps / Taylorsville Road intersection during the PM peak period
- Poor levels of service at Taylorsville Road / Stone Lakes Drive intersection during the AM peak period (Currently this is mitigated by a police officer controlling traffic during the arrival / departure period for school traffic.)
- Proposed new Retail Development west of interchange and strong traffic growth in the area

5.3 ALTERNATIVES

- **Alternative 1** – Signalize Taylorsville Road / Stone Lakes Drive / St. Michael's Church Drive intersection in conjunction with the new development as proposed in the Birch, Trautwein and Mims, Inc. Tyler Retail Center traffic study. Also included as part of the signal analysis is the evaluation of geometric improvements to Taylorsville Road and St. Michael's Church Drive as proposed in the BTM study. These improvements include:
  - Add Southbound left turn lane on St. Michael's Church Drive at Taylorsville
  - Add Westbound right turn lane on Taylorsville at St. Michael's Church Drive
  - Add 2nd Eastbound thru lane on Taylorsville from west of study area to existing 5-lane section
- **Alternative 2** – Improve Southbound I-265 Ramp / Intersection Operations
- **Alternative 3** – Improve Northbound I-265 Ramp / Intersection Operations
- **Alternative 4** – Install signal coordination system (possibly demand responsive) with multiple timing plans.

Refer to Figure 5-2 for a graphical representation of these alternatives.

FIGURE 5-2: I-265 & TAYLORSVILLE ROAD (KY 155) INTERCHANGE

ALTERNATIVES

- Alt. 1 - Install Traffic Signal at Taylorsville Road / Stone Lakes Drive (Int. 3) along with the following improvements:

  Construct Exclusive WB Right and SB Left Turn Lanes

  Construct 2nd EB Thru Lane

- Alt. 2 - Optimize Signal Timing for I-265 SB Ramps / Taylorsville Road Intersection and Construct Dual Left Turn Lanes
- Alt. 3 - Optimize Signal Timing for I-265 NB Ramps / Taylorsville Road Intersection
- Alt. 4 - Install Signal Coordination System (Possibly Demand Responsive)

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

5.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Install Traffic Signal at Taylorsville Road / Stone Lakes Drive

Traffic and Safety

Level of Service Analysis – As shown in the existing conditions analysis, both the northbound and southbound movements at this unsignalized intersection currently operate at LOS F in the AM peak period. These same movements operate better in the PM peak period (LOS D and C). Using the same traffic volumes and lane configurations, the intersection was analyzed with a new signal. The intersection was also analyzed assuming two stages of improvements based on recommendations by BTM. The first is construction of turn lanes including a southbound left turn lane on St. Michael's Church Drive and a westbound right turn lane on Taylorsville Road to St. Michael's Church Drive. The second includes the previous improvements plus an additional eastbound through lane. The levels of service for these different scenarios are shown in the following table (Table 5-3).

Table 5-3: Alternative 1 Level of Service and Delay Comparison for Taylorsville Road / Stone Lakes Drive

			AM		PM	
Intersection	**Scenario**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Taylorsville Road / Stone Lakes Drive	Unsignalized	NB	*	F[1]	33.7	D
		SB	578.6	F[1]	21.4	C
	Signalized – Existing Configuration	**Whole Int.**	**40.1**	**D**	**32.5**	**C**
	Signalized – WB Right Turn and SB Left Turn Lane	**Whole Int.**	**27.7**	**C**	**24.8**	**C**
	Signalized – Previous Improvements Plus 2nd EB Thru Lane	**Whole Int.**	**27.4**	**C**	**13.5**	**B**

[1] The LOS and delay shown here are those computed by HCS based on the AM peak volumes. The actual LOS for the side streets is better during the AM peak period because traffic is controlled by a police office during the peak periods for school traffic.

Signalization of the intersection improves traffic operations, particularly in the AM peak. However, the level of service (LOS D) is still below the preferred threshold of LOS C. With the proposed westbound right turn and southbound left turn lanes, the intersection operates at LOS C during both peak periods. With all movements operating at LOS C or better, it is likely that police control will no longer be needed to maintain good traffic operations.

Adding a second eastbound through lane has little impact on the AM peak period, but does further improve the LOS during the PM peak. The second eastbound through lane is not necessary to achieve a good level of service for this intersection, but was proposed by BTM in order to improve traffic flow for intersections to the west where the main entrance to the development would be located. Since it was part of the proposed development plan, it was included as part of this analysis.

Queue Length Analysis – As mentioned in the existing conditions analysis, the southbound movement at this intersection experiences queues that exceed the available storage during the AM peak period. Queues were also observed in the westbound direction on Taylorsville Road due to police control of the intersection (to let school traffic exit in a reasonable amount of time). According to the HCS method, installation of a traffic signal resolves this queuing issue such that queues do not exceed the available storage.

Signal Warrant Analysis – A traffic signal warrant evaluation was performed to determine if the intersection meets or exceeds any of the signal warrants as outlined in the Manual of Uniform Traffic Control Devices (MUTCD). Of the four of warrants that are most relevant to this intersection, at least two are met as outlined below.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Only four hours of data was collected during the original traffic count, therefore there is insufficient data to determine if the 8-hour warrant is met.
- Warrant 2: Four-Hour Vehicular Volume – For this analysis, Stone Lakes Drive was considered to be the minor street with Taylorsville Road as the major street. The four hours of data obtained during the AM and PM traffic counts were used as the basis for this warrant analysis. Figure 4C-2 in the MUTCD was used as the threshold curve. Because the speed on Taylorsville through this intersection exceeds 40 mph, the reduced warrant was used. The traffic volumes for all four hours plotted above the threshold curve. **Based on these traffic volumes, this warrant is currently met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must exceed the threshold curve shown on Figure 4C-4 in the MUTCD. From the traffic count data, the highest peak hour is from 7-8 AM. The traffic volumes during this hour plot above the threshold curve. **This warrant is satisfied.**
- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection. Based on the crash rate analysis, there are no known crash problems on Taylorsville Road; **therefore this warrant is not met**.

According to the level of service, queuing, and signal warrant analysis, installation of a traffic signal at the intersection of Taylorsville Road / Stone Lanes Drive / St. Michael's Church Drive is recommended. Installation of the signal would solve the poor levels of service during the AM peak period and would not require a police office to be present to direct traffic during the beginning of school. It would also solve the southbound queuing problem during the AM peak period. To provide optimal intersection operations, the improvements proposed in the BTM study should also be completed as part of the signal installation which includes construction of a southbound left turn lane, a separate westbound right turn lane, and a second eastbound through lane. Installation of the signal and the improvements should be completed as part of (and should be funded by) the proposed development.

Community / Environmental Impacts – These improvements are proposed in conjunction with the proposed Tyler Retail Center development project. Any environmental issues would be addressed by the entity responsible for constructing the improvements. There are no known community impacts.

Costs – These improvements were proposed in conjunction with the Tyler Retail Center development and would be funded by the developer.

Alternative 2 – Improve Southbound I-265 Ramp / Intersection Operations

Traffic and Safety – The existing levels of service calculated for the southbound I-265 ramp intersection showed that the left-turn movement operates poorly during the PM peak period. The first option analyzed to improve traffic operations was signal optimization. The resulting levels of service for both the existing signal timing and the optimized signal timing are shown in Table 5-4.

Table 5-4: Alternative 2 Level of Service and Delay Comparison for Taylorsville Road / I-265 Southbound Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Taylorsville Road / I-265 SB Ramps	Existing	Eastbound	25.9	C	28.2	C
		Westbound	8.7	A	8.2	A
		Southbound	53.3	D	**228.0**	**F**
		Whole Int.	**21.5**	**C**	**102.4**	**F**
	Signal Optimized	Eastbound	13.4	B	32.7	C
		Westbound	7.3	A	20.6	C
		Southbound	27.8	C	30.4	C
		Whole Int.	**12.7**	**B**	**28.6**	**C**
	Signal Optimized and Dual Left Turns	Eastbound	13.4	B	32.7	C
		Westbound	7.3	A	20.6	C
		Southbound	22.5	C	23.0	C
		Whole Int.	**11.7**	**B**	**25.6**	**C**

Optimization of the signal timing for this ramp intersection improves all movements to LOS C or better, which meets or exceeds the desirable LOS C threshold. Signal optimization also reduces the southbound left queue such that it is less than the available storage, but this is assuming the available vehicle storage extends past the striped lanes to where the ramp narrows down below eighteen feet (where two cars cannot pass). The actual striped turn lane is approximately 300 feet in length, and queue length for the southbound left turn is 780 feet which does exceed the striped lane. Therefore, either the turn lane should be extended to account for this queue, or a dual left turn lane should be considered.

Based on traffic volumes, dual southbound left turn lanes are a reasonable alternative since the PM peak period left turning traffic is 532 vehicles per hour. Typically dual left turn lanes are considered if the peak hour volumes exceed 300 vehicles per hour which in this case they do. Analyzing intersection operations with dual southbound left turn lanes shows that the levels of service are at or above the desirable LOS C threshold. The queue length is reduced from 780 feet to 335 feet.

To maximize intersection operations and accommodate the heavy left turn flow and lengthy queues, the signal timing should be optimized and dual left turn lanes should be constructed. To accommodate the peak hour queues, they should extend back from the intersection for a minimum distance of 350 feet, but could be extended further if possible. These improvements will also be beneficial in the future as traffic volumes through the interchange continue to grow.

Community / Environmental Impacts – It is expected that these improvements can be made within the existing right-of-way, with few if any environmental impacts.

Costs – The estimated order of magnitude cost for this alternative is $260,000 in year 2005 dollars.

Alternative 3 – Improve Northbound I-265 Ramp / Intersection Operations

Traffic and Safety – The existing levels of service calculated for the I-265 Northbound Ramp intersection showed the northbound left turn movement operates poorly during the AM peak period. Signal timing optimization was examined to determine if this would improve the level of service to a desirable level. The resulting levels of service for both the existing signal timing and the optimized signal timing are shown in Table 5-5.

Table 5-5: Alternative 3 Level of Service and Delay Comparison for Taylorsville Road / I-265 Northbound Ramps

			AM		PM	
Intersection	**Scenario**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Taylorsville Road / I-265 NB Ramps	Existing	Eastbound	8.4	A	9.2	A
		Westbound	26.1	C	25.1	C
		Northbound	**116.3**	**F**	47.8	D
		Whole Int.	**41.3**	**D**	**16.3**	**B**
	Signal Optimized	Eastbound	11.8	B	8.0	A
		Westbound	22.7	C	14.7	B
		Northbound	34.0	C	28.2	C
		Whole Int.	**21.0**	**C**	**11.3**	**B**

Optimization of the signal timing improves all movements to LOS C or better, which meets or exceeds the desirable LOS C threshold. The existing queue length for the northbound left was found to exceed the available vehicle storage for the existing signal timing, sometimes hindering right turning vehicles from using the exclusive turn lane at the end of the ramp. Signal optimization reduces the northbound left queue to be equal to the available storage length. For this intersection the northbound left turn volume in the PM peak period is 372 vehicles per hour. This exceeds the typical threshold of 300 vehicles per hour for consideration of dual left turn lanes, however, because signal optimization improves intersection operations to a desirable level and the queue length does not yet exceed the available storage, dual left turn lanes are not necessary at this time. Instead, the most reasonable and cost-effective option would be to optimize the signal timing and monitor traffic volumes on this ramp. As traffic volumes increase, levels of service may decline and queue lengths may increase such that the left turn queue blocks right-turning traffic. At that time either extension of the left turn lane or dual left turn lanes should be considered.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The cost for improving the timing at this intersection is minimal.

Alternative 3 – Install Signal Coordination System

Traffic and Safety – Installation of a coordinated signal system at this interchange with multiple "time-of-day" timing plans appropriate for current peak and off-peak traffic flows could benefit overall traffic operating conditions. This could help maximize progression through the interchange intersections. An initial assessment of the benefit of using a coordinated system at this location showed modest delay reduction (<5%) over simply optimizing the signal timing. However, given the challenges of maintaining optimal signal timing and maximizing progression through a series of interchange intersections due to annual, monthly, weekly, daily, and even hourly fluctuations in traffic flows, it may be beneficial to consider a demand responsive coordinated signal system. With this type of system, as traffic flows increase, decrease, or shift, the system can switch to the most appropriate timing plan. A demand responsive system would require the installation of at least five new loop detectors (in addition to those that are already present) to monitor traffic flows as well as improved hardware and software and a link to District 5 and Central Office. This could be a reasonable, small system for testing such a system.

As shown, optimizing the timing at the two intersections will result in important benefits to the traveling public. Use of a demand-responsive coordinated signal system could be used to further enhance these gains, as well as to facilitate continued good operations in the future. Furthermore, as traffic flows increase in the future, the demand responsive coordinated system could offer timing plan flexibility without the need to conduct field traffic counts or change timing plans in the field. Overall, this may be a good opportunity for this type of system.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $40,000 – $90,000 in year 2005 dollars. The exact cost depends on the type of system installed and the amount of new hardware and software (both in the field and in the office). Some of the cost will be loop cuts that will need to be made in the roadway with loop wire installed. Shielded loop wire would likely need to be installed from the loops to the controller cabinet (unless a wireless system is employed). Other possible signal system elements could include a loop amp detector in the controller cabinet, software, communications equipment, new controller and/or computer equipment (if necessary), pull boxes, and conduit.

5.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed I-265 / Taylorsville Road Interchange alternatives is provided in Table 5-6.

Table 5-6: I-265 / Taylorsville Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Install Traffic Signal at Taylorsville Road / Stone Lakes Drive with Intersection Improvements	●	●	◗	●	●	◗	**YES (To Be Completed by Developer)**
2	I-265 SB Ramp / Intersection Improvements - Signal Optimization and Dual Southbound Left Turn Lanes	●	●	◗	◗	●	◗	**YES**
3	I-265 NB Ramp / Intersection Improvements - Signal Optimization	●	●	◗	◗	●	●	**YES**
4	Install Signal Coordination System	○	◗	◗	◗	●	●	**YES** (Possibly longer term)

Ratings Guide: ○ = Poor ◗ = Fair ● = Good

5.6 RECOMMENDATION AND PHASING

Several improvements are recommended for the I-265 / Taylorsville Road interchange area. The BTM recommendations for the St. Michael's Church Drive intersection in the Tyler Retail Center study should be completed as part of the proposed development. In addition to a signal, the improvements include a westbound right turn lane on Taylorsville, a southbound left turn lane on St. Michael's Church Drive, and a second eastbound through lane on Taylorsville Road. For the I-265 Southbound Ramp / Taylorsville Road intersection, construction of a second left turn lane is recommended along with signal timing optimization. The left turn lanes should be extended back from Taylorsville Road a minimum distance of 350 feet to provide adequate vehicles storage length. At this time, only signal timing optimization is recommended for the I-265 Northbound Ramp / Taylorsville Road intersection. However, it is recommended that traffic volumes be monitored on this ramp and lengthening the northbound left turn lane or dual left turn lanes be considered in the future if traffic volumes increase substantially. A signal system could also be considered for this interchange to help provide good operations in the future as traffic volumes continue to grow.

6.0 I-265 / OLD HENRY ROAD (KY 3084) INTERCHANGE

6.1 INTRODUCTION AND STUDY AREA

The study area for the I-265 / Old Henry Road (KY 3084) interchange consists of the intersections listed below. Refer to Figure 6-1 for the limits of the study area.

1. Old Henry Road (KY 3084) / Nelson Miller Parkway
2. Old Henry Road (KY 3084) / I-265 Southbound Ramps
3. Old Henry Road (KY 3084) / I-265 Northbound Ramps

6.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns

The current average daily traffic volume for Old Henry Road is approximately 9,100 according to the Highway Information System (HIS) database. Through the interchange, the AM peak traffic flow is from the east on Old Henry Road to the west and south. The reverse is true for the PM peak, with the heaviest traffic flow from the south to the east with approximately 700 vehicles turning right from the northbound exit ramp. Just west of the interchange is the intersection of Old Henry Road / Nelson Miller Parkway which leads into a business park. As development occurs, traffic volumes going through this intersection are likely to increase.

Geometrics / Right-of-way

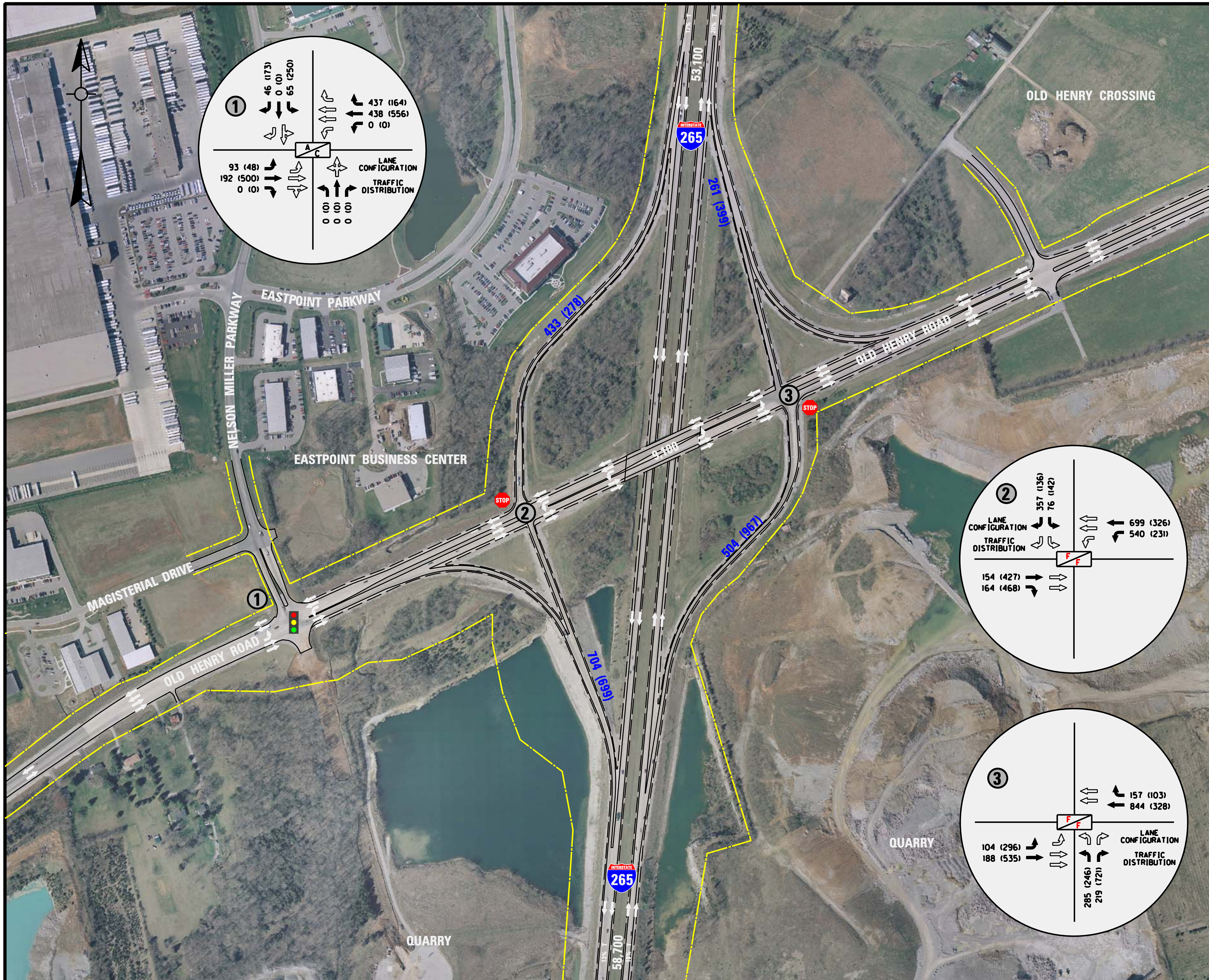
An evaluation of the existing interchange features revealed the following:

- Interchange is a typical diamond without traffic signals at the ramp intersections (during this study a new traffic signal was installed at the I-265 Northbound Ramps intersection with Old Henry Road)
- Old Henry Road / Nelson Miller Parkway intersection to the west is signalized
- Old Henry Road is a four lane divided highway with turn lanes, narrowing to 2 lanes at either end of the state maintained portion of the highway
- Interchange is the west end of proposed Old Henry Road–Crestwood Connector
- Posted speed limit through the interchange is 45 mph

Land Use, Future Development, and Historic Traffic Volumes

Some land uses in the vicinity of the interchange include the Eastpoint Business Center northwest of the interchange and a quarry southwest and southeast of the interchange. Significant developable land is located east of the interchange, including the proposed 120 acre Old Henry Crossing development, with over 1 million square feet of proposed office and retail development as well as residential and hotel components. An analysis of historic traffic volumes showed that since 1987, traffic has grown at approximately 6 to 7 percent per year on I-265. There was insufficient data to calculate a growth rate for Old Henry Road, but based on the current development rate, traffic volumes may increase substantially in the future.

