Traffic Analysis Report

I-5 Interchanges 14 and 19

(Green Springs and North Ashland Interchanges)

City of Ashland
Jackson County

Prepared for

Oregon Department of Transportation, Region 3

3500 NW Stewart Parkway

Roseburg, Oregon 97470

Prepared by

David Evans and Associates, Inc.

2100 SW River Parkway

Portland, Oregon

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Executive Summary

This Traffic Analysis Report was undertaken to assess the bridge and roadway configuration needed in connection with OTIA III Bridge Bundle #314. The analysis focused on the year of construction (2010) and a twenty-year horizon (2030).

No-build conditions were analyzed for traffic for analysis years 2006, 2010 and 2030. MUTCD traffic signal warrants and TPAU preliminary traffic signal warrants were analyzed. The principal focus, however, was to determine what lane configuration would be needed to meet mobility standards and to accommodate year 2030 traffic.

At the North Ashland Interchange 19 (South Valley View Road), a three-lane bridge was determined to be adequate. Signalization of the ramp terminals is not necessary for at least several years based on the application of preliminary traffic signal warrants. Based on ODOT mobility standards, however, installation of traffic signals at the ramp terminals is likely to be necessary to accommodate year 2030 traffic volumes. As an alternative to signalization of the northbound ramp terminal, a loop ramp could be used at the northbound ramp terminal to accommodate the high volumes of traffic from northbound South Valley View Road to northbound I-5. A portion of both the northbound and southbound exit ramps will need two lanes as they approach the ramp terminals to allow separate left-turn and right-turn lanes. Eventually, the southbound exit ramp will likely require a dedicated, free-flow, right-turn lane onto southbound South Valley View Road to assure that the mobility standard specified in the Highway Design Manual is met. Widening of South Valley View Road to five lanes, a project specified in the County's Transportation System Plan, would allow this to be accommodated.

At the Green Springs Interchange 14 (Ashland Street), a four-lane bridge was determined to be adequate to accommodate design hour volumes. Due to geometric constraints a four-lane bridge would likely require design exceptions for the westbound left-turn lane length and taper length between the westbound and eastbound left-turn lanes. A five-lane bridge appears to allow all ODOT design standards to be met on the bridge. High volumes of left turns from eastbound Ashland Street to northbound I-5 are sufficient to require dual left-turn lanes by 2030. Traffic signal warrants are currently met for the southbound ramp terminal and installation of traffic signals at the northbound ramp terminal will, in connection, with the dual turn lanes avoid long queues for the eastbound-to-northbound left turn and avoid operational problems. Two lanes will be required for a portion of the exit ramp to allow separate lanes for left and right turns at the ramp terminals. Right turns from the southbound exit ramp to westbound Ashland Street can be accommodated with a dedicated receiving lane on westbound Ashland Street.

1 Introduction and Background

The OTIA III Bundle #314 bridge replacement, safety project will improve interstate interchange bridges crossing I-5 at the Green Springs Interchange (Exit 14) and the North Ashland Interchange (Exit 19), and will include ramp improvements. The OTIA III projects managed by the Oregon Bridge Delivery Partners (OBDP) are scheduled to be let in August 2008, with estimated completion by year 2010. Additionally, the Oregon Department of Transportation, Region 3, has identified the need to prepare Interchange Area Management Plans (IAMPs) for both interchanges, which will address public improvement investments via land use and transportation management planning.

The work currently planned under the OTIA III Bridge Bundle #314 is intended to address immediate repair and safety improvements for the interchanges, allowing for future planned improvement projects identified in the regional, county and city Transportation System Plans (TSPs) to address long-term capacity improvements.

Critical design elements of the Bridge Bundle #314 improvements at each interchange can be selected based in part on the results of the traffic analysis contained in this Traffic Analysis Report. This report focuses on the design of the bridges, ramp terminals, and roadway sections immediately adjacent to them and is intended to inform and support the scope of the OTIA III Bundle #314 bridge replacement, safety improvement project. The purpose of this Traffic Analysis Report is: 1) to provide ODOT and OBDP with adequate safety and operations analysis information to select appropriate bridge improvements for the Bundle #314 projects, and 2) for subsequent use in IAMP projects at the Green Springs and North Ashland Interchanges.

This report provides an inventory of existing roadways, provides existing and future traffic operations analyses, safety analysis, and evaluates several build options at each interchange designed to address existing or projected transportation needs.

2 Analysis Area

2.1 Interchange 14 (Green Springs Interchange)

This interchange consists of a standard diamond interchange in the southeastern area of Ashland. Ashland Street (OR 66, Green Springs Highway) is the cross street, which connects with OR 99 (Rogue Valley Highway) approximately one mile to the west. All four quadrants of the interchange area are developed.

The existing bridge carries two lanes of OR 66 over I-5. The ramp terminals are unsignalized. Both the bridge and ramps are functionally obsolete to serve long-range transportation needs.

The Green Springs Interchange analysis area stretches along OR 66 between its intersections with Tolman Creek Road and Oak Knoll Drive/East Main Street. The overall length of this roadway segment is just under one mile and is characterized by suburban commercial development with multiple closely-spaced private accesses.

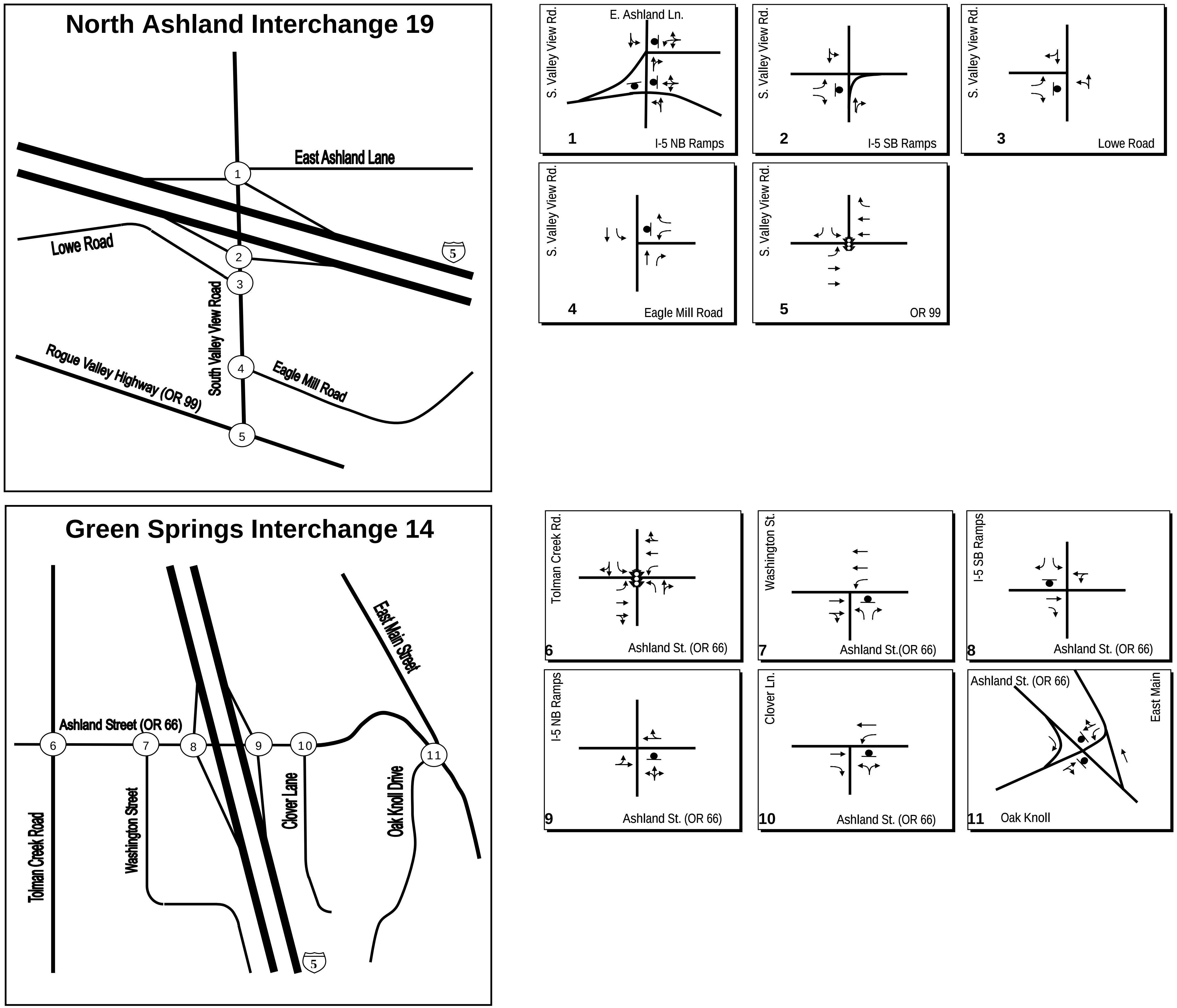
2.2 Interchange 19 (North Ashland Interchange)

This interchange has a standard diamond configuration. South Valley View Road, a Jackson County facility, is the interchange crossroad and provides access between the Rogue Valley Highway (OR 99) and I-5. The intersection angle of South Valley View Road and I-5 is slightly skewed but the existing ramp terminals intersect South Valley View Road at a 90-degree angle. While located outside of

Ashland's Urban Growth Boundary, this interchange is heavily used by commuters within the Rogue Valley MPO area. The segment of South Valley View Road between OR 99 and I-5 carries the highest traffic volumes of any Jackson County roadway.

The North Ashland Interchange analysis area consists of all the major intersections along South Valley View Road from East Ashland Lane located directly north of the I-5 northbound ramp terminal intersection, to OR 99, which lies about 2500 feet south of the interchange.

Figure 1 illustrates both analysis areas, showing existing lane configurations and traffic control at all analysis area intersections. Table 1 provides roadway names, jurisdictional authorities, functional classifications, posted speeds (if available), number of lanes and operational standards for analysis area roadways. This information was collected through a site visit and review of the 1999 Oregon Highway Plan, the Transportation System Plans for Jackson County and the City of Ashland, and the Ashland Municipal Code.



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NOT TO SCALE

LEGEND

- = Traffic Signal
- = Stop Sign
- = Allowable turning movement for desinated lane
- = Intersection number

FIGURE 1

Existing Lane Configurations and Traffic Control Devices

I-5 Interchanges 14 & 19

Table 1. Analysis Area Roadway Inventory

	Roadway/Highway Name	Jurisdiction	ODOT Functional Classification	City/County Functional Classification	Posted Speed	Lanes	OHP Mobility Standard[1,2]	HDM Mobility Standard[1,3]	City/ County Operational Standard[1]
	I-5 (Pacific Highway No. 1)	ODOT	Interstate Hwy, NHS[9], FR[10]	-	65	4	0.80	0.75	-
	I-5 Ramp terminal Intersections	ODOT	Interstate Hwy, NHS[9], FR[10]	-	-	1	0.85, 0.90[4]	0.75	-
Interchange 14	Ashland Street (OR 66, Green Springs Highway)	ODOT	District Hwy	Boulevard	35	2,3,5[5]	0.90	0.85	0.90[6]
	Tolman Creek Road	City of Ashland	-	Avenue	30	2	-	-	0.90[6]
	Washington Street	City of Ashland	-	Neighborhood Collector	25	2	-	-	0.90[6]
	Clover Lane	City of Ashland	-	Local Street	-	2	-	-	0.90[6]
	Oak Knoll Drive	City of Ashland	-	Avenue	-	2	-	-	0.90[6]
	East Main Street	City of Ashland	-	Avenue	-	2	-	-	0.90[6]
Interchange 19	OR 99 (Rogue Valley Hwy)	ODOT	District Hwy	Arterial	45,55[7]	5	0.90	0.85	0.95[8]
	South Valley View Road	Jackson County	-	Arterial	45	3	-	-	0.95[8]
	Eagle Mill Road	Jackson County	-	Minor Collector	45	2	-	-	0.95[8]
	Lowe Road	Jackson County	-	Local Street	-	2	-	-	0.95[8]
	East Ashland Lane	Jackson County	-	Local Street	-	2	-	-	0.95[8]

Notes:
1. Values shown are volume to capacity (v/c) ratios.
2. Source: 1999 Oregon Highway Plan (OHP), Table 6. Standards apply to planning projects through the planning horizon (2030).
3. Source: 2003 Highway Design Manual (HDM), Table 10-1. Standards apply to project design through the planning horizon (2030).
4. Operational standard for interchange ramp terminals shall be the smaller of 0.85 or the value of the v/c for the crossroad. Within an MPO, v/c ratio may be as high as 0.90 if 95^{th} percentile queue does not extend into deceleration portion of the ramp (Source: 1999 OHP).
5. Five lanes west of I-5, 3 lanes between I-5 and Sutton Place, 2 lanes between Sutton Place and Oak Knoll Drive/East Main Street
6. Ashland Municipal Code requires that traffic operations on City facilities do not exceed capacity (v/c < 1.00) and defers to ODOT standards for intersections with State highways within the City. OHP District Highway mobility standard is shown.
7. 45 mph south of I-5, 55 mph north of I-5.
8. Source: Jackson County Transportation System Plan.
9. NHS: National Highway System
10. FR: Freight Route

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3 Traffic Counts

Traffic counts were conducted on May 16, 2006. The counts consisted of 16-hour and 3-hour AM and PM peak period counts at analysis area intersections, and a 24-hour count on the I-5 mainline. The counts included full FHWA 13-class vehicle classifications. Raw traffic data at all count locations are provided in Appendix A. Table 2 below provides a list of all intersection count locations including the count type.

Table 2. Intersection Turning Movement Count Locations and Types

Location	Type of Count
North Ashland Interchange 19 Analysis Area	
I-5 mainline, both directions, South of North Ashland Interchange	24 hour
South Valley View Road at East Ashland Lane	16-hour (6 AM – 10 PM)
North Ashland Interchange 19 : Northbound Ramps at South Valley View Road	16-hour (6 AM – 10 PM)
North Ashland Interchange 19: Southbound Ramps at South Valley View Road	16-hour (6 AM – 10 PM)
South Valley View Road at Lowe Road	3-hour AM and PM
South Valley View Road at Eagle Mill Road	3-hour AM and PM
South Valley View Drive at Rogue Valley Highway (OR 99)	16-hour (6 AM – 10 PM)
Green Springs Interchange 14 Analysis Area	
OR 66 (Ashland Street) at Tolman Creek Road	16-hour (6 AM – 10 PM)
Ashland Street at Washington Street	3-hour AM and PM
Green Springs Interchange 14: Southbound Ramps at Ashland Street	16-hour (6 AM – 10 PM)
Green Springs Interchange 14: Northbound Ramps at Ashland Street	16-hour (6 AM – 10 PM)
Ashland Street at Clover Lane	3-hour AM and PM
Ashland Street at Main Street / Oak Knoll Drive	3-hour AM and PM

Note: All 16-hour and 3-hour counts conducted May 16, 2006. 24-hour I-5 mainline count conducted from 22:00 May 15 to 22:00 May 16, 2006.

4 Developing 30th Hour Volumes

The ODOT Transportation Planning Analysis Unit (TPAU) has developed procedures for calculating current year 30th highest hour traffic volumes (30th HV). These procedures are outlined in the TPAU Analysis Procedures Manual. 30th hour traffic volumes usually occur during the peak month of the year and are calculated by applying a seasonal factor to the peak hour volumes. Typically the peak hour volumes occur during the PM peak hour. Occasionally the unique directional distribution and other factors associated with AM peak hour operations can result in the 30th highest hour occurring during the AM peak hour at some intersections. Most intersections in the analysis area, and the system as a whole, experience higher traffic volumes during the PM peak hour than during the AM peak period. However, the southbound ramp terminal at Interchange 14 experiences roughly equivalent volumes during each of the AM and PM peak hours. Separate AM peak hour analyses were conducted at this interchange and are discussed in Section 11 of this report.

After reviewing the TPAU methodologies for determination of seasonal factors, it was determined that use of the TPAU Seasonal Trend Table would be the most appropriate. The Seasonal Trend Table averages statewide seasonal trends according to highway type.

Table 3 below provides selected data from the table for three seasonal trend types: Interstate, Summer and Commuter. The values provided in the ‘May 15’ column represent the inverse of the decimal percent of average daily traffic (ADT). For example, interstate volumes collected during mid-May are 98% (1/1.0200) of average. The values provided in the ‘Peak SF’ column represent the inverse of the decimal percent of ADT during the peak month of the year. To determine a seasonal factor, the seasonal trend factor for the applicable time of year (May

15 column) is divided by the peak seasonal trend factor. The calculated seasonal factors resulting from the Interstate and Summer seasonal trends were 1.18 and 1.17, respectively; the seasonal factor resulting from the Commuter seasonal trend was 1.04.

