13 SIGNALIZED INTERSECTION ANALYSIS

13.1 <u>Purpose</u>

This chapter presents commonly used signalized intersection deterministic analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Simulation procedures are covered in APM Chapter 15. Software settings are covered in Appendix 12/13. Topics covered include:

- Turn Lanes at Signalized Intersections
- Signalized Intersection Capacity Analysis
- Signal Progression Analysis
- Estimating Queue Lengths at Signalized Intersections

The scope of this chapter is limited to auto mode analysis at signalized intersections. A complete evaluation of signalized intersections requires a broader evaluation including of non-auto modes. Refer to APM Chapter 10 for modal considerations such as for left and right turn lanes, and to Chapter 14 for multimodal analysis procedures such as MMLOS. The need for other evaluations such as per the Traffic Manual and HDM should be coordinated with Region Roadway/Traffic or Traffic-Roadway Section.

13.2 <u>Criteria for Turn Lanes at Signalized Intersections</u>

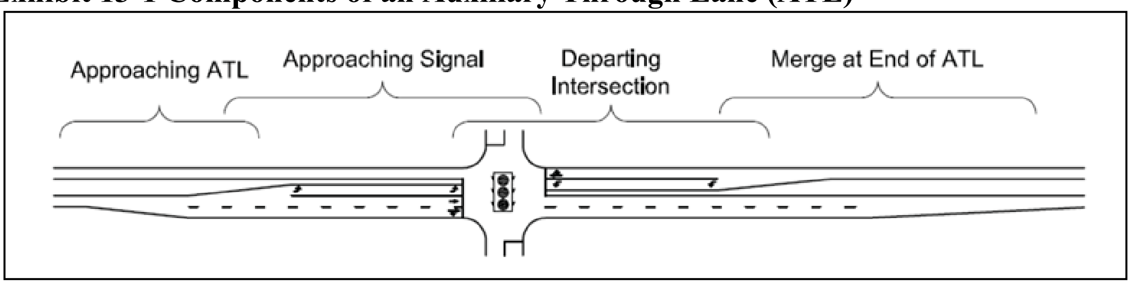
Turn lanes at signalized intersections are determined differently than at unsignalized intersections. At signalized intersections a left turn lane is always desirable, while a right turn lane is generally determined based on signal capacity needs. At signalized intersections, installation of turn lanes must be consistent with the requirements in ODOT's Traffic Signal Policy and Guidelines and the Traffic Manual and approval must be received.

13.3 Auxiliary Through Lanes at Signalized Intersections

The following procedure is intended for the analysis of existing ATLs only. Installation of an Auxiliary Through Lane (ATL) on a state highway is generally not allowed. ODOT has reviewed NCHRP Report 707 Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections. While the document provides discussion about the use of auxiliary through lanes and creates a potential process to follow when considering installing an auxiliary through lane, it was found the research and analysis was neither comprehensive nor definitive enough to fully support the recommendations. Therefore, the installation of an auxiliary through lane on the state highway system will require approval from the State Traffic-Roadway Engineer and will be considered on a case-bv-case basis.

An Auxiliary Through Lane (ATL) is a limited length through lane added midblock upstream and downstream of a signalized intersection (Exhibit 13-1). Configurations different than shown in the exhibit, including when accesses are present, are not considered ATLs and are add/drop lane areas instead, which are not covered in this section. Typically, the ATL form has been used as a way to meet an operational standard with future street widening deferred to a later date. ATLs are more commonly found on local rather than state facilities.

Exhibit 13-1 Components of an Auxiliary Through Lane (ATL)



13.3.1 ATL Issues

There are several issues regarding ATLs and transit, access points, pedestrians and bicycles and other conditions. Overall, ATLs are discouraged and in some cases should be reconfigured. A few of the issues are identified below:

- Access points within ATLs can be both safety and operational concerns.
- Pedestrian crossing distance and time is longer at an ATL, which can lead to longer exposure, cycle times and increased delay.
- Transit stop locations can be a problem within ATLs. Without a transit pull-out, the presence of a bus will reduce ATL utilization.
- Some ATLs may be used as a passing lane, causing a safety concern in the speed differential between the two lanes during congested hours.

13.3.2 TL Analysis

As noted above, in the review of NCHRP Report 707, ODOT does not fully support recommendations regarding the NCHRP 707 (1) procedure for estimating the lane utilization, taper length, or prescribed length of an ATL.

For analysis of an existing ATL, the lane utilization should be measured in the field if possible. If the lane utilization cannot be measured, assume a lane utilization of 15% for a shared ATL with one continuous through lane, 12% if two continuous through lanes exist. Add 3% for an exclusive right turn lane.