FIGURE 6-1: I-265 & OLD HENRY ROAD (KY 3084) INTERCHANGE



KEY ISSUES / DEFICIENCIES

- Poor levels of service on both STOP controlled ramp approaches
- Queuing on NB ramp
- Drivers exceeding the posted speed limit on Old Henry Road
- Expected future development and associated traffic growth

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP-CONTROLLED INTERSECTION
- 67,800 2004 AVERAGE DAILY TRAFFIC
- 980 (1080) 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T PERCENT TRUCKS
- B/E 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600
GRAPHIC SCALE IN FEET

Traffic Operations / Level of Service Analysis

Both AM and PM peak period turning movement counts were provided by KYTC. For each of the key intersections, AM and PM peak hour volumes are shown on Figure 6-1. Existing levels of service and delay based on these volumes are shown in Table 6-1.

Table 6-1: 2004 Intersection Levels of Service for I-265 / Old Henry Road

			AM		PM	
Intersection	**Type**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Old Henry Road / Nelson Miller Pkwy	Signalized	**Whole Int.**	**5.9**	**A**	**34.6**	**C**
Old Henry Road / I-265 SB Ramps	STOP Controlled on Ramp	WB Left	9.7	A	9.9	A
		SB Left	*	F	227.5	F
		SB Right	21.1	C	10.5	B
Old Henry Road / I-265 NB Ramps	STOP Controlled on Ramp	EB Left	11.7	B	9.2	A
		NB Left	502.4	F	*	F
		NB Right	10.5	B	147.0	F
Old Henry Road / I-265 NB Ramps	Signalized	**Whole Int.**	**16.6**	**B**	**31.6**	**C**

According to the HCM method of analysis, the stop controlled approaches to both ramp intersections operate poorly during the peak periods. Field observations indicate that this may be a somewhat conservative assessment; however queues do build, especially on the northbound ramp. In an effort to improve intersection operations at the northbound ramp intersection, KYTC recently approved the installation of a new traffic signal at this location and have subsequently installed the signal. As shown in Table 6-1, this should address the LOS deficiency at that intersection. However, based on queue lengths from HCS output, queue lengths for both the left and right turn movements that exceed that available storage may still exist. In fact, the signal installation may cause the eastbound left turn movement to queue beyond the turn bay length. For the southbound ramp, there are no queues exceeding the available storage.

In addition to the turning movement counts, KYTC also provided a recent speed study performed for Old Henry Road. The study indicated that the 85th percentile speeds west of the interchange were over 50 mph and east of the intersection speeds increased to over 55 mph in one location.

Safety / Crash Analysis

The crash analysis did not reveal a crash rate problem on I-265. For Old Henry Road, inadequate crash data was available to complete a crash rate analysis.

Key Issues / Deficiencies

Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor levels of service on both STOP controlled ramp approaches
- Queuing on the northbound ramp
- Drivers exceeding the posted speed limit on Old Henry Road
- Expected future development and associated traffic growth

6.3 RANGE OF ALTERNATIVES

In response to issues / deficiencies identified in the existing conditions analysis, the following alternatives were developed.

- **Alternative 1** – Install traffic signal at Old Henry Road / I-265 Northbound ramps intersection (already implemented by KYTC)
- **Alternative 2** – Install traffic signal at Old Henry Road / I-265 Southbound ramps intersection

6.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Install Traffic Signal at Old Henry Road / I-265 Northbound Ramps

<u>Level of Service Analysis</u> – As shown in the existing conditions analysis, the northbound left turn movement has a poor LOS (LOS F) in the AM peak period, and both the northbound left and right turns are LOS F in the PM peak period. Using the same traffic volumes and lane configurations, the intersection was analyzed with a signal. The level of service analysis with the signal indicates that the intersection would operate acceptably (LOS B in the AM peak, LOS C in the PM peak).

<u>Queue Length Analysis</u> – An analysis of the existing queue lengths showed that they exceed the current storage for the left turn during the AM peak period, and both the left and right turns during the PM peak period. Field observations have shown that traffic on the ramp does back up during some PM peak periods. With the installation of a traffic signal, queue lengths are reduced but the queue length for the northbound right still exceeds the available storage during the PM peak. However, with the storage available on the ramp prior to the start of the two-lane section there is adequate storage for the right turn movement. Allowing right turns on red will reduce the queues, and impacts to the left turn traffic should be minimal.

<u>Signal Warrant Analysis</u> – As part of this alternative, the MUTCD signal warrants were reviewed. Because the speed limit is higher than 40 mph, the 70% factor threshold volumes were used. As expected, Warrant 1 – Eight Hour Vehicular Volume, Warrant 2 – Four Hour Vehicular Volume, and Warrant 3 – Peak Hour Volumes were all met.

The above analysis confirms the fact that a traffic signal is beneficial and warranted in this location. It will improve intersection operations to a desirable level of service and should reduce queue lengths. With the signal already in place, the only recommendation from this study would be to monitor traffic volumes and queuing issues on the ramp, particularly for the northbound right turn traffic during the PM peak period. While the right turn queue may exceed the available storage during the PM peak period, intersection operations should not be impacted and the left turn traffic should not be impacted significantly. If traffic monitoring reveals that the right turn queue is impacting traffic flow, the turn lanes could be extended back to accommodate this queue.

Alternative 2 – Install Traffic Signal at Old Henry Road / I-265 Southbound Ramps

Traffic and Safety –

Level of Service Analysis – For this intersection, the movement currently experiencing a poor LOS is the southbound left turn movement in both the AM and PM peak periods. Using the same traffic volumes and lane configurations, the intersection was analyzed with a signal. The levels of service for the signalized intersection indicate the intersection would operate acceptably at a LOS C in the AM peak period and LOS B in the PM peak period.

Queue Length Analysis – The existing conditions analysis showed that no queues exceed the available storage at this intersection. If a signal was installed, the queues for both the westbound left turn and the southbound right turn would build during the AM peak period such that they might exceed the available storage.

Signal Warrant Analysis – A traffic signal warrant evaluation was also performed to determine if the intersection meets or exceeds any of the MUTCD signal warrants. The most relevant warrants for this analysis are listed along with a brief definition and a discussion of how they compare to the given conditions. Because the speed limit is higher than 40 mph, the 70% factor threshold volumes were used.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Only four hours of data was collected during the original traffic count, therefore there is insufficient data to determine if the 8-hour warrant is met. If signalization of this intersection is pursued, additional fill-in counts should be collected.
- Warrant 2: Four-Hour Vehicular Volume – For this analysis, the southbound off-ramp approach is the minor street and Old Henry Road is the major street. The four hours of data obtained during the AM and PM traffic counts were used as the basis for the analysis. Figure 4C-2 in the MUTCD was used as the threshold curve. The volumes for all four hours plotted above the threshold curve shown for an intersection with two lanes on both the major and minor approaches. **Based on these traffic volumes, this warrant is currently met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must exceed the threshold curve shown on MUTCD Figure 4C-4. From the traffic count data, the highest peak hour is from 8-9 AM. The traffic volumes during this hour plot above the threshold curve. **Therefore, this warrant is satisfied.**

Installation of a traffic signal is warranted at this location. It would improve intersection operations such that the southbound left turn movement operates above the desirable level of service threshold, and traffic volumes are high enough such that Warrants 2 and 3 are met. However, based on a queue length analysis using HCS output, installing a traffic signal may actually cause queuing problems for the westbound left and southbound right turn movements during the AM peak period where there currently are none. Also, the heaviest turn volume is the right turn traffic (357 vehicles per hour during the AM peak period), for which traffic flow will likely not be improved through installation of a traffic signal. The southbound left turn volumes are much lower compared to the right turn traffic (76 in the AM peak period and 142 vehicles per hour during the PM peak period). Due to the fact that installation of a traffic signal at this location would not serve the heaviest traffic flow and could lead to queuing issues, it is not recommended at this time, but the intersection should continue to be monitored in the future.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for Alternative 2 is $125,000 in 2005 dollars.

6.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A comparison of the alternatives proposed for improvements to the I-265 / Old Henry Road Interchange area are listed in Table 6-2 below.

Table 6-2: I-265 / Old Henry Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Install Traffic Signal at I-265 NB Ramps / Old Henry Road	●	●	●	◗	●	●	**YES (Already Completed by KYTC)**
2	Install Traffic Signal at I-265 SB Ramps / Old Henry Road	●	◗	◗	◗	●	●	**NO**

Ratings Guide: ○ = Poor ◗ = Fair ● = Good

6.6 RECOMMENDATION AND PHASING

The analysis performed for the northbound ramp intersection supports the recent installation of a traffic signal at this location by KYTC. Therefore, the only recommendation for this intersection would be to monitor traffic volumes to ensure the right turn queues do not cause significant delay or block the left turn vehicles on a regular basis. For the southbound ramp intersection, at this time a do-nothing approach is recommended. A traffic signal could be installed at this location, but it could negatively impact traffic flow for the heavy right turn movement and may lead to queuing problems where there currently are none. Traffic volumes should be monitored at this location as well, and if significant changes occur then the intersection may need to be re-evaluated for signalization.

7.0 I-265 / LAGRANGE ROAD (KY 146) INTERCHANGE

7.1 INTRODUCTION AND STUDY AREA

The study area for the I-265 / LaGrange Road (KY 146) interchange consists of the intersections listed below. Refer to Figure 7-1 for the limits of the study area.

1. LaGrange Road (KY 146) / I-265 South Ramps
2. LaGrange Road (KY 146) / I-265 North Ramps
3. LaGrange Road (KY 146) / Factory Lane / Chamberlain Lane

7.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns
The current average daily traffic volumes for I-265 came from the Highway Information System (HIS) database, and are listed below.

- Approximately 17,500 ADT on LaGrange Road north of I-265
- Approximately 7,100 ADT on LaGrange Road south of I-265

Traffic volumes are higher north of the interchange on LaGrange Road based on current ADT volumes. Major flows through the study area are from the north to the south and east in the AM with the reverse flows in the PM.

Geometrics / Right-of-way
An evaluation of the existing interchange features revealed the following:

- This interchange is a partial cloverleaf with all ramps located east of LaGrange Road
- Development west of LaGrange Road is constrained by the railroad. There are over 30 trains per day on this CSX mainline, which blocks the Chamberlain Lane leg of the LaGrange Road / Factory Lane / Chamberlain Lane intersection when a train is present.
- The stop controlled north exit ramp intersection is approximately 400 feet from the LaGrange Road / Factory Lane / Chamberlain Lane intersection

Land Use, Future Development, and Historic Traffic Growth
Major existing traffic generators in the vicinity of the interchange include the Ford Plant to the northwest, retail to the north, and industrial / warehouse / office development to the southeast. In addition, the area has also experienced significant residential growth during the past several years. An analysis of historic traffic volumes showed that traffic grew at the following rates between 1985 and 2004:

KY 146 South of I-265	KY 146 North of I-265	I-265 East of LaGrange Rd	I-265 West of LaGrange Rd
2%	4%	6%	7%

FIGURE 7-1: I-265 & LAGRANGE ROAD (KY 146) INTERCHANGE

KEY ISSUES / DEFICIENCIES

- Poor levels of service at all study intersections
- Queues present at all study intersections
- Short turn bay lengths
- Trains exacerbate queues and poor LOS

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP-CONTROLLED INTERSECTION
- 67,800 2004 AVERAGE DAILY TRAFFIC
- 980 (1080) 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T PERCENT TRUCKS
- 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600

GRAPHIC SCALE IN FEET

According to this analysis, traffic has been growing at a higher rate on I-265 compared to LaGrange Road. In particular, traffic growth on LaGrange Road south of I-265 has been relatively flat based on the most recent 10 years of data.

Traffic Operations / Level of Service Analysis

Peak period turning movement counts were conducted on 10/19/04. The AM and PM peak hour volumes from these counts are shown on Figure 7-1. Existing levels of service and delay are shown in Table 7-1.

Table 7-1: 2004 Intersection Levels of Service for I-265 / LaGrange Road

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
LaGrange Road / I-265 South Ramps	Signalized	Westbound	48.3	D	40.3	D
		Northbound	25.0	C	30.5	C
		Southbound	**91.9**	**F**	**227.7**	**F**
		Whole Int.	**76.5**	**E**	**133.3**	**F**
LaGrange Road / I-265 North Ramps	STOP Controlled on Ramp	WB Left	**73.4**	**F**	**49.5**	**E**
		WB Right	15.5	C	**227.0**	**F**
		SB Left	8.4	A	13.1	B
LaGrange Road / Factory Lane	Signalized	Eastbound	53.2	D	**122.3**	**F**
		Westbound	50.7	D	53.8	D
		Northbound	35.2	D	**60.7**	**E**
		Southbound	**81.6**	**F**	**57.5**	**E**
		Whole Int.	**59.0**	**E**	**73.3**	**E**

The level of service analysis showed that all intersections operate at a poor LOS. When trains are present and block the intersection, traffic operations further deteriorate. Some areas of particular concern with regard to queuing include: the northbound left turn bay from LaGrange Road to Chamberlain Lane which is short, the north exit ramp which has long queues during the PM peak period, the southbound left at the south ramps, southbound and northbound through at Factory Lane, northbound right and southbound right on LaGrange Road at Factory Lane, westbound left and right on Factory Lane, and eastbound on Chamberlain Lane (refer to Table 7-2). Also, field observations showed that for the high volume of vehicles turning left onto I-265 at the south intersection during the PM peak period, it took several cycles of the signal for vehicles to complete the turn onto the ramp.

Table 7-2: Existing Queuing Issues for I-265 / LaGrange Road Interchange

Int.	Approach / Movement	Design Hour	95th Percentile Queue	Queue Length (ft)	Available Storage Length (ft)	Notes
I-265 South Ramps / LaGrange Road	SB Left	AM	55.8	1395	590	**EXCEEDS** available storage
		PM	87.7	2193	590	**EXCEEDS** available storage
I-265 North Ramps / LaGrange Road	WB Right	PM	35.9	898	590[1]	**EXCEEDS** available storage
LaGrange Road / Factory Lane / Chamberlain Lane	EB Right	PM	45.2	1130	880	**EXCEEDS** available storage
	WB Left and Right	AM	14.3	358	200	**EXCEEDS** available storage
		PM	20.4	510	200	**EXCEEDS** available storage
	NB Left	AM	9.4	235	160	**EXCEEDS** available storage
	NB Right	AM	22.6	565	330	**EXCEEDS** available storage
		PM	60.2	1505	330	**EXCEEDS** available storage
	SB Right	AM	47.4	1185	670	**EXCEEDS** available storage

[1]For this ramp, it was assumed that there are two lanes up to the point (590 feet) where the ramp narrows to below 18 feet, or where two cars can no longer pass. The total ramp length is approximately 1,840 feet.

Safety / Crash Analysis

The crash analysis did not show a crash rate problem (see Table 7-3 below).

Table 7-3: 2001 – 2003 Crash Analysis for I-265 / LaGrange Road Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-265	**21**	**6**	**0**	**42**	**74**	**121**	**0.35**
KY 146	**58**	**22**	**0**	**240**	**272**	**366**	**0.66**

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies

Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor levels of service at all study intersections
- Queues present at all study intersections
- Short turn bay lengths
- Trains exacerbate queues and poor LOS

7.3 RANGE OF ALTERNATIVES

Below is a list of potential improvement alternatives based on identified traffic operations or safety issues from the existing conditions analysis. Refer to Figure 7-2 for the location of these improvements on a map.

- **Alternative 1** – Install signal at LaGrange Road / I-265 North Ramps intersection and coordinate with LaGrange Road / Chamberlain Lane Intersection
- **Alternative 2** – LaGrange Road / Factory Lane / Chamberlain Lane Intersection Improvements
- **Alternative 3** – Improve southbound left turn lane operations on LaGrange Road at south ramps (three options)
- **Alternative 4** – Extend the through lane north of Factory Lane
- **Alternative 5** – Utilize ITS technology on I-265 in advance of the LaGrange interchange (and possibly south of Old Henry Road) to warn drivers with a dynamic message sign (DMS) when a train is crossing Chamberlain Lane, so they can use alternates routes to reach destinations on Chamberlain Lane.

7.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Install Traffic Signal at LaGrange Road / I-265 North Ramps

Traffic and Safety –

Level of Service Analysis – For this intersection, both the westbound left and right turn movements currently experience a poor LOS (LOS E and F) during the PM peak period. The westbound left turn also has a poor LOS during the AM peak period. Using the same traffic volumes and lane configurations, the intersection was analyzed with a new signal. The level of service for the whole intersection indicates that the intersection would operate acceptably. Refer to Table 7-4 for more details.

