R.G. SKINNER PARKWAY, PHASE 2
ROUNDABOUT DESIGN CRITERIA PACKAGE
MAY 2014

R.G. SKINNER PARKWAY, PHASE 2
ROUNDABOUT DESIGN CRITERIA PACKAGE
MAY 2014

The purpose of this report is to document the analysis and design decisions used determine the number of lanes required and the geometry of the proposed extension to R.G. Skinner Parkway and the proposed Phase 2 roundabout. R.G. Skinner Parkway is a proposed roadway which connects the interchange currently under construction of SR-9B and the southern terminus of the existing R.G. Skinner Parkway near Atlantic Coast High School. R.G. Skinner Parkway will serve as a second access to the high school and a connection directly to SR-9B to relieve congestion in the vicinity of the I-295/Baymeadows Road Interchange. Figure 1 illustrates the general location of the Parkway Corridor.

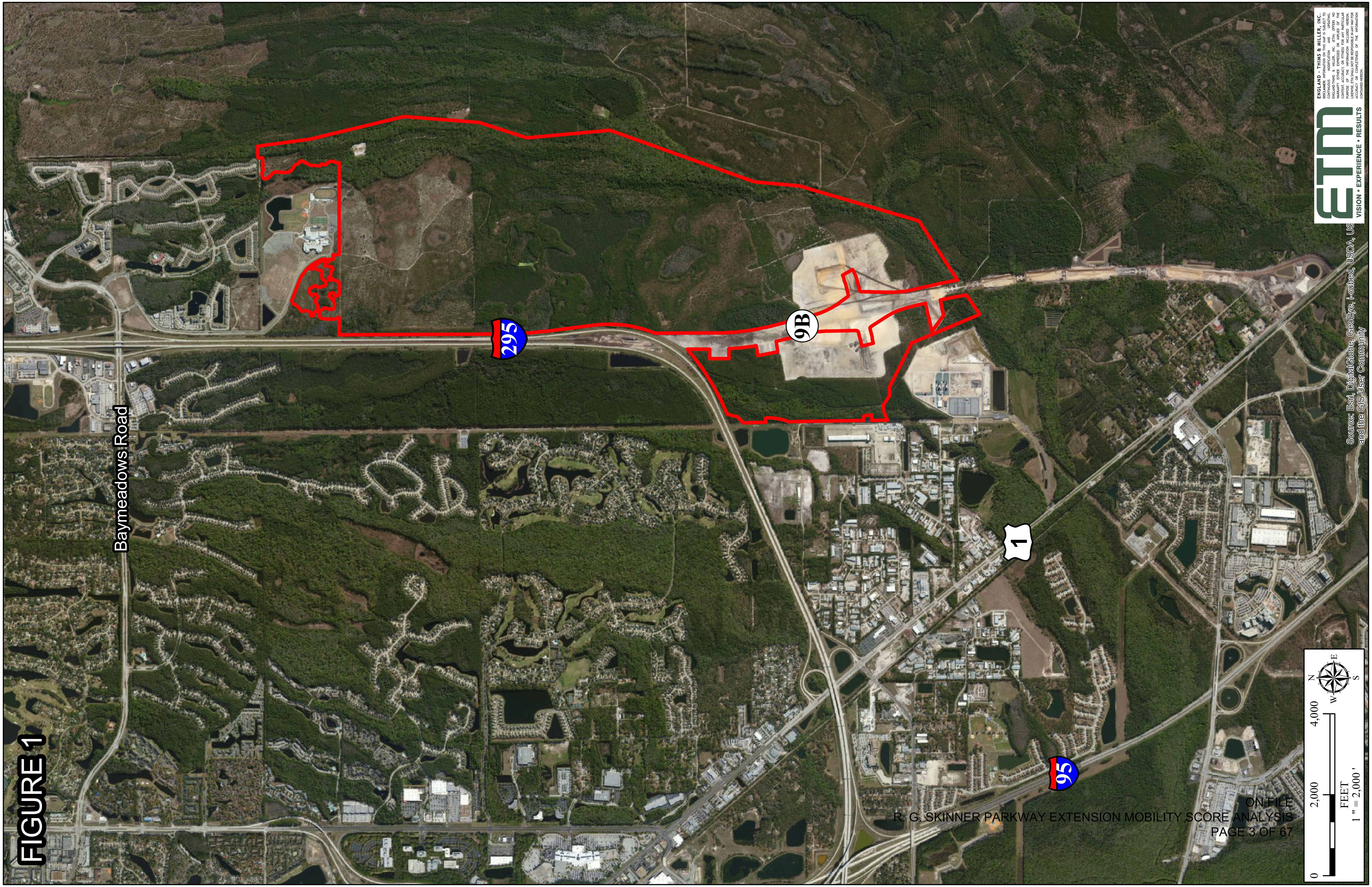


FIGURE 1
Baymeadows Road
295
9B
1
95
ENGLAND - THIMS & MILLER, INC.
DISCLAIMER: INFORMATION ON THIS MAP IS SUBJECT TO CONTINUOUS MODIFICATION AND UPDATING. ENGLAND-THIMS & MILLER, INC. (ETM) OFFERS NO WARRANTY EITHER EXPRESSED OR IMPLIED OF THE CONTENT, ACCURACY, OR FITNESS FOR ANY PARTICULAR PURPOSE OF THE INFORMATION INCLUDED HEREON. LIKEWISE, ETM SHALL NOT BE RESPONSIBLE IN ANY WAY FOR ACCURACY OR COMPLETENESS OF THE INFORMATION CONTAINED HEREON.
ETM
VISION • EXPERIENCE • RESULTS
Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, US
and the GIS User Community
N
E
S
W
0
2,000
4,000
FEET
1 " = 2,000 '
ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 3 OF 67

DESIGN REFERENCES

NCHRP (National Cooperative Highway Research Program) REPORT 672
In Cooperation with: USDOT/FHWA (Second Edition)

Technical Summary, Roundabouts - FHWA-SA-10-006
U.S. Department of Transportation/ Federal Highway Administration

DESIGN CRITERIA AND PARAMETERS

PROJECT DESCRIPTION

The R.G. Skinner Parkway Phase 2 roundabout is designed to accommodate the future 4-lane divided urban R.G. Skinner Parkway and a future 2-lane undivided urban roadway connection. The roundabout and the approaches have been modified to accommodate the interim 2-lane undivided urban configuration of R.G. Skinner Parkway. The improvements will also include sidewalks, pedestrian crossings and drainage improvements.

GEOMETRIC ELEMENTS DEFINITIONS

Central Island

The central island is the raised area in the center of a roundabout around which traffic circulates.

Splitter Island

A splitter island is a raised or painted area on an approach used to separate entering from exiting traffic, deflect and slow entering traffic, and provide storage space for pedestrians crossing the road in two stages.

Circulatory Roadway

The circulatory roadway is the curved path used by vehicles to travel in a counterclockwise fashion around the central island.

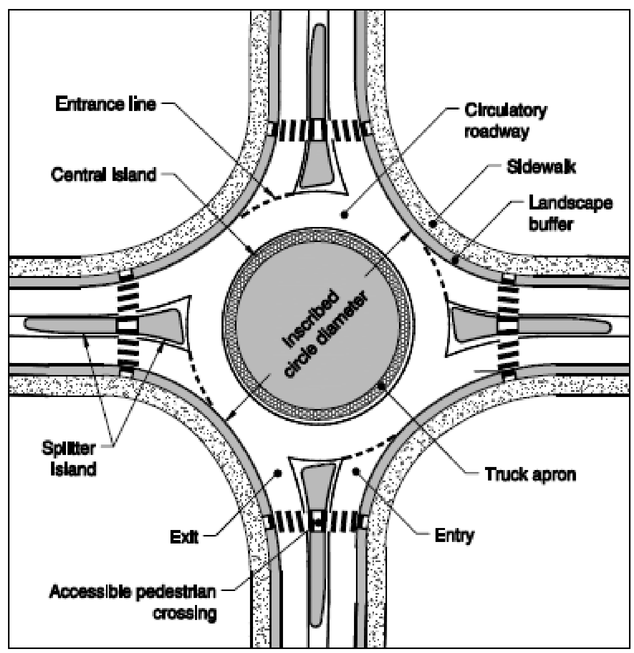


Exhibit 6-2

Apron

If required on smaller roundabouts to accommodate the wheel tracking of large vehicles, an apron is the mountable portion of the central island adjacent to the circulatory roadway.

Entrance Line

The entrance line marks the point of entry into the circulatory roadway. This line is physically an extension of the circulatory roadway edge line but functions as a yield or give-way line in the absence of a separate yield line. Entering vehicles must yield to any circulating traffic coming from the left before crossing this line into the circulatory roadway.

Accessible Pedestrian Crossings

Accessible pedestrian crossings should be provided at all roundabouts. The crossing location is set back from the yield line, and the splitter island is cut to allow pedestrians, wheelchairs, strollers, and bicycles to pass through.

Bicycle Treatments

Bicycle treatments at roundabouts provide bicyclists the option of traveling through the roundabout either as a vehicle or as a pedestrian, depending on the bicyclist's level of comfort.

Inscribed Circle Diameter

The inscribed circle diameter is the basic parameter used to define the size of a roundabout. It is measured between the outer edges of the circulatory roadway.

Circulatory Roadway Width

The circulatory roadway width defines the roadway width for vehicle circulation around the central island. It is measured as the width between the outer edge of this roadway and the central island. It does not include the width of any mountable apron, which is defined to be part of the central island.

Approach Width

The approach width is the width of the roadway used by approaching traffic upstream of any changes in width associated with the roundabout. The approach width is typically no more than half of the total width of the roadway.

Departure Width

The departure width is the width of the roadway used by departing traffic downstream of any changes in width associated with the roundabout. The departure width is typically no more than half of the total width of the roadway.

Entry Width

The width of the entry where it meets the inscribed circle, measured perpendicularly from the right edge of the entry to the intersection point of the left edge line and the inscribed circle.

Exit Width

The exit width defines the width of the exit where it meets the inscribed circle. It is measured perpendicularly from the right edge of the exit to the intersection point of the left edge line and the inscribed circle.

Entry Radius

The entry radius is the minimum radius of curvature of the outside curb at the entry.

Exit Radius

The exit radius is the minimum radius of curvature of the outside curb at the exit.

GENERAL DESIGN PROCESS

Exhibit 6-1 provides a general outline for the design process, incorporating elements of project planning, preliminary design, and final design into an iterative process.

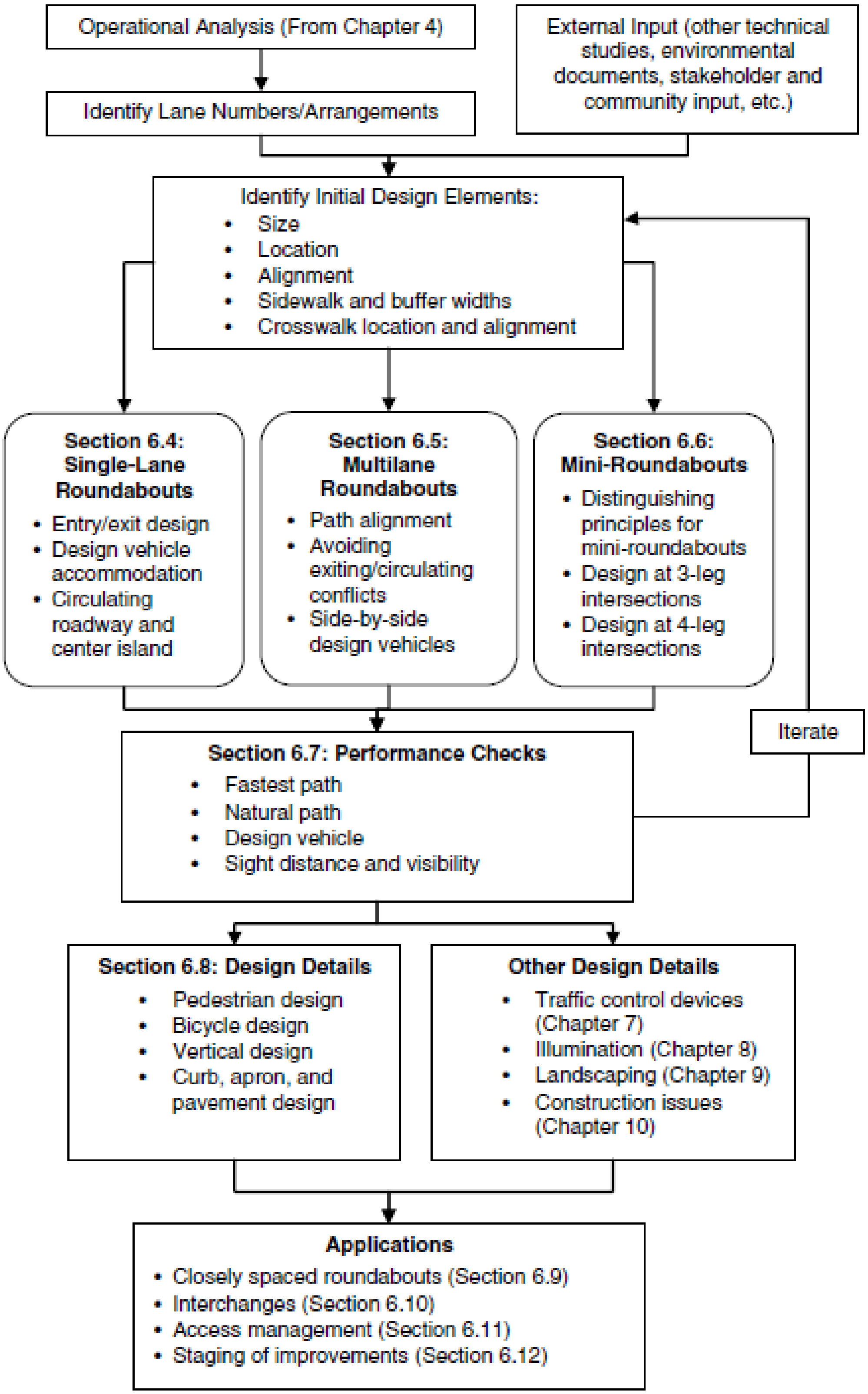


Exhibit 6-1

R.G. Skinner Parkway Traffic Analysis:

Purpose

The purpose of this report is to document the analysis conducted to determine the number of lanes required on the proposed extension to R.G. Skinner Parkway (Parkway) and the storage lengths necessary at the proposed development pods. The Parkway is a proposed roadway which connects the interchange currently under construction of SR-9B and the southern terminus of the Parkway near Atlantic Coast High School. The Parkway will serve as a second access to the high school and a connection directly to SR-9B to relieve congestion in the vicinity of the I-295/Baymeadows Road Interchange. Figure 1 illustrates the general location of the Parkway Corridor.

Traffic Estimates

Traffic estimates for the Parkway were estimated using a combination of the NERPM regional planning model and the Institute of Transportation Engineers ***Trip Generation Manual***. The NERPM model was used to forecast future traffic volumes at the SR-9B interchange as part of the Interchange Justification Report. This model was also used to approximate the travel pattern associated with the various development pods along the proposed Parkway. Attached as Figure 2, is a development plan for the lands adjacent to the Parkway. As shown, the adjacent land is envisioned to be residential in nature for the northern portion of the route and a mix of uses near the interchange with SR-9B. Table 1 is a summary of the land uses that were used to estimate the traffic from the adjacent lands. Due to the proximity of the high school, traffic volumes for both morning and afternoon peak hours were developed. Table 2 lists the morning and afternoon peak hour volumes for the various portions of the Parkway and their lanage requirements and operation condition. The lanage requirements and levels of service were based on the Florida Department of Transportation's ***Quality and Level of Service Handbook.*** Figure 3 illustrates the 2035 morning and afternoon turning movement volumes for the first section of the Parkway to be constructed.

Operational Analysis

An operational analysis of the proposed Parkway was conducted using the traffic estimates developed above and the Synchro. The levels of service of the intersections are shown in Figure 3 and depicted along with the left turn storage requirements. Table 3 tabulates the level of service and left turn queue requirements to accommodate the projected 2035 morning and afternoon peak hour volumes.

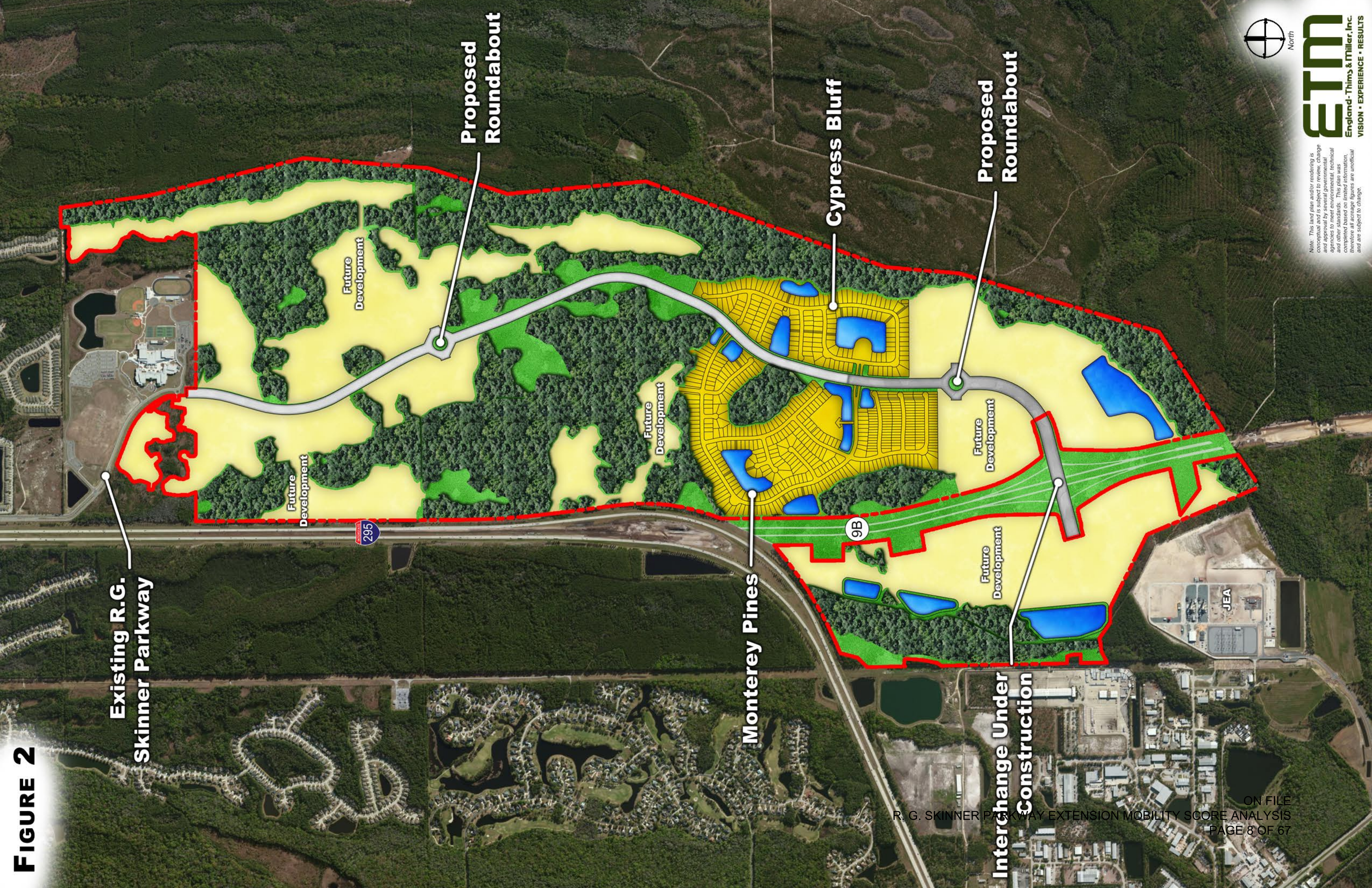

Proposed Roundabout
Cypress Bluff
Proposed Roundabout
North
England-Thims & Miller, Inc.
VISION • EXPERIENCE • RESULTS
Note: This land plan and/or rendering is conceptual and is subject to review, change and approval by several governmental agencies to meet environmental, technical and other standards. This plan was completed based on limited information, therefore all acreage figures are unofficial and are subject to change.
Future Development
Future Development
Future Development
Future Development
Future Development
295
9B
JEA
Existing R.G. Skinner Parkway
Monterey Pines
Interchange Under Construction