To analyze the adequacy of the taper or ATL length, such as for performing microsimulation, the analyst needs to work with the designer to determine what lengths should be used. This may be an iterative process where the analyst runs the analysis with several different lengths to determine the impact of length on the analysis results.

13.4 Signalized Intersection Analysis

Signalized intersection control can generally be classified into three categories; pre-timed, semi-actuated and fully actuated operations. A pre-timed signal has the cycle length, phases, green times and change phases all preset to be constant for every cycle. A semi-actuated signal operates by designating a “main street” that is served until actuation from the “side street” occurs. Under this type of operation, the cycle length and green times may vary based on vehicle demand. ODOT has effectively upgraded all formerly semi-actuated intersections to fully actuate. A fully actuated signal allows detection on all legs and phases of the intersection and cycle lengths and green times are determined based on the demand for each movement.

In addition to the type of signal operating, each signalized intersection has characteristics associated with it related to how the timing of a signal is allocated over a cycle. These characteristics relate to phases, intervals, change intervals, green time, lost time, yellow and all-red clearance times and effective green time. All of these characteristics can be part of signalized operations and can affect the overall intersection operations. For more information on characteristics of signals and signal operations analysis, refer to Chapter 16 of the HCM.

13.4.1 Saturation Flow Rates

As previously discussed in Chapter 3, saturation flow rates are critical components in the analysis of signalized intersection capacity and can be defined as the flow in vehicles per hour that can be accommodated by a lane group assuming that the green phase is displayed 100 percent of the time. Saturation flow rates can be measured in the field or calculated by applying adjustment factors to a default “ideal” saturation flow rate. For more information regarding the calculation and application of saturation flow rates, refer to Chapter 3.

Chapter 31 of the HCM 7th Edition provides adjusted saturation flow rates for through movements, along with saturation flow adjustment factors for protected and permitted left turns, that reflect the presence of connected and automated vehicles (CAVs) in the traffic stream. CAVs offer the potential to increase the saturation flow rate by being able to cooperatively form platoons that have shorter headways between platooned vehicles than human-driven vehicles can achieve safely. These shorter headways allow more vehicles to enter an intersection per hour of green time, increasing the capacity of through and protected left movements. In addition, they can result in longer gaps in opposing traffic that can be used by permitted left-turn movements. Both effects can result in higher movement capacities, particularly at higher percentages (>60–80%) of CAVs in the traffic stream. Appendix 6B provides guidance on estimating saturation flow rates for use in longer-range planning analyses testing the potential effects of CAVs on signalized intersection and arterial capacity.

As of 2022, no vehicles were available commercially that met the definition of a CAV for the purposes of the capacity adjustments provided for signalized intersection analyses in the HCM (i.e., a vehicle with an operating cooperative adaptive cruise control system that is capable of communicating with other vehicles and driving without human intervention in any situation). The saturation flow rate adjustments presented in Appendix 6B are intended for use only in longer-range planning analyses. That appendix also provides guidance on estimating the percentage of CAVs in the traffic stream in a future year and example problems.

Because CAVs are not yet commercially available, saturation flow rate adjustments for CAVs should not be made in near-term analyses such as traffic impact studies.

13.4.2 Right Turn on Red

Oregon law permits a right-turn movement by a vehicle facing a circular red or a red arrow indication after stopping and yielding to pedestrians and any conflicting vehicles, unless posted otherwise. For future conditions, an engineering study should be performed to evaluate appropriate traffic control options such as RTOR prohibition for safety reasons – contact Region Traffic for guidance. Warrants for turn prohibitions are found in OAR 734-020-0020. Additional guidance is found in MUTCD Section 2B.54. Region Traffic Engineer/Manager approval is required for No Turn On Red signs. The remainder of this section assumes that it has been determined that RTOR will not be prohibited.

For existing conditions, the HCM advises that counts may be used to obtain the RTOR volume, which is then subtracted from the total right turn volume in the analysis. However, it is often not practical to obtain RTOR counts.

For future conditions or where RTOR counts are not obtained, the HCM does not provide a methodology to estimate right turn on red (RTOR) volume. The following options for analysis can be considered.