Table 7-4: Alternative 1 Level of Service and Delay Comparison for LaGrange Road / I-265 North Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
LaGrange Road / I-265 North Ramps	Unsignalized	WB Left	**73.4**	**F**	**49.5**	**E**
		WB Right	15.5	C	**227.0**	**F**
		SB Left	8.4	A	13.1	B
	Signalized	**Whole Int.**	**10.3**	**B**	**19.4**	**B**

Queue Length Analysis – Existing queue lengths for the unsignalized intersection were evaluated to determine if the current storage is exceeded during peak periods. Only the queue length for the westbound right during the PM peak period was found to exceed the available storage. Signalization of this intersection is expected to reduce the queue for this movement to 690 feet, but this is still approximately 100 feet more than the

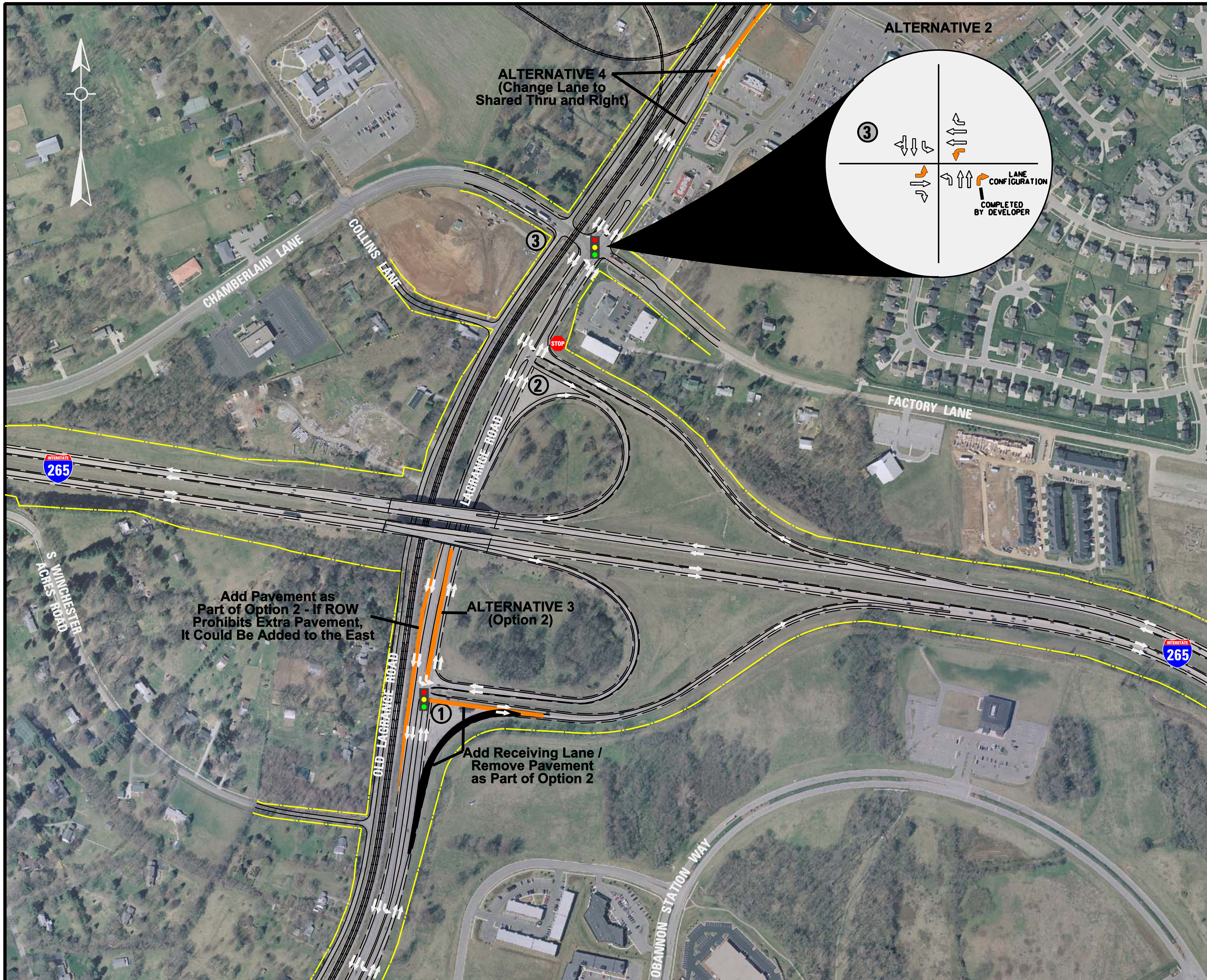


FIGURE 7-2: I-265 & LAGRANGE ROAD (KY 146) INTERCHANGE

ALTERNATIVES

- Alt. 1 - Install Traffic Signal at I-265 North Ramps / LaGrange Road (Intersection 2)
- Alt. 2 - Add NB Right Turn Lane, EB Left Turn Lane, and WB Left Turn Lane at LaGrange Road / Chamberlain Lane Intersection
- Alt. 3 - Improve SB Left Turn Lane Operations on LaGrange Road at South Ramps

  Option 1 - Modify Signal Phasing to Allow Protected / Permitted for the SB Left Turn

  Option 2 - Add 2nd SB Left Turn Lane

  Opton 3 - Lengthen the SB Left Turn Lane and Modify Signal Timing
- Alt. 4 - Extend NB Through Lane on LaGrange Road North From the Kroger Driveway to Forest Springs Drive or Beyond
- Alt. 5 - Advance Train Warning System Using ITS Technology To Warn of Chamberlain Lane Blockage

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

current available storage. While it may be beneficial to stripe the ramp for two lanes and extend these back 100 feet, it may not be necessary given the shorter left turn queues. After the signal is operational, traffic volumes could be monitored to determine if long right turn queues impact intersection operations.

<u>Signal Warrant Analysis</u> – A traffic signal warrant evaluation was also performed to determine if the intersection meets or exceeds any of the MUTCD signal warrants. The four most relevant warrants are listed below along with a brief definition and a discussion of how they compare to the given conditions.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Initially, only four hours of data was collected during the original traffic count. To determine if this warrant is met, an additional fill-in traffic count was conducted on March 23, 2005 from 9:00 AM to 4:00 PM. Using the 70% reduction factor due to the high posted speed (45 mph), the volumes exceed the given threshold for both Condition A and B. **Therefore, this warrant is currently met.**
- Warrant 2: Four-Hour Vehicular Volume – For this analysis, the North off-ramp approach is the minor street and LaGrange Road is the major street. The four hours of data obtained during the AM and PM traffic counts were used as the basis for this warrant analysis. Figure 4C-2 in the MUTCD was used as the threshold curve. The traffic volumes for all four hours plotted above the threshold curve for an intersection with two lanes on the major approach and one lane on the minor approach. **This warrant is currently met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must exceed the given threshold curve as shown on Figure 4C-4 in the MUTCD. From the traffic count data, the highest peak hour is from 4-5 PM. The volumes during this hour plot above the threshold curve; **therefore, this warrant is satisfied.**
- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection. Based on the crash rate analysis, there is no crash problem on LaGrange Road. **Therefore, this warrant is not met.**

Based on the level of service, queuing, and signal warrant analysis for this intersection, a traffic signal could be justified at this location. Installation would solve the existing poor levels of service, and it meets Warrants 1, 2, and 3 as outlined in the MUTCD. While installation of the signal may not solve the right turn queuing issues on the ramp, it does reduce it such that it should not affect the overall intersection operations.

However, this intersection is located very close to the signalized intersection of LaGrange Road / Factory Lane (approximately 330 feet). Therefore, the recommendation of signalizing the north ramps intersection depends in large part on the potential interaction between the two signals. This alternative would include interconnecting this signal with an upgraded signal at Factory Lane. To determine how well they would work together, the existing roadway network with a signal at the north ramps was set up in Synchro 6.0 and simulated. Both signals appeared to function well during the AM peak period, and no blockages were shown for any of the intersection approaches or the mainline (LaGrange Road). For the PM peak period, traffic was observed to queue from the LaGrange / Factory Road intersection back to the north ramps intersection, at times not allowing right turn traffic to turn during the green phase. While this is a problem, it also occurs without the signal at the north ramp intersection. Therefore, for the most part, the signals should interact well to move traffic through this part of the interchange. However, it may be possible (and beneficial) to delay this improvement until some of the other improvements (such as Alternatives 2 and 4) can be made, because they will help this installation work more effectively.

Community / Environmental Impacts – There are no known adverse impacts associated with installing a traffic signal at the north ramp intersection. To obtain greater spacing between this intersection and LaGrange Road / Factory Lane intersection, the north ramps could be shifted to the south if the design speed for the loop ramp was reduced. However, moving the ramps does not initially appear desirable as it would be costly, impact the on-ramp operations, and result in a relatively small increase in queuing capacity.

Costs – The estimated order of magnitude cost for the signal installation, Factory Lane signal upgrade, and coordination / interconnect system is $150,000 in year 2005 dollars.

Alternative 2 – LaGrange Road / Factory Lane / Chamberlain Lane Intersection Improvements

Traffic and Safety – The existing conditions analysis showed that both the eastbound and southbound approaches operate at LOS F during a peak period. In addition, several queuing issues were identified at this intersection. To improve intersection operations, several options were proposed. They include signal optimization and adding a right turn lane in the northbound direction and left turn lanes in the eastbound and westbound directions. The right turn lane is proposed to reduce the existing PM peak period queues in this direction, and the eastbound and westbound left turn lanes are proposed to improve intersection operations specifically for these approaches. Since the beginning of this study, the northbound right turn lane has been constructed at this location by a developer. Refer to Table 7-5 for the levels of service and delay for each option.

Adding a second left turn lane was considered for the northbound approach to provide additional storage for when a train is present across Chamberlain lane. However, the preliminary level of service analysis showed that it did not significantly improve the level of service at the intersection and it would be difficult to construct given the existing intersection alignment (the through lanes would have to be realigned to accommodate the left turn lane). As other options appear to resolve the queuing issues for this movement, it was not analyzed further.

Simply optimizing the signal timing does improve intersection operations, but the eastbound and westbound movements still operate at a LOS E in the PM peak period. Adding a northbound right turn lane and separate eastbound and westbound turn lanes improves the LOS to a C for the whole intersection (with overlap phases as well). All queuing issues are resolved with this option except for the westbound left and the northbound through. Traffic will continue to back up to the north ramp intersection during the PM peak period, possibly blocking access to the northbound right turn lane. However, because the HCS method is conservative and the fact that there are two through lanes, the northbound queues may not impede northbound right turn traffic to the degree as suggested by HCS. During the course of this study, the northbound right turn lane was constructed by a developer in response to a land development approval requirement. Therefore, it is recommended that the other two turn lanes (eastbound and westbound left turns) be constructed in conjunction with signal timing optimization.

To further improve operations at this intersection Alternative 4 could be implemented as discussed later in this section. This will result in a more balanced and efficient northbound through lane utilization, leading in turn to higher movement capacity and reducing the potential for queue spillback south through the north ramps intersection.

Table 7-5: Alternative 2 Level of Service and Delay Comparison for LaGrange Road / Chamberlain Lane

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
LaGrange Road / Chamberlain Lane	Existing	Eastbound	53.2	D	**122.3**	**F**
		Westbound	50.7	D	53.8	D
		Northbound	35.2	D	**60.7**	**E**
		Southbound	**81.6**	**F**	**57.5**	**E**
		Whole Int.	**59.0**	**E**	**73.3**	**E**
	Signal Optimized	Eastbound	34.8	C	**62.8**	**E**
		Westbound	49.3	D	**76.8**	**E**
		Northbound	14.5	B	46.7	D
		Southbound	36.1	D	36.2	D
		Whole Int.	**31.0**	**C**	**53.1**	**D**
	Optimized Plus Dual NB Right and Separate EB/WB Left Turn Lanes	Eastbound	40.4	D	31.9	D
		Westbound	49.9	D	**55.8**	**E**
		Northbound	18.8	B	32.3	C
		Southbound	37.0	D	36.8	D
		Whole Int.	**33.4**	**C**	**36.8**	**C**

Community / Environmental Impacts – There are no known major adverse impacts associated with this alternative – the eastbound and westbound left turn lanes should be able to be constructed within the existing right-of-way. However, coordination with the railroad to make improvements (especially the eastbound left-turn lane) at this intersection could be challenging. To avoid impacts an urban typical section with appropriate drainage may be necessary.

Costs – The estimated order of magnitude cost for this alternative is $500,000 in year 2005 dollars.

Alternative 3 – Improve Southbound Left Turn Lane Operations on LaGrange Road at South Ramps

Three different options were considered to improve the operations of the southbound left turn lane on LaGrange Road at the south ramps intersection. These include the following:

Option 1 – Modify signal phasing to allow protected / permitted for the southbound left turn movement.

Option 2 – Add 2nd southbound left turn lane.

Option 3 – Lengthen the southbound left turn lane and modify the signal timing. In addition, the green time extension (or passage time) should be increased and/or a new advance loop detector should be constructed in the left turn (possibly 150 feet prior to the stop bar). Full actuation at the intersection could also be considered. The goal of these improvements would be to increase the throughput of the existing single lane.

Traffic and Safety – The existing level of service for the southbound left at the I-265 south ramps intersection is LOS F for both the AM and PM peak periods. As a result, the queue length for this movement exceeds the available storage in the turn lane. This queue problem was highlighted by the HCS queue calculations. It was also observed in the field. To improve intersection operations, three options were proposed. The resulting levels of service for Options 1 and 2 are shown in Table 7-6 below.

Table 7-6: Alternative 3 Level of Service and Delay Comparison for LaGrange Road / I-265 South Ramps

			AM		PM	
Intersection	**Scenario**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
LaGrange Road / I-265 South Ramps	Existing	SB Left	**91.9**	**F**	**227.7**	**F**
		Whole Int.	**76.5**	**E**	**133.3**	**F**
	Option 1: Protected / Permitted Phasing Change	SB Left	11.6	**B**	25.5	C
		Whole Int.	**19.1**	**B**	**24.5**	**C**
	Option 2: Dual SB Lefts	SB Left	17.0	B	28.5	C
		Whole Int.	**15.5**	**B**	**23.0**	**C**
	Option 3: Lengthened Turn Lane and Modified Signal Timing	SB Left	19.8	B	47.9	D
		Whole Int.	**17.7**	**B**	**34.1**	**C**

Currently, the signal timing for this intersection only allows for the left turn movement to operate as protected. According to KYTC staff, two fatal crashes occurred fairly soon after the signal was installed. At that time a decision was made to restrict southbound left turns to protected only due to safety concerns. However, at present the interchange area is not highlighted as a high crash area. Allowing vehicles to turn left during the northbound / southbound through phase improves the LOS to C or better for both the southbound left movement and the whole intersection. It also reduces the queue lengths such that they do not exceed the available storage. Therefore, KYTC requested that protected / permitted phasing be reconsidered.

According to the Institute of Transportation Engineers Traffic Engineering Handbook, 5th Edition (ITE, 1999), protected only phasing is recommended if two of the following conditions are met:

- Peak 15-min flow rate for the left turning traffic is greater than 320 vph **(YES – 531 vph in the PM Peak Hour)**
- Peak 15-min flow rate for the opposing traffic is greater than 1,100 vph **(NO)**
- Opposing traffic speed limit is greater than or equal to 45 mph **(YES – The posted speed limit is 45 mph)**
- There are two or more left-turn lanes **(NO)**

As shown, the first and third conditions are currently met. In addition, the Handbook lists a number of other conditions for which protected only phasing is recommended, including a left turning volume exceeding 320 vph with a heavy vehicle percentage of greater than 2.5%. This condition is also met as the AM and PM volumes and percentages are 504 (6% trucks) and 531 (3% trucks) respectively. Therefore, it appears that maintaining protected only phasing at this intersection is appropriate.

Adding a second southbound left turn lane (Option 2) also improves the level of service for this movement (and the whole intersection) to an acceptable LOS. In addition, the queue lengths are reduced such that they would not exceed the available storage assuming the second left turn lane was constructed to be the same length as the first turn lane. Furthermore, the high left turn volumes of 504 during the AM peak period and 531 during the PM peak period are above the threshold of 300 vehicles per hour which is typically used as the initial threshold for dual turn lanes. Therefore, dual left turn lanes are appropriate for this intersection, however, given the cost and the potential benefits of modifying the existing signal as discussed below, this is a longer term recommendation.

Option 3 includes modifying the existing signal timing and possibly lengthening the turn lane. Modifying the timing is the most important part of this option as it can help achieve an acceptable overall LOS for the intersection during both peak periods as shown in Table 7-6 (LOS C or better). It will also reduce turn lane queue lengths. With this option, the signal should be retimed. As part of these changes, increasing the vehicle extension time for the southbound left turns and/or decreasing the maximum green times for the northbound through movement would be beneficial. An advance loop could also be added to the turn lane approximately 150 feet back to provide for green extension for the southbound left turn signal.

In addition, the intersection could be upgraded to be fully actuated as part of the improvements. This change would allow for the elimination of a default mainline green for the northbound movement, which uses time that would be better allocated to the heavy turn movements. The purpose of these changes is to reallocate green time from the northbound through movement and apply it to the southbound and westbound left movements. The changes will also allow the southbound green to extend even when there are longer headways between vehicles. The northbound through movement has relatively small volumes except in the PM peak, but even then it has only 350 vehicles per hour per lane. Northbound queues may increase, but the volumes are not excessive and there is considerable storage.

Therefore, in the near-term it is recommended that signal timing improvements be made to this intersection, with the possible addition of an advance loop in the turn lane. Turn lane lengthening and/or full actuation would be part of a follow-up mid-term improvement. Finally, in the long term, it is recommended that a double left turn lane be installed.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative, though right-of-way is certainly limited in this area by the presence of the CSX railroad line.

Costs – The estimated order of magnitude cost for making the signal timing improvements and adding the loop is under $10,000. The cost of full actuation could be closer to $50,000 and the rough cost of extending the turn lane could be another $150,000 in 2005 dollars. The order of magnitude cost of constructing a second left turn lane is approximately $500,000 in 2005 dollars. This assumes no new right-of-way and that the two lanes merge on the ramp and the existing northbound right turn lane is brought to the intersection and is no longer a free flow lane.

Alternative 4 – Extend Through Lane North from Kroger Driveway to Forest Springs Drive (or even to Reamers Road if possible)

Traffic and Safety – This alternative was proposed at one of the project team meetings. From a traffic flow and safety perspective it should provide better traffic operations from the LaGrange / Factory Road intersection north to Forest Springs Drive. Widening to Reamers Road would provide even more benefit. It will provide improved lane utilization for the rightmost northbound through lane at Chamberlain Lane (the lane that currently drops at the Kroger driveway) and thereby increase capacity at the intersection as shown in Table 7-7. In addition, from a system operations perspective extending the through lane will reduce delay by approximately six percent during the peak period. Overall, this is a good project to reduce delay and further improve traffic operations north of the interchange.

Table 7-7: Alternative 4 Level of Service and Delay Comparison for LaGrange Road / Chamberlain Lane

Intersection	Scenario	With Lane Extension	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
LaGrange Road / Chamberlain Lane	Signal Optimized	**NO**	31.0	C	53.1	D
		YES	30.8	C	49.1	D
	Optimized Plus Dual NB Right and Separate EB/WB Left Turn Lanes	**NO**	33.4	C	36.8	D
		YES	30.0	C	30.4	C

Community / Environmental Impacts – The right-of-way in this area is tight. An urban section may be required and property acquisition could be an issue. Pedestrian and bicycle circulation should also be considered.

Costs – The estimated order of magnitude cost for this alternative is approximately $250,000 in 2005 dollars assuming no new right-of-way and a rural typical section. However, new right-of-way and/or an urban typical section may be required to implement this improvement, which could increase project costs. The cost would also increase if the project limits were extended to Reamers Road, which would be even more beneficial.

Alternative 5 – Advance Train Warning System Using ITS Technology

This alternative would use Intelligent Transportation System (ITS) technology on I-265 northbound in advance of the LaGrange Road interchange to warn drivers with a dynamic message sign that a train is crossing Chamberlain Lane and that they should use alternate routes.

This would consist of a device tied to the interlock for the railroad. The device would be actuated when the crossing gates go down. When there is such a condition, a wireless 900 MHz Frequency Hopped Spread Spectrum (FHSS) signal would then be relayed to the dynamic message sign(s) indicating that a train is blocking Chamberlain Lane. The message could read “Train Blocking Chamberlain Lane at KY 146” and “Use Alternate Route” flashed across two messages. One sign could be located south of the LaGrange interchange and a second could be located south of the Old Henry Road interchange to divert traffic headed for Factory Lane or points east on LaGrange Road.