FIGURE 2

Table 1 – Land Use Totals

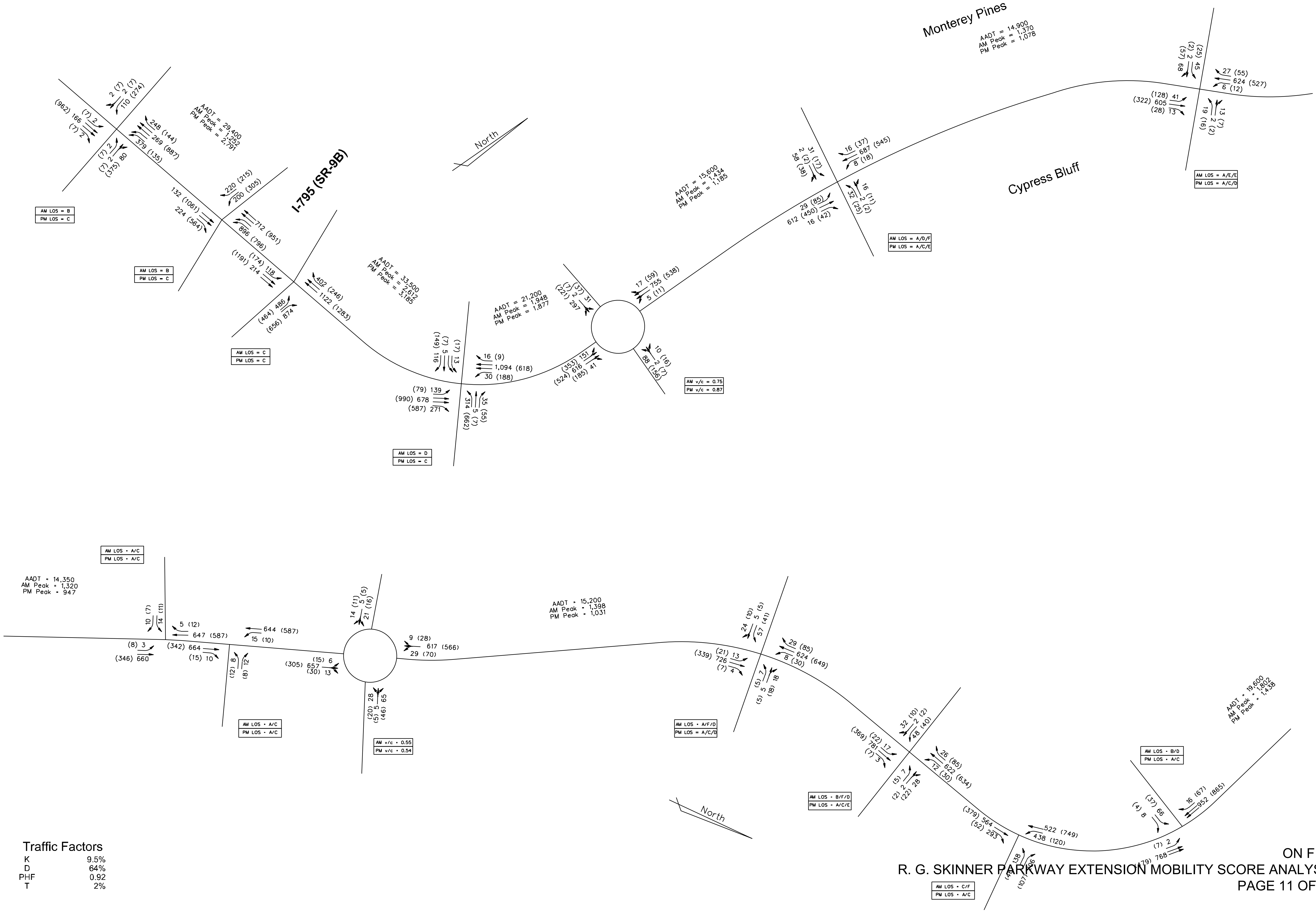
Land Use	Quantity	Unit
Warehouse	800,000	Square Feet
Single Family Residential	1,500	Dwelling Units
Multi Family Residential	1,600	Dwelling Units
Assisted Living Facility	250	Beds
Hotel	350	Rooms
City Park	10	Acres
School	2,500	Students
Office	320,000	Square Feet
Commercial	1,070,000	Square Feet

Source: ETM, 2014

Table 2 – Roadway Segment Lane Requirements

Road Segment	AM Peak Hour			PM Peak Hour		
	Volume	Lanes	LOS	Volume	Lanes	LOS
West of SR-9B	1,250 vph	4-In	B	2,791 vph	4-In	C
SR-9B to the south Traffic Circle	2,612 vph	4-In	C	3,185 vph	4-In	C
South Traffic Circle to the south Entrance to Parcels 10/11	1,434 vph	2-In	C	1,185 vph	2-In	C
South Entrance to Parcels 10/11 to North Traffic Circle	1,370 vph	2-In	C	1,078 vph	2-In	C
North Traffic Circle to existing south Terminus	1,398 vph	2-In	C	1,031 vph	2-In	C

Source: ETM, FDOT Generalized Level of Service Manual, 2009 - Table 4, Class I Arterials



Traffic Factors

K	9.5%
D	64%
PHF	0.92
T	2%

PLANS PREPARED UNDER THE DIRECTION OF:

REVISIONS:

ETM NO.
DRAWN BY:
DESIGNED BY:
CHECKED BY:
DATE:

England-Thims & Miller, Inc.
14775 Old St. Augustine Road
Jacksonville, FL 32258
TEL (904) 642-8990
FAX (904) 646-9485
CA - 00002584 LC - 0000316

ETM
VISION • EXPERIENCE • RESULTS

2035 Traffic Estimates

RG Skinner Parkway Extension for The Parc Group

DRAWING NUMBER

Figure 3

Table 3 – Intersection Levels of Service and Left Turn Queue Requirements

Intersection	Movement	AM Peak Hour LOS	PM Peak Hour LOS	Required Storage
R.G. Skinner Parkway and SB Ramps		B	C	
	SB LT			325'
	WB LT			250'
R.G. Skinner Parkway and NB Ramps		C	C	
	NB Lt			250'
	EB Lt			200'
R.G. Skinner Parkway and Mixed Use		B	C	
Entrance	SB Lt			100'
	NB Lt			150'
	EB Lt			100'
	WB Lt			100'
R.G. Skinner Parkway and South entrance		A	A	
To Cypress Bluff/Monterey Pines	SB Lt			100'
	NB Lt			150'
	EB Lt			100'
	WB Lt			100'
RG Skinner Parkway and North entrance		A/E	A/D	
To Parcels 10/11	SB Lt			100'
	NB Lt			150'
	EB Lt			100'
	WB Lt			100'

Source: ETM, Synchro Software using Highway Capacity Manual Procedures.

IDENTIFY LANE NUMBERS / ARRANGEMENTS

Lane Number Determination: Using Average Daily Traffic:

Design Element	Mini-Roundabout	Single-Lane Roundabout	Multilane Roundabout
Desirable maximum entry design speed	15 to 20 mph (25 to 30 km/h)	20 to 25 mph (30 to 40 km/h)	25 to 30 mph (40 to 50 km/h)
Maximum number of entering lanes per approach	1	1	2+
Typical inscribed circle diameter	45 to 90 ft (13 to 27 m)	90 to 180 ft (27 to 55 m)	150 to 300 ft (46 to 91 m)
Central island treatment	Fully traversable	Raised (may have traversable apron)	Raised (may have traversable apron)
Typical daily service volumes on 4-leg roundabout below which may be expected to operate without requiring a detailed capacity analysis (veh/day)*	Up to approximately 15,000	Up to approximately 25,000	Up to approximately 45,000 for two-lane roundabout

*Operational analysis needed to verify upper limit for specific applications or for roundabouts with more than two lanes or four legs.

Exhibit 1-9

With the year 2035 projected average daily traffic volume of 14,350 south of the roundabout and 15,200 north of the roundabout the volumes are well within the range for a single-lane roundabout. However, since this roadway section is being constructed as two-lanes of a future four-lane divided urban roadway a two-lane roundabout design is proposed, with interim conditions (markings) limiting the circulation to a two-lane roundabout. This configuration allows the proposed roundabout to be constructed to the full outside diameter, so that only modification to the signing and marking will be needed within the limits of the roundabout for the final four-lane configuration. According to exhibit 1-9, this two-lane roundabout can maintain a capacity of up to approximately 45,000, with the single lane interim capacity of approximately 25,000 far exceeding the 2035 projections.

Lane Number Determination: Using Exhibit 3-14 Volume Thresholds for Determining the Number of Entry Lanes Required

Using the Peak Hour Traffic from the previous section, R.G. Skinner Parkway:

2030 PM Peak
NB Entry: 30+305+15+5+16+29 = **400**
WB Entry: 46+5+20+305+15+21 = **412**
SB Entry: 28+566+70+5+20+15 = **704**
EB Entry: 11+5+16+566+70+20 = **688**

2030 AM Peak
NB Entry: 13+657+6+5+21+29 = **731**
WB Entry: 65+5+28+657+6+21 = **782**
SB Entry: 13+657+6+5+21+29 = **731**
EB Entry: 14+5+21+617+29+28 = **714**

Volume Range (sum of entering and conflicting volumes)	Number of Lanes Required
0 to 1,000 veh/h	▪ Single-lane entry likely to be sufficient
1,000 to 1,300 veh/h	▪ Two-lane entry may be needed ▪ Single-lane may be sufficient based upon more detailed analysis.
1,300 to 1,800 veh/h	▪ Two-lane entry likely to be sufficient
Above 1,800 veh/h	▪ More than two entering lanes may be required ▪ A more detailed capacity evaluation should be conducted to verify lane numbers and arrangements.

Source: New York State Department of Transportation

Based on the traffic numbers a single-lane roundabout is sufficient. However, knowing that R.G. Skinner Parkway is being constructed as 2-lanes of a future 4-lane facility, a 2-lane roundabout was designed and marked as a single lane roundabout.

IDENTIFY INITIAL DESIGN ELEMENTS

Size

The inscribed circle diameter for a single-lane roundabout typically needs to be at least 105 ft to accommodate a WB-50 (WB-15) design vehicle; a larger diameter is typically needed for design vehicles larger than a WB-50 (WB-15).Diameters in the range of 150 to 300 ft are typical for multi-lane roundabouts.

An inscribed diameter of 200 feet has been provided for the R.G. Skinner Parkway Phase 2 roundabout.

Location / Alignment

The common starting point in design is to center the roundabout so that the centerline of each leg passes through the center of the inscribed circle (radial alignment). This location typically allows the geometry of the roundabout to be adequately designed such that vehicles will maintain slow speeds through both the entries and the exits. The radial alignment also makes the central island more conspicuous to approaching drivers and minimizes roadway modification required upstream of the intersection.

As a new alignment with no existing outside connections, the Alternative 2: Alignment through Center was chosen for this location. This layout combined with the larger inscribed diameter keeps the alignment changes local to the roundabout and manages speed throughout the roundabout entry and exit.

Entry Alignment

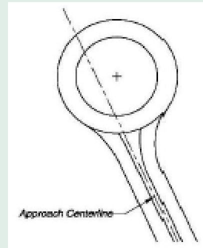
Question

Should the approach alignment run through the center of the inscribed circle? Or is it acceptable to offset the approach centerline to one side?

Design Principle

The alignment does not have to pass through the center of the roundabout; however, it has a primary effect on the entry/exit design. The optimal alignment allows for an entry design that provides adequate deflection and speed control while also providing appropriate view angles to drivers and balancing property impacts/costs.

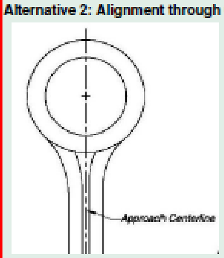
Alternative 1: Offset Alignment to the Left of Center



ADVANTAGES:
- Allows for increased deflection
- Beneficial for accommodating large trucks with small inscribed circle diameter—allows for larger entry radius while maintaining deflection and speed control
- May reduce impacts to right-side of roadway

TRADE-OFFS
- Increased exit radius or tangential exit reduces control of exit speeds and acceleration through crosswalk area
- May create greater impacts to the left side of the roadway

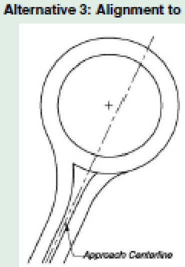
Alternative 2: Alignment through Center of Roundabout



ADVANTAGES:
- Reduces amount of alignment changes along the approach roadway to keep impacts more localized to intersection
- Allows for some exit curvature to encourage drivers to maintain slower speeds through the exit

TRADE-OFFS
- Increased exit radius reduces control of exit speeds/acceleration through crosswalk area
- May require a slightly larger inscribed circle diameter (compared to offset-left design) to provide the same level of speed control

Alternative 3: Alignment to Right of Center



ADVANTAGES:
- Could be used for large inscribed circle diameter roundabouts where speed control objectives can still be met
- Although not commonly used, this strategy may be appropriate in some instances (provided that speed objectives are met) to minimize impacts, improve view angles, etc.

TRADE-OFFS
- Often more difficult to achieve speed control objectives, particularly at small diameter roundabouts
- Increases the amount of exit curvature that must be negotiated

Exhibit 6-10

Sidewalk and Buffer Widths

Section 6.8.1.1 recommends a setback distance of 5 ft should be used with a sidewalk width of 6 ft. However this section continues on to recommend areas with heavy pedestrian volumes and where access to bicyclists is present the sidewalk should be increased to a minimum of 10 ft and additional setbacks are desirable.

The design of the R.G. Skinner Parkway roundabout provides a 20 ft plus setback similar to Exhibit 6-64 below, and a since the design allows for a shared use with bicycles a 12 ft multiuse path is proposed around the roundabout.

Crosswalk Location and Alignment

A typical and minimum crosswalk setback of 20 ft is recommended.

This design provides a 20 ft or greater setback at all locations.

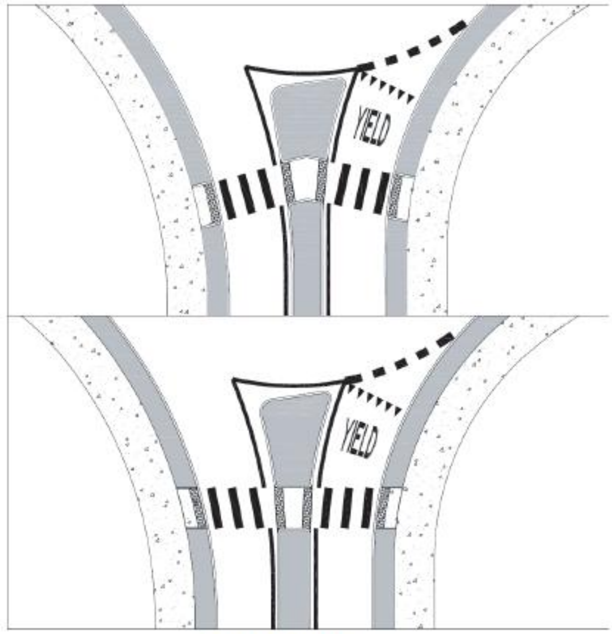
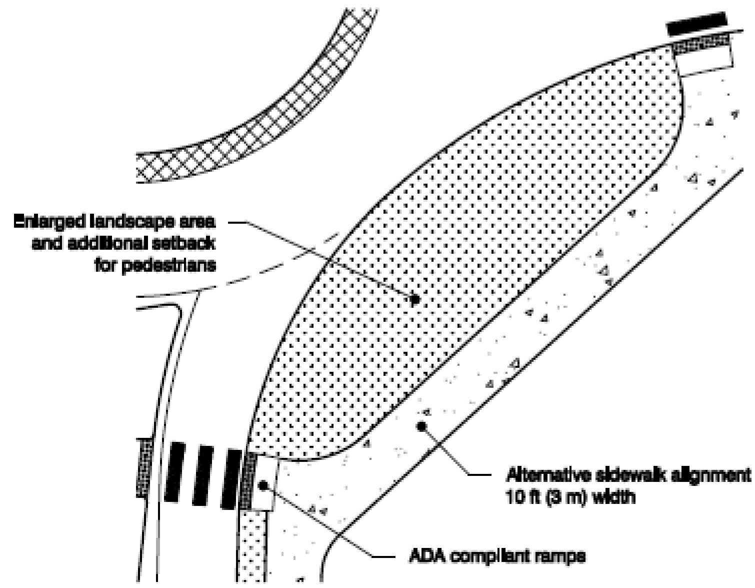


Exhibit 6-66

Exhibit 6-64
Alternative Sidewalk Treatments

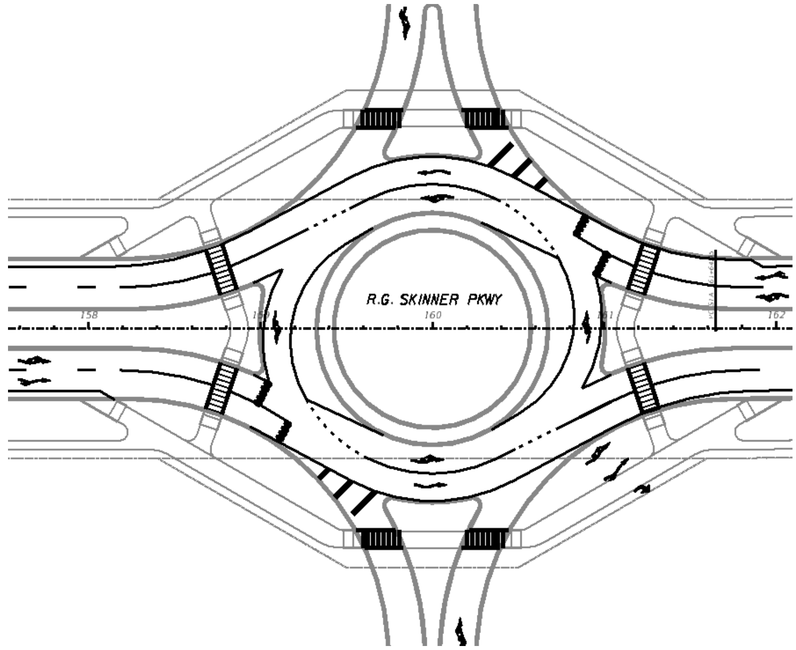


MULTILANE ROUNDABOUTS

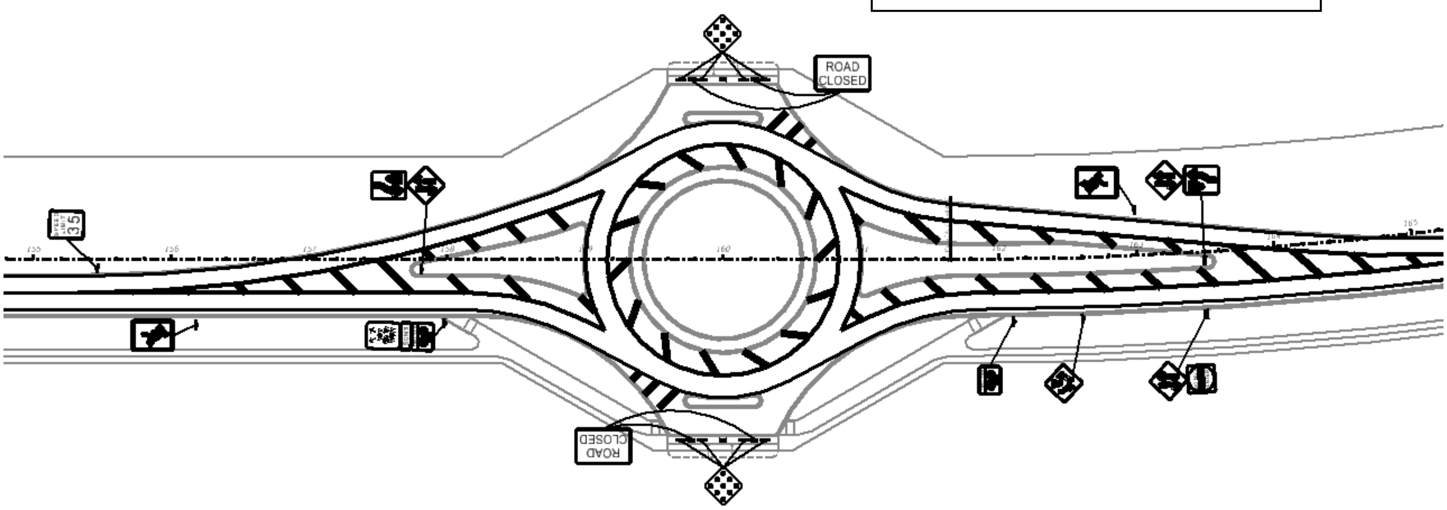
Multilane roundabout geometric design criteria are contained in Chapter 6, Section 6.5 of the NCHRP Report 672. (Attached as Appendix A.)