Table 3. Seasonal Factors From Seasonal Trend Table Data

Seasonal Trend	May 15†	Peak SF†	Calculated SF
Interstate	1.0200	0.8669	1.18
Summer	0.9776	0.8378	1.17
Commuter	0.9366	0.9000	1.04
Average of Summer, Commuter (50/50)	0.9571	0.8689	1.10
Weighted Average of Summer, Commuter (25/75)	0.9469	0.8845	1.07

†Source: 2005 Seasonal Trend Table

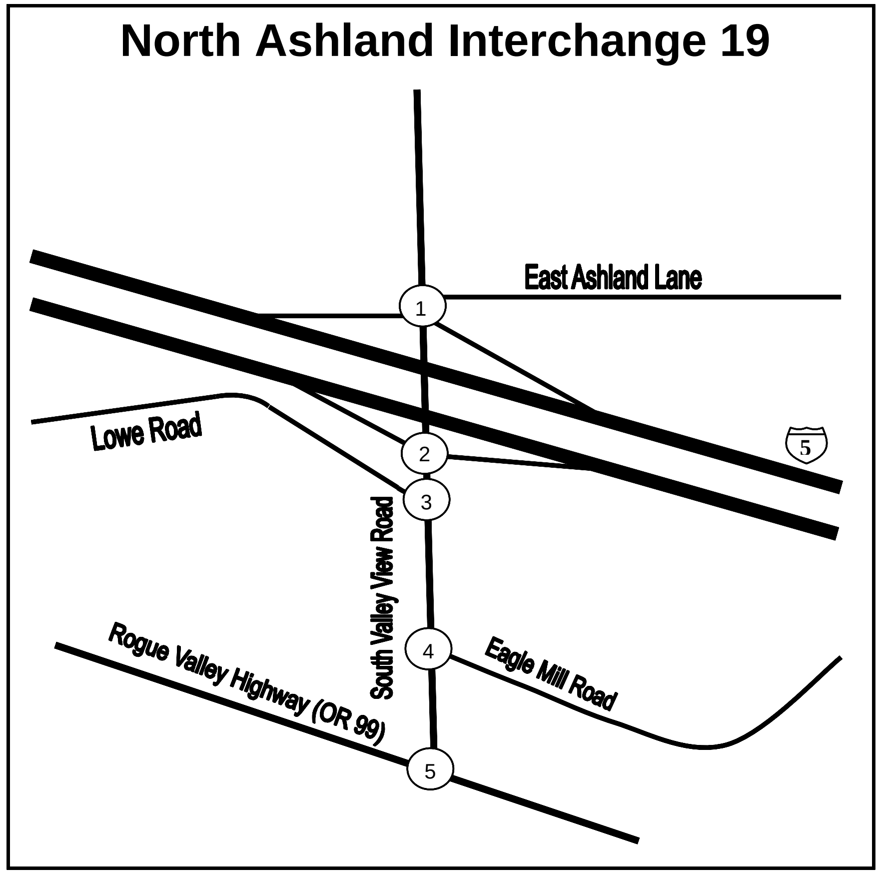
The Analysis Procedures Manual notes that it is appropriate to average Summer and Commuter trends in some mid-sized cities. A straight average of these two trends yields a seasonal factor of 1.10. However, recognizing that the area is more strongly characterized by commuter travel patterns between Ashland and Medford, a weighted average of seasonal factors for the Summer and Commuter seasonal trends was also calculated. Applying a weighting of 25%/75% to Summer and Commuter seasonal trends, respectively, yields a seasonal factor of 1.07.

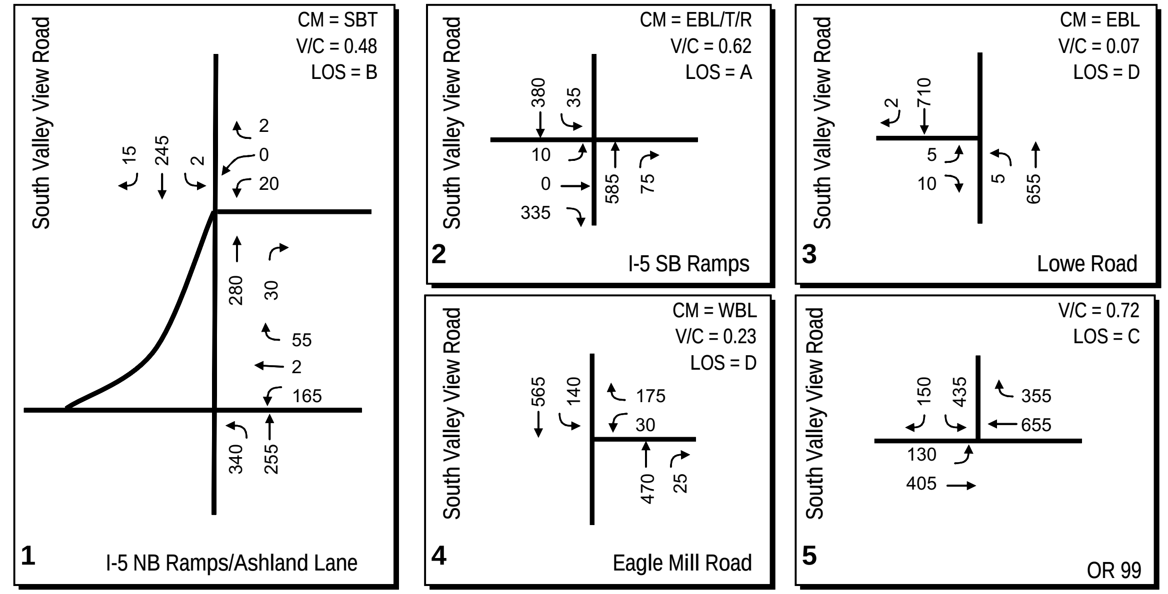
Based on a thorough evaluation of the best available data and consultation with TPAU staff three seasonal factors were derived and applied to the peak hour volumes, depending on the type of facility.

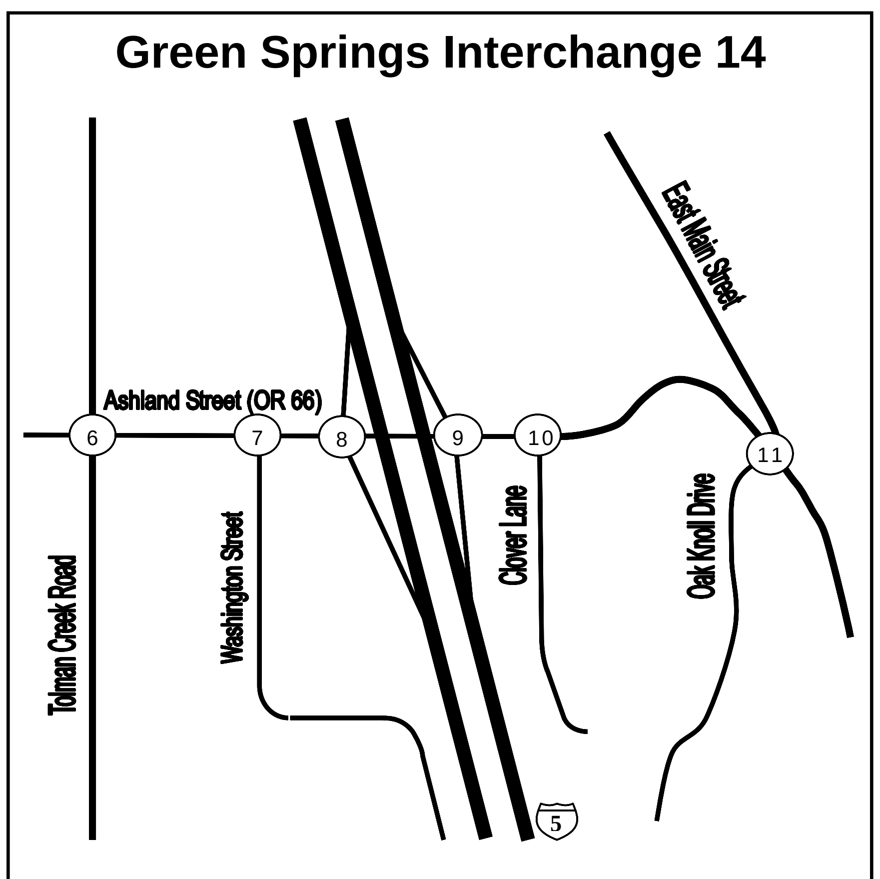
- A seasonal factor of **1.07** was applied to all **non-ramp terminal intersections**.
- A seasonal factor of **1.18** was applied to traffic volumes on the **I-5 mainline**.
- The seasonal factor for **interchange ramp terminal intersections** was determined by averaging the seasonal factors for non-freeway and freeway, or 1.07 and 1.18, respectively. The resulting seasonal factor was **1.13**.

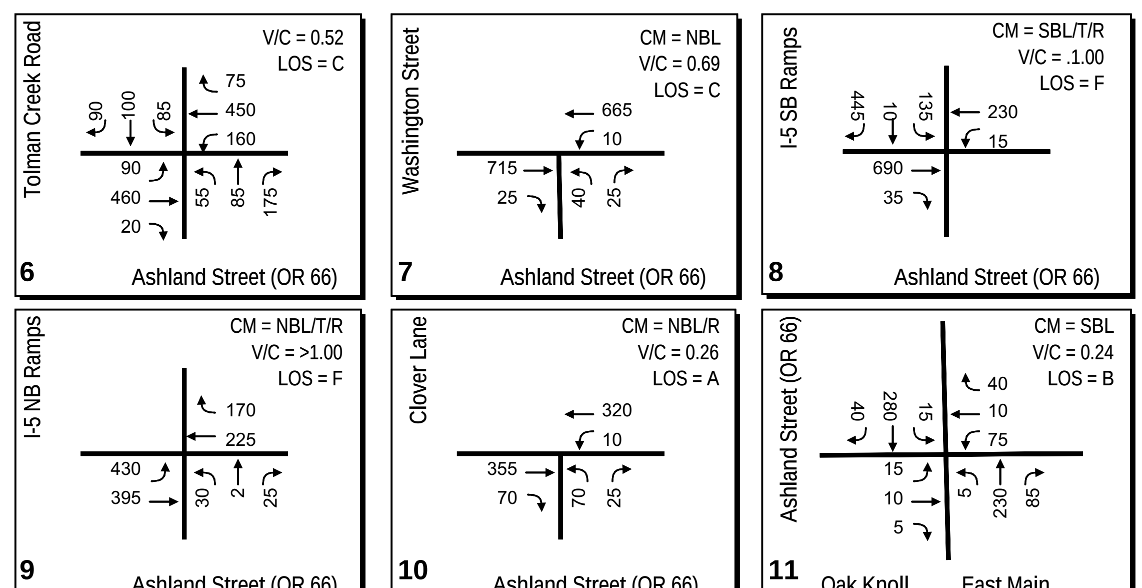
All traffic counts were conducted on the same day. Therefore, within a reasonable margin of error, and after accounting for the effects of driveways, the count volumes were essentially balanced between intersections. By applying different seasonal factors at different intersections the volumes were no longer balanced, and they needed to be manually balanced between intersections. Manual balancing consisted mostly of modifying intersection volumes upward so that entering volumes at each intersection roughly equaled exiting volumes at each adjacent intersection. This is a conservative method because after balancing, traffic volumes at many intersections were increased by more than the seasonal factors indicated on Table 3.

Figure 2 illustrates 30th highest hour volumes, balanced between intersections. Note that the volumes are not perfectly balanced however. This is due to the different seasonal factors applied to different intersections, and the effect of driveways. A table showing raw and adjusted traffic volumes for the common peak hour is provided in Appendix B.









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(11) = Intersection number
123 = Turning movement volume
↰ = Allowable turning movement volume
CM = Critical movement (TWSC)
V/C = Intersection volume to capacity
Critical volume to capacity (TWSC)
LOS = Intersection Level of Service
Critical Level of Service (TWSC)
TWSC = Two-way stop-controlled

FIGURE 2

Existing Conditions (2006)
Balanced PM Peak Hour Volumes
(30th Highest Hour Volumes)
Traffic Operations Analysis Summary

I-5 Interchanges 14 & 19

5 Traffic Operations Standards and Procedures

5.1 Volume to Capacity Ratio

Transportation engineers have established various standards for measuring traffic capacity and quality of service of roadways at intersections. A comparison of traffic volume demand to intersection capacity is one method of evaluating how well an intersection, roadway segment, or merge/diverge segment is operating. This comparison is presented as a volume-to-capacity (v/c) ratio. A v/c ratio of less than 1.0 indicates that the volume is less than capacity. When it is closer to 0.0, traffic conditions are generally good with little congestion and low delays for most intersection movements. As the v/c ratio approaches 1.0, traffic becomes more congested and unstable with longer delays.

ODOT applies two sets of operational standards (mobility standards) to different types of projects. For planning and project analysis of existing conditions and no-build conditions the applicable mobility standards are found in Table 6 of the *1999 Oregon Highway Plan* (OHP). For planning and project analysis of build alternatives, the applicable mobility standards are specified in Table 10-1 of the *2003 Highway Design Manual* (HDM). Mobility standards are dependent on the roadway classification and area type and apply during peak operating conditions through the planning horizon year, which is year 2030. Both are presented in terms of v/c ratios, and they are shown in Table 1.

5.2 Level of Service

Another standard for measuring traffic capacity and quality of service of roadways at intersections is level of service (LOS). At both stop-controlled and signalized intersections, LOS is a function of control delay, which includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. Six standards have been established ranging from LOS A where there is little or no delay, to LOS F, where there is delay of more than 50 seconds at unsignalized intersections, or more than 80 seconds at signalized intersections.

It should be noted that, although delays can sometimes be long for some movements at a stop-controlled intersection, the v/c ratio may indicate that there is adequate capacity to process the demand for that movement. Similarly at signalized intersections, some movements, particularly side street approaches or left turns onto side streets, may experience longer delays because they receive only a small portion of the green time during a signal cycle but their v/c ratio may be relatively low. For these reasons it is important to examine both v/c ratio and LOS when evaluating overall intersection operations. Both are evaluated in the analyses that follow. It should be noted that all of the roadway jurisdictions use v/c, not LOS, as a measure of performance.

5.3 Traffic Operations Analysis Procedures

All of the intersection operations were evaluated using the methodology outlined in the *2000 Highway Capacity Manual* (HCM). Synchro analysis software was used to generate the HCM reports from which the v/c ratios were derived. This report presents 95^{th} percentile queuing and delay results that have been generated by SimTraffic simulation software. The SimTraffic results were derived from the average of five randomly seeded simulation model runs. LOS results were then calculated based on the delay from SimTraffic simulation. While Synchro provides HCM LOS results, SimTraffic simulation can more accurately represent the impact of nearby intersections to delay and queuing. Synchro (HCM) looks at each intersection in isolation. All Synchro and SimTraffic output sheets can be found in Appendix C. The Highway Capacity Software (HCS) tool was used to conduct freeway merge and diverge analyses. All HCM Merge analysis output sheets can be found in Appendix D.

CORSIM is another traffic simulation program that allows for user control of trip assignment through the ability to set vehicle-type specific turn percentages and set predefined vehicle routes. These abilities can be useful under certain circumstances, such as when a pronounced lane imbalance is present. However, it appears that traffic conditions that would warrant use of the CORSIM program are not present at any of the analysis area intersections.

6 Existing Traffic Operations

This section summarizes the traffic operations analysis that was conducted for the study area intersections under existing (seasonally adjusted) traffic volume conditions.

Table 4 summarizes the results for all analysis area intersections and also presents agency operational standards to enable comparison with intersection results. Table 5 summarizes queuing on critical approach legs at the same intersections. Critical movements at unsignalized intersections are typically the minor street left turns or, in the case of single-lane approaches, the minor street approaches. These movements are required to yield to all other movements at the intersection and thus are subject to the longest delays and have least capacity. Left turns from the major street are also subject to delays since motorists making these maneuvers must also yield to on-coming major street traffic. Bold numbers in the tables represent v/c ratios that exceed the mobility standards and approaches with long queues.