Two potentially challenging implementation issues for this alternative involve railroad cooperation and gaining authorization to display a non-freeway message on a freeway sign. The proposed message for this ITS train warning system is not standard since the message board on the mainline pertains to another roadway. However, it is justifiable because it is a safety message that is pertinent to the off-ramp / exit and acts to relieve congestion for the system.

Traffic and Safety – Currently, traffic is stopped with little warning when trains pass. During peak times, this backs up the northbound left-turn lane at the LaGrange Road / Chamberlain Lane intersection. This in turn affects through vehicles on LaGrange Road and vehicles existing the I-265 north ramp. A warning system would inform drivers of the blockage, providing them with the opportunity to avoid the area and use alternate routes. For example, advance warning would also allow northbound motorists destined for Chamberlain Lane to travel one exit north to the Westport Road exit. Drivers headed to the Crestwood area may also choose to exit at Old Henry Road to avoid the area.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – For the cabinet, controller, interconnect, Dynamic Message Signs (DMSs) (2) and communications, the estimated costs are roughly $155,000 to $325,000 in 2005 dollars. The largest portion of which is for the DMSs – roughly $104,000 to $260,000 depending on type and size (small freestanding vs. large overhead).

7.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed I-265 / LaGrange Road Interchange alternatives is provided in Table 7-7.

Table 7-8: I-265 / LaGrange Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Install Traffic Signal at I-265 North Ramps / LaGrange Road	●	●	◗	◗	●	●	**YES** (After 2 and 4 if possible)
2	Add NB Right, EB Left, and WB Left Turn Lanes at LaGrange Road / Chamberlain Lane Intersection	●	●	◗	◗	●	◗	**YES**
3-1	Option 1: Modify Signal Phasing to Allow Protected / Permitted	●	◗	●	○	◗	●	**NO**
3-2	Option 2: Add 2nd SB Left Turn Lane on LaGrange Road at South Ramps	●	●	●	◗	◗	◗	**YES (Long-term)**
3-3	Option 3: Modify Signal Timing and Extend Left Turn Lane	●	●	●	◗	●	●	**YES (Short-Term)**
4	Extend Through Lane North of Factory Lane	◗	◗	◗	◗	◗	◗	**YES**
5	Advance Warning System Using ITS Technology	●	●	●	◗	●	◗	**YES**

Ratings Guide: ○ = Poor ◗ = Fair ● = Good

7.6 RECOMMENDATION AND PHASING

To address identified operating and safety deficiencies from the existing conditions analysis, the following improvements are recommended.

- Install traffic signal at I-265 north ramps / LaGrange Road intersection. Traffic should be monitored to ensure the westbound right turn is not impeded by queues from the LaGrange Road / Factory Lane / Chamberlain Lane intersection upstream that typically extend to this intersection. This signal should be coordinated with the upstream signal at LaGrange Road / Factory Lane / Chamberlain Lane to facilitate traffic flow and ensure that queuing is not an issue.

- Optimize signal timing at the intersection of LaGrange Road / Factory Lane / Chamberlain Lane and construct an eastbound and westbound left turn lane, thereby separating the turn lanes from the through lanes. The northbound right turn lane has already been completed by a developer.

- In the near-term make signal timing improvements to LaGrange Road / South Ramps intersection, with the possible addition of an advance loop in the turn lane. Turn lane lengthening and/or full actuation could be part of a follow-up mid-term improvement. Finally, in the long term, it is recommended that a double left turn lane be installed (see below).

- In the longer term, construct a second southbound left turn lane at the I-265 south ramps / LaGrange Road intersection to accommodate existing queues and improve intersection levels of service. The receiving single lane on the ramp will need to be widened to two lanes to accept the turning traffic. The northbound right turn lane should be removed, forcing right turning traffic to go through the signalized intersection. This is suggested to reduce merging issues on the ramp.

- Extend the northbound through lane from Kroger to Forest Springs Drive (or even to Reamers Road) if funding is available.

- Utilize ITS technology on I-265 in advance of the LaGrange interchange (and possibly south of Old Henry Road) to warn drivers with a dynamic message sign (DMS) when a train is crossing Chamberlain Lane, so they can use alternates routes to reach Chamberlain Lane and other destinations in the area.

8.0 I-64 / BLANKENBAKER PARKWAY (KY 913) INTERCHANGE

8.1 INTRODUCTION AND STUDY AREA

The study area for the I-64 / Blankenbaker Parkway (KY 913) interchange consists of the intersections listed below (refer to Figure 8-1).

1. Blankenbaker Parkway (KY 913) / Bluegrass Parkway
2. Blankenbaker Parkway (KY 913) / I-64 Eastbound Ramps
3. Blankenbaker Parkway (KY 913) / I-64 Westbound Ramps
4. Blankenbaker Parkway (KY 913) / Ellingsworth Lane

8.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns

The current average daily traffic volumes for I-64 come from the Highway Information System (HIS) database, and are listed below.

- Approximately 25,200 ADT on Blankenbaker Parkway north of I-64
- Approximately 19,300 ADT on Blankenbaker Parkway south of I-64

Geometrics / Right-of-way

This interchange is a partial cloverleaf with both loop ramps east of Blankenbaker. Due to the short distance between the two loop ramps, weaving and merging are issues.

Land Use, Future Development, and Historic Traffic Growth

Major traffic generators in the vicinity of the interchange include the Bluegrass Industrial Park, Southeast Christian Church (SECC), and major retail developments. Two new developments are proposed near the interchange – Citigroup Call Center and the Ellingsworth office development. Phase I of the Citigroup Call Center just opened adding approximately 160 trips in the AM peak and approximately 300 trips in the PM peak. Phase II is approved and scheduled to open in 2006, adding approximately 390 trips in the AM peak and approximately 240 trips in the PM peak. The proposed Ellingsworth development could add approximately 450 trips during the PM peak hour. Additional growth is expected in the area both north and south of the interchange.

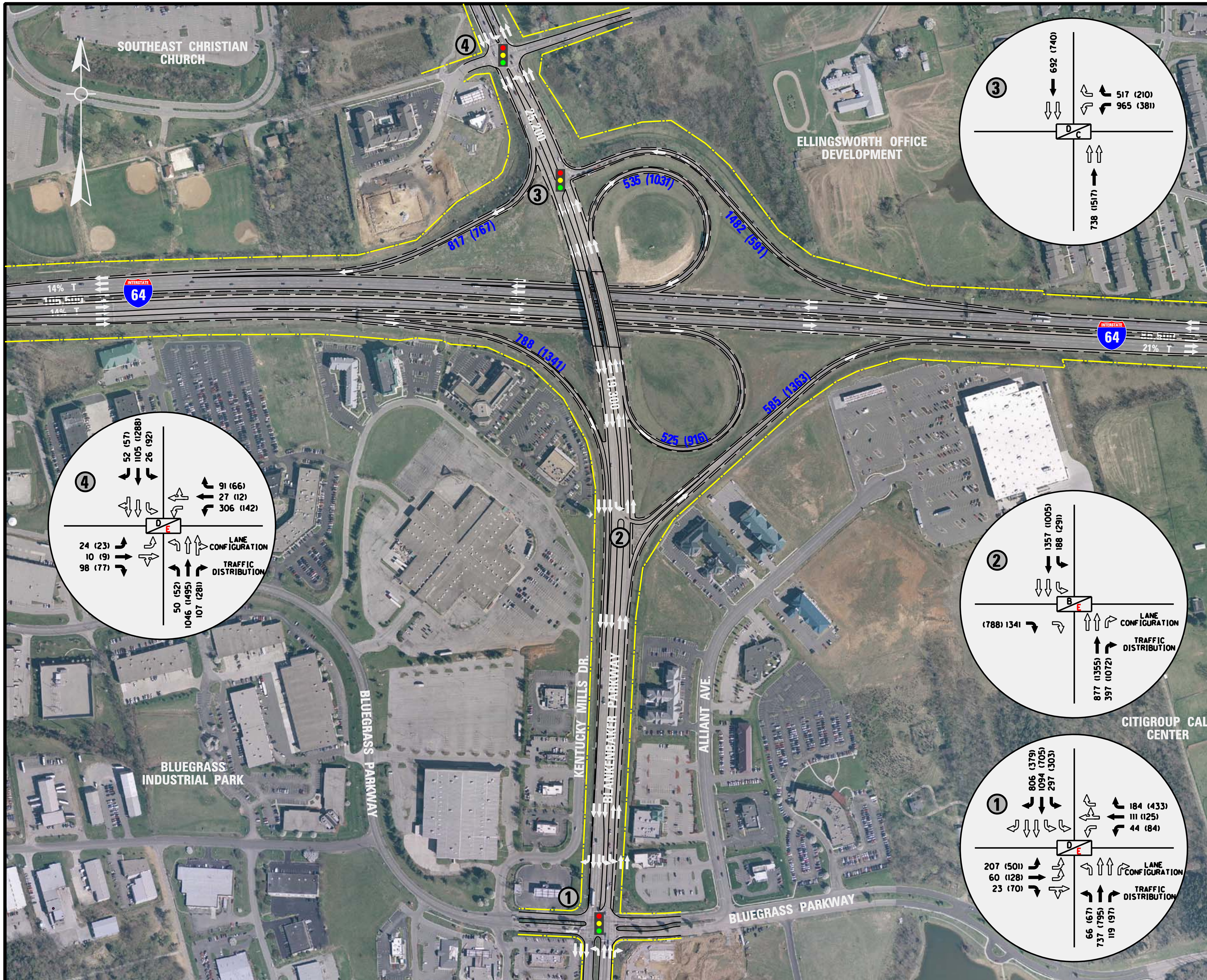
An analysis of the historic traffic volumes showed that traffic has been growing at the following rates between 1987 and 2004:

KY 913 South of I-64	KY 913 North of I-64	I-64 East of Blankenbaker	I-64 West of Blankenbaker
7%	12%	6%	9%

Traffic Operations / Level of Service Analysis

Peak period turning movement counts were conducted on 10/14/04. Additional data was acquired from the Citigroup Call Center Traffic Study (JJG, 2004) and the Ellingsworth Office Traffic Study (BTM, 2004). For each of the key intersections, AM and

FIGURE 8-1: I-64 & BLANKENBAKER PARKWAY (KY 913) INTERCHANGE



KEY ISSUES / DEFICIENCIES

- Major existing traffic generators including the office park and SECC
- Significant ongoing development in the area
- Poor levels of service for one or more movements at each intersection.
- Heavy ramp volumes and weave on Blankenbaker
- High crash rate on Blankenbaker Parkway south of Bluegrass Parkway.

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP-CONTROLLED INTERSECTION
- 67,800 — 2004 AVERAGE DAILY TRAFFIC
- 980 (1080) — 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T — PERCENT TRUCKS
- B/E — 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600
GRAPHIC SCALE IN FEET

PM peak hour volumes are shown on Figure 8-1. Existing levels of service and delay are shown on Table 8-1.

Table 8-1: 2004 Intersection Levels of Service for I-64 / Blankenbaker Parkway

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Blankenbaker Pkwy/ Bluegrass Pkwy	Signalized	Eastbound	54.1	D	**113.9**	**F**
		Westbound	52.3	D	**70.0**	**E**
		Northbound	39.4	D	40.6	D
		Southbound	37.9	D	33.3	C
		Whole Int.	**40.9**	**D**	**57.1**	**E**
Blankenbaker Pkwy / I-64 EB Ramps	Unsignalized	**Southbound Left**	**12.2**	**B**	**35.2**	**E**
Blankenbaker Pkwy / I-64 WB Ramps	Signalized	Westbound	**65.5**	**E***	24.7	C*
		Northbound	31.0	C	36.9	D
		Southbound	30.2	C	17.4	B
		Whole Int.	**48.3**	**D***	**29.3**	**C***
Blankenbaker Pkwy/ Ellingsworth Lane	Signalized	Eastbound	38.1	D	37.4	D
		Westbound	**109.5**	**F**	40.0	D
		Northbound	26.6	C	**104.1**	**F**
		Southbound	37.9	C	31.8	C
		Whole Int.	**39.1**	**D**	**69.3**	**E**

*To achieve this LOS, drivers use the shoulder on this ramp to maintain two travel lanes

Safety / Crash Analysis

The crash analysis showed that there was a crash rate problem on Blankenbaker Parkway (see Table 8-2). Further analysis for this section of Blankenbaker was performed to determine a more specific location. After breaking the section into spots and performing the same analysis for the spot locations, the area with a crash rate higher than the critical crash rate was the portion of Blankenbaker south of Bluegrass Parkway (91 crashes occurred in this "spot"). Therefore, the crash rate problem is just south of the study area and is therefore not addressed by any of the proposed improvements since it is outside the study area.

Table 8-2: 2001 – 2003 Crash Analysis for I-64 / Blankenbaker Parkway Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-64	**91**	**27**	**0**	**64**	**74**	**110**	**0.58**
KY 913	**147**	**44**	**0**	**411**	**332**	**352**	**1.17**

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies
Based on the existing conditions analysis, the key issues / deficiencies are:

- Major existing traffic generators including the office park and SECC
- Significant ongoing development in the area
- Poor levels of service for one or more movements at each intersection
- Heavy ramp volumes and weaving on Blankenbaker Parkway

8.3 FUTURE ANALYSIS SCENARIO

This study was designed to specifically evaluate the existing conditions and recommend improvements for identified deficiencies. Therefore, up to this point the alternatives have been evaluated based on the existing traffic volumes. For this interchange, it was determined that as a result of recent and imminent development in the vicinity of the interchange, near future development volumes for 2006 would be used for the alternatives analysis. These developments include the Citigroup Call Center and the Ellingsworth Office Development. The Citigroup Call Center is located near the intersection of Bluegrass Parkway and Tucker Station Road, south of I-64. Phase I of the Citigroup Call Center recently opened, employing approximately 2,130 people with 500 of those employees coming from the nearby call center which recently closed. Phase II will result in the addition of 1,200 jobs at the call center as well as 1,400 jobs in an office development at the same site. Phase II is scheduled to be completed in 2006. The Ellingsworth office development is located on Ellingsworth Lane, north of I-64. Currently this development is in the review stages and may move to the construction phase in the future. Therefore, traffic volumes were generated for a full development scenario for the year 2006.

Traffic Volumes
To determine the traffic volumes for the 2006 full build scenario the base traffic volumes obtained from the peak period traffic counts were increased by two percent per year to reflect future background traffic growth. A growth factor of two percent per year was used in both traffic studies for the new and proposed developments and was used for this analysis as well. Additional traffic generated by the recent and new developments that would go through the study intersections was added to the background volumes based on turning movement percentages.

Using the new traffic volumes for the 2006 build scenario, a level of service analysis was prepared as the baseline for alternatives comparison (refer to Table 8-3).

Table 8-3: 2006 Intersection Levels of Service for I-64 / Blankenbaker Parkway

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Blankenbaker Pkwy/ Bluegrass Pkwy	Signalized	Eastbound	**55.5**	**E**	**132.2**	**F**
		Westbound	**63.9**	**E**	**469.5**	**F**
		Northbound	42.8	D	41.6	D
		Southbound	**111.4**	**F**	51.0	D
		Whole Int.	**85.6**	**F**	**142.2**	**F**
Blankenbaker Pkwy / I-64 EB Ramps	Unsignalized	**Southbound Left**	**15.8**	**C**	**179.8**	**F**
Blankenbaker Pkwy / I-64 WB Ramps	Signalized	Westbound	**110.7**	**F***	27.4	C*
		Northbound	**59.6**	**E**	**87.2**	**F**
		Southbound	34.9	C	20.0	C
		Whole Int.	**77.3**	**E***	**55.6**	**E***
Blankenbaker Pkwy/ Ellingsworth Lane	Signalized	Eastbound	38.3	D	37.5	D
		Westbound	**227.3**	**F**	**338.8**	**F**
		Northbound	**90.8**	**F**	**192.3**	**F**
		Southbound	31.5	C	45.3	D
		Whole Int.	**86.1**	**F**	**153.7**	**E**

*To achieve this LOS, drivers use the shoulder on this ramp to maintain two travel lanes

Queue lengths that exceed the available storage are an issue and in fact worsen due to the higher traffic volumes resulting from the new and proposed developments. Locations that have queue lengths exceeding the current storage are:

- Westbound Left at the Ellingsworth Lane / Blankenbaker Parkway Intersection
- Both the Westbound Left and Right at the I-64 Westbound Off-Ramp Intersection
- The Southbound Left at the I-64 Eastbound On-Ramp Intersection
- Many approaches to the Blankenbaker Parkway / Bluegrass Parkway Intersection

8.4 EVENT PEAK TRAFFIC DATA COLLECTION

Additional data collection was performed at this interchange to capture all traffic peaking characteristics which includes peak flows to / from the nearby Southeast Christian Church. Therefore, traffic counts and field observations were performed on several Sundays and during the church's annual Easter Pageant production.

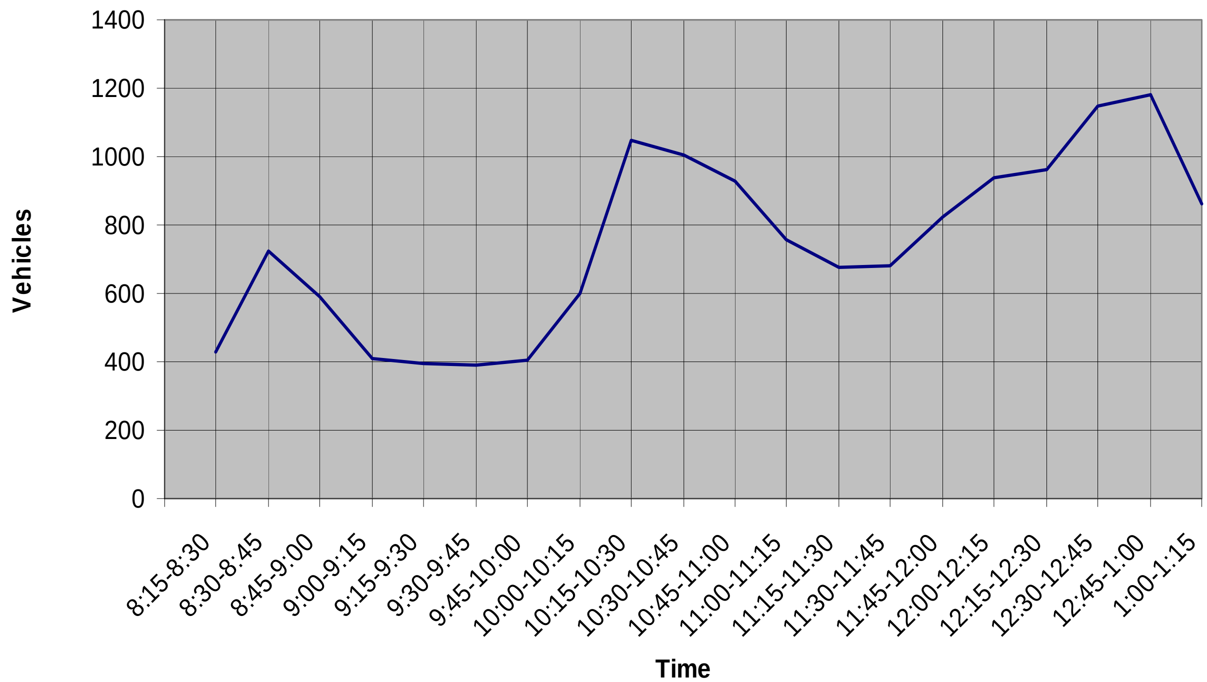
Traffic Count Procedure

The first step in the data collection process was to select a typical Sunday to perform the traffic counts. Sunday, April 17, 2005 was selected as a typical day for traffic, with no special events occurring at the church. To capture the critical traffic flows through the interchange, it was decided that volume data would be collected on all of the major ramps to / from I-64 as well as on Blankenbaker Parkway from just north of the I-64 Westbound Off-Ramp to just south of the I-64 Eastbound On-Ramp. To collect the data, road tubes were laid out the previous afternoon at seven key locations. In addition, a manual turning movement count was performed at the I-64 Westbound Off-Ramp on Sunday morning to provide the turn volumes from the ramp.[3]

Traffic Volumes

The Sunday morning traffic volumes were analyzed with particular attention given to the inbound flows to the interchange to highlight the traffic peaking characteristics as shown in Figure 8-2. Three peak periods were identified: 1) 8:15-9:15 AM, 2) 10:15-11:15 AM, and 3) 12:00-1:00 PM. The first two periods correspond to the beginning times of the two services at Southeast Christian Church (9:00 AM and 11:00 AM). The third peak corresponds to the end of the second service (approximately 12:15 PM) as well as to an increase in traffic headed to the retail and restaurant establishments south of the interchange. Refer to Figure 8-3 for a graphical depiction of the traffic flow through the interchange during these three peak periods.