Lane Numbers and Arrangements

Multilane roundabouts have at least one approach with at least two lanes on the entries or exits. The number of lanes can vary from approach to approach as long as they are appropriately assigned by lane designation signs and markings. Likewise, the number of lanes within the circulatory roadway may vary depending upon the number of entering and exiting lanes. The important principle is that the design requires continuity between the entering, circulating, and exiting lanes such that lane changes are not needed to navigate the roundabout. The driver should be able to select the appropriate lane upstream of the entry and stay within that lane through the roundabout to the intended exit without any lane changes. This principle is consistent with the design of all types of intersections.



ULTIMATE CONFIGURATION



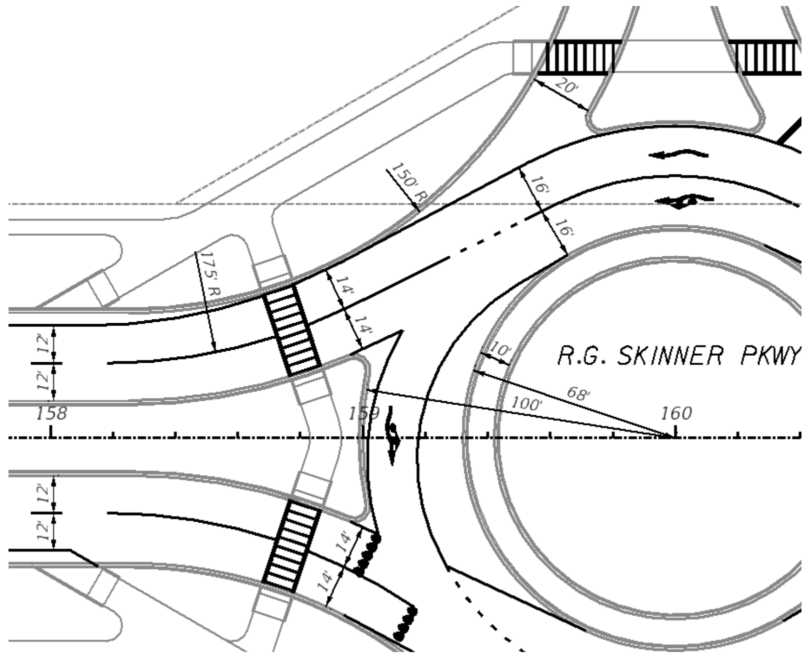
INTERIM CONFIGURATION

The R.G. Skinner roundabout has been designed to carry the future 4-lane divided R.G. Skinner Parkway and a 2-lane divided cross road. The lanes have been arranged to allow the driver to select a lane prior to entering the roundabout and proceed through the circulation to the preferred exit without changing lanes. Interim conditions limit the roundabout to a single lane roundabout.

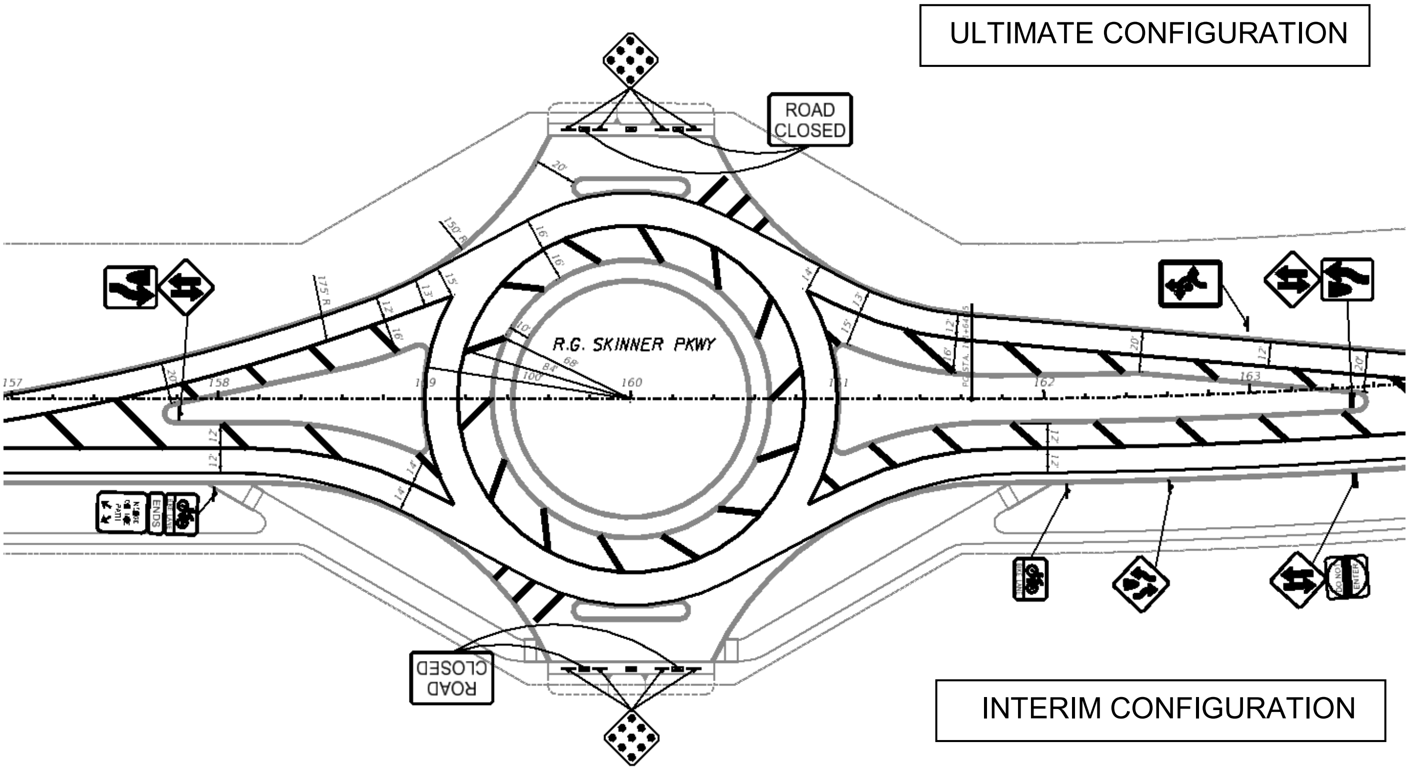
Entry Design and Approach Alignment

Typical entry widths for two-lane entrances range from 24 to 30 ft. However, values higher or lower than this range may be appropriate for site-specific design vehicle and speed requirements for critical vehicle paths. The entry width should be primarily determined based upon the number of lanes identified in the operational analysis combined with the turning requirements for the design vehicle. Entry radii for multilane roundabouts should typically exceed 65 ft (20 m) to encourage adequate natural paths and avoid sideswipe collisions on entry.

Each approach to the R.G. Skinner roundabout has a 28 ft entry width for the R.G. Skinner approaches, a 20' entry width for the side street approached and a minimum curb radius of 150 ft, to allow the potential for larger vehicle turning radii.



ULTIMATE CONFIGURATION



INTERIM CONFIGURATION

Center Island and Circulating Roadway

The central island of a roundabout is the raised, mainly non-traversable area surrounded by the circulatory roadway. It may also include a traversable truck apron. The island is typically landscaped for aesthetic reasons and to enhance driver recognition of the roundabout upon approach. A circular central island is preferred because the constant-radius circulatory roadway helps promote constant speeds around the central island. The size of the central island is dependent upon the inscribed circle diameter and the required circulatory roadway width.

The R.G. Skinner Parkway roundabout is designed with a raised non-traversable area with an internal diameter of 136 ft wrapped with an additional 10 ft mountable truck apron.

The circulatory roadway width is usually governed by the design criteria relating to the types of vehicles that may need to be accommodated adjacent to one another through a multilane roundabout. The combination of vehicle types to be accommodated side-by-side is dependent upon the specific site traffic conditions.

Multilane circulatory roadway lane widths typically range from 14 to 16 ft (4.3 to 4.9 m). Use of these values results in a total circulating width of 28 to 32 ft (8.5 to 9.8 m) for a two-lane circulatory roadway.

With the proximity of the commercial area and the high school, the single unit truck and school bus were chosen as the design vehicles. The large circulating radius combined with 16 ft lanes allow for single unit trucks and school busses to circulate through the roundabout without encroaching in the adjacent lane. Larger vehicles like the WB-67 can navigate the roundabout but will have minor encroachments in the adjacent lane.

At multilane roundabouts, the circulatory roadway width may also be variable depending upon the number of lanes and the design vehicle turning requirements. A constant width is not required throughout the entire circulatory roadway, and it is desirable to provide only the minimum width necessary to serve the required lane configurations within that specific portion of the roundabout.

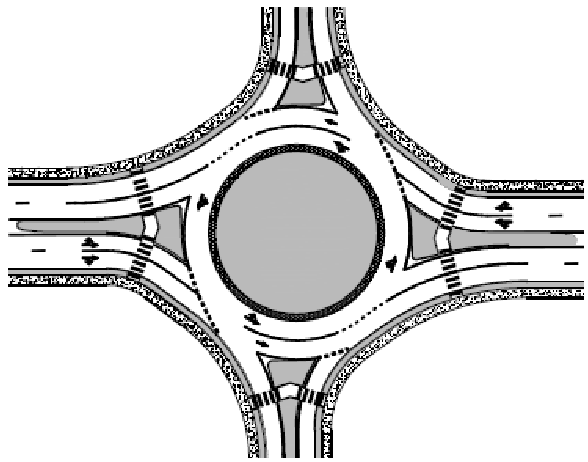


Exhibit 6-26
Multilane Major Street with Single Lane on Minor Street

In some instances, the circulatory roadway width may actually need to be wider than the corresponding entrance that is feeding that portion of the roundabout. For example, in situations where two consecutive entries require exclusive left turns, a portion of the circulatory roadway will need to contain an extra lane and spiral markings to enable all vehicles to reach their intended exits without being trapped or changing lanes. This situation is illustrated in Exhibit 6-27, where a portion of the circulatory roadway is required to have three lanes despite the fact that all of the entries have only two lanes.

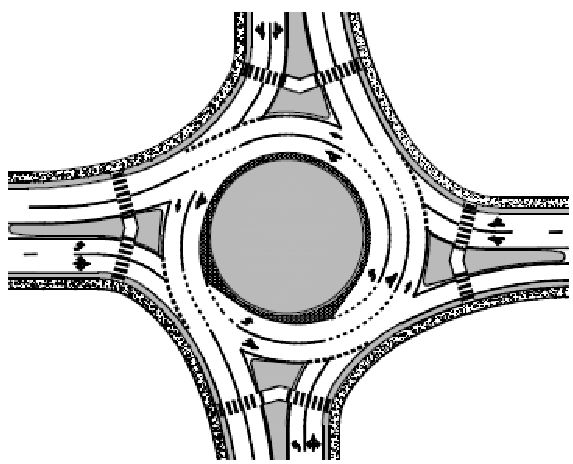


Exhibit 6-27
Two-Lane Roundabout with Consecutive Double-Lefts

With the proximity to the 9B Interchange and the traffic models (covered in later sections of this report) showing a required 2-laned roadway between the interchange and the roundabout, the R.G. Skinner Parkway roundabout has been designed to allow 2 through lanes on R.G. Skinner Parkway as well as 1 entry lane from each side road approach. In allowing 2 entry lanes on each R.G. Skinner approach additional “spiral” lanes have been included to maintain the flow through the roundabout without overlapping movements.

Exit Design

The exit curb radii are usually larger than the entry curb radii in order to minimize the likelihood of congestion and crashes at the exits. This, however, is balanced by the need to maintain slow speeds through the pedestrian crossing on exit. The exit design is also influenced by the design environment (urban versus rural), pedestrian demand, the design vehicle, and physical constraints. Generally, exit curb radii should be no less than 50 ft, with values of 100 to 200 ft being more common.

Each exit from the roundabout on R.G. Skinner has been designed with 175 ft radii to minimize the congestion while maintaining speed control and pedestrian safety. Each of the R.G. Skinner Parkway exits are designed with 2 lanes exiting from the roundabout, while the side road exits are designed with 1 exit lane from the roundabout.

PERFORMANCE CHECKS

Fastest Path

The fastest path allowed by the geometry determines the negotiation speed for that particular movement into, through, and exiting the roundabout. It is the smoothest, flattest path possible for a single vehicle, in the absence of other traffic and ignoring all lane markings. The fastest path is drawn for a vehicle traversing through the entry, around the central island, and out the relevant exit.

Site Category	Recommended Maximum Theoretical Entry Design Speed
Mini-Roundabout	20 mph (30 km/h)
Single Lane	25 mph (40 km/h)
Multilane	25 to 30 mph (40 to 50 km/h)

Exhibit 6-47

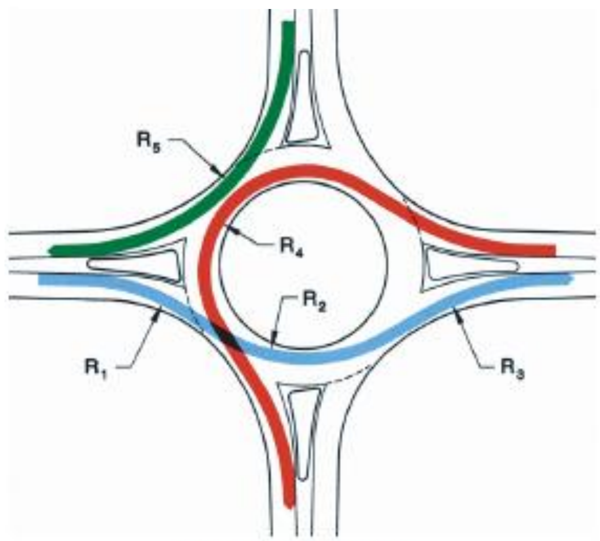


Exhibit 6-46

The critical path radii must be checked for each approach. *R1*, the *entry path radius,* is the minimum radius on the fastest through path prior to the entrance line. *R2*, the *circulating path radius,* is the minimum radius on the fastest through path around the central island. *R3*, the *exit path radius,* is the minimum radius on the fastest through path into the exit. It is important to note that these vehicular path radii are not the same as the curb radii. When drawing the path, a short length of tangent should be drawn between consecutive curves to account for the time it takes for a driver to turn the steering wheel.

Consistency between the speeds of various movements within the intersection can help to minimize the crash rate between conflicting traffic streams. Relative speeds between conflicting traffic streams and between consecutive geometric elements should be minimized such that the maximum speed differential between movements should be no more than approximately 10 to 15 mph. As with other design elements, speed consistency should be balanced with other objectives in establishing a design.

According to the fastest path layout (See below right), the following radii and associated speeds were provided for the entrance, circulation and exit paths:

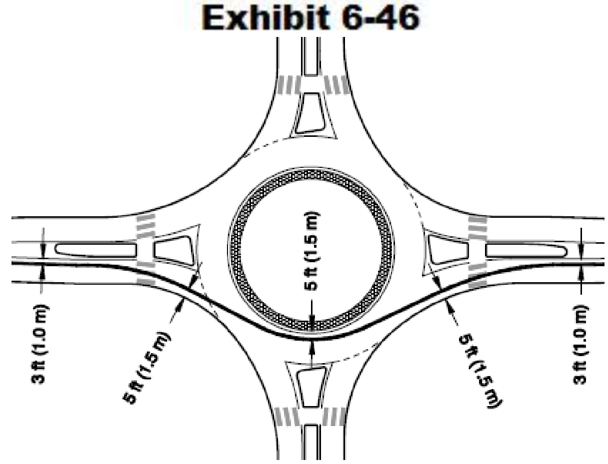


Exhibit 6-48

R. G. Skinner Parkway:
R1 (NB) = 303 ft (28 MPH)
R1 (SB = 278 ft (28 MPH)
R2 & R4 = 73 ft (18 MPH)
R3 = 342 ft (30 MPH)
R5 = 155 ft (25 MPH)

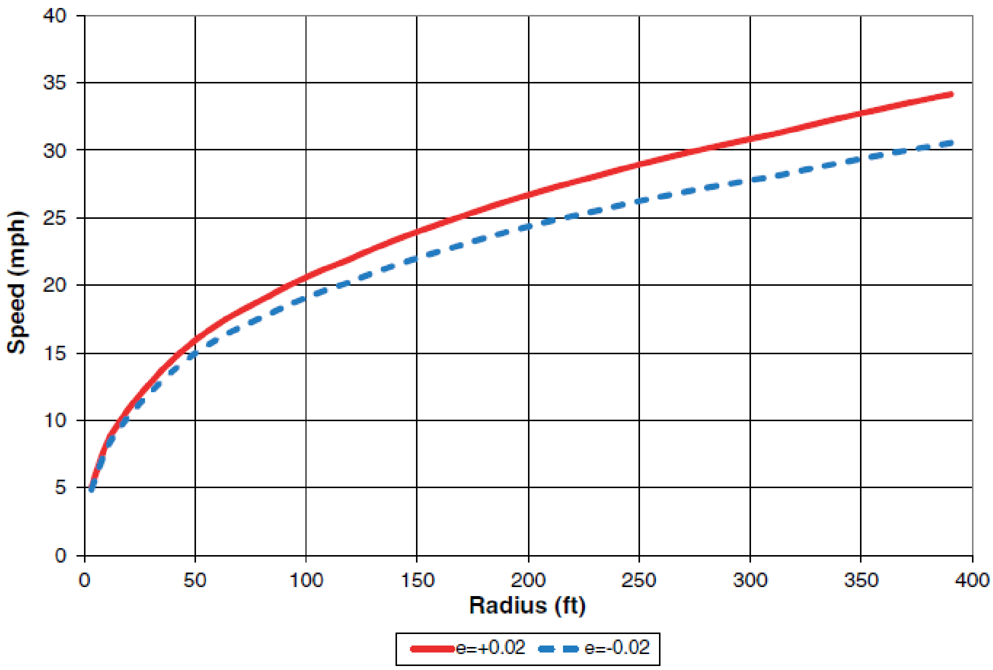
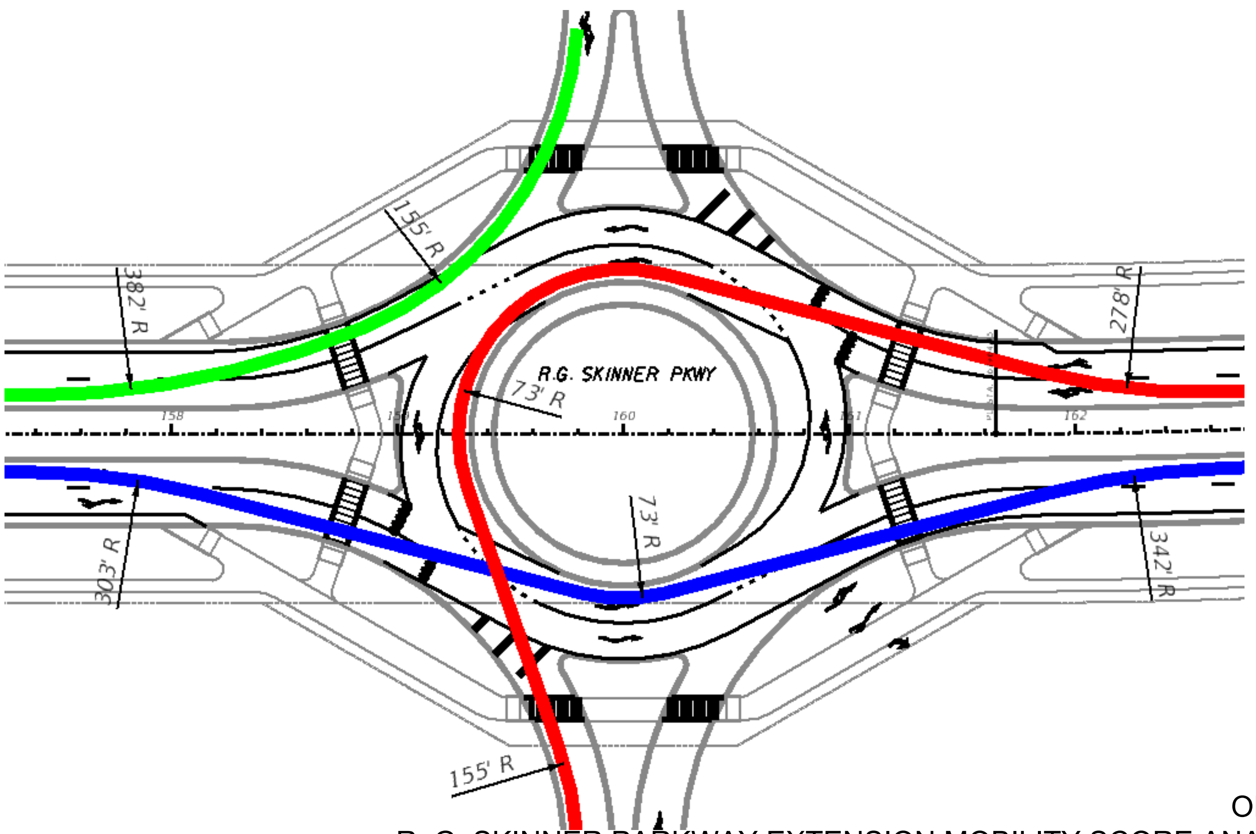


