# FINAL

# I-4/SR 33 INTERCHANGE IMPROVEMENT CONCEPT REPORT

Polk County, Florida

## FLORIDA DEPARTMENT OF TRANSPORTATION
## DISTRICT ONE

## FINANCIAL PROJECT ID: 430185-1-22-01

February 2014

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# 1.0 INTRODUCTION

## 1.1 PROJECT LOCATION

The I-4/SR 33 interchange is located within the City of Lakeland incorporated limits in the northwest portion of Polk County. This interchange is located at Mile Marker 37.877 on I-4 and is designated as Exit 38. The I-4/SR 33 interchange is located approximately 2.5 miles to the west of the I-4/Polk Parkway (SR 570) interchange and approximately 3.4 miles to the east of the I-4/Socrum Loop Road (CR 582)/SR 33 interchange. The project location is illustrated in **Figure 1-1**.

## 1.2 PROJECT BACKGROUND

The I-4/SR 33 interchange was included in the FDOT District One I-4 Project Development & Environment (PD&E) Study that was approved by the Federal Highway Administration (FHWA) in 1999. The FHWA-approved PD&E concept for this interchange was to maintain the current diamond interchange configuration and laneage on SR 33 and widen the I-4 mainline. In the 10-year period following the I-4 PD&E study, several Developments of Regional Impact (DRIs) located in the vicinity of the I-4/SR 33 interchange have been prepared and approved. These DRIs include the Williams DRI, Bridgewater DRI, and Polk Commerce Center DRI. All three of these DRIs are mixed use developments that include single and multi-family residential, retail, and office/business park development.

The Williams DRI is located south of I-4 and east of SR 33 while the Polk Commerce Center DRI is located south of I-4 and east of the Polk Parkway. A majority of the Bridgewater DRI is located south of I-4 and west of SR 33; however, there is also a portion located on the east side of SR 33 (between Old Combee Road and SR 33 to the west of N. Combee Road). In addition, the proposed Rockefeller Group Park of Commerce development is located to the north of SR 33 and to the east of Tomkow Road on the 112-acre site of the former USA International Speedway. This planned development will consist of approximately 1.2 million square feet of warehousing, distribution, and light manufacturing.

In July 2008, FDOT District One initiated a preliminary interchange improvement feasibility study to identify both short-term and long-term improvements for the I-4/SR 33 interchange. This study was initiated at the request of the City of Lakeland due to growing concerns regarding the potential for negative impacts to occur at the existing interchange as a result of these DRI's. This interchange feasibility study was completed in April 2009; however, no subsequent interchange improvement studies were initiated upon the completion of the feasibility study.

In May 2012, FDOT District One initiated a PD&E study for the portion of SR 33 from Old Combee Road to north of Tomkow Road to document the need for widening this roadway, to determine the specific geometric improvements that should be implemented within the study corridor, and to


**FIGURE 1-1: PROJECT LOCATION MAP**

quantify the costs and environmental impacts of the recommended improvements. The SR 33 PD&E study limits include the interchange at I-4; therefore, the development and evaluation of alternative geometric improvements for the interchange was conducted as a part of this study. More recently, District One has programmed the final design of the I-4/SR 33 interchange improvements into FDOT's Approved Five-Year Work Program for FY 2013/2014.

## 1.3 PROJECT PURPOSE

As the planned development in the vicinity of the existing interchange occurs, the delay experienced by vehicles exiting the I-4 mainline and turning onto SR 33 (as well as the vehicles making the reverse movement) will increase significantly. This increased vehicle delay will result in longer vehicle queues which in turn, increases the potential for more vehicle crashes to occur in the interchange area. The purpose of this report is to document the long-term geometric improvements that should be implemented at the I-4/SR 33 interchange to ensure that this interchange has sufficient capacity to accommodate the future year traffic volumes that are projected to occur due to future residential and commercial development. The provision of additional capacity at this interchange is expected to reduce the potential for any negative impacts to occur on the I-4 ramps, as well as on the I-4 mainline upstream of the interchange off-ramps. This report has been prepared in accordance with FDOT Policy No. 000-525-015-f: Approval of New or Modified Access to Limited Access Facilities, FDOT Procedure No. 525-030-160-g: The Interchange Handbook, and FDOT Procedure No. 525-030-120-g: The Project Traffic Forecasting Handbook.

# 2.0 EXISTING GEOMETRIC CONDITIONS

## 2.1 ROADWAY

I-4 is a six-lane divided east-west limited access facility with a posted speed limit of 70 miles per hour (mph). I-4 is functionally classified as an urban principal arterial-interstate. The current median width is approximately 64 feet; however, the distance between the eastbound and westbound bridges in the center of the interchange is 80 feet. The existing right-of-way is generally 300 feet. The 1,500-foot crest vertical curve has a K-value of 250 which only allows for a maximum design speed of 55 mph based on Volume 1 of the FDOT's Plans Preparation Manual (PPM). In addition, the two 500-foot approach sag vertical curves have K-values of 167 which allow for a maximum design speed of 60 mph.

SR 33 is a two-lane undivided roadway both south and north of the I-4 interchange and has a posted speed limit of 60 mph. In general, SR 33 has a southwest-northeast orientation. For the purposes of this report, the portion of SR 33 between University Boulevard/Firstpark Boulevard N. and the westbound I-4 on-/off-ramps will be referred to as a north-south roadway. Since the portion of SR 33 north of the interchange has more of an east-west alignment, this portion will be referred to as an east-west roadway. SR 33 is functionally classified as an urban minor arterial. Within the interchange area, the northbound and southbound SR 33 travel lanes are separated by a raised grass median. Guardrail also exists in the median and on the outside of the travel lanes in the immediate vicinity of the I-4 bridge piers.

The existing I-4/SR 33 interchange is a rural diamond interchange configuration that has single lane on- and off-ramps in all four quadrants. I-4 crosses over SR 33 on a 135°/45° skew angle. Single left-turn and right-turn lanes are provided on SR 33 and on the I-4 off-ramps. The right-turn lanes on both SR 33 and the I-4 off-ramps are channelized. There is approximately 325 feet of left-turn vehicle storage provided on the westbound off-ramp prior to the beginning of the channelized right-turn lane. Based on an average vehicle spacing of 25 feet, the westbound right-turn vehicles are able to access the right-turn lane if the westbound left-turn queue is less than or equal to 13 vehicles. Similarly, there is approximately 125 feet of left-turn vehicle storage provided on the eastbound off-ramp prior to the beginning of the channelized right-turn lane. Based on an average vehicle spacing of 25 feet, the eastbound right-turn vehicles are able to access the right-turn lane if the eastbound left-turn queue is less than or equal to five vehicles.

The distance between the two unsignalized ramp terminal intersections is approximately 800 feet. The left-turn movements from the I-4 off-ramps onto SR 33 operate under stop sign control while the left-turn movements from SR 33 onto the I-4 on-ramps must yield to oncoming vehicles. The southbound left-turn lane for the eastbound I-4 on-ramp and the northbound left-turn lane for the westbound I-4 on-ramp both have approximately 50 feet of full width queue storage and 105 feet of taper. These left-turn lanes do not provide the minimum deceleration length that is required for a 60 mph roadway based on Standard Index 301 of the PPM. All four right-turn movements are channelized and controlled by yield signs. Currently, there are no acceleration/deceleration lanes provided on SR 33 for the right-turn movements. An aerial photograph of the existing I-4/SR 33 interchange is provided in **Figure 2-1**.

The University Boulevard/Firstpark Boulevard N. intersection is a four-legged intersection located approximately 0.48 miles south of the eastbound I-4 ramp terminal intersection. University

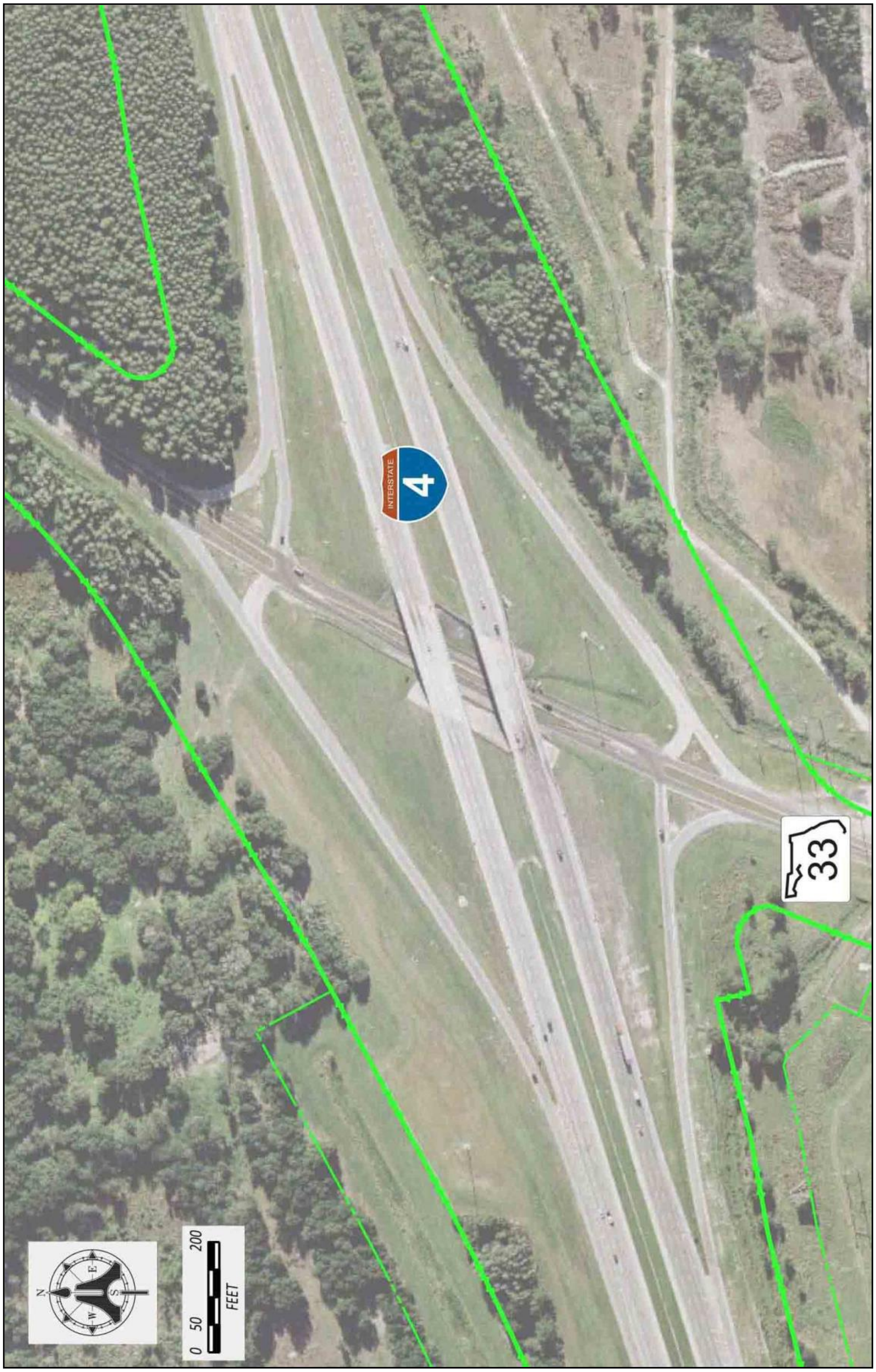


FIGURE 2-1: EXISTING I-4/SR 33 INTERCHANGE

Boulevard is a four-lane divided roadway located to the east of SR 33 that extends over to the Polk Parkway. This roadway was constructed and opened to traffic in 2012 to provide access to the Williams DRI and the USF Polytechnic Campus. Firstpark Boulevard N. is a two-lane undivided roadway located to the west of SR 33 that serves as the northern entrance/exit to the Firstpark at Bridgewater Industrial Park. Approximately 500 feet to the west of the intersection, this roadway transitions to a four-lane divided roadway. There is a traffic signal at this intersection; however, this signal is currently displaying flashing red for University Boulevard and Firstpark Boulevard N. and flashing yellow for SR 33.

The Tomkow Road intersection is an unsignalized T-intersection located approximately 0.20 miles north of the westbound I-4 ramp terminal intersection. Tomkow Road is a two-lane undivided roadway located to the northwest of SR 33 that extends northward to Old Polk City Road. Although Tomkow Road is a T-intersection, there is an entrance/exit to a small park-and-ride lot located on the south side of SR 33 approximately 30 feet to the west of Tomkow Road (centerline-to-centerline). Although there is no dedicated (i.e., marked) left-turn lane at the entrance to the park-and-ride lot, dashed lines extend from the eastbound designated left-turn lane (serving Tomkow Road) all the way across the Tomkow Road intersection and connect to the painted median/traffic separator on the other side of the intersection. Field observations indicated that most of the westbound SR 33 vehicles turning left into this lot were actually turning from this center lane and not from the westbound through lane. Even in the cases when an eastbound left-turn vehicle was waiting to turn onto Tomkow Road, the westbound left-turn vehicles were driving past the eastbound left-turn vehicles and accessing the center lane while waiting to make the left-turn movement (due to the offset of Tomkow Road and the park-and-ride lot entrance). In essence, this lane is operating as a “de-facto” two-way center left-turn lane even though it is not marked as one. **Figure 2-2** depicts the existing laneage for the I-4/SR 33 interchange as well as the University Boulevard/Firstpark Boulevard N. and Tomkow Road intersections.

## 2.2 BRIDGES

I-4 crosses over SR 33 via two independent structures (i.e., bridges). These structures were first constructed in 1961 and were subsequently widened from four lanes to six lanes in 2004 as a part of the I-4 six-lane Design/Build project that was completed in 2006. The eastbound I-4 bridge (Bridge No. 160182) was last inspected on July 26, 2012 and has a sufficiency rating of 96.4 and a health index of 88.60. The westbound I-4 bridge (Bridge No. 160181) was also last inspected on July 26, 2012 and has a sufficiency rating of 96.4 and a health index of 89.11. Both of these bridges are in good condition relative to their age (i.e., approximately 52 years). The existing vertical clearance over SR 33 is 14.9 feet and satisfies the minimum allowed by AASHTO (i.e., 14 feet) in highly developed urban areas if there is an alternate route that provides 16 feet of clearance. However, this existing vertical clearance is significantly less than the minimum required by the FDOT’s PPM (i.e., 16.5 feet) and as a result, the I-4 bridges are considered to be “functionally obsolete.” It should be noted that a vertical clearance design exception was approved by FHWA during the design of the six-lane bridges. The distance between the center pier and the two outside piers is 40 feet. Currently, the depth of the bridge pier footings is approximately 1.89 feet below grade which does not meet the 3-foot minimum requirement specified in Section 13.5 (Pier Details – Footings) of the Structures Detailing Manual. A typical section of SR 33 under the I-4 bridges is provided in **Figure 2-3**.

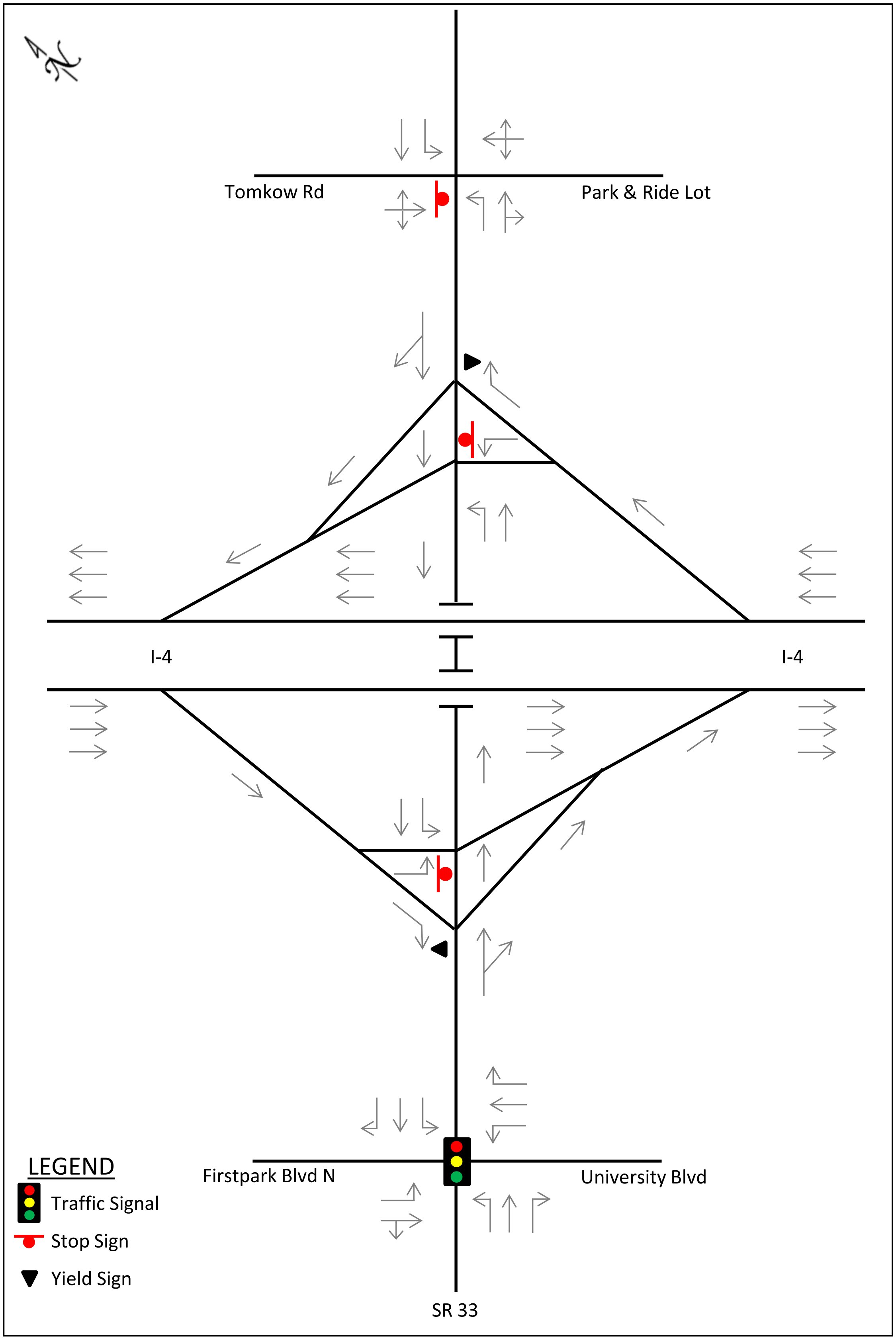


**FIGURE 2-2: EXISTING YEAR (2012) INTERCHANGE/INTERSECTION GEOMETRY**

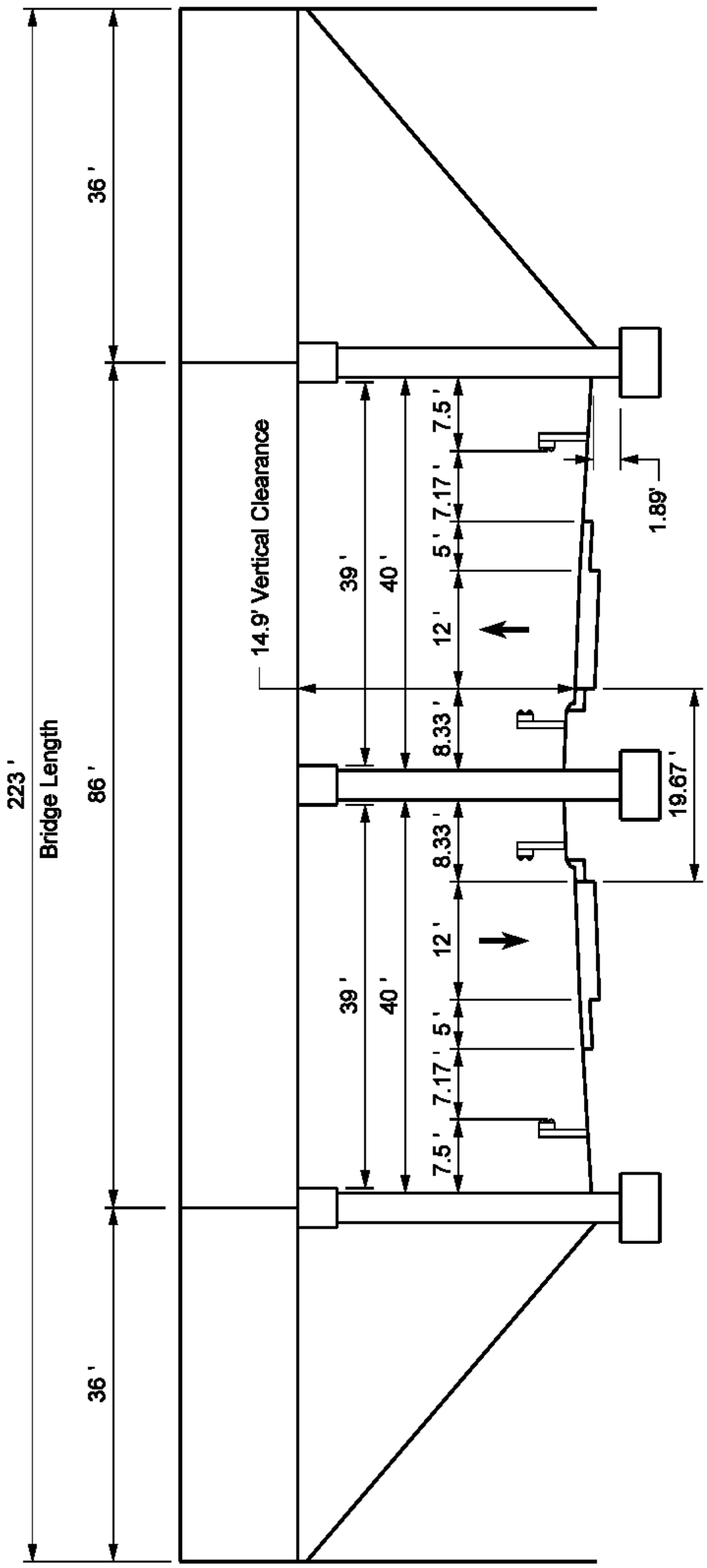


**FIGURE 2-3: EXISTING TYPICAL SECTION – SR 33 UNDER I-4 LOOKING NORTH**

# 3.0 METHODOLOGY

## 3.1 OVERVIEW

This section documents the methodologies that were used in the preparation of this report including the traffic data collection, travel demand forecasting, and traffic operations analyses. The methodologies that were used are consistent with the Methodology Letter of Understanding (MLOU) that was signed by FHWA on July 30, 2013. A copy of this MLOU is provided in **Appendix A**.

## 3.2 ANALYSIS YEARS

The analysis years used in this project are as follows:

- Existing Year – 2012
- Opening Year – 2016
- Design Year – 2036

These years correspond to the analysis years that were used in the SR 33 PD&E study. Additional interim year analyses were also conducted to estimate the approximate time frame when the capacity of the existing interchange was exceeded.

## 3.3 AREA OF INFLUENCE

The area of influence is illustrated in **Figure 3-1**. This area extends along the I-4 mainline from just west of the on-/off-ramps to just east of the on-/off-ramps and includes the ramp merge/diverge areas. The area of influence also extends along SR 33 from the University Boulevard/Firstpark Boulevard N. intersection to the Tomkow Road intersection.

## 3.4 TRAFFIC DATA COLLECTION/EXISTING TRAFFIC VOLUMES

A traffic count program was previously conducted in September and October of 2012 in support of the SR 33 PD&E study. Twenty-four (24) hour bi-directional volume counts were conducted on September 6, 2012 at the following locations:

- SR 33 south of University Boulevard/Firstpark Boulevard N.
- University Boulevard east of SR 33
- Firstpark Boulevard N. west of SR 33
- SR 33 between University Boulevard/Firstpark Boulevard N. and the eastbound I-4 ramps
- The eastbound I-4 on- and off-ramps
- SR 33 between the eastbound I-4 ramps and the westbound I-4 ramps
- The westbound I-4 on- and off-ramps
- SR 33 between the westbound I-4 ramps and Tomkow Road
- Tomkow Road north of SR 33
- SR 33 east of Tomkow Road

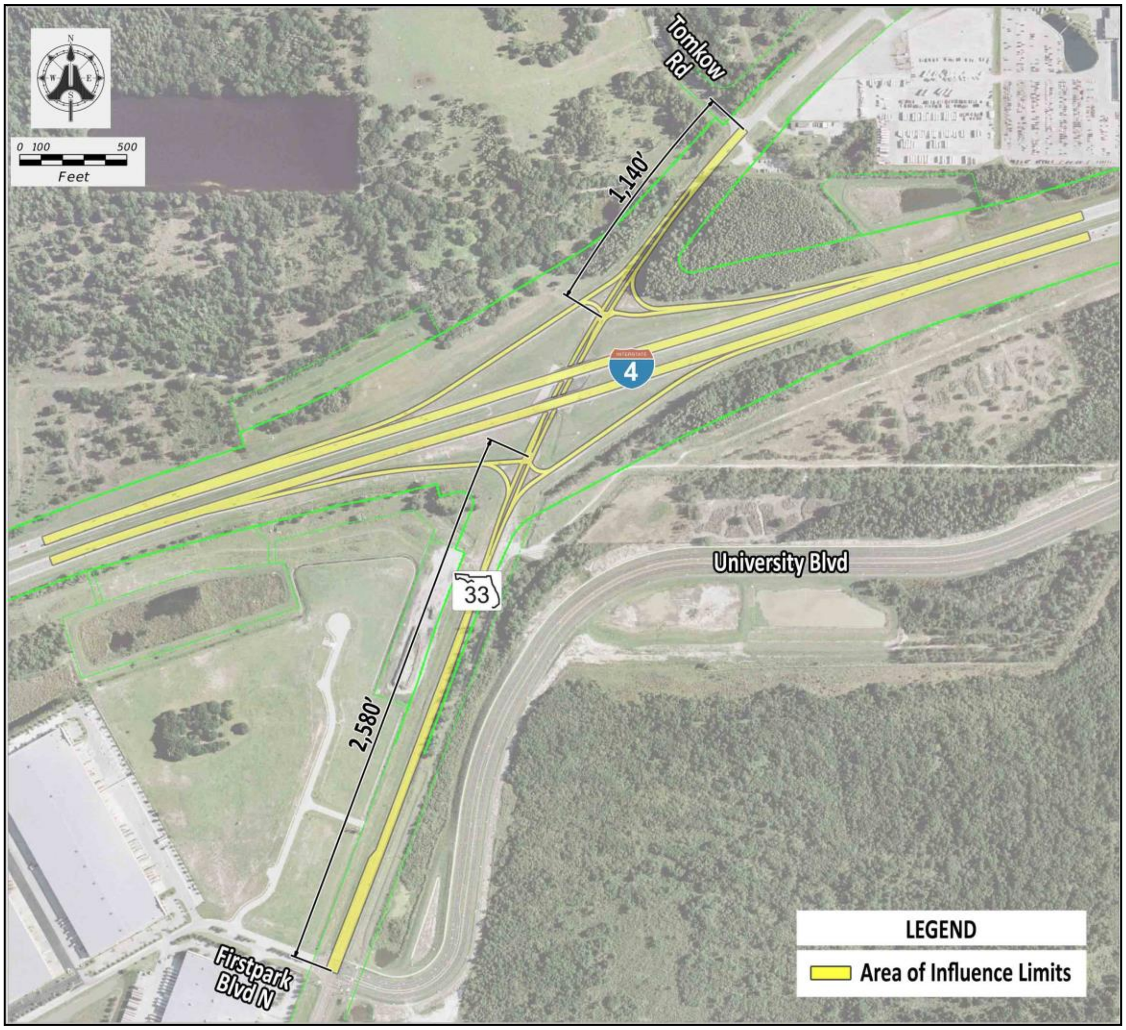


**FIGURE 3-1: AREA OF INFLUENCE**

A 72-hour vehicle classification count was also conducted on SR 33 north of the N. Combee Road (SR 659) intersection between September 4, 2012 and September 6, 2012. This location is approximately 0.67 miles to the south of University Boulevard/Firstpark Boulevard N. intersection. The bi-directional volume count data and vehicle classification count data is provided in **Appendix B**.

The 2012 Annual Average Daily Traffic (AADT) volumes were calculated by multiplying the 24-hour count data by seasonal and axle adjustment factors. According to the 2012 Peak Season Factor Category Report, the Polk County (Countywide) and I-4 weekly adjustment factors associated with the week of September 2nd through September 8th are equal to 1.07 and 1.09, respectively. These weekly adjustment factors are provided in **Appendix C**. The 2012 Weekly Axle Factor Category Report indicates that the axle adjustment factor for the portion of I-4 from US 98 to the Osceola County line (which includes the I-4/SR 33 interchange) is 0.90. The 72-hour bi-directional vehicle classification count on SR 33 was used to calculate an axle adjustment factor equal to 0.86 which is slightly higher than the 0.82 value contained in the FDOT database. Since the axle adjustment factor that was calculated using the September 6th vehicle classification count was extremely close to the three-day average axle adjustment factor and all of the 24-hour volume counts were conducted on September 6th; the use of the 0.86 axle adjustment factor was viewed as being more accurate. The axle adjustment factors are also provided in **Appendix C**.

**Table 3-1** summarizes the two-way 24-hour volumes obtained from the traffic counts, as well as the estimated 2012 AADT volumes. Since the only "existing" land use located along University Boulevard is the initial phase of the Florida Polytechnic University which is still under construction, a majority of the vehicles that were counted on University Boulevard east of SR 33 were associated with the ongoing construction. Consequently, the use of a weekly adjustment factor greater than 1.00 was not appropriate for this facility at this time.