Figure 8-2: Total Inbound Traffic Volumes



[3] Due to an equipment problem, the manual count was repeated on Sunday, May 8, 2005. This second count was conducted from 8:00 AM to 1:00 PM, capturing the peak periods before and after each of the services. The northbound through volumes counted on May 8 were compared to the April 17 through volumes to determine if an adjustment factor was required to make the data compatible. The May 8 counts were found to be 6% higher than the earlier April 17 counts, therefore no adjustments were made.

Figure 8-3: I-64 / Blankenbaker Parkway
Sunday Morning Peak Period Volumes

A closer examination of the data reveals that some of the specific interchange movements have unique peaking characteristics. For example, the westbound off-ramp exhibits two morning peaks correlating with the start of each service as shown in Figure 8-4. Separating the left and right turning traffic on this ramp shows that the right turning traffic has the same characteristics, but the left-turning traffic generally increases over the course of the morning as shown in Figure 8-5.

Figure 8-4: Westbound Off-Ramp Volume

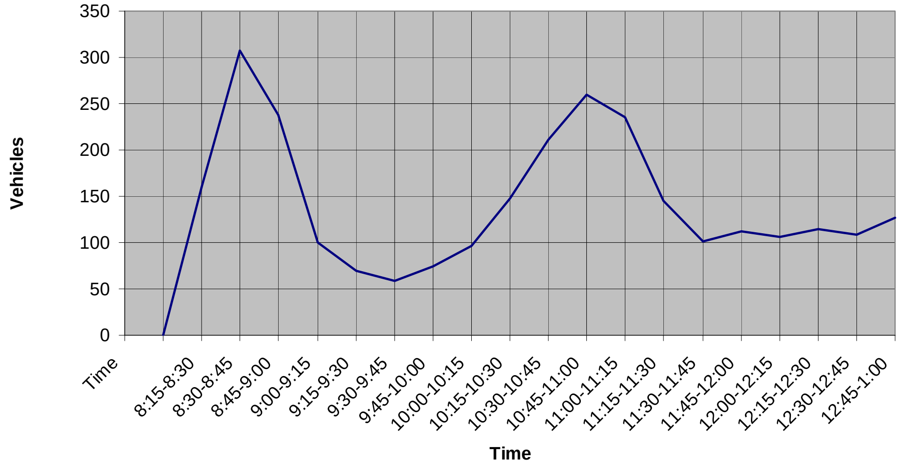
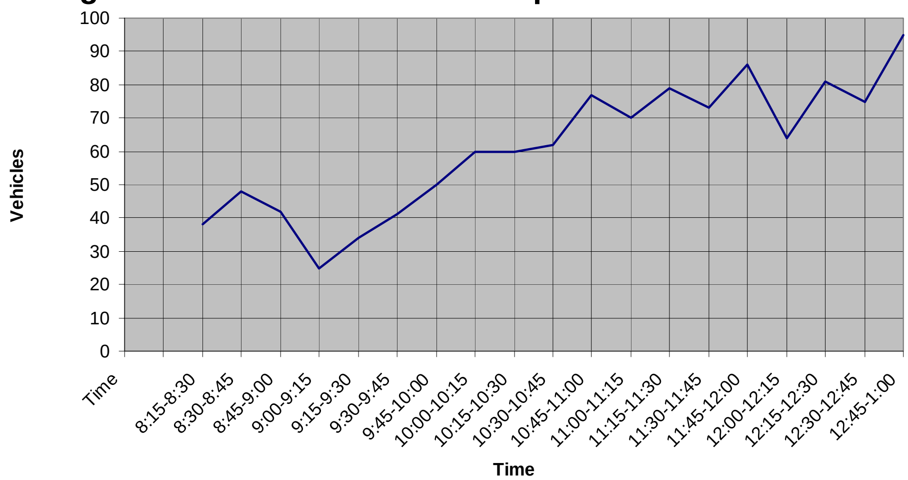


Figure 8-5: Westbound Off-Ramp Left Turn Volumes



Field Observations

During the major event traffic periods at Southeast Christian Church (including Sunday mornings) the church employs police to control traffic from the I-64 Westbound Off-Ramp intersection north to Watterson Trail. Cones are also used to limit vehicle weaving in this stretch of highway. On the two Sundays that the counts were conducted queues were observed extending back approximately 800 feet on the I-64 Westbound Off-Ramp. Northbound queues on Blankenbaker extended approximately 600 feet back onto the bridge over I-64. In the southbound direction, there was considerable congestion from the I-64 Westbound On-Ramp north to the main church driveway and even to Watterson Trail. Southbound left queues were also observed at the I-64 Eastbound On-Ramp intersection, extending north past the start of the left turn lane.

While an attempt was made to capture a “typical Sunday”, based on observations on other Sundays and on conversations with a traffic control officer there during the count periods, these volumes appear to be moderate compared to volumes experienced on some other Sundays. Field observations were also conducted earlier in the year (March 2005) during the church’s annual Easter Pageant production. Traffic flows prior to this event, which runs nearly every day for two weeks in the spring, were considerably heavier than those observed during the two Sunday count periods. Traffic on the I-64 Westbound Off-Ramp backed up over 1,000 feet and northbound Blankenbaker traffic backed up south past the Bluegrass Parkway. The I-64 eastbound to northbound loop off-ramp also backed up over 1,200 feet to the diverge with the I-64 eastbound to southbound off-ramp. This poor operating condition lasted for a considerable time, even after the event began at 7:00 pm.

Operational Deficiencies
Key locations identified as experiencing delay and long queues as a result of high event traffic volumes included the southbound through lanes from the I-64 Westbound On-Ramp intersection north to the church, the I-64 Westbound Off-Ramp (especially due to right-turning traffic), the northbound approach to the Westbound Off-Ramp intersection (mainly during very heavy event traffic periods), and the southbound left-turn at the I-64 Eastbound On-Ramp intersection.

8.5 RANGE OF ALTERNATIVES

- **Alternative 1** – Install signal at Blankenbaker Parkway / I-64 Eastbound Ramps Intersection
- **Alternative 2** – I-64 Westbound Off-Ramp Improvements
- **Alternative 3** – Blankenbaker Parkway / Ellingsworth Lane Intersection Improvements
- **Alternative 4** – Blankenbaker Parkway / Bluegrass Parkway Intersection Improvements
- **Alternative 5** – Add Third Southbound Lane from Southeast Christian Church to the I-64 WB On-Ramp

Figure 8-6 shows these alternatives on an aerial photo.

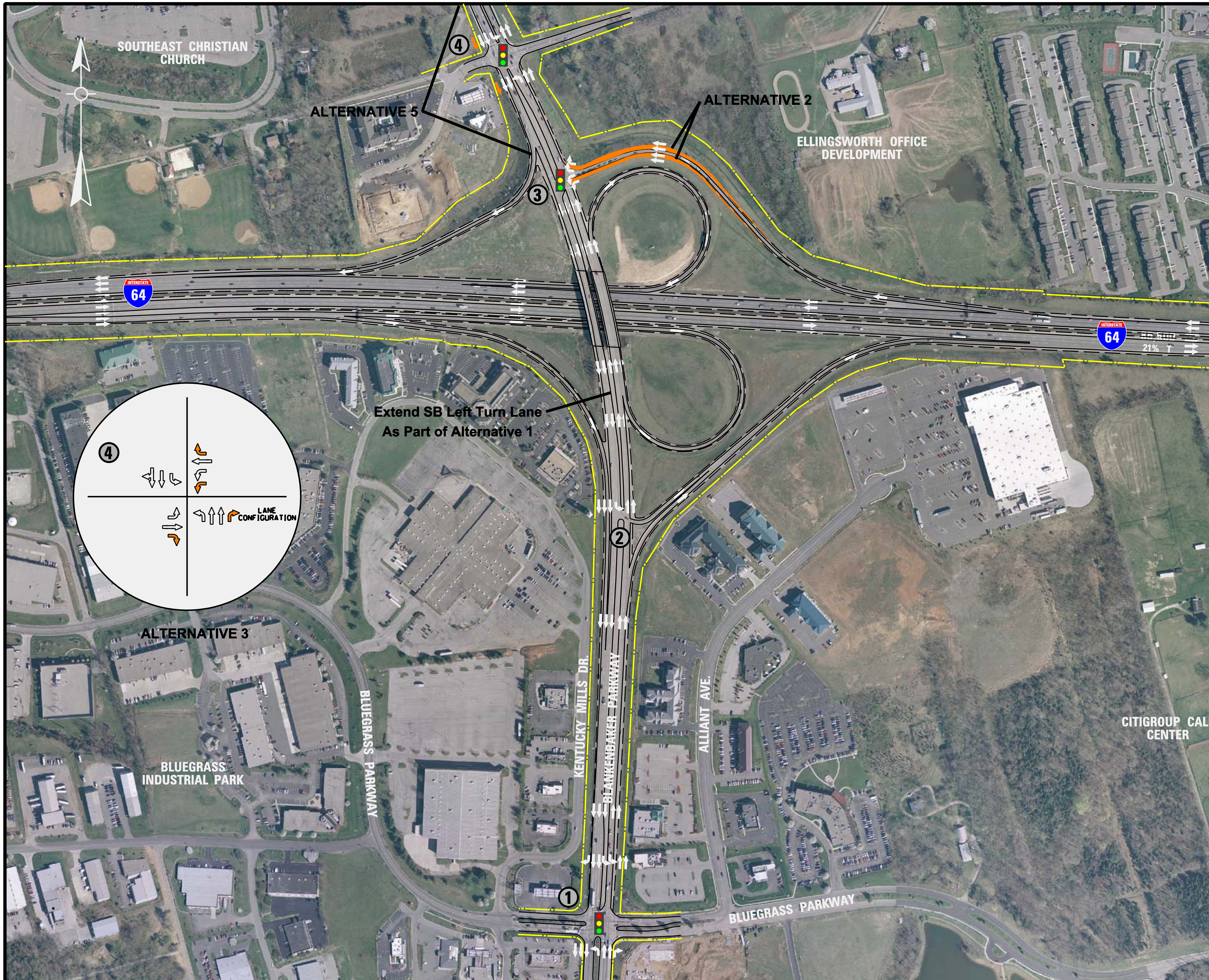
8.6 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Install Traffic Signal at Blankenbaker Parkway / I-64 Eastbound Ramps Intersection

Traffic and Safety –

<u>Level of Service Analysis</u> – The Highway Capacity Software (HCS) package was used to evaluate operating conditions for this intersection. The initial 2006 level of service analysis showed that the southbound left turn onto the I-64 eastbound ramp is experiencing significant delay and poor levels of service in the PM peak (LOS F) hour while the AM peak hour appears to be operating at an acceptable level (LOS C). Installation of a traffic signal at this location will improve the operating conditions for the southbound left turn (LOS B), at the expense of northbound through traffic. However, the level of service will remain above the threshold for desirable traffic operations. Signal control will not be necessary for the southbound through traffic since this movement does not conflict with any other movements through the intersection. Instead, green arrows can be used to direct through traffic (similar to the signal control used for the Hurstbourne Parkway / I-64 Eastbound On-Ramp).

FIGURE 8-6: I-64 & BLANKENBAKER PARKWAY (KY 913) INTERCHANGE



ALTERNATIVES

- Alt. 1 - Install Traffic Signal at I-64 EB Ramps / Blankenbaker Parkway Intersection (Int. 2) and Lengthen SB Left Turn Lane
- Alt. 2 - Construct Dual Left and Right Turn Lanes on I-64 WB Off-Ramp
- Alt. 3 - At Blankenbaker Parkway / Ellingsworth Lane Optimize Signal Timing and Construct Dual WB Left Turn Lanes and Exclusive NB, EB and WB Right Turn Lanes
- Alt. 4 - Optimize Signal Timing at Blankenbaker Parkway / Bluegrass Parkway and Modify Phasing / Lanes to Allow for WB Free-Flow Right Turn Movement
- Alt. 5 - Add 3rd SB Lane from Southeast Christian Church to the I-64 WB On-Ramp

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

<u>Queue Length Analysis</u> – Based on the existing and future development analysis, queue lengths for the southbound left turn lane typically back up during peak hours. The current storage length for left turning vehicles is approximately 270 feet. With the intersection signalized, the southbound left turn queue becomes 183 feet in the AM peak and 383 feet during the PM peak. Therefore, even with the signal in place, the available storage is exceeded during the PM peak by approximately 110 feet. To provide adequate storage with signal installation, it is recommended that the southbound left turn lane be extended back to the bridge. This should be sufficient length to accommodate the PM peak queues.

<u>Signal Warrant Analysis</u> – A traffic signal warrant evaluation was also performed to determine if the intersection meets or exceeds any of the MUTCD signal warrants. The most relevant warrants for the analysis of this intersection are listed below along with a brief discussion of how they compare to the given conditions.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Initially, only four hours of data was collected. Therefore, a fill-in traffic count was conducted on March 24, 2005 from 9:00 AM to 4:00 PM to assemble the required eight hours of data. Using the 70% adjustment factor since the speed on the major street exceeds 40 mph; eight hours exceed the threshold values on Table 4C-1 for both Condition A and B. **Therefore, this warrant is currently met.**
- Warrant 2: Four-Hour Vehicular Volume – For this analysis, the southbound left turn movement is considered to be the minor street volume since it experiences delay due to crossing the major street. The major street volume is the northbound through traffic since it is the only movement that conflicts with the southbound left turning traffic. The four hours of data obtained during the AM and PM traffic counts were used as the basis for this warrant analysis. Figure 4C-2 in the MUTCD with the 70% adjustment factor was used as the threshold curve. This figure is recommended for use if the speed limit on the major street exceeds 40 mph (the posted speed limit on Blankenbaker Parkway is 45 mph). The traffic volumes for all four hours plotted above the threshold curve shown for an intersection with two or more lanes on the major approach (northbound through) and one lane on the minor approach (southbound left). **Based on these traffic volumes, this warrant is currently met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must exceed the threshold curve shown on Figure 4C-4 in the MUTCD. The threshold curve with the 70% adjustment factor is used again since the posted speed limit on Blankenbaker Parkway exceeds 40 mph. This intersection meets the peak hour warrant in both the AM and PM peak hours. **Therefore this warrant is satisfied based on the current volumes.**
- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection. Three criteria must be met including:

    1) Adequate trial of alternatives with failure to reduce the crash frequency;

2) Five or more reported crashes within a 12-month period which could be corrected by installation of a traffic signal; and
3) The traffic volume (in vehicles per hour) exceeds the 80 percent threshold for the eight-hour vehicular volume warrant.

PB is unaware of any safety improvements being tried at this intersection. As for the second criterion, the initial crash analysis for the interchange indicated that there is a high crash rate on Blankenbaker Parkway. However, after breaking the section into spots and performing the same analysis for the spot locations, the only area with a crash rate higher than the critical crash rate was the portion of Blankenbaker Parkway south of Bluegrass Parkway. The area in the vicinity of the intersection of Blankenbaker Parkway / I-64 Eastbound On-Ramp was below the critical crash rate. West of the intersection there were 18 total crashes over a period of three years. These crashes were rear end crashes (8), mid-block collisions (5), sideswipes in the same direction (3), or collisions with fixed objects (2). Most, if not all, of these crashes were in the same direction and many are unlikely to be mitigated with the installation of a traffic signal. The third evaluation criterion is met, with volumes exceeding the 80% threshold for both Condition A and B for at least eight hours during the day. **However, with only one of the criteria being met at this time, the crash warrant is not currently met.**

Based on the analysis presented in this section, it appears reasonable to recommend the installation of a traffic signal at this location. There is an identified level of service problem which is fixed by installing a traffic signal, and traffic volumes are high enough such that Warrants 1, 2, and 3 are met. There is also the potential for safety issues due to the higher volumes and the speed of traffic northbound on Blankenbaker Parkway. Without a traffic signal, during the peak event periods including both weekday peaks and those associated with Southeast Christian Church, the southbound left turn lane backs up beyond the current bay length. However, the queuing issue for the southbound left turn is not addressed through installation of a signal. Therefore the turn lane should be extended to accommodate the queue which means that the turn lane should be extended back to the bridge. This should be done in conjunction with the signal installation to ensure that the southbound left turn traffic does not back up and impede through traffic.

Community / Environmental Impacts – There are no known adverse environmental or community impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $160,000 in 2005 dollars.

Alternative 2 – I-64 Westbound Off-Ramp Improvements

Traffic and Safety – The baseline 2006 development analysis showed that during the AM peak period the westbound movement operates at a LOS F and during the PM peak period the northbound movement operates at a level of service F. As mentioned before, this is assuming separate right and left turn lanes at the intersection approach since this is how traffic operates currently. At a minimum, this ramp should be widened to accommodate striping for separate right and left turn lanes.

Some options to improve the level of service and reduce the delay include optimizing the signal timing, providing dual left turn lanes, and providing both dual left and right turn lanes. Table 8-4 shows the resulting levels of service and delay from these improvements along with the existing conditions for comparison. For all analysis scenarios the 2006 future development volumes are used.