Exhibit 6-52
Speed–Radius Relationship

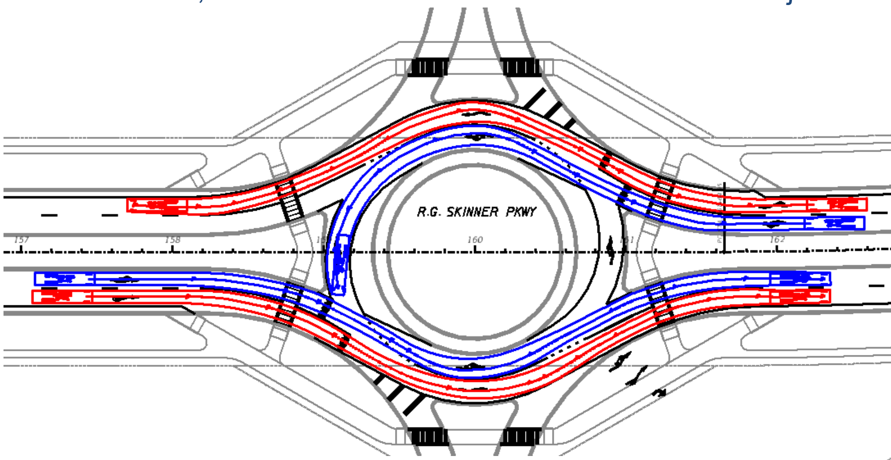


Natural Path

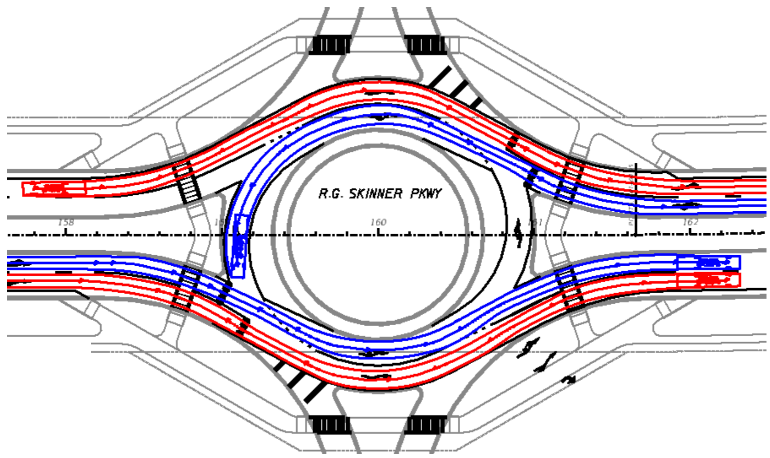
In addition to evaluating the fastest path, at multilane roundabouts the engineer should also consider the natural vehicle paths.

Design Vehicle

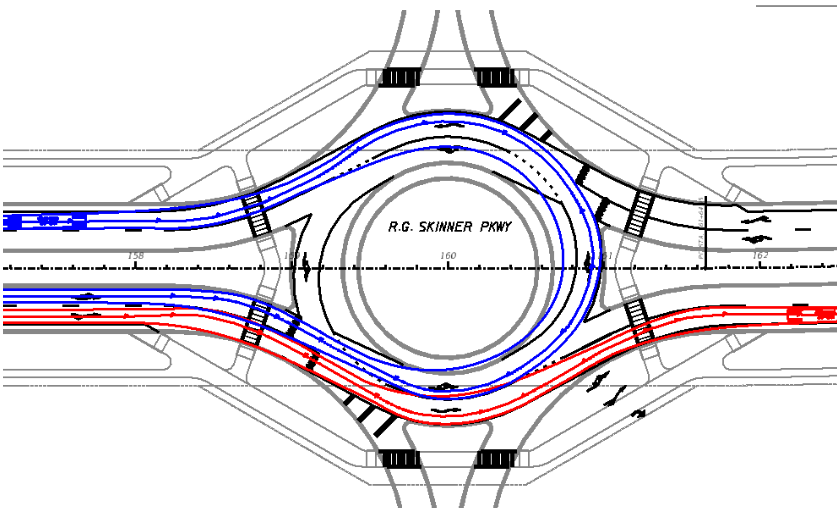
Autoturn was used to analyze SU-40, BUS-40, WB-65 and WB-67 movements. The R.G. Skinner Parkway roundabout is designed to allow the SU-40 and BUS-40 design vehicles to navigate the roundabout simultaneously. (See some typical layouts below). The larger WB-62 and WB- 67 design vehicles can utilize the roundabout, but their movements will encroach on the adjacent lanes and utilize the truck apron.



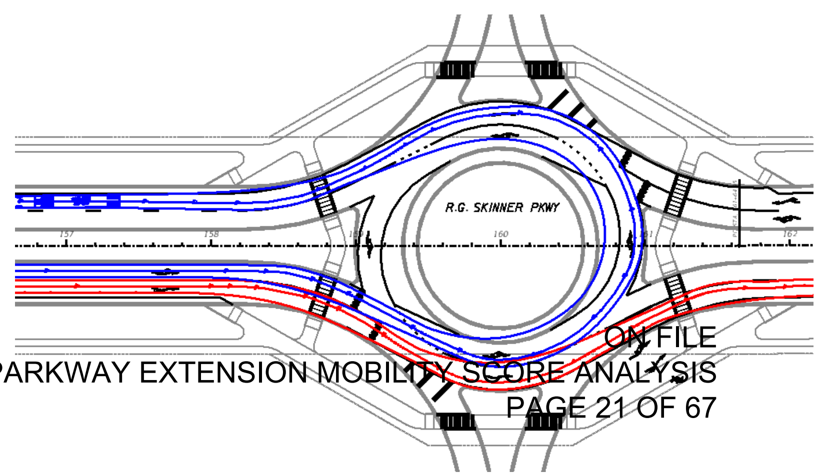
SU-40 Vehicle



Bus-40 Vehicle



WB-62 Vehicle



WB-67 Vehicle

Sight Distance and Visibility

The two most relevant aspects of sight distance for roundabouts are <u>stopping sight distance</u> and <u>intersection sight distance</u>. At roundabouts, a minimum of three critical types of locations should be checked: approach sight distance, sight distance on circulatory roadway and sight distance to crosswalk on exit.

Speed (km/h)	Computed Distance* (m)	Speed (mph)	Computed Distance* (ft)
10	8.1	10	46.4
20	18.5	15	77.0
30	31.2	20	112.4
40	46.2	25	152.7
50	63.4	30	197.8
60	83.0	35	247.8
70	104.9	40	302.7
80	129.0	45	362.5
90	155.5	50	427.2
100	184.2	55	496.7

* Assumes 2.5 s perception–braking time, 3.4 m/s² (11.2 ft/s²) driver deceleration

Exhibit 6-54

Stopping Sight Distance

Approach:

Each of the approaches of R.G. Skinner Parkway have stopping sight distance greater than 300 ft exceeding the distance required for the roadway approach design speed of 35 MPH. (247.8 ft)

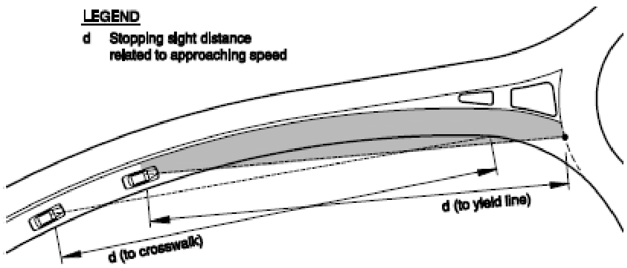


Exhibit 6-55

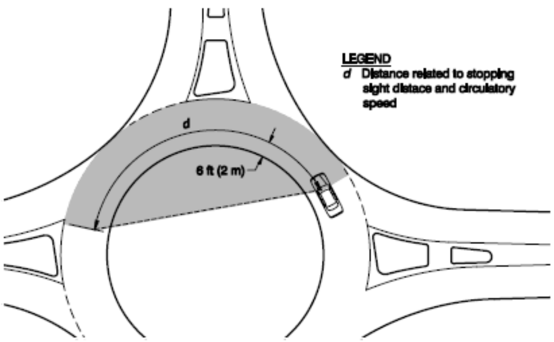


Exhibit 6-56

Circulating:

The size of the roundabout allows for the stopping sight distance of greater than 150 ft for all approaches for the circulatory roadway, exceeding the minimum distance required for 20 MPH (112.4 ft).

Crosswalk:

The stopping sight distance provided for the crosswalk exceeds 150 ft for all approaches, exceeding the minimum distance required for 20 MPH (112.4 ft).

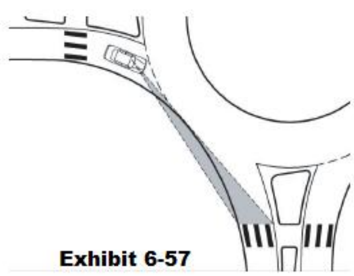


Exhibit 6-57

Intersection Sight Distance

Intersection sight distance is the distance required for a driver without the right-of-way to perceive and react to the presence of conflicting vehicles. Intersection sight distance is achieved through the establishment of *sight triangles* that allow a driver to see and safely react to potentially conflicting vehicles. At roundabouts, the only locations requiring evaluation of intersection sight distance are the entries.

Intersection sight distance is traditionally measured through the determination of a sight triangle. This triangle is bounded by a length of roadway defining a limit away from the intersection on each of the two conflicting approaches and by a line connecting those two limits. For roundabouts, these legs should be assumed to follow the curvature of the roadway, and thus distances should be measured not as straight lines but as distances along the vehicular path.

Entering Stream is composed of vehicles from the immediate upstream entry. The speed for this movement can be approximated by taking the average of the theoretical entering (*R1*) speed and the circulating (*R2*) speed.

Theoretical Entering Speed Calculation

((R1 + R2) / 2) = ((28 MPH + 18 MPH) / 2) = 23 MPH

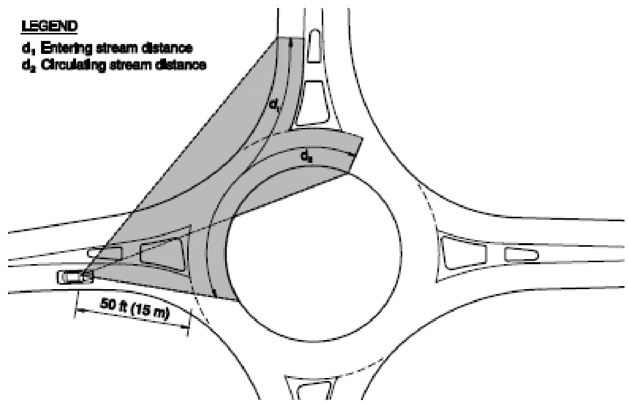


Exhibit 6-58

Circulating stream is composed of vehicles that enter the roundabout prior to the immediate upstream entry. This speed can be approximated by taking the speed of left-turning vehicles (path with radius *R4*).

Theoretical Circulation Speed Calculation

R2 = R4 = 18 MPH

Conflicting Approach Speed (mph)	Computed Distance (ft)	Conflicting Approach Speed (km/h)	Computed Distance (m)
10	73.4	20	27.8
15	110.1	25	34.8
20	146.8	30	41.7
25	183.5	35	48.7
30	220.2	40	55.6

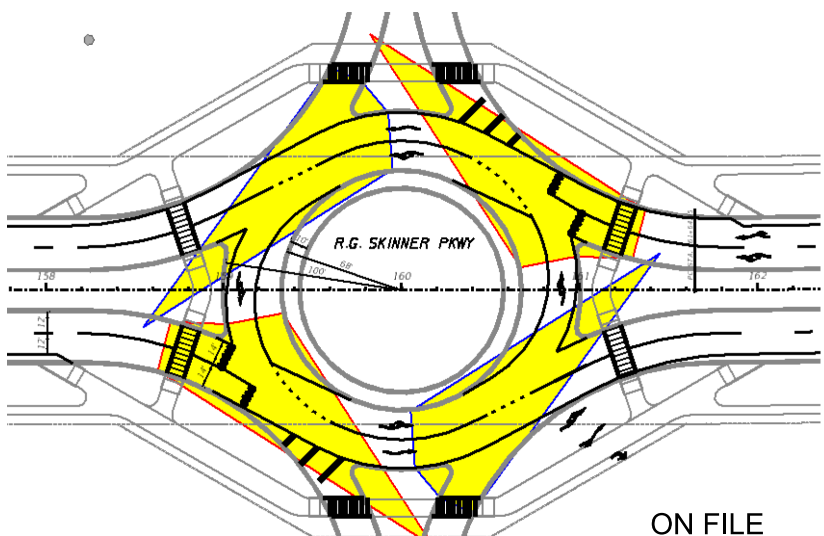
Note: Computed distances are based on a critical headway of 5.0 s.

Exhibit 6-59

R.G. Skinner Parkway Sight Triangles:

Entering Speed 25 MPH = 183.5 ft.

Circulation Speed 20 MPH = 146.8 ft



Appendix A

NCHRP (National Cooperative Highway Research Program) REPORT 672

Section 6.5: Multilane Roundabouts

the material used for the sidewalks so that pedestrians are not encouraged to cross the circulatory roadway. In addition, the truck apron features should be designed to encourage heavy vehicles to use this portion of the central island when necessary. If the colored or textured pavement appears to be for aesthetics only, truck drivers may be discouraged to traverse the apron (*12*). Exhibit 6-22 illustrates an example of applying aesthetic pavement treatments to the truck apron. Some agencies have used waffle block material as part of the truck apron, as shown in Exhibit 6-23. This provides additional truck apron width for the occasional large vehicle without adding additional impervious area.

Exhibit 6-22
Example of Aesthetic Truck Apron Treatments

(a) Arcata, California

(b) Santa Barbara, California

Exhibit 6-23
Example of Waffle Blocks Used within a Truck Apron

Killingworth, Connecticut

6.5 MULTILANE ROUNDABOUTS

The principles and design process described previously apply to multilane roundabouts but in a more complex way. Because multiple traffic streams may enter, circulate through, and exit the roundabout side-by-side, the engineer also should consider how these traffic streams interact with each other. The geometry of the roundabout should provide adequate alignment and establish appropriate lane configurations for vehicles in adjacent entry lanes to be able to negotiate the roundabout geometry without competing for the same space. Otherwise, operational and/or safety deficiencies may occur.

Multilane roundabout design tends to be less forgiving than single-lane roundabout design. Multilane design can have a direct impact on vehicle alignment and lane choice, which can affect both the safety performance and capacity. Capacity, safety, property impacts, and costs are interrelated, and a balance of these

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 25 OF 67

components becomes more difficult with multilane roundabout design. Due to this balancing of design elements that is required to meet the design principles, the use or creation of boilerplate or standard designs is discouraged.

The design of pavement markings and signs at a multilane roundabout is also critical to achieving predicted capacities and optimal overall operations. Geometry, pavement markings, and signs must be designed together to create a comprehensive system to guide and regulate road users who are traversing roundabouts. The marking plan should be integral to the preliminary design phase of a project. Chapter 7 provides additional detail on the design of pavement markings and signs for multilane roundabouts.

In addition to the fundamental principles outlined in Section 6.2, other key considerations for all multilane roundabouts include:

- Lane arrangements to allow drivers to select the appropriate lane on entry and navigate through the roundabout without changing lanes,
- Alignment of vehicles at the entrance line into the correct lane within the circulatory roadway,
- Accommodation of side-by-side vehicles through the roundabout (i.e., a truck or bus traveling adjacent to a passenger car),
- Alignment of the legs to prevent exiting–circulating conflicts, and
- Accommodation for all travel modes.

The reader should also refer to Section 6.4 on single-lane roundabouts as some design elements [such as central islands (Section 6.4.4)] are not described again in this multilane roundabouts section because the information is not substantially different for multilane design. Section 6.8 also provides additional information pertaining to design of pedestrian and bicycle facilities.

6.5.1 LANE NUMBERS AND ARRANGEMENTS

Multilane roundabouts have at least one approach with at least two lanes on the entries or exits. The number of lanes can vary from approach to approach as long as they are appropriately assigned by lane designation signs and markings. Likewise, the number of lanes within the circulatory roadway may vary depending upon the number of entering and exiting lanes. The important principle is that the design requires continuity between the entering, circulating, and exiting lanes such that lane changes are not needed to navigate the roundabout. The driver should be able to select the appropriate lane upstream of the entry and stay within that lane through the roundabout to the intended exit without any lane changes. This principle is consistent with the design of all types of intersections.

The number of lanes provided at the roundabout should be the minimum needed for the existing and anticipated demand as determined by the operational analysis. The engineer is discouraged from providing additional lanes that are not needed for capacity purposes as these additional lanes can reduce the safety effectiveness at the intersection. If additional lanes are needed for future conditions, a phased design approach should be considered that would allow for future expansion.

On multilane roundabouts, it is also desirable to achieve balanced lane utilization in order to be able to achieve predicted capacity. There are a number of design variables that can produce lane imbalance, such as poorly designed entry or exit alignments or turning movement patterns. There is also a need to recognize possible downstream system variables, such as a major trip generator, interchange ramp, or bottleneck at a downstream intersection. All of these variables may influence lane choice at a roundabout.

6.5.2 ENTRY WIDTH

The required entry width for any given design is dependent upon the number of lanes and design vehicle. A typical entry width for a two-lane entry ranges from 24 to 30 ft (7.3 to 9.1 m) for a two-lane entry and from 36 to 45 ft (11.0 to 13.7 m) for a three-lane entry. Typical widths for individual lanes at entry range from 12 to 15 ft (3.7 to 4.6 m). The entry width should be primarily determined based upon the number of lanes identified in the operational analysis combined with the turning requirements for the design vehicle. Excessive entry width may not produce capacity benefits if the entry width cannot be fully used by traffic.