Table 4. Existing (Year 2006) 30th Highest Hour Traffic Operations Analysis Results

Intersection	Critical Movement	v/c Ratio	LOS	OHP Std.[1]	HDM Std.[2]	J.C. Std.[3]
Green Springs Interchange 14 Analysis Area						
Ashland St (OR 66) & Tolman Creek Rd	n/a[4]	0.52	C	0.90	0.85	-
Ashland St (OR 66) & Washington St	NBL	0.69	C	0.90	0.85	-
Ashland St (OR 66) & I-5 SB Ramps	SBL/T/R	**>1.00**	**F**	0.85	0.75	-
Ashland St (OR 66) & I-5 NB Ramps	NBL/T/R	**>1.00**	**F**	0.85	0.75	-
Ashland St (OR 66) & Clover Ln	NBL/R	0.26	A	0.90	0.85	-
Ashland St (OR 66) & E. Main/Oak Knoll	SBL[5]	0.24	B	0.90	0.85	-
North Ashland Interchange 19 Analysis Area						
S. Valley View Rd & OR 99	n/a[4]	0.72	C	0.90	0.85	0.95
S. Valley View Rd & Eagle Mill Rd	WBL	0.23	D	-	-	0.95
S. Valley View Rd & Lowe Rd	EBL	0.07	D	-	-	0.95
S. Valley View Rd & I-5 SB Ramps	EBL/T/R	0.62	A	0.85	0.75	0.95
S. Valley View Rd & I-5 NB Ramps	WBL/T/R	0.48	C	0.85	0.75	0.95
S. Valley View Rd & E. Ashland Lne	WBL/R	0.05	C	-	-	0.95

Notes:
1. 1999 Oregon Highway Plan Mobility Standards (Table 6)
2. 2003 ODOT Highway Design Manual Mobility Standards (Table 10-1)
3. Operational standards for Jackson County roadway facilities (Source: Jackson County Transportation System Plan)
4. Signalized intersection. LOS and v/c are for overall intersection.
5. Refers to left turn from Main Street to Ashland Street.

Table 5. Existing (Year 2006) 30th Highest Hour 95th Percentile Queues

Intersection	Movement	95% Queue
Green Springs Interchange 14 Analysis Area		
Ashland St (OR 66) & Tolman Creek Rd	EBL	125
	WBL	150[1]
	NBL	100[1]
	SBL	100[1]
Ashland St (OR 66) & Washington St	WBL	25
	NBL	75[1]
Ashland St (OR 66) & I-5 SB Ramps	WBL/T	50
	SBL/T/R	**675**[2,3]
Ashland St (OR 66) & I-5 NB Ramps	**EBL/T**	**600**[4]
	NBL/T/R	**375**
Ashland St (OR 66) & Clover Ln	WBL/T	50
	NBL/R	75
Ashland St (OR 66) & E. Main/Oak Knoll	NBL/T/R	50
	SBL	75[1]
North Ashland Interchange 19 Analysis Area		
S. Valley View Rd & OR 99	**SBL/R**	**775**[4]
	EBL	175[1]
	WBR	125[1]
S. Valley View Rd & Eagle Mill Rd	WBL/R	175
	SBL	75
S. Valley View Rd & Lowe Rd	EBL/R	25
	NBL/T	25
S. Valley View Rd & I-5 SB Ramps	EBL/T/R	125
	SBL/T	75
S. Valley View Rd & I-5 NB Ramps	WBL/T/R	175
	SBT	175[4]
S. Valley View Rd & E. Ashland Ln	WBL/R	50

Notes:
1. Storage bay at or above capacity.
2. Queue extends beyond extent of SimTraffic model link. Therefore, queue may be longer than shown.
3. Queue extends into ramp deceleration area.
4. Queue extends into adjacent intersection(s).

Most analysis area intersections currently operate with acceptable v/c ratios, queuing and levels of service. However, traffic operations at some key intersections do not meet mobility standards, and drivers experience long delays and substantial queuing. A discussion of these intersections follows.

It should be noted that it is impossible to measure existing intersection operating conditions at a v/c of greater than 1.00, even though Table 4 appears to indicate otherwise. Volume to capacity ratios in excess of 1.00 cannot occur; they actually result from the application of seasonal adjustment factors and growth factors to existing volumes. The v/c ratios greater than 1.00 shown in Table 4 resulted from analysis based on seasonally adjusted volumes that were inflated to approximate 30th highest hour traffic volumes.

6.1 Green Springs Interchange 14 Northbound and Southbound Ramp Terminals

Under adjusted volume conditions that approximate the existing 30th highest hour, analysis shows that critical v/c ratios at both ramp terminal intersections exceed mobility standards and also exceed available capacity. Queuing on both exit ramps, and especially the southbound ramp, is significant because of limited gaps on Ashland Street for left turning vehicles, combined with substantial right turning volumes (on the southbound exit ramp). The 95th percentile queue on the southbound exit ramp is calculated to extend into the deceleration area of the ramp. Left turning vehicles from eastbound Ashland Street to the northbound I-5 entrance ramp are delayed because of conflicts with high westbound through volumes. These delays result in queuing over the bridge that is calculated to spill over into the southbound ramp terminal intersection.

Compounding the operational problems at the ramp terminal intersections are the presence of many accesses to Ashland Street very close to the interchange ramp terminal intersections. These public streets (Clover Lane and Washington Street) and private driveways create potential vehicular conflicts and delay that may impact operations at the interchange. ODOT interchange area access spacing standards, as stated in OAR 734-051 (Division 51), specify that no approaches should be located within 1320 feet of ramp terminal intersections along the cross street. While Division 51 standards are clearly not attainable in a developed area such as this, it is desirable to move in the direction of attaining the standards through access management techniques such as consolidation or elimination of accesses and implementation of turn prohibitions. Discussion of specific access management strategies is beyond the scope of this report and will be addressed in the future Interchange Area Management Plan (IAMP). A general discussion of access management as it pertains to the Green Springs Interchange area is contained in Section 12 of this report.

6.2 North Ashland Interchange 19 Northbound and Southbound Ramp Terminals

Traffic operations analyses at the North Ashland Interchange revealed that both ramp terminals currently meet ODOT OHP mobility standards. However, under adjusted volume conditions that approximate existing 30th highest hour volumes, queuing on the southbound exit ramp is calculated to extend into the deceleration portion of the ramp. Queue lengths are expected to lengthen as traffic volumes increase in the future.

The northbound ramp terminal intersection has an unconventional configuration in which left turning vehicles from South Valley View Road to the I-5 northbound entrance ramp have a free movement. All other approaches, consisting of the I-5 northbound exit ramp and southbound South Valley View Road, must yield to the northbound through and left turning movements. This type of intersection control violates driver expectation and is generally not recommended for new construction. However, since the heaviest movement at this intersection is the left turn from northbound South Valley View Road to I-5 northbound, this configuration provides optimal capacity of any unsignalized configuration. As the safety analysis revealed, and as described in the following section, there is no evidence of an elevated crash rate at this intersection related to the unconventional intersection control.

Similar to the Green Springs Interchange, there are many access points located within close proximity to the ramp terminals. Directly north of the northbound ramp terminal, East Ashland Lane, as well as several private residential driveways, intersect with South Valley View Road. East Ashland Lane provides access to a few homes and experiences very low volumes. Directly south of the southbound ramp terminal, Lowe Road, as well as several driveways (gas stations, a hotel and fast food restaurant) intersect South Valley View Road. Traffic volumes to the south of the interchange are significantly higher than those to the north of the interchange. As traffic volumes increase, the potential conflicts and delays associated with these accesses will have an increasingly significant impact on traffic operations at the interchange. Further discussion of access management as it pertains to the North Ashland Interchange area is contained in Section 12 of this report.

6.3 South Valley View Road at Rogue Valley Highway (OR 99)

This intersection currently has a v/c ratio of 0.72 during 30th highest hour traffic volume conditions. This meets the OHP mobility standard of 0.90 for District-level highways. However, queuing on the single-lane southbound approach is significant, at nearly 800 feet. Future traffic operations are expected to worsen with major queuing on the southbound approach leg. The Jackson County TSP recognizes the need for improvements to South Valley View Road between the North Ashland Interchange and OR 99, as this segment of arterial carries the highest traffic volume of any County roadway. Proposed improvements to this segment of South Valley View Road consist of an upgrade to a five-lane arterial. The TSP classifies the project as a Tier 1 – financially constrained long-range (2014-2023) roadway improvement project.

6.4 Merge/Diverge Analysis

Analyses were conducted using HCM methodology for each of the merge and diverge segments for the entrance and exit ramps at each interchange under existing 30th highest hour traffic volume conditions. The analyses showed that traffic operations at each of the ramp merge and diverge sections meet the OHP mobility standard for interstate freeways. Results of this analysis are provided in Appendix D.

7 Safety Analysis

A safety analysis was conducted to determine if there were any significant documented safety issues within the analysis area and to recommend measures at specific locations or general strategies for improving overall safety.

The safety analysis included a review of crash history data supplied by the ODOT Crash Analysis and Reporting Unit for the period between January 1, 2002 and December 31, 2004, which are the three most recent full years for which crash data is available. It should be noted that the crashes listed are only the crashes reported. The analysis also examined ODOT Safety Priority Index System (SPIS) data and compared calculated crash rates from analysis area roadways with statewide averages.

The process for analyzing the safety data provided was to determine the location and frequency of crashes occurring in the study area. Crashes were totaled by *segment* and by *intersection*. After being summarized and placed into the appropriate segment, crash rates for each roadway segment and intersection influence area were calculated and compared to statewide averages.

7.1 Calculation of Crash Rates

The crash rates were calculated from ODOT-provided crash data. For a crash to be considered associated with an intersection, it must occur within 0.05 mile (265 feet) of the intersection. Beyond this region, crash data is placed in the segment category. It should be noted that this analysis only accounts for those crashes that were reported. In Oregon, legally reportable crashes are those involving death, bodily injury or damage to any one person's property in excess of $1,000 (August 31, 1997 thru December 31, 2003) or $1,500 (after January 1, 2004).

Intersection and segment crash rates were calculated using the following equations.

$$rate_{\text{int}} = \frac{(Crashes \cdot 1{,}000{,}000)}{(365 \cdot Years \cdot ADT)} \quad \text{and} \quad rate_{segment} = \frac{(Crashes \cdot 1{,}000{,}000)}{(365 \cdot Years \cdot Length \cdot ADT)}, \text{ where}$$

$Rate_{int}$ = Crash rate per Million Entering Vehicles (MEV)

$Rate_{segment}$ = Crash rate per Million Vehicle Miles Traveled (MVMT)

Crashes = Number of crashes during the time segment

Years = Number of years being studied

ADT = Average Daily Traffic volume

Length = Length of roadway segment being studied (for segment rates).

The number of crashes was determined from ODOT crash data. At intersections, the sum of all PM peak hour entering volumes from each leg were multiplied by ten to estimate an intersection ADT. For roadway segments the ADT was determined using the ODOT Volume Tables. Crash rates were then calculated for the entire three-year study period.

7.2 SPIS Data

The Safety Priority Index System (SPIS) is a method developed by ODOT for prioritizing locations where funding for safety improvements can be spent most efficiently and effectively. Based on crash data, the SPIS score is influenced by three components: crash frequency, crash rate, and crash severity. Three years of crash data are analyzed to determine a SPIS score for a specific location. SPIS locations meet one of two criteria during the previous three years: (1) three or more crashes at the same location, or (2) one or more fatal crashes at the same location. ODOT produces a list of the sites with the top 10% SPIS scores each year. For the year 2004, which includes crash data for 2001, 2002, and 2003, the SPIS scores at or above 44.49 are in the top 10%.

There is one Top 10% SPIS location in the analysis area of Interchanges 14 and 19, and it is located between milepost 1.27 and 1.37 on OR 66. This is the segment of Ashland Street between and including the northbound and southbound ramp terminals.

7.3 Intersection Crashes

Documented crashes between the years 2002 and 2004 were summarized by location for each of the study intersections. For the purposes of this crash analysis, the study intersections consist only of the ramp terminals and intersections directly adjacent to them. After sorting crashes by location, intersection crash rates were calculated. Table 6 shows the ADT that was determined for each intersection and the calculated crash rates.

Table 6. Study Area Intersection Crash Rates

Intersection	ADT	3 Year Crash Rate
Green Springs Interchange 14: Southbound Ramps at Ashland Street	13,730	0.27
Green Springs Interchange 14: Northbound Ramps at Ashland Street	11,990	0.30
Ashland Street at Clover Lane	7,980	0.34
North Ashland Interchange 19: Northbound Ramps at S.Valley View Road/E. Ashland Lane	9,210	0.00
North Ashland Interchange 19: Southbound Ramps at S. Valley View Road	12,680	0.00
South Valley View Road at Lowe Road	8,760	0.42

The safety analysis showed that none of the intersections in the study area has a crash rate significantly greater than that of the surrounding area. As a rule of thumb, intersections with crash rates of 1.0 or above are potentially problematic and are candidates for further investigation. As Table 6 shows, all of the intersection crash rates are well below the threshold value of 1.0.

There were no reported crashes during the three year study period at either of the ramp terminals of the North Ashland Interchange. However, there were four crashes resulting in three injuries at the intersection of South Valley View Road at Lowe Road, which is very close to the interchange, directly to the south of the southbound ramp terminal intersection. The primary type of crashes at this location are turning and rear end. While the intersection spacing is much closer than what is advised under current interchange design standards, there is no evidence that the crashes were caused by the close spacing. However, future improvements to the interchange should include a relocation of the intersection to a point further south.

7.4 Segment Crashes

Crashes were also examined along Interstate 5 in the vicinity of Interchanges 14 and 19. The crashes were examined on I-5 northbound from the Interchange 14 northbound exit ramp (MP 13.96) to the Interchange 19 northbound entrance ramp (MP 19.40), and on I-5 southbound from the Interchange 19 southbound exit ramp (MP 19.26) to the Interchange 14 southbound entrance ramp (MP 13.96). Table 7 below provides a summary of these crashes.

Table 7. Freeway Crash Data

Location	2002	2003	2004	Total
Northbound				
Green Springs Interchange 14 Exit Ramp (MP 13.96)	1	0	0	1
Green Springs Interchange 14 Entrance Ramp (MP 14.37)	0	0	0	0
North Ashland Interchange 19 Exit Ramp (MP 18.96)	1	0	0	1
North Ashland Interchange 19 Entrance Ramp (MP 19.40)	0	0	0	0
Southbound				
North Ashland Interchange 19 Exit Ramp (MP 19.26)	0	0	0	0
North Ashland Interchange 19 Entrance Ramp (MP 18.91)	0	0	0	0
Green Springs Interchange 14 Exit Ramp (MP 14.37)	0	0	0	0
Green Springs Interchange 14 Entrance Ramp (MP 13.96)	0	0	0	0

As Table 7 shows, there were very few crashes during the three year study period along any of the freeway segments. The segment crash rates for both northbound and southbound I-5 is 0.01. The comparable statewide average is 0.50. As would be expected, neither northbound nor southbound directions have SPIS scores high enough to rank in the Top 10%.

Segment crash rates were calculated and examined along all roadway segments in the analysis area and compared to statewide averages for comparable roadway segments. All segment crash rates – excluding intersection areas – were well below the statewide averages, which indicated that there are no significant safety issues along any of the roadway segments. A large majority of the documented crashes occurred in the vicinity of intersections.

7.5 Crash Analysis Conclusions

Overall, there are no apparent crash patterns at any of the study area intersections, and no single intersection shows a significant safety problem. However, taken together, the crashes on Ashland Street between the I-5 northbound and southbound ramps warrant a SPIS score that ranks in the top 10% statewide. A total of eleven crashes were reported during the three-year study period on Ashland Street between Clover Lane and the I-5 southbound ramps. These crashes resulted in a total of six injuries. The primary types of crashes documented along this segment are turning and rear end. Rear end crashes are often caused by driver inattention in congested conditions, while the turning crashes may be a symptom of drivers accepting small gaps at unsignalized intersections.

One change that would likely alter the crash statistics would be installing traffic signals at the ramp terminals. Traffic signals would create gaps in the traffic stream to allow vehicles to make turning maneuvers onto Ashland Street and the interchange ramps. Approval of new signals requires that certain traffic volume warrants are met. The Traffic Analysis Report will evaluate preliminary signal warrants at the ramp terminals to determine their appropriateness.

Another strategy that can enhance safety in the interchange vicinity would be the consolidation or elimination of access points. When driveways are spaced too closely the number of potential conflicts increase dramatically and capacity is diminished. Specific access management recommendations are beyond the scope of this report, but an access management plan will be a major component of the future IAMP.

Finally, the geometric improvements associated with construction of a new interchange overcrossing of I-5 at Ashland Street is another improvement that could improve safety conditions by improving sight distance.