Table 8-4: Alternative 2 Level of Service and Delay Comparison for Blankenbaker Parkway / I-64 Westbound Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Blankenbaker Parkway / I-64 WB Ramps	Existing Conditions	Westbound	**110.7**	**F**	27.4	C
		Northbound	**59.6**	**E**	**87.2**	**F**
		Southbound	34.9	C	20.0	C
		Whole Int.	**77.3**	**E**	**55.6**	**E**
	Optimized	Westbound	**107.9**	**F**	34.9	C
		Northbound	50.1	D	37.6	D
		Southbound	34.5	C	14.8	D
		Whole Int.	**73.0**	**E**	**30.5**	**C**
	Dual Lefts	Westbound	25.4	C	28.3	C
		Northbound	28.3	C	14.8	B
		Southbound	18.6	B	6.8	A
		Whole Int.	**24.7**	**C**	**15.2**	**B**
	Dual Lefts and Rights	Westbound	15.8	B	23.3	C
		Northbound	28.3	C	14.8	B
		Southbound	18.6	B	6.8	A
		Whole Int.	**20.2**	**C**	**14.2**	**B**

Optimizing the signal timing theoretically will improve traffic operations such that most movements operate at LOS D or better, but the westbound left still operates at LOS F in the AM peak. Constructing dual left turn lanes will improve all movements to LOS C or better including the AM westbound left. The ultimate build option is to construct both dual right and left turn lanes. This option provides the best LOS, but it is similar to only constructing dual left turn lanes during the typical commute peak periods.

For each of the proposed improvement scenarios, a queue length analysis was prepared to determine if any queue lengths exceed the available storage. For the options of constructing dual left turn lanes and dual right turn lanes, the queue length is provided to determine the appropriate length of the turn lanes. The current available storage length is given for comparison purposes. Table 8-5 shows the results of this analysis. For each scenario only the maximum queue is given for each movement.

Table 8-5: Alternative 2 Queue Length Evaluation for Blankenbaker Parkway / I-64 Westbound Ramps

Int.	Approach / Movement	Design Hour	95th Percentile Queue	Queue Length (ft)	Available Storage Length (ft)	Notes
Existing	WB Left	AM	113.2	2830	230	**EXCEEDS** available storage
	WB Right	AM	35.4	885	230	**EXCEEDS** available storage
Optimized	WB Left	AM	115.4	2885	230	**EXCEEDS** available storage
	WB Right	AM	36.9	923	230	**EXCEEDS** available storage
Dual Lefts	WB Left	AM	20.5	513	230	**EXCEEDS** available storage
	WB Right	AM	29.7	743	230	**EXCEEDS** available storage
Dual Lefts and Rights	WB Left	AM	20.5	513	230	**EXCEEDS** available storage
	WB Right	AM	11.3	283	230	**EXCEEDS** available storage

For this analysis, the available storage length was assumed to be the length of the ramp from the intersection approach to where it narrowed down to one travel lane. The actual total length of the ramp is approximately 1,970 feet. According to Table 8-5, both the queues for the left and right turn movements exceed the available storage. As more improvements are made, the queue lengths become shorter, but still exceed the current turn lane storage length. Therefore, even with adding turn lanes, the length of the turn lanes will need to be extended past the wider portion of the ramp and dual left and right turn lanes will need to be constructed to accommodate the existing queues.

An additional aspect to consider in selecting the appropriate number of turn lanes for this ramp is the impact of traffic volumes associated with weekend event peak traffic conditions. The I-64 WB Off-Ramp is a primary access point for traffic headed toward Southeast Christian Church from the east. Therefore, the right turn from the ramp toward the church is a heavy movement during peak periods on Sundays, Saturday evenings, and other special events. The peak right turn volume during the Sunday count was 652 vehicles per hour, with 259 in the peak 15 minute period. Field observations during the traffic count showed that queues extended back approximately 800 feet. However, during other field observation periods traffic has backed up onto the I-64 mainline. Because it is only a single lane ramp, most right turning vehicles on the ramp move to the right and drive on a portion of the shoulder during these peak periods. This allows left turning vehicles to pass on the left. Based on the high right turn volumes on Sundays, Saturday evenings, and during other special events at the church, dual right turn lanes would also be beneficial to reduce delay and improve ramp operations (including increased queue storage space). A preliminary analysis of the intersection showed that the added right turn lane could reduce delay and queues during event peak periods at this intersection. However, additional downstream improvements would be required to fully realize the benefits of this improvement. In addition, traffic control plans would have to be developed to facilitate safe operations at the intersection when under police control.

The northbound approach to the westbound off-ramp intersection frequently backs up during peak event periods due to the high ramp volume, the queue backup from the vicinity of the church entries, and weaving on the bridge between the eastbound off-ramp and westbound on-ramp. The improvements discussed above would mitigate the ramp volume conflict. The queues from the church entry locations can only be improved through downstream improvements on Blankenbaker Parkway and on church property. The weave condition on the bridge is difficult to improve without major changes to the interchange design (i.e. the addition of a flyover). Therefore, the westbound off-ramp improvements are recommended as the best near-term improvement to address these queue and capacity issues.

Based on this analysis, constructing dual left turns would provide optimal traffic flow through the intersection for average weekday traffic; however both dual left and right turn lanes are required to provide adequate queuing storage, and dual right turn lanes are necessary to handle the event peak traffic conditions. Therefore, both dual left and right lanes are recommended. To accommodate the longest queue, the left turn lanes should extend a length of 550 feet at a minimum, but could be extended farther if possible. The right turn lanes should be a minimum of 750 feet, but longer if possible. If necessary, the center lane could be designed as a shared or optional left and right turn lane (similar to the I-65 exit ramp to Preston Highway at Old Grade Lane).

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative; however there is the potential for environmental issues associated with the land north of the ramp.

Costs – The estimated order of magnitude cost for this alternative is $950,000 in year 2005 dollars.

Alternative 3 – Blankenbaker Parkway / Ellingsworth Lane Intersection Improvements

Traffic and Safety – Based on the existing conditions analysis using the 2006 development volumes, both the northbound and westbound movements are expected to operate poorly. As part of the Ellingsworth Office Development Study by Birch, Trautwein and Mims, Inc. there are recommendations for improvements to the intersection of Blankenbaker Parkway / Ellingsworth Lane. These improvements include dual westbound left turn lanes, an exclusive northbound right turn lane, and signal timing improvements. All of these improvements target the identified operational deficiencies. The levels of service and delay for these improvements as well as for the 2006 existing conditions for comparison are shown in Table 8-6.

Table 8-6: Alternative 3 Level of Service and Delay Comparison for Blankenbaker Parkway / Ellingsworth Lane

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Blankenbaker Pkwy/ Ellingsworth Lane	2006 Existing Conditions	Eastbound	38.3	D	37.5	D
		Westbound	**227.3**	**F**	**338.8**	**F**
		Northbound	**90.8**	**F**	**192.3**	**F**
		Southbound	31.5	C	45.3	D
		Whole Int.	**86.1**	**F**	**153.7**	**F**
	2006 Optimized & Turn Lanes	Eastbound	48.7	D	**62.0**	**E**
		Westbound	49.2	D	**73.0**	**E**
		Northbound	24.6	C	38.4	D
		Southbound	29.5	C	34.4	C
		Whole Int.	**30.6**	**C**	**42.0**	**D**
	2006 Separate EB and WB Right Turn Lanes	Eastbound	32.3	C	45.9	D
		Westbound	35.0	C	54.4	D
		Northbound	20.4	C	32.3	C
		Southbound	34.6	C	29.7	C
		Whole Int.	**28.0**	**C**	**34.5**	**C**

The proposed turn lanes and signal timing optimization improves the overall operating conditions. However with the recent and proposed development in place, the eastbound and westbound movements will still operate poorly at LOS E during the PM peak period. To improve operations for these movements, separate eastbound and westbound right turn lanes could be constructed (these were not part of the BTM study). Not only do they provide additional storage for turning vehicles and improved flow, but separating these turn movements allows for them to be included as an overlap phase. With these additional turn lanes in place, the intersection operations for each movement are LOS D or better (LOS C overall).

Also mentioned as part of the 2006 future analysis scenario were queuing issues for the westbound left turn movement. The improvements discussed above will reduce the queue length, but will not improve operations such that the queue is less than the available 170 feet of storage. To provide adequate storage, the turn lane should be extended to a total length of 500 feet or as far back as possible.

The northbound left turn into the gas station at this intersection does not appear to exceed the maximum available storage, but it was identified at a project team meeting that this turn lane is very short and can only accommodate a few cars at a time. If possible, this turn lane should be extended back to provide adequate storage, especially with the expected traffic growth in the future. The westbound off-ramp intersection is located just south of this intersection, so the turn lane should extend back to this intersection.

Based on the analysis for this intersection the recommendation to achieve good levels of service is to construct the improvements proposed in the development study (dual westbound lefts, an exclusive northbound right, and signal optimization) as well as separating the eastbound and westbound right turn lanes from the through lanes, making them exclusive. To provide adequate storage for the westbound left turns, the new dual left turn lanes should be constructed to a distance of 500 feet back from Blankenbaker, or as far back as possible. Also, the northbound left turn lane on Blankenbaker Parkway should be extended back from the intersection as far as possible.

Community / Environmental Impacts – There are no known adverse environmental impacts associated with this alternative. There could be some property impacts, but these should be minimal (if any) as construction is likely to remain within the existing right-of-way.

Costs – The estimated order of magnitude cost for this alternative is $570,000 in year 2005 dollars. However, the majority of this cost would be born by the developer to construct the second westbound left turn lane and the northbound right turn lane. The cost for constructing the eastbound and westbound right turn lanes as well as the extension of the northbound left turn lane may fall to KYTC.

Alternative 4 – Blankenbaker Parkway / Bluegrass Parkway Intersection Improvements

Traffic and Safety – This intersection currently operates poorly, with traffic operations only worsening with the 2006 development volumes. In the Citigroup Call Center Study by Jordan Jones and Goulding there are recommendations for improvements to the intersection of Blankenbaker Parkway / Bluegrass Parkway which included dual southbound left turn lanes and signal timing improvements. The study also included a proposal to widen the right turn lane on westbound Bluegrass Parkway and provide a 600 foot northbound auxiliary on Blankenbaker Parkway to facilitate a free flow right turn movement. Since the study was completed, a second left turn lane has been added in the southbound direction on Blankenbaker Parkway at Bluegrass Parkway. Therefore the first option analyzed was improved signal timing. As shown in Table 8-7, intersection operations were only improved slightly. While all movements operate poorly during the PM peak, only the eastbound and westbound operate poorly during the AM peak period. If the westbound right turn traffic could operate in a free-flow manner, this would moderately improve intersection operations. However, the whole intersection as well as all approaches except the westbound approach would still operate at a poor LOS. The westbound approach would become LOS D in the PM peak, but the westbound left would operate at LOS E. Therefore, the next improvement option considered was to separate the right turn lanes such that they were exclusive from the through movements. This resulted in adequate levels of service during the AM peak period and reduced delay during the PM peak period but did not fully fix the level of service problem. Aside from proposing widening Blankenbaker to six lanes, it appears that this intersection is operating at capacity, therefore signal optimization and turn lane additions will decrease delay for the intersection but will not resolve the LOS problems.

Table 8-7: Alternative 4 Level of Service and Delay Comparison for Blankenbaker Parkway / Bluegrass Parkway

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Blankenbaker Pkwy/ Bluegrass Pkwy	2006 Existing Conditions	Eastbound	**55.5**	**E**	**132.2**	**F**
		Westbound	**63.9**	**E**	**469.5**	**F**
		Northbound	42.8	D	41.6	D
		Southbound	**111.4**	**F**	51.0	D
		Whole Int.	**85.6**	**F**	**142.2**	**F**
	2006 Optimized	Eastbound	**69.2**	**E**	**114.4**	**F**
		Westbound	**64.0**	**E**	**201.1**	**F**
		Northbound	52.5	D	**86.3**	**F**
		Southbound	40.7	D	**57.0**	**E**
		Whole Int.	**47.9**	**D**	**101.1**	**F**
	2006 Optimized with Separate EB and WB Right Turn Lanes	Eastbound	54.9	D	**68.1**	**F**
		Westbound	53.5	D	**67.0**	**E**
		Northbound	37.0	D	**73.2**	**E**
		Southbound	28.1	C	51.9	D
		Whole Int.	**33.8**	**C**	**62.7**	**E**

Community / Environmental Impacts – There are no known environmental or community impacts associated with this alternative.

Costs – The estimated cost for optimizing the signal timing is minimal. If the westbound right turn lane is widened and a lane is added on Blankenbaker Parkway to receive this traffic, this cost would be the responsibility of the developer.

Alternative 5 – Add Third Southbound Lane from Southeast Christian Church to the I-64 WB On-Ramp

Traffic and Safety – Currently there are two southbound through lanes from the church to the I-64 Westbound On-Ramp. During peak event traffic flow periods (i.e. when an event or service ends), these lanes are typically at capacity. During the Sunday count, the highest peak hour flow counted was 1,922 vehicles per hour (for the two lanes) with a peak 15-minute flow of 570. The restraining factors on the capacity of this flow include side street traffic at Ellingsworth Lane as well as the high density / low speed nature of the traffic, which results in low and relatively unstable traffic flow characteristics (in vehicles per hour). One option to increase capacity and improve traffic flow would be the addition of a third southbound lane. If the third lane were added, it would be the most beneficial if the on-ramp were widened to two lanes because the event peak ramp volume (894) is nearly as high as the event peak southbound through volume (1,028).

Community / Environmental Impacts – Right-of-way could be an issue for construction of the third lane and/or widening the ramp since the available right-of-way is limited west of Blankenbaker Parkway and there is a steep embankment on the west side of the roadway. In addition, there is a gas station located just off of Blankenbaker Parkway in that vicinity.

Costs – The third lane could be costly due to the steep embankment on the west side of the roadway and the need to tie into the existing on-ramp. Ramp widening would also likely be costly due to the extensive ramp merge and taper areas that could be required on I-64 westbound. Overall, this could be a beneficial project, but may difficult and costly to implement. If, in the future, improvements are made in this area on Blankenbaker Parkway it may be feasible to complete this project in conjunction with any future improvements.

8.7 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed I-64 / Blankenbaker Parkway Interchange alternatives is provided in Table 8-8.

Table 8-8: I-64 / Blankenbaker Parkway Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Install Traffic Signal at I-64 EB Ramps / Blankenbaker Parkway Intersection	●	◗	◗	●	●	●	**YES**
2	I-64 WB Off-Ramp Improvements – Dual Left and Right Turn Lanes	●	●	●	◗	◗	○	**YES**
3	Blankenbaker Parkway / Ellingsworth Lane Intersection Improvements – Dual WB Lefts, Exclusive NB, EB, and WB Rights, Signal Optimization, and Extend NB Left Turn Lane	●	●	◗	◗	●	◗	**YES**
4	Blankenbaker Parkway / Bluegrass Parkway Intersection Improvements – Signal Timing Optimization and Phasing / Lane Improvements to allow for WB free-flow right turn movement	◗	◗	◗	◗	◗	●	**YES**
5	Third SB Lane from Southeast Christian Church to I-64 WB On-Ramp	◗	●	◗	◗	◗	○	**NO (Could be Considered in Future)**

Ratings Guide: ○ **= Poor** ◗ **= Fair** ● **= Good**

8.8 RECOMMENDATION AND PHASING

For this interchange and the intersections within the study area, several improvements are recommended to address level of service, queuing, and safety issues.

- Install traffic signal at the intersection of Blankenbaker Parkway / I-64 Eastbound On-Ramp and lengthen the southbound left turn lane back to the bridge.

- Construct dual left and right turn lanes on the I-64 Westbound Off-Ramp. The right turn lanes are proposed to be a minimum of 750 feet and the left turn lanes are proposed to be a minimum of 550 feet, but longer lengths for both could beneficial.

- At the Blankenbaker Parkway / Ellingsworth Lane intersection, construct dual westbound left turn lanes, an exclusive northbound right turn lane, separate the westbound and eastbound right turn lanes, and optimize the traffic signal. For the westbound dual lefts, these should be constructed to extend 500 feet back from Blankenbaker Parkway. (Construction of the second westbound left turn lane and the northbound right turn lane are the developer's responsibility.)

- Signal optimization at Blankenbaker / Bluegrass Parkway intersection and provide improvements to allow for a free flow right turn movement from Bluegrass Parkway to northbound Blankenbaker Parkway. This includes widening the westbound right turn lane on Bluegrass Parkway and providing a third lane for a distance of 600 feet on Blankenbaker Parkway. If possible, the receiving lane could be extended further north, possibly to the I-64 Eastbound On-Ramp. (Construction costs to provide the free-flow right turn lane are the developer's responsibility.)

Installation of a traffic signal at the intersection of Blankenbaker Parkway / I-64 Eastbound On-Ramp is currently warranted, and it is recommended that it be installed by 2006 assuming traffic continues to grow as expected in the next few years. As for the other improvements, it is recommended that they be completed as soon as funding is available.

9.0 KY 841 / STONE STREET ROAD INTERCHANGE

9.1 INTRODUCTION AND STUDY AREA

The study area for the KY 841 / Stone Street Road interchange consists of the intersections listed below. Refer to Figure 9-1 for the limits of the study area.

1. Stone Street Road / KY 841 Eastbound Ramps
2. Stone Street Road / KY 841 Westbound Ramps

9.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns
The majority of traffic flow for this interchange is between the north and east directions. Traffic volumes are relatively low throughout the interchange, particularly the Stone Street Road / KY 841 Eastbound Ramps intersection which has low traffic volumes except for the southbound left-turn movement onto KY 841 eastbound.

Geometrics / Right-of-way
An evaluation of the existing interchange features revealed the following:

- The interchange is a simple diamond without traffic signals.
- There is a railroad line that crosses Stone Street north of the interchange, but it appears to have minimal affect on the interchange operations.
- The exit ramps are single lane ramps, but the westbound ramp widens to two lanes 450 feet before the intersection (the eastbound exit ramp flares at the intersection).

Land Use, Future Development, and Historic Traffic Growth
In the immediate vicinity of the interchange, there is limited development. The topography around the interchange includes some steep slope areas which may be a limiting factor for development in the area. An analysis of historic traffic volumes for KY 841 showed annual increases of approximately 6-7% between 1984 and 2004. Stone Street Road is not a state highway; therefore historic volume data was not available.

Traffic Operations / Level of Service Analysis
Peak period turning movement counts were conducted in October 2004. Follow-up field observations were conducted in February and April 2005. For the two key intersections, AM and PM peak hour volumes are shown on Figure 9-1. Existing levels of service and delay using the highway capacity manual method are shown on Table 9-1.

Table 9-1: 2004 Intersection Levels of Service for KY 841 / Stone Street Road

			AM		PM	
Intersection	**Type**	**Approach**	**Avg. Delay**	**LOS**	**Avg. Delay**	**LOS**
Stone Street Rd / KY 841 EB Ramps	Unsignalized	Eastbound Left	**372.2**	**F**	**145.0**	**F**
Stone Street Rd / KY 841 WB Ramps	Unsignalized	Westbound Left	23.3	C	18.3	C
		Westbound Right	12.2	B	**62.9**	**F**

FIGURE 9-1: KY 841 & STONE STREET ROAD INTERCHANGE

KEY ISSUES / DEFICIENCIES

- Poor levels of service on ramps from KY 841 to Stone Street Road.
- Field observations showed that the right turn movement on the WB Exit Ramp backs up frequently in the PM peak period with queue lengths averaging 10 vehicles. Vehicles turning left from the WB Exit Ramp experienced some delay while waiting for an adequate clearance gap, however, no queue lengths longer than 3 or 4 vehicles were observed.