There are two FDOT portable count stations located on I-4 in the vicinity of the I-4/SR 33 interchange. These count stations are as follows:

- Station No. 160114 (located to the west of the I-4/SR 33 interchange)
- Station No. 160113 (located to the east of the I-4/SR 33 interchange)

In addition, there is also one portable count station located on each of the four interchange ramps. These count stations are as follows:

- Station No. 16320090 (on the eastbound I-4 off-ramp)
- Station No. 16320091 (on the westbound I-4 on-ramp)
- Station No. 16320092 (on the eastbound I-4 on-ramp)
- Station No. 16320093 (on the westbound I-4 off-ramp)

**Table 3-2** provides a comparison of the 2012 AADT volumes obtained from the FDOT count stations and the 2012 AADT volumes estimated from the 24-hour traffic counts for the interchange ramps. This table also includes the 2012 I-4 mainline AADT volumes that are associated with the two mainline count stations. A review of **Table 3-2** indicates that the two AADT volumes associated with each ramp are exactly the same for three of the four locations. Although the 2012 westbound on-ramp AADT volume obtained from the FDOT count station is lower than the AADT volume estimated from the traffic count, it is much closer to the 2012 eastbound off-ramp AADT volume. Typically, the AADT volumes on reciprocal ramps at conventional diamond interchanges (i.e., where all movements are allowed) are approximately the same and the FDOT count station volumes at the I-4/SR 33 interchange ramps support this. Consequently, the eastbound off-ramp and westbound on-ramp AADT volumes estimated from the 24-hour traffic counts were averaged and this average value (i.e., 3,800 vpd) was used as the 2012 AADT volume for each of these two ramps.

## TABLE 3-1: EXISTING YEAR (2012) AADT VOLUMES

| Location | 24-Hour Volume | SF (1) | AF (2) | AADT Volume | AADT Volume (3) |
|---|---|---|---|---|---|
| SR 33 south of Firstpark Boulevard N./University Boulevard | 10,628 | 1.07 | 0.86 (4) | 9,780 | 9,800 |
| SR 33 between Firstpark Boulevard N./University Boulevard and Eastbound I-4 On-/Off-Ramps | 11,381 | 1.07 | 0.86 (4) | 10,473 | 10,500 |
| SR 33 between Eastbound I-4 On-/Off-Ramps and Westbound I-4 On-/Off-Ramps | 12,834 | 1.07 | 0.86 (4) | 11,810 | 11,800 |
| SR 33 between Westbound I-4 On-/Off-Ramps and Tomkow Road | 13,488 | 1.07 | 0.86 (4) | 12,412 | 12,400 |
| SR 33 east of Tomkow Road | 10,187 | 1.07 | 0.86 (4) | 9,374 | 9,400 |
| Firstpark Boulevard N. west of SR 33 | 1,970 | 1.07 | 0.86 (4) | 1,813 | 1,800 |
| University Boulevard east of SR 33 | 731 | N/A | 0.86 (4) | 629 | 630 |
| Eastbound I-4 Off-Ramp west of SR 33 | 3,432 | 1.09 | 0.90 | 3,367 | 3,400 |
| Eastbound I-4 On-Ramp east of SR 33 | 2,829 | 1.09 | 0.90 | 2,775 | 2,800 |
| Westbound I-4 Off-Ramp east of SR 33 | 2,819 | 1.09 | 0.90 | 2,765 | 2,800 |
| Westbound I-4 On-Ramp west of SR 33 | 4,332 | 1.09 | 0.90 | 4,250 | 4,250 |
| Tomkow Road north of SR 33 | 2,722 | 1.07 | 0.86 (4) | 2,505 | 2,500 |

(1) 2012 Weekly Seasonal Adjustment Factor obtained from the FDOT Database

(2) 2012 Weekly Axle Adjustment Factor obtained from the FDOT Database

(3) Rounded AADT Volume

(4) 2012 Axle Adjustment Factor calculated based on vehicle classification count data obtained between 9/4/2012 and 9/6/2012

The 2012 FDOT count station ramp volumes to and from the east (5,600 vpd) were subtracted from the 2012 FDOT count station volume on the I-4 mainline east of the SR 33 interchange (68,000 vpd) to obtain an estimate of the 2012 AADT volume on I-4 in the “middle’ of the interchange. The 2012 FDOT count station ramp volumes to and from the west (7,100 vpd) were subsequently added to this volume resulting in a 2012 AADT volume of 69,500 vpd for the I-4 mainline west of the SR 33 interchange. This volume was significantly lower than the 2012 AADT volume obtained from the FDOT count station on I-4 west of SR 33. A review of the Historical AADT Report for the I-4 mainline east of SR 33 indicated that the 2012 and 2011 AADT volumes were exactly the same. In contrast, a review of the Historical AADT Report for the I-4 mainline west of SR 33 indicated that the 2011 AADT volume was equal to 70,000 vpd. Since the 2011 AADT volume on I-4 to the west of SR 33 compared favorably to the 2012 AADT volume that was obtained by adding and subtracting the 2012 ramp volumes from the 2012 AADT volume on I-4 to the east of SR 33, the 2011 AADT volume was used as the 2012 AADT volume for this location. The 2012 AADT volumes for the study area are graphically illustrated in **Figure 3-2**.

**TABLE 3-2: EXISTING AADT VOLUME COMPARISON**

| Location | 2012 FDOT Count Station Volume | 2012 Adjusted 24-Hour Count Volume | 2012 Final Volume |
|---|---|---|---|
| I-4 West of SR 33 | 74,000 (1) | N/A | 70,000 (3) |
| I-4 East of SR 33 | 68,000 (2) | N/A | 68,000 (2) |
| I-4 Eastbound Off-Ramp | 3,400 | 3,400 | 3,800 (4) |
| I-4 Westbound On-Ramp | 3,700 | 4,250 | 3,800 (4) |
| I-4 Westbound Off-Ramp | 2,800 | 2,800 | 2,800 |
| I-4 Eastbound On-Ramp | 2,800 | 2,800 | 2,800 |

(1) 2012 AADT volume from FDOT Count Station No. 160114

(2) 2012 AADT volume from FDOT Count Station No. 160113

(3) 2011 AADT volume from FDOT Count Station No. 160114

(4) Calculated as [(3,400 + 4,250)/2]

Eight-hour manual turning movement counts were conducted at the study area intersections on October 18, 2012. The turning movement counts were conducted from 6:00 a.m. to 9:00 a.m. and from 1:00 p.m. to 6:00 p.m. The peak hour intersection turning movement count data is provided in **Appendix D**. The 2012 I-4 mainline hourly volumes obtained from the synopsis reports contained in the FDOT's Florida Traffic Online website were used in combination with the 2012 I-4 ramp volumes obtained from the peak hour turning movement counts conducted at the ramp terminal intersections to derive the 2012 a.m. and p.m. peak hour volumes for the I-4/SR 33 interchange. Adjustments were made to the a.m. and p.m. peak hour volumes for balancing purposes. **Figure 3-3** and **Figure 3-4** graphically illustrate the adjusted 2012 a.m. and p.m. peak hour volumes for the study area, respectively.

Eight-hour manual turning movement counts were also conducted at the Gourmet Foods International driveway and the two existing Manheim of Lakeland Auto Auction driveways. Both of these businesses are located to the east of Tomkow Road. Gourmet Foods International is located on the north side of SR 33 while the Auto Auction is located on the south side of SR 33. The Gourmet Foods International turning movement counts were conducted from 6:00 a.m. to 9:00 a.m. and from 1:00 p.m. to 6:00 pm. on October 25, 2012; while the Auto Auction turning movement counts were conducted on October 3, 2012 from 12:30 p.m. to 8:30 p.m. These counts were conducted to obtain information that could be used during the development of the preliminary SR 33 access management plan. Auctions are only conducted at this location on Wednesdays between the hours of 2:00 p.m. and 8:00 p.m. (although people start arriving on Wednesdays as early as 12:30 p.m.); therefore, the Auto Auction turning movement counts were conducted during the "peak hours" of this land use.

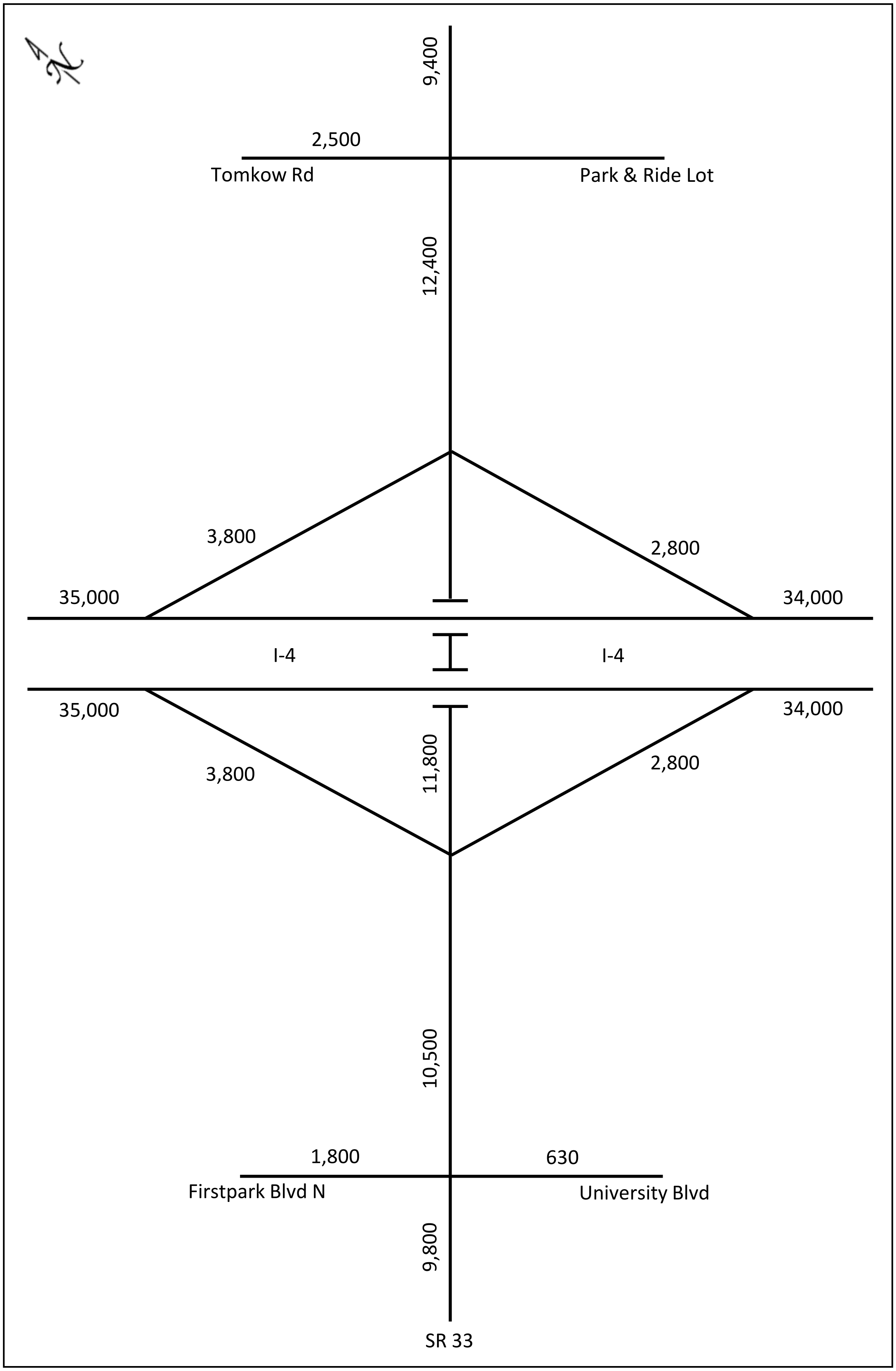


**FIGURE 3-2: EXISTING YEAR (2012) AADT VOLUMES**

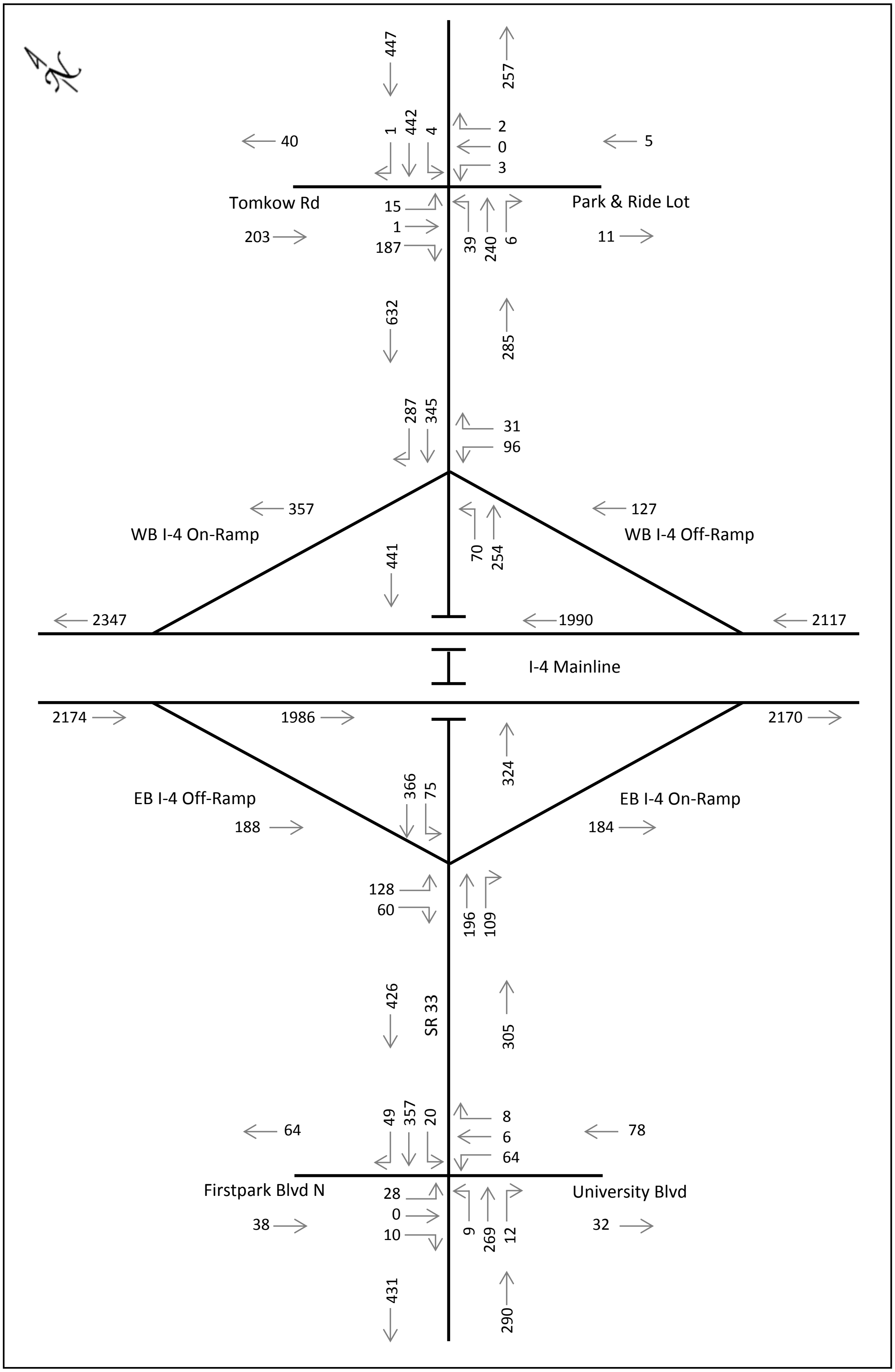


**FIGURE 3-3: EXISTING YEAR (2012) AM PEAK HOUR VOLUMES**

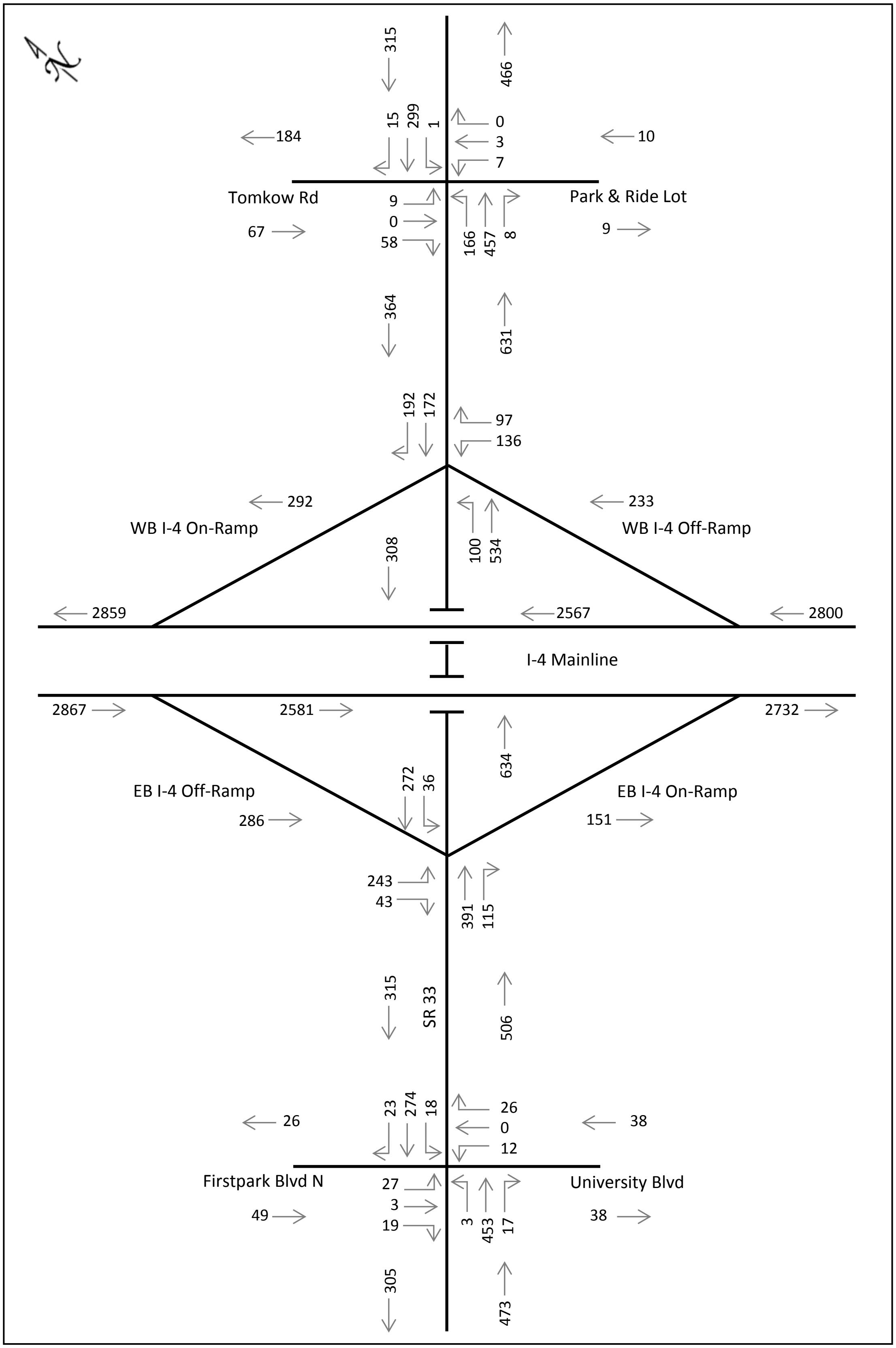


**FIGURE 3-4: EXISTING YEAR (2012) PM PEAK HOUR VOLUMES**

## 3.5 TRAFFIC FORECASTING/FUTURE YEAR TRAFFIC VOLUMES

The future year AADT volumes that are documented in this report were originally developed in support of the SR 33 PD&E study. A detailed discussion of the future year traffic forecasting methodology that was employed for the SR 33 PD&E study is contained in the Final SR 33 Travel Demand Forecasting Technical Memorandum (September 2013). A CD containing this technical memorandum is provided in **Appendix E**. This section of the report provides a summary of the overall process that was followed to derive the opening year and design year AADT volumes for the I-4/SR 33 interchange.

The first step in the future year traffic forecasting involved running the Polk County Transportation Planning Organization's (TPO's) 2007 Base Year travel demand model, as well as the TPO's 2035 travel demand model that represents their Cost Feasible Long Range Transportation Plan (which is commonly referred to as the 2035 Mobility Vision Plan). It should be noted that the widening (i.e., four-laning) of SR 33 from east of Old Combee Road/Deeson Pointe Boulevard to east of Tomkow Road is included in the TPO's 2035 Mobility Vision Plan as a cost-feasible transportation improvement. Consequently, the TPO's 2035 travel demand model includes a four-lane SR 33 roadway. The 2007 and 2035 Peak Season Weekday Average Daily Traffic (PSWADT) volumes obtained from these original models were converted to 2035 AADT volumes and reviewed for reasonableness. **Table 3-3** provides a comparison of the 2007 and 2035 model AADT volumes. As indicated in this table, minimal growth in daily traffic was projected to occur for the portion of SR 33 north of the interchange. In addition, the 2035 AADT volumes projected for Tomkow Road north of SR 33 and N. Combee Road south of SR 33 were slightly lower than the 2007 model AADT volumes.

**TABLE 3-3: ORIGINAL POLK TPO MODEL AADT VOLUME COMPARISON**

| Roadway | Location | 2007 Polk TPO Model | 2035 Polk TPO Model | Increase |
|---|---|---|---|---|
| SR 33 | North of N. Combee Rd. | 4,200 | 18,800 | 14,600 |
| | South of I-4 | 4,600 | 12,100 | 7,500 |
| | North of I-4 | 14,300 | 14,900 | 600 |
| | East of Tomkow Rd. | 9,900 | 11,100 | 1,200 |
| I-4* | West of CR 582/SR 33 | 69,800 | 98,400 | 28,600 |
| | West of SR 33 | 69,400 | 99,400 | 30,000 |
| | East of SR 33 | 64,200 | 99,200 | 35,000 |
| | East of Polk Parkway | 64,300 | 105,000 | 40,700 |
| Old Combee Rd. | South of SR 33 | 9,600 | 16,100 | 6,500 |
| N. Combee Rd. | South of SR 33 | 4,000 | 3,800 | -200 |
| Tomkow Rd. | North of SR 33 | 4,300 | 3,800 | -500 |

*The AADT volumes for I-4 were derived using a Model Output Conversion Factor (MOCF) equal to 0.94

The next step in the future year traffic forecasting involved a review of the validation accuracy associated with the Polk TPO's 2007 Base Year travel demand model. **Table 3-4** provides a comparison of the 2007 AADT volumes obtained from the base year model and the actual 2007 AADT volumes. The 2007 model AADT volumes on SR 33 south and north of the interchange were 4,600 vehicles per day (vpd) and 14,300 vpd, respectively. In contrast, the actual 2007 AADT volumes at these two locations were 9,300 vpd and 12,100 vpd. Consequently, the 2007 base year

**TABLE 3-4: 2007 AADT VOLUME COMPARISON – ORIGINAL TPO MODEL VS. REVISED TPO MODEL**

| Roadway | Location | 2007 Actual | 2007 Original TPO Model | | 2007 Revised TPO Model | |
|---|---|---|---|---|---|---|
| | | AADT Volume | AADT Volume | % Difference | AADT Volume | % Difference |
| SR 33 | South of I-4 | 9,300 | 4,600 | -50.5% | 6,800 | -26.9% |
| | North of I-4 | 12,100 | 14,300 | 18.2% | 15,100 | 24.8% |
| I-4* | West of SR 33 | 75,000 | 69,400 | -7.5% | 73,000 | -2.7% |
| | East of SR 33 | 68,500 | 64,200 | -6.3% | 68,500 | 0.0% |
| N. Combee Rd. | South of SR 33 | 6,900 | 4,000 | -42.0% | 5,700 | -17.4% |

* An I-4 specific MOCF of 0.94 was used to calculate these AADT volumes

model was underestimating the volume on SR 33 to the south of the interchange by 4,700 vpd (approximately 51%) and overestimating the volume on SR 33 to the north of the interchange by 2,200 vpd (approximately 18%). In addition, the 2007 model AADT volume on N. Combee Road to the southeast of SR 33 was 4,000 vpd, while the actual 2007 AADT volume was 6,900 vpd. The 2007 base year model was underestimating the volume on this roadway by 2,900 vpd (approximately 42%).

**Table 3-4** also illustrates that the original 2007 model AADT volumes on I-4 west and east of the interchange were both lower than the actual 2007 AADT volumes. The 2007 model AADT volume on I-4 west of the interchange was 5,600 vpd (approximately 8%) lower than the actual AADT volume, while the 2007 model AADT volume on I-4 east of the interchange was 4,300 vpd (approximately 6%) lower than the actual AADT volume.

Modifications were made to the original Polk TPO base year model to improve the validation accuracy of the model within the interchange area. These included modifications to the travel demand model roadway network characteristics (i.e., facility types, speeds and capacities), Traffic Analysis Zone (TAZ) structure and centroid connectors, and land use data. The revised 2007 model AADT volumes are also provided in **Table 3-4**. A review of this table indicates that the revised 2007 model AADT volumes are closer to the actual 2007 AADT volumes for four of the five locations.

Modifications were also made to the original Polk TPO 2035 Mobility Vision Plan travel demand model. Some of these modifications were necessary to ensure consistency with the revised base year model while others were necessary to correct roadway network coding errors in the original

Polk TPO 2035 travel demand model. As stated earlier in **Section 1.2** of this report, there are also three Developments of Regional Impact (DRIs) located in close proximity to the I-4/SR 33 interchange. These are the Williams DRI, Bridgewater DRI, and Polk Commerce Center DRI. The Williams DRI is located immediately south of I-4 between SR 33 and the Polk Parkway while the Polk Commerce Center DRI is located immediately east of the Polk Parkway between I-4 and Saddle Creek Road. A majority of the Bridgewater DRI is located on the west side of SR 33 (between SR 33 and I-4); however, there is also a portion located on the east side of SR 33. In addition, the proposed Rockefeller Group Park of Commerce development is located to the north of SR 33 and to the east of Tomkow Road on the site of the former USA International Speedway.

A comparison of the land use data that is included in the DRIs and the Rockefeller Group Park of Commerce and the land use data that was included in the Polk TPO's 2035 model for those TAZs that comprise these planned developments was also conducted. The results of this comparison indicated that the original 2035 model contained significantly lower amounts of land use than the development levels that were contained in the DRI documents. Consequently, some of the land use data contained in the original Polk TPO 2035 model was modified to more accurately reflect the amount of future land use that is anticipated to occur as a result of these large developments.

The revised Polk TPO 2035 travel demand model was run and the 2035 PSWADT volumes were converted to AADT volumes. **Table 3-5** provides a comparison of the revised 2035 model AADT volumes and the original 2035 model AADT volumes. Roadway network plots of the revised 2035 AADT volumes are provided in **Appendix F**. The 2012 AADT volumes that were derived from the PD&E study traffic counts are also included in **Table 3-5**. A review of this table indicates that significantly higher AADT volumes are projected for the I-4 mainline, SR 33, and the I-4/SR 33 interchange ramps with the revised 2035 travel demand model. The 2035 AADT volumes on the I-4 mainline west and east of SR 33 are projected to be 13,600 vpd and 10,000 vpd higher, respectively with the revised 2035 model. Compared to the 2012 AADT volumes for these two locations, the revised 2035 model AADT volumes represent increases of approximately 61% (or 2.6%/year). The revised 2035 model AADT volumes on SR 33 between University Boulevard/Firstpark Boulevard N. and Tomkow Road are between 7,300 vpd and 16,600 vpd higher than the original 2035 model AADT volumes.

Growth trend analyses were conducted for SR 33 using historic AADT volumes obtained from FDOT Count Station Nos. 160118 and 160027. These count stations are located on SR 33 just south and just north of the I-4/SR 33 interchange. These growth trend analyses were conducted based on the AADT volumes recorded for the years 1997 through 2011 as well as the 2012 AADT volumes estimated from the PD&E study traffic counts. The growth trend analyses yielded 2035 AADT volumes equal to 13,000 vpd (south of I-4) and 16,600 vpd (north of I-4). The 2035 volume for SR 33 south of I-4 represents a 24.0% increase over the existing (2012) volume while the 2035 volume for SR 33 north of I-4 represents a 34.0% increase over this same 23-year time period.

**TABLE 3-5: 2035 AADT VOLUME COMPARISON**

| Roadway | Segment | | AADT Volume | | |
|---|---|---|---|---|---|
| | From | To | Existing (2012) | Original 2035 TPO Model | Revised 2035 TPO Model |
| SR 33 | N. Combee Rd. | University Blvd. | 9,950 | 20,500 | 32,900 |
| | University Blvd. | I-4 EB Ramps | 10,500 | 12,100 | 28,700 |
| | I-4 EB Ramps | I-4 WB Ramps | 11,800 | 13,600 | 26,700 |
| | I-4 WB Ramps | Tomkow Rd. | 12,400 | 14,900 | 22,200 |
| | Tomkow Rd. | E. of Tomkow Rd. | 9,400 | 11,100 | 14,500 |
| University Blvd. | SR 33 | Reasearch Way W. | 630 | 12,900 | 38,600 |
| Tomkow Rd. | SR 33 | Old Polk City Rd. | 2,500 | 3,800 | 7,700 |
| Old Combee Rd. | Lake Parker Dr. | SR 33 | 10,500 | 16,100 | 15,400 |
| N. Combee Rd. | Old Combee Rd. | SR 33 | 8,200 | 3,800 | 12,550 |
| I-4* | CR 582/SR33 | SR 33 | 70,000 | 99,400 | 113,000 |
| | SR 33 | Polk Parkway East | 68,000 | 99,200 | 109,200 |
| I-4 Ramps* | EB Off-Ramp | | 3,400 | 4,000 | 9,100 |
| | EB On-Ramp | | 2,800 | 3,900 | 7,100 |
| | WB Off-Ramp | | 2,800 | 3,900 | 7,300 |
| | WB On-Ramp | | 4,250 | 4,000 | 9,100 |

* The 2035 AADT volumes for these locations were derived using an MOCF equal to 0.94

Copies of these two growth trend analyses are also provided in **Appendix F**. It should be noted that the $R^2$ values associated with these growth trend analyses are extremely low (i.e., 25.7% and 33.8%, respectively). This statistic measures how well the linear growth trend equation (i.e., the straight line) “fits” the data points. A review of the graphs of the growth trend analyses indicates that many of the data points (i.e., the historic volumes) are either higher or lower than the volumes that were estimated from the growth trend equation.