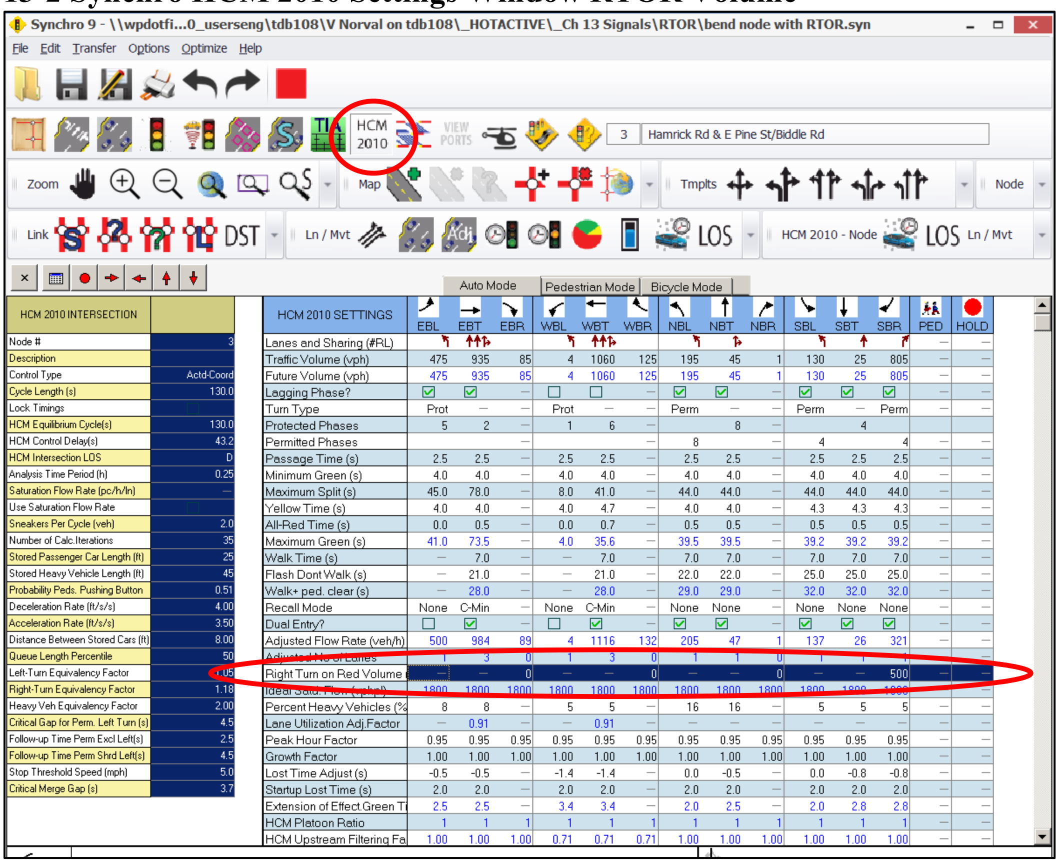
1. No reduction for RTOR – The HCM recommends not applying a reduction for RTOR for future conditions. This provides a conservative result. If v/c ratio and queuing are not an issue, no further RTOR analysis may be deemed necessary; however for simulation

RTOR should be accounted for. The operational benefit of RTOR is a function of the volume of right turns, volume of conflicting traffic and signal timing/phasing.

2. Synchro –RTOR can be enabled by checking the RTOR box in the Lane settings window. In this method, a saturated flow rate for RTOR is calculated. The right turn on red saturated flow rate (sRTOR) is the potential volume if the signal was red 100% of the time. In order to reflect RTOR in the HCM 2010 report, a RTOR volume must be entered in the HCM 2010 settings window as shown in Exhibit 13-2 below. The Synchro estimated RTOR volume (vRTOR) can be obtained from this equation:

   vRTOR = sRTOR * r/C, where r/C is the red to cycle ratio

Exhibit 13-2 Synchro HCM 2010 Settings Window RTOR Volume



3. SIDRA – Check the Turn on Red Checkbox to identify approaches where RTOR is allowed. SIDRA will then internally calculate and apply the RTOR volume reduction similar to Synchro.

4. Vistro – In Vistro the analyst can select the approaches where RTOR is allowed either using a global setting or by approach. However, Vistro does not calculate the RTOR volume so it must be input manually. First, assume no RTOR and determine if there is a v/c ratio or queuing problem. If no v/c ratio or queuing problem is found, no further analysis is necessary. If a v/c ratio or queuing problem is found assuming no RTOR, options for estimating RTOR in Vistro include using Synchro to obtain the RTOR volume, or one of the following steps.

5. Planning level method for shared through/right lanes - can be used for estimating RTOR volume in Vistro (2). This method addresses only vehicle conflicts with the RTOR movement. It does not address pedestrian conflicts or bicycles in the traffic stream. A significant volume of pedestrians may warrant posting of no right turn on red signage.