LEGEND

- EXISTING EDGE OF PAVEMENT
- EXISTING EDGE OF TRAVEL WAY
- EXISTING RIGHT OF WAY
- SIGNALIZED INTERSECTION
- STOP: STOP-CONTROLLED INTERSECTION
- 67,800: 2004 AVERAGE DAILY TRAFFIC
- 980 (1080): 2004 AM (PM) PEAK HOUR VOLUMES
- 3% T: PERCENT TRUCKS
- B/E: 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600
GRAPHIC SCALE IN FEET

While the HCM method shows the left turn from the eastbound off-ramp operates at LOS F, no significant queues or delays were observed at this intersection during the count and subsequent follow-up observation periods. Therefore, the traffic conditions at the eastbound ramp intersection do not appear to be as poor as indicated in Table 9-1. Furthermore, it is important to note that the volume of traffic turning left from the eastbound exit ramp is relatively modest at 20 vehicles in the AM peak hour and 59 vehicles in the PM peak hour.

The westbound ramp intersection also shows a poor level of service in the PM peak period for the right-turn movement. This poor operating condition was observed on more than one occasion, with average delays even longer than that shown on at least one occasion. Queue lengths for this movement were also evaluated using the HCM method to determine if the current storage is exceeded during peak periods. The current storage length for left and right turning vehicles is approximately 450 feet for each lane. The 95th percentile queue is shown on Table 9-2. The calculated queue length exceeds the storage for the WB right turn in the PM peak. Field observations performed on February 16 and 17 confirmed that vehicles back up to near the KY 841 mainline for the right turn onto Stone Street (but were never observed backing onto the mainline). This occurrence was not observed on every weekday that field staff was present, but was observed on more than one occasion. The delay during these times was greater than that indicated by the highway capacity software in Table 9-1. There was little delay or queuing observed for westbound left turning vehicles.

Table 9-2: Queue Length Evaluation for Stone Street Road / KY 841 WB Ramps Intersection

Approach / Movement	Design Hour	95th Percentile Queue	Queue Length (ft)	Available Storage Length (ft)	Notes
WB Right	AM	2.8	70	450	Does **NOT** exceed available storage
	PM	22.5	563	450	**EXCEEDS** available storage

Safety / Crash Analysis

The crash analysis for KY 841 did not show a crash rate problem for that highway. Detailed crash information was not available for Stone Street Road since it is a local road. Lines-of-sight at the two intersections appear to be adequate.

Key Issues / Deficiencies

Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor operating conditions and long delays at the study intersections, especially on the westbound exit ramp from KY 841 to Stone Street Road.
- Field observations showed that the right turn movement on the westbound exit ramp backs up frequently in the PM peak period.
- Vehicles turning left from the eastbound exit ramp experienced some delay while waiting for an adequate clearance gap; however, queue lengths were very short if present at all.

9.3 RANGE OF ALTERNATIVES

A number of potential improvement alternatives were developed to address the identified deficiencies. They include:

- **Alternative 1A** – Install traffic signal at the Stone Street Road / KY 841 eastbound off-ramp intersection
- **Alternative 1B** – Install traffic signal at the Stone Street Road / KY 841 westbound off-ramp intersection
- **Alternative 2** – Add a northbound auxiliary lane on Stone Street Road to better accommodate right turning traffic from the westbound KY 841 exit ramp. The right-turn would be converted from a STOP control to a free-flow movement with appropriate channelization and signage.
- **Alternative 4** – Extend the turn lanes on the KY 841 eastbound exit ramp to increase vehicle storage.

Figure 9-2 shows these alternatives on an aerial photo.

9.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1A – Install Traffic Signal at Stone Street Road / KY 841 Eastbound Ramps

Traffic and Safety –

Level of Service Analysis – According to the HCS method, the eastbound left movement for the intersection experiences significant delay and poor level of service during the AM and PM peak periods. The addition of a signal would improve the levels of service to LOS C or better for all movements (LOS B overall).

Queue Length Analysis – There do not appear to be major queuing issues at this intersection today, though the HCM method does show 95th percentile queues extending back about 100 to 120 feet for the eastbound left. With the installation of a signal the maximum queue drops to 75 feet.

Signal Warrant Analysis – A traffic signal warrant evaluation was also performed to determine if the intersection meets or exceeds any of the MUTCD signal warrants. According to the MUTCD, there are eight warrants used to justify the installation of a traffic signal, four of which are relevant to this intersection. These four warrants are listed below along with a brief definition and a discussion of how they compare to the given conditions.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Only four hours of data was collected during the original traffic count, therefore there is insufficient data to determine if the 8-hour warrant is met. If signalization of this

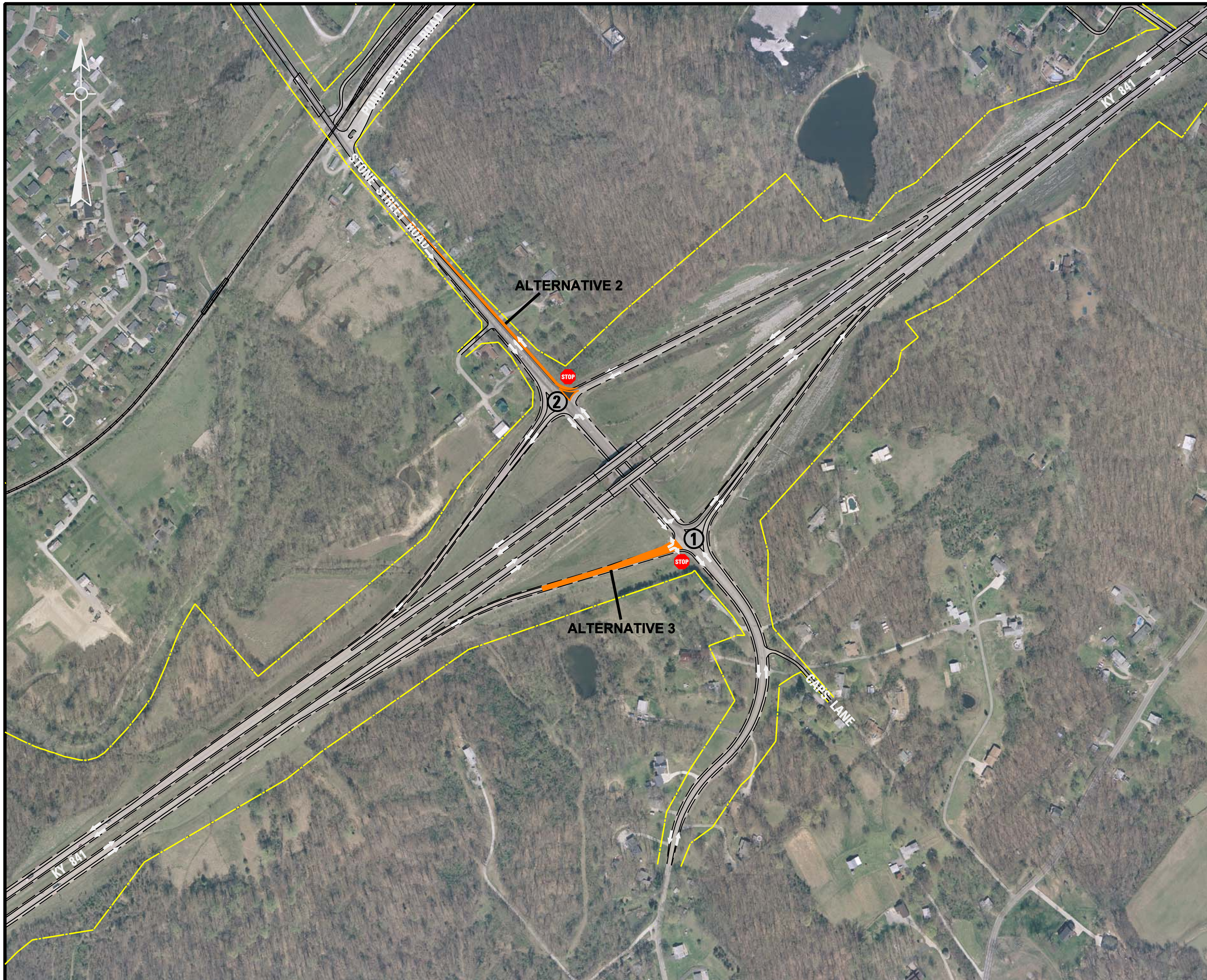


FIGURE 9-2: KY 841 & STONE STREET ROAD INTERCHANGE

ALTERNATIVES

- Alt. 1A - Install Traffic Signal at KY 841 EB Off-Ramp / Stone Street Road (Int. 1)
- Alt. 1B - Install Traffic Signal at KY 841 WB Off-Ramp / Stone Street Road (Int. 2)
- Alt. 2 - Construct NB Auxiliary Lane for Traffic Turning Right onto Stone Street Road from the WB KY 841 Off-Ramp
- Alt. 3 - Add Turn Lanes to KY 841 EB Off-Ramp

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

intersection is selected as a recommended alternative, additional fill-in counts should be collected to provide justification for intersection signalization.

- Warrant 2: Four-Hour Vehicular Volume – For this analysis, the eastbound off-ramp approach was the minor street and Stone Street is the major street. The four hours of data obtained during the AM and PM traffic counts were used as the basis for this warrant analysis. Figure 4C-2 in the MUTCD was used as the threshold curve. The traffic volumes for all four hours did not plot above the threshold curve shown for an intersection with two lanes on the major approach and one lane on the minor approach. **Based on these traffic volumes, this warrant is not met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must be such that they exceed the given threshold curve as shown on Figure 4C-4 in the MUTCD. From the traffic count data, the highest peak hour is from 7-8 AM. The traffic volumes during this hour plot below the threshold curve. **Therefore, this warrant is not satisfied.**
- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection. Because Stone Street Road is a local road, crash information was not available. As a result, there is insufficient data to determine if this warrant is met.

Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $125,000 in year 2005 dollars.

Overall, the benefit of this signal installation is small and it would likely increase overall intersection delay. Furthermore, it does not meet the two traffic volume warrants for which data is available. Therefore, this signal installation is not recommended.

Alternative 1B – Install Traffic Signal at Stone Street Road / KY 841 WB Ramps

Traffic and Safety –

<u>Level of Service Analysis</u> – The existing level of service analysis showed that the westbound right-turn movement experiences significant delay and a poor level of service in the PM peak period. Signalizing the intersection (using the same traffic volumes and intersection configuration) results in LOS C or better for all movements. Refer to Table 9-3 for more details.

Table 9-3: Alternative 1B Level of Service and Delay Comparison for Stone Street Road / KY 841 WB Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Stone Street Rd / KY 841 WB Ramps	Existing Unsignalized	Westbound Left	23.3	C	18.3	C
		Westbound Right	12.2	B	**62.9**	**F**
	Signalized	Westbound	3.1	A	24.1	C
		Northbound	14.1	B	30.0	C
		Southbound	9.1	A	3.6	A
		Whole Int.	**7.5**	**A**	**17.1**	**B**

Queue Length Analysis – Based on the level of service analysis, the average delay is fairly low. However, given the single lane and the high right-turn volume the 95th percentile queue can still be expected to extend up the ramp past the end of the current left turn lane. Essentially, the signal will address the delay issue, but long queues may still build.

Signal Warrant Analysis – A traffic signal warrant evaluation was performed to determine if the intersection meets or exceeds any of the signal warrants as outlined in the Manual of Uniform Traffic Control Devices (MUTCD). The three warrants which are most relevant to this intersection are discussed below.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. Initially, only four hours of data was collected during the original traffic count. To determine if this warrant is met, a fill-in traffic count was conducted on March 22, 2005 from 9:00 AM to 4:00 PM. Assuming speeds in excess of 40 mph on Stone Street, the volumes exceed the reduced threshold values on Table 4C-1 for Condition A. **Therefore, this warrant is met** (assuming the high speed reduction).
- Warrant 2: Four-Hour Vehicular Volume – The westbound off-ramp is the minor street and Stone Street is the major street. The traffic volumes for the four highest hours plotted above the threshold curve (Figure 4C-2) for an intersection with one lane on the major approach and two lanes on the minor approach. **Based on these traffic volumes, this warrant is currently met.**
- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must be such that they exceed the given threshold curve as shown on Figure 4C-4 in the MUTCD. From the traffic count data, the highest peak hour is from 4-5 PM. The traffic volumes during this hour plotted above the threshold curve. **Therefore, this warrant is satisfied.**

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $125,000 in year 2005 dollars.

Alternative 2 – Construct Northbound Through Lane for Traffic Turning Right onto Stone Street Road from the Westbound KY 841 Off-Ramp

Traffic and Safety – For this interchange, the westbound right turn movement carries the highest volume of traffic (in the PM peak period). This results in a poor level of service for this movement as well as long delays, and a queue length that exceeds the available storage for that lane. In an attempt to reduce delay and the queue lengths, the construction of a northbound auxiliary lane for westbound right turning traffic was proposed. This alternative would allow the right-turn movement to operate as a free-flow lane. Drivers would not have to wait for an acceptable gap in traffic to complete the turn. A drawback of this alternative is that pedestrians on that side of the roadway would have to cross a free-flow ramp, however few if any pedestrians were observed during the count periods. Overall, this option would improve the delay and level of service for the right-turn movement. Given the relatively modest cost, this alternative is recommended for additional more detailed examination and potential implementation.

Community / Environmental Impacts – Right-of-way is somewhat limited along Stone Street in this area, however the addition of a single auxiliary lane may be possible without further right-of-way acquisition. The existing residential driveways would be tied back into the widened roadway. There are no known environmental issues associated with the proposed project.

Costs – The estimated order of magnitude cost for this alternative is approximately $200,000 in year 2005 dollars.

Alternative 3 – Extend Turn Lanes on KY 841 Eastbound Off-Ramp

Traffic and Safety – According to the existing conditions level of service analysis, the eastbound left turn off of the ramp experiences poor levels of service (LOS F) and long average delay in both the AM and PM peak periods. Currently, the eastbound Off-ramp is one lane that flares out at the intersection approach to provide room for two vehicles (right and left turning traffic). The ramp could be widened to two lanes to provide a separate lane for the left turn movement and the right turn movement. This would provide additional capacity for vehicles turning left. Evaluation of traffic volumes on this ramp revealed that they are very low (the highest volume is 59 vehicles during the PM peak period for the left turn movement). Widening the ramp to provide additional storage will not improve intersection LOS and few queues were actually observed.

Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $130,000 in year 2005 dollars.

Again, this alternative seems unwarranted given the low ramp traffic volumes and lack of observed queues. It is therefore not recommended at this time.

9.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A graphical summary evaluation of the proposed KY 841 / Stone Street Road interchange alternatives is provided in Table 9-4.

Table 9-4: KY 841 / Stone Street Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1A	Install Traffic Signal at KY 841 EB Off-Ramp / Stone Street Road	◗	◗	◗	◗	●	●	**NO**
1B	Install Traffic Signal at KY 841 WB Off-Ramp / Stone Street Road	◗	◗	◗	◗	●	●	**NO**
2	Construct NB Through Lane for Traffic Turning Right onto Stone Street Road from the WB KY 841 Off-Ramp	●	●	●	◗	○	◗	**YES**
3	Add Turn Lanes to KY 841 EB Off-Ramp	◗	◗	○	◗	●	●	**NO**

Ratings Guide: ○ = Poor ◗ = Fair ● = Good

9.6 RECOMMENDATION AND PHASING

For this interchange there is only one intersection that requires improvement: the Stone Street / KY 841 Westbound ramps intersection. To facilitate the right turn from the ramp onto Stone Street there are two possible options – install a signal or add the right turn into a northbound auxiliary lane. Of these the auxiliary lane appears to offer the best operating condition for this relatively undeveloped low traffic area, handling what is one of the two heaviest flows through the entire interchange. None of the other proposed projects are recommended at this time.

10.0 I-65 / BROOKS ROAD (KY 1526) INTERCHANGE

10.1 INTRODUCTION AND STUDY AREA

The study area for the I-65 / Brooks Road (KY 1526) interchange consists of the intersections listed below. Refer to Figure 10-1 for the limits of the study area.

1. Brooks Road (KY 1526) / Willabrook Drive / East Blue Lick Road
2. Brooks Road (KY 1526) / I-65 Southbound Ramps
3. Brooks Road (KY 1526) / I-65 Northbound Ramps

10.2 EXISTING CONDITIONS

Current Traffic Volumes and Traffic Patterns

The current average daily traffic volumes for I-65 came from the Highway Information System (HIS) database, and are listed below.

- Approximately 10,400 ADT on Brooks Road west of I-65
- Approximately 18,700 ADT on Brooks Road east of I-65

The major traffic flows though the interchange are toward the north in the AM and from the north in the PM. Truck traffic makes up a substantial portion of the traffic volumes through the interchange. This is mainly due to the proximity of a truck stop southwest of the interchange. Peak hour truck percentages on Brooks Road west of the interchange were 14% in the AM and 11% in the PM.

Geometrics / Right-of-way

An evaluation of the existing interchange features revealed the following:

- Typical diamond interchange with off ramps widening to two lanes near the intersection approaches
- The intersection of Brooks Road / East Blue Lick Road is somewhat atypical in the northbound direction with two separate STOP controlled approaches leading up to the intersection (one on East Blue Lick Road and one from the Pilot gas station [Sarah Way]). Safety and access at this location are concerns, particularly with the high volume of trucks that use this station as a truck stop.

Land Use, Future Development, and Historic Traffic Growth

As mentioned before, a truck stop / travel station is located southwest of the interchange. Future development in the area includes a Jewish Hospital facility planned for the southeast corner of the interchange and a 65 room Holiday Inn Express planned for the northeast corner. Overall, the county growth is strong. Analysis of historic traffic volumes showed that traffic in the vicinity of the interchange has been growing at approximately 5% per year since 1990 with slightly less growth on Brooks Road west of I-65 (3% per year).