Significant increases in both population and employment are projected to occur between 2012 and 2035 for several of the TAZs in the vicinity of the I-4/SR 33 interchange. Given the magnitude of the projected growth in study area population and employment, the 2035 AADT volumes projected for the SR 33 study corridor using the revised 2035 Polk TPO model were viewed as being reasonable.

Growth trend analyses were also conducted for I-4 using historic AADT volumes obtained from FDOT Count Station Nos. 160114 and 160113. As stated earlier in **Section 3.4** of this report, these two count stations are located on I-4 to the west and east of the I-4/SR 33 interchange. The growth trend analyses were conducted based on the AADT volumes recorded for the years 1997 through 2012. The growth trend analyses yielded 2035 AADT volumes equal to 94,700 vpd (west of SR 33) and 97,100 vpd (east of SR 33). The 2035 volume for I-4 west of SR 33 represents a 28% increase over the 23-year period while the 2035 volume for I-4 east of SR 33 represents a 43% increase over this same time period. Copies of these two growth trend analyses are also contained in **Appendix F.**

Although the 2035 I-4 mainline volumes estimated from the growth trend analyses compare favorably to the 2035 volumes estimated from the original 2035 Polk TPO model, the 2035 I-4 ramp volumes estimated from the original Polk TPO model are not significantly higher than the 2012 ramp volumes. Since the historic growth trend analysis methodology is unable to take into account the impact of future land use growth on future travel demand, and significant increases in future year population and employment are projected to occur for several TAZs in the study area; the 2035 AADT volumes projected for the I-4 mainline and the I-4/SR 33 interchange ramps using the revised 2035 Polk TPO model were once again viewed as being the most reasonable future year projections.

Since the design year established for this project is 2036, the design year AADT volumes were derived by extrapolation using the existing (2012) and revised 2035 model AADT volumes. An opening year of 2016 was also established for this project and the opening year AADT volumes were derived through interpolation using the existing (2012) and revised 2035 model AADT volumes. The 2016 and 2036 AADT volumes are graphically illustrated in **Figure 3-5**.

The design year a.m. and p.m. peak hour volumes were derived with the use of the FDOT's TURNS5 software. The 2012 and 2035 AADT volumes were used as input along with a K-factor of 9.0%, D-factors of 53.0% (for I-4) and 55.4% (for SR 33), and the existing peak hour turning movement percentages. The 2036 peak hour volumes obtained from the TURNS5 software were subsequently reviewed for reasonableness. Based on this review it was determined that manual adjustments to the output were appropriate for one or more of the following reasons:

- To increase individual movement volumes that were estimated to be less than the 2012 volumes

- To reduce individual movement volumes that were estimated to be significantly higher than the 2012 volumes (if this significant increase was not viewed as being reasonable)

- To eliminate any differences between departure volumes and approach volumes at adjacent intersections

- To better reflect the design year peak hour K- and D-factors on the I-4 mainline and the interchange on- and off-ramps

The TURNS5 output is provided in **Appendix G**.

The opening year peak hour volumes were subsequently derived by interpolating between the 2012 peak hour volumes and the 2036 peak hour volumes. The opening year peak hour volumes are provided in **Figure 3-6** and **Figure 3-7**, while the design year peak hour volumes are provided in **Figure 3-8** and **Figure 3-9**.

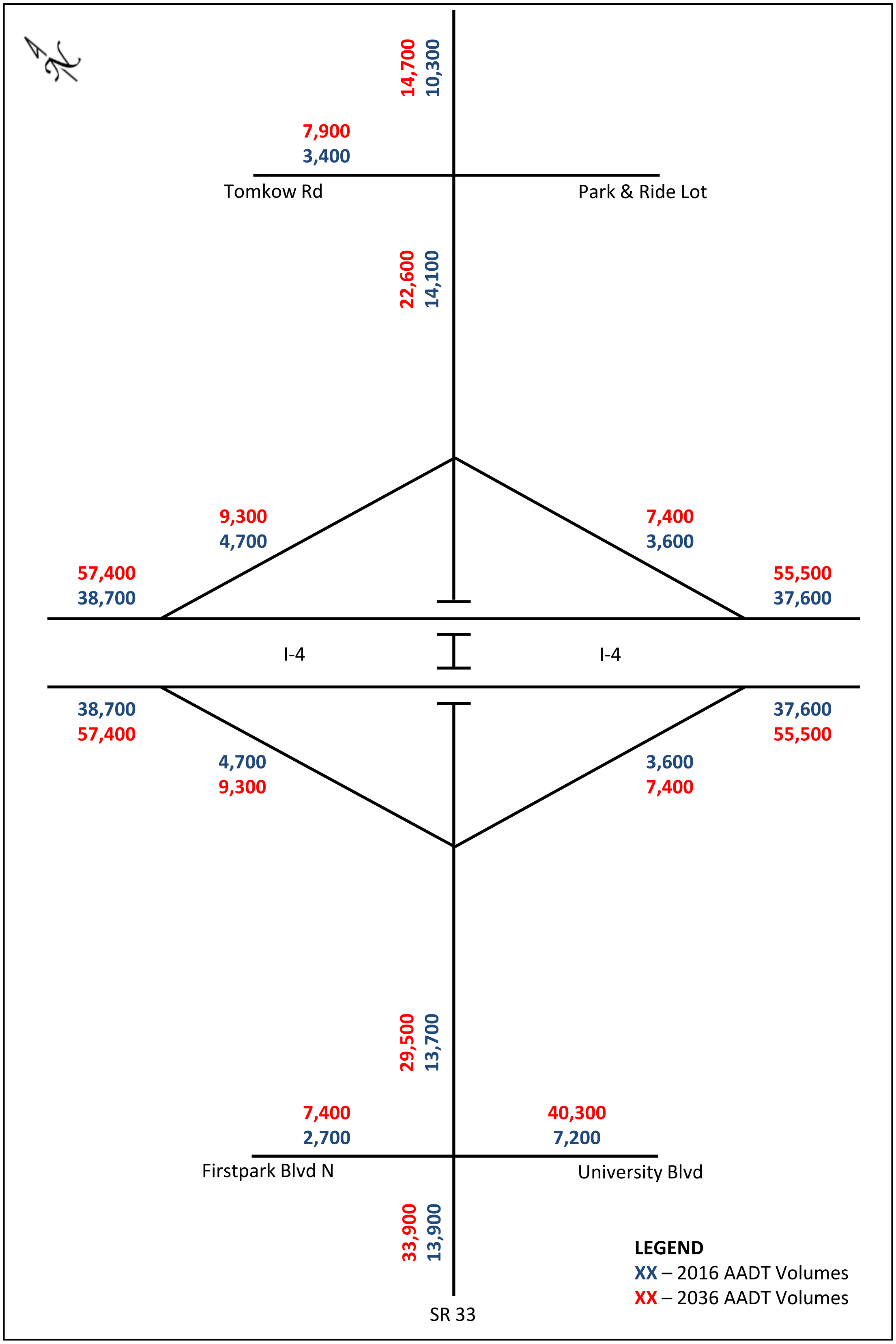


**FIGURE 3-5: OPENING YEAR (2016) AND DESIGN YEAR (2036) AADT VOLUMES**

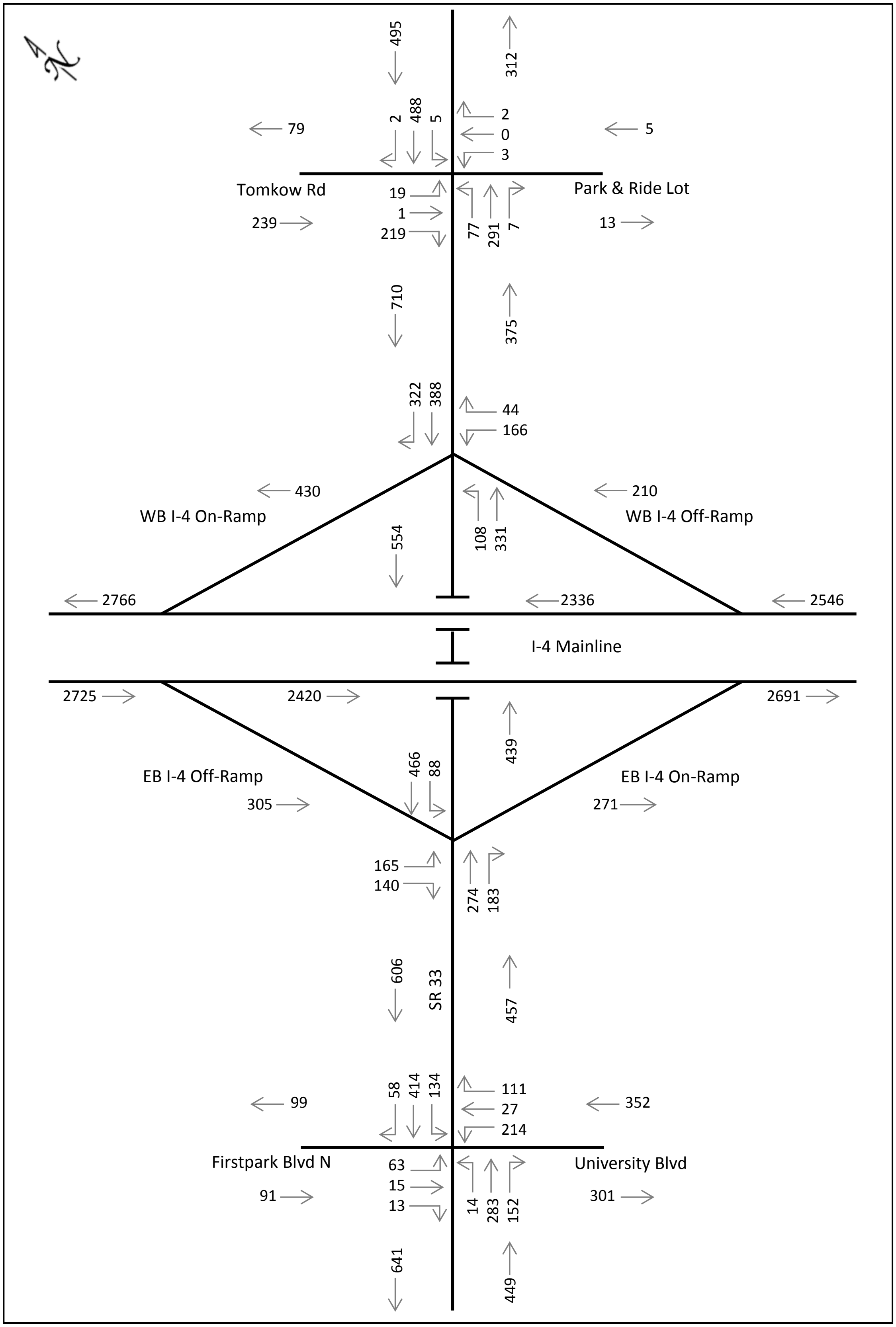


**FIGURE 3-6: OPENING YEAR (2016) AM PEAK HOUR VOLUMES**

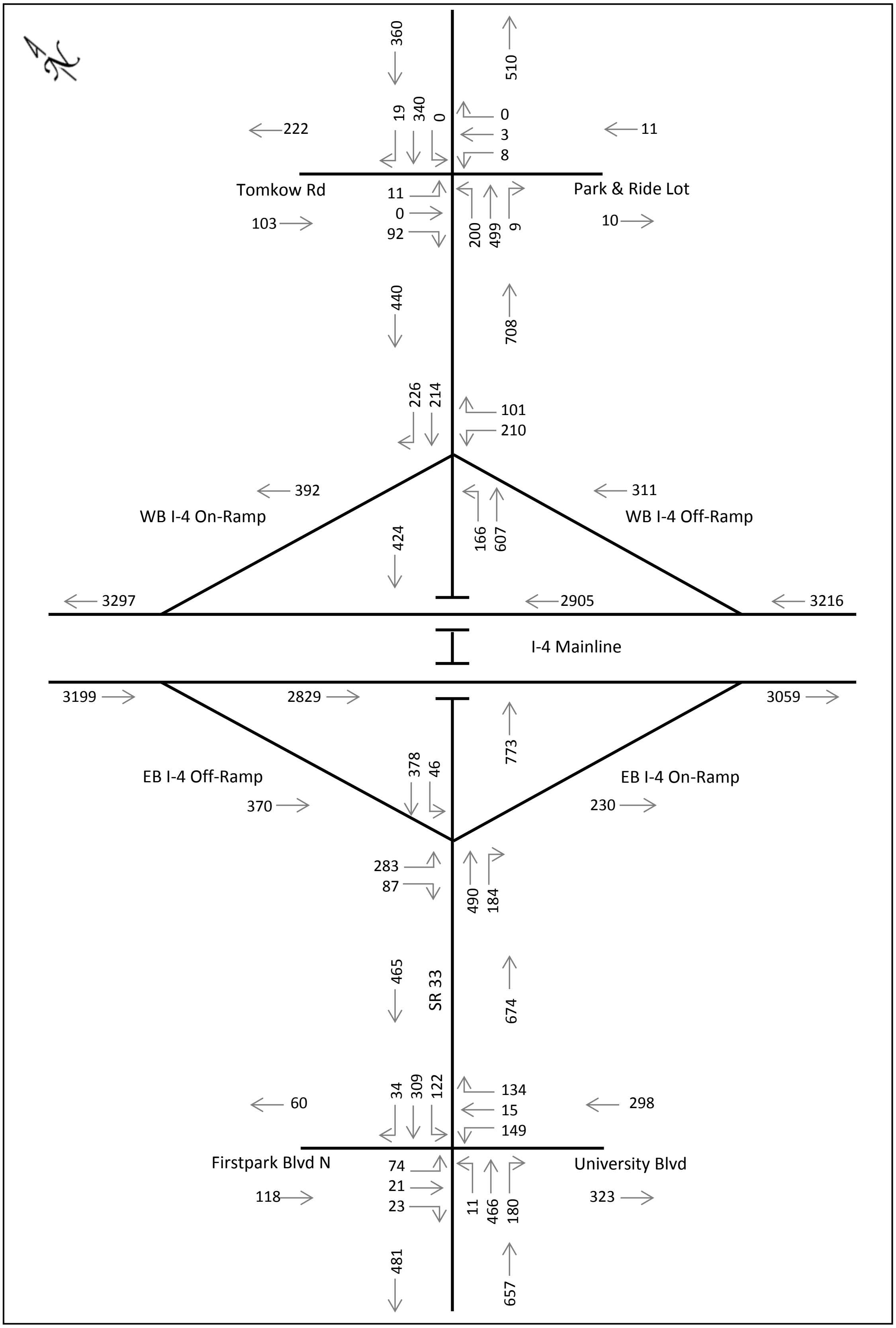


**FIGURE 3-7: OPENING YEAR (2016) PM PEAK HOUR VOLUMES**

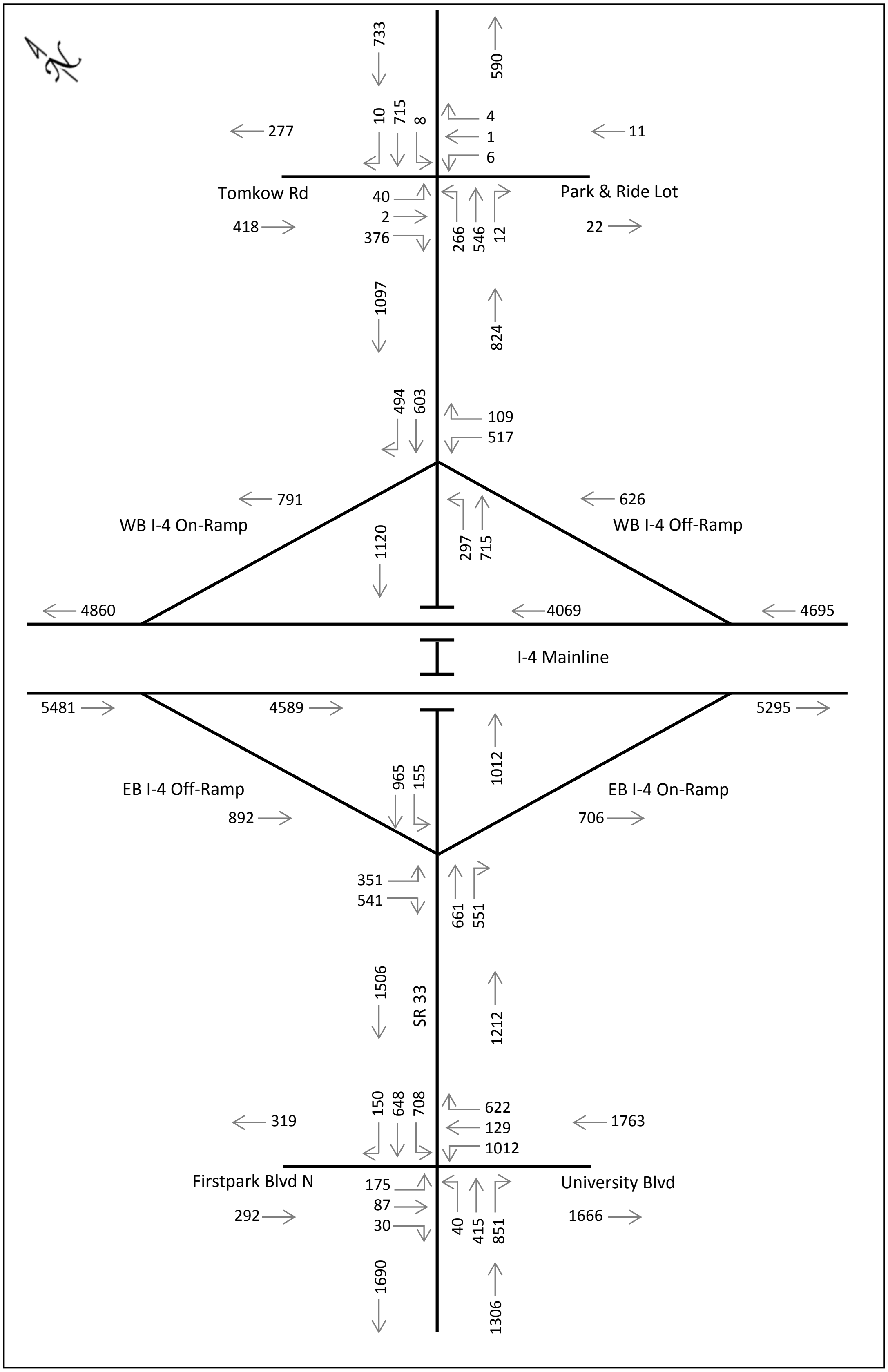


**FIGURE 3-8: DESIGN YEAR (2036) AM PEAK HOUR VOLUMES**

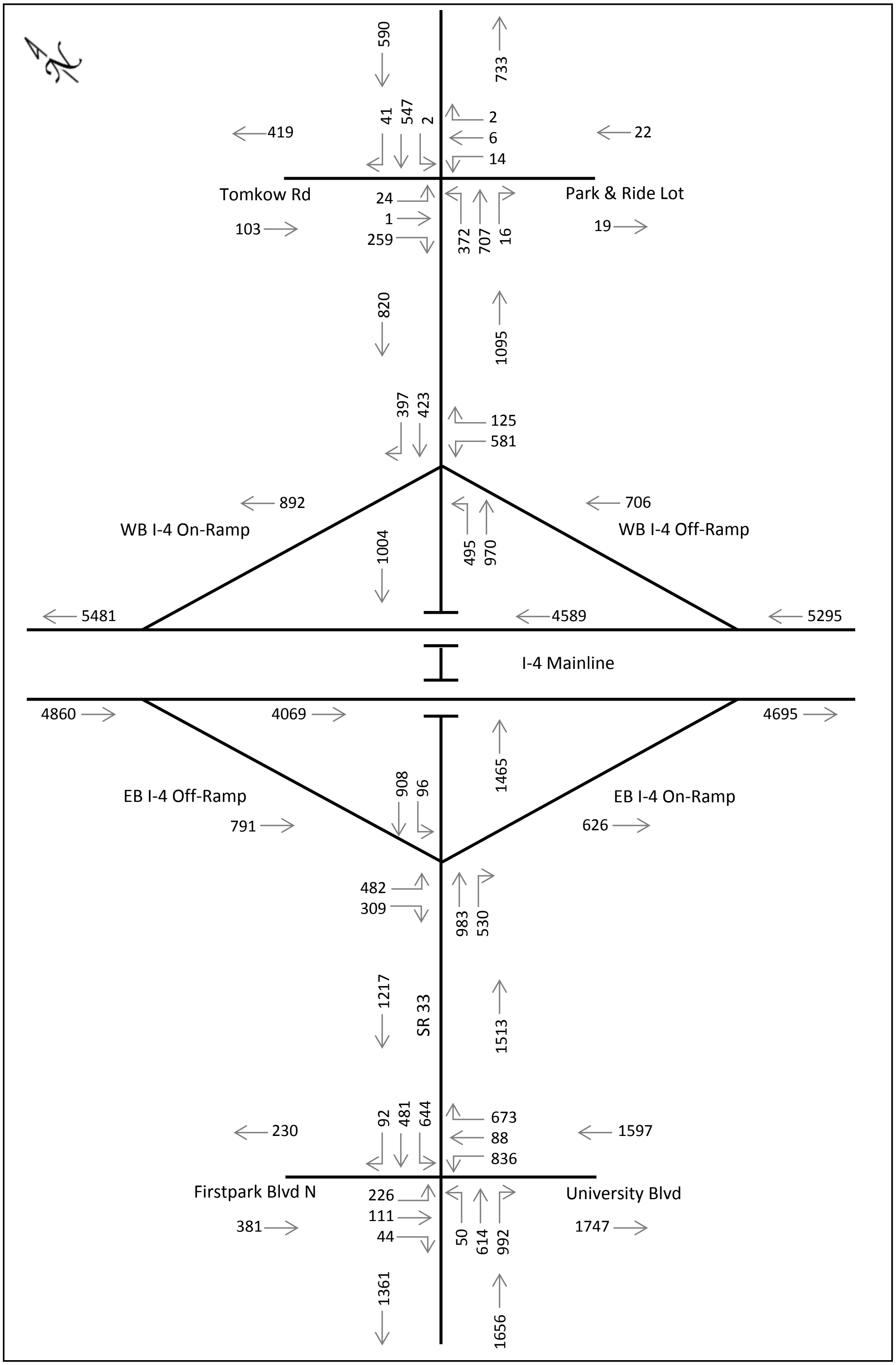


**FIGURE 3-9: DESIGN YEAR (2036) PM PEAK HOUR VOLUMES**

# 4.0 EXISTING CONDITIONS TRAFFIC OPERATIONS ANALYSIS

The existing conditions peak hour traffic operations analysis included an analysis of the four interchange ramp merge/diverge areas. The 2010 Highway Capacity Software (HCS) was used to conduct these analyses. The I-4 ramp merge/diverge area analyses were conducted using the following factors:

- Heavy Vehicle Percentage = 7.0%
- Driver Population Factor ($f_p$) = 1.00
- Peak Hour Factor (PHF) = 0.92
- Mainline Free Flow Speed (FFS) = 70 mph
- Ramp Free Flow Speed ($S_{FR}$) = 45 mph

The results of the existing conditions I-4 ramp merge/diverge area analyses are summarized in **Table 4-1**. In the a.m. peak hour, all four merge/diverge areas are operating at Level of Service B. In the p.m. peak hour, the two merge areas are operating at Level of Service B while the two diverge areas are operating at Level of Service C. The 2012 ramp merge/diverge area analysis summary sheets are provided in **Appendix H**.

**TABLE 4-1: EXISTING YEAR (2012) MERGE/DIVERGE AREA LEVELS OF SERVICE**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 2,174 | 188 | 17.4 | B | 2,867 | 286 | 21.7 | C |
| EB I-4 Merge Area | 1,986 | 184 | 12.4 | B | 2,581 | 151 | 15.3 | B |
| WB I-4 Diverge Area | 2,117 | 127 | 16.9 | B | 2,800 | 233 | 21.2 | C |
| WB I-4 Merge Area | 1,990 | 357 | 13.9 | B | 2,567 | 292 | 16.4 | B |

The existing conditions peak hour traffic operations analysis also included an analysis of the four intersections located within the area of influence. Since three of the four existing intersections are currently unsignalized and the fourth (i.e., the University Boulevard/Firstpark Boulevard N. intersection) is currently operating under two-way stop control; the 2010 HCS was also used to conduct these analyses. The average existing a.m. and p.m. PHF's that were calculated and used to conduct the roadway segment analyses for the SR 33 PD&E study, were also used for the SR 33 movements in the unsignalized intersection analyses. The specific PHF's calculated from the 2012 turning movement counts were used in the unsignalized intersection analyses for the cross street approaches because many of the cross street approaches are experiencing significant fluctuations in traffic flow (i.e., peaking characteristics) within the peak hour. Similarly, the average a.m. and p.m. peak hour truck percentages that were calculated and used to conduct the SR 33 PD&E study roadway segment analyses were also used for the SR 33 through movements in the unsignalized intersection analyses. The specific peak hour truck percentages that were calculated from the 2012 turning movement counts were used in the unsignalized intersection analyses for all of the other intersection movements.

The results of the existing conditions intersection analyses are summarized in **Table 4-2**. All of the individual movements at the eastbound and westbound I-4 ramp terminal intersections are projected to operate at Level of Service D or better during the a.m. and p.m. peak hours. The two movements that are operating at Level of Service D during the p.m. peak hour are the eastbound and westbound off-ramp left-turn movements.

**TABLE 4-2: EXISTING YEAR (2012) PEAK HOUR INTERSECTION OPERATIONS**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (unsignalized) | Northbound | LT | 0.01 | 8.2 | A | 0.00 | 8.3 | A |
| | Southbound | LT | 0.02 | 8.0 | A | 0.02 | 8.4 | A |
| | Eastbound | LT | 0.19 | 24.0 | C | 0.24 | 25.2 | D |
| | Eastbound | TH/RT | 0.02 | 10.5 | B | 0.07 | 11.2 | B |
| | Westbound | LT | 0.26 | 21.2 | C | 0.09 | 20.3 | C |
| | Westbound | TH | 0.02 | 16.7 | C | 0.00 | 0.0 | N/A |
| | Westbound | RT | 0.01 | 9.8 | A | 0.09 | 11.7 | B |
| I-4 Eastbound Ramps (unsignalized) | Eastbound | LT | 0.41 | 20.0 | C | 0.63 | 27.2 | D |
| | Eastbound | RT | 0.13 | 11.9 | B | 0.07 | 24.8 | C |
| | Southbound | LT | 0.06 | 7.9 | A | 0.03 | 8.4 | A |
| I-4 Westbound Ramps (unsignalized) | Westbound | LT | 0.30 | 19.6 | C | 0.50 | 27.1 | D |
| | Westbound | RT | 0.04 | 10.1 | B | 0.23 | 14.2 | B |
| | Northbound | LT | 0.08 | 8.9 | A | 0.08 | 8.0 | A |
| Tomkow Road (unsignalized) | Eastbound | LT | 0.04 | 8.4 | A | 0.14 | 8.4 | A |
| | Westbound | LT | 0.00 | 7.8 | A | 0.00 | 8.3 | A |
| | Northbound | LT/TH/RT | 0.05 | 23.0 | C | 0.17 | 36.0 | E |
| | Southbound | LT/TH/RT | 0.43 | 16.6 | C | 0.18 | 14.5 | B |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

Queue length observations were also conducted for the eastbound and westbound I-4 off-ramps on October 18, 2012. These observations were conducted on the same day that the eight-hour turning movement counts were conducted at the ramp terminal intersections. The maximum number of queued vehicles that were observed during each 15-minute interval was recorded separately for both the left-turn and right-turn lanes on the off-ramps. These observations are provided in **Appendix D**. The queue length data indicated the following:

- With one exception, the maximum left-turn queues were always greater than or equal to the maximum right-turn queues at both off-ramps.
- The longest left-turn queues recorded during the morning hours at both ramps occurred during the 60-minute period from 7:15 a.m. to 8:15 a.m. The maximum left-turn queues were 10 vehicles on the westbound off-ramp and 5 vehicles on the eastbound off-ramp.
- The longest left-turn queues recorded during the afternoon hours at both ramps occurred during the 60-minute period from 4:15 p.m. to 5:15 p.m. The maximum left-turn queues were 9 vehicles on the westbound off-ramp and 11 vehicles on the eastbound off-ramp.