   The method estimates RTOR volume using the following model (3).

$$N_{RTOR} = \min(X_r, 1.0) \times \left(\frac{1-p}{p}\right) \times \frac{3600}{C}$$

   where

   N_{RTOR} = expected number of RTORs expressed as an hourly flow rate for the analysis period
   Xr = demand volume-to-capacity ratio for the shared lane subject approach
   p = proportion of through vehicles to the total approach volume in the shared lane (veh/h)
   C = average cycle length (s) during the analysis period.

   The RTOR volume is deducted from the total right turn volume.

6. Planning method for exclusive right turn lanes – Assumes 50% of right turn volumes turn right on red, unless high pedestrian traffic or sight distance constraints are present, in which case assume 30% RTOR (4).

7. Wisconsin method for exclusive right turn lanes – Applies a reduction factor to the total right turn volume as follows[1]:
   - Single Right-Turn Lanes at Intersections: 0.62
   - Single Right-Turn Lanes at Interchanges: 0.34
   - Dual Right-Turn Lanes (Intersections and Interchanges): 0.70

Example 13-1 Planning Level RTOR Method for Shared Through/Right Lane

A two-lane approach has one through lane and one shared through/right lane. The through volume is 760 vph and the right turn volume is 250 vph. The v/c ratio for the shared lane group is 0.97. The cycle length is 100 sec.

Xr = 0.97

Total lane group volume = 760 + 250 = 1010 vph

Assuming balanced lane volumes, the volume in each lane is 1010/2 = 505 vph

Shared lane through volume = 505 – 250 = 255 vph

[1] https://wisconsindot.gov/dtsdManuals/traffic-ops/manuals-and-standards/teops/16-15.pdf

p = proportion of through vehicles in shared lane = 255/505 = 0.50

$$N_{RTOR} = \min(0.97, 1.0) \times \left(\frac{1-0.50}{0.50}\right) \times \frac{3600}{100} = 35 \text{ vph}$$

Therefore, for this example the RTOR volume can be estimated as 35 vph.

13.4.3 Critical Movement Analysis

The critical movement analysis method is a planning-level tool to estimate capacity of a signalized intersection with existing or forecasted volumes. It is for estimation only; not to report final v/c ratios or compare to mobility targets. The analysis requires intersection approach volumes, number of lanes, and lane assignments per approach.

Each movement pair in conflict (e.g. westbound left and eastbound through) are added for a total volume. Identify the highest total (or critical movement pair) for each roadway. If available, use lane utilization for duplicate lane assignments on an approach. If lane utilization data does not exist, then use an even distribution. The critical movement pairs for each roadway are summed and compared with the thresholds shown in Exhibit 13-3.

Exhibit 13-3 Intersection Performance Assessment by Critical Volume

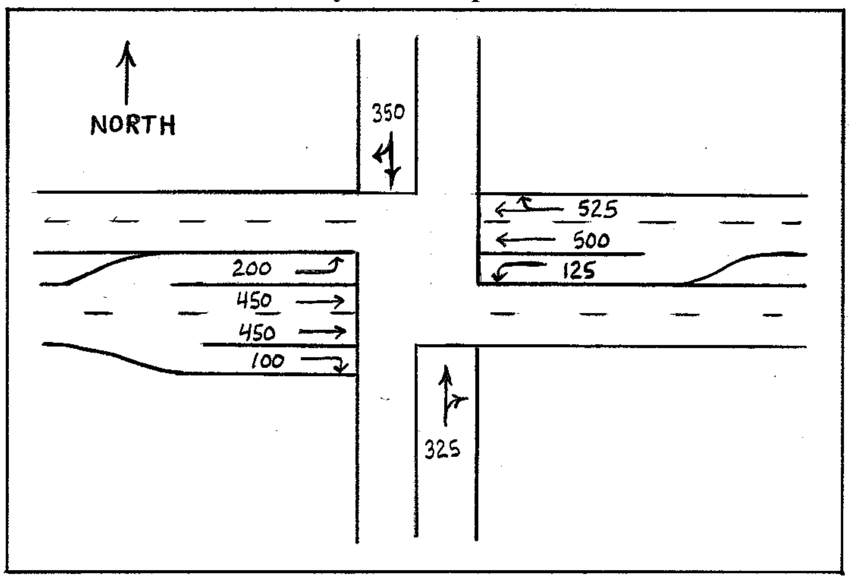
Sum of Critical Volumes (Vehicles/Hour/Lane)	Performance
0 to 1,200	Under Capacity
1,201 to 1,400	Near Capacity
1,401 and Above	Over Capacity

Critical movement analysis only estimates an intersection's capacity. It does not estimate vehicle delay, level of service or vehicle queue lengths.