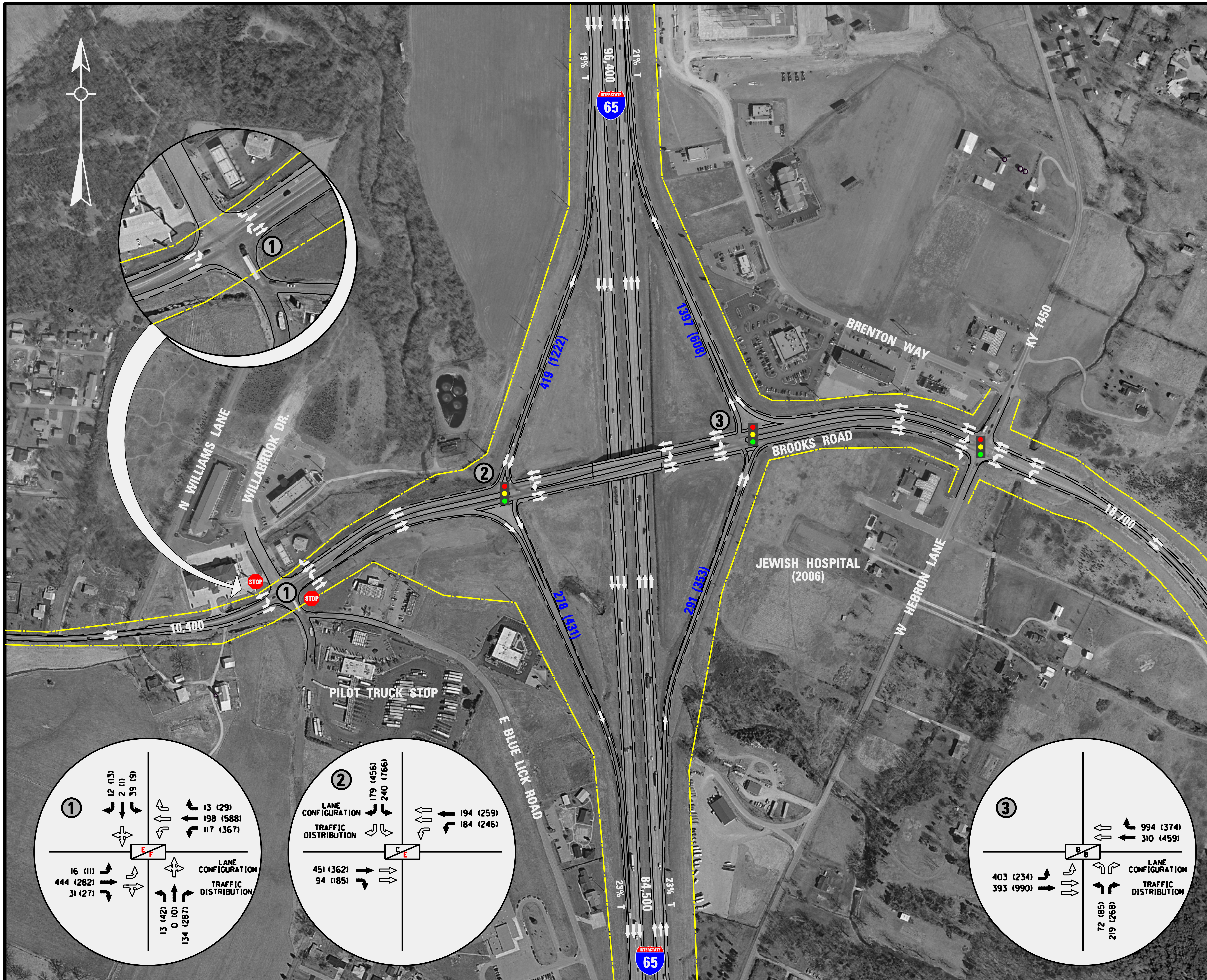


FIGURE 10-1: I-65 & BROOKS ROAD (KY 1526) INTERCHANGE

KEY ISSUES / DEFICIENCIES

- Poor levels of service at two of the intersections
- Poor access and circulation in the vicinity East Blue Lick Road
- Safety concerns for KY 1526

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION
67,800 2004 AVERAGE DAILY TRAFFIC
980 (1080) 2004 AM (PM) PEAK HOUR VOLUMES
3% T PERCENT TRUCKS
B/E 2004 LEVEL OF SERVICE (AM/PM)

200 0 200 400 600
GRAPHIC SCALE IN FEET

Traffic Operations / Level of Service Analysis

Peak period turning movement counts were conducted on 10/12/04 and 10/13/04. Turning movement counts for Brooks Road / East Blue Lick Road were provided by KYTC. For each of the key intersections, AM and PM peak hour volumes are shown on Figure 10-1. Existing levels of service and delay are shown on Table 10-1.

Table 10-1: 2004 Intersection Levels of Service for I-65 / Brooks Road

Intersection	Type	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Brooks Road / East Blue Lick Road	Two-Way STOP Controlled	EB Left	7.7	A	9.0	A
		WB Left	9.8	A	10.1	B
		Northbound	19.0	C	**345.9**	**F**
		Southbound	**45.5**	**E**	**183.3**	**F**
Brooks Road / I-65 SB Ramps	Signalized	Eastbound	44.8	D	44.1	D
		Westbound	17.4	B	17.7	B
		Southbound	27.2	C	**119.6**	**F**
		Whole Int.	**30.7**	**C**	**78.1**	**E**
Brooks Road / I-65 NB Ramps	Signalized	Eastbound	12.3	B	12.1	B
		Westbound	29.4	C	31.1	C
		Northbound	24.4	C	25.0	C
		Whole Int.	**17.9**	**B**	**18.2**	**B**

As shown on Table 10-1, the two-way stop-controlled intersection at East Blue Lick Road operates poorly during both peak periods. In the PM peak, the northbound queue also exceeds the small available storage. Specifically, the queue backs up through the two approaches to this leg of the intersection.

The Brooks Road / I-65 Southbound off-ramp intersection also operates poorly in the PM peak. Significant queues form on the ramp during this period, sometimes extending back to the vicinity of the mainline.

Safety / Crash Analysis

The crash analysis did show a crash rate problem on Brooks Road (see Table 10-2). Nearly half of the crashes in the section occurred west of the southbound ramps and half of these were angle crashes. The fatal crash on KY 1526 occurred on March 20, 2002, during the middle of the day, east of the interchange near the intersection of KY 1526 and KY 1450.

Table 10-2: 2001 – 2003 Crash Analysis for I-65 / Brooks Road Interchange

Highway	Crashes in Study Area			Section Crash Rate	Statewide Ave. Crash Rate	Statewide Critical Crash Rate	Critical Rate Factor*
	Total	Injury	Fatal				
I-65	**47**	**13**	**1**	**53**	**74**	**114**	**0.47**
KY 1526	**108**	**38**	**1**	**636**	**272**	**589**	**1.08**

Sources: Crash data from KYTC, Statewide Rates from KTC Research Report KTC-04-25/KSP2-04-1F, Analysis of Traffic Crash Data in Kentucky (1999 - 2004)
*Critical rate factor is section rate / statewide critical rate

Key Issues / Deficiencies
Based on the existing conditions analysis, the key issues / deficiencies are:

- Poor levels of service at two of the intersections
- Poor access and circulation in the vicinity of East Blue Lick Road
- Queuing issues (northbound on East Blue Lick Road and southbound on the I-65 southbound exit ramp)
- Safety concerns for KY 1526 (especially just west of the interchange)

10.3 RANGE OF ALTERNATIVES

To address identified traffic and safety issues, the following alternatives are proposed:

- **Alternative 1** – Install signal at Brooks Road / East Blue Lick Road
- **Alternative 2** – Redesign of Northbound approach at Brooks Road / East Blue Lick Road
- **Alternative 3** – I-65 Southbound Off-Ramp Improvements

Figure 10-2 shows each of these alternatives on an aerial photo of the study area.

10.4 ANALYSIS AND EVALUATION OF ALTERNATIVES

Alternative 1 – Install Traffic Signal at Brooks Road / East Blue Lick Road

Traffic and Safety –

<u>Level of Service Analysis</u> – As shown in the existing conditions analysis, the northbound movement has a poor LOS in the PM peak (LOS F), and the southbound movement is LOS E and LOS F in the AM and PM peak periods respectively. Using the same traffic volumes and lane configurations, the intersection was analyzed with a signal. The levels of service for the whole intersection indicate that it would operate acceptably with the new signal (LOS B in the AM peak and LOS C in the PM peak).

<u>Queue Length Analysis</u> – As discussed previously in the existing conditions analysis, the current northbound approach queues on East Blue Lick Road exceed the available storage during the PM peak period. The installation of a traffic signal will not improve this condition. Queues will still exceed the approximately 60 feet available between the intersection and the two STOP controlled legs to the south. Also, as a result of installing the signal, queues exceeding the available storage in the westbound left turn lane are likely to develop. The maximum queue length is 350 feet during the PM peak period; therefore the westbound left turn lane should be extended to 350 - 400 feet in total length to accommodate the maximum expected queue.

FIGURE 10-2: I-65 & BROOKS ROAD (KY 1526) INTERCHANGE

ALTERNATIVES

- Alt. 1 - Install Traffic Signal at Brooks Road / East Blue Lick Road Intersection (Int. 1) and Lengthen WB Left Turn Lane
- Alt. 2 - Redesign Northbound Approach at Brooks Road / East Blue Lick Road - New Signage and Striping
- Alt. 3 - I-65 Southbound Off-Ramp Improvements Including Signal Optimization and Dual Left Turn Lanes

LEGEND

EXISTING EDGE OF PAVEMENT
EXISTING EDGE OF TRAVEL WAY
EXISTING RIGHT OF WAY
SIGNALIZED INTERSECTION
STOP-CONTROLLED INTERSECTION

200 0 200 400 600
GRAPHIC SCALE IN FEET

Signal Warrant Analysis – A traffic signal warrant evaluation was performed to determine if the intersection meets any of the MUTCD signal warrants. Of the eight MUTCD warrants used to justify the installation of a traffic signal, four are relevant to this intersection. These four warrants are listed below along with a brief definition and a discussion of how they compare to the given conditions. For Warrants 1, 2, and 3, the 70% factor threshold volumes were used since the speed on the major road (Brooks Road) exceeds 40 mph.

- Warrant 1: Eight-Hour Vehicular Volume – To satisfy this warrant, a minimum hourly volume must be exceeded for eight hours during an average day. This study used 12 hours of turning movement counts collected by KYTC in August of 2004. **Based on these traffic volumes, this warrant is currently met.**

- Warrant 2: Four-Hour Vehicular Volume – For this analysis, the East Blue Lick Road / Willabrook Drive approaches were considered to be the minor street. For each hour, the approach with the highest volumes was used. The four hours of data obtained during the AM and PM traffic counts were used as the basis for this warrant analysis. Figure 4C-2 in the MUTCD was used as the threshold curve. The traffic volumes for all four hours plotted above the threshold curve shown for an intersection with one lane on the major approach and one lane on the minor approach. **Based on these traffic volumes, this warrant is currently met.**

- Warrant 3: Peak Hour – For this warrant, traffic volumes during one hour must be such that they exceed the given threshold curve as shown on Figure 4C-4 in the MUTCD. From the traffic count data, the highest peak hour is from 3-4 PM. The traffic volumes during this hour plot above the threshold curve. **Therefore, this warrant is satisfied.**

- Warrant 7: Crash Experience – This warrant is used when the primary reason for installing a signal is due to a history of severe and frequent crashes in the vicinity of the intersection. The crash rate analysis did show that there is a crash rate problem on Brooks Road. Within 0.05 miles of this intersection, 26 crashes occurred between 2001 and 2003. One crash was on the west side of the intersection, and the remaining 25 occurred on the east legs of the intersection. Of the 26 crashes, 17 were angle crashes that may have been prevented by a traffic signal at this location. According to this crash analysis, this warrant may be met, but should not be the only basis for signal installation. Other measures to reduce the crash problem should be investigated before this warrant is used as justification for a traffic signal.

Based on the above analysis, installation of a signal is justified at this location. The traffic volumes are high enough such that Warrants 1, 2 and 3 are met, and there is sufficient evidence that there is a crash problem in the vicinity of the intersection, thereby providing possible justification for Warrant 7. In addition, installation of a traffic signal will improve intersection operations as shown by the improvements in LOS. The westbound left turn lane will need to be lengthened to accommodate potential queues resulting from installation of the signal. The only operational deficiency identified that signal installation will not address is the northbound queuing problems on East Blue Lick Road.

Community / Environmental Impacts – There are no known adverse impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $190,000 in year 2005 dollars.

Alternative 2 – Redesign Northbound Approach at Brooks Road / East Blue Lick Road

Traffic and Safety – As a result of potential development south and east of East Blue Lick Road, development plans have been prepared by QK4 that propose some improvements to this approach. They primarily consist of improved signage and striping to define the actual travel lanes. This would be a useful improvement to reduce driver confusion and improve safety for this approach. The striping would prohibit vehicles from pulling alongside another vehicle at the approach to Brooks Street, and would define the entrance / exits to the gas station.

While the proposed improvements would increase the safety for this approach, they do nothing to improving the queuing issue. One possible option to provide adequate vehicle storage for the northbound approach would be to restrict traffic flow on Sarah Way. Northbound traffic would be prohibited, thereby forcing traffic headed back to the interchange to make a left from a new exit on East Blue Lick Road to leave the gas station. The problem with this option would be that if development occurs south and west of the gas station, any traffic headed north towards Brooks would have to have access on East Blue Lick Road instead of Sarah Way. A second option would be to realign East Blue Lick Road south of the gas station. This provides a longer approach to Brooks and greater storage length for turning vehicles. The old portion of East Blue Lick Road could potentially be left open for local traffic to the gas station and businesses located north of the realignment. A third option would be to buy the gas station and any surrounding property and reconstruct the approach to form a standard four-leg intersection.

Because of the potential property and development impacts and high expected cost of these options, the preferred short term improvement alternative is to enhance safety through new signage and striping as proposed by QK4 and do nothing to increase vehicle storage for this approach. However, with the significant ongoing development and the fact that more development is planned for the area, this intersection could potentially be improved as part of a larger project in the area to provide good access in the future.

Community / Environmental Impacts – There are no known impacts associated with the proposed signage and striping plans. Significant impacts are expected with any of the potential options to increase vehicle storage on East Blue Lick Road, including major property impacts.

Costs – The estimated order of magnitude cost for this alternative is $30,000 in year 2005 dollars. The cost estimate is based on signage and striping improvements only.

Alternative 3 – I-65 Southbound Off-Ramp Improvements

Traffic and Safety – A review of the existing conditions analysis showed that the Brooks Road / I-65 Southbound Ramps intersection has a poor level of service for the southbound movement during the PM peak period. Options to improve the overall operations for this movement include signal optimization, dual left turn lanes, and lengthening the left turn lane(s). The level of service analysis for signal optimization and the construction of dual left turn lanes is presented in Table 10-3 as well as the existing conditions level of service analysis for comparison.

Table 10-3: Alternative 3 Level of Service and Delay Comparison for Brooks Road / I-65 Southbound Ramps

Intersection	Scenario	Approach	AM Avg. Delay	AM LOS	PM Avg. Delay	PM LOS
Brooks Road / I-65 SB Ramps	Existing	Eastbound	44.8	D	44.1	D
		Westbound	17.4	B	17.7	B
		Southbound	27.2	C	**119.6**	**F**
		Whole Int.	**30.7**	**C**	**78.1**	**E**
	Signal Optimized	Eastbound	14.6	B	48.5	D
		Westbound	7.1	A	33.7	C
		Northbound	22.9	C	29.3	C
		Whole Int.	**14.4**	**B**	**34.5**	**C**
	Dual Lefts	Eastbound	44.8	D	44.1	D
		Westbound	17.4	B	17.7	B
		Northbound	25.1	C	31.0	C
		Whole Int.	**30.1**	**C**	**30.5**	**C**
	Signal Optimized and Dual Lefts	Eastbound	14.6	B	22.8	C
		Westbound	7.1	A	18.8	B
		Northbound	18.9	B	14.8	B
		Whole Int.	**13.3**	**B**	**17.6**	**B**

As shown in Table 10-3, simply optimizing the signal timing improves intersection operations to a LOS C for the PM peak. Adding a second left turn lane on the ramp and optimizing the signal further reduces the delay and improves the level of service from a LOS F to a LOS B. While the AM peak period volume of 240 may not be high enough to justify a second left turn lane, the PM peak period volume of 766 is very high.

To determine the impact of these improvements on reducing queuing for the left turn traffic on the ramp, queue lengths were evaluated for each improvement option. Table 10-4 shows the results of this analysis.

Table 10-4: Alternative 3 Queue Length Evaluation for Southbound Left on I-65 Southbound Off-Ramp

Approach / Movement	Design Hour	95th Percentile Queue	Queue Length (ft)	Available Storage Length (ft)	Notes
SB Left - Existing	AM	14.1	353	160	**EXCEEDS** available storage
	PM	80.4	2,010	160	**EXCEEDS** available storage
SB Left - Optimized	AM	9.4	235	160	**EXCEEDS** available storage
	PM	45.9	1148	160	**EXCEEDS** available storage
SB Left – Optimized and Dual Lefts	AM	4.3	108	160	Does **NOT** exceed available storage
	PM	12.6	315	160	**EXCEEDS** available storage

As shown in Table 10-4, the existing queue length for the southbound left turn movement greatly exceeds the available storage. However, there is additional storage on the ramp other than what is listed in the table. Prior to the two lane section is a single lane shared by both right and left turning traffic that extends back to the diverge on I-65. The length of the single lane portion of the ramp is approximately 1,400 feet, which combined with the storage length for the exclusive southbound left turn, is still less than the calculated queue length.

Optimizing the signal timing does reduce the southbound left queue, but it still remains very high during the PM peak period. Constructing dual left turn lanes reduces the queue to 315 feet in the PM peak, but this is still higher than the exclusive available storage length for this movement. To provide adequate storage for the southbound left turn movement, dual left turn lanes should be constructed and should be approximately 350 feet in length from the intersection with Brooks Street.

Based on this analysis, signal optimization would take care of the LOS problems at this intersection, but would not address the queuing issue for the southbound left turn traffic. To provide desirable traffic operations and adequate storage on the ramp, the signal timing should be optimized, the ramp should be widened to accommodate dual left turn lanes, and the turn lanes should be approximately 350 feet in length. Mainline detection and preemption was proposed at a project team meeting as a possible alternative, but if these improvements are implemented, this should not be necessary.

Community / Environmental Impacts – There are no known adverse environmental or community impacts associated with this alternative.

Costs – The estimated order of magnitude cost for this alternative is $370,000 in year 2005 dollars.

10.5 SUMMARY EVALUATION AND COMPARISON OF ALTERNATIVES

A comparison of the three alternatives proposed for improvements to the I-65 / Brooks Road Interchange area is listed in Table 10-5 below. For better comparison, the proposed improvements are further outlined by evaluation category in this table based on the analysis from the previous section.

Table 10-5: I-65 / Brooks Road Alternative Summary Evaluation and Comparison Matrix

Alt.	Description	Traffic				Community / Environmental Impacts	Cost	**Recommendation**
		Congestion	Operations	Use	Safety			
1	Install Traffic Signal at Brooks Road / East Blue Lick Road Intersection	●	●	◗	●	●	●	**YES**
2	Redesign of NB Approach at Brooks Road / East Blue Lick Road Intersection – New Signage and Striping	◗	◗	◗	◗	●	●	**YES**
3	I-65 SB Off-Ramp Improvements – Signal Optimization, Dual Lefts, Extended Turn Lanes	●	●	●	◗	●	◗	**YES**

Ratings Guide: ○ **= Poor** ◗ **= Fair** ● **= Good**

10.6 RECOMMENDATION AND PHASING

All three alternatives propose projects that should improve identified traffic and safety problems through this interchange. With minimal to no negative impacts to the community and environment and low costs, all seem to be feasible and cost-effective methods to improve traffic operations. Therefore, all three projects are recommended and are listed below.

- Install Traffic Signal at Brooks Road / East Blue Lick Road Intersection (and Lengthen Westbound Left Turn Lane to 350 – 400 feet)
- Redesign of Northbound Approach at Brooks Road / East Blue Lick Road through New Signage and Striping

- Improvements to I-65 Southbound Ramp and Intersection Including Signal Optimization, Dual Left Turn Lanes, and Extending the Turn Lanes to a Total Length of 350 feet.

Installation of a traffic signal at Brooks / East Blue Lick is proposed to address poor levels of service at this intersection as well as potentially reduce the crash problem on the east legs of the intersection. The westbound left turn lane should be lengthened at the time of the signal installation to prevent queues from backing up on Brooks Road and blocking the through traffic. To further improve safety at this intersection, the northbound approach should be resigned and restriped as proposed in the development plans by QK4. This should provide clearer direction for vehicles at the intersection and reduce conflict.

Signal optimization of the I-65 Southbound Ramp / Brooks Road traffic signal should be implemented first since it could reduce delay and queues on the ramp. To ultimately alleviate the long queues on this ramp, the dual left turn lanes should be constructed and extended for a length of 350 feet.