The crash history at South Valley View Road and Lowe Road do not indicate a significant safety problem. However, to ensure long term safe and efficient operation of the North Ashland Interchange, it is recommended that future interchange improvements include a realignment of Lowe Road so that it intersects with South Valley View Road at a point further south. As previously noted, the future IAMP will contain more detail with respect to access management recommendations.

8 Future Volume Development

In addition to current year conditions, traffic operations analysis were conducted under year 2010 and 2030 traffic volume scenarios to determine future needs and evaluate various build alternatives. Traffic volumes have been developed and are discussed below.

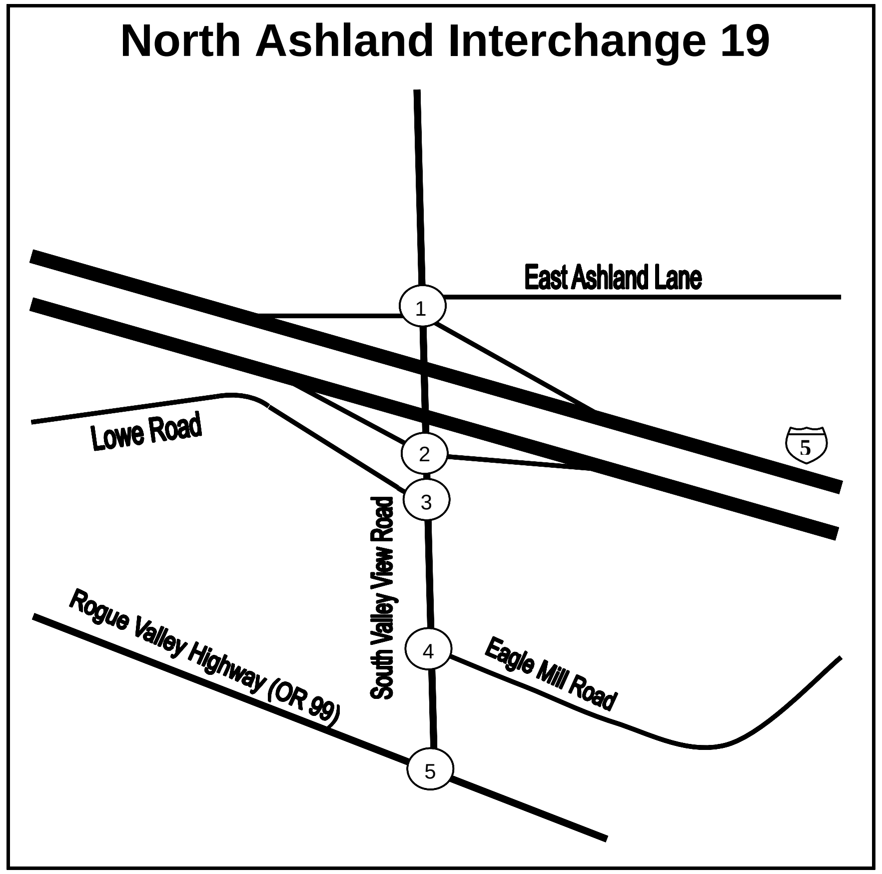
Future traffic volumes were developed by applying an annual growth rate to existing 30th hour volumes. An annual growth rate was determined from the Primary 2024 Future Volume Table, which is published each year by TPAU and uses traffic volumes from previous years to project future volumes. The table projects year 2024 Average Daily Traffic (ADT) at all count locations listed in the Transportation Volume Tables. Annual growth rates at locations within the analysis area were found to be between 2.0% and 2.3%, with an average of 2.1%. One location on OR 99 was found to have a growth rate of 1.3%. Therefore, a linear growth rate of 2.1% was applied to all 30th hour volumes in the analysis area, with the exception of volumes on OR 99 for which a growth rate of 1.3% was applied. Use of this methodology was approved by TPAU.

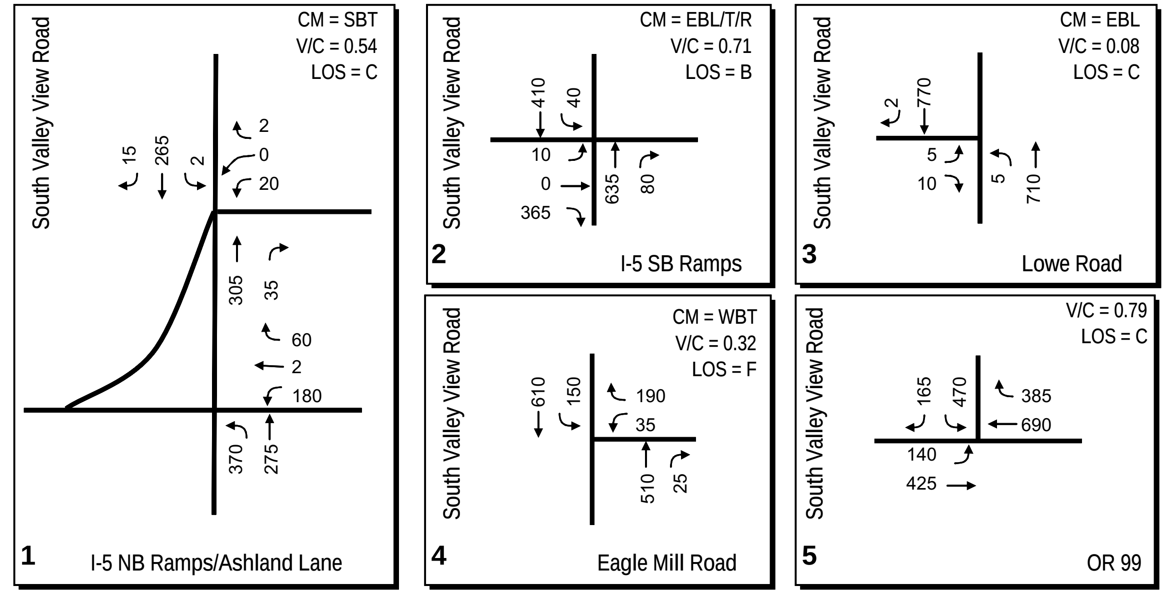
Another method for developing future volumes is a travel demand model. A Rogue Valley MPO travel demand model currently exists, but does not include the Ashland area. A new model that includes the Ashland area is currently under development, but will not be operational until late 2006. The schedule constraints of this project preclude use of the RVMPO model for future volume development.

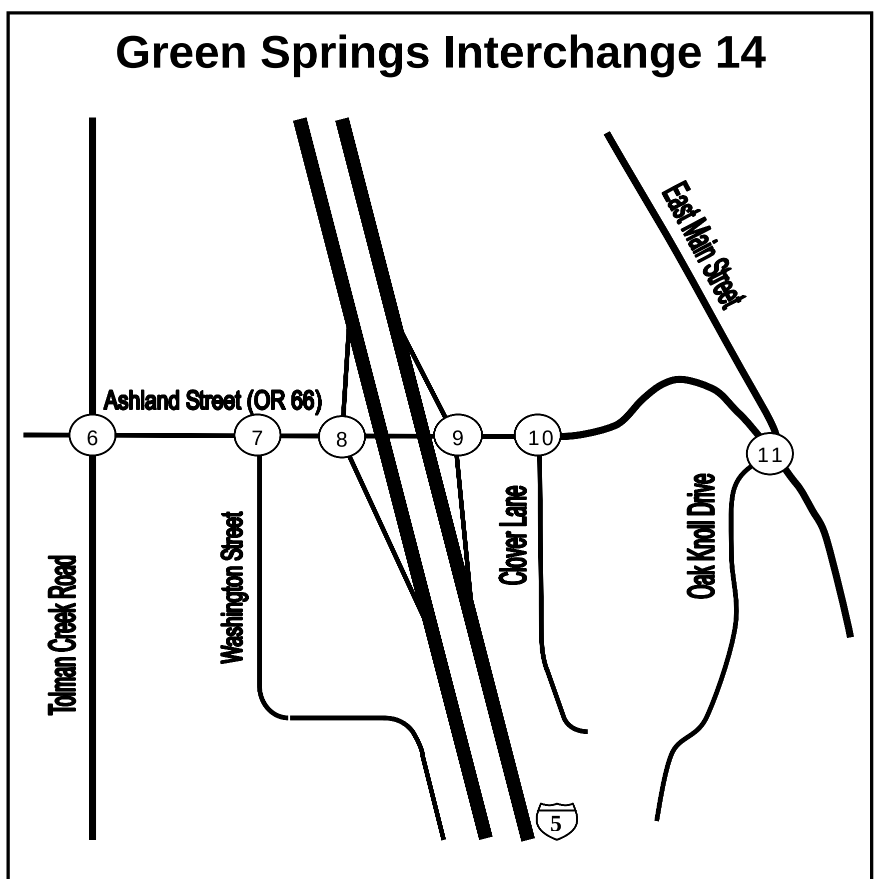
Figures 3 and 4 illustrate year 2010 and 2030 design hour volumes, respectively, balanced between intersections. The figures also show key operations analysis results that will be described in greater detail in the following section. A table showing future year design hour traffic volumes is provided in Appendix B.

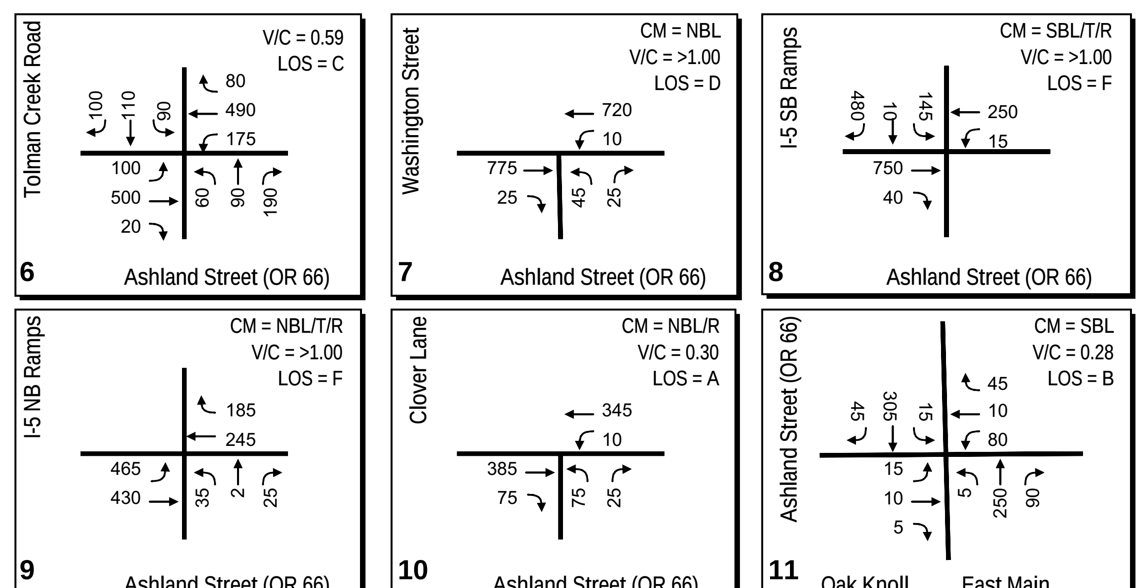
9 Future Traffic Operations Analysis – No-Build Scenario

Operations analyses were conducted at all of the analysis area intersections under no-build conditions and two design hour scenarios: year 2010, which is the estimated year of build; and year 2030, which is the plan horizon year. Table 8 summarizes the v/c and LOS results from these analyses, plus year 2006 30th highest hour analyses. Table 8 also presents agency operational standards to enable comparison with intersection results. Table 9 summarizes queuing on critical approach legs at the same intersections. Bold numbers in the tables represent v/c ratios that exceed the OHP mobility standards and approaches with long queues.









NOT TO SCALE

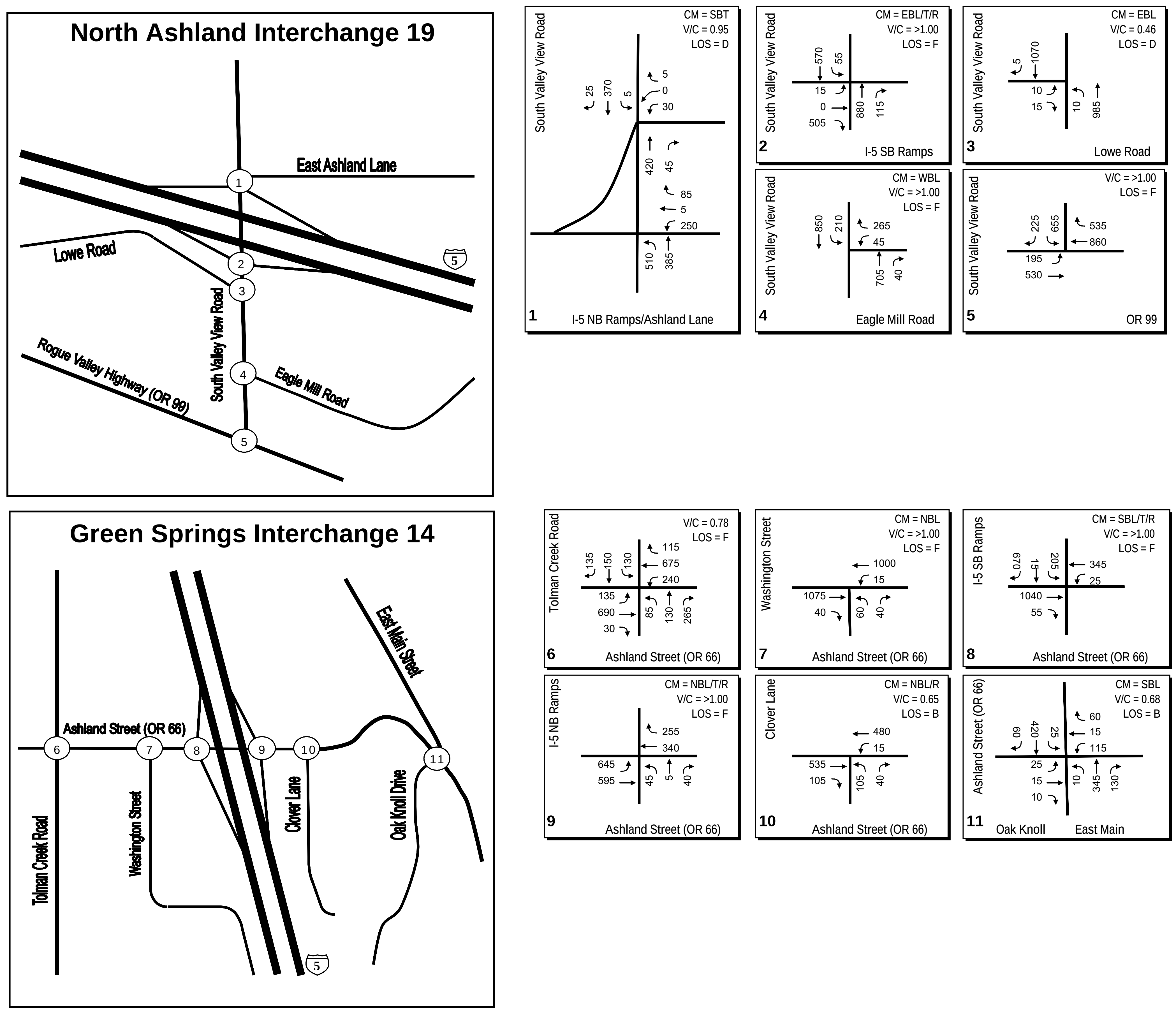
LEGEND

(11) = Intersection number
123 = Turning movement volume
↰ = Allowable turning movement volume
CM = Critical movement (TWSC)
V/C = Intersection volume to capacity
Critical volume to capacity (TWSC)
LOS = Intersection Level of Service
Critical Level of Service (TWSC)
TWSC = Two-way stop-controlled

FIGURE 3

Year 2010 No-Build
Design Hour Volumes
Traffic Operations Analysis Summary

I-5 Interchanges 14 & 19



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NOT TO SCALE

LEGEND

(11) = Intersection number
123 = Turning movement volume
↰ = Allowable turning movement volume
CM = Critical movement (TWSC)
V/C = Intersection volume to capacity
Critical volume to capacity (TWSC)
LOS = Intersection Level of Service
Critical Level of Service (TWSC)
TWSC = Two-way stop-controlled

FIGURE 4

Year 2030 No-Build
Design Hour Volumes
Traffic Operations Analysis Summary

I-5 Interchanges 14 & 19

Table 8. Existing and Future Year No Build Traffic Operations Analysis Results

Intersection	Critical Movement	2006 30th HV v/c	2006 30th HV LOS	2010 Design Hour v/c	2010 Design Hour LOS	2030 Design Hour v/c	2030 Design Hour LOS	OHP Std.[1]	HDM Std.[2]	J.C. Std.[3]
Green Springs Interchange 14 Analysis Area										
Ashland St (OR 66) & Tolman Creek Rd	n/a[4]	0.52	C	0.59	C	0.78	F	0.90	0.85	-
Ashland St (OR 66) & Washington St	NBL	0.69	C	**>1.00**	**D**	**>1.00**	**F**	0.90	0.85	-
Ashland St (OR 66) & I-5 SB Ramps	SBL/T/R	**>1.00**	**F**	**>1.00**	**F**	**>1.00**	**F**	0.85	0.75	-
Ashland St (OR 66) & I-5 NB Ramps	EBL/T	0.45	C	0.50	D	0.82	F	0.85	0.75	
	NBL/T/R	**>1.00**	**F**	**>1.00**	**F**	**>1.00**	**F**	0.85	0.75	-
Ashland St (OR 66) & Clover Lane	NBL/R	0.26	A	0.30	A	0.65	B	0.90	0.85	-
Ashland St (OR 66) & E. Main/Oak Knoll	SBL[5]	0.24	B	0.28	B	0.68	B	0.90	0.85	-
North Ashland Interchange 19 Analysis Area										
S. Valley View Rd & OR 99	n/a[4]	0.72	C	0.79	C	**>1.00**	**F**	0.90	0.85	0.95
S. Valley View Rd & Eagle Mill Road	WBL	0.23	D	0.32	F	**>1.00**	**F**	-	-	0.95
S. Valley View Rd & Lowe Road	EBL	0.07	D	0.08	C	0.46	D	-	-	0.95
S. Valley View Rd & I-5 SB Ramps	EBL/T/R	0.62	A	0.71	B	**>1.00**	**F**	0.85	0.75	0.95
S. Valley View Rd & I-5 NB Ramps	SBT	0.48	B	0.54	C	**0.95**	**D**	0.85	0.75	0.95
S. Valley View Rd & E. Ashland Ln	WBL/R	0.05	C	0.06	D	0.12	F	-	-	0.95

Notes:

1. 1999 Oregon Highway Plan Mobility Standard (Table 6)
2. 2003 ODOT Highway Design Manual Mobility Standard (Table 10-1)
3. Operational standards for Jackson County roadway facilities (Source: Jackson County Transportation System Plan)
4. Signalized intersection. LOS and v/c are for overall intersection.
5. Refers to left turn from Main Street to Ashland Street.