In addition, the left-turn vehicle queues on the westbound I-4 off-ramp did not prohibit the right-turning vehicles from accessing the right-turn lane at any time during the eight-hour period, In contrast, there were multiple occasions where the left-turn vehicle queues on the eastbound I-4 off-ramp did not allow access to the right-turn lane. All of these occurred during the afternoon hours, with maximum left-turn vehicle queues in the range of 10 to 11 vehicles occurring between 4:15 p.m. and 5:15 p.m. These observations suggested that the p.m. peak hour average vehicle delay for the eastbound right-turn movement that was obtained from the HCS analysis (i.e., 10.7 seconds/vehicle) may be lower than the actual delay.

The existing conditions HCS analyses that were conducted for the I-4 ramp terminal intersections included separate left-turn and right-turn lanes for the eastbound and westbound I-4 off-ramp approaches. As stated earlier in **Section 2.1**, the I-4 off-ramps are single lane ramps that provide channelized right-turn lanes in the vicinity of SR 33. As long as the left-turn vehicle queues do not extend back and block the access to the channelized right-turn lanes, the right-turn vehicle delays are independent of the left-turn vehicle delays. However, once the left-turn vehicle queues block the access to the channelized right-turn lanes, the right-turn vehicle delays become affected by the left-turn vehicle delays and their delays start to approximate the delay associated with a single shared left-turn/right-turn lane. Since the eastbound left-turn queues did extend back and block the access to the eastbound right-turn lane for at least a portion of each 15-minute interval between 4:15 p.m. and 6:00 p.m., the weighted average approach delay value of 24.8 seconds/vehicle was used as the estimate of the average vehicle delay for the eastbound right-turn movement and is included in **Table 4-2**.

With one exception, all of the movements at the University Boulevard/Firstpark Boulevard N. and Tomkow Road intersections are also operating at Level of Service D or better during both peak hours. In the p.m. peak hour, the northbound left-turn, through, and right-turn movements exiting the park-and-ride lot across from Tomkow Road are operating at Level of Service E. The 2012 intersection analysis summary sheets are also provided in **Appendix H**.

# 5.0 EXISTING CONDITIONS CRASH DATA ANALYSIS

Crash data for the five-year period from January 1, 2007 through December 31, 2011 was obtained from the FDOT's State Safety Office. The SR 33 crash data covered the approximately 0.40-mile portion of the roadway from Milepost 8.237 (approximately 500 feet south of the eastbound I-4 and northbound SR 33 right-turn roadways) to Milepost 8.638 (approximately 500 feet northeast of the westbound I-4 and southbound SR 33 right-turn roadways).The I-4 crash data also covered the 0.95-mile portion of the I-4 mainline from Milepost 11.882 (approximately 0.25 miles to the west of the eastbound I-4 off-ramp) to Milepost 12.832 (approximately 0.25 miles to the east of the westbound I-4 off-ramp), as well as the four interchange ramps.

**Table 5-1** summarizes the number of crashes, fatalities, and injuries that occurred during each of the five years between 2007 and 2011. A total of 163 crashes involving 257 vehicles occurred during this five-year period and these crashes resulted in one fatality and 98 injuries. A majority of the crashes (114) occurred on the I-4 mainline; however 28 crashes occurred on SR 33 within the interchange area and another 21 crashes occurred on the I-4 on- and off-ramps.

**TABLE 5-1: CRASH HISTORY (2007 – 2011)**

| Roadway | Year | No. of Crashes | No. of Vehicles | No. of Fatalities | No. of Injuries |
|---|---|---|---|---|---|
| SR 33 (1) | 2007 | 8 | 14 | 0 | 4 |
| | 2008 | 7 | 12 | 0 | 9 |
| | 2009 | 7 | 12 | 1 | 4 |
| | 2010 | 4 | 6 | 0 | 3 |
| | 2011 | 2 | 3 | 0 | 3 |
| | **Subtotal** | **28** | **47** | **1** | **23** |
| I-4 On-/Off-Ramps | 2007 | 1 | 1 | 0 | 1 |
| | 2008 | 5 | 10 | 0 | 3 |
| | 2009 | 10 | 19 | 0 | 13 |
| | 2010 | 2 | 3 | 0 | 0 |
| | 2011 | 3 | 6 | 0 | 3 |
| | **Subtotal** | **21** | **39** | **0** | **20** |
| I-4 Mainline (2) | 2007 | 20 | 32 | 0 | 15 |
| | 2008 | 22 | 30 | 0 | 11 |
| | 2009 | 28 | 42 | 0 | 9 |
| | 2010 | 25 | 39 | 0 | 7 |
| | 2011 | 19 | 28 | 0 | 13 |
| | **Subtotal** | **114** | **171** | **0** | **55** |
| **Total** | | **163** | **257** | **1** | **98** |

(1) From Milepost 8.237 to Milepost 8.638

(2) From Milepost 11.882 to Milepost 12.832

**Table 5-2** summarizes the lighting, weather, and roadway surface conditions that were present at the time of the crashes. A review of this table indicates that the majority of the crashes occurred during daylight hours (approximately 72.4%), non-rainy weather (approximately 77.3%) and on dry pavement conditions (approximately 68.7%). Therefore, a majority of the crashes were not influenced by poor visibility and/or slippery roadway surface conditions.

**TABLE 5-2: CRASH CONDITIONS (2007 – 2011)**

| Lighting | | |
|---|---|---|
| **Condition** | **No. of Occurrences** | **% of Occurrences** |
| Daylight | 118 | 72.39% |
| Dark (Street Light) | 28 | 17.18% |
| Dark (No Street Light) | 8 | 4.91% |
| Dusk | 6 | 3.68% |
| Dawn | 3 | 1.84% |
| **Total** | **163** | **100.00%** |
| **Weather** | | |
| **Condition** | **No. of Occurrences** | **% of Occurrences** |
| Clear | 80 | 49.08% |
| Cloudy | 46 | 28.22% |
| Rain | 37 | 22.70% |
| Fog | 0 | 0.00% |
| **Total** | **163** | **100.00%** |
| **Road Surface** | | |
| **Condition** | **No. of Occurrences** | **% of Occurrences** |
| Dry | 112 | 68.71% |
| Wet | 51 | 31.29% |
| Slippery | 0 | 0.00% |
| **Total** | **163** | **100.00%** |

**Table 5-3** summarizes the primary locations of the crashes that occurred within the I-4/SR 33 interchange area. The three highest frequency locations on SR 33 are at the eastbound I-4 right-turn lane onto southbound SR 33, the westbound I-4 right-turn lane onto northbound SR 33, and underneath the two I-4 bridges. These three locations accounted for 50.0% of the total crashes on SR 33 during the five-year period. The one fatality that was recorded during this five-year period occurred at the westbound I-4 right-turn lane onto northbound SR 33. It should be noted that alcohol was involved with this crash. A majority of the interchange ramp crashes occurred on either the eastbound I-4 off-ramp (approximately 61.9%) or the westbound I-4 off-ramp (approximately 23.8%). Approximately 49.0% of the crashes that occurred on the I-4 mainline occurred at one of the four ramp merge/diverge areas, with the eastbound I-4 on-ramp merge area having the highest number of crashes of the four merge/diverge areas (22).

**TABLE 5-3: CRASH LOCATIONS (2007 – 2001)**

| SR 33 | | |
|---|---|---|
| **Crash Location** | **No. of Occurrences** | **% of Occurrences** |
| EB I-4 Right-Turn Lane (onto SB SR 33) | 6 | 21.43% |
| WB I-4 Right-Turn Lane (onto NB SR 33) | 5 | 17.86% |
| Under the I-4 Bridges | 3 | 10.71% |
| NB SR 33 Right-Turn Lane (onto EB I-4) | 2 | 7.14% |
| EB I-4 Unsignalized Intersection | 2 | 7.14% |
| SB SR 33 Right-Turn Lane (onto WB I-4) | 1 | 3.57% |
| WB I-4 Unsignalized Intersection | 0 | 0.00% |
| Other Locations | 9 | 32.14% |
| **Total** | **28** | **100.00%** |
| **I-4 On-/Off-Ramps** | | |
| **Crash Location** | **No. of Occurrences** | **% of Occurrences** |
| EB I-4 Off-Ramp | 13 | 61.90% |
| WB I-4 Off-Ramp | 5 | 23.81% |
| EB I-4 On-Ramp | 3 | 14.29% |
| WB I-4 On-Ramp | 0 | 0.00% |
| **Total** | **21** | **100.00%** |
| **I-4 Mainline** | | |
| **Crash Location** | **No. of Occurrences** | **% of Occurrences** |
| EB I-4 On-Ramp Merge Area (1) | 22 | 19.30% |
| WB I-4 On-Ramp Merge Area (1) | 15 | 13.16% |
| EB I-4 Off-Ramp Diverge Area (1) | 14 | 12.28% |
| On the SR 33 Bridges | 8 | 7.02% |
| WB I-4 Off-Ramp Diverge Area (1) | 5 | 4.39% |
| Other Locations | 50 | 43.86% |
| **Total** | **114** | **100.00%** |

(1) ≤ 500 feet (approximately 0.095 miles) upstream or downstream of the ramp milepost number

**Table 5-4** summarizes the types of crashes that occurred within the interchange area. Sixteen of the 28 crashes that occurred on SR 33 (approximately 57.1%) were either angle, left-turn or sideswipe crashes. It is very likely that the existing geometric conditions on SR 33 had an influence on these types of crashes. As stated earlier in **Section 2.0** of this report, both of the ramp terminal intersections are currently unsignalized and there are no acceleration/deceleration lanes provided on SR 33 for the right-turn movements. Approximately 52.4 % of the crashes on the I-4 ramps were

**TABLE 5-4: CRASH TYPES (2007 – 2011)**

| SR 33 | | |
|---|---|---|
| **Crash Type** | **No. of Occurrences** | **% of Occurrences** |
| Angle | 8 | 28.57% |
| Left-Turn | 6 | 21.43% |
| Hit Guardrail | 4 | 14.29% |
| Sideswipe | 2 | 7.14% |
| Hit Sign/Sign Post | 2 | 7.14% |
| Ran Into Ditch/Culvert | 1 | 3.57% |
| Overturned | 1 | 3.57% |
| Hit Fixed Object Above Road | 1 | 3.57% |
| Unspecified | 3 | 10.71% |
| **Total** | **28** | **100.00%** |
| **I-4 On-/Off-Ramps** | | |
| **Crash Type** | **No. of Occurrences** | **% of Occurrences** |
| Rear-End | 11 | 52.38% |
| Angle | 3 | 14.29% |
| Backed Into | 2 | 9.52% |
| Hit Concrete Barrier Wall | 2 | 9.52% |
| Hit Fence | 1 | 4.76% |
| Ran Into Ditch/Culvert | 1 | 4.76% |
| Head-On | 1 | 4.76% |
| **Total** | **21** | **100.00%** |
| **I-4 Mainline** | | |
| **Crash Type** | **No. of Occurrences** | **% of Occurrences** |
| Hit Guardrail | 27 | 23.68% |
| Rear-End | 15 | 13.16% |
| Angle | 13 | 11.40% |
| Sideswipe | 9 | 7.89% |
| Overturned | 7 | 6.14% |
| Hit Concrete Barrier Wall | 6 | 5.26% |
| Ran Into Ditch/Culvert | 5 | 4.39% |
| Hit Fence | 4 | 3.51% |
| Hit Movable Object On Road | 4 | 3.51% |
| Hit Motor Vehicle on Side of Road | 3 | 2.63% |
| Hit Other Fixed Object | 2 | 1.75% |
| Cargo Loss | 2 | 1.75% |
| Separation of Units | 2 | 1.75% |
| Hit Sign/Sign Post | 1 | 0.88% |
| Hit Bridge/Pier/Abutment/Rail | 1 | 0.88% |
| Hit Animal | 1 | 0.88% |
| Unspecified | 12 | 10.53% |
| **Total** | **114** | **100.00%** |

rear-end crashes. This type of crash is often the most frequently occurring type of crash on diamond interchange ramps at unsignalized intersections. The two most prevalent types of crashes that occurred on the I-4 mainline in the vicinity of the I-4/SR 33 interchange were guardrail crashes (approximately 23.7 percent) and rear-end crashes (approximately 13.2%). However, it should also be noted that angle crashes and sideswipe crashes combined represent approximately 19.3% of the total I-4 mainline crashes.

The actual crash rate on I-4 for the years 2007 through 2011 was 0.980 crashes per million vehicle-miles of travel. The statewide average crash rate for urban interstate facilities during this same five-year period was 0.685 crashes per million vehicle-miles of travel. Consequently, the section of I-4 within the study area has experienced a higher crash rate than the statewide average for similar facilities. The actual crash rate on the portion of SR 33 within the immediate interchange area (i.e., from Milepost 8.288 to Milepost 8.590) for the years 2007 through 2011 was 7.105 crashes per million vehicle-miles of travel. The statewide average crash rate for similar facilities during this same five-year period was 2.514 crashes per million vehicle-miles of travel. Therefore, the actual crash rate for this portion of SR 33 is almost three times higher than the statewide average.

# 6.0 ALTERNATIVES CONSIDERED

Four alternative interchanges were evaluated and are documented in this report. These alternatives include one No-Build Alternative, one Transportation Systems Management (TSM) Alternative, and two Build Alternatives. The alternatives are described below:

- **No-Build Alternative –** The existing diamond interchange configuration with the existing laneage on both SR 33 and the I-4 ramps. The existing unsignalized ramp terminal intersections are also maintained.

- **TSM Alternative** – The existing diamond interchange configuration with roundabouts at the ramp terminals. The existing laneage on both SR 33 and the I-4 ramps was maintained, however, the conventional unsignalized intersections at the ramp terminals were replaced with one lane roundabouts.

- **Build Alternative No. 1 –** A diamond interchange configuration with four through lanes on SR 33 and additional turn lanes on both SR 33 and the I-4 off-ramps. The ramp terminal intersections are signalized.

- **Build Alternative No. 2 –** A diverging diamond interchange configuration with four through lanes on SR 33. The ramp terminal intersections are signalized.

# 7.0 FUTURE CONDITIONS TRAFFIC OPERATIONS ANALYSIS

With one exception, the opening year (2016) and design year (2036) I-4 ramp merge/diverge area analyses were conducted using the same factors that were incorporated into the Existing Year (2012) analyses. The opening year analyses used a PHF equal to 0.92, while a PHF value of 0.95 was used in the design year analyses.

## 7.1 NO-BUILD ALTERNATIVE

The No-Build Alternative interchange and intersection geometrics that were analyzed are graphically illustrated in **Figure 7-1**. The results of the No-Build Alternative opening year ramp merge/diverge area analyses are summarized in **Table 7-1**. All four of the merge/diverge areas are projected to operate at Level of Service C or better during both the a.m. and p.m. peak hours in the year 2016. **Table 7-2** summarizes the results of the design year ramp merge/diverge area analyses. Three of the four merge/diverge areas are projected to operate at Level of Service D or better during both the a.m. and p.m. peak hours in the year 2036. The eastbound I-4 off-ramp is projected to operate at Level of Service E in the a.m. peak hour and Level of Service D in the p.m. peak hour. The 2016 and 2036 No-Build Alternative ramp merge/diverge area analysis summary sheets are provided in **Appendix I**.

**TABLE 7-1: OPENING YEAR (2016) MERGE/DIVERGE AREA LEVELS OF SERVICE – NO-BUILD ALTERNATIVE**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 2,725 | 305 | 20.9 | C | 3,199 | 370 | 23.7 | C |
| EB I-4 Merge Area | 2,420 | 271 | 15.4 | B | 2,829 | 230 | 17.3 | B |
| WB I-4 Diverge Area | 2,546 | 210 | 19.7 | B | 3,216 | 311 | 23.7 | C |
| WB I-4 Merge Area | 2,336 | 430 | 16.3 | B | 2,905 | 392 | 19.0 | B |

**TABLE 7-2: DESIGN YEAR (2036) MERGE/DIVERGE AREA LEVELS OF SERVICE – NO-BUILD ALTERNATIVE**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 5,481 | 892 | 35.0 | E | 4,860 | 791 | 32.1 | D |
| EB I-4 Merge Area | 4,589 | 706 | 29.5 | D | 4,069 | 626 | 26.2 | C |
| WB I-4 Diverge Area | 4,695 | 626 | 31.1 | D | 5,295 | 706 | 33.9 | D |
| WB I-4 Merge Area | 4,069 | 791 | 27.5 | C | 4,589 | 892 | 31.0 | D |

The results of the No-Build Alternative opening year intersection analyses are summarized in **Table 7-3**. The No-Build Alternative assumed that the existing unsignalized I-4 ramp terminal intersections would remain in place as well as the existing unsignalized Tomkow Road intersection. These three unsignalized intersections were analyzed using the 2010 HCS. As stated earlier in **Section 2.1**, the

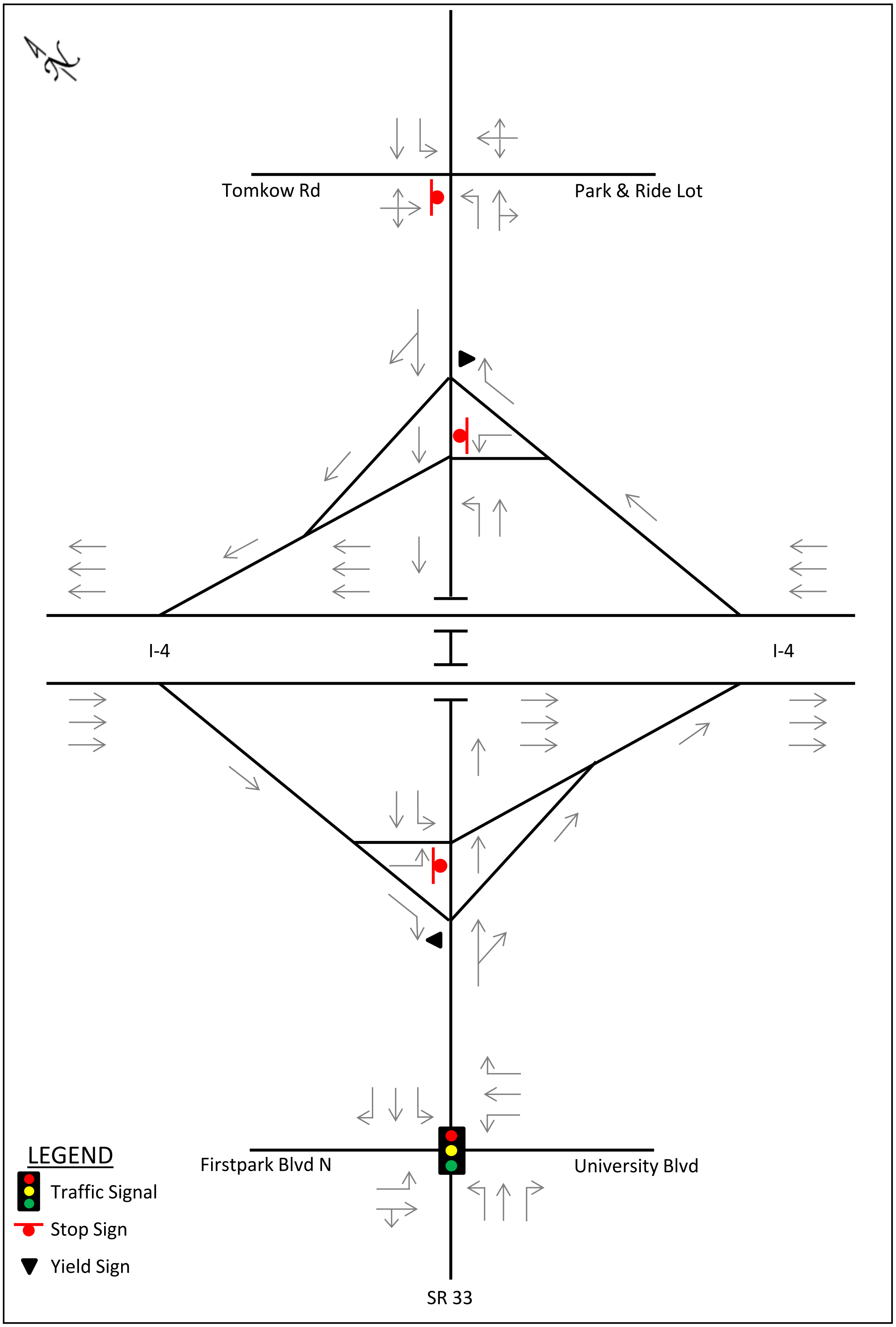


**FIGURE 7-1: DESIGN YEAR (2036) INTERCHANGE/INTERSECTION GEOMETRY – NO-BUILD ALTERNATIVE**

existing University Boulevard/Firstpark Boulevard N. intersection is currently operating as a two-way stop controlled intersection because the traffic signal that was installed as part of the University Boulevard construction is currently displaying flashing yellow for SR 33 and flashing red for University Boulevard/Firstpark Boulevard N. This intersection was analyzed both as an unsignalized two-way stop controlled intersection and as a signalized intersection. The signalized intersection analysis was conducted using the SYNCHRO software (Version 8).

### TABLE 7-3: OPENING YEAR (2016) PEAK HOUR INTERSECTION OPERATIONS – NO-BUILD ALTERNATIVE

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C [1] | Delay [2] | LOS [3] | V/C [1] | Delay [2] | LOS [3] |
| University Boulevard/Firstpark Boulevard N. (Unsignalized) | Eastbound | LT | 1.53 | 406.5 | F | 3.20 | 1,168.0 | F |
| | Eastbound | TH/RT | 0.19 | 24.5 | C | 0.38 | 29.0 | D |
| | Westbound | LT | 1.94 | 503.9 | F | 2.76 | 879.2 | F |
| | Westbound | TH | 0.19 | 30.6 | D | 0.17 | 30.1 | D |
| | Westbound | RT | 0.19 | 11.0 | B | 0.46 | 16.5 | C |
| | Northbound | LT | 0.01 | 8.4 | A | 0.01 | 8.5 | A |
| | Southbound | LT | 0.14 | 8.9 | A | 0.14 | 9.6 | A |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.24 | 16.3 | B | 0.22 | 17.2 | B |
| | Eastbound | TH/RT | 0.06 | 13.6 | B | 0.11 | 14.2 | B |
| | Eastbound | Approach | N/A | 15.4 | B | N/A | 16.1 | B |
| | Westbound | LT | 0.47 | 18.9 | B | 0.37 | 19.7 | B |
| | Westbound | TH | 0.05 | 18.5 | B | 0.04 | 21.4 | C |
| | Westbound | RT | 0.16 | 0.5 | A | 0.24 | 0.9 | A |
| | Westbound | Approach | N/A | 13.1 | B | N/A | 11.3 | B |
| | Northbound | LT | 0.07 | 16.6 | B | 0.05 | 13.7 | B |
| | Northbound | TH | 0.67 | 27.9 | C | 0.83 | 31.9 | C |
| | Northbound | RT | 0.11 | 0.2 | A | 0.12 | 0.1 | A |
| | Northbound | Approach | N/A | 18.2 | B | N/A | 22.9 | C |
| | Southbound | LT | 0.31 | 16.3 | B | 0.33 | 16.2 | B |
| | Southbound | TH | 0.72 | 25.5 | C | 0.44 | 14.4 | B |
| | Southbound | RT | 0.10 | 0.3 | A | 0.06 | 0.2 | A |
| | Southbound | Approach | N/A | 21.0 | C | N/A | 13.8 | B |
| | Overall Intersection | | N/A | 18.0 | B | N/A | 17.4 | B |
| I-4 Eastbound Ramps (Unsignalized) | Eastbound | LT | 0.68 | 36.7 | E | 0.94 | 70.8 | F |
| | Eastbound | RT | 0.36 | 15.9 | C | 0.16 | 57.1 | F |
| | Southbound | LT | 0.08 | 8.1 | A | 0.05 | 8.7 | A |
| I-4 Westbound Ramps (Unsignalized) | Westbound | LT | 0.66 | 41.1 | E | 1.07 | 125.8 | F |
| | Westbound | RT | 0.07 | 10.8 | B | 0.26 | 90.2 | F |
| | Northbound | LT | 0.12 | 9.3 | A | 0.14 | 8.3 | A |
| Tomkow Road (Unsignalized) | Eastbound | LT | 0.08 | 8.7 | A | 0.18 | 8.7 | A |
| | Westbound | LT | 0.00 | 7.9 | A | 0.00 | 8.5 | A |
| | Northbound | LT/TH/RT | 0.08 | 35.0 | D | 0.29 | 59.7 | F |
| | Southbound | LT/TH/RT | 0.58 | 22.2 | C | 0.29 | 16.4 | C |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

**Table 7-3** indicates that the eastbound and westbound left-turn movements at the I-4 ramp terminal intersections are projected to operate at Level of Service E in the a.m. peak hour with v/c ratios equal to 0.68 and 0.66, respectively. In the p.m. peak hour, both of these left-turn movements are projected to operate at Level of Service F with v/c ratios equal to 0.94 and 1.07, respectively. The average p.m. peak hour vehicle delays for the eastbound and westbound left-turn movements were estimated to be approximately 71 seconds/vehicle and 126 seconds/vehicle. In contrast, the initial average p.m. peak hour vehicle delays for the eastbound and westbound right-turn movements were estimated to be approximately 12 seconds/vehicle and 16 seconds/vehicle.

The approach that was used in the existing conditions analysis to obtain a more reasonable estimate of the eastbound off-ramp right-turn vehicle delay in the p.m. peak hour was also used to estimate the p.m. peak hour right-turn vehicle delays in the opening year analyses. Since the westbound left-turn movement is projected to operate over capacity in the p.m. peak hour, this methodology was utilized for the westbound right-turn movement as well as the eastbound right-turn movement. The analysis results indicated that the overall average eastbound and westbound approach delays were estimated to be approximately 57 seconds/vehicle and 90 seconds/vehicle, respectively. The use of these delay values as estimates for the right-turn vehicle delays was viewed as being more reasonable considering the magnitude of the p.m. peak hour v/c ratios for the left-turn movements and the amount of left-turn storage provided between SR 33 and the entrances to the channelized right-turn lanes.

A majority of the movements at the University Boulevard/Firstpark Boulevard N. intersection are projected to operate at Level of Service D or better during both peak hours under two-way stop control. Only the westbound and eastbound left-turn movements from University Boulevard and Firstpark Boulevard N. are projected to operate at Level of Service F. The v/c ratios for both of these movements are projected to be greater than 1.5. All of the movements at this intersection are projected to operate at Level of Service C or better during both peak hours under full signal control. These results indicate that the existing flashing yellow (SR 33)/flashing red (University Boulevard/Firstpark Boulevard N.) operations at this intersection will need to be converted to standard traffic signal operations before the year 2016 to reduce the future delays experienced by the cross street left-turn vehicles.

All of the movements at the Tomkow Road intersection are projected to operate at Level of Service D or better during the a.m. peak hour. In the p.m. peak hour, the northbound left-turn, through, and right-turn movements exiting the park-and-ride lot across from Tomkow Road are projected to operate at Level of Service F. Although these movements are projected to operate at Level of Service F, it should be noted that the northbound volume is very low (i.e., 11 vehicles), and as a result, the v/c ratio is only 0.29.

The results of the No-Build Alternative design year intersection analyses are summarized in **Table 7-4**. The eastbound and westbound left-turn movements at the I-4 ramp terminal intersections are projected to operate at Level of Service F during both the a.m. and p.m. peak hours. The v/c ratios for these left-turn movements are estimated to be greater than or equal to 3.58. It should be noted that the HCS software was unable to calculate a v/c ratio for the p.m. peak hour westbound left-turn movement since there is no capacity available for this movement. The eastbound and westbound right-turn movements at the I-4 ramp terminal intersections are also expected to operate at Level of Service F during both the a.m. and p.m. peak hours, due in large part, to the severe overcapacity conditions projected for the left-turn movements and the inadequate lengths of the right-turn lanes.

**TABLE 7-4: DESIGN YEAR (2036) PEAK HOUR INTERSECTION OPERATIONS – NO-BUILD ALTERNATIVE**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.68 | 44.4 | D | 0.71 | 45.8 | D |
| | Eastbound | TH/RT | 0.78 | 92.5 | F | 0.88 | 103.4 | F |
| | Eastbound | Approach | N/A | 63.7 | E | N/A | 69.1 | E |
| | Westbound | LT | 1.63 | 321.1 | F | 1.64 | 372.2 | F |
| | Westbound | TH | 0.23 | 38.4 | D | 0.20 | 46.1 | D |
| | Westbound | RT | 0.69 | 13.6 | B | 0.88 | 32.0 | C |
| | Westbound | Approach | N/A | 192.0 | F | N/A | 187.3 | F |
| | Northbound | LT | 0.46 | 66.3 | E | 0.46 | 55.2 | E |
| | Northbound | TH | 1.21 | 165.7 | F | 1.34 | 208.0 | F |
| | Northbound | RT | 0.58 | 1.6 | A | 0.68 | 2.5 | A |
| | Northbound | Approach | N/A | 55.8 | E | N/A | 80.3 | F |
| | Southbound | LT | 1.61 | 319.7 | F | 1.58 | 309.9 | F |
| | Southbound | TH | 0.88 | 55.1 | E | 0.61 | 34.8 | C |
| | Southbound | RT | 0.21 | 4.5 | A | 0.12 | 1.9 | A |
| | Southbound | Approach | N/A | 174.4 | F | N/A | 177.9 | F |
| | Overall intersection | | N/A | 142.3 | F | N/A | 139.1 | F |
| I-4 Eastbound Ramps (Unsignalized) | Eastbound | LT | 3.58 | 1,249.0 | F | 5.34 | 2,040.0 | F |
| | Eastbound | RT | 2.01 | 792.4 | F | 1.06 | 1,284.0 | F |
| | Southbound | LT | 0.19 | 10.0 | B | 0.15 | 11.5 | B |
| I-4 Westbound Ramps (Unsignalized) | Westbound | LT | 9.22 | 3,832.0 | F | * | ** | F |
| | Westbound | RT | 0.28 | 3,171.0 | F | 0.47 | *** | F |
| | Northbound | LT | 0.34 | 10.9 | B | 0.48 | 11.3 | B |
| Tomkow Road (Unsignalized) | Eastbound | LT | 0.34 | 11.6 | B | 0.42 | 11.6 | B |
| | Westbound | LT | 0.01 | 8.6 | A | 0.00 | 9.2 | A |
| | Northbound | LT/TH/RT | * | ** | F | 2.00 | 1,145.0 | F |
| | Southbound | LT/TH/RT | 2.19 | 592.0 | F | 1.90 | 475.8 | F |

(1) Volume-to-Capacity Ratio
(2) Average Delay (seconds/vehicle)
(3) Level of Service
* Theoretically, the capacity for this movement is equal to zero; therefore, the v/c ratio is infinite
** No estimate of delay is provided since the v/c ratio is infinite
*** No estimate of delay is provided since the v/c ratio for the westbound left-turn movement is infinite

The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service F overall during both the a.m. and p.m. peak hours. In addition, three of the individual movements (i.e., the westbound left-turn, southbound left-turn and northbound through movements) are projected to have v/c ratios greater than 1.2 during both peak hours.