Example 13-2 Critical Movement Analysis

The Critical Movement figure shows the signalized intersection of a five-lane highway with a two-lane cross street. For this intersection, conduct critical movement analysis.

Critical Movement Analysis Example



Solution:

For the east-west roadway, the conflict pairs include:

- 200 (EB LT) + 525 (WB TH/RT) = 725
- 200 (EB LT) + 500 (WB TH) = 700
- 125 (WB LT) + 450 (highest EB TH) = 575
- 125 (WB LT) + 100 (EB RT) = 225

The highest conflict pair is EB LT and WB TH/RT. Therefore, the critical movement volume for the east-west roadway is 725 vehicles.

For the north-south roadway, the conflict pairs include:

- 350 (SB TH/RT) = 350
- 325 (NB TH/RT) = 325

For these approaches there are no conflicting movements, thus the highest total approach volume is the north-south critical movement, 350 vehicles. The sum of the critical movement volumes for the intersection:

725 (east-west) + 350 (north-south) = 1,075

Compared to the thresholds shown in Exhibit 13-3, this intersection is estimated to operate under capacity.

13.4.4 Critical Intersection v/c Ratio

For signalized intersections, the OHP v/c ratio is based on the critical intersection v/c ratio, not the movement v/c ratio as explained in Action 1F of the OHP. The critical intersection v/c ratio is also known as Xc in the HCM. It involves summing the flow ratios of the critical movements. This value is not generally affected by the approach green times (except in cases with shared left turns). See HCM equation below.

Critical Intersection Volume to Capacity Ratio (for signalized intersections)

$$X_C = \left(\frac{C}{C - L}\right) \sum_{i \in ci} y_{c,i}$$

With

$$L = \sum_{i \in ci} l_{t,i}$$

Where

X_C = critical intersection volume to capacity ratio

C = cycle length (sec)

$y_{c,i}$ = critical flow ratio for phase $i = \frac{v_i}{(Ns_i)}$

$L_{t,i}$ = phase i lost time $= l_{1,i} + l_{2,i}$ (sec)

ci = set of critical phases on the critical path

L = cycle lost time (sec)

v_i = lane group flow rate for phase i

N = number of lanes for lane group i

s_i = lane group saturation flow rate for phase i

Options for calculating the critical intersection v/c ratio:

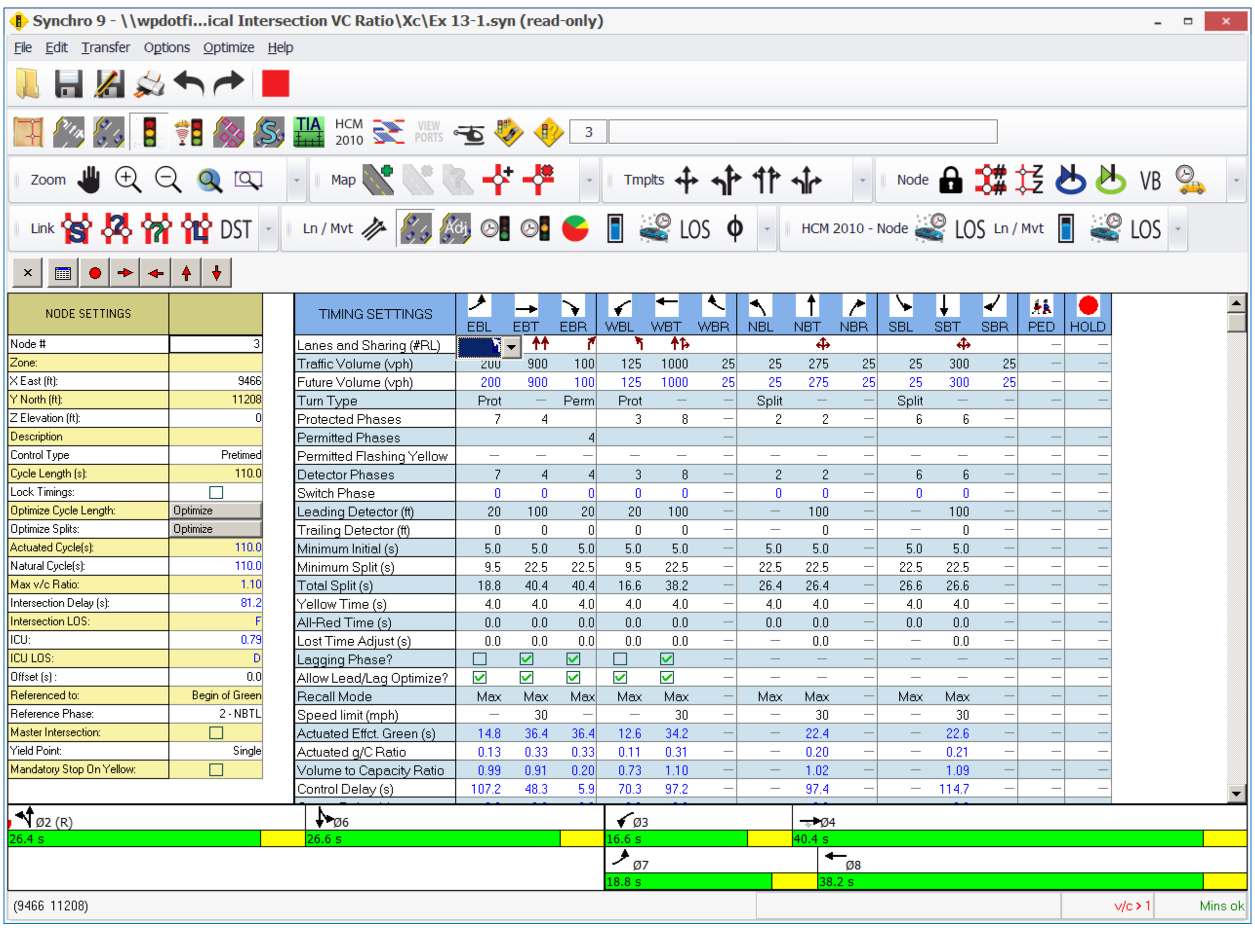
1. Vistro – In Vistro, the critical intersection v/c ratio is calculated automatically and reported out. Not requiring separate calculation of this value is one of the advantages of using Vistro, particularly where several intersections and/or alternatives are involved.
2. Synchro – In Synchro, the critical intersection v/c ratio is provided as part of the HCM 2000 report but is not provided in the HCM 2010/6th Edition report. For an HCM Synchro analysis it must be post-processed. The critical movements may be identified from the Synchro HCM 2000 report. For those movements, the critical flow ratios can be calculated manually using flow rates pulled from the Synchro HCM 2010/6th Edition report. The critical intersection v/c ratio can then be calculated. The procedure to post-process Synchro HCM output is illustrated in Example 13-3.