Table 9. Existing and Future Year No Build 95th Percentile Queues

Intersection	Movement	95th Percentile Queue (ft) Existing	2010	2030
Green Springs Interchange 14 Analysis Area				
Ashland St (OR 66) & Tolman Creek Rd	EBL	125	150[1]	200[1]
	WBL	150[1]	150[1]	125[1]
	NBL	100[1]	100[1]	150[1,2]
	SBL	100[1]	125[1]	125[1,2]
Ashland St (OR 66) & Washington St	WBL	25	25	25
	NBL	75[1]	75[1]	50[3]
Ashland St (OR 66) & I-5 SB Ramps	WBL/T	50	50	75
	SBL/T/R	**675[2,5]**	**850[2,5]**	**650[2,5]**
Ashland St (OR 66) & I-5 NB Ramps	EBL/T	**600[4]**	**825[4]**	**4425[4]**
	NBL/T/R	**375**	**625**	**675[5]**
Ashland St (OR 66) & Clover Ln	WBL/T	50	25	50
	NBL/R	75	75	100
Ashland St (OR 66) & E. Main/Oak Knoll	NBL/T/R	50	50	50
	SBL	75[1]	75[1]	75[1]
North Ashland Interchange 19 Analysis Area				
S. Valley View Rd & OR 99	SBL/R	**775[4]**	**850[4]**	**1785[4]**
	EBL	175[1]	175[1]	250[1]
	WBR	125[1]	125[1]	225[1,6]
S. Valley View Rd & Eagle Mill Rd	WBL/R	175	**550**	**1575[7]**
	SBL	75	100[1]	150[1]
S. Valley View Rd & Lowe Rd	EBL/R	25	50	50
	NBL/T	25	50	75
S. Valley View Rd & I-5 SB Ramps	EBL/TR	125	225	**1575[5]**
	SBL/T	75	100	100
S. Valley View Rd & I-5 NB Ramps	WBL/T/R	175	**275**	**1550**
	SBT	175[4]	200[4]	**1350[4]**
S. Valley View Rd & E. Ashland Ln	WBL/R	50	50	**475**

Notes:
1. Storage bay at or above capacity.
2. Queue extends beyond extent of SimTraffic model link. Therefore, queue may be longer than shown.
3. Because of standing queue on eastbound Ashland Street, right turning vehicles block access for left turning vehicles to northbound left turn bay.
4. Queue extends into adjacent intersection(s).
5. Queue extends into ramp deceleration area.
6. Queue spills over into adjacent lane creating a blocked condition.
7. Left turns are blocked due to queuing along South Valley View from OR 99 intersection.

Results for future year no-build traffic operations analysis showed marked degradation at most key intersections.

At the Green Springs Interchange 14, where existing 30th highest hour critical v/c ratios at both ramps terminal intersections are greater than 1.00, future operations are expected to be characterized by more delay and queuing on intersection and ramp approaches. Without mitigation, queuing on the southbound exit ramp will frequently extend into the deceleration area of the ramp, creating a potential safety problem. The increased demand for the eastbound left turn from Ashland Street to the I-5 northbound entrance ramp will generate queues that extend through and well beyond the adjacent southbound ramp terminal intersection.

No-build traffic operations at both ramp terminals of the North Ashland Interchange 19 are expected to meet mobility standards under year 2010 conditions. However, under 2030 traffic volume conditions, the critical v/c is calculated to be 0.95, which exceeds the OHP mobility standard if no mitigation is implemented. The v/c at the southbound ramp terminal is calculated to be greater than 1.00 prior to the plan horizon year. Queuing on the exit ramp will be significant and will extend into the deceleration area of the ramp, creating a potential safety problem.

The v/c ratio at the intersection of South Valley View Road and OR 99 under no-build conditions is expected to be greater than 1.00 prior to the planning horizon year. If South Valley View Road remains a two-lane road, queuing on the southbound approach will be substantial and is likely to impact operations as far north as the North Ashland Interchange.

As noted in the discussion on existing conditions, an intersection cannot actually operate at a v/c of greater than 1.00. Future v/c ratios greater than 1.00 indicate that anticipated demand will exceed capacity.

10 Signal Warrant Analysis

The need for traffic signals at intersections is established by evaluating existing and projected traffic conditions against traffic signal warrants contained in the *2003 Manual on Uniform Traffic Control Devices* (MUTCD). The MUTCD provides eight signal warrants that consider different conditions under which a new signal may be warranted. The most commonly applied signal warrants are based on traffic volumes, although the MUTCD contains signal warrants based on crash experience, coordinated signal systems, and warrants for signals at pedestrian and school crossings.

MUTCD Warrant 1, Eight-Hour Vehicular Volume, was analyzed at each ramp terminal intersection for existing conditions (year 2006). Warrant 1 requires that certain volume thresholds are met for each of any eight (8) hours in an *average* day, and has two condition requirements. Condition A is met when a high volume of traffic is present on an intersecting roadway, and Condition B is met when mainline volumes are so high that traffic on the minor street experiences excessive delay. MUTCD Warrant 1 is met when *either* Condition A or Condition B are met.

For years 2010 and 2030 conditions TPAU preliminary traffic signal warrants were evaluated. The TPAU preliminary warrants are based on MUTCD warrants, but require less data. TPAU developed these warrants for the purpose of projecting future traffic signal needs.

OAR 731-020 provides specific guidance on the application of signal warrants in Oregon. Meeting signal warrants does not guarantee that a signal shall be installed. For installation of signals, OAR 731-020-0490 (1) provides that traffic volumes sufficient to meet signal warrants must be met within three years of signal installation. Since the planning horizon used in this analysis is 2030, which greatly exceeds the three year specification in OAR 731-020, the preliminary traffic signal warrants rather than the MUTCD signal warrants are appropriate for this planning analysis.

As specified in OAR 734-020-0460 (2), before a signal can be installed a comprehensive traffic engineering study must be undertaken at a future date when traffic volumes actually meet, or can be predicted to occur within three years that meet MUTCD warrants. A The traffic signal investigation must include an evaluation of traffic conditions and physical characteristics of the proposed traffic signal location. OAR 734-020-460 (4) requires additional analyses that includes evaluations of speeds, highway type, grades, sight distance, existing level of service and conflicting accesses. OAR 734-020-0480 requires progression analyses to be conducted to evaluate the impact of a new signal on the existing or future signal system. The State Traffic Engineer will make the final decision on the installation of a signal.

Traffic signal warrants are assessed using average traffic volumes rather than the 30th highest hour volumes used for roadway and intersection design. To calculate average traffic volumes, adjustment factors were applied to the existing 16-hour traffic counts taken in May 2006. The adjustment factors were equivalent to the values shown in the 'May 15' column of the Seasonal Trend Table, reproduced in Table 3 of this report (0.98345 for interchange ramp volumes and 0.9469 for cross street volumes).

Signal warrant analysis results are summarized in Table 10. Complete signal warrant worksheets are contained in Appendix E.

Table 10. Signal Warrant Analysis Summary

Location	Year	Warrant 1, Eight-Hour Vehicular Volume: Condition A	Warrant 1, Eight-Hour Vehicular Volume: Condition B	TPAU Preliminary Traffic Signal Warrant: Case A	TPAU Preliminary Traffic Signal Warrant: Case B
Green Springs Interchange 14					
Ashland St (OR 66) & I-5 SB Ramps	2006	MET	NOT MET		
	2010			NOT MET	NOT MET
	2030			MET	MET
Ashland St (OR 66) & I-5 NB Ramps	2006	NOT MET	NOT MET		
	2010			NOT MET	NOT MET
	2030			NOT MET	NOT MET
North Ashland Interchange 19					
S. Valley View Road & I-5 SB Ramps	2006	NOT MET	NOT MET		
	2010			NOT MET	NOT MET
	2030			NOT MET	NOT MET
S. Valley View Road & I-5 NB Ramps	2006	NOT MET	NOT MET		
	2010			NOT MET	NOT MET
	2030			NOT MET	NOT MET

The signal warrant analyses showed that only the intersection of Ashland Street at the I-5 southbound ramp terminals meet MUTCD and TPAU preliminary signal warrants. However, most of the interchange build options evaluated in this report include signalized ramp terminals, and signalization is likely to be required at some time in the future to prevent failing intersection operations and excessive queuing on the exit ramps. At a minimum, junction boxes and conduit should be included in the design of the Bridge Bundle #314 improvements to accommodate future signals, regardless of whether the ramp terminals are signalized at this time. Traffic signal warrant analysis results for each intersection is discussed below.

10.1 Ashland Street (OR 66) & I-5 Southbound Ramps

MUTCD Signal Warrant 1, Condition A, is met under existing traffic volume conditions, and TPAU Preliminary Warrant Condition B is met under future conditions. This intersection has significant left and right turning volumes on the southbound exit ramp and high cross street volumes on Ashland Street. The high cross street volumes limit the availability of gaps and result in failing v/c conditions with significant queuing that is calculated to spill into the deceleration area of the southbound exit ramp. Because of the substantial volumes of left turning vehicles and their potential for interfering with right turning vehicles, 25% of the exit ramp right turning volume was included in the minor street approach volume when conducting the MUTCD warrant analysis. A right turn reduction was also applied for the TPAU preliminary signal warrant analysis consistent with the methodology presented in the Analysis Procedures Manual.

10.2 Ashland Street (OR 66) & I-5 Northbound Ramps

MUTCD Warrant 1 is not met at this intersection under existing conditions, and the TPAU preliminary warrant is not met through the planning horizon. However, as noted in the previous section, this intersection currently experiences significant operational problems that are expected to worsen through the planning horizon. The high volumes on Ashland Street allow few gaps in the traffic stream for left turning vehicles on the northbound exit ramp. Furthermore, the high volumes of eastbound left turning vehicles and the lack of gaps afforded to them generate queues that extend through the southbound ramp terminal intersection. The queue spillover from the northbound ramp terminal exacerbates operational problems at the southbound ramp terminal.

Although signal warrants based on traffic volumes are met only at the southbound ramp terminal, signalization of only the southbound ramp terminal without also signalizing the northbound ramp terminal would counteract the operational benefits of signalization at the southbound ramp terminals.

Other warrants, particularly Warrant 8, Roadway Network, or Warrant 7, Crash Experience (location is a top 10% SPIS site), may provide justification for installation of a traffic signal, and installation of a signal may well be necessary to solve the operational problems including queuing and v/c problems related to the eastbound to northbound left turn.

All build options for Interchange 14 include signalized intersections at the northbound and southbound ramp terminals.

10.3 North Ashland Interchange 19

MUTCD Warrant 1 is not met for existing conditions, nor are the TPAU preliminary signal warrants met for future conditions at either the northbound or southbound ramp terminals. Despite these results, signalization should remain an option at the interchange ramp terminals, and a future warrant analysis may prove that traffic signals are appropriate. A number of factors must be considered, and sound engineering judgment must be employed, when conducting a signal warrant investigation. Some items to consider for future signal warrant investigations are discussed below.

For major streets with speeds of 40 mph or greater, or in isolated communities with populations of less than 10,000 residents, MUTCD methodology allows volume thresholds for meeting traffic signal warrants to be reduced to 70 percent of standard warrant values. This reduced volume threshold also applies to TPAU preliminary warrants. The posted speed of South Valley View Road through the interchange is 35 mph. Directly north of the northbound ramp terminal the posted speed changes to 45 mph. If the documented 85th percentile speed is equal to or greater than 40 mph, then it could be suggested that the reduced warrant threshold would be appropriate. Application of the 70 percent values based on the isolated community standard may or may not be appropriate depending on the interpretation of what constitutes a rural or urban area. The Interchange 19 area has a rural character and is located outside of the Ashland Urban Growth Boundary. However, the interchange is located inside the Rogue Valley MPO boundary, and the traffic composition is largely regular commuter traffic within the region. Notwithstanding the above discussion, neither the northbound nor southbound ramp terminals appear to meet 70 percent of the standard TPAU preliminary signal warrants through the planning horizon.

At times it is appropriate to include some right turns in the volume calculation. However, at both Interchange 19 ramp terminals none of the right turns were counted in the volume calculations for preliminary signal warrant analyses. The TPAU Analysis Procedures Manual provides specific guidelines regarding application of right-turn reductions. These guidelines dictate when to apply reductions and, when applicable, the portion of right turns that should be included. The warrant analyses conducted for this Traffic Analysis Report adhered to the TPAU guidelines. By excluding all right turns, the major movement at the southbound ramp terminal (i.e., southbound I-5 exit ramp to southbound South Valley View Road) is not reflected in the intersection volumes. This, in turn, reduces the likelihood of meeting signal warrants. The ability of motorists to turn right depends on provision of separate left turn and right turn lanes. Should queues of left-turning vehicles grow long enough that they inhibit the ability of right-turning vehicles to get to the intersection, some of the right-turning vehicles must be counted in the ramp approach volumes when assessing volume warrants.

Failure to meet preliminary signal warrants at this time should not be viewed as a definitive disqualification for signalization as a possible future solution. The traffic operations analysis contained in Section 9 of this report shows both ramp terminals failing to meet operations standards within the planning horizon under their current, unsignalized conditions. Signalization may in fact be the appropriate solution as traffic volumes increase and interchange operations deteriorate.

Should Region 3 decide to pursue signalization of one or both of the ramp terminals, the requisite traffic engineering investigation should consider the possibility of including some right turns in the intersection volumes, as well as evaluating whether using 70 percent of standard warrants are appropriate. If it is determined that either one of these modifications should be made, signal warrants may be found to be met at one or both ramp terminals.

All Interchange 19 Build Options except 19-1 include signalization at the southbound ramp terminals. Build Option 19-3 includes a loop ramp at the northbound ramp terminal in lieu of a traffic signal. A loop ramp would remove the heaviest movement from the intersection and would eliminate the need for a signal at the northbound ramp terminals through the planning horizon.

11 Future Traffic Operations Analysis – Build Options

Each of the build options was analyzed to determine the intersection v/c, LOS and 95th percentile queue. The traffic operations analysis results contained in this report, in combination with relevant cost and design data, should be employed to select an appropriate build option. Once a build option is selected, the designer should consider the traffic operations analysis results to determine appropriate design attributes for the selected build option, such as queue storage and ramp lengths. This report summarizes operations results and 95th percentile queue lengths, but does not present specific recommendations for design features, as this is the purview of the designer.