The northbound and southbound approaches at the Tomkow Road intersection are both projected to operate at Level of Service F during the a.m. and p.m. peak hours. Overcapacity conditions are projected to occur during the peak hours since the v/c ratios for both of these approaches are greater than 1.00. These results indicate that a traffic signal may need to be implemented at the existing Tomkow Road intersection prior to or by the design year to continue providing acceptable operations for the cross street movements. The 2016 and 2036 No-Build Alternative intersection analysis summary sheets are also provided in **Appendix I**.

## 7.2 TSM ALTERNATIVE

The TSM Alternative interchange and intersection geometrics that were analyzed are graphically illustrated in **Figure 7-2**. The results of the TSM Alternative opening year ramp merge/diverge area analyses are summarized in **Table 7-5**. All four of the merge/diverge areas are projected to operate at Level of Service C or better during both the a.m. and p.m. peak hours in the year 2016. **Table 7-6** summarizes the results of the design year ramp merge/diverge area analyses. Three of the four merge/diverge areas are projected to operate at Level of Service D or better during both the a.m.

**TABLE 7-5: OPENING YEAR (2016) MERGE/DIVERGE AREA LEVELS OF SERVICE – TSM ALTERNATIVE**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 2,725 | 305 | 20.9 | C | 3,199 | 370 | 23.7 | C |
| EB I-4 Merge Area | 2,420 | 271 | 15.4 | B | 2,829 | 230 | 17.3 | B |
| WB I-4 Diverge Area | 2,546 | 210 | 19.7 | B | 3,216 | 311 | 23.7 | C |
| WB I-4 Merge Area | 2,336 | 430 | 16.3 | B | 2,905 | 392 | 19.0 | B |

**TABLE 7-6: DESIGN YEAR (2036) MERGE/DIVERGE AREA LEVELS OF SERVICE – TSM ALTERNATIVE**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 5,481 | 892 | 35.0 | E | 4,860 | 791 | 32.1 | D |
| EB I-4 Merge Area | 4,589 | 706 | 29.5 | D | 4,069 | 626 | 26.2 | C |
| WB I-4 Diverge Area | 4,695 | 626 | 31.1 | D | 5,295 | 706 | 33.9 | D |
| WB I-4 Merge Area | 4,069 | 791 | 27.5 | C | 4,589 | 892 | 31.0 | D |

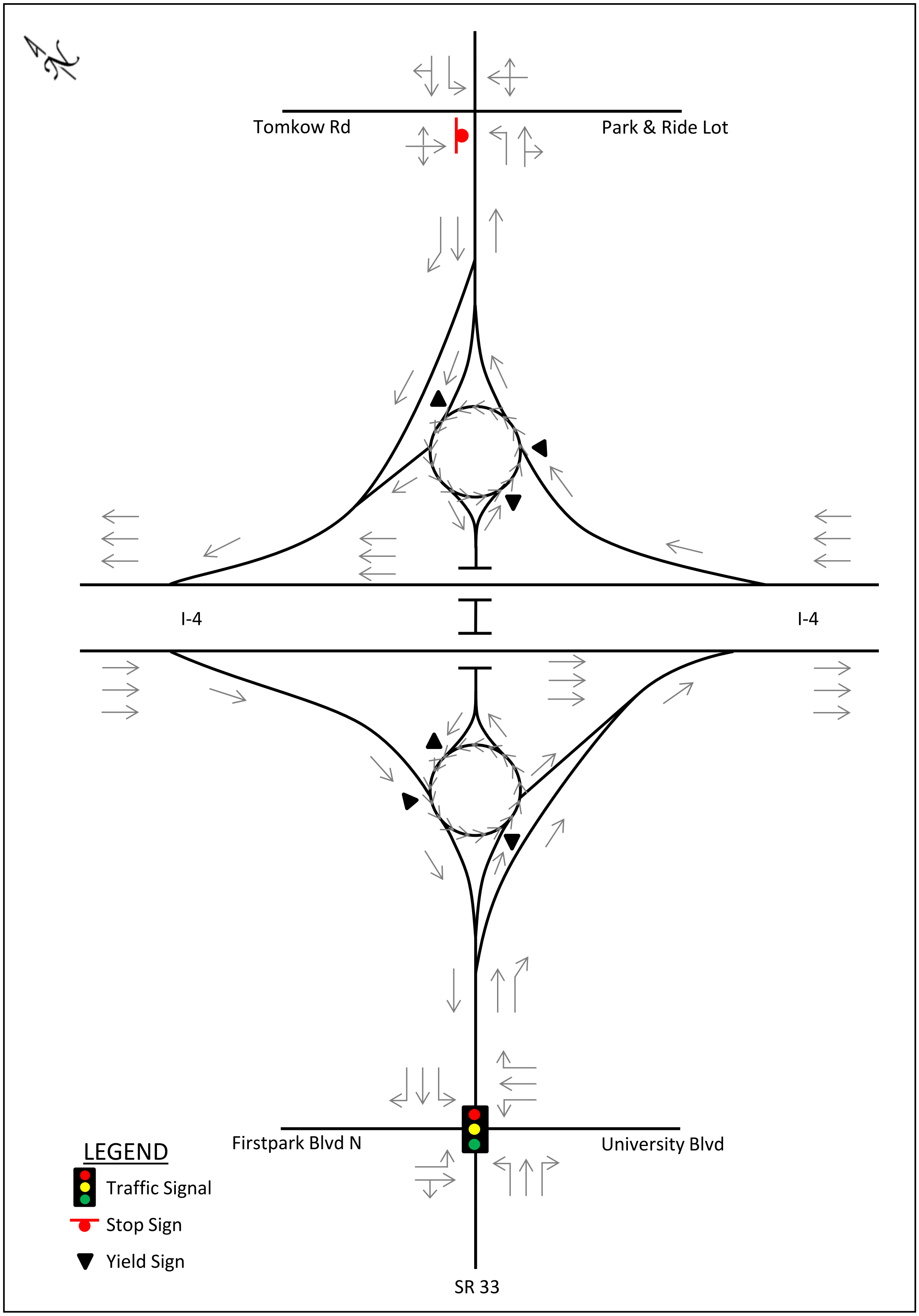


**FIGURE 7-2: DESIGN YEAR (2036) INTERCHANGE/INTERSECTION GEOMETRY – TSM ALTERNATIVE**

and p.m. peak hours in the year 2036. The eastbound I-4 off-ramp is projected to operate at Level of Service E in the a.m. peak hour and Level of Service D in the p.m. peak hour. The 2016 and 2036 TSM Alternative ramp merge/diverge area analysis summary sheets are provided in **Appendix J**.

The results of the TSM Alternative opening year intersection analyses are summarized in **Table 7-7**. The TSM Alternative assumed that single lane roundabouts would be constructed at the existing I-4 ramp terminal intersections. The existing Tomkow Road intersection was assumed to remain unsignalized while the University Boulevard/Firstpark Boulevard N. intersection was analyzed both as an unsignalized intersection and a signalized intersection.

The initial HCS roundabout analysis results indicated that the southbound approach at the westbound I-4 ramp terminal intersection and the northbound approach at the eastbound I-4 ramp terminal intersection were projected to be overcapacity during the a.m. and p.m. peak hours, respectively. The v/c ratios for the southbound and northbound intersection approaches were estimated to be 1.11 and 1.06, respectively. Consequently, the opening year roundabout analyses were modified to include the provision of yield controlled bypass lanes for the southbound and northbound right-turn movements. The results of these roundabout analyses are summarized in **Table 7-7**. In the a.m. peak hour, all of the movements at both of the ramp terminal intersections are projected to operate at Level of Service D or better. In the p.m. peak hour, the westbound left-turn and right-turn movements are projected to operate at Level of Service E, while all of the other movements are projected to operate at Level of Service C or better.

A majority of the movements at the University Boulevard/Firstpark Boulevard N. intersection are projected to operate at Level of Service D or better during both peak hours under two-way stop control. Only the westbound and eastbound left-turn movements from University Boulevard and Firstpark Boulevard N. are projected to operate at Level of Service F. The v/c ratios for both of these movements are projected to be greater than 1.53. All of the movements at this intersection are projected to operate at Level of Service C or better during both peak hours under full signal control. These results indicate that the existing flashing yellow (SR 33)/flashing red (University Boulevard/Firstpark Boulevard N.) operations at this intersection will need to be converted to standard traffic signal operations before the year 2016 to reduce the future delays experienced by the cross street left-turn vehicles.

In the a.m. peak hour, all of the movements at the Tomkow Road intersection are projected to operate at Level of Service D or better. In the p.m. peak hour, the northbound left-turn, through, and right-turn movements exiting the park-and-ride lot across from Tomkow Road are projected to operate at Level of Service F. Although these movements are projected to operate at Level of Service F, it should be noted that the northbound volume is very low (i.e., 11 vehicles), and as a result, the v/c ratio is only 0.29.

## TABLE 7-7: OPENING YEAR (2016) PEAK HOUR INTERSECTION OPERATIONS – TSM ALTERNATIVE

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Unsignalized) | Eastbound | LT | 1.53 | 406.5 | F | 3.20 | 1,168.0 | F |
| | Eastbound | TH/RT | 0.19 | 24.5 | C | 0.38 | 29.0 | D |
| | Westbound | LT | 1.94 | 503.9 | F | 2.76 | 879.2 | F |
| | Westbound | TH | 0.19 | 30.6 | D | 0.17 | 30.1 | D |
| | Westbound | RT | 0.19 | 11.0 | B | 0.46 | 16.5 | C |
| | Northbound | LT | 0.01 | 8.4 | A | 0.01 | 8.5 | A |
| | Southbound | LT | 0.14 | 8.9 | A | 0.14 | 9.6 | A |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.24 | 16.3 | B | 0.22 | 17.2 | B |
| | Eastbound | TH/RT | 0.06 | 13.6 | B | 0.11 | 14.2 | B |
| | Eastbound | Approach | N/A | 15.4 | B | N/A | 16.1 | B |
| | Westbound | LT | 0.47 | 18.9 | B | 0.37 | 19.7 | B |
| | Westbound | TH | 0.05 | 18.5 | B | 0.04 | 21.4 | C |
| | Westbound | RT | 0.16 | 0.5 | A | 0.24 | 0.9 | A |
| | Westbound | Approach | N/A | 13.1 | B | N/A | 11.3 | B |
| | Northbound | LT | 0.07 | 16.6 | B | 0.05 | 13.7 | B |
| | Northbound | TH | 0.67 | 27.9 | C | 0.83 | 31.9 | C |
| | Northbound | RT | 0.11 | 0.2 | A | 0.12 | 0.1 | A |
| | Northbound | Approach | N/A | 18.2 | B | N/A | 22.9 | C |
| | Southbound | LT | 0.31 | 16.3 | B | 0.33 | 16.2 | B |
| | Southbound | TH | 0.72 | 25.5 | C | 0.44 | 14.4 | B |
| | Southbound | RT | 0.10 | 0.3 | A | 0.06 | 0.2 | A |
| | Southbound | Approach | N/A | 21.0 | C | N/A | 13.8 | B |
| | Overall Intersection | | N/A | 18.0 | B | N/A | 17.4 | B |
| I-4 EB Ramps (Roundabout) | Eastbound | LT | 0.69 | 26.5 | D | 0.66 | 20.6 | C |
| | Eastbound | RT | 0.69 | 26.5 | D | 0.66 | 20.6 | C |
| | Eastbound | Approach | N/A | 26.5 | D | N/A | 20.6 | C |
| | Northbound | TH | 0.41 | 10.5 | B | 0.76 | 23.6 | C |
| | Northbound | RT | 0.00 | 4.1 | A | 0.00 | 3.8 | A |
| | Northbound | Approach | N/A | 10.5 | B | N/A | 23.6 | C |
| | Southbound | LT | 0.61 | 12.2 | B | 0.45 | 8.6 | A |
| | Southbound | TH | 0.61 | 12.2 | B | 0.45 | 8.6 | A |
| | Southbound | Approach | N/A | 12.2 | B | N/A | 8.6 | A |
| | Overall Intersection | | N/A | 15.6 | C | N/A | 17.8 | C |
| I-4 WB Ramps (Roundabout) | Westbound | LT | 0.43 | 13.7 | B | 0.82 | 42.0 | E |
| | Westbound | RT | 0.43 | 13.7 | B | 0.82 | 42.0 | E |
| | Westbound | Approach | N/A | 13.7 | B | N/A | 42.0 | E |
| | Northbound | LT | 0.52 | 10.5 | B | 0.82 | 22.0 | C |
| | Northbound | TH | 0.52 | 10.5 | B | 0.82 | 22.0 | C |
| | Northbound | Approach | N/A | 10.5 | B | N/A | 22.0 | C |
| | Southbound | TH | 0.63 | 17.4 | C | 0.35 | 10.4 | B |
| | Southbound | RT | 0.00 | 4.4 | A | 0.00 | 4.4 | A |
| | Southbound | Approach | N/A | 17.4 | C | N/A | 10.4 | B |
| | Overall Intersection | | N/A | 13.8 | B | N/A | 24.9 | C |
| Tomkow Road (Unsignalized) | Eastbound | LT | 0.08 | 8.7 | A | 0.18 | 8.7 | A |
| | Westbound | LT | 0.00 | 7.9 | A | 0.00 | 8.5 | A |
| | Northbound | LT/TH/RT | 0.08 | 35.0 | D | 0.29 | 59.7 | F |
| | Southbound | LT/TH/RT | 0.58 | 22.2 | C | 0.29 | 16.4 | C |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

The results of the TSM Alternative design year intersection analyses are summarized in **Table 7-8**. With two exceptions, all of the movements at the I-4 ramp terminal intersections are projected to operate at Level of Service E or F during both peak hours. Although the northbound and southbound right-turn movements are projected to operate at Level of Service A, the severe overcapacity conditions projected for the northbound and southbound through movements are expected to result in the formation of long through vehicle queues. **Table 7-9** summarizes the design year a.m. and p.m. peak hour 95$^{th}$- percentile queue length estimates obtained from the HCS roundabout analyses. Based on these queue length estimates, the beginning of the northbound and southbound right-turn bypass lane channelization would need to occur 1,650 feet and 850 feet upstream of the roundabouts.

The severe overcapacity conditions projected for the eastbound and westbound off-ramp movements are also expected to result in the formation of long off-ramp queues. The 95$^{th}$-percentile a.m. peak hour queue length for the eastbound off-ramp is estimated to be 2,100 feet while the 95$^{th}$-percentile p.m. peak hour queue length for the westbound off-ramp is estimated to be 1,775 feet. Since the length of the existing off-ramps (as measured from the SR 33 stop bars to the beginning of the I-4 mainline gore areas) is approximately 1,250 feet; the 95$^{th}$-percentile ramp queues would be expected to extend back onto the I-4 mainline creating an unsafe condition. Although the off-ramp delays and queue lengths could be reduced with the provision of separate yield controlled bypass lanes for the eastbound and westbound right-turn movements, the analysis results provided in **Table 7-8** also indicate that the northbound and southbound left-turn and through movements are projected to be significantly overcapacity during both peak hours. These results help demonstrate the need to widen SR 33 from two lanes to four lanes prior to the year 2036.

The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service F overall during both the a.m. and p.m. peak hours. In addition, three of the individual movements (i.e., the westbound left-turn, southbound left-turn and northbound through movements) are projected to have v/c ratios greater than 1.2 during both peak hours.

The northbound and southbound approaches at the Tomkow Road intersection are both projected to operate at Level of Service F during the a.m. and p.m. peak hours. Overcapacity conditions are projected to occur during the peak hours since the v/c ratios for both of these approaches are greater than 1.00. These results indicate that a traffic signal may need to be implemented at the existing Tomkow Road intersection prior to or by the design year to continue providing acceptable operations for the cross street movements. The 2016 and 2036 No-Build Alternative intersection analysis summary sheets are also provided in **Appendix J**.

Additional interim year intersection analyses were conducted for the I-4 ramp terminal intersections to obtain an estimate of the “useful life” of the single lane roundabouts. The additional intersection analyses were conducted for the years 2021 and 2026 and the analysis results are summarized in **Table 7-10** and **Table 7-11**, respectively. In the year 2021, overcapacity conditions are projected to occur for the eastbound off-ramp approach in the a.m. peak hour (with a v/c ratio equal to 1.04) and the westbound off-ramp approach in the p.m. peak hour (with a v/c ratio equal to 1.23).

**TABLE 7-8: DESIGN YEAR (2036) PEAK HOUR INTERSECTION OPERATIONS – TSM ALTERNATIVE**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.68 | 44.4 | D | 0.71 | 45.8 | D |
| | Eastbound | TH/RT | 0.78 | 92.5 | F | 0.88 | 103.4 | F |
| | Eastbound | Approach | N/A | 63.7 | E | N/A | 69.1 | E |
| | Westbound | LT | 1.63 | 321.1 | F | 1.64 | 372.2 | F |
| | Westbound | TH | 0.23 | 38.4 | D | 0.20 | 46.1 | D |
| | Westbound | RT | 0.69 | 13.6 | B | 0.88 | 32.0 | C |
| | Westbound | Approach | N/A | 192.0 | F | N/A | 187.3 | F |
| | Northbound | LT | 0.46 | 66.3 | E | 0.46 | 55.2 | E |
| | Northbound | TH | 1.21 | 165.7 | F | 1.34 | 208.0 | F |
| | Northbound | RT | 0.58 | 1.6 | A | 0.68 | 2.5 | A |
| | Northbound | Approach | N/A | 55.8 | E | N/A | 80.3 | F |
| | Southbound | LT | 1.61 | 319.7 | F | 1.58 | 309.9 | F |
| | Southbound | TH | 0.88 | 55.1 | E | 0.61 | 34.8 | C |
| | Southbound | RT | 0.21 | 4.5 | A | 0.12 | 1.9 | A |
| | Southbound | Approach | N/A | 174.4 | F | N/A | 177.9 | F |
| | Overall intersection | | N/A | 142.3 | F | N/A | 139.1 | F |
| I-4 EB Ramps (Roundabout) | Eastbound | LT | 3.14 | 998.0 | F | 2.44 | 681.9 | F |
| | Eastbound | RT | 3.14 | 998.0 | F | 2.44 | 681.9 | F |
| | Eastbound | Approach | N/A | 998.0 | F | N/A | 681.9 | F |
| | Northbound | TH | 1.16 | 115.4 | F | 1.88 | 420.0 | F |
| | Northbound | RT | 0.00 | 4.1 | A | 0.00 | 3.8 | A |
| | Northbound | Approach | N/A | 115.4 | F | N/A | 420.0 | F |
| | Southbound | LT | 1.12 | 83.4 | F | 1.00 | 47.8 | E |
| | Southbound | TH | 1.12 | 83.4 | F | 1.00 | 47.8 | E |
| | Southbound | Approach | N/A | 83.4 | F | N/A | 47.8 | E |
| | Overall Intersection | | N/A | 396.6 | F | N/A | 360.1 | F |
| I-4 WB Ramps (Roundabout) | Westbound | LT | 1.95 | 463.7 | F | 3.66 | 1,243.7 | F |
| | Westbound | RT | 1.95 | 463.7 | F | 3.66 | 1,243.7 | F |
| | Westbound | Approach | N/A | 463.7 | F | N/A | 1,243.7 | F |
| | Northbound | LT | 1.01 | 49.8 | E | 1.46 | 225.8 | F |
| | Northbound | TH | 1.01 | 49.8 | E | 1.46 | 225.8 | F |
| | Northbound | Approach | N/A | 49.8 | E | N/A | 225.8 | F |
| | Southbound | TH | 1.50 | 263.3 | F | 1.42 | 237.1 | F |
| | Southbound | RT | 0.00 | 4.8 | A | 0.00 | 6.0 | A |
| | Southbound | Approach | N/A | 263.3 | F | N/A | 237.1 | F |
| | Overall Intersection | | N/A | 222.9 | F | N/A | 504.7 | F |
| Tomkow Road (Unsignalized) | Eastbound | LT | 0.34 | 11.6 | B | 0.42 | 11.6 | B |
| | Westbound | LT | 0.01 | 8.6 | A | 0.00 | 9.2 | A |
| | Northbound | LT/TH/RT | * | ** | F | 2.00 | 1,145.0 | F |
| | Southbound | LT/TH/RT | 2.19 | 592.0 | F | 1.90 | 475.8 | F |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

* Theoretically, the capacity for this movement is equal to zero. Therefore, the v/c ratio is infinite.

** No estimate of delay is provided since the v/c ratio is infinite.

**TABLE 7-9: DESIGN YEAR (2036) 95TH-PERCENTILE QUEUE LENGTHS – TSM ALTERNATIVE**

| Intersection | Approach | Movement | AM Peak Hour Queue | | PM Peak Hour Queue | |
|---|---|---|---|---|---|---|
| | | | No. of Vehicles | Feet (1) | No. of Vehicles | Feet (1) |
| I-4 EB Ramps (Roundabout) | Eastbound | LT/RT | 84 | 2,100 | 66 | 1,650 |
| | Northbound | TH | 23 | 575 | 66 | 1,650 |
| | Southbound | LT/TH | 30 | 750 | 20 | 500 |
| I-4 WB Ramps (Roundabout) | Westbound | LT/RT | 46 | 1,150 | 71 | 1,775 |
| | Northbound | LT/TH | 21 | 525 | 69 | 1,725 |
| | Southbound | TH | 34 | 850 | 24 | 600 |

(1) Queue length assumes an average vehicle spacing of 25 feet

**TABLE 7-10: INTERIM YEAR (2021) PEAK HOUR ROUNDABOUT OPERATIONS – TSM ALTERNATIVE**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| I-4 EB Ramps (Roundabout) | Eastbound | LT | 1.04 | 83.4 | F | 0.93 | 50.6 | F |
| | Eastbound | RT | 1.04 | 83.4 | F | 0.93 | 50.6 | F |
| | Eastbound | Approach | N/A | 83.4 | F | N/A | 50.6 | F |
| | Northbound | TH | 0.55 | 13.7 | B | 0.98 | 54.2 | F |
| | Northbound | RT | 0.00 | 3.9 | A | 0.00 | 3.7 | A |
| | Northbound | Approach | N/A | 13.7 | B | N/A | 54.2 | F |
| | Southbound | LT | 0.72 | 15.1 | C | 0.58 | 11.0 | B |
| | Southbound | TH | 0.72 | 15.1 | C | 0.58 | 11.0 | B |
| | Southbound | Approach | N/A | 15.1 | C | N/A | 11.0 | B |
| | Overall Intersection | | N/A | 35.1 | E | N/A | 38.3 | E |
| I-4 WB Ramps (Roundabout) | Westbound | LT | 0.64 | 21.0 | C | 1.23 | 159.2 | F |
| | Westbound | RT | 0.64 | 21.0 | C | 1.23 | 159.2 | F |
| | Westbound | Approach | N/A | 21.0 | C | N/A | 159.2 | F |
| | Northbound | LT | 0.60 | 11.4 | B | 0.96 | 39.3 | E |
| | Northbound | TH | 0.60 | 11.4 | B | 0.96 | 39.3 | E |
| | Northbound | Approach | N/A | 11.4 | B | N/A | 39.3 | E |
| | Southbound | TH | 0.73 | 22.6 | C | 0.51 | 15.6 | C |
| | Southbound | RT | 0.00 | 4.1 | A | 0.00 | 4.6 | A |
| | Southbound | Approach | N/A | 22.6 | C | N/A | 15.6 | C |
| | Overall Intersection | | N/A | 17.4 | C | N/A | 65.6 | F |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

**TABLE 7-11: INTERIM YEAR (2026) PEAK HOUR ROUNDABOUT OPERATIONS – TSM ALTERNATIVE**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| I-4 EB Ramps (Roundabout) | Eastbound | LT | 1.53 | 277.4 | F | 1.29 | 173.1 | F |
| | Eastbound | RT | 1.53 | 277.4 | F | 1.29 | 173.1 | F |
| | Eastbound | Approach | N/A | 277.4 | F | N/A | 173.1 | F |
| | Northbound | TH | 0.71 | 20.8 | C | 1.23 | 137.0 | F |
| | Northbound | RT | 0.00 | 3.9 | A | 0.00 | 3.7 | A |
| | Northbound | Approach | N/A | 20.8 | C | N/A | 137.0 | F |
| | Southbound | LT | 0.84 | 22.1 | C | 0.71 | 14.9 | B |
| | Southbound | TH | 0.84 | 22.1 | C | 0.71 | 14.9 | B |
| | Southbound | Approach | N/A | 22.1 | C | N/A | 14.9 | B |
| | Overall Intersection | | N/A | 102.1 | F | N/A | 104.4 | F |
| I-4 WB Ramps (Roundabout) | Westbound | LT | 0.94 | 58.5 | F | 1.79 | 398.4 | F |
| | Westbound | RT | 0.94 | 58.5 | F | 1.79 | 398.4 | F |
| | Westbound | Approach | N/A | 58.5 | F | N/A | 398.4 | F |
| | Northbound | LT | 0.72 | 15.4 | C | 1.12 | 83.4 | F |
| | Northbound | TH | 0.72 | 15.4 | C | 1.12 | 83.4 | F |
| | Northbound | Approach | N/A | 15.4 | C | N/A | 83.4 | F |
| | Southbound | TH | 0.91 | 45.8 | E | 0.72 | 28.8 | D |
| | Southbound | RT | 0.00 | 4.3 | A | 0.00 | 5.0 | A |
| | Southbound | Approach | N/A | 45.8 | E | N/A | 28.8 | D |
| | Overall Intersection | | N/A | 35.6 | E | N/A | 156.8 | F |

(1) Volume-to-Capacity Ratio
(2) Average Delay (seconds/vehicle)
(3) Level of Service

These off-ramp approaches are projected to operate at Level of Service F during these peak hours with average delays of approximately 83 seconds/vehicle and 159 seconds/vehicle, respectively. The 2021 p.m. peak hour analyses also indicate that the northbound movements are approaching capacity in the p.m. peak hour. The v/c ratio for the northbound through movement at the eastbound I-4 ramp terminal intersection is estimated to be 0.98 while the v/c ratio for the northbound through and left-turn movements at the westbound I-4 ramp terminal intersection is estimated to be 0.96.

The 2026 p.m. peak hour analyses indicate that the northbound movements are significantly overcapacity. The v/c ratio for the northbound through movement at the eastbound I-4 ramp terminal intersection is estimated to be 1.22 while the v/c ratio for the northbound through and left-turn movements at the westbound I-4 ramp terminal intersection is estimated to be 1.12. These movements are projected to operate at Level of Service F with average delays of approximately 136 seconds/vehicle and 83 seconds/vehicle, respectively. The eastbound off-ramp approach is projected to have v/c ratios equal to 1.53 and 1.29 in the a.m. and p.m. peak hours, while the westbound off-ramp approach is projected to have a v/c ratio equal to 1.79 in the p.m. peak hour. These ramps are projected to have average delays ranging between 173 seconds/vehicle and 398 seconds/vehicle. These results suggest that the capacity of the single lane roundabouts at the I-4/SR 33 interchange will likely be exceeded sometime between the years 2021 and 2022 (i.e., five or six years after the opening year). The 2021 and 2026 roundabout analysis summary sheets are also provided in **Appendix J**.

## 7.3 BUILD ALTERNATIVE NO. 1

The Build Alternative No. 1 diamond interchange concept is depicted in **Figure 7-3** and on the plan sheets provided in **Appendix K**. Dual left-turn lanes are provided on both the eastbound and westbound I-4 off-ramps and dual right-turn lanes are provided on the eastbound I-4 off-ramp. In addition, dual left-turn lanes are also provided on northbound SR 33 at the entrance to the westbound I-4 on-ramp. Build Alternative No. 1 also provides a longer deceleration lane on the I-4 mainline for the eastbound off-ramp. The length of this deceleration lane is increased from 215 feet to 300 feet. This alternative interchange improvement concept provides a 2,850-foot crest vertical curve that has a K-value of 506 and two 600-foot approach sag vertical curves that have K-values of 206. These vertical curves allow for a maximum design speed of 70 mph based on Volume 1 of the FDOT's PPM. This interchange concept also provides 16.5 feet of vertical clearance under the I-4 bridges which satisfies the minimum required by the FDOT's PPM. The proposed typical section of SR 33 under the I-4 bridges is provided in **Figure 7-4**.