For locations where additional entry capacity is required, there are generally two options:

1. Adding a full lane upstream of the roundabout and maintaining parallel lanes through the entry geometry; or
2. Widening the approach gradually (flaring) through the entry geometry.

Exhibit 6-24 and Exhibit 6-25 illustrate these two widening options.

Approach flaring may provide an effective means of increasing capacity without requiring as much right-of-way as a full lane addition. In addition, U.K. research suggests that length of flare affects capacity without a direct effect on safety. Although this research has not been replicated in the United States, the U.K. findings suggest that the crash frequency for two approaches with the same entry width will be identical whether they have parallel entry lanes or flared entry

Exhibit 6-24
Approach Widening by Adding a Full Lane

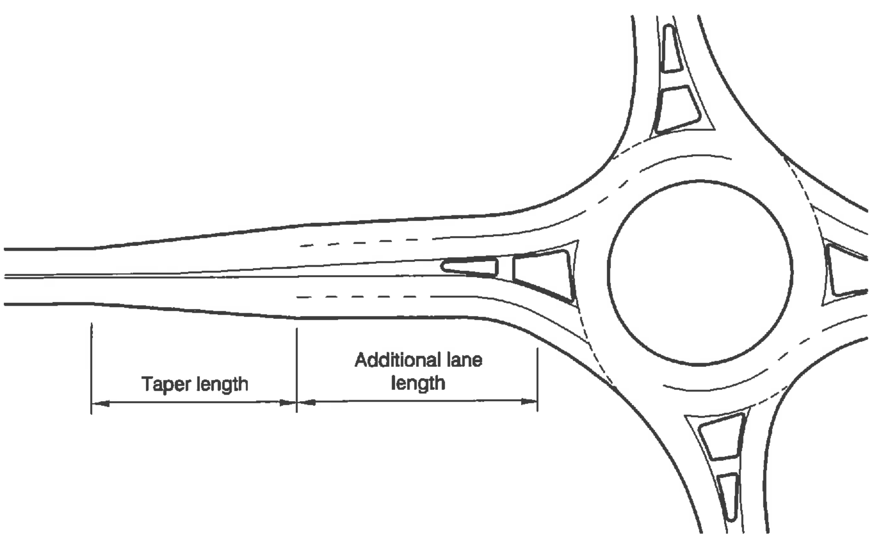
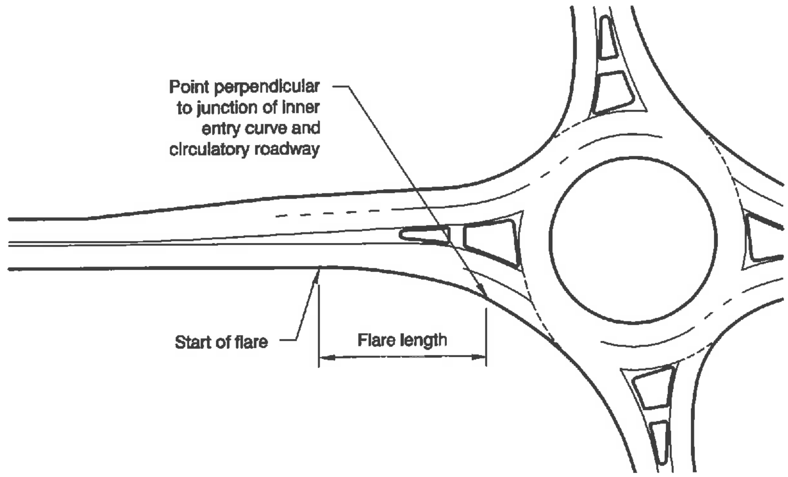


Exhibit 6-25
Approach Widening by Entry Flaring



designs. Entry widths should therefore be minimized and flare lengths maximized to achieve the desired capacity with minimal effect on crashes.

6.5.3 CIRCULATORY ROADWAY WIDTHS

The circulatory roadway width is usually governed by the design criteria relating to the types of vehicles that may need to be accommodated adjacent to one another through a multilane roundabout. The provision of pavement markings within the circulatory roadway (discussed in Chapter 7) may require extra space and the use of a truck apron to support lane discipline for trucks and cars circulating. The combination of vehicle types to be accommodated side-by-side is dependent upon the specific site traffic conditions, and requirements for side-by-side design vehicles may vary by individual state or local jurisdiction. Further research on this topic is underway at the time of this publication, and the reader is advised to look to the latest guidance for the conditions being explored.

If the entering traffic is predominantly passenger cars and single-unit trucks (AASHTO P and SU design vehicles, respectively), where semi-trailer traffic is infrequent, it may be appropriate to design the width for two passenger vehicles or a passenger car and a single-unit truck side-by-side. If semi-trailer traffic is relatively frequent (greater than 10%), it may be necessary to provide sufficient width for the simultaneous passage of a semi-trailer in combination with a P or SU vehicle.

Multilane circulatory roadway lane widths typically range from 14 to 16 ft (4.3 to 4.9 m). Use of these values results in a total circulating width of 28 to 32 ft (8.5 to 9.8 m) for a two-lane circulatory roadway and 42 to 48 ft (12.8 to 14.6 m) total width for a three-lane circulatory roadway.

At multilane roundabouts, the circulatory roadway width may also be variable depending upon the number of lanes and the design vehicle turning requirements. A constant width is not required throughout the entire circulatory roadway, and it is desirable to provide only the minimum width necessary to serve the required lane configurations within that specific portion of the roundabout. A common combination is two entering and exiting lanes along the major roadway, but only single entering and exiting lanes on the minor street. This combination is illustrated in Exhibit 6-26. In this example, the portion of circulatory roadway that serves the

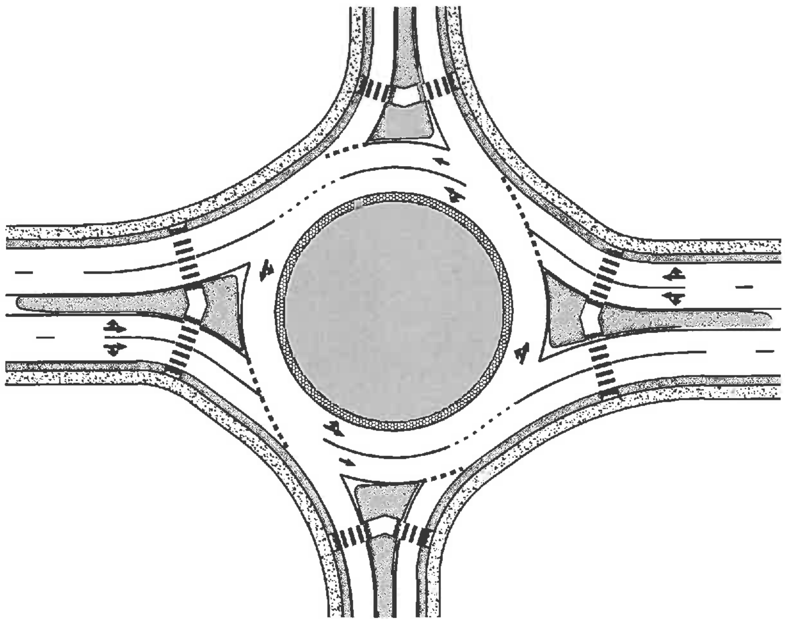


Exhibit 6-26
Multilane Major Street with Single Lane on Minor Street

minor street has been reduced to a single lane to provide consistency in the lane configurations. For the portions of a multilane roundabout where the circulatory roadway is reduced to a single lane, the guidance for circulatory roadway width contained in Section 6.4.3 should be used.

In some instances, the circulatory roadway width may actually need to be wider than the corresponding entrance that is feeding that portion of the roundabout. For example, in situations where two consecutive entries require exclusive left turns, a portion of the circulatory roadway will need to contain an extra lane and spiral markings to enable all vehicles to reach their intended exits without being trapped or changing lanes. This situation is illustrated in Exhibit 6-27,

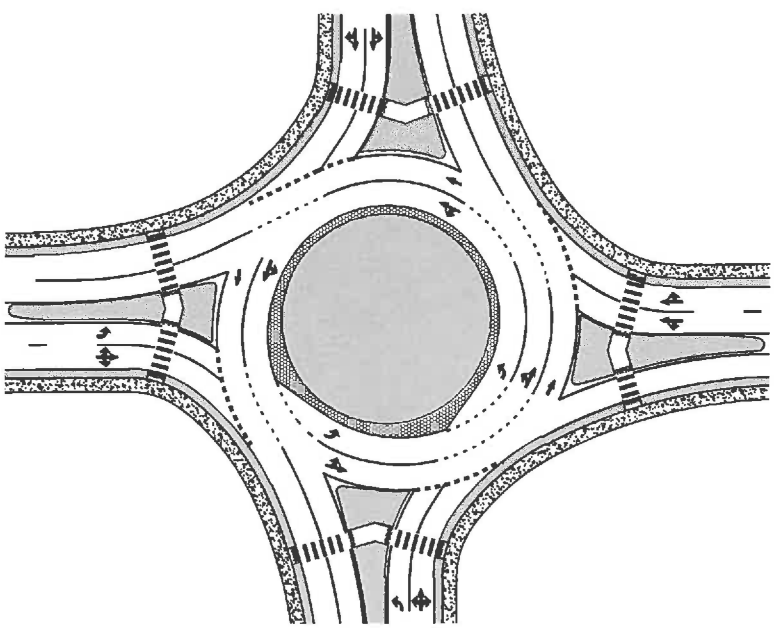


Exhibit 6-27
Two-Lane Roundabout with Consecutive Double-Lefts

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 29 OF 67

where a portion of the circulatory roadway is required to have three lanes despite the fact that all of the entries have only two lanes.

6.5.4 ENTRY GEOMETRY AND APPROACH ALIGNMENT

At multilane roundabouts, the design of the entry curvature should balance the competing objectives of speed control, adequate alignment of the natural paths, and the need for appropriate visibility lines. This often requires several iterations of design to identify the appropriate roundabout size, location, and approach alignments.

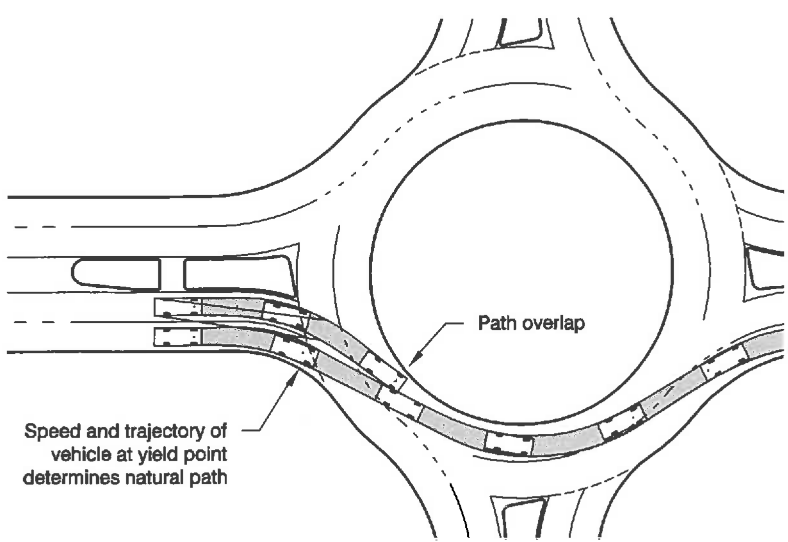
Individual geometric parameters also play a role in the balanced entry design. For example, entry radii are one key parameter that is often used to control vehicle speeds. The use of small entry radii may produce low entry speeds but often leads to path overlap on the entry since vehicles will cut across lanes to avoid running into the central island. Small entry radii may also result in an increase in single-vehicle crashes onto the central island.

Increasing vehicle path curvature decreases relative speeds between entering and circulating vehicles but also increases side friction between adjacent traffic streams in multilane roundabouts.

Entry radii for multilane roundabouts should typically exceed 65 ft (20 m) to encourage adequate natural paths and avoid sideswipe collisions on entry. Engineers should avoid the use of overly tight geometrics in order to achieve the fastest-path objectives. Overly small [less than 45 ft (13.7 m)] entry radii can result in conflicts between adjacent traffic streams, which may result in poor lane use and reduced capacity. Similarly, the R_1 fastest-path radius should also not be excessively small. If R_1 is too small, vehicle path overlap may result, reducing the operational efficiency and increasing potential for crashes. Values for R_1 in the range of 175 to 275 ft (53 to 84 m) are generally preferable. This results in a design speed of 25 to 30 mph (40 to 50 km/h).

Vehicle path overlap is a type of conflict that occurs when the natural path of the adjacent lanes cross one another. It occurs most commonly at entries, where the geometry of the right (outside) lane tends to lead vehicles into the left (inside) circulatory lane. However, vehicle path overlap can also occur at exits where the geometry tends to lead vehicles from the left-hand lane into the right-hand exit lane. Exhibit 6-28 illustrates an example of entry vehicle path overlap.

Exhibit 6-28
Entry Vehicle Path Overlap



ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 30 OF 67

The engineer should balance the need to control entry speed with the need to provide good path alignment at multilane entries. The desired result of the entry design is for vehicles to naturally be aligned into their correct lane within the circulatory roadway, as illustrated in Exhibit 6-29. This can be done a variety of ways that can vary significantly depending on site-specific conditions. Therefore, it may not be possible to specify a single method for designing multilane roundabouts since this can preclude the needed flexibility in design. Regardless of the specific design technique employed, the engineer should maintain the overall design principles of speed management presented in Section 6.2.

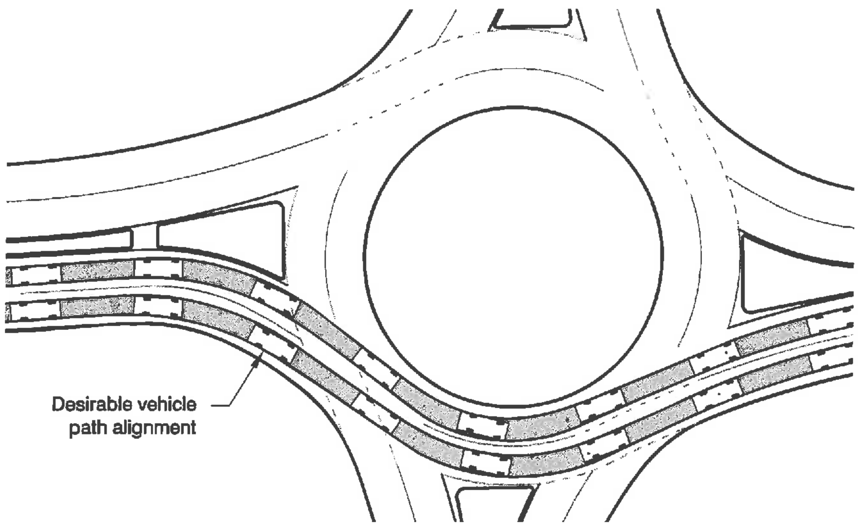


Exhibit 6-29
Desirable Vehicle Path Alignment

One possible technique to promote good path alignment is shown in Exhibit 6-30 using a compound curve or tangent along the outside curb. The design consists of an initial small-radius entry curve set back from the edge of the circulatory roadway. A short section of a large-radius curve or tangent is provided between the entry curve and the circulatory roadway to align vehicles into the proper circulatory lane at the entrance line. Care should be taken in determining the optimal location

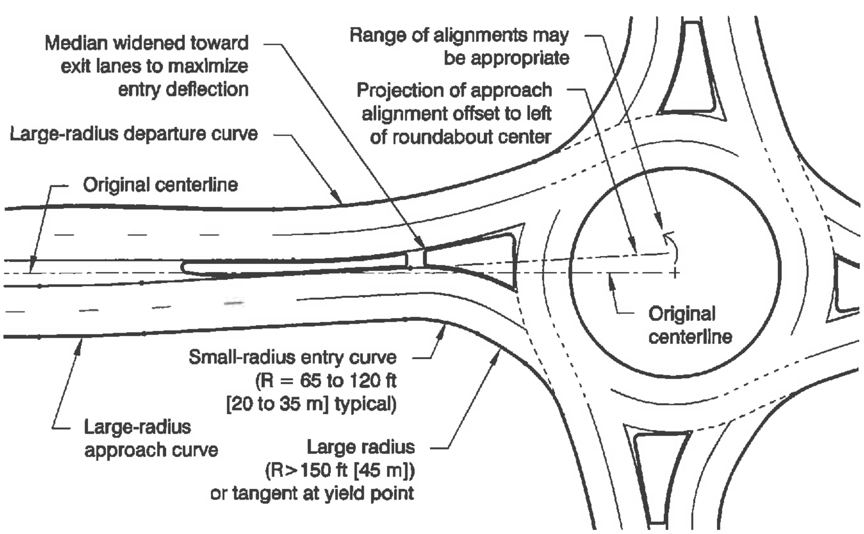


Exhibit 6-30
Example Minor Approach Offset to Increase Entry Deflection

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 31 OF 67

of the entry curve from the entrance line. If it is located too close to the circulatory roadway, the tangent (or large radius portion of the compound curve) will be too short, and the design may still have path alignment issues. However, if the entry curve is located too far away from the circulatory roadway, it can result in inadequate deflection (i.e., entry speeds too fast).

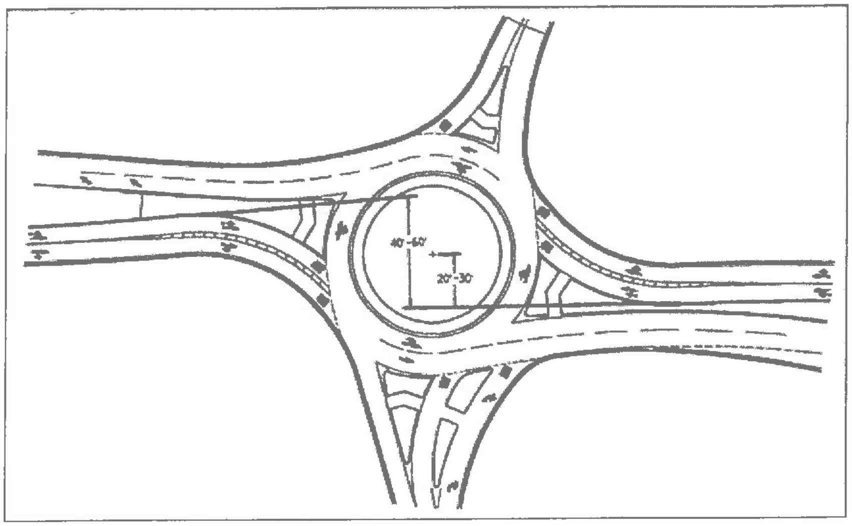
For the method illustrated in Exhibit 6-30, entry curve radii commonly range from approximately 65 to 120 ft (20 to 35 m) and are set back at least 20 ft (6 m) from the edge of the circulatory roadway. A tangent or large-radius [greater than 150 ft (45 m)] curve is then fitted between the entry curve and the outside edge of the circulatory roadway.

An alternative method for designing the entry curves to a multilane roundabout is to use a single-radius entry curve rather than a small curve and tangent. This is similar in some regards to a single-lane design; however, larger radii are typically required to provide adequate vehicle alignment. Care must be taken when using a single entry curve to meet both the speed control and vehicle natural path alignment objectives. If the circulatory roadway is sufficiently wide relative to the entry, entry curves can be designed tangential to a design circle offset 5 ft (1.5 m) from the central island rather than to the central island. This improves the curvature and deflection that is achieved on the inside (splitter island) edge of the entry. Regardless of the method used, it is desirable for the inside (splitter island) curb to block the through path of the left lane to promote adequate deflection.

Another key factor in multilane roundabout design is to recognize that achieving adequate deflection on entry and meeting the principles is independent of the centerline of the approaching roadways. As discussed in Section 6.3, the centerlines of approach roadways do not need to pass through the center of the inscribed circle. It is acceptable design practice for multilane roundabouts to have an offset-left alignment, and in many cases this may provide a useful tool for achieving additional deflection and speed control.