All analysis described in this section was performed using the Synchro and SimTraffic programs. As noted in Section 5.3, traffic conditions that would warrant the use of the CORSIM program (e.g., field observed lane imbalances) were not present.

Four build options were developed for the Green Springs Interchange 14, and five build options were developed for the North Ashland Interchange 19. The objectives of the build options were to: 1) address existing and anticipated operational deficiencies; and 2) help ODOT and OBDP determine the extent of immediate improvements associated with the OTIA III Bridge Bundle #314 project.

All build options assumed that two lanes were provided on the exit ramp approaches to the ramp terminals to accommodate separate left and right turns. The required length of queue storage varies depending on the operational characteristics associated with each build option.

Like the no-build analyses, the Synchro and SimTraffic software tools were used to evaluate traffic operations for each of the build options. When an alternative included signalization, signal timing was coordinated between the intersections and optimized to maximize capacity within the limits of usual operational parameters. An attempt was made to approximate realistic signal timing consistent with ODOT signal policy.

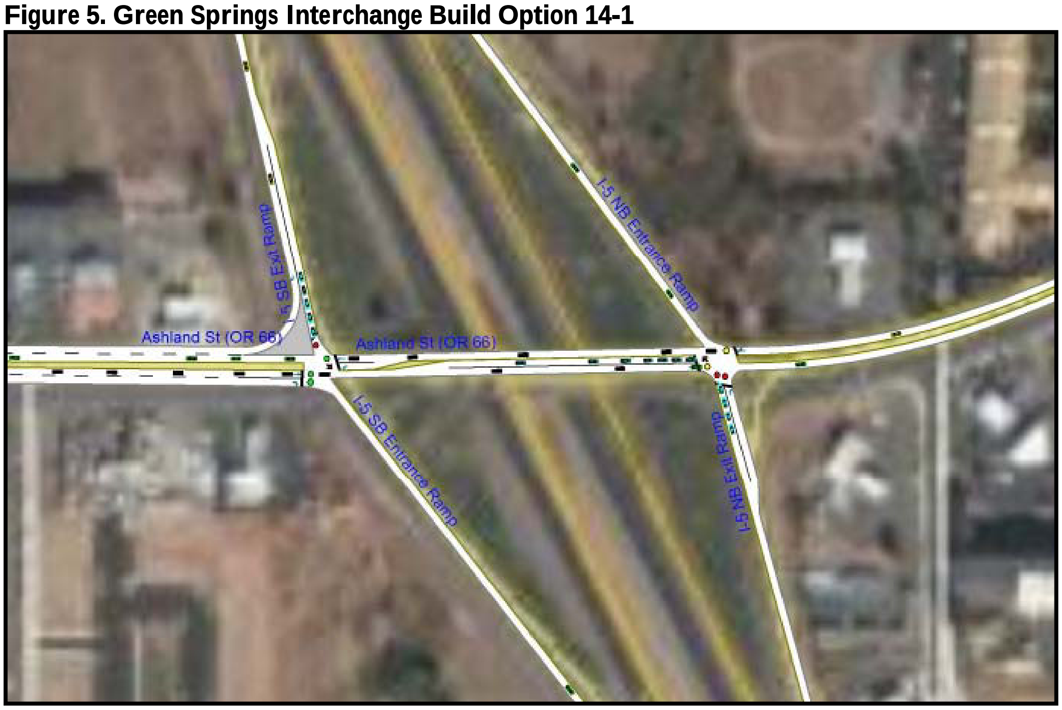
A detailed discussion of each build option, including analysis results, is provided below. Figures 5-13 illustrate each option, and Tables 11-14 list summary analysis results for the options.

11.1 Green Springs Interchange Build Option 14-1

Build Option 14-1 consists of a three lane bridge over I-5 with traffic signals at both northbound and southbound ramp terminals. The right turning movement from the I-5 southbound exit ramp is channelized with a dedicated receiving lane on westbound Ashland Street as shown on Figure 5.

Operations analysis resulted in intersection v/c ratios of 0.83 for the southbound ramp terminal and 0.93 for the northbound ramp terminal under year 2030 conditions, neither of which meet the HDM mobility standard. A three-lane bridge does not appear to provide adequate length for back-to-back left-turn lanes at the southbound and northbound ramp terminals, plus the needed taper between them. Furthermore, queuing analysis indicates that the single eastbound left turn lane is not sufficient to accommodate the expected 95th percentile queues. For these reasons, this build option does not appear to be adequate to accommodate design hour traffic volumes for year 2030.

Figure 5. Green Springs Interchange Build Option 14-1



Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.2 Green Springs Interchange Build Option 14-2

Build Option 14-2 consists of a four-lane bridge and like Build Option 14.1, includes traffic signals at both ramp terminal intersections. Build Option 14-2 is shown in Figure 6. Four lanes accommodate dual eastbound left turn lanes for the high-volume eastbound Ashland Street to northbound I-5 movement; the left-hand eastbound left-turn lane is located back-to-back with the westbound left-turn lane.

Note that dual left turn lanes require two receiving lanes on the northbound I-5 entrance ramp. The dual receiving lanes should be carried as far down the ramp as possible, but may merge into one lane prior to joining the I-5 mainline. The HCM does not provide a methodology to assess merging of two single-lane facilities or a lane drop from two lanes to one. However, the design hour volumes on the northbound entrance ramp are far below the capacity of the ramp, so it can accommodate a merge from two lanes to one lane prior to joining the mainline. The more critical factor is whether the I-5 mainline can absorb the design hour ramp volumes if they join the interstate in a single lane as opposed to dual lanes. The HCM provides a methodology for assessing merge movements on freeways. The analysis is applicable to either single or dual-ramps merging with multi-lane freeway sections. HCM merge analysis was conducted assuming design hour volumes and a single merge lane, and it showed that design year traffic volumes on the ramp could be accommodated by a single-lane ramp. Analysis details are contained in Appendix D.

Build Option 14-2 also includes a channelized right turn lane with a dedicated receiving lane for southbound traffic exiting I-5 onto westbound Ashland Street.

Analysis results show that v/c ratios at both ramp terminal intersections would meet HDM mobility standards. However, existing geometric constraints (i.e., the distance between ramp terminals) may result in a westbound left-turn lane and taper between the eastbound and westbound left-turn lanes that do not meet ODOT design standards. Preliminary measurements indicate that ODOT design standards require longer minimum left turn storage lane and taper lengths than what could be provided in Build Option 14-2. A four-lane bridge option does not appear to simultaneously provide for: 1) sufficient storage for the 95th percentile queues for the design hour volume on the eastbound approach to the northbound I-5 ramp terminal; 2) a standard 100-foot storage bay for the westbound approach at the southbound ramp terminal; 3) a standard taper associated with a 40-mph design speed; and 4) a distance of approximately 535 feet between ramp terminals. It is important to note that this effects only the inside left turn lane of the dual lanes from eastbound Ashland Street to northbound I-5; the outside left turn lane queue storage is determined only by the distance between the ramp terminals. The fact that only one of the two turn lanes is limited by this configuration may be an important consideration when selecting the preferred design.

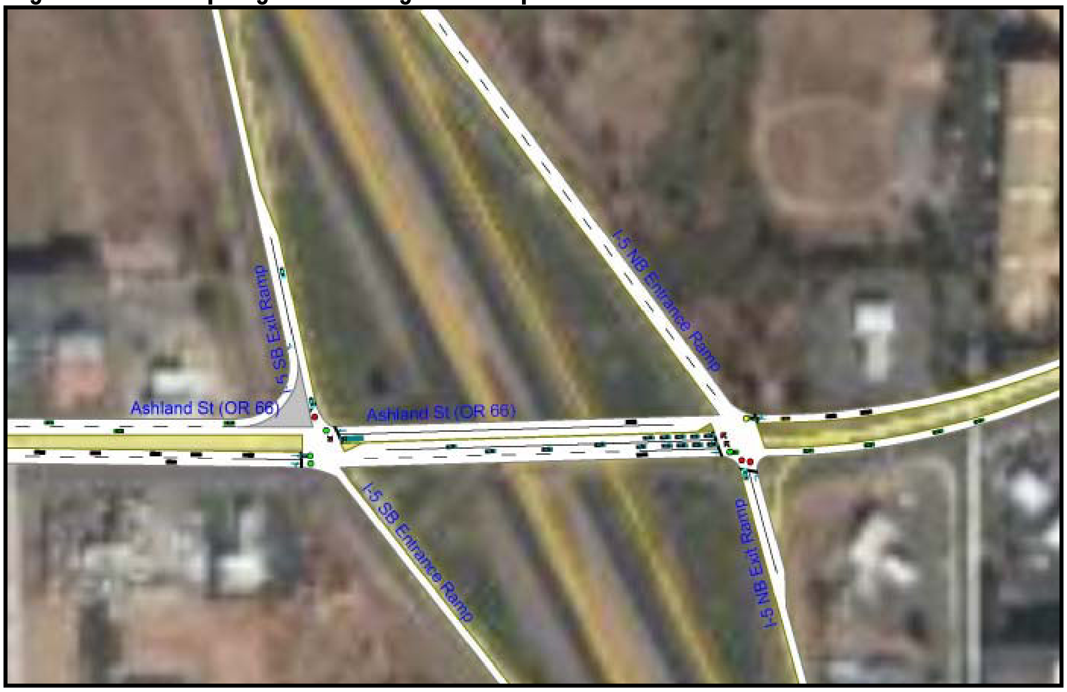
Figure 6. Green Springs Interchange Build Option 14-2

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.3 Green Springs Interchange Build Option 14-3

Build Option 14-3 features a five-lane bridge. Build Option 14-3 contains all the attributes of Build Option 14-2, including signalization at the northbound and southbound ramp terminals, dual eastbound left-turn lanes, and a dedicated receiving lane on westbound Ashland Street for right-turning vehicles from the southbound exit ramp. This option adds an additional lane between the ramp terminals, resulting in a five-lane bridge. The lane configuration between the ramp terminals consists of single through lanes in both the eastbound and westbound directions, and dual eastbound left-turn lanes located side-by-side with a single westbound left-turn lane. Operationally, the ramp terminals associated with this option would function identically to those of Build Option 14-2. The side-by-side left-turn lane configuration would simply allow all ODOT design standards to be met on the bridge since left-turn queue storage would be limited only by the distance between the ramp terminals and no tapers would be required. The implications of a five-lane bridge have not been assessed with regard to the impact on the sections of Ashland Street to the east and to the west of the ramp terminals. These issues are appropriately dealt with by the roadway designer.

Figure 7. Green Springs Interchange Build Option 14-3



Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.4 Green Springs Interchange Build Option 14-4

In response to Agency and OBDB requests, traffic operations were analyzed at the Green Springs Interchange for a single-point urban interchange, or SPUI. This interchange type brings all ramp terminals to a single intersection, and is shown in Figure 8. Like the other build options, traffic exiting from southbound I-5 would have a channelized right turn with a dedicated receiving lane on westbound Ashland Street. Traffic operations analysis results show that the SPUI would operate with a v/c of 0.71 under design year traffic volumes.

Note that the traffic analysis was performed using a SPUI concept with the minimum number of lanes required to achieve acceptable ODOT mobility standards. Several of the approaches to the SPUI modeled for Interchange 14 include only a single approach lane. Typically SPUIs have multiple lanes for each approach. The fact that single lanes appear to provide a v/c ratio meeting mobility standards should not be taken as a recommendation that this is an appropriate design. Adding additional lanes for the approaches would improve the v/c ratio and may be necessary to meet other applicable design standards. Changes to the configuration may also alter the queue storage requirements reported in Table 12.

Figure 8. Green Springs Interchange Build Option 14-4: SPUI

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.5 Analysis of AM Peak Hour at Interchange 14

All analysis descriptions and summaries described above pertain to the PM peak hour. Total entering volumes at all but one of the analysis area ramp terminal intersections were higher during the PM peak hour than during the AM peak hour. However, traffic volumes at the Interchange 14 southbound ramp terminal were roughly equivalent during the AM and PM peak hours. As a check, traffic operations and queuing analysis was performed at Interchange 14 for Build Options 14-2 and 14-3 under projected year 2030 AM peak hour traffic volumes. The purpose of this analysis was to ensure that interchange design decisions based on design hour (PM peak hour) traffic operations were adequate to accommodate the volumes and directional distribution associated with the AM peak hour as well as the PM peak hour. The analysis showed that both Build Options 14-2 and 14-3 could adequately accommodate design year AM peak hour traffic volumes. Detailed analysis results are contained in Appendix C.

11.6 North Ashland Interchange Build Option 19-1

Build Option 19-1 improvements are limited to a three lane bridge over Interstate 5 and two lanes for portions of the northbound and southbound I-5 exit ramp approaches to South Valley View Road to provide separate left turn and right turn lanes. No traffic signals are included. The unconventional intersection control at the northbound ramp terminal is retained (free left turn from northbound South Valley View Road to the northbound I-5 entrance ramp). A portion of exit ramp approaches to ramp terminals are widened to two lanes to provide separate lanes for left turns and right turns. Build Option 19-1 is shown in Figure 9.

Analysis results show that critical v/c ratios would not meet HDM mobility standards under year 2030 design hour traffic volume conditions. Build Option 19-1 provides minimal operational benefits over the current interchange configuration.

Conventional stop control was also examined as an alternative to the existing intersection control at the northbound ramp terminal. Conventional control would consist of free movements on the northbound and southbound South Valley View Road approaches and stop control on the northbound I-5 exit ramp approach. Analysis showed that conventional intersection control would result in worse operations than existing intersection control under design hour conditions. For conventional control, the v/c was calculated to be greater than 1.00 and with very long queues.

Figure 9. North Ashland Interchange Build Option 19-1

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.7 North Ashland Interchange Build Option 19-2

This build option consists of a three-lane bridge and signalization at the northbound and southbound I-5 ramp terminals. Portions of the northbound and southbound exit ramps would be widened to two lanes to provide for separate left and right turns on the exit ramps at each signalized ramp terminal. Build Option 19-2 is shown in Figure 10. Analysis results show that under year 2030 traffic volume conditions, the northbound ramp terminal intersection would meet HDM mobility standards, but the southbound ramp terminal would not. The predicted sub-standard operational performance at the southbound ramp terminal is caused by limited capacity to accommodate the right-turn movement from the southbound I-5 exit ramp to southbound South Valley View Road south of the interchange ramp terminal, not the number of lanes on the bridge. A three-lane bridge section appears to simultaneously provide sufficient storage for the 95th percentile queues for both northbound and southbound left turns using year 2030 design hour volumes while meeting ODOT design standards for left turn lane storage and lane taper lengths.

Figure 10. North Ashland Interchange Build Option 19-2

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.8 North Ashland Interchange Build Option 19-3

This option, shown in Figure 11, consists of a traffic signal at the southbound ramp terminal and a loop ramp in NE quadrant of interchange to accommodate the heavy left turn movement from northbound South Valley View Road to the northbound I-5 entrance ramp.

Analysis results show that the northbound ramp terminal would meet HDM mobility standards, but the southbound ramp terminal would not. The southbound ramp terminal analysis results are nearly identical to those of Build Option 19-2. Also, like in Build Option 19-2, the capacity limitation at the southbound ramp terminal is not associated with the number of lanes on the bridge nor the northbound loop ramp, but rather it is related to limited capacity for the right turn from the southbound I-5 exit ramp to southbound South Valley View Road. Option 19-4 provides a solution that allows the mobility standard to be met at the southbound ramp terminal that could be implemented in conjunction with a loop ramp at the northbound ramp terminal.

All-way stop control would be necessary at the northbound ramp terminal intersection to meet HDM mobility standards. Analysis of the intersection based on free movements on South Valley View Road with stop-control on the exit ramp resulted in a v/c of 0.99 on the stop-controlled I-5 northbound exit ramp.

Note that East Ashland Lane and Lowe Road are not shown intersecting South Valley View Road in their current locations near the interchange ramp terminals. Build Option 19-3 assumes that East Ashland Lane and Lowe Road would be realigned to intersect with South Valley View Road at points further from the ramp terminals, consistent with Division 51 access management standards for interchange areas. The designer may conclude that a relocation of East Ashland Lane would be necessary to accommodate a loop ramp. This may depend on the design speed of the ramp and various other geometric and right-of-way considerations. Further discussion regarding access management issues at both interchanges is contained in Section 12 of this report.