3. SIDRA – In SIDRA, the critical intersection v/c ratio is not reported out, but critical movements are identified from which it can be calculated using the HCM equation. The procedure to post-process critical intersection v/c ratio from SIDRA output is illustrated in Example 13-4.

Once the critical movements have been identified, the critical intersection v/c ratio can be calculated using the HCM equation. Critical movements may be identified using either CMA analysis (for protected phasing only) or, if using Synchro, from the Synchro HCM 2000 report.

Example 13-3 Calculating Critical Intersection v/c Ratio in Synchro

Example 13-1 was coded and signal timing optimized in Synchro. It was assumed that the signalized intersection had protected left turn signal phasing on the east and west approaches and split phasing on the north and south approaches. See Synchro signal timing settings window below.



NODE SETTINGS	
Node #	3
Zone:	
X East (ft):	9466
Y North (ft):	11208
Z Elevation (ft):	0
Description	
Control Type	Pretimed
Cycle Length (s):	110.0
Lock Timings:	☐
Optimize Cycle Length:	Optimize
Optimize Splits:	Optimize
Actuated Cycle(s):	110.0
Natural Cycle(s):	110.0
Max v/c Ratio:	1.10
Intersection Delay (s):	81.2
Intersection LOS:	F
ICU:	0.79
ICU LOS:	D
Offset (s):	0.0
Referenced to:	Begin of Green
Reference Phase:	2 - NBTL
Master Intersection:	☐
Yield Point:	Single
Mandatory Stop On Yellow:	☐