Build Alternative No. 1 includes improvements to the University Boulevard/Firstpark Boulevard N. intersection. In addition to the two northbound and southbound through lanes, dual left-turn lanes are provided on the southbound and westbound intersection approaches while dual right-turn lanes are provided on the northbound intersection approach. Build Alternative No. 1 also includes a realignment of Tomkow Road. The existing Tomkow Road intersection and the existing entrance/exit to the park-and-ride lot are both currently located within the existing limited access right-of-way for the interchange. The Tomkow Road intersection is located approximately 720 feet north/east of the beginning of the southbound SR 33 right-turn lane onto westbound I-4. Similarly, the park-and-ride lot entrance/exit is located approximately 775 feet north/east of the westbound I-4 right-turn lane onto northbound SR 33. The proposed diamond interchange concept shifts both the southbound SR 33 right-turn lane and the westbound I-4 right-turn lane further to the north/east of their current junctions with the SR 33 mainline. The beginning of the southbound SR 33 right-turn lane is located at the existing Tomkow Road intersection while the stop bar for the westbound I-4 right-turn lane is located approximately 715 feet to the south/west of the park-and-ride lot access.

Although signalization of the westbound I-4 right-turn lane and the Tomkow Road intersection (in combination with prohibiting any right-turn-on-red movements) would eliminate any high-speed merging and weaving conflicts between the right-turn vehicles and the northbound/southbound SR 33 vehicles; the close proximity of these right-turn lanes to the existing Tomkow Road intersection precludes the ability to provide drivers with adequate advanced signing for both Tomkow Road and the westbound I-4 on-ramp. Consequently, Build Alternative No. 1 also includes a realignment of Tomkow Road. Approximately 240 feet north of the existing intersection, Tomkow Road is realigned to run parallel to SR 33 within the existing right-of-way that exists on the north side of SR 33. The relocated SR 33/Tomkow Road intersection is located approximately 1,450 feet east of the existing intersection. The Tomkow Road realignment concept is depicted in **Figure 7-5**.

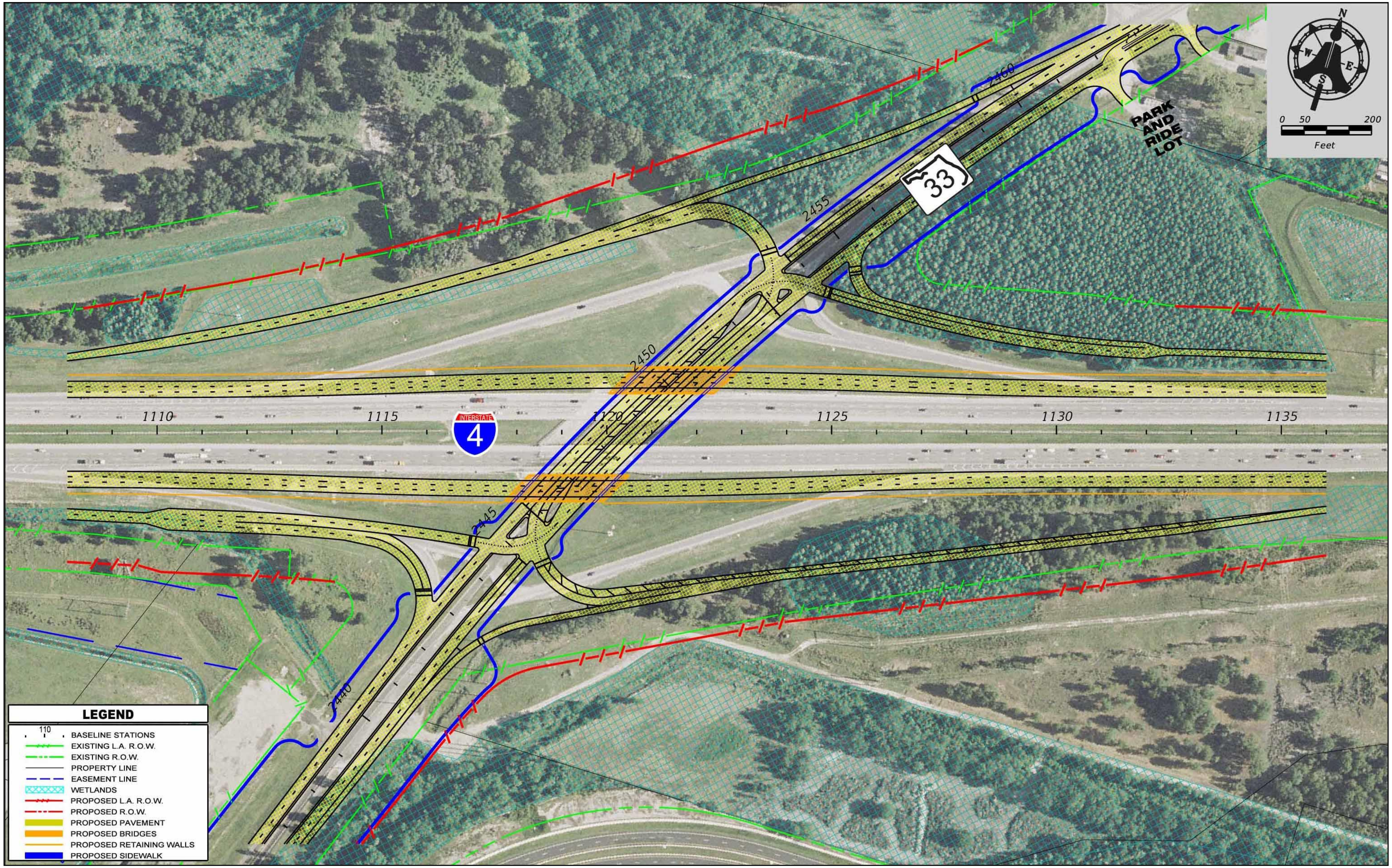


**FIGURE 7-3: BUILD ALTERNATIVE NO. 1 (DIAMOND INTERCHANGE)**

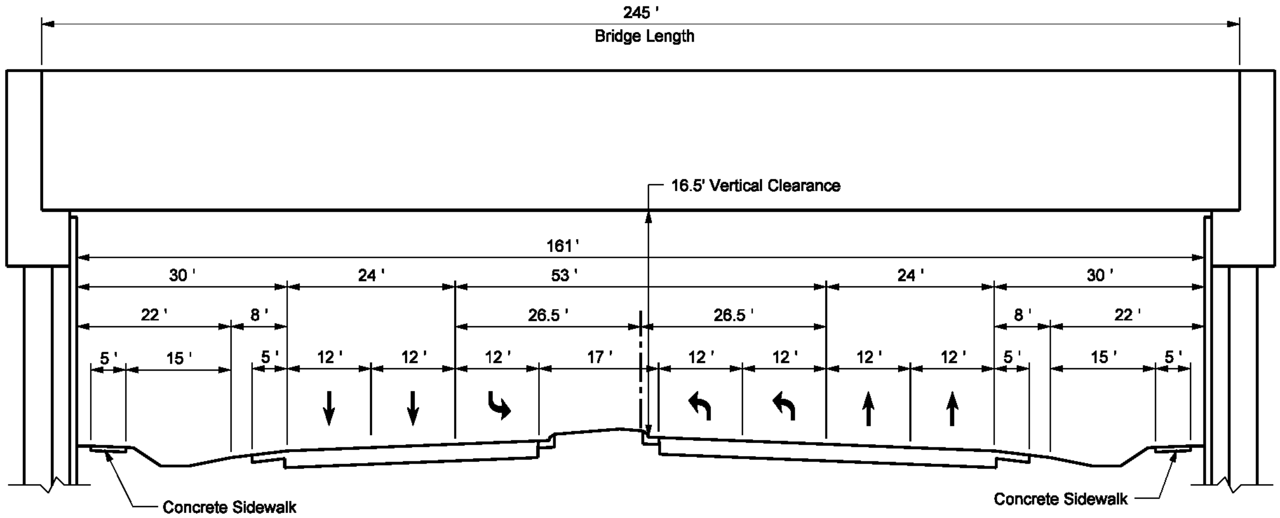


**FIGURE 7-4: BUILD ALTERNATIVE NO. 1 PROPOSED TYPICAL SECTION – SR 33 UNDER I-4 LOOKING NORTH**

The realigned Tomkow Road intersects SR 33 directly across from the easternmost active entrance/exit to the Auto Auction and a full median opening is proposed for this location. Although the Auto Auction has three connections to SR 33, the easternmost connection is gated and is not currently used by this business. The westernmost Auto Auction entrance/exit would only have right-in/right-out access. A westbound directional median opening is also proposed for the park-and-ride lot entrance to accommodate left-turn movements into this facility. This directional median opening eliminates the need to accommodate U-turn movements at the westbound I-4 ramp terminal intersection.

The peak hour volumes that were used to conduct the relocated Tomkow Road intersection analysis were derived by manually redistributing several of the design year peak hour movement volumes that were previously used to conduct the Tomkow Road intersection analysis for the No-Build (and TSM) Alternative. In addition, a 1.5% per year growth rate was applied to the existing peak hour Auto Auction turning movement volumes to derive the design year peak hour volumes for this land use. Several of these volumes were also manually redistributed to reflect the relocation of the Tomkow Road intersection and the right-in/right-out only access provided at the western Auto Auction driveway. Since a.m. peak hour turning movement counts were not conducted at the Auto Auction driveways, only a p.m. peak hour analysis was conducted for this intersection. As stated earlier in **Section 3.4** of this report, auctions do not start until 2:00 p.m. on Wednesdays and consequently, the volume of traffic entering and exiting this facility during the a.m. peak hour is significantly lower than the p.m. peak hour. Therefore, the design year a.m. peak hour traffic operations would be expected to be significantly better than the p.m. peak hour traffic operations at this intersection. The opening year and design year p.m. peak hour volumes that were used to conduct the analysis of the relocated Tomkow Road intersection are graphically illustrated in **Figure 7-6** and **Figure 7-7**, respectively. The design year interchange and intersection geometrics that were analyzed for Build Alternative No. 1 are graphically illustrated in **Figure 7-8**.

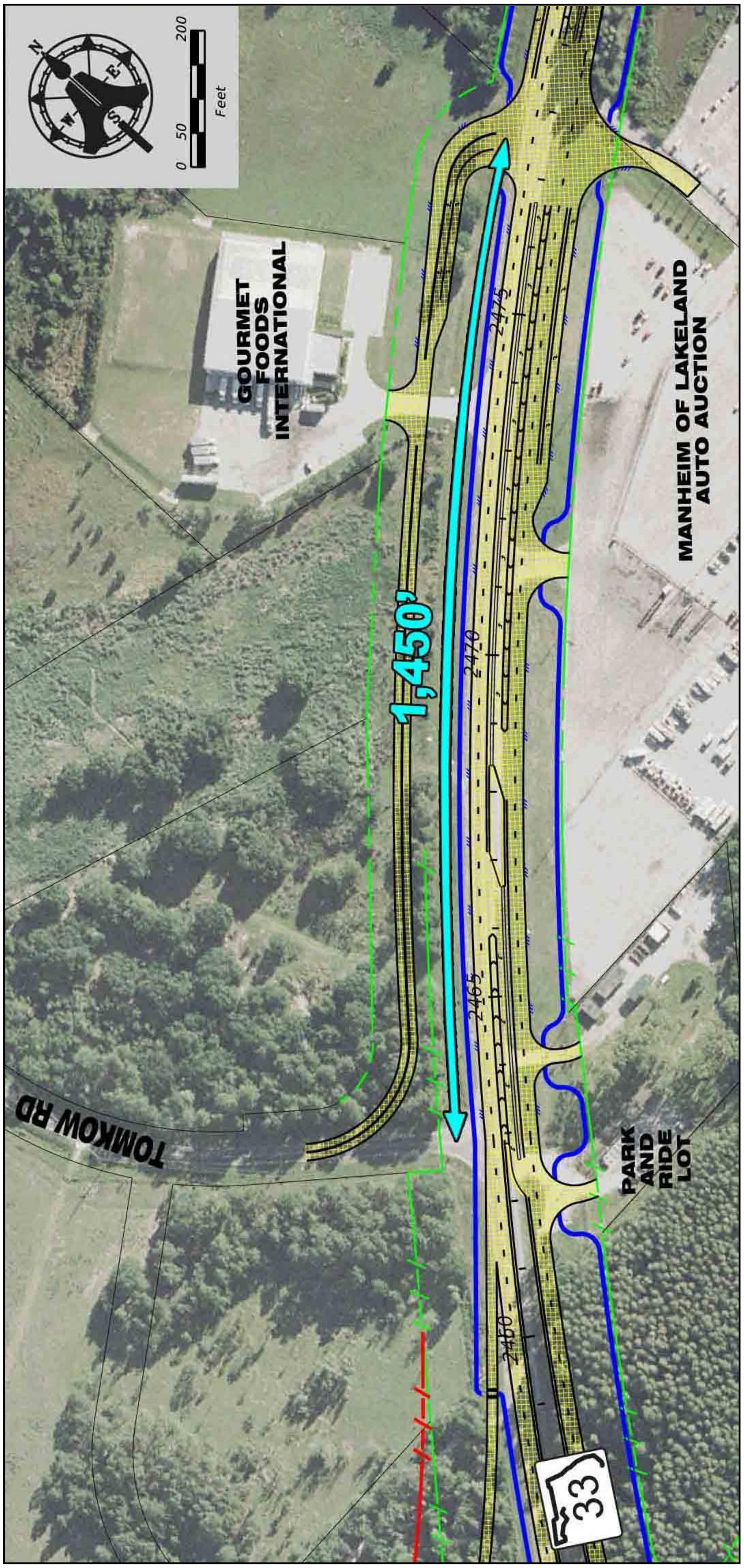


**FIGURE 7-5: TOMKOW ROAD REALIGNMENT**

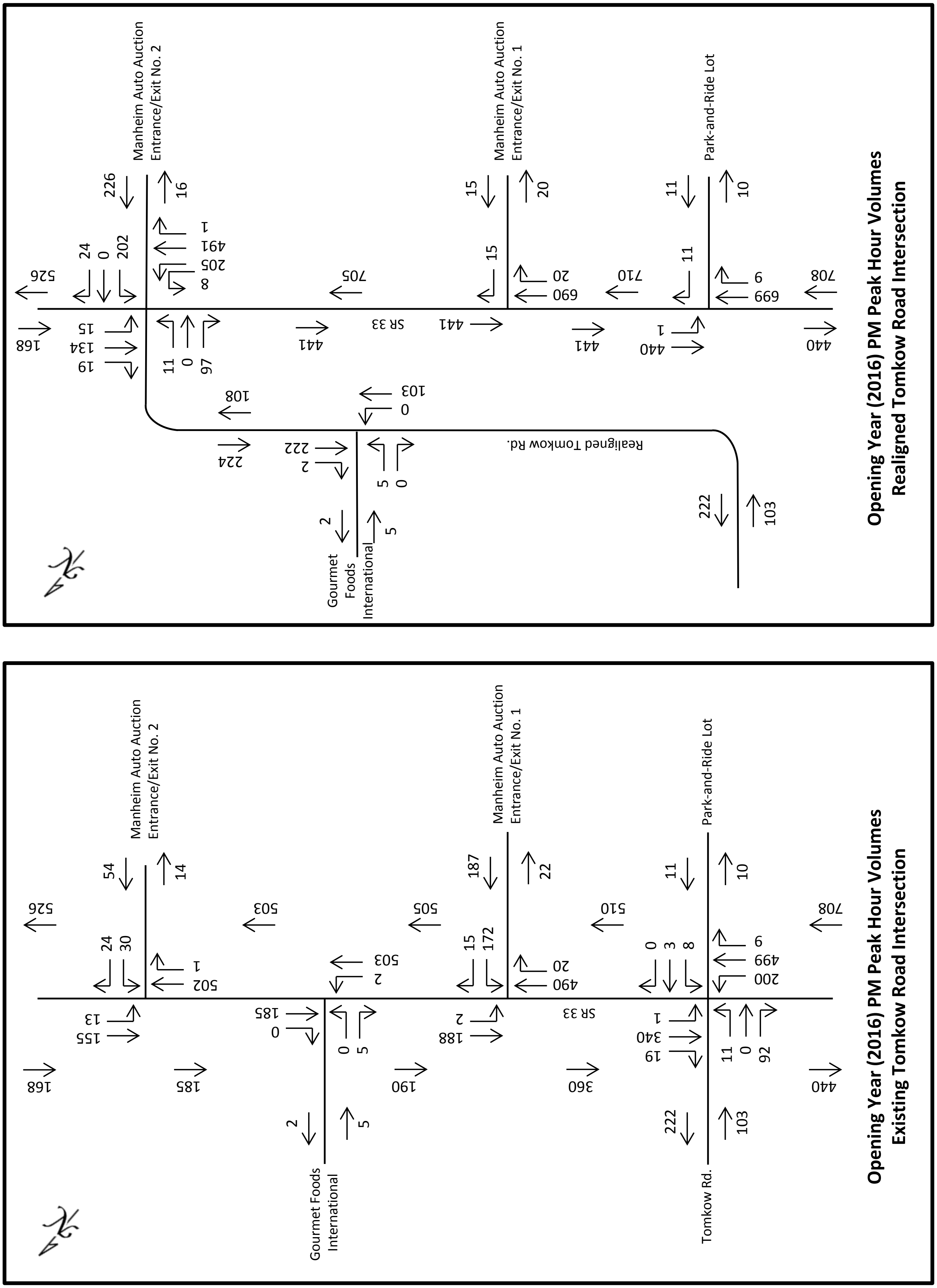


**FIGURE 7-6: REDISTRIBUTED OPENING YEAR (2016) PM PEAK HOUR VOLUMES**

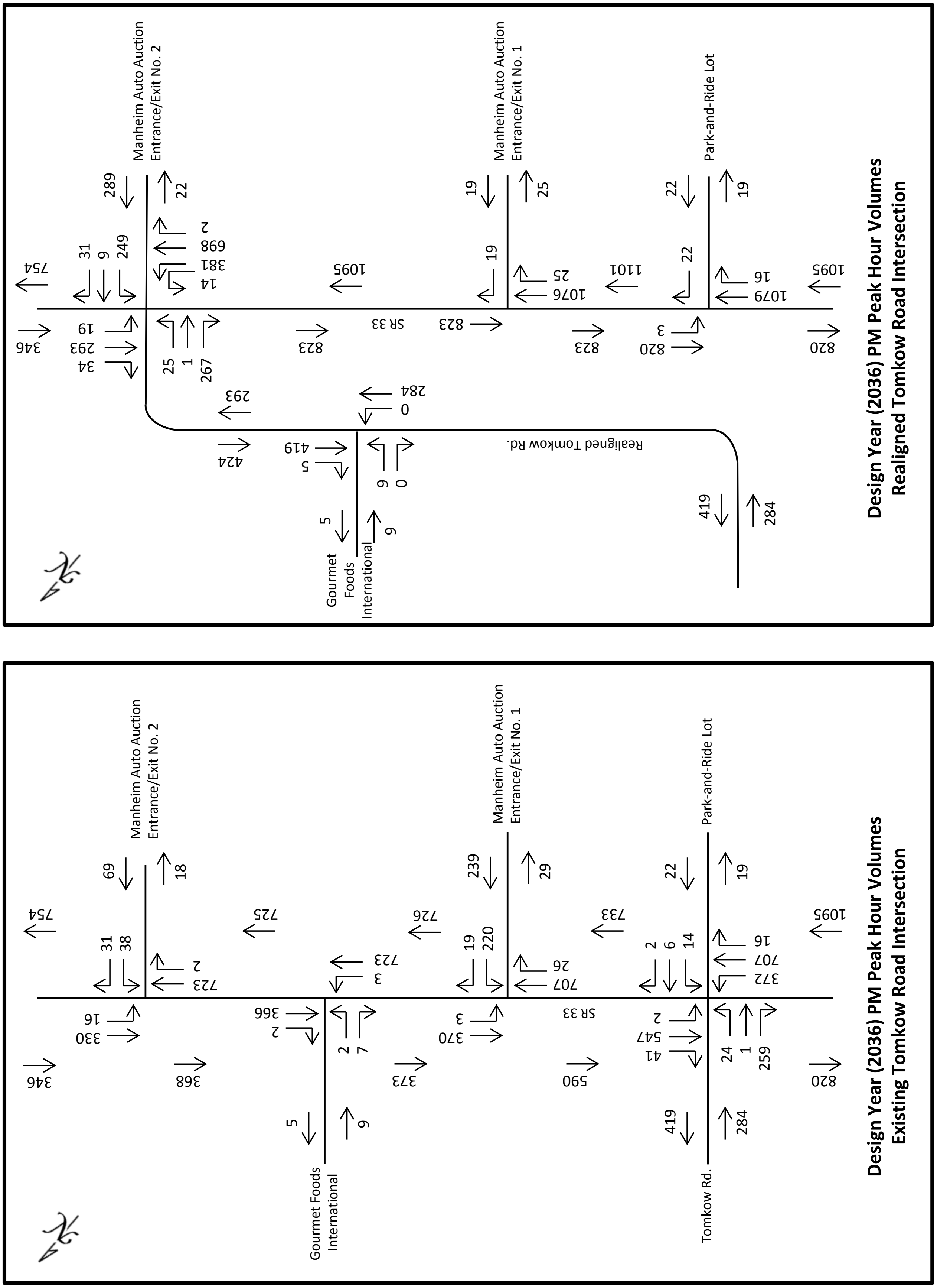


**FIGURE 7-7: REDISTRIBUTED DESIGN YEAR (2036) PM PEAK HOUR VOLUMES**

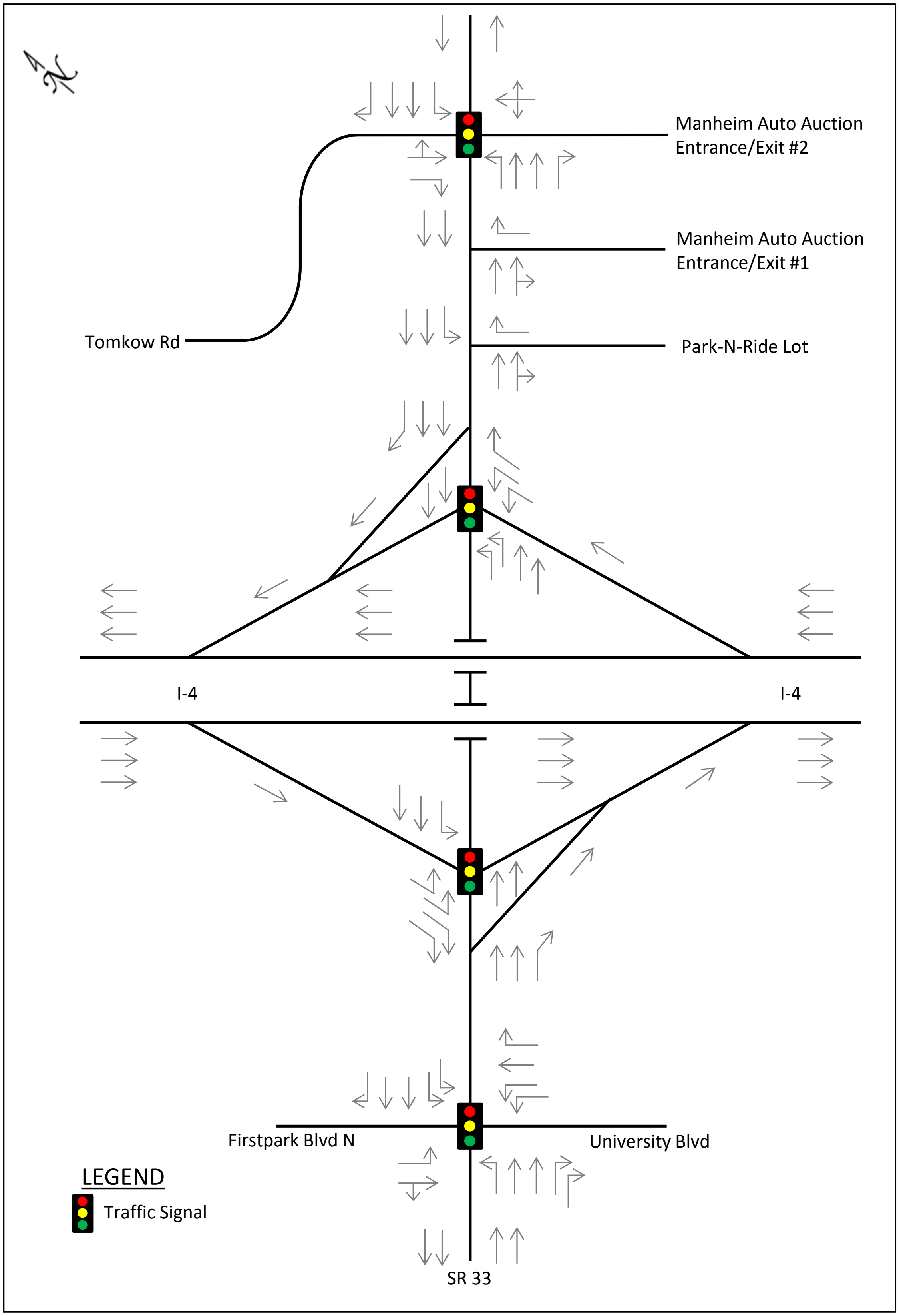


**FIGURE 7-8: DESIGN YEAR (2036) INTERCHANGE/INTERSECTION GEOMETRY – BUILD ALTERNATIVE NO. 1**

The results of the Build Alternative No. 1 opening year ramp merge/diverge area analyses are summarized in **Table 7-12**. All four of the merge/diverge areas are projected to operate at Level of Service C or better during both the a.m. and p.m. peak hours in the year 2016. **Table 7-13** summarizes the results of the design year ramp merge/diverge area analyses. All four of the merge/diverge areas are projected to operate at Level of Service D or better during both the a.m. and p.m. peak hours in the year 2036. The 2016 and 2036 ramp merge/diverge area analysis summary sheets for Build Alternative No. 1 are provided in **Appendix L**.

**TABLE 7-12: OPENING YEAR (2016) MERGE/DIVERGE AREA LEVELS OF SERVICE – BUILD ALTERNATIVE NO. 1**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 2,725 | 305 | 20.1 | C | 3,199 | 370 | 22.9 | C |
| EB I-4 Merge Area | 2,420 | 271 | 15.4 | B | 2,829 | 230 | 17.3 | B |
| WB I-4 Diverge Area | 2,546 | 210 | 19.7 | B | 3,216 | 311 | 23.7 | C |
| WB I-4 Merge Area | 2,336 | 430 | 16.5 | B | 2,905 | 392 | 19.2 | B |

**TABLE 7-13: DESIGN YEAR (2036) MERGE/DIVERGE AREA LEVELS OF SERVICE – BUILD ALTERNATIVE NO. 1**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 5,481 | 892 | 34.2 | D | 4,860 | 791 | 32.1 | D |
| EB I-4 Merge Area | 4,589 | 706 | 29.5 | D | 4,069 | 626 | 26.2 | C |
| WB I-4 Diverge Area | 4,695 | 626 | 31.1 | D | 5,295 | 706 | 33.9 | D |
| WB I-4 Merge Area | 4,069 | 791 | 27.7 | C | 4,589 | 892 | 31.2 | D |

Signalized intersection analyses were conducted for the I-4 ramp terminal intersections and the University Boulevard/Firstpark Boulevard N. intersection using the SYNCHRO software (Version 8). The realigned Tomkow Road intersection was analyzed as an unsignalized intersection (using the HCS) and as a signalized intersection (using the SYNCHRO software). An unsignalized intersection analysis was also conducted for the directional median opening at the existing park-and-ride lot. The results of the design year intersection analyses conducted for Build Alternative No. 1 are summarized in **Table 7-14**.

Both of the I-4 ramp terminal intersections are projected to operate at Level of Service B overall during the a.m. and p.m. peak hours. With one exception, all of the individual movements at these two intersections are projected to operate at Level of Service C or better during both peak hours. The westbound left-turn movement is projected to operate at Level of Service D during the p.m. peak hour. The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service D overall during both the a.m. and p.m. peak hours. There are a few individual movements that are projected to operate at Level of Service E or F during one or both peak hours; however, the v/c ratios associated with these movements are all projected to be less than 1.00.