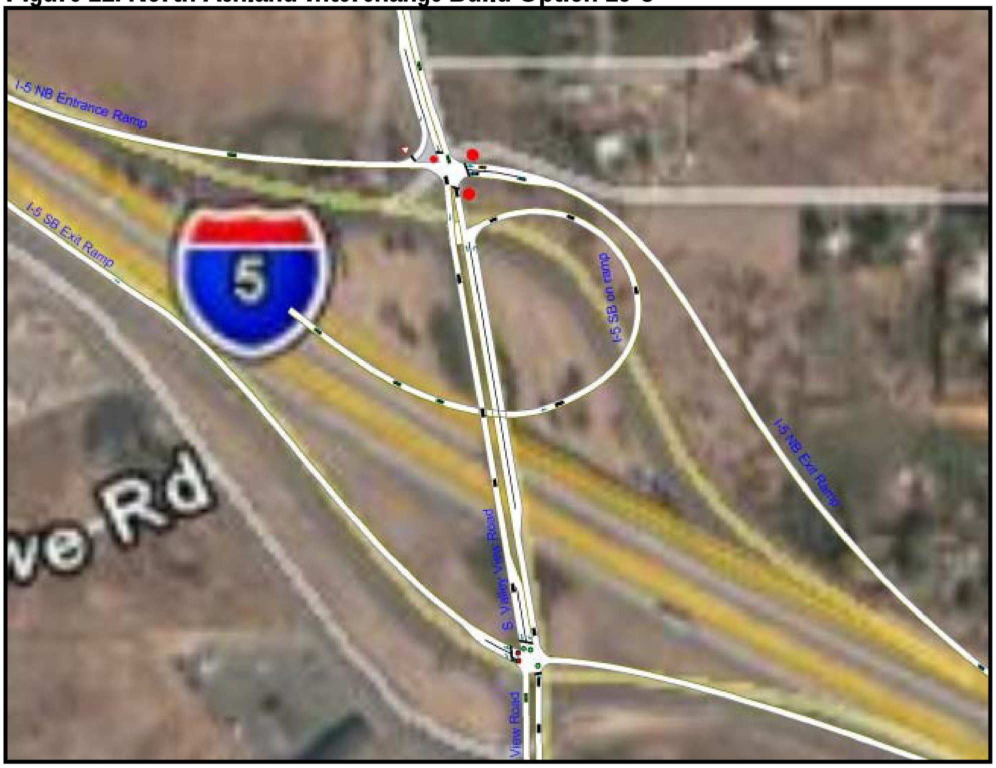
A loop ramp would likely require redesign of the existing northbound entrance ramp to allow adequate spacing between adjacent ramps or accommodating merging of the ramps prior to joining the northbound I-5 mainline. Operationally, either option (i.e., merging the ramps prior to joining I-5 or providing two separate merge points) would function adequately, as described below.

HCM merge analysis was performed to assess year 2030 design hour traffic operations of a scenario consisting of two entrance ramps accessing northbound I-5 at Interchange 19. For the analysis, the two ramps were assumed to be 1000 feet apart, which is the minimum ODOT spacing. The analysis showed that the ramp influence areas would operate with acceptable v/c ratios. Analysis worksheets are contained in Appendix D. The conclusion that operations would be acceptable assuming the two ramps are spaced 1000 feet apart should not be taken as a recommendation that this distance be used. ODOT's recommended or desirable spacing may be more appropriate based on other factors such as cost or the availability of right of way.

Another option would be to merge the ramps together prior to joining northbound I-5. As discussed in Section 11.2 of this report, the HCM does not provide a methodology to assess merging of two single-lane facilities or a lane drop from two lanes to one. However, the design hour volumes on the northbound entrance ramp are well below the capacity of a single ramp, so it could be expected accommodate a merge from two lanes to one lane prior to joining the mainline. The distance provided for merging of two single-lane ramps should be as long as possible.

If a second northbound entrance ramp at Interchange 19 is added, or the existing ramp is relocated, an Access Modification Request would need to be prepared to address the requirements of the Federal Highway Administration.

Figure 11. North Ashland Interchange Build Option 19-3



Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.9 Loop Ramp Design

As described in the previous section, Build Option 19-3 uses a loop ramp to accommodate the northbound traffic from South Valley View Road to northbound I-5. From a traffic operations standpoint, the loop ramp has certain advantages as well as certain disadvantages. The geometric design of the ramp is a function of the roadway designer and is typically undertaken during preliminary engineering. The following discussion may prove useful as the roadway designer considers various design options.

ODOT's *Highway Design Manual* lists the Ramp Design Speed in Table 9-3. For a highway design speed of 50 mph, the desirable ramp design speed is 45 mph and the minimum design speed of the ramp is 25 mph. For a highway design speed of 55 mph, the desirable ramp design speed is 45 mph and the minimum design speed of the ramp is 30 mph.

ODOT's *Highway Design Manual* lists Single Lane Ramp Capacity in Table 9-3. This table shows ramp design speed and level of service. This table, however, does not list either lane capacity or v/c ratios. The v/c ratio is the criterion upon which ODOT assesses compliance with the mobility standard. The *Highway Capacity Manual* does, however, provide information on capacity in Exhibit 25-3: Approximate Capacity of Roadway Ramps. This exhibit describes capacity of both single-lane and two-lane ramps according to the free-flow speed of ramps. The lowest ramp capacity listed is 1800 passenger cars per hour for a single lane ramp with a free-flow speed of less than 20 mph. At higher speeds, the ramp capacity increases. Since the projected year 2030 traffic volume on the optional loop ramp is approximately 550 vehicles during the peak hour, it is clear that ODOT's mobility standard is easily met even if one performs the calculation of the v/c ratio for the ramp based on a capacity of 1800 passenger vehicles per hour. Increasing the design speed and the free-flow speed of the ramp is not necessary to satisfy ODOT's mobility standard. The roadway designer need not be concerned about selecting a particular ramp design speed to meet traffic operations standards. Any speed above the minimum ramp design speed specified in ODOT's *Highway Design Manual* will satisfy mobility standards for a potential entrance ramp at the North Ashland Interchange. Other factors such as the available right of way are more likely to be important when selecting design speed for a potential loop ramp.

The roadway designer is also cautioned to consider that a lower speed on the ramp will also likely require a longer section for acceleration before the ramp joins the freeway main line. This factor may or may not be important in selecting the design speed for a potential loop ramp.

11.10 North Ashland Interchange Build Option 19-4

Build Option 19-4, shown in Figure 12, consists of all the features of Build Option 19-2 and adds a dedicated receiving lane on southbound South Valley View Road that accommodates a free right turn movement from a channelized right turn lane on the southbound I-5 exit ramp to southbound South Valley View Road. Ideally the additional southbound lane on South Valley View road should extend all the way to OR 99. Carrying two lanes southbound from the interchange to OR 99 avoids all issues related to merging. However, in lieu of the full length, the lane should be carried as far as possible. Analysis shows that this build option results in both ramp terminals meeting HDM mobility standards for the design year. Under year 2030 traffic volume conditions the southbound ramp terminal would operate with a v/c of 0.64 and the northbound ramp terminal would operate with a v/c of 0.74. Traffic control and traffic operations at the northbound ramp terminal are identical to those from Build Option 19-2.

Figure 12. North Ashland Interchange Build Option 19-4

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.11 North Ashland Interchange Build Option 19-5

In response to Agency and OBDB requests, traffic operations were analyzed at the North Ashland Interchange for a four-lane bridge. Build Option 19-5, shown in Figure 13, consists of all the features of Build Option 19-4, but increases the width of the bridge to accommodate four lanes. The primary attribute of a four-lane bridge option is that it increases the amount of storage available for left turn movements at both ramp terminals and avoids any issues related to taper lengths that would be associated with back-to-back left turns at ramp terminals. The v/c at each ramp terminal was calculated to be identical to those of Build Option 19-4. This indicates that a fourth lane provides no traffic operations benefit at the ramp terminals over a three-lane structure. A four lane bridge avoids any issue relating to tapers associated with left turn lanes by allowing side-by-side left turn lanes. Preliminary measurements indicate that the available left turn queue storage associated with a four-lane bridge section would be approximately 700 feet at both ramp terminals, which far exceeds the calculated 95th percentile queue for the design year.

Figure 13. North Ashland Interchange Build Option 19-5

Note: Figure intended for illustration purposes only. Lane configurations are exaggerated for clarity.

11.12 Improvements to South Valley View Road

Build Options 19-4 and 19-5 assume that South Valley View Road contains two southbound through lanes for an appropriate distance south of the ramp terminal such that it provides a dedicated receiving lane for right-turning vehicles coming from the southbound I-5 exit ramp. Analysis indicates that an additional southbound lane on South Valley View Road would be required to meet HDM mobility standards at the southbound ramp terminals in year 2030. Two southbound lanes would ideally extend south all the way to OR 99. This will be necessary within the planning horizon to prevent operational problems at the intersection of South Valley View Road at OR 99 from impacting the interchange. Since the HCM does not provide a methodology for assessing a lane drop from two lanes to one lane, a definitive assessment of the minimum distance for the two lanes cannot be specified. Lacking a method to specify a minimum distance, the recommendation is that the two lanes be carried as far as possible.

Table 11 Build Options Analysis Summary

Build Option[1]	Ramp Terminal Intersection	2010 Design Year v/c	2010 Design Year LOS	2030 Design Year v/c	2030 Design Year LOS
Green Springs Interchange 14					
14-1. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of the exit ramp to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	0.59	B	**0.83**	C
	NB Ramp Terminal	0.67	C	**0.93**	D
14-2. Four-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of the exit ramp to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	0.42	A	0.59	B
	NB Ramp Terminal	0.54	B	0.75	B
14-3. Five-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of the exit ramp to provide for separate left and right turns lanes at the ramp terminals.	SB Ramp Terminal	0.42	A	0.59	B
	NB Ramp Terminal	0.54	B	0.75	B
14-4. Single-Point Urban Interchange (SPUI)	SPUI Intersection	0.52	B	0.71[2]	D
North Ashland Interchange 19					
19-1. Three-lane bridge, maintain existing intersection control, widening of a portion of the exit ramp to provide for separate left and right turns lanes at the ramp terminals.	SB Ramp Terminal	0.63	A	**>1.00**	F
	NB Ramp Terminal	0.57	A	**0.85**	F
19-2. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of the exit ramp to provide for separate left and right turns lanes at the ramp terminals.	SB Ramp Terminal	0.51	A	**0.84**	A
	NB Ramp Terminal	0.52	B	0.74	B
19-3. Loop ramp (no signal) at northbound ramp terminal, three-lane bridge, signal at SB ramp terminal, widening of a portion of the exit ramp to provide for separate left and right turns lanes at the ramp terminals.	SB Ramp Terminal	0.58	B	**0.85**	B
	NB Ramp Terminal	0.43	A	0.71[3]	B
19-4. Same as Build Option 19-2, plus dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp.	SB Ramp Terminal	0.48	A	0.64	A
	NB Ramp Terminal	0.51	B	0.74	B
19-5. Four-lane bridge, signals at SB and NB ramp terminals, dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	0.47	A	0.64	A
	NB Ramp Terminal	0.52	B	0.74	B

Notes:
1. Descriptions are summaries of major features only. See report narrative for detailed descriptions of each build option.
2. Signalized intersection only; does not include operations of right-turn lanes from ramps.
3. Shows highest v/c approach per methodology outlined in the TPAU Analysis Procedures Manual.

Table 12. Build Options Queuing Summary; Green Springs Interchange 14

Build Option	Ramp Terminal Intersection	Movement	95th Percentile Queue (ft) 2010 Design Year	95th Percentile Queue (ft) 2030 Design Year
14-1. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	WBL	50	75
		SBL	200	325
	NB Ramp Terminal	EBL	425	475
		NBR	50	75
14-2. Four-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	WBL	50	50
		SBL	150	200
	NB Ramp Terminal	EBL	150	275
		NBL/T	50	100
14-3. Five-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	WBL	25	50
		SBL	150	200
	NB Ramp Terminal	EBL	150	275
		NBL/T	75	100
14-4. Single-Point Urban Interchange (SPUI)	SPUI Intersection	SBL	125	300
		NBL	75	75
		NBR	25	50
		EBL	200	275
		WBL	50	75

Table 13. Build Options Queuing Summary; North Ashland Interchange 19

Build Option	Ramp Terminal Intersection	Movement	95th Percentile Queue (ft) 2010 Design Year	95th Percentile Queue (ft) 2030 Design Year
19-1. Three-lane bridge, maintain existing intersection control, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	EBL/T	50	**1275**
		SBL	50	75
	NB Ramp Terminal	WBL	125	**1400**
		SBT	75	100
19-2. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	EBL/T	50	50
		SBL	75	100
	NB Ramp Terminal	NBL	200	325
		WBL	150	250
19-3. Loop ramp (no signal) at northbound ramp terminal,Three-lane bridge, signal at SB ramp terminal, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	EBL/T	50	50
		SBL	75	100
	NB Ramp Terminal	WBL	75	100
		NBT	100	125
		SBT	100	125
19-4. Same as Build Option 19-2, plus dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp.	SB Ramp Terminal	EBL/T	50	50
		SBL	50	75
	NB Ramp Terminal	NBL	175	375
		WBL	150	250
19-5. Four-lane bridge, signals at SB and NB ramp terminals, dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	SB Ramp Terminal	EBL/T	50	50
		SBL	50	75
	NB Ramp Terminal	NBL	175	375
		WBL	175	275

Table 14. Summary of Build Option Adequacy, Potential Design and ROW Issues

Build Option	Meets HDM Mobility Std?	Potential Issues for Designer
Green Springs Interchange 14		
14-1. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	NO	Inadequate queue storage for EBL.
14-2. Four-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	YES	May not provide simultaneously for 1) sufficient storage for design year EBL 95th percentile queues; 2) standard 100-foot storage bay for the westbound approach at the southbound ramp terminal; 3) a standard taper associated with a 40-mph design speed; and 4) a distance of approximately 535 feet between ramp terminals.
14-3. Five-lane bridge with dual EB left turn lane, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	YES	Additional width of bridge relative to Option 14-2 may have implications for the width of Ashland Street east and west of the ramp terminals.
14-4. Single-Point Urban Interchange (SPUI)	YES	Single lane approaches on several approaches appear adequate to meet mobility standards, but may not be appropriate for other reasons.
North Ashland Interchange 19		
19-1. Three-lane bridge, maintain existing intersection control, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	NO	
19-2. Three-lane bridge, signals at SB and NB ramp terminals, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	NO[1]	
19-3. Loop ramp (no signal) at northbound ramp terminal,Three-lane bridge, signal at SB ramp terminal, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	NO[1]	Appropriate design speed must be selected for loop ramp. Existing northbound entrance ramp will likely need to be redesigned. Designer must choose whether to merge ramps prior to merging with I-5 or have two entrance ramps merge with I-5 with appropriate distances between them. Access Modification Request will be needed to address FHWA requirements.
19-4. Same as Build Option 19-2, plus dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp.	YES	Dedicated receiving lane on southbound South Valley View Road should be carried as far as possible.
19-5. Four-lane bridge, signals at SB and NB ramp terminals, dedicated receiving lane on SB Valley View Road to accommodate right turning vehicles from I-5 SB exit ramp, widening of a portion of exit ramps to provide separate left and right turn lanes at the ramp terminals.	YES	Dedicated receiving lane on southbound South Valley View Road should be carried as far as possible.

1. Southbound ramp terminal would meet mobility standard if dedicated receiving lane is provided on southbound South Valley View Road, as shown in the analysis for Build Option 19-4.

12 Access Management Standards

The current access in the vicinity of both the Green Springs Interchange 14 and North Ashland Interchange 19 does not conform to the OHP Access Management Standards.

OHP Table 16: Minimum spacing standards applicable to freeway interchanges with two-lane crossroads specifies the following minimums for rural area type:

- 1320 feet distance to the first approach on the right; right in/right out
- 1320 feet distance to the first intersection where left turns are allowed
- 1320 feet distance between the last right in/right out approach and the start of taper for the entrance ramp

The distances repeated above from Table 16 relating to rural area type are probably applicable to the current situation for North Ashland Interchange 19.

OHP Table 16: Minimum spacing standards applicable to freeway interchanges with two-lane crossroads specifies the following minimums for fully developed urban area type:

- 750 feet distance to the first approach on the right; right in/right out
- 1320 feet distance to the first intersection where left turns are allowed
- 750 feet distance between the last right in/right out approach and the start of taper for the entrance ramp

The distances repeated above from Table 16 relating to fully developed urban area type are probably applicable to the current situation at Green Springs Interchange 14. It is worth noting, however, that increasing the number of lanes on Ashland Street over I-5 may cause stricter spacing standards to apply. OHP Table 17: Minimum spacing standards applicable to freeway interchanges with multi-lane crossroads specifies a longer distance (990 feet, instead of 750 feet) as the minimum distance between the last approach road and the start of the taper for the entrance ramp.