Exhibit 6-31 illustrates an example of a design technique to enhance the entry deflection by shifting the approach alignment further toward the left of the roundabout center. This technique of offsetting the approach alignment left of the roundabout center is effective at increasing entry deflection. However, it also reduces the deflection of the exit on the same leg, where it is desirable to keep speeds relatively low within the pedestrian crosswalk location. Therefore, the distance of the approach offset from the roundabout center should be balanced with the other design objectives to maximize safety for pedestrians. Exhibit 6-32 illustrates an example of this technique being applied for a partial three-lane roundabout.

Other important components of the design of an entry are sight distance and visibility, as discussed in Section 6.2.6. The angle of visibility to the left must be adequate for entering drivers to comfortably view oncoming traffic from the immediate upstream entry or from the circulatory roadway. This requires that the vehicles be staggered at the entrance line such that vehicles nearest to the outside curb can see in front of the vehicle in the adjacent lane to the left of them. The design of the entry must balance the design objective of providing speed control with providing appropriate angles of visibility for drivers. Additional details on measuring angles of visibility are provided in Section 6.7.4.



Source: Wisconsin Department of Transportation (*7*)

Exhibit 6-31
Example of Major Approach Offset to Increase Entry Deflection

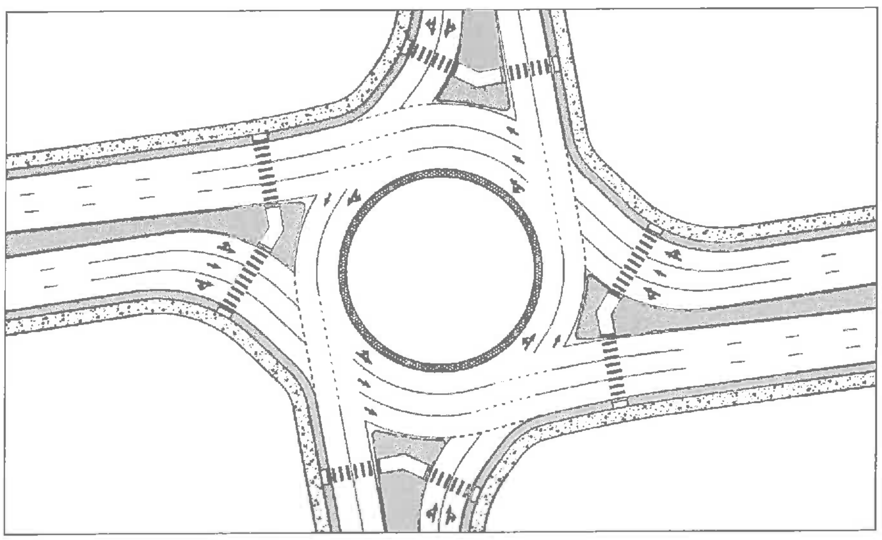


Exhibit 6-32
Example of a Partial Three Lane Roundabout with an Offset Approach Alignment

As discussed previously for single-lane roundabouts, a useful surrogate for capturing the effects of entry speed, path alignment, and visibility to the left is entry angle (phi). Typical entry angles are between 20° and 40°. Additional detail on entry angle can be found in the Wisconsin Department of Transportation *Roundabout Guide* (*7*) and design guidance from the United Kingdom (*9, 10*).

6.5.5 SPLITTER ISLANDS

For multilane roundabouts, the entry geometry is typically established first to identify a design that adequately controls fastest-path entry speeds, avoids entry path overlap, and accommodates the design vehicle. The splitter island is then developed in conjunction with the exit design to provide an adequate median width for the pedestrian refuge and for sign placement. Adequate median width should be provided to accommodate necessary equipment and pedestrian design elements where signalized pedestrian crossings are used. Additional details

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 33 OF 67

regarding the minimum dimensions and design details for splitter islands are provided under the discussion of single-lane roundabouts in Section 6.4.1. Additional discussion of pedestrian crosswalk design is provided in Section 6.8.1 and considerations for signalized pedestrian crossing are discussed in Chapter 7.

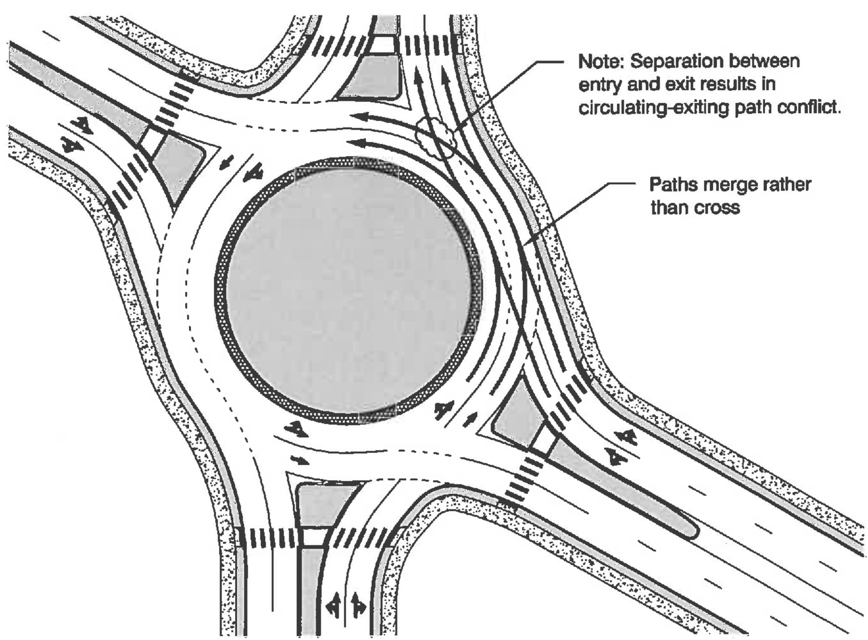
6.5.6 EXIT CURVES

As with the entries, the design of the exit curvature at multilane roundabouts is more complex than at single-lane roundabouts. Conflicts can occur between exiting and circulating vehicles if appropriate lane assignments are not provided. Inadequate horizontal design of the exits can also result in exit vehicle path overlap, similar to that occurring at entries. The radii of exit curves are commonly larger than those used at the entry as a consequence of other factors (entry alignment, diameter, etc.); larger exit curve radii are also typically used to promote good vehicle path alignment. However, the design should be balanced to maintain low speeds at the pedestrian crossing at the exit.

To promote good path alignment at the exit, the exit radius at a multilane roundabout should not be too small. At single-lane roundabouts, it is acceptable to use a minimal exit radius in order to control exit speeds and maximize pedestrian safety. However, if the exit radius on a multilane exit is too small, traffic on the inside of the circulatory roadway will tend to exit into the outside exit lane on a more comfortable turning radius.

Problems can also occur when the design allows for too much separation between entries and subsequent exits. Large separations between legs causes entering vehicles to join next to circulating traffic that may be intending to exit at the next leg, rather than crossing the path of the exiting vehicles. This can create conflicts at the exit point between exiting and circulating vehicles, as shown in Exhibit 6-33.

Exhibit 6-33
Exit–Circulating Conflict Caused by Large Separation between Legs

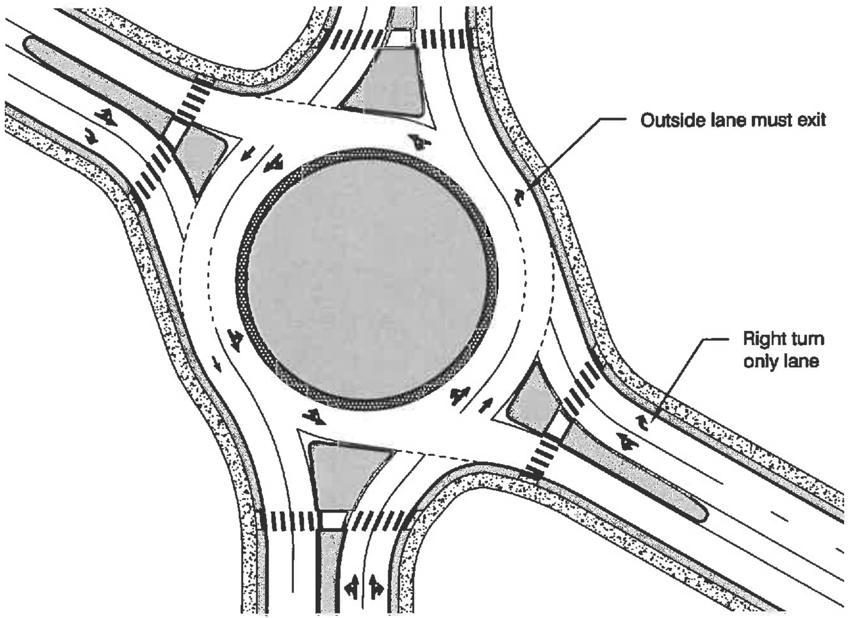


Source: California Department of Transportation (*1*)

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 34 OF 67

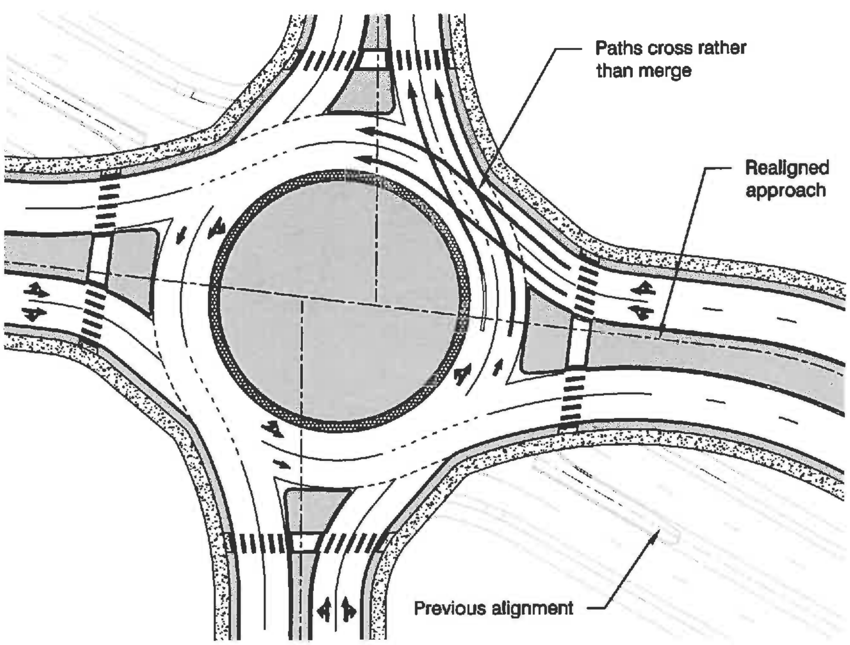
Exhibit 6-34 illustrates a possible low-cost fix that involves modifications to the lane arrangements using a combination of striping and physical modifications. This may be acceptable if the traffic volumes are compatible. A better solution is illustrated in Exhibit 6-35, which involves realignment of the approach legs to have the paths of entering vehicles cross the paths of the circulating traffic (rather than merging) to eliminate the conflict.

Exhibit 6-34
Possible Lane Configuration Modifications to Resolve Exit-Circulating Conflicts



Source: California Department of Transportation (*1*)

Exhibit 6-35
Realignment to Resolve Exit-Circulating Conflicts



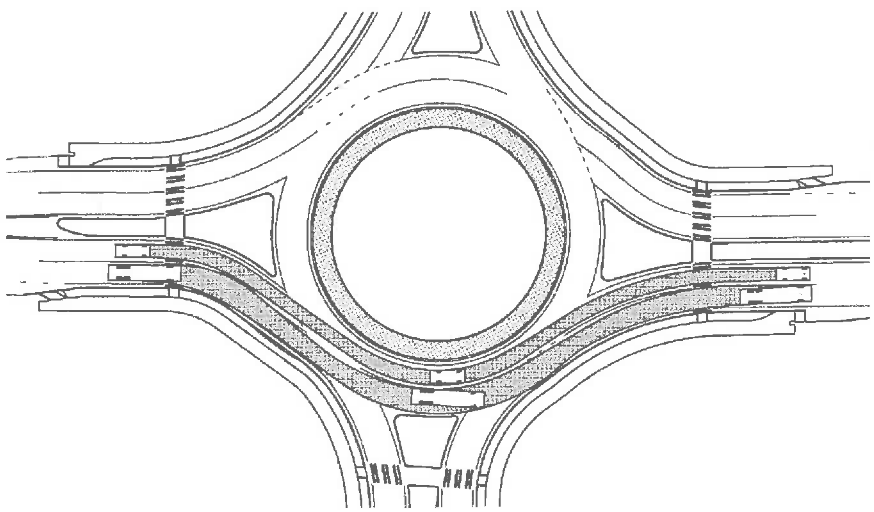
Source: California Department of Transportation (*1*)

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 35 OF 67

6.5.7 DESIGN VEHICLE CONSIDERATIONS

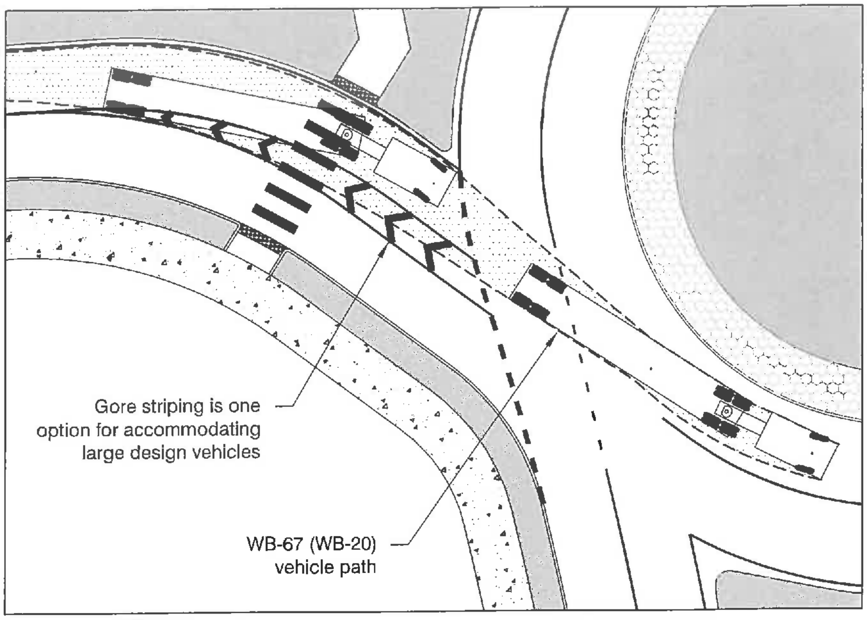
Design vehicle considerations should be made for both tracking on the entry/exit and within the circulatory roadway (as previously discussed in Section 6.5.3). The percentage of trucks and lane utilization is an important consideration when determining whether the design will allow trucks to use two lanes or accommodate them to stay within their own lane. The frequency of a particular design vehicle is also an important consideration. For instance, a particular roundabout may have infrequent use by WB-67-size tractor-trailers and is thus designed to allow the WB-67 to claim both lanes to navigate through. However, the same location could have frequent bus service that would dictate the need to accommodate buses within their own lane to travel adjacent to a passenger car (see Exhibit 6-36). Therefore, a particular roundabout may have multiple design vehicles depending upon the unique site characteristics.

Exhibit 6-36
Side-by-Side Navigation for a Bus and Passenger Car



Where the design dictates the need to accommodate large design vehicles within their own lane, there are a number of design considerations that come into play. A larger inscribed circle diameter and entry/exit radii may be required to maintain speed control and accommodate the design vehicle. A technique that has been used in the United States on the entry is to provide gore striping—a striped vane island between the entry lanes—to help center the vehicles within the lane and allow a cushion for off-tracking by the design vehicle. This technique is illustrated in Exhibit 6-37. The actual dimensions used may vary depending on the individual design; however, one state (*11*) identified the use of two 12 ft (3.6 m) lanes and a 6 ft (1.8 m) wide gore area for an entrance with a total width of 30 ft (9 m).

Another technique for accommodating the design vehicle within the circulatory roadway is to use a wider lane width for the outside lane and a narrower lane width for the inside lane. For example, for a 32 ft (9.8 m) circulatory roadway width, an inside width of 15 ft (4.6 m) and an outside width of 17 ft (5.2 m) could be used. This would provide an extra two feet of circulating width for trucks in the outside lane. Large trucks in the inside lane would use the truck apron to accommodate any off tracking. Eliminating all overlap for the outside lane may not always be desirable or feasible, as this may dictate a much larger inscribed circle diameter than desired for overall safety performance for all vehicle types and the context.



Source: New York State Department of Transportation (*11*)

Exhibit 6-37
WB-67 (WB-20) Truck Path with Gore Striping at Entry

6.5.8 OTHER DESIGN PRACTICES

Throughout the world there continues to be advancement in the design practices for multilane roundabouts. One practice initiated in the Netherlands and being tested elsewhere is the turbo-roundabout (*13*). This style of multilane design has two key features that distinguish it from other multilane roundabouts:

- Entries are perpendicular to the circulatory roadway, and
- Raised lane dividers are used within the circulatory roadway to guide drivers to the appropriate exit.

This treatment has not been used in the United States at the time of this writing.

6.6 MINI-ROUNDABOUTS

A mini-roundabout is an intersection design form that can be used in place of stop control or signalization at physically constrained intersections to help improve safety and reduce delays. Typically characterized by a small diameter and traversable islands, mini-roundabouts are best suited to environments where speeds are already low and environmental constraints would preclude the use of a larger roundabout with a raised central island. Exhibit 6-38 presents the characteristics of a mini-roundabout.