TIMING SETTINGS	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR	PED	HOLD
Lanes and Sharing (#RL)													—	—
Traffic Volume (vph)	200	900	100	125	1000	25	25	275	25	25	300	25	—	—
Future Volume (vph)	200	900	100	125	1000	25	25	275	25	25	300	25	—	—
Turn Type	Prot	—	Perm	Prot	—	—	Split	—	—	Split	—	—	—	—
Protected Phases	7	4		3	8	—	2	2	—	6	6	—		
Permitted Phases			4			—			—			—	—	—
Permitted Flashing Yellow	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Detector Phases	7	4	4	3	8	—	2	2	—	6	6	—	—	—
Switch Phase	0	0	0	0	0	—	0	0	—	0	0	—	—	—
Leading Detector (ft)	20	100	20	20	100	—	—	100	—	—	100	—	—	—
Trailing Detector (ft)	0	0	0	0	0	—	—	0	—	—	0	—	—	—
Minimum Initial (s)	5.0	5.0	5.0	5.0	5.0	—	5.0	5.0	—	5.0	5.0	—	—	—
Minimum Split (s)	9.5	22.5	22.5	9.5	22.5	—	22.5	22.5	—	22.5	22.5	—	—	—
Total Split (s)	18.8	40.4	40.4	16.6	38.2	—	26.4	26.4	—	26.6	26.6	—	—	—
Yellow Time (s)	4.0	4.0	4.0	4.0	4.0	—	4.0	4.0	—	4.0	4.0	—	—	—
All-Red Time (s)	0.0	0.0	0.0	0.0	0.0	—	0.0	0.0	—	0.0	0.0	—	—	—
Lost Time Adjust (s)	0.0	0.0	0.0	0.0	0.0	—	—	0.0	—	—	0.0	—	—	—
Lagging Phase?	☐	☑	☑	☐	☑	—	—	—	—	—	—	—	—	—
Allow Lead/Lag Optimize?	☑	☑	☑	☑	☑	—	—	—	—	—	—	—	—	—
Recall Mode	Max	Max	Max	Max	Max	—	Max	Max	—	Max	Max	—	—	—
Speed limit (mph)	—	30	—	—	30	—	—	30	—	—	30	—	—	—
Actuated Effct. Green (s)	14.8	36.4	36.4	12.6	34.2	—	—	22.4	—	—	22.6	—	—	—
Actuated g/C Ratio	0.13	0.33	0.33	0.11	0.31	—	—	0.20	—	—	0.21	—	—	—
Volume to Capacity Ratio	0.99	0.91	0.20	0.73	1.10	—	—	1.02	—	—	1.09	—	—	—
Control Delay (s)	107.2	48.3	5.9	70.3	97.2	—	—	97.4	—	—	114.7	—	—	—

In the Synchro HCM 2000 report, the critical movements are those identified with a ‘c’ as shown below:

HCM Signalized Intersection Capacity Analysis

3: 01/08/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	200	900	100	125	1000	25	25	275	25	25	300	25
Future Volume (vph)	200	900	100	125	1000	25	25	275	25	25	300	25
Ideal Flow (vphpl)	1750	1750	1750	1750	1750	1750	1750	1750	1750	1750	1750	1750
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0			4.0			4.0	
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95			1.00			1.00	
Frt	1.00	1.00	0.85	1.00	1.00			0.99			0.99	
Flt Protected	0.95	1.00	1.00	0.95	1.00			1.00			1.00	
Satd. Flow (prot)	1630	3260	1458	1630	3248			1692			1693	
Flt Permitted	0.95	1.00	1.00	0.95	1.00			1.00			1.00	
Satd. Flow (perm)	1630	3260	1458	1630	3248			1692			1693	
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	217	978	109	136	1087	27	27	299	27	27	326	27
RTOR Reduction (vph)	0	0	73	0	1	0	0	2	0	0	2	0
Lane Group Flow (vph)	217	978	36	136	1113	0	0	351	0	0	378	0
Turn Type	Prot	NA	Perm	Prot	NA		Split	NA		Split	NA	
Protected Phases	7	4		3	8		2	2		6	6	
Permitted Phases			4									
Actuated Green, G (s)	14.8	36.4	36.4	12.6	34.2			22.4			22.6	
Effective Green, g (s)	14.8	36.4	36.4	12.6	34.2			22.4			22.6	
Actuated g/C Ratio	0.13	0.33	0.33	0.11	0.31			0.20			0.21	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0			4.0			4.0	
Lane Grp Cap (vph)	[illegible]	1078	482	186	[illegible]			344			347	
v/s Ratio Prot	c0.13	0.30		0.08	c0.34			c0.21			c0.22	
v/s Ratio Perm			0.02									
v/c Ratio	0.99	0.91	0.07	0.73	1.10			1.02			1.09	
Uniform Delay, d1	47.5	35.2	25.2	47.1	37.9			43.8			43.7	
Progression Factor	1.00	1.00	1.00	1.00	1.00			1.00			1.00	
Incremental Delay, d2	58.5	12.6	0.3	22.2	60.8			53.5			74.0	
Delay (s)	106.1	47.7	25.5	69.3	98.7			97.3			117.7	
Level of Service	F	D	C	E	F			F			F	
Approach Delay (s)		55.6			95.5			97.3			117.7	
Approach LOS		E			F			F			F	

Intersection Summary			
HCM 2000 Control Delay	82.4	HCM 2000 Level of Service	F
HCM 2000 Volume to Capacity ratio	1.05		
Actuated Cycle Length (s)	110.0	Sum of lost time (s)	16.0
Intersection Capacity Utilization	78.9%	ICU Level of Service	D
Analysis Period (min)	15		
c Critical Lane Group			

01/08/2018 Baseline

Synchro 9 Report
Page 1

Critical movements

After identifying the critical movements, adjusted flow rates and saturated flow rate values for each can be pulled from the Synchro HCM 2010 report as shown below.