12.1 List on Non-Conforming Accesses

The following non-conforming accesses are noted in the vicinity of North Ashland Interchange 19:

- The intersection of East and West Butler Lane is less than 1320 feet from the northbound ramp terminal. The distance is estimated to be approximately 1100 feet.
- The intersection of East Ashland Lane is less than 1320 feet from the northbound ramp terminal. The distance is estimated to be approximately 200 feet.
- The intersection of Lowe Road is less than 1320 feet from the southbound ramp terminal. The distance is estimated to be approximately 200 feet.
- The driveways serving high-volume commercial establishments on both sides of South Valley View Road are less than 1320 feet from the southbound ramp terminal. The distance is estimated to be approximately 700 feet.
- Numerous driveways serving fields and individual residences are also present both north and south of the interchange.

No determination has been made as to which public and private approaches have valid access permits.

The following non-conforming accesses are noted in the vicinity of Green Springs Interchange 14:

- The intersection of Tolman Creek Road is less than 1320 feet from the southbound ramp terminal. The distance is estimated to be approximately 1100 feet.
- The intersection of Washington Street is less than 1320 feet from the southbound ramp terminal. The distance is estimated to be approximately 350 feet.
- The intersection of Clover Lane is less than 1320 feet from the northbound ramp terminal. The distance is estimated to be approximately 250 feet.
- The intersection of Sutton Place is less than 1320 feet from the northbound ramp terminal. The distance is estimated to be approximately 750 feet.
- Driveways serving high-volume commercial establishments on both sides of Ashland Street to the east and west of I-5 are less than 1320 feet from the ramp terminals.

No determination has been made as to which public and private approaches have valid access permits.

12.2 Changes to Access Management in Connection with Bridge Bundle 314

If it is determined that construction of improvements associated with Bridge Bundle 314 is the appropriate time to bring the roadways into compliance with OHP Access Management Standards, several changes will need to be implemented at both Interchange 19 (North Ashland) and Interchange 14 (Green Springs).

At Interchange 19, the following changes to the local street network would be necessary to fully comply with the OHP Access Management Standards:

1) Relocate the intersection of South Valley View Road and West/East Butler Lane by relocating the intersection of West/East Butler Lane to the north by a distance of approximately 250 feet. (The intersection is now approximately 1100 feet from the interchange's northbound ramp terminal.)
2) Relocate East Ashland Lane to do one of the following:
   a) Relocate it to the north by a distance of approximately 600 feet and restrict it to right-in, right-out movements, or
   b) Eliminating its intersection with South Valley View Road and creating a new intersection with East Butler Lane instead.
3) Relocate Lowe Road to do one of the following:
   a) Relocate it to the south by a distance of approximately 600 feet and restrict it to right-in, right-out movements, or
   b) Relocate it to the south by a distance of approximately 1300 feet to align with the intersection of Eagle Mill Road.

At Interchange 14, the following changes to the local street network would be necessary to fully comply with the OHP Access Management Standards:

1) Relocate the intersection of Ashland Street and Tollman Creek Road to the west by a distance of approximately 250 feet. (The intersection is now approximately 1100 feet from the interchange's southbound ramp terminal.)

2) Relocate the intersection of Ashland Street and Washington Street to the west by a distance of about 700 feet (if OHP Table 17 applies) or by a distance of approximately 400 feet (if OHP Table 16 applies) and restrict the intersection of Ashland Street and Washington Street Lane to right-in, right-out movements
3) Relocate the intersection of Ashland Street and Clover Lane to the east by a distance of about 750 feet (if OHP Table 17 applies) or by a distance of approximately 450 feet (if OHP Table 16 applies) and restrict the intersection to right-in, right-out movements
4) Relocate the intersection of Ashland Street and Sutton Place to the east by a distance of about 250 feet (if OHP Table 17 applies) and restrict the intersection to right-in, right out movements. If OHP Table 16 applies, relocating the intersection does not appear to be necessary, but restrict the intersection to right-in, right-out movements.

12.3 Access Management Differences Between Build Options

The current configuration at both interchanges is such that the OHP Access Management Standards are not met. There is little reason to believe that different access management standards are applicable depending upon which build option is selected.

There is also not likely to be a significant cost difference associated with bringing the roadway system into compliance with OHP Access Management Standards depending upon which build option is selected. If an existing local access road needs to be relocated and restricted to right in, right out movement, it is unlikely to be either more difficult or more costly if the bridge at the interchange is three, four, or five lanes wide.

Factors that increase the importance of complying with the OHP Access Management Standards include traffic safety. The provision of a free-flow right-turn lane from the exit ramp at either interchange (as is assumed under some build options) probably increases the desirability of implementing the OHP Access Management Standards.

The relocation of Lowe Road, for example, and the restriction to right in, right out movements is desirable under all build alternatives simply to comply with the OHP Access Management Standards. It is deemed to be slightly more important to comply with OHP Access Management Standards under Build Option 19-4 than with 19-2. This is because with a dedicated receiving lane allowing free-flow entry to southbound South Valley View Road, southbound traffic on the exit ramp can be expected to be traveling somewhat faster on South Valley View Road than traffic would under stop- or yield-control conditions at the ramp terminal. Increasing the distance between the ramp terminal and the intersection with Lowe Road helps to assure adequate sight distance, allowing motorists to better judge the adequacy of gaps in traffic for their turning movements. The faster speeds likely with the dedicated receiving lane associated with Option 19-4 provide additional justification for changes to the access at Lowe Road, but is not significant difference between 19-2 and 19-4.

Similar arguments can be made relative to the importance of access control on the north side of the North Ashland Interchange. East Ashland Lane is much closer to the northbound ramp terminal than is permissible under OHP Access Management Standards. Relocation of East Ashland Lane is desirable under all build options. Should Option 19-3, which features a loop ramp, be selected, the roadway designer may uncover additional geometric reasons requiring relocation of East Ashland Lane.

Changes to access control at the Green Springs Interchange will also be needed to comply with OHP standards. Like the situation at the southbound ramp at North Ashland Interchange, the possibility of a dedicated receiving lane and higher speeds on eastbound Ashland Street suggest it would be desirable to implement changes to improve sight distance and safety along Ashland Street. Changes on the east side of I-5, including changes to Clover Lane are also desirable under all build options at Interchange 14.

13 Bicycle and Pedestrian Facilities

In the preparation of this Traffic Analysis Report, the traffic engineer has assumed that provisions would be made for pedestrians and bicyclists.

Oregon Revised Statues (ORS) govern the provision of bicycle and pedestrian facilities. ORS 366.514 (Use of highway fund for footpaths and bicycle trails) specifies, in part, that "…reasonable amounts shall be expended as necessary to provide footpaths and bicycle trails, including curb cuts or ramps as part of the project. Footpaths and bicycle trails, including curb cuts or ramps as part of the project, shall be provided wherever a highway, road or street is being constructed, reconstructed or relocated."

ORS 366.514 does provide for exceptions. ORS 366.514 (2) states:

> "Footpaths and trails are not required to be established under subsection (1) of this section:
>
> (a) Where the establishment of such paths and trails would be contrary to public safety;
>
> (b) If the cost of establishing such paths and trails would be excessively disproportionate to the need or probable use; or
>
> (c) Where sparsity of population, other available ways or other factors indicate an absence of any need for such paths and trails."

There are numerous examples of interchanges being designed with specific facilities for pedestrians and bicyclists. The South Medford Interchange, which is scheduled for construction during 2006 through 2009, is one example near Ashland. No evidence has been presented to suggest that improvements designed to accommodate pedestrians and bicyclists at the interchanges at Green Springs or North Ashland would be contrary to public safety.

The cost of providing bicycle and pedestrian facilities has not been estimated. The cost of building the project without bicycle and pedestrian facilities has not been estimated. Such cost estimates are beyond the scope of this Traffic Analysis Report, so no judgment can be made as to whether the cost of providing such facilities would be "excessively disproportionate."

The Ashland area is well known for outdoor activities, including bicycling and walking. Local governments in the area have invested in trails such as those in the Bear Creek Greenway. The presence of the Bear Creek Greenway enhances the opportunities for north-south travel in the valley. Though some may argue to the contrary, the Bear Creek Greenway probably helps justify providing bicycle and pedestrian facilities. We judge it unlikely that the "absence of any need" criterion could be met to justify elimination of facilities for bicyclists and pedestrians at either interchange.

ODOT's usual signal plans provide for preformed detector loops in bicycle lanes and pushbuttons for actuation by pedestrians. The design for the South Medford Interchange used these features. Low volumes of bicycle and pedestrian activity during the peak hour are unlikely to have any measurable impact on traffic signal operations when actuated signals are used. Actuation by pedestrians may cause more time to be given to a particular movement than would be required for the vehicles for that cycle. Pedestrians and bicyclists may also cause motorists to be delayed when making certain turning movements, such as right turns. Potential conflicts between motorists and pedestrians and bicyclists were judged to be so low at both intersections that no adjustments were considered when evaluating peak hour traffic operations.

The decision by the traffic engineer to evaluate peak hour traffic operations without specifically testing for the presence of pedestrians should not be used as justification by the roadway or signal designers to eliminate crosswalks, curb ramps, pedestrian signals, or detection. The roadway and signal designers should comply with appropriate provisions from ODOT's *Highway Design Manual* and the *Manual on Uniform Traffic Control Devices.*

A free-flow movement, such as the provision of a separate right-turn lane and dedicated receiving lane on westbound Ashland Street or southbound South Valley View Road, is clearly less than desirable from the perspective of pedestrians and cyclists. The general consensus of bicycle and pedestrian experts is that signalized ramp terminals is preferred to designs featuring free-flow movements.

If a free-flow movement is selected for the southbound right-turning movement at either interchange, the roadway designer may have a few different alternatives. Depending on the design speed chosen for the ramp, the designer may have an option of having the right-turn lane cross the bicycle lane or crossing the ramp at an acute angle. References that may prove useful to the roadway designer include ODOT's *1995 Oregon Bicycle and Pedestrian Plan*. The document provides illustrations showing the concepts applicable to merging and exit lanes. The designer is specifically referred to Section II.7. Intersections and to Figures 106 and 108.

ODOT's Bicycle and Pedestrian Program specialists may provide additional guidance or suggestions to meet the needs of bicyclists and pedestrians while meeting mobility standards.

14 Conclusions

14.1 Interchange 19 Overview

At the North Ashland Interchange 19 (South Valley View Road), a three-lane bridge was determined to be adequate. Signalization of the ramp terminals is not necessary for at least several years based on the application of preliminary traffic signal warrants. Based on ODOT mobility standards, however, installation of traffic signals at the ramp terminals is likely to be necessary to accommodate year 2030 traffic volumes.

As an alternative to signalization of the northbound ramp terminal, a loop ramp could be used at the northbound ramp terminal to accommodate the high volumes of traffic from northbound South Valley View Road to northbound I-5. With a loop ramp to accommodate this high-volume movement, signalization of the northbound ramp terminal is unlikely to be needed. From a traffic operations standpoint, a loop ramp is a viable option to meet traffic needs at the northbound ramp terminal.

A portion of both the northbound and southbound exit ramps will need two lanes as they approach the ramp terminals to allow separate left-turn and right-turn lanes. The length of the two-lane section of ramp needed to accommodate queues at the ramp terminal depends upon whether or not traffic signals are installed. With signalization of the ramp terminals, relatively short left-turn and right-turn lanes will accommodate future traffic. Without signalization, long queues extending down the ramp must be anticipated.

The southbound exit ramp will eventually require a dedicated, free-flow, right-turn lane onto southbound South Valley View Road to assure that the mobility standard specified in the Highway Design Manual is met. This is a key feature of Build Options 19-4 and 19-5. Widening of South Valley View Road to five lanes, a project specified in the County's Transportation System Plan, would allow this to be accommodated.

Neither Lowe Road nor East Ashland Lane meet the spacing standard specified in Table 16 of the OHP. Both streets should be relocated to locations farther from the ramp terminals.

14.2 Interchange 19 Conclusions

In connection with the OTIA III Bridge Bundle #314:

- A three-lane bridge appears to provide adequate width to accommodate appropriate left turn lanes for both northbound and southbound ramp terminals and adequate tapers for back-to-back left turn lanes.
- Providing separate left-turn and right-turn lanes on the northbound exit ramp for a distance of approximately 300 feet accommodates the calculated 95^{th} percentile queue for 2030 traffic assuming signalization of the ramp terminal.
- Providing separate left-turn and right-turn lanes on the southbound exit ramp for a distance of approximately 150 feet accommodates the 95^{th} percentile queue for 2030 traffic assuming signalization of the ramp terminal.
- Signalization of the southbound ramp terminal will likely be necessary by the end of the planning period.
- Providing a dedicated receiving lane for the southbound ramp terminal appears to be needed to meet HDM mobility standards in year 2030. Others will need to assess whether this should be provided in connection with the Bridge Bundle 314 or in connection with widening of South Valley View Road (a County project listed in the County TSP).
- Providing conduits and junction boxes at each ramp terminal in anticipation of future signalization may reduce costs of a future signalization project.
- From a traffic operations standpoint, a loop ramp is a viable option to meet traffic needs at the northbound ramp terminal.
- Retaining existing diamond configuration and intersection control for northbound ramp but designing the bridge such that it could accommodate a loop ramp may provide future flexibility. This may require the relocation of East Ashland Lane.
- Converting the northbound ramp terminal to conventional intersection control would result in worse traffic operations than retaining the existing intersection control. A conventional stop control configuration would result in failure and over-capacity conditions prior to the planning horizon year.

14.3 Interchange 14 Overview

At the Green Springs Interchange 14 (Ashland Street), at least a four-lane bridge was determined to be necessary to accommodate dual left-turn lanes at the northbound ramp terminal. Due to geometric constraints a four-lane bridge would likely require design exceptions for the westbound left-turn lane length and taper length between the westbound and eastbound left-turn lanes. A five-lane bridge appears be necessary to meet all ODOT design standards without design exceptions. High volumes of left turns from eastbound Ashland Street to northbound I-5 are sufficient to require dual left-turn lanes by 2030.

Traffic signal warrants are currently met for the southbound ramp terminal and installation of traffic signals at the northbound ramp terminal will, in connection, with the dual turn lanes avoid long queues for the eastbound-to-northbound left turn and avoid operational problems. Without dual turn lanes, insufficient storage will be available to keep queues of left-turning vehicles interfering with the intersection of the southbound ramp terminal with Ashland Street.

A single-point urban interchange, or SPUI, was found to provide adequate capacity to accommodate projected year 2030 traffic volumes and is a viable alternative from a traffic operations standpoint.

Portions of both the northbound and southbound exit ramps will need two lanes as they approach the ramp terminals to allow separate left-turn and right-turn lanes.

A free-flow right-turn lane from the southbound exit ramp to a dedicated receiving lane on westbound Ashland Street will assure that the mobility standard specified in the Highway Design Manual is met.

14.4 Interchange 14 Conclusions

In connection with the OTIA III Bridge Bundle #314:

- A five-lane bridge that provides for side-by-side left turn lanes meets HDM mobility standards while allowing all design standards for left-turn bay and taper lengths without design exceptions.
- A four-lane bridge that provides for back-to-back left turn lanes meets HDM mobility standards, but is likely to require design exceptions relating to turn lane storage or tapers associated with back-to-back left turn lanes.
- Based on traffic signal volume warrants, installation of traffic signals at both ramp terminals appears to be appropriate in connection with Bridge Bundle 314, but additional analysis and a comprehensive engineering study appears necessary to comply with OAR 734-020-0460.
- Providing separate left-turn and right-turn lanes on the northbound exit ramp for a distance of approximately 150 feet accommodates the calculated 95th percentile queue for 2030 traffic assuming signalization of the ramp terminal.
- Providing separate left-turn and right-turn lanes on the southbound exit ramp for a distance of approximately 250 feet accomodates the calculated 95th percentile queue for 2030 traffic assuming signalization of the ramp terminal.
- Providing a dedicated right-turn lane for the southbound ramp terminal and a dedicated receiving lane on westbound Ashland Street helps assure that the HDM mobility standard will be met at the intersection of Ashland Street and the I-5 southbound ramp terminal.
- Providing dual receiving lanes on the northbound entrance ramp will be necessary to accommodate the dual left turn from eastbound Ashland Street. The dual receiving lanes should be carried as far down the ramp as possible, but may merge into one lane prior to joining the I-5 mainline.
- From a traffic operations standpoint, a SPUI is a viable option to meet traffic needs at the interchange.