Mini-roundabouts operate in the same manner as larger roundabouts, with yield control on all entries and counterclockwise circulation around a central island. Due to the small footprint, large vehicles are typically required to travel over the fully traversable central island, as shown in Exhibit 6-38. To help promote safe operations, the design generally aligns passenger cars in such a way as

ON FILE
R. G. SKINNER PARKWAY EXTENSION MOBILITY SCORE ANALYSIS
PAGE 37 OF 67

Appendix B

SYNCRO Reports
2035 AM Peak Hour Traffic

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↱		↰	↱		↰	↑↑	↱	↰↰	↑↑	↱
Volume (vph)	110	2	2	2	2	80	2	166	2	379	269	248
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0		4.0	4.0		4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	1.00		1.00	1.00		1.00	0.95	1.00	0.97	0.95	1.00
Frt	1.00	0.93		1.00	0.85		1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	1723		1770	1590		1770	3539	1583	3433	3539	1583
Flt Permitted	0.70	1.00		0.76	1.00		0.57	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1303	1723		1407	1590		1068	3539	1583	3433	3539	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	120	2	2	2	2	87	2	180	2	412	292	270
RTOR Reduction (vph)	0	2	0	0	74	0	0	0	1	0	0	71
Lane Group Flow (vph)	120	2	0	2	15	0	2	180	1	412	292	199
Turn Type	Perm			Perm			Perm		Perm	Prot		Perm
Protected Phases		4			8			2		1	6	
Permitted Phases	4			8			2		2			6
Actuated Green, G (s)	10.4	10.4		10.4	10.4		29.6	29.6	29.6	18.0	51.6	51.6
Effective Green, g (s)	10.4	10.4		10.4	10.4		29.6	29.6	29.6	18.0	51.6	51.6
Actuated g/C Ratio	0.15	0.15		0.15	0.15		0.42	0.42	0.42	0.26	0.74	0.74
Clearance Time (s)	4.0	4.0		4.0	4.0		4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	194	256		209	236		452	1496	669	883	2609	1167
v/s Ratio Prot		0.00			0.01			0.05		c0.12	0.08	
v/s Ratio Perm	c0.09			0.00			0.00		0.00			c0.13
v/c Ratio	0.62	0.01		0.01	0.06		0.00	0.12	0.00	0.47	0.11	0.17
Uniform Delay, d1	27.9	25.4		25.4	25.6		11.7	12.3	11.7	21.9	2.6	2.8
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00	1.00	0.73	0.40	0.10
Incremental Delay, d2	5.8	0.0		0.0	0.1		0.0	0.2	0.0	0.4	0.1	0.3
Delay (s)	33.7	25.4		25.4	25.7		11.7	12.4	11.7	16.5	1.1	0.6
Level of Service	C	C		C	C		B	B	B	B	A	A
Approach Delay (s)		33.4			25.7			12.4			7.5	
Approach LOS		C			C			B			A	

Intersection Summary			
HCM Average Control Delay	11.7	HCM Level of Service	B
HCM Volume to Capacity ratio	0.35		
Actuated Cycle Length (s)	70.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	38.2%	ICU Level of Service	A
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↱						↑↑	↱	↰↰	↑↑	
Volume (vph)	200	5	220	0	0	0	0	132	224	896	712	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0						4.0	4.0	4.0	4.0	
Lane Util. Factor	1.00	1.00						0.95	1.00	0.97	0.95	
Frt	1.00	0.85						1.00	0.85	1.00	1.00	
Flt Protected	0.95	1.00						1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1770	1589						3539	1583	3433	3539	
Flt Permitted	0.95	1.00						1.00	1.00	0.95	1.00	
Satd. Flow (perm)	1770	1589						3539	1583	3433	3539	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	217	5	239	0	0	0	0	143	243	974	774	0
RTOR Reduction (vph)	0	181	0	0	0	0	0	0	187	0	0	0
Lane Group Flow (vph)	217	63	0	0	0	0	0	143	56	974	774	0
Turn Type	Perm								Perm	Prot		
Protected Phases		4						2		1	6	
Permitted Phases	4								2			
Actuated Green, G (s)	16.0	16.0						16.0	16.0	26.0	46.0	
Effective Green, g (s)	16.0	16.0						16.0	16.0	26.0	46.0	
Actuated g/C Ratio	0.23	0.23						0.23	0.23	0.37	0.66	
Clearance Time (s)	4.0	4.0						4.0	4.0	4.0	4.0	
Lane Grp Cap (vph)	405	363						809	362	1275	2326	
v/s Ratio Prot		0.04						0.04		c0.28	c0.22	
v/s Ratio Perm	c0.12								0.04			
v/c Ratio	0.54	0.17						0.18	0.15	0.76	0.33	
Uniform Delay, d1	23.7	21.7						21.7	21.6	19.3	5.3	
Progression Factor	1.00	1.00						0.59	1.22	0.69	0.25	
Incremental Delay, d2	5.0	1.0						0.5	0.9	3.1	0.3	
Delay (s)	28.7	22.7						13.3	27.2	16.4	1.6	
Level of Service	C	C						B	C	B	A	
Approach Delay (s)		25.6			0.0			22.0			9.9	
Approach LOS		C			A			C			A	

Intersection Summary			
HCM Average Control Delay	14.5	HCM Level of Service	B
HCM Volume to Capacity ratio	0.56		
Actuated Cycle Length (s)	70.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	101.9%	ICU Level of Service	G
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations				↰↰	↱		↰	↑↑			↑↑	↱
Volume (vph)	0	0	0	486	5	874	118	214	0	0	1122	402
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)				4.0	4.0		4.0	4.0			4.0	4.0
Lane Util. Factor				0.97	1.00		1.00	0.95			0.95	1.00
Frt				1.00	0.85		1.00	1.00			1.00	0.85
Flt Protected				0.95	1.00		0.95	1.00			1.00	1.00
Satd. Flow (prot)				3433	1585		1770	3539			3539	1583
Flt Permitted				0.95	1.00		0.12	1.00			1.00	1.00
Satd. Flow (perm)				3433	1585		230	3539			3539	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	0	0	528	5	950	128	233	0	0	1220	437
RTOR Reduction (vph)	0	0	0	0	374	0	0	0	0	0	0	237
Lane Group Flow (vph)	0	0	0	528	581	0	128	233	0	0	1220	200
Turn Type				Prot			pm+pt					Perm
Protected Phases				3	8		5	2			6	
Permitted Phases							2					6
Actuated Green, G (s)				26.4	26.4		35.6	35.6			28.4	28.4
Effective Green, g (s)				26.4	26.4		35.6	35.6			28.4	28.4
Actuated g/C Ratio				0.38	0.38		0.51	0.51			0.41	0.41
Clearance Time (s)				4.0	4.0		4.0	4.0			4.0	4.0
Vehicle Extension (s)				3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)				1295	598		187	1800			1436	642
v/s Ratio Prot				0.15	c0.37		c0.03	0.07			c0.34	
v/s Ratio Perm							0.32					0.13
v/c Ratio				0.41	0.97		0.68	0.13			0.85	0.31
Uniform Delay, d1				16.0	21.4		13.6	9.0			18.9	14.1
Progression Factor				1.00	1.00		1.24	0.23			0.72	0.28
Incremental Delay, d2				0.2	29.5		9.3	0.1			5.3	1.0
Delay (s)				16.3	50.9		26.3	2.2			18.8	4.9
Level of Service				B	D		C	A			B	A
Approach Delay (s)		0.0			38.6			10.8			15.2	
Approach LOS		A			D			B			B	

Intersection Summary			
HCM Average Control Delay	24.6	HCM Level of Service	C
HCM Volume to Capacity ratio	0.90		
Actuated Cycle Length (s)	70.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	101.9%	ICU Level of Service	G
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	30	1094	16	139	678	271	13	5	116	314	5	35
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1770	1863	1583	1770	1863	1583
Flt Permitted	0.34	1.00	1.00	0.18	1.00	1.00	0.75	1.00	1.00	0.75	1.00	1.00
Satd. Flow (perm)	637	3539	1583	335	3539	1583	1405	1863	1583	1405	1863	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	33	1189	17	151	737	295	14	5	126	341	5	38
RTOR Reduction (vph)	0	0	7	0	0	114	0	0	60	0	0	28
Lane Group Flow (vph)	33	1189	10	151	737	181	14	5	66	341	5	10
Turn Type	Perm		Perm	Perm		Perm	Perm		Perm	Perm		Perm
Protected Phases		4			8			2			6	
Permitted Phases	4		4	8		8	2		2	6		6
Actuated Green, G (s)	43.0	43.0	43.0	43.0	43.0	43.0	19.0	19.0	19.0	19.0	19.0	19.0
Effective Green, g (s)	43.0	43.0	43.0	43.0	43.0	43.0	19.0	19.0	19.0	19.0	19.0	19.0
Actuated g/C Ratio	0.61	0.61	0.61	0.61	0.61	0.61	0.27	0.27	0.27	0.27	0.27	0.27
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	391	2174	972	206	2174	972	381	506	430	381	506	430
v/s Ratio Prot		0.34			0.21			0.00			0.00	
v/s Ratio Perm	0.05		0.01	c0.45		0.11	0.01		0.04	c0.24		0.01
v/c Ratio	0.08	0.55	0.01	0.73	0.34	0.19	0.04	0.01	0.15	0.90	0.01	0.02
Uniform Delay, d1	5.5	7.8	5.2	9.5	6.6	5.9	18.8	18.6	19.4	24.5	18.6	18.7
Progression Factor	1.00	1.00	1.00	0.84	0.97	1.07	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	0.4	1.0	0.0	12.2	0.2	0.2	0.2	0.0	0.8	26.0	0.0	0.1
Delay (s)	5.9	8.8	5.3	20.2	6.6	6.5	18.9	18.7	20.1	50.5	18.7	18.8
Level of Service	A	A	A	C	A	A	B	B	C	D	B	B
Approach Delay (s)		8.7			8.3			20.0			47.0	
Approach LOS		A			A			B			D	

Intersection Summary			
HCM Average Control Delay	14.1	HCM Level of Service	B
HCM Volume to Capacity ratio	0.78		
Actuated Cycle Length (s)	70.0	Sum of lost time (s)	8.0
Intersection Capacity Utilization	72.0%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Right Turn Channelized												
Volume (veh/h)	5	755	17	151	616	41	31	5	297	88	5	10
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	5	821	18	164	670	45	34	5	323	96	5	11
Approach Volume (veh/h)		845			878			362			112	
Crossing Volume (veh/h)		265			45			922			867	
High Capacity (veh/h)		1125			1337			664			694	
High v/c (veh/h)		0.75			0.66			0.55			0.16	
Low Capacity (veh/h)		926			1118			518			544	
Low v/c (veh/h)		0.91			0.79			0.70			0.21	

Intersection Summary			
Maximum v/c High	0.75		
Maximum v/c Low	0.91		
Intersection Capacity Utilization	83.7%	ICU Level of Service	E

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (veh/h)	8	687	16	29	612	16	31	2	58	32	2	16
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	9	747	17	32	665	17	34	2	63	35	2	17
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	683			764			1511	1510	747	1557	1510	665
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	683			764			1511	1510	747	1557	1510	665
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	99			96			63	98	85	53	98	96
cM capacity (veh/h)	910			849			90	115	413	74	115	460

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	9	747	17	32	665	17	34	65	35	20
Volume Left	9	0	0	32	0	0	34	0	35	0
Volume Right	0	0	17	0	0	17	0	63	0	17
cSH	910	1700	1700	849	1700	1700	90	380	74	345
Volume to Capacity	0.01	0.44	0.01	0.04	0.39	0.01	0.37	0.17	0.47	0.06
Queue Length 95th (ft)	1	0	0	3	0	0	37	15	48	4
Control Delay (s)	9.0	0.0	0.0	9.4	0.0	0.0	66.9	16.4	91.2	16.1
Lane LOS	A			A			F	C	F	C
Approach Delay (s)	0.1			0.4			33.6		64.2	
Approach LOS							D		F	

Intersection Summary			
Average Delay	4.4		
Intersection Capacity Utilization	51.3%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	SEL	SET	SER	NWL	NWT	NWR	NEL	NET	NER	SWL	SWT	SWR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↱		↰	↱	
Volume (veh/h)	6	624	27	41	605	13	45	2	68	19	2	13
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	7	678	29	45	658	14	49	2	74	21	2	14
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	672			708			1453	1452	678	1513	1467	658
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	672			708			1453	1452	678	1513	1467	658
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	99			95			51	98	84	73	98	97
cM capacity (veh/h)	919			891			99	123	452	78	120	464

Direction, Lane #	SE 1	SE 2	SE 3	NW 1	NW 2	NW 3	NE 1	NE 2	SW 1	SW 2
Volume Total	7	678	29	45	658	14	49	76	21	16
Volume Left	7	0	0	45	0	0	49	0	21	0
Volume Right	0	0	29	0	0	14	0	74	0	14
cSH	919	1700	1700	891	1700	1700	99	420	78	336
Volume to Capacity	0.01	0.40	0.02	0.05	0.39	0.01	0.49	0.18	0.27	0.05
Queue Length 95th (ft)	1	0	0	4	0	0	54	16	24	4
Control Delay (s)	8.9	0.0	0.0	9.3	0.0	0.0	72.6	15.5	67.4	16.2
Lane LOS	A			A			F	C	F	C
Approach Delay (s)	0.1			0.6			37.8		44.8	
Approach LOS							E		E	

Intersection Summary			
Average Delay	4.3		
Intersection Capacity Utilization	49.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑	↱	↰	↑	↰	↱
Volume (veh/h)	647	5	3	660	14	10
Sign Control	Free			Free	Stop	
Grade	0%			0%	0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	703	5	3	717	15	11
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)						
pX, platoon unblocked						
vC, conflicting volume			709		1427	703
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			709		1427	703
tC, single (s)			4.1		6.4	6.2
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		90	98
cM capacity (veh/h)			890		148	437

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	NB 1	NB 2
Volume Total	703	5	3	717	15	11
Volume Left	0	0	3	0	15	0
Volume Right	0	5	0	0	0	11
cSH	1700	1700	890	1700	148	437
Volume to Capacity	0.41	0.00	0.00	0.42	0.10	0.02
Queue Length 95th (ft)	0	0	0	0	8	2
Control Delay (s)	0.0	0.0	9.1	0.0	32.0	13.4
Lane LOS			A		D	B
Approach Delay (s)	0.0		0.0		24.3	
Approach LOS					C	

Intersection Summary			
Average Delay	0.5		
Intersection Capacity Utilization	44.7%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations	↰	↑	↑	↱	↰	↱
Volume (veh/h)	15	644	664	10	8	12
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	16	700	722	11	9	13
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)			388			
pX, platoon unblocked						
vC, conflicting volume	733				1454	722
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	733				1454	722
tC, single (s)	4.1				6.4	6.2
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	98				94	97
cM capacity (veh/h)	872				141	427

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	SB 1	SB 2
Volume Total	16	700	722	11	9	13
Volume Left	16	0	0	0	9	0
Volume Right	0	0	0	11	0	13
cSH	872	1700	1700	1700	141	427
Volume to Capacity	0.02	0.41	0.42	0.01	0.06	0.03
Queue Length 95th (ft)	1	0	0	0	5	2
Control Delay (s)	9.2	0.0	0.0	0.0	32.3	13.7
Lane LOS	A				D	B
Approach Delay (s)	0.2		0.0		21.1	
Approach LOS					C	

Intersection Summary			
Average Delay	0.4		
Intersection Capacity Utilization	44.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Right Turn Channelized												
Volume (veh/h)	29	617	9	6	657	13	21	2	14	28	2	65
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	32	671	10	7	714	14	23	2	15	30	2	71
Approach Volume (veh/h)		712			735			40			103	
Crossing Volume (veh/h)		39			57			733			743	
High Capacity (veh/h)		1343			1325			774			767	
High v/c (veh/h)		0.53			0.55			0.05			0.13	
Low Capacity (veh/h)		1123			1107			614			608	
Low v/c (veh/h)		0.63			0.66			0.07			0.17	

Intersection Summary			
Maximum v/c High	0.55		
Maximum v/c Low	0.66		
Intersection Capacity Utilization	64.4%	ICU Level of Service	C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↑↱		↰	↑↱	
Volume (veh/h)	8	624	29	13	726	4	57	2	24	7	2	18
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	9	678	32	14	789	4	62	2	26	8	2	20
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	793			710			1534	1517	678	1540	1545	789
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	793			710			1534	1517	678	1540	1545	789
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	99			98			29	98	94	91	98	95
cM capacity (veh/h)	828			889			87	116	452	86	112	391

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	9	678	32	14	789	4	62	28	8	22
Volume Left	9	0	0	14	0	0	62	0	8	0
Volume Right	0	0	32	0	0	4	0	26	0	20
cSH	828	1700	1700	889	1700	1700	87	370	86	312
Volume to Capacity	0.01	0.40	0.02	0.02	0.46	0.00	0.71	0.08	0.09	0.07
Queue Length 95th (ft)	1	0	0	1	0	0	87	6	7	6
Control Delay (s)	9.4	0.0	0.0	9.1	0.0	0.0	113.2	15.5	51.1	17.4
Lane LOS	A			A			F	C	F	C
Approach Delay (s)	0.1			0.2			82.6		26.1	
Approach LOS							F		D	

Intersection Summary			
Average Delay	5.1		
Intersection Capacity Utilization	54.7%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↑↱		↰	↑↱	
Volume (veh/h)	12	622	26	17	781	3	48	5	32	7	5	28
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	13	676	28	18	849	3	52	5	35	8	5	30
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	852			704			1621	1591	676	1626	1616	849
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	852			704			1621	1591	676	1626	1616	849
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	98			98			26	95	92	89	95	92
cM capacity (veh/h)	787			893			70	103	453	71	100	361

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	13	676	28	18	849	3	52	40	8	36
Volume Left	13	0	0	18	0	0	52	0	8	0
Volume Right	0	0	28	0	0	3	0	35	0	30
cSH	787	1700	1700	893	1700	1700	70	311	71	258
Volume to Capacity	0.02	0.40	0.02	0.02	0.50	0.00	0.74	0.13	0.11	0.14
Queue Length 95th (ft)	1	0	0	2	0	0	86	11	9	12
Control Delay (s)	9.7	0.0	0.0	9.1	0.0	0.0	140.6	18.3	62.0	21.2
Lane LOS	A			A			F	C	F	C
Approach Delay (s)	0.2			0.2			87.4		28.3	
Approach LOS							F		D	

Intersection Summary			
Average Delay	5.6		
Intersection Capacity Utilization	57.1%	ICU Level of Service	B
Analysis Period (min)	15		

Movement	EBL	EBT	WBT	WBR	SEL	SER
Lane Configurations						
Volume (veh/h)	438	522	564	293	138	206
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	476	567	613	318	150	224
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)						
pX, platoon unblocked						
vC, conflicting volume	932				2133	613
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	932				2133	613
tC, single (s)	4.1				6.4	6.2
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	35				0	55
cM capacity (veh/h)	735				19	492

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	SE 1	SE 2
Volume Total	476	567	613	318	150	224
Volume Left	476	0	0	0	150	0
Volume Right	0	0	0	318	0	224
cSH	735	1700	1700	1700	19	492
Volume to Capacity	0.65	0.33	0.36	0.19	7.85	0.45
Queue Length 95th (ft)	120	0	0	0	Err	58
Control Delay (s)	18.5	0.0	0.0	0.0	Err	18.3
Lane LOS	C				F	C
Approach Delay (s)	8.4		0.0		4022.2	
Approach LOS					F	

Intersection Summary			
Average Delay	644.0		
Intersection Capacity Utilization	71.6%	ICU Level of Service	C
Analysis Period (min)	15		

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Volume (veh/h)	66	8	952	16	2	768
Sign Control	Stop		Free			Free
Grade	0%		0%			0%
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	72	9	1035	17	2	835
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type			None			None
Median storage veh)						
Upstream signal (ft)						590
pX, platoon unblocked						
vC, conflicting volume	1874	1035			1052	
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	1874	1035			1052	
tC, single (s)	6.4	6.2			4.1	
tC, 2 stage (s)						
tF (s)	3.5	3.3			2.2	
p0 queue free %	9	97			100	
cM capacity (veh/h)	79	282			662	

Direction, Lane #	WB 1	WB 2	NB 1	NB 2	SB 1	SB 2
Volume Total	72	9	1035	17	2	835
Volume Left	72	0	0	0	2	0
Volume Right	0	9	0	17	0	0
cSH	79	282	1700	1700	662	1700
Volume to Capacity	0.91	0.03	0.61	0.01	0.00	0.49
Queue Length 95th (ft)	119	2	0	0	0	0
Control Delay (s)	169.4	18.2	0.0	0.0	10.5	0.0
Lane LOS	F	C			B	
Approach Delay (s)	153.0		0.0		0.0	
Approach LOS	F					

Intersection Summary			
Average Delay	6.3		
Intersection Capacity Utilization	60.4%	ICU Level of Service	B
Analysis Period (min)	15		

Appendix C

SYNCRO Reports
2035 PM Peak Hour Traffic

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	274	2	2	2	2	375	2	962	2	135	887	144
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0		4.0	4.0		4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	1.00		1.00	1.00		1.00	0.95	1.00	0.97	0.95	1.00
Frt	1.00	0.93		1.00	0.85		1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	1723		1770	1585		1770	3539	1583	3433	3539	1583
Flt Permitted	0.38	1.00		0.76	1.00		0.25	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	705	1723		1407	1585		475	3539	1583	3433	3539	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	298	2	2	2	2	408	2	1046	2	147	964	157
RTOR Reduction (vph)	0	1	0	0	95	0	0	0	1	0	0	81
Lane Group Flow (vph)	298	3	0	2	315	0	2	1046	1	147	964	76
Turn Type	Perm			Perm			Perm		Perm	Prot		Perm
Protected Phases		4			8			2		1	6	
Permitted Phases	4			8			2		2			6
Actuated Green, G (s)	39.6	39.6		39.6	39.6		32.3	32.3	32.3	6.1	42.4	42.4
Effective Green, g (s)	39.6	39.6		39.6	39.6		32.3	32.3	32.3	6.1	42.4	42.4
Actuated g/C Ratio	0.44	0.44		0.44	0.44		0.36	0.36	0.36	0.07	0.47	0.47
Clearance Time (s)	4.0	4.0		4.0	4.0		4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	310	758		619	697		170	1270	568	233	1667	746
v/s Ratio Prot		0.00			0.20			c0.30		0.04	c0.27	
v/s Ratio Perm	c0.42			0.00			0.00		0.00			0.05
v/c Ratio	0.96	0.00		0.00	0.45		0.01	0.82	0.00	0.63	0.58	0.10
Uniform Delay, d1	24.5	14.1		14.1	17.6		18.6	26.3	18.5	40.9	17.3	13.2
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00	1.00	1.15	0.64	0.73
Incremental Delay, d2	40.6	0.0		0.0	0.5		0.1	6.1	0.0	5.0	1.3	0.3
Delay (s)	65.0	14.1		14.1	18.1		18.7	32.4	18.5	52.0	12.4	9.9
Level of Service	E	B		B	B		B	C	B	D	B	A
Approach Delay (s)		64.3			18.1			32.3			16.7	
Approach LOS		E			B			C			B	