## TABLE 7-14: DESIGN YEAR (2036) PEAK HOUR INTERSECTION OPERATIONS – BUILD ALTERNATIVE NO. 1

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.64 | 38.6 | D | 0.74 | 46.5 | D |
| | Eastbound | TH/RT | 0.70 | 77.4 | E | 0.80 | 84.6 | F |
| | Eastbound | Approach | N/A | 54.2 | D | N/A | 62.0 | E |
| | Westbound | LT | 0.93 | 58.2 | E | 0.92 | 63.6 | E |
| | Westbound | TH | 0.22 | 34.7 | C | 0.18 | 38.5 | D |
| | Westbound | RT | 0.68 | 11.9 | B | 0.84 | 25.6 | C |
| | Westbound | Approach | N/A | 40.1 | D | N/A | 46.2 | D |
| | Northbound | LT | 0.34 | 56.3 | E | 0.34 | 48.8 | D |
| | Northbound | TH | 0.74 | 62.3 | E | 0.81 | 59.1 | E |
| | Northbound | RT | 0.60 | 11.9 | B | 0.70 | 13.6 | B |
| | Northbound | Approach | N/A | 29.2 | C | N/A | 31.6 | C |
| | Southbound | LT | 0.92 | 68.6 | E | 0.90 | 67.8 | E |
| | Southbound | TH | 0.51 | 34.2 | C | 0.34 | 27.3 | C |
| | Southbound | RT | 0.23 | 5.0 | A | 0.13 | 1.7 | A |
| | Southbound | Approach | N/A | 47.4 | D | N/A | 46.8 | D |
| | Overall Intersection | | N/A | 40.3 | D | N/A | 42.6 | D |
| I-4 EB Ramps (Signalized) | Eastbound | LT | 0.50 | 25.8 | C | 0.70 | 30.7 | C |
| | Eastbound | RT | 0.55 | 4.4 | A | 0.39 | 4.5 | A |
| | Eastbound | Approach | N/A | 12.8 | B | N/A | 20.5 | C |
| | Northbound | TH | 0.58 | 20.9 | C | 0.60 | 15.0 | B |
| | Northbound | Approach | 0.58 | 20.9 | C | 0.60 | 15.0 | B |
| | Southbound | LT | 0.30 | 5.1 | A | 0.41 | 16.8 | B |
| | Southbound | TH | 0.77 | 16.3 | B | 0.82 | 15.6 | B |
| | Southbound | Approach | N/A | 14.7 | B | N/A | 15.7 | B |
| | Overall Intersection | | N/A | 15.6 | B | N/A | 16.8 | B |
| I-4 WB Ramps (Signalized) | Westbound | LT | 0.73 | 31.4 | C | 0.84 | 38.3 | D |
| | Westbound | RT | 0.27 | 6.6 | A | 0.27 | 3.1 | A |
| | Westbound | Approach | N/A | 27.1 | C | N/A | 32.1 | C |
| | Northbound | LT | 0.56 | 19.7 | B | 0.73 | 18.5 | B |
| | Northbound | TH | 0.62 | 13.2 | B | 0.59 | 9.3 | A |
| | Northbound | Approach | N/A | 15.1 | B | N/A | 12.4 | B |
| | Southbound | TH | 0.48 | 18.7 | B | 0.38 | 18.9 | B |
| | Southbound | Approach | 0.48 | 18.7 | B | 0.38 | 18.9 | B |
| | Overall Intersection | | N/A | 19.4 | B | N/A | 18.8 | B |
| Tomkow Road (Unsignalized) | Eastbound | LT | N/A | N/A | N/A | 0.35 | 9.7 | A |
| | Westbound | LT | N/A | N/A | N/A | 0.02 | 9.2 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 5.22 | 2,042.0 | F |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.35 | 75.1 | F |
| | Southbound | RT | N/A | N/A | N/A | 0.32 | 11.0 | B |
| Tomkow Road (Signalized) | Eastbound | LT | N/A | N/A | N/A | 0.84 | 29.2 | C |
| | Eastbound | TH | N/A | N/A | N/A | 0.44 | 10.2 | B |
| | Eastbound | RT | N/A | N/A | N/A | 0.00 | 0.0 | A |
| | Eastbound | Approach | N/A | N/A | N/A | N/A | 17.0 | B |
| | Westbound | LT | N/A | N/A | N/A | 0.06 | 7.5 | A |
| | Westbound | TH | N/A | N/A | N/A | 0.18 | 8.1 | A |
| | Westbound | RT | N/A | N/A | N/A | 0.05 | 2.8 | A |
| | Westbound | Approach | N/A | N/A | N/A | N/A | 7.5 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 0.66 | 28.7 | C |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.06 | 18.2 | B |
| | Southbound | RT | N/A | N/A | N/A | 0.41 | 4.9 | A |
| | Southbound | Approach | N/A | N/A | N/A | N/A | 6.1 | A |
| | Overall Intersection | | N/A | N/A | N/A | N/A | 15.5 | B |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

The results of the p.m. peak hour unsignalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that several movements are projected to operate at Level of Service F. The southbound left-turn and through movements are projected to have an average delay of approximately 75 seconds/vehicle while the northbound left-turn, through, and right-turn movements are projected to have an average delay of 2,042 seconds/vehicle. Although Level of Service F operations are projected for the southbound left-turn and through movements, the v/c ratio associated with these movements is low (i.e., 0.35). The results of the p.m. peak hour signalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that all movements are projected to operate at Level of Service C or better with the implementation of a traffic signal. The 2036 intersection analyses for Build Alternative No. 1 are provided in **Appendix L**.

The opening year interchange and intersection geometrics that were analyzed for Build Alternative No. 1 are graphically illustrated in **Figure 7-9**. Although the FDOT District One Adopted Five-Year Work Program includes funding for the final design of the widening of SR 33 from Old Combee Road to north of Tomkow Road (as well as the interchange improvements), there is currently no construction funding programmed. Consequently, it is likely that the interchange improvements would be constructed prior to the widening of the remaining portion of SR 33. The Build Alternative No. 1 interchange improvement concept plans provided in **Appendix K** incorporate the transition from four lanes to two lanes at the University Boulevard/Firstpark Boulevard N. intersection. In the southbound direction, the inside through lane is terminated as a left-turn only lane. In the northbound direction, the second through lane is added via a free-flow westbound right-turn lane.

The results of the opening year intersection analyses conducted for Build Alternative No. 1 are summarized in **Table 7-15**. Both of the I-4 ramp terminal intersections are projected to operate at Level of Service B or better overall during the a.m. and p.m. peak hours. In addition, all of the individual movements are projected to operate at Level of Service C or better during both peak hours. The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service C overall during both the a.m. and p.m. peak hours.

**Table 7-15** also indicates that a majority of the movements at the relocated Tomkow Road intersection are projected to operate at Level of Service C or better in the p.m. peak hour with two-way stop control. However, the northbound left-turn, through, and right-turn movements are projected to operate at Level of Service F with an average delay of approximately 422 seconds/vehicle. The v/c ratio for this single shared lane is projected to be 1.81. The results of this analysis indicate that vehicles exiting the Auto Auction could experience significant p.m. peak hour delays in the opening year if this intersection operates as a two-way stop controlled intersection. The results of the p.m. peak hour signalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that all movements are projected to operate at Level of Service C or better with the implementation of a traffic signal. The 2016 intersection analyses for Build Alternative No. 1 are provided in **Appendix L**.

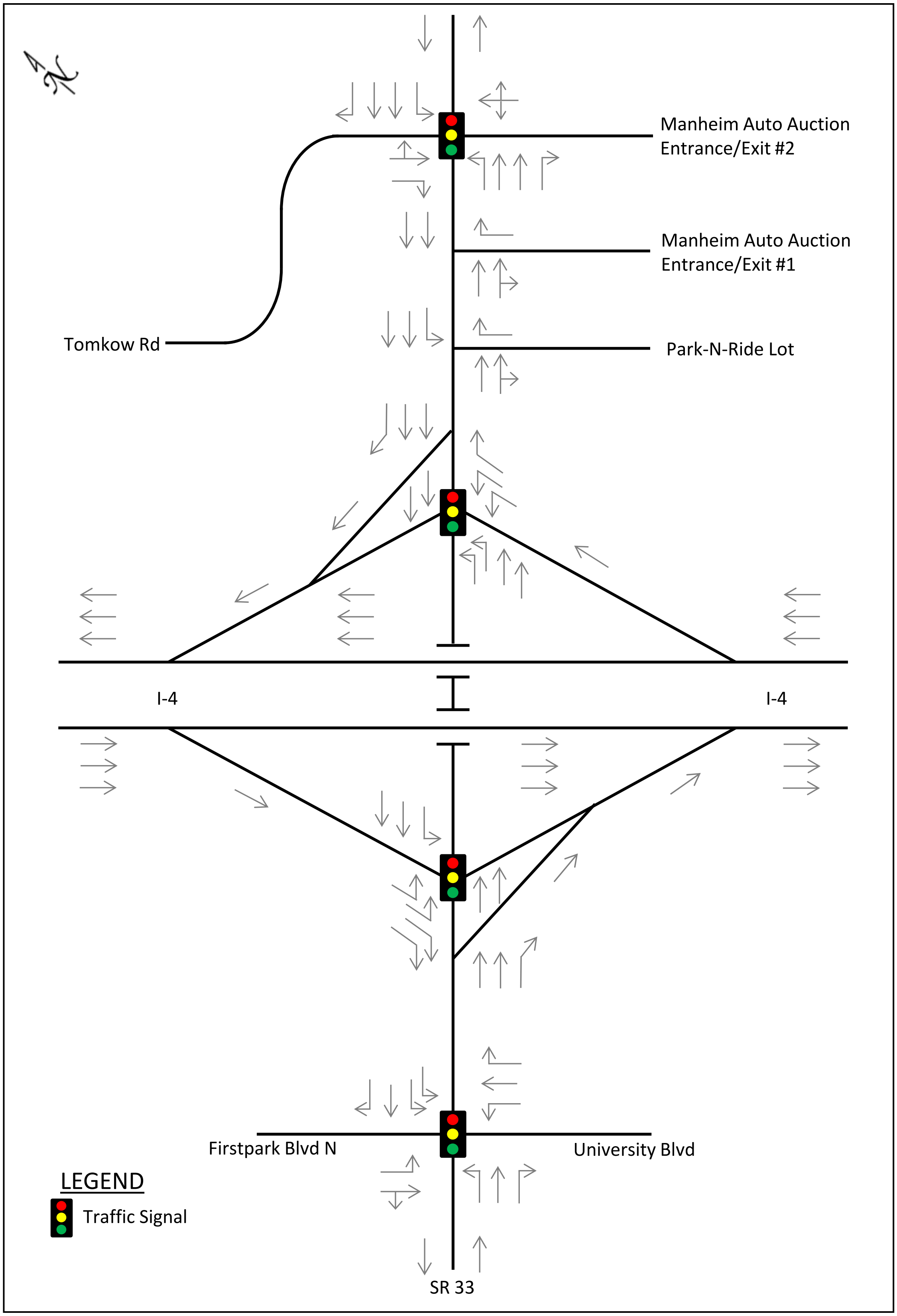


**FIGURE 7-9: OPENING YEAR (2016) INTERCHANGE/INTERSECTION GEOMETRY – BUILD ALTERNATIVE NO. 1**

## TABLE 7-15: OPENING YEAR (2016) PEAK HOUR INTERSECTION OPERATIONS – BUILD ALTERNATIVE NO. 1

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C [1] | Delay [2] | LOS [3] | V/C [1] | Delay [2] | LOS [3] |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.24 | 16.2 | B | 0.20 | 17.0 | B |
| | Eastbound | TH/RT | 0.06 | 13.6 | B | 0.11 | 14.2 | B |
| | Eastbound | Approach | N/A | 15.4 | B | N/A | 16.0 | B |
| | Westbound | LT | 0.51 | 25.9 | C | 0.47 | 34.0 | C |
| | Westbound | TH | 0.05 | 20.0 | B | 0.04 | 24.1 | C |
| | Westbound | RT | 0.07 | 0.1 | A | 0.09 | 0.1 | A |
| | Westbound | Approach | N/A | 17.3 | B | N/A | 18.2 | B |
| | Northbound | LT | 0.07 | 16.6 | B | 0.05 | 13.7 | B |
| | Northbound | TH | 0.67 | 27.8 | C | 0.83 | 31.9 | C |
| | Northbound | RT | 0.29 | 3.4 | A | 0.27 | 3.5 | A |
| | Northbound | Approach | N/A | 19.2 | B | N/A | 23.8 | C |
| | Southbound | LT | 0.52 | 35.3 | D | 0.51 | 38.0 | D |
| | Southbound | TH | 0.72 | 25.4 | C | 0.44 | 14.4 | B |
| | Southbound | RT | 0.10 | 0.3 | A | 0.06 | 0.2 | A |
| | Southbound | Approach | N/A | 25.2 | C | N/A | 19.6 | B |
| | Overall Intersection | | N/A | 21.0 | C | N/A | 20.9 | C |
| I-4 EB Ramps (Signalized) | Eastbound | LT | 0.39 | 27.3 | C | 0.54 | 27.8 | C |
| | Eastbound | RT | 0.31 | 3.7 | A | 0.17 | 0.7 | A |
| | Eastbound | Approach | N/A | 16.5 | B | N/A | 21.4 | C |
| | Northbound | TH | 0.20 | 12.7 | B | 0.27 | 8.4 | A |
| | Northbound | Approach | 0.20 | 12.7 | B | 0.27 | 8.4 | A |
| | Southbound | LT | 0.11 | 0.5 | A | 0.12 | 3.0 | A |
| | Southbound | TH | 0.31 | 5.1 | A | 0.31 | 6.8 | A |
| | Southbound | Approach | N/A | 4.4 | A | N/A | 6.4 | A |
| | Overall Intersection | | N/A | 9.6 | A | N/A | 11.5 | B |
| I-4 WB Ramps (Signalized) | Westbound | LT | 0.43 | 28.2 | C | 0.40 | 25.3 | C |
| | Westbound | RT | 0.13 | 0.7 | A | 0.26 | 1.7 | A |
| | Westbound | Approach | N/A | 22.4 | C | N/A | 17.6 | B |
| | Northbound | LT | 0.36 | 23.8 | C | 0.31 | 14.9 | B |
| | Northbound | TH | 0.25 | 5.3 | A | 0.34 | 5.7 | A |
| | Northbound | Approach | N/A | 9.9 | A | N/A | 7.7 | A |
| | Southbound | TH | 0.26 | 10.5 | B | 0.18 | 15.1 | B |
| | Southbound | Approach | 0.26 | 10.5 | B | 0.18 | 15.1 | B |
| | Overall Intersection | | N/A | 12.6 | B | N/A | 11.3 | B |
| Tomkow Road (Unsignalized) | Eastbound | LT | N/A | N/A | N/A | 0.16 | 8.0 | A |
| | Westbound | LT | N/A | N/A | N/A | 0.01 | 8.5 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 1.81 | 421.5 | F |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.05 | 20.7 | C |
| | Southbound | RT | N/A | N/A | N/A | 0.12 | 9.1 | A |
| Tomkow Road (Signalized) | Eastbound | LT | N/A | N/A | N/A | 0.61 | 22.0 | C |
| | Eastbound | TH | N/A | N/A | N/A | 0.53 | 16.2 | B |
| | Eastbound | RT | N/A | N/A | N/A | 0.00 | 0.0 | A |
| | Eastbound | Approach | N/A | N/A | N/A | N/A | 17.9 | B |
| | Westbound | LT | N/A | N/A | N/A | 0.07 | 12.0 | B |
| | Westbound | TH | N/A | N/A | N/A | 0.14 | 12.3 | B |
| | Westbound | RT | N/A | N/A | N/A | 0.04 | 3.2 | A |
| | Westbound | Approach | N/A | N/A | N/A | N/A | 11.3 | B |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 0.35 | 10.0 | A |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.02 | 9.3 | A |
| | Southbound | RT | N/A | N/A | N/A | 0.12 | 3.1 | A |
| | Southbound | Approach | N/A | N/A | N/A | N/A | 3.8 | A |
| | Overall Intersection | | N/A | N/A | N/A | N/A | 14.2 | B |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

*Although the results of the signalized intersection analyses indicate that the cross street vehicle delays at the relocated Tomkow Road intersection are projected to improve significantly with the implementation of traffic signal control, this does not imply that a traffic signal will be provided at this intersection in the opening year or the design year. The decision to install a traffic signal at this intersection will be made during the final design phase of the project and will be based on the results of a traffic signal warrant study conducted by the FDOT.*

Additional analyses were conducted to obtain an estimate of the “useful life” of the opening year geometry at the University Boulevard/Firstpark Boulevard N. intersection. The results of these additional analyses indicate that Level of Service D operations are projected for the overall intersection in the year 2026. Approximately 86.0% of the total intersection capacity is projected to be utilized in 2026 during the p.m. peak hour and all of the critical movements are projected to have v/c ratios greater than or equal to 0.97. The 2026 intersection analyses for the University Boulevard/Firstpark Boulevard N. intersection are also provided in **Appendix L.**

## 7.4 BUILD ALTERNATIVE NO. 2

The Build Alternative No. 2 diverging diamond interchange concept is depicted in **Figure 7-10** and on the plan sheets provided in **Appendix M**. Dual left-turn lanes are provided on both the eastbound and westbound I-4 off-ramps and dual right-turn lanes are provided on the eastbound I-4 off-ramp. Build Alternative No. 2 also provides a 300-foot deceleration lane on the I-4 mainline for the eastbound off-ramp. This alternative interchange improvement concept provides the same vertical profile and vertical clearance over SR 33 that is provided with Build Alternative No. 1. The proposed typical section of SR 33 under the I-4 bridges is illustrated in **Figure 7-11**. The proposed realignment of Tomkow Road that is included in Build Alternative No. 1 is also included in Build Alternative No. 2.

The results of the Build Alternative No. 2 opening year ramp merge/diverge area analyses are summarized in **Table 7-16**. All four of the merge/diverge areas are projected to operate at Level of Service C or better during both the a.m. and p.m. peak hours in the year 2016. **Table 7-17** summarizes the results of the design year ramp merge/diverge area analyses. All four of the merge/diverge areas are projected to operate at Level of Service D or better during both the a.m. and p.m. peak hours in the year 2036. The 2016 and 2036 ramp merge/diverge area analysis summary sheets for Build Alternative No. 2 are provided in **Appendix N**.

**Figure 7-12** illustrates the design year interchange and intersection geometrics that were analyzed for Build Alternative No. 2. It should be noted that the University Boulevard/Firstpark Boulevard N. and relocated Tomkow Road intersection geometrics are the same for Build Alternatives No. 1 and No. 2.

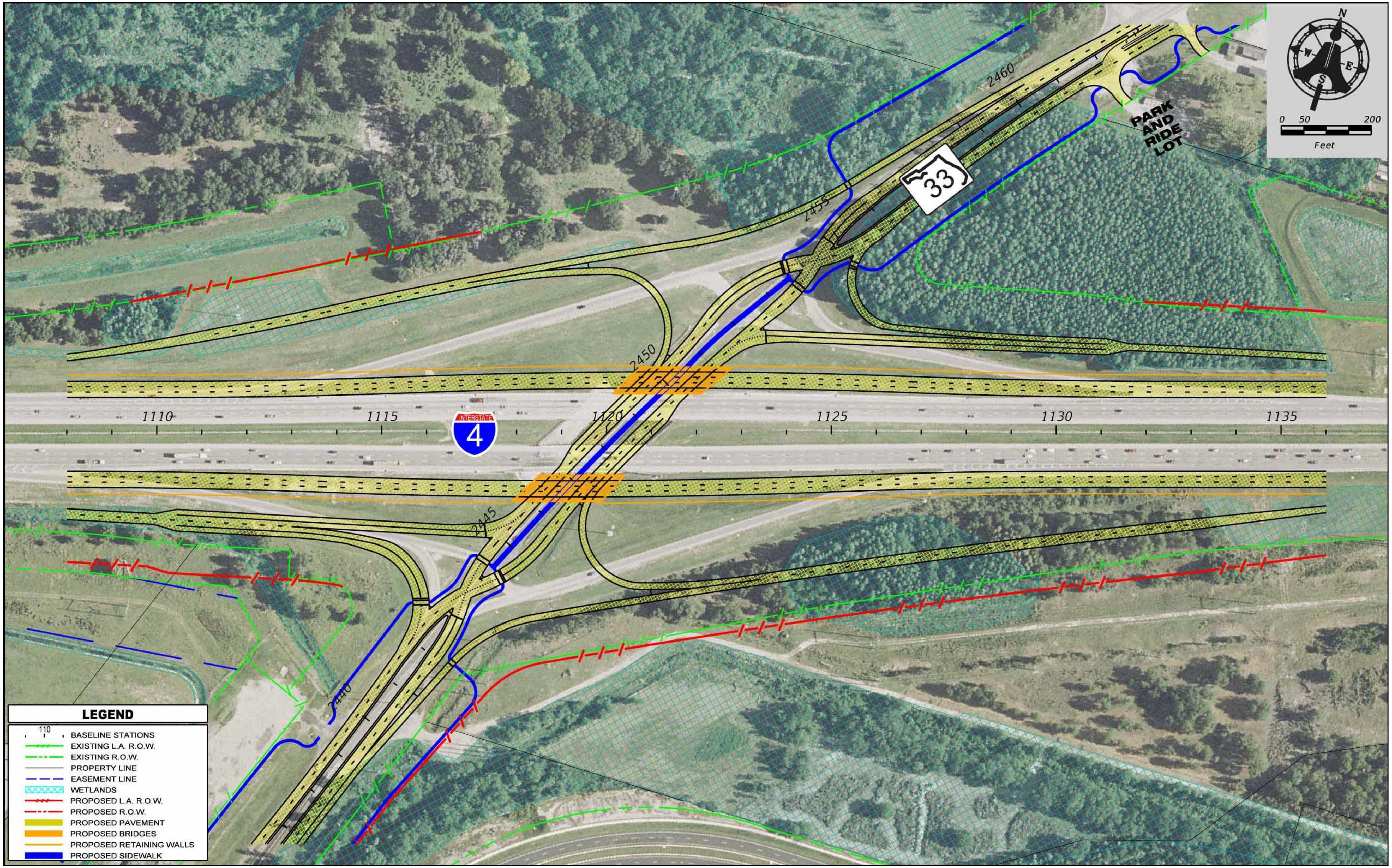


**FIGURE 7-10: BUILD ALTERNATIVE NO. 2 (DIVERGING DIAMOND INTERCHANGE)**

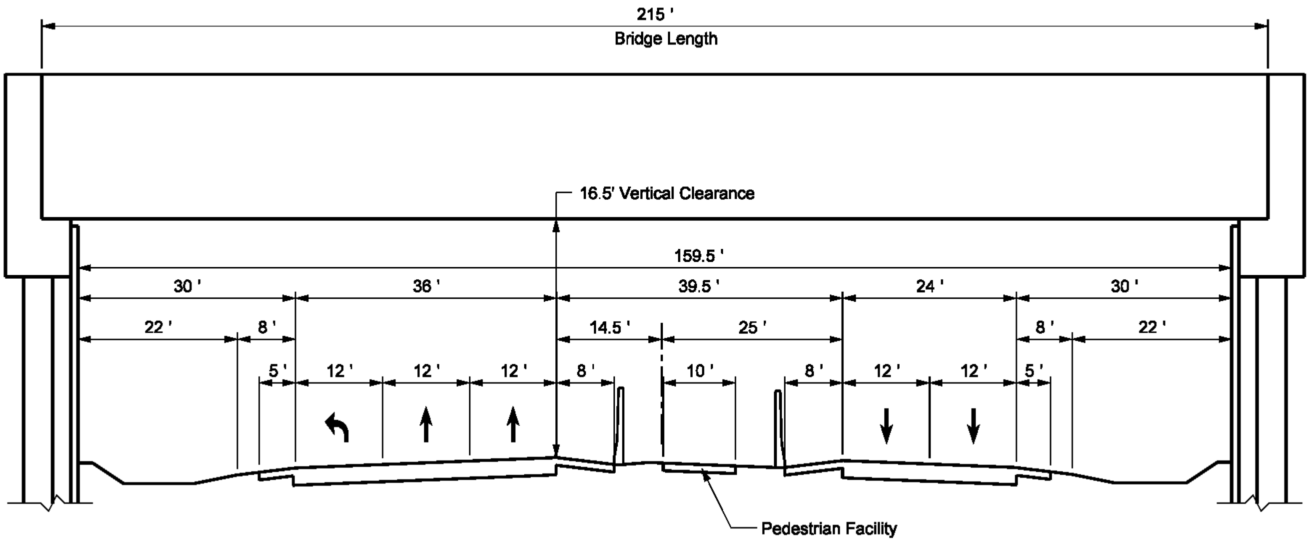


**FIGURE 7-11: BUILD ALTERNATIVE NO. 2 PROPOSED TYPICAL SECTION – SR 33 UNDER I-4 LOOKING NORTH**

**TABLE 7-16: OPENING YEAR (2016) MERGE/DIVERGE AREA LEVELS OF SERVICE – BUILD ALTERNATIVE NO. 2**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 2,725 | 305 | 20.1 | C | 3,199 | 370 | 22.9 | C |
| EB I-4 Merge Area | 2,420 | 271 | 15.4 | B | 2,829 | 230 | 17.3 | B |
| WB I-4 Diverge Area | 2,546 | 210 | 19.7 | B | 3,216 | 311 | 23.7 | C |
| WB I-4 Merge Area | 2,336 | 430 | 16.5 | B | 2,905 | 392 | 19.2 | B |

**TABLE 7-17: DESIGN YEAR (2036) MERGE/DIVERGE AREA LEVELS OF SERVICE – BUILD ALTERNATIVE NO. 2**

| Location | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|
| | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service | Mainline Volume | Ramp Volume | Density (pc/mi/ln) | Level of Service |
| EB I-4 Diverge Area | 5,481 | 892 | 34.2 | D | 4,860 | 791 | 32.1 | D |
| EB I-4 Merge Area | 4,589 | 706 | 29.5 | D | 4,069 | 626 | 26.2 | C |
| WB I-4 Diverge Area | 4,695 | 626 | 31.1 | D | 5,295 | 706 | 33.9 | D |
| WB I-4 Merge Area | 4,069 | 791 | 27.7 | C | 4,589 | 892 | 31.2 | D |

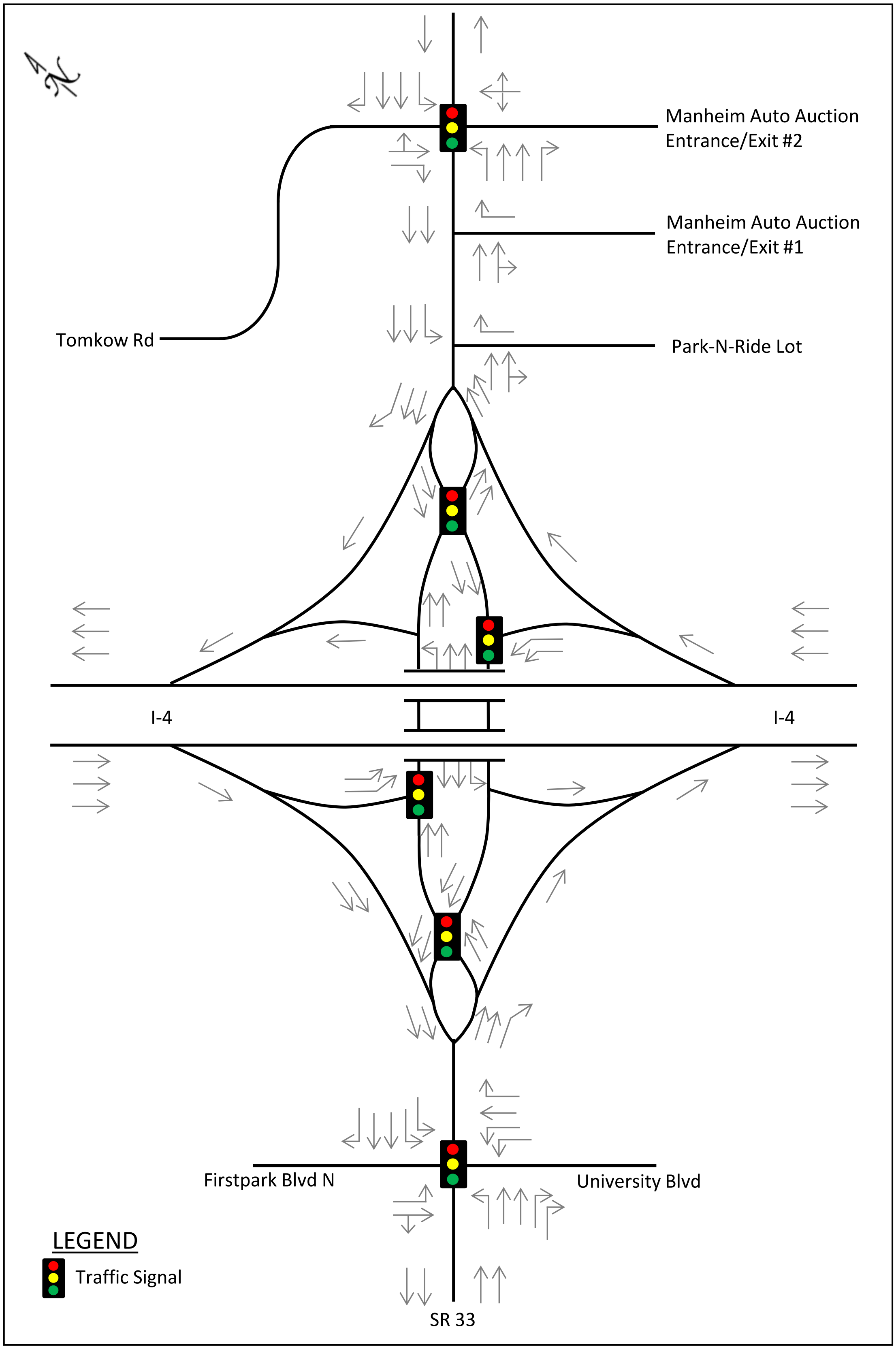


**FIGURE 7-12: DESIGN YEAR (2036) INTERCHANGE/INTERSECTION GEOMETRY – BUILD ALTERNATIVE NO. 2**

Signalized intersection analyses were conducted for the I-4 ramp terminal intersections and the University Boulevard/Firstpark Boulevard N. intersection using the SYNCHRO software. The realigned Tomkow Road intersection was analyzed as an unsignalized intersection (using the HCS) and as a signalized intersection (using the SYNCHRO software). An unsignalized intersection analysis was also conducted for the directional median opening at the existing park-and-ride lot. The results of the design year intersection analyses conducted for Build Alternative No. 2 are summarized in **Table 7-18**.