Intersection Summary			
HCM Average Control Delay	27.0	HCM Level of Service	C
HCM Volume to Capacity ratio	0.89		
Actuated Cycle Length (s)	90.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	82.3%	ICU Level of Service	E
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↱						↑↑	↱	↰↰	↑↑	
Volume (vph)	305	5	215	0	0	0	0	1061	564	796	951	0
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0						4.0	4.0	4.0	4.0	
Lane Util. Factor	1.00	1.00						0.95	1.00	0.97	0.95	
Frt	1.00	0.85						1.00	0.85	1.00	1.00	
Flt Protected	0.95	1.00						1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1770	1589						3539	1583	3433	3539	
Flt Permitted	0.95	1.00						1.00	1.00	0.95	1.00	
Satd. Flow (perm)	1770	1589						3539	1583	3433	3539	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	332	5	234	0	0	0	0	1153	613	865	1034	0
RTOR Reduction (vph)	0	117	0	0	0	0	0	0	282	0	0	0
Lane Group Flow (vph)	332	122	0	0	0	0	0	1153	331	865	1034	0
Turn Type	Perm								Perm	Prot		
Protected Phases		4						2		1	6	
Permitted Phases	4								2			
Actuated Green, G (s)	20.0	20.0						33.0	33.0	25.0	62.0	
Effective Green, g (s)	20.0	20.0						33.0	33.0	25.0	62.0	
Actuated g/C Ratio	0.22	0.22						0.37	0.37	0.28	0.69	
Clearance Time (s)	4.0	4.0						4.0	4.0	4.0	4.0	
Lane Grp Cap (vph)	393	353						1298	580	954	2438	
v/s Ratio Prot		0.08						c0.33		c0.25	0.29	
v/s Ratio Perm	c0.19								0.21			
v/c Ratio	0.84	0.34						0.89	0.57	0.91	0.42	
Uniform Delay, d1	33.5	29.5						26.8	22.8	31.4	6.2	
Progression Factor	1.00	1.00						0.75	0.75	0.63	0.73	
Incremental Delay, d2	19.5	2.7						6.3	2.6	7.9	0.3	
Delay (s)	53.0	32.1						26.4	19.8	27.7	4.8	
Level of Service	D	C						C	B	C	A	
Approach Delay (s)		44.3			0.0			24.1			15.2	
Approach LOS		D			A			C			B	

Intersection Summary			
HCM Average Control Delay	22.8	HCM Level of Service	C
HCM Volume to Capacity ratio	0.88		
Actuated Cycle Length (s)	90.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	96.0%	ICU Level of Service	F
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Volume (vph)	0	0	0	464	5	656	174	1192	0	0	1283	246
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)				4.0	4.0		4.0	4.0			4.0	4.0
Lane Util. Factor				0.97	1.00		1.00	0.95			0.95	1.00
Frt				1.00	0.85		1.00	1.00			1.00	0.85
Flt Protected				0.95	1.00		0.95	1.00			1.00	1.00
Satd. Flow (prot)				3433	1585		1770	3539			3539	1583
Flt Permitted				0.95	1.00		0.10	1.00			1.00	1.00
Satd. Flow (perm)				3433	1585		191	3539			3539	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	0	0	0	504	5	713	189	1296	0	0	1395	267
RTOR Reduction (vph)	0	0	0	0	19	0	0	0	0	0	0	102
Lane Group Flow (vph)	0	0	0	504	699	0	189	1296	0	0	1395	165
Turn Type				Prot			pm+pt					Perm
Protected Phases				3	8		5	2			6	
Permitted Phases							2					6
Actuated Green, G (s)				37.0	37.0		45.0	45.0			35.0	35.0
Effective Green, g (s)				37.0	37.0		45.0	45.0			35.0	35.0
Actuated g/C Ratio				0.41	0.41		0.50	0.50			0.39	0.39
Clearance Time (s)				4.0	4.0		4.0	4.0			4.0	4.0
Vehicle Extension (s)				3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)				1411	652		201	1770			1376	616
v/s Ratio Prot				0.15	c0.44		c0.06	0.37			c0.39	
v/s Ratio Perm							0.41					0.10
v/c Ratio				0.36	1.07		0.94	0.73			1.01	0.27
Uniform Delay, d1				18.3	26.5		20.8	17.7			27.5	18.8
Progression Factor				1.00	1.00		1.93	0.13			0.56	0.55
Incremental Delay, d2				0.2	55.9		28.1	1.2			22.4	0.6
Delay (s)				18.4	82.4		68.2	3.5			37.9	10.9
Level of Service				B	F		E	A			D	B
Approach Delay (s)		0.0			56.0			11.7			33.5	
Approach LOS		A			E			B			C	

Intersection Summary			
HCM Average Control Delay	32.4	HCM Level of Service	C
HCM Volume to Capacity ratio	1.04		
Actuated Cycle Length (s)	90.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	96.0%	ICU Level of Service	F
Analysis Period (min)	15		
c Critical Lane Group			

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑↑	↱	↰	↑↑	↱	↰	↑	↱	↰↰	↑	↱
Volume (vph)	188	918	9	79	990	587	17	5	149	662	5	55
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	1.00	1.00	1.00	0.97	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1770	1863	1583	3433	1863	1583
Flt Permitted	0.16	1.00	1.00	0.16	1.00	1.00	0.75	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	290	3539	1583	290	3539	1583	1405	1863	1583	3433	1863	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	204	998	10	86	1076	638	18	5	162	720	5	60
RTOR Reduction (vph)	0	0	4	0	0	226	0	0	88	0	0	35
Lane Group Flow (vph)	204	998	6	86	1076	412	18	5	74	720	5	25
Turn Type	pm+pt		Perm	pm+pt		pm+ov	pm+pt		Perm	Prot		Perm
Protected Phases	7	4		3	8	1	5	2		1	6	
Permitted Phases	4		4	8		8	2		2			6
Actuated Green, G (s)	32.4	32.4	32.4	28.9	28.9	48.9	20.0	18.4	18.4	20.0	36.8	36.8
Effective Green, g (s)	32.4	32.4	32.4	28.9	28.9	48.9	20.0	18.4	18.4	20.0	36.8	36.8
Actuated g/C Ratio	0.36	0.36	0.36	0.32	0.32	0.54	0.22	0.20	0.20	0.22	0.41	0.41
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	215	1274	570	146	1136	860	319	381	324	763	762	647
v/s Ratio Prot	0.07	c0.28		0.02	c0.30	0.11	0.00	0.00		c0.21	0.00	
v/s Ratio Perm	0.27		0.00	0.17		0.15	0.01		c0.05			0.02
v/c Ratio	0.95	0.78	0.01	0.59	0.95	0.48	0.06	0.01	0.23	0.94	0.01	0.04
Uniform Delay, d1	36.1	25.7	18.5	25.0	29.8	12.7	27.5	28.6	29.9	34.4	15.8	16.0
Progression Factor	1.00	1.00	1.00	0.70	0.71	0.56	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	49.4	4.9	0.0	3.0	9.2	0.2	0.1	0.1	1.7	20.0	0.0	0.1
Delay (s)	85.5	30.5	18.5	20.5	30.4	7.3	27.6	28.6	31.5	54.4	15.8	16.1
Level of Service	F	C	B	C	C	A	C	C	C	D	B	B
Approach Delay (s)		39.7			21.7			31.1			51.2	
Approach LOS		D			C			C			D	

Intersection Summary			
HCM Average Control Delay	33.5	HCM Level of Service	C
HCM Volume to Capacity ratio	0.74		
Actuated Cycle Length (s)	90.0	Sum of lost time (s)	12.0
Intersection Capacity Utilization	73.3%	ICU Level of Service	D
Analysis Period (min)	15		

c Critical Lane Group

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Right Turn Channelized												
Volume (veh/h)	11	538	59	353	524	185	37	5	221	156	5	16
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	12	585	64	384	570	201	40	5	240	170	5	17
Approach Volume (veh/h)		661			1154			286			192	
Crossing Volume (veh/h)		559			58			766			993	
High Capacity (veh/h)		891			1324			753			626	
High v/c (veh/h)		0.74			0.87			0.38			0.31	
Low Capacity (veh/h)		716			1106			596			486	
Low v/c (veh/h)		0.92			1.04			0.48			0.40	

Intersection Summary			
Maximum v/c High	0.87		
Maximum v/c Low	1.04		
Intersection Capacity Utilization	86.9%	ICU Level of Service	E

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↱		↰	↱	
Volume (veh/h)	18	545	37	85	450	42	17	2	38	25	2	11
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	20	592	40	92	489	46	18	2	41	27	2	12
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	535			633			1318	1351	592	1348	1346	489
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	535			633			1318	1351	592	1348	1346	489
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	98			90			84	98	92	74	98	98
cM capacity (veh/h)	1033			950			118	133	506	106	134	579

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	20	592	40	92	489	46	18	43	27	14
Volume Left	20	0	0	92	0	0	18	0	27	0
Volume Right	0	0	40	0	0	46	0	41	0	12
cSH	1033	1700	1700	950	1700	1700	118	444	106	383
Volume to Capacity	0.02	0.35	0.02	0.10	0.29	0.03	0.16	0.10	0.26	0.04
Queue Length 95th (ft)	1	0	0	8	0	0	13	8	24	3
Control Delay (s)	8.6	0.0	0.0	9.2	0.0	0.0	40.9	14.0	50.3	14.8
Lane LOS	A			A			E	B	F	B
Approach Delay (s)	0.3			1.4			22.0		38.1	
Approach LOS							C		E	

Intersection Summary			
Average Delay	2.9		
Intersection Capacity Utilization	51.4%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	SEL	SET	SER	NWL	NWT	NWR	NEL	NET	NER	SWL	SWT	SWR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↱		↰	↱	
Volume (veh/h)	12	527	55	128	322	28	25	2	57	16	2	7
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	13	573	60	139	350	30	27	2	62	17	2	8
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	380			633			1236	1258	573	1290	1287	350
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	380			633			1236	1258	573	1290	1287	350
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	99			85			79	98	88	84	98	99
cM capacity (veh/h)	1178			950			132	144	519	108	139	693

Direction, Lane #	SE 1	SE 2	SE 3	NW 1	NW 2	NW 3	NE 1	NE 2	SW 1	SW 2
Volume Total	13	573	60	139	350	30	27	64	17	10
Volume Left	13	0	0	139	0	0	27	0	17	0
Volume Right	0	0	60	0	0	30	0	62	0	8
cSH	1178	1700	1700	950	1700	1700	132	477	108	367
Volume to Capacity	0.01	0.34	0.04	0.15	0.21	0.02	0.21	0.13	0.16	0.03
Queue Length 95th (ft)	1	0	0	13	0	0	18	12	14	2
Control Delay (s)	8.1	0.0	0.0	9.4	0.0	0.0	39.3	13.7	44.8	15.1
Lane LOS	A			A			E	B	E	C
Approach Delay (s)	0.2			2.5			21.3		34.1	
Approach LOS							C		D	

Intersection Summary			
Average Delay	3.3		
Intersection Capacity Utilization	52.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑	↱	↰	↑	↰	↱
Volume (veh/h)	587	12	8	346	11	7
Sign Control	Free			Free	Stop	
Grade	0%			0%	0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	638	13	9	376	12	8
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)						
pX, platoon unblocked						
vC, conflicting volume			651		1032	638
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			651		1032	638
tC, single (s)			4.1		6.4	6.2
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			99		95	98
cM capacity (veh/h)			935		256	477

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	NB 1	NB 2
Volume Total	638	13	9	376	12	8
Volume Left	0	0	9	0	12	0
Volume Right	0	13	0	0	0	8
cSH	1700	1700	935	1700	256	477
Volume to Capacity	0.38	0.01	0.01	0.22	0.05	0.02
Queue Length 95th (ft)	0	0	1	0	4	1
Control Delay (s)	0.0	0.0	8.9	0.0	19.8	12.7
Lane LOS			A		C	B
Approach Delay (s)	0.0		0.2		17.0	
Approach LOS					C	

Intersection Summary			
Average Delay	0.4		
Intersection Capacity Utilization	40.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations	↰	↑	↑	↱	↰	↱
Volume (veh/h)	10	587	342	15	12	8
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	11	638	372	16	13	9
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)			388			
pX, platoon unblocked						
vC, conflicting volume	388				1032	372
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	388				1032	372
tC, single (s)	4.1				6.4	6.2
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	99				95	99
cM capacity (veh/h)	1170				256	674

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	SB 1	SB 2
Volume Total	11	638	372	16	13	9
Volume Left	11	0	0	0	13	0
Volume Right	0	0	0	16	0	9
cSH	1170	1700	1700	1700	256	674
Volume to Capacity	0.01	0.38	0.22	0.01	0.05	0.01
Queue Length 95th (ft)	1	0	0	0	4	1
Control Delay (s)	8.1	0.0	0.0	0.0	19.8	10.4
Lane LOS	A				C	B
Approach Delay (s)	0.1		0.0		16.1	
Approach LOS					C	

Intersection Summary			
Average Delay	0.4		
Intersection Capacity Utilization	40.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Right Turn Channelized												
Volume (veh/h)	70	566	28	15	305	30	16	2	11	20	2	46
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	76	615	30	16	332	33	17	2	12	22	2	50
Approach Volume (veh/h)		722			380			32			74	
Crossing Volume (veh/h)		40			96			713			365	
High Capacity (veh/h)		1342			1285			786			1039	
High v/c (veh/h)		0.54			0.30			0.04			0.07	
Low Capacity (veh/h)		1122			1071			625			849	
Low v/c (veh/h)		0.64			0.36			0.05			0.09	

Intersection Summary			
Maximum v/c High	0.54		
Maximum v/c Low	0.64		
Intersection Capacity Utilization	68.3%	ICU Level of Service	C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↑↱		↰	↑↱	
Volume (veh/h)	30	649	85	21	339	7	41	2	10	5	2	18
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	33	705	92	23	368	8	45	2	11	5	2	20
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	376			798			1205	1192	705	1197	1277	368
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	376			798			1205	1192	705	1197	1277	368
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	97			97			70	99	98	96	99	97
cM capacity (veh/h)	1182			824			148	177	436	151	157	677

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	33	705	92	23	368	8	45	13	5	22
Volume Left	33	0	0	23	0	0	45	0	5	0
Volume Right	0	0	92	0	0	8	0	11	0	20
cSH	1182	1700	1700	824	1700	1700	148	350	151	509
Volume to Capacity	0.03	0.41	0.05	0.03	0.22	0.00	0.30	0.04	0.04	0.04
Queue Length 95th (ft)	2	0	0	2	0	0	30	3	3	3
Control Delay (s)	8.1	0.0	0.0	9.5	0.0	0.0	39.5	15.7	29.8	12.4
Lane LOS	A			A			E	C	D	B
Approach Delay (s)	0.3			0.5			34.1		15.9	
Approach LOS							D		C	

Intersection Summary			
Average Delay	2.2		
Intersection Capacity Utilization	49.8%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰	↑	↱	↰	↱		↰	↱	
Volume (veh/h)	30	634	85	22	369	7	40	5	10	5	5	22
Sign Control		Free			Free			Stop			Stop	
Grade		0%			0%			0%			0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	33	689	92	24	401	8	43	5	11	5	5	24
Pedestrians												
Lane Width (ft)												
Walking Speed (ft/s)												
Percent Blockage												
Right turn flare (veh)												
Median type		None			None							
Median storage veh)												
Upstream signal (ft)												
pX, platoon unblocked												
vC, conflicting volume	409			782			1230	1211	689	1217	1296	401
vC1, stage 1 conf vol												
vC2, stage 2 conf vol												
vCu, unblocked vol	409			782			1230	1211	689	1217	1296	401
tC, single (s)	4.1			4.1			7.1	6.5	6.2	7.1	6.5	6.2
tC, 2 stage (s)												
tF (s)	2.2			2.2			3.5	4.0	3.3	3.5	4.0	3.3
p0 queue free %	97			97			69	97	98	96	96	96
cM capacity (veh/h)	1150			836			139	172	446	144	153	649

Direction, Lane #	EB 1	EB 2	EB 3	WB 1	WB 2	WB 3	NB 1	NB 2	SB 1	SB 2
Volume Total	33	689	92	24	401	8	43	16	5	29
Volume Left	33	0	0	24	0	0	43	0	5	0
Volume Right	0	0	92	0	0	8	0	11	0	24
cSH	1150	1700	1700	836	1700	1700	139	291	144	406
Volume to Capacity	0.03	0.41	0.05	0.03	0.24	0.00	0.31	0.06	0.04	0.07
Queue Length 95th (ft)	2	0	0	2	0	0	31	4	3	6
Control Delay (s)	8.2	0.0	0.0	9.4	0.0	0.0	42.4	18.1	31.0	14.6
Lane LOS	A			A			E	C	D	B
Approach Delay (s)	0.3			0.5			35.8		17.1	
Approach LOS							E		C	

Intersection Summary			
Average Delay	2.4		
Intersection Capacity Utilization	48.9%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	EBL	EBT	WBT	WBR	SBL	SBR
Lane Configurations						
Volume (veh/h)	120	749	379	52	46	107
Sign Control		Free	Free		Stop	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	130	814	412	57	50	116
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None	None			
Median storage veh)						
Upstream signal (ft)						
pX, platoon unblocked						
vC, conflicting volume	468				1487	412
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	468				1487	412
tC, single (s)	4.1				6.4	6.2
tC, 2 stage (s)						
tF (s)	2.2				3.5	3.3
p0 queue free %	88				59	82
cM capacity (veh/h)	1093				121	640

Direction, Lane #	EB 1	EB 2	WB 1	WB 2	SB 1	SB 2
Volume Total	130	814	412	57	50	116
Volume Left	130	0	0	0	50	0
Volume Right	0	0	0	57	0	116
cSH	1093	1700	1700	1700	121	640
Volume to Capacity	0.12	0.48	0.24	0.03	0.41	0.18
Queue Length 95th (ft)	10	0	0	0	44	16
Control Delay (s)	8.7	0.0	0.0	0.0	54.6	11.9
Lane LOS	A				F	B
Approach Delay (s)	1.2		0.0		24.7	
Approach LOS					C	

Intersection Summary			
Average Delay	3.3		
Intersection Capacity Utilization	49.4%	ICU Level of Service	A
Analysis Period (min)	15		

Movement	WBL2	WBL	NBL	NBR	NER	NER2
Lane Configurations						
Volume (veh/h)	7	479	37	4	865	67
Sign Control		Free	Stop		Free	
Grade		0%	0%		0%	
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	8	521	40	4	940	73
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type		None			None	
Median storage veh)						
Upstream signal (ft)		590				
pX, platoon unblocked						
vC, conflicting volume	1013		1476	940		
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol	1013		1476	940		
tC, single (s)	4.1		6.4	6.2		
tC, 2 stage (s)						
tF (s)	2.2		3.5	3.3		
p0 queue free %	99		71	99		
cM capacity (veh/h)	684		137	320		

Direction, Lane #	WB 1	WB 2	NB 1	NE 1	NE 2
Volume Total	8	521	45	940	73
Volume Left	8	0	40	0	0
Volume Right	0	0	4	0	73
cSH	684	1700	146	1700	1700
Volume to Capacity	0.01	0.31	0.31	0.55	0.04
Queue Length 95th (ft)	1	0	30	0	0
Control Delay (s)	10.3	0.0	40.3	0.0	0.0
Lane LOS	B		E		
Approach Delay (s)	0.1		40.3	0.0	
Approach LOS			E		

Intersection Summary			
Average Delay	1.2		
Intersection Capacity Utilization	56.9%	ICU Level of Service	B
Analysis Period (min)	15		