Both of the I-4 ramp terminal intersections are projected to operate at Level of Service B overall during the a.m. and p.m. peak hours. In addition, all of the individual movements at these two intersections are projected to operate at Level of Service C or better. Unlike Build Alternative No. 1, there would be no delay for the northbound and southbound SR 33 left-turn movements onto the I-4 ramps with Build Alternative No. 2. The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service D overall during both the a.m. and p.m. peak hours. There are a few individual movements that are projected to operate at Level of Service E or F during one or both peak hours; however, the v/c ratios associated with these movements are all projected to be less than 1.00.

The results of the p.m. peak hour unsignalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that several movements are projected to operate at Level of Service F. The southbound left-turn and through movements are projected to have an average delay of approximately 75 seconds/vehicle while the northbound left-turn, through, and right-turn movements are projected to have an average delay of 2,042 seconds/vehicle. Although Level of Service F operations are projected for the southbound left-turn and through movements, the v/c ratio associated with these movements is low (i.e., 0.35). The results of the p.m. peak hour signalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that all movements are projected to operate at Level of Service C or better with the implementation of a traffic signal. The 2016 and 2036 intersection analysis summary sheets for Build Alternative No. 2 are provided in **Appendix N**.

The opening year interchange and intersection geometrics that were analyzed for Build Alternative No. 2 are graphically illustrated in **Figure 7-13**. The Build Alternative No. 2 interchange improvement concept plans provided in **Appendix M** also incorporate the transition from four lanes to two lanes at the University Boulevard/Firstpark Boulevard N. intersection. In the southbound direction, the inside through lane is terminated as a left-turn only lane. In the northbound direction, the second through lane is added via a free-flow westbound right-turn lane. This is the same transitional geometry that was included with Build Alternative No. 1.

The results of the opening year intersection analyses conducted for Build Alternative No. 2 are summarized in **Table 7-19**. Both of the I-4 ramp terminal intersections are projected to operate at Level of Service B or better overall during the a.m. and p.m. peak hours. In addition, all of the individual movements are projected to operate at Level of Service C or better during both peak hours. The University Boulevard/Firstpark Boulevard N. intersection is projected to operate at Level of Service C overall during both the a.m. and p.m. peak hours.

**TABLE 7-18: DESIGN YEAR (2036) PEAK HOUR INTERSECTION OPERATIONS – BUILD ALTERNATIVE NO. 2**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.64 | 38.6 | D | 0.74 | 46.5 | D |
| | Eastbound | TH/RT | 0.70 | 77.4 | E | 0.80 | 84.6 | F |
| | Eastbound | Approach | N/A | 54.2 | D | N/A | 62.0 | E |
| | Westbound | LT | 0.93 | 58.2 | E | 0.92 | 63.6 | E |
| | Westbound | TH | 0.22 | 34.7 | C | 0.18 | 38.5 | D |
| | Westbound | RT | 0.68 | 11.9 | B | 0.84 | 25.6 | C |
| | Westbound | Approach | N/A | 40.1 | D | N/A | 46.2 | D |
| | Northbound | LT | 0.34 | 56.3 | E | 0.34 | 48.8 | D |
| | Northbound | TH | 0.74 | 62.3 | E | 0.81 | 59.1 | E |
| | Northbound | RT | 0.60 | 11.9 | B | 0.70 | 13.6 | B |
| | Northbound | Approach | N/A | 29.2 | C | N/A | 31.6 | C |
| | Southbound | LT | 0.92 | 68.6 | E | 0.90 | 67.8 | E |
| | Southbound | TH | 0.51 | 34.2 | C | 0.34 | 27.3 | C |
| | Southbound | RT | 0.23 | 5.0 | A | 0.13 | 1.7 | A |
| | Southbound | Approach | N/A | 47.4 | D | N/A | 46.8 | D |
| | Overall Intersection | | N/A | 40.3 | D | N/A | 42.6 | D |
| I-4 EB Ramps (Signalized) | Eastbound | LT | 0.23 | 10.9 | B | 0.39 | 16.0 | B |
| | Eastbound | RT | 0.54 | 15.7 | B | 0.26 | 9.4 | A |
| | Eastbound | Approach | N/A | 13.8 | B | N/A | 13.4 | B |
| | Northbound | TH | 0.56 | 19.5 | B | 0.67 | 17.6 | B |
| | Northbound | Approach | 0.56 | 19.5 | B | 0.67 | 17.6 | B |
| | Southbound | LT | N/A | N/A | N/A | N/A | N/A | N/A |
| | Southbound | TH | 0.62 | 11.1 | B | 0.71 | 18.7 | B |
| | Southbound | Approach | 0.62 | 11.1 | B | 0.71 | 18.7 | B |
| | Overall Intersection | | N/A | 14.3 | B | N/A | 16.7 | B |
| I-4 WB Ramps (Signalized) | Westbound | LT | 0.45 | 18.1 | B | 0.41 | 13.7 | B |
| | Westbound | RT | 0.15 | 5.1 | A | 0.22 | 15.1 | B |
| | Westbound | Approach | N/A | 15.8 | B | N/A | 13.9 | B |
| | Northbound | LT | N/A | N/A | N/A | N/A | N/A | N/A |
| | Northbound | TH | 0.60 | 28.5 | C | 0.66 | 19.3 | B |
| | Northbound | Approach | 0.60 | 28.5 | C | 0.66 | 19.3 | B |
| | Southbound | TH | 0.39 | 12.3 | B | 0.33 | 15.4 | B |
| | Southbound | Approach | 0.39 | 12.3 | B | 0.33 | 15.4 | B |
| | Overall Intersection | | N/A | 19.4 | B | N/A | 16.7 | B |
| Tomkow Road (Unsignalized) | Eastbound | LT | N/A | N/A | N/A | 0.35 | 9.7 | A |
| | Westbound | LT | N/A | N/A | N/A | 0.02 | 9.2 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 5.22 | 2042.0 | F |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.35 | 75.1 | F |
| | Southbound | RT | N/A | N/A | N/A | 0.32 | 11.0 | B |
| Tomkow Road (Signalized) | Eastbound | LT | N/A | N/A | N/A | 0.84 | 29.2 | C |
| | Eastbound | TH | N/A | N/A | N/A | 0.44 | 10.2 | B |
| | Eastbound | RT | N/A | N/A | N/A | 0.00 | 0.0 | A |
| | Eastbound | Approach | N/A | N/A | N/A | N/A | 17.0 | B |
| | Westbound | LT | N/A | N/A | N/A | 0.06 | 7.5 | A |
| | Westbound | TH | N/A | N/A | N/A | 0.18 | 8.1 | A |
| | Westbound | RT | N/A | N/A | N/A | 0.05 | 2.8 | A |
| | Westbound | Approach | N/A | N/A | N/A | N/A | 7.5 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 0.66 | 28.7 | C |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.06 | 18.2 | B |
| | Southbound | RT | N/A | N/A | N/A | 0.41 | 4.9 | A |
| | Southbound | Approach | N/A | N/A | N/A | N/A | 6.1 | A |
| | Overall Intersection | | N/A | N/A | N/A | N/A | 15.5 | B |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

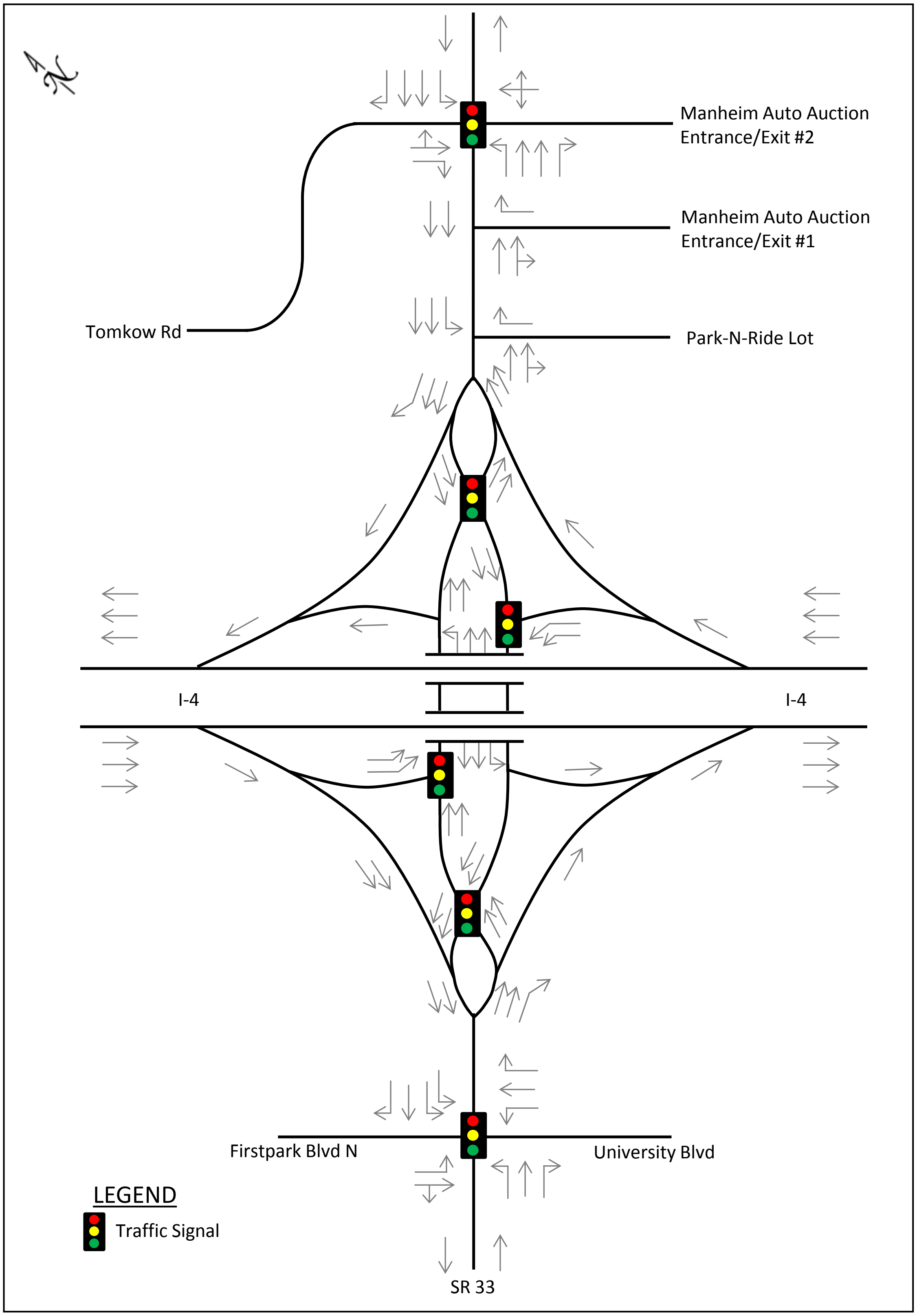


**FIGURE 7-13: OPENING YEAR (2016) INTERCHANGE/INTERSECTION GEOMETRY – BUILD ALTERNATIVE NO. 2**

**TABLE 7-19: OPENING YEAR (2016) PEAK HOUR INTERSECTION OPERATIONS – BUILD ALTERNATIVE NO. 2**

| Intersection | Approach | Movement | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | V/C (1) | Delay (2) | LOS (3) | V/C (1) | Delay (2) | LOS (3) |
| University Boulevard/Firstpark Boulevard N. (Signalized) | Eastbound | LT | 0.24 | 16.2 | B | 0.20 | 17.0 | B |
| | Eastbound | TH/RT | 0.06 | 13.6 | B | 0.11 | 14.2 | B |
| | Eastbound | Approach | N/A | 15.4 | B | N/A | 16.0 | B |
| | Westbound | LT | 0.51 | 25.9 | C | 0.47 | 34.0 | C |
| | Westbound | TH | 0.05 | 20.0 | B | 0.04 | 24.1 | C |
| | Westbound | RT | 0.07 | 0.1 | A | 0.09 | 0.1 | A |
| | Westbound | Approach | N/A | 17.3 | B | N/A | 18.2 | B |
| | Northbound | LT | 0.07 | 16.6 | B | 0.05 | 13.7 | B |
| | Northbound | TH | 0.67 | 27.8 | C | 0.83 | 31.9 | C |
| | Northbound | RT | 0.29 | 3.4 | A | 0.27 | 3.5 | A |
| | Northbound | Approach | N/A | 19.2 | B | N/A | 23.8 | C |
| | Southbound | LT | 0.52 | 35.3 | D | 0.51 | 38.0 | D |
| | Southbound | TH | 0.72 | 25.4 | C | 0.44 | 14.4 | B |
| | Southbound | RT | 0.10 | 0.3 | A | 0.06 | 0.2 | A |
| | Southbound | Approach | N/A | 25.2 | C | N/A | 19.6 | B |
| | Overall Intersection | | N/A | 21.0 | C | N/A | 20.9 | C |
| I-4 EB Ramps (Signalized) | Eastbound | LT | 0.10 | 7.5 | A | 0.25 | 14.3 | B |
| | Eastbound | RT | 0.17 | 0.4 | A | 0.07 | 0.1 | A |
| | Eastbound | Approach | N/A | 4.2 | A | N/A | 11.0 | B |
| | Northbound | TH | 0.37 | 19.8 | B | 0.35 | 12.1 | B |
| | Northbound | Approach | 0.37 | 19.8 | B | 0.35 | 12.1 | B |
| | Southbound | LT | N/A | N/A | N/A | N/A | N/A | N/A |
| | Southbound | TH | 0.28 | 6.6 | A | 0.32 | 15.8 | B |
| | Southbound | Approach | 0.28 | 6.6 | A | 0.32 | 15.8 | B |
| | Overall Intersection | | N/A | 9.4 | A | N/A | 12.9 | B |
| I-4 WB Ramps (Signalized) | Westbound | LT | 0.24 | 18.1 | B | 0.15 | 10.5 | B |
| | Westbound | RT | 0.06 | 8.2 | A | 0.19 | 14.5 | B |
| | Westbound | Approach | N/A | 16.0 | B | N/A | 11.8 | B |
| | Northbound | LT | N/A | N/A | N/A | N/A | N/A | N/A |
| | Northbound | TH | 0.45 | 28.7 | C | 0.43 | 11.5 | B |
| | Northbound | Approach | 0.45 | 28.7 | C | 0.43 | 11.5 | B |
| | Southbound | TH | 0.23 | 8.0 | A | 0.18 | 13.8 | B |
| | Southbound | Approach | 0.23 | 8.0 | A | 0.18 | 13.8 | B |
| | Overall Intersection | | N/A | 17.2 | B | N/A | 12.0 | B |
| Tomkow Road (Unsignalized) | Eastbound | LT | N/A | N/A | N/A | 0.16 | 8.0 | A |
| | Westbound | LT | N/A | N/A | N/A | 0.01 | 8.5 | A |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 1.81 | 421.5 | F |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.05 | 20.7 | C |
| | Southbound | RT | N/A | N/A | N/A | 0.12 | 9.1 | A |
| Tomkow Road (Signalized) | Eastbound | LT | N/A | N/A | N/A | 0.61 | 21.9 | C |
| | Eastbound | TH | N/A | N/A | N/A | 0.53 | 16.2 | B |
| | Eastbound | RT | N/A | N/A | N/A | 0.00 | 0.0 | A |
| | Eastbound | Approach | N/A | N/A | N/A | N/A | 17.9 | B |
| | Westbound | LT | N/A | N/A | N/A | 0.07 | 12.0 | B |
| | Westbound | TH | N/A | N/A | N/A | 0.14 | 12.3 | B |
| | Westbound | RT | N/A | N/A | N/A | 0.04 | 3.2 | A |
| | Westbound | Approach | N/A | N/A | N/A | N/A | 11.3 | B |
| | Northbound | LT/TH/RT | N/A | N/A | N/A | 0.35 | 10.0 | A |
| | Southbound | LT/TH | N/A | N/A | N/A | 0.02 | 9.2 | A |
| | Southbound | RT | N/A | N/A | N/A | 0.12 | 3.1 | A |
| | Southbound | Approach | N/A | N/A | N/A | N/A | 3.7 | A |
| | Overall Intersection | | N/A | N/A | N/A | N/A | 14.2 | B |

(1) Volume-to-Capacity Ratio

(2) Average Delay (seconds/vehicle)

(3) Level of Service

**Table 7-19** also indicates that a majority of the movements at the relocated Tomkow Road intersection are projected to operate at Level of Service C or better in the p.m. peak hour with two-way stop control. However, the northbound left-turn, through, and right-turn movements are projected to operate at Level of Service F with an average delay of approximately 422 seconds/vehicle. The v/c ratio for this single shared lane is projected to be 1.81. The results of this analysis indicate that vehicles exiting the Auto Auction could experience significant p.m. peak hour delays in the opening year if this intersection operates as a two-way stop controlled intersection. The results of the p.m. peak hour signalized intersection analysis conducted for the realigned Tomkow Road intersection indicate that all movements are projected to operate at Level of Service C or better with the implementation of a traffic signal. The 2016 intersection analyses for Build Alternative No. 2 are provided in **Appendix N**.

*Although the results of the signalized intersection analyses indicate that the cross street vehicle delays at the relocated Tomkow Road intersection are projected to improve significantly with the implementation of traffic signal control, this does not imply that a traffic signal will be provided at this intersection in the opening year or the design year. The decision to install a traffic signal at this intersection will be made during the final design phase of the project and will be based on the results of a traffic signal warrant study conducted by the FDOT.*

# 8.0 COMPARATIVE EVALUATION OF ALTERNATIVES

The results of the traffic operations analyses conducted for the No-Build Alternative and the TSM Alternative indicate that Level of Service F operations are projected to occur at both of the I-4 ramp terminal intersections. Consequently, neither of these two alternatives satisfies the purpose and need of the project. **Table 8-1** provides a comparison of the design year peak hour traffic operations projected to occur at the I-4 ramp terminal intersections with Build Alternatives No. 1 and No. 2. The performance measures included in this table are the capacity utilization (expressed as a percentage of the total capacity), maximum v/c ratio, average delay, and level of service.

**Table 8-1** indicates that similar operations are projected to occur for both of the Build Alternatives. Build Alternative No. 1 provides more “reserve” (i.e., unused) capacity at the eastbound I-4 ramp terminal intersection while Build Alternative No. 2 provides more reserve capacity at the westbound I-4 ramp terminal. The average a.m. peak hour delays at the westbound I-4 ramp terminal and the average p.m. peak hour delays at the eastbound I-4 ramp terminal are the same for both Build Alternatives. Build Alternative No. 2 is projected to have slightly lower average delays at the eastbound I-4 ramp terminal in the a.m. peak hour (14.3 seconds/vehicle vs. 15.6 seconds/vehicle) and at the westbound I-4 ramp terminal in the p.m. peak hour (16.7 seconds/vehicle vs. 18.8 seconds/vehicle). **Table 8-1** also provides a comparison of the overall average delays for the entire interchange. In the a.m. peak hour, the overall average interchange delay is approximately 16.6 seconds/vehicle for Build Alternative No. 2 and 17.3 seconds/vehicle for Build Alternative No. 1. In the p.m. peak hour, the overall average interchange delay is approximately 16.7 seconds/vehicle for Build Alternative No. 2 and 17.8 seconds/vehicle for Build Alternative No. 1.

**Table 8-2** provides a comparison of the design year peak hour 95$^{th}$-percentile off-ramp queue length estimates obtained from the SYNCHRO analyses. In general, this table indicates that the left-turn vehicle queues are projected to be longer with Build Alternative No. 1 while the right-turn vehicle queues are projected to be longer with Build Alternative No. 2. However, it should be noted that the differences between the 95$^{th}$-percentile queue length estimates are not significant. None of the off-ramp movement queues are projected to exceed 210 feet in length during either peak hour. Consequently, both Build Alternatives will be able to safely accommodate the design year peak hour vehicle queues on the off-ramps and avoid any potential queuing conditions on the I-4 mainline.

Preliminary cost estimates were developed for both of the Build Alternatives and are summarized in **Table 8-3**. The costs include final design, right-of-way, wetland mitigation, construction, and construction engineering inspection (assumed to be 15.0% of the construction cost). **Table 8-3** indicates that Build Alternative No. 1 is estimated to cost approximately $51,655,000 while Build Alternative No. 2 is estimated to cost approximately $51,010,000.

## TABLE 8-1: DESIGN YEAR (2036) PEAK HOUR INTERSECTION OPERATIONS COMPARISON

| Alternative | Intersection | AM Peak Hour | | | | PM Peak Hour | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Capacity Utilization | Maximum V/C [1] | Avg. Delay (sec/veh) | LOS [2] | Capacity Utilization | Maximum V/C [1] | Avg. Delay (sec/veh) | LOS [2] |
| Build Alternative No. 1 | EB I-4 On-/Off-Ramps | 53.9% | 0.77 (SB TH) | 15.6 | B | 57.9% | 0.82 (SB TH) | 16.8 | B |
| | WB I-4 On-/Off-Ramps | 52.1% | 0.73 (WB LT) | 19.4 | B | 54.4% | 0.84 (WB LT) | 18.8 | B |
| Build Alternative No. 2 | EB I-4 On-/Off-Ramps | 67.5% | 0.62 (SB TH) | 14.3 | B | 78.5% | 0.71 (SB TH) | 16.7 | B |
| | WB I-4 On-/Off-Ramps | 45.9% | 0.60 (NB TH) | 19.4 | B | 48.1% | 0.66 (NB TH) | 16.7 | B |

| Alternative | Intersection | Peak Hour | EB I-4 On-/Off-Ramps | | WB I-4 On-/Off-Ramps | | Total Delay (sec) | Avg. Delay (sec/veh) | LOS [2] |
|---|---|---|---|---|---|---|---|---|---|
| | | | Avg. Delay | Volume | Avg. Delay | Volume | | | |
| Build Alternative No. 1 | Both Intersections | AM Peak | 15.6 | 3,224 | 19.4 | 2,735 | 103,353.40 | 17.34 | B |
| | Both Intersections | PM Peak | 16.8 | 3,308 | 18.8 | 2,991 | 111,805.20 | 17.75 | B |
| Build Alternative No. 2 | Both Intersections | AM Peak | 14.3 | 3,224 | 19.4 | 2,735 | 99,162.20 | 16.64 | B |
| | Both Intersections | PM Peak | 16.7 | 3,308 | 16.7 | 2,991 | 105,193.30 | 16.70 | B |

(1) Volume-to-Capacity Ratio
(2) Level of Service

## TABLE 8-2: DESIGN YEAR (2036) PEAK HOUR QUEUE LENGTH COMPARISON

| Peak Hour | I-4 Off-Ramp Movement | 95% Queue Length (in feet) | |
|---|---|---|---|
| | | Build Alternative No. 1 | Build Alternative No. 2 |
| AM Peak Hour | EB LT | 108 | 68 |
| | EB RT | 39 | 132 |
| | WB LT | 160 | 129 |
| | WB RT | 36 | 33 |
| PM Peak Hour | EB LT | 152 | 113 |
| | EB RT | 31 | 60 |
| | WB LT | 208 | 124 |
| | WB RT | 19 | 72 |

**TABLE 8-3: PRELIMINARY COST ESTIMATES**

| Project Component | Build Alternative No. 1 Diamond Interchange | Build Alternative No. 2 Diverging Diamond Interchange |
|---|---|---|
| Design | $4,530,000 | $4,530,000 |
| Right-of-Way Acquisition | $2,179,000 | $2,167,000 |
| Wetland Mitigation | $2,206,000 | $2,145,000 |
| Construction Cost | $37,165,000 | $36,668,000 |
| Construction Engineering & Inspection | $5,575,000 | $5,500,000 |
| Preliminary Estimate of Total Project Cost | $51,655,000 | $51,010,000 |

In addition to the projected peak hour traffic operations and cost estimates associated with the two Build Alternatives, there are several other factors that should be considered. These other considerations include the following:

- SR 33 design speeds within the interchange area
- SR 33 through traffic flow during off-peak hours
- Driver expectancy

Build Alternative No.1 provides for a 55 mph design speed within the interchange area, while Build Alternative No. 2 provides for a 30 mph design speed within the interchange area. The lower design speed associated with Build Alternative No. 2 is due to the horizontal curves that are located in the areas where the northbound and southbound SR 33 lanes cross each other. As previously stated in **Section 2.1** of this report, the existing posted speed limit on SR 33 (including the interchange area) is 60 mph. Although a five (or 10) mph reduction in speed approaching the interchange would likely be viewed as reasonable from the driver's perspective, a 30 mph reduction in speed would likely be viewed by drivers as unreasonable and could increase the potential for speeding to occur as vehicles approach the interchange area. This increased speed could increase the potential for crashes to occur.

During the off-peak hours for the eastbound and westbound I-4 off-ramps, the traffic signals at the ramp terminal intersections with Build Alternative No. 1 can minimize the amount of “green time” that is provided for the off-ramp movements which maximizes the amount of green time that is provided for the SR 33 through movements. This results in fewer stops and longer durations of “free-flow” conditions for both through movements. Although the traffic signals that control the eastbound and westbound I-4 off-ramp movements with Build Alternative No. 2 can be designed to reduce the amount of green time that is provided for these movements during off-peak hours, there will be significantly more stops and more delay for the SR 33 through movements during off-peak hours. This is due to the need to alternate green time between the northbound and southbound through movements at the “cross-over” intersections.

The vehicle movements and traffic signal operations associated with conventional diamond interchanges are well understood by drivers in Polk County since this type of interchange has been in existence for a long time. In addition, since the interchange configuration provided with Build Alternative No. 1 is the same as the existing configuration (with additional laneage being provided on SR 33 and at the ramp terminal intersections), the driver expectancy associated with traveling through this interchange would be extremely high. In contrast, the level of driver expectancy associated with Build Alternative No. 2 would be significantly lower than Build Alternative No. 1 since this type of interchange configuration is not currently in operation anywhere within Polk County. It is possible that the cross-over through movements and traffic signal operations associated with a diverging diamond interchange could cause some wrong way movements to occur due to driver confusion (especially with non-commuter traffic and during the first few months after implementation).

# 9.0 CONCLUSIONS/RECOMMENDATIONS

Based on the magnitude of the future year traffic volumes that are projected to occur at the I-4/SR 33 interchange; there exists a need to widen SR 33 from two lanes to four lanes and signalize the ramp terminal intersections. Although the overall average peak hour vehicle delays for the interchange are projected to be lower with Build Alternative No. 2 than with Build Alternative No. 1, the magnitude of the differences is small. In the design year, the differences between the overall average peak hour vehicle delays are less than or equal to 1.1 seconds/vehicle. Both Build Alternatives are projected to provide Level of Service B operations at the ramp terminal intersections. The preliminary cost estimate for Build Alternative No. 2 is also lower than the cost estimate for Build Alternative No. 1; however, the magnitude of the cost difference is relatively small (i.e., approximately $645,000 or 1.3%).

Build Alternative No. 2 has a significantly lower design speed than Build Alternative No. 1 within the interchange area (30 mph vs. 55 mph). The level of driver expectancy associated with Build Alternative No. 2 would be significantly lower than Build Alternative No. 1 since this interchange configuration is not currently in operation anywhere throughout Polk County. Although Build Alternative No. 2 is projected to result in lower vehicle delay during the peak hours, it is also likely that this alternative will result in higher vehicle delay during periods of low ramp volumes due to the need to alternate green time between the northbound and southbound through movements at the cross-over intersections. Although the disadvantages associated with Build Alternative No. 2 are somewhat qualitative in nature, the differences in the vehicle delays and costs estimated for the two Build Alternatives are not significant enough to justify the construction of a diverging diamond interchange at this location. Consequently, the conventional diamond interchange improvement concept (Build Alternative No. 1) is recommended for approval by FDOT.

# 10.0 CONCEPTUAL FUNDING PLAN/CONSTRUCTION SCHEDULE

FDOT District One is currently funding the SR 33 PD&E study and this study is scheduled for completion in May 2014. District One has also recently programmed the final design phase of the I-4/SR 33 interchange into FDOT's Five-Year Work Program for FY 2013/2014.

**APPENDICES ARE LOCATED ON THE CD THAT IS ATTACHED TO THE BACK COVER**

# APPENDIX A

METHODOLOGY LETTER OF UNDERSTANDING

## *APPENDIX B*

BI-DIRECTIONAL VOLUME AND VEHICLE CLASSIFICATION COUNT DATA

## *APPENDIX C*

WEEKLY AND AXLE ADJUSTMENT FACTORS

# *APPENDIX D*

PEAK HOUR INTERSECTION TURNING MOVEMENT COUNT DATA

## *APPENDIX E*

TRAVEL DEMAND FORECASTING
TECHNICAL MEMORANDUM (CD)

## *APPENDIX F*

REVISED YEAR 2035 TRAVEL DEMAND MODEL AADT VOLUME PLOT AND HISTORIC GROWTH TREND ANALYSES

## *APPENDIX G*

TURNS5 OUTPUT

# APPENDIX H

EXISTING CONDITIONS TRAFFIC ANALYSES

## *APPENDIX I*

NO-BUILD ALTERNATIVE TRAFFIC ANALYSES

## *APPENDIX J*

TSM ALTERNATIVE TRAFFIC ANALYSES

## *APPENDIX K*

BUILD ALTERNATIVE NO. 1
CONCEPT PLAN SHEETS

## *APPENDIX L*

BUILD ALTERNATIVE NO. 1
TRAFFIC ANALYSES

# *APPENDIX M*

BUILD ALTERNATIVE NO. 2
CONCEPT PLAN SHEETS

# *APPENDIX N*

BUILD ALTERNATIVE NO. 2
TRAFFIC ANALYSES