HCM 2010 Signalized Intersection Summary
3: 01/08/2018

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	200	900	100	125	1000	25	25	275	25	25	300	25
Future Volume (veh/h)	200	900	100	125	1000	25	25	275	25	25	300	25
Number	7	4	14	3	8	18	5	2	12	1	6	16
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Adj Sat Flow, veh/h/ln	1716	1716	1716	1716	1716	1750	1750	1716	1750	1750	1716	1750
Adj Flow Rate, veh/h	217	978	109	136	1087	27	27	299	27	27	326	27
Adj No. of Lanes	1	2	1	1	2	0	0	1	0	0	1	0
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	220	1079	483	187	1011	25	26	291	26	25	298	25
Arrive On Green	0.13	0.33	0.33	0.11	0.31	0.31	0.20	0.20	0.20	0.21	0.21	0.21
Sat Flow, veh/h	1634	3260	1458	1634	3251	81	[illegible]	1428	[illegible]	[illegible]	1449	120
Grp Volume(v), veh/h	217	978	109	136	543	569	353	0	0	380	0	0
Grp Sat Flow(s),veh/h/ln	1634	1630	1458	1634	1630	1701	1686	0	0	1689	0	0
Q Serve(g_s), s	14.6	31.5	5.9	8.8	34.2	34.2	22.4	0.0	0.0	22.6	0.0	0.0
Cycle Q Clear(g_c), s	14.6	31.5	5.9	8.8	34.2	34.2	22.4	0.0	0.0	22.6	0.0	0.0
Prop In Lane	1.00		1.00	1.00		0.05	0.08		0.08	0.07		0.07
Lane Grp Cap(c), veh/h	220	1079	483	187	507	529	343	0	0	347	0	0
V/C Ratio(X)	0.99	0.91	0.23	0.73	1.08	1.08	1.03	0.00	0.00	1.10	0.00	0.00
Avail Cap(c_a), veh/h	220	1079	483	187	507	529	343	0	0	347	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	47.5	35.2	26.6	47.0	37.9	37.9	43.8	0.0	0.0	43.7	0.0	0.0
Incr Delay (d2), s/veh	57.5	12.5	1.1	21.7	61.8	61.0	55.9	0.0	0.0	76.4	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	10.1	16.0	2.5	5.1	24.0	25.0	15.7	0.0	0.0	17.9	0.0	0.0
LnGrp Delay(d),s/veh	105.0	47.7	27.7	68.8	99.7	98.9	99.7	0.0	0.0	120.1	0.0	0.0
LnGrp LOS	F	D	C	E	F	F	F			F		
Approach Vol, veh/h		1304			1250			353			380	
Approach Delay, s/veh		55.5			96.0			99.7			120.1	
Approach LOS		E			F			F			F	

Timer	1	2	3	4	5	6	7	8
Assigned Phs		2	3	4		6	7	8
Phs Duration (G+Y+Rc), s		26.4	16.6	40.4		26.6	18.8	38.2
Change Period (Y+Rc), s		4.0	4.0	4.0		4.0	4.0	4.0
Max Green Setting (Gmax), s		22.4	12.6	36.4		22.6	14.8	34.2
Max Q Clear Time (g_c+I1), s		24.4	10.8	33.5		24.6	16.6	36.2
Green Ext Time (p_c), s		0.0	0.1	2.6		0.0	0.0	0.0

Intersection Summary	
HCM 2010 Ctrl Delay	83.1
HCM 2010 LOS	F

01/08/2018 Baseline

Synchro 9 Report
Page 1

Critical movements

Adjusted flow rate

Saturated flow rate

Flow ratios are calculated for each critical movement lane group by dividing the adjusted flow rate by the saturated flow rate:

EB Left = 217/1634 = 0.13

WB Through/right = 0.32 = 1087/3251 = 0.33

NB All = 299/1428 = 0.19

SB All = 326/1449 = 0.22

The sum of all critical movement flow ratios is then calculated: .13 + .33 + .19 + .22 = 0.87

Cycle length = 110 sec

Lost time per phase = 4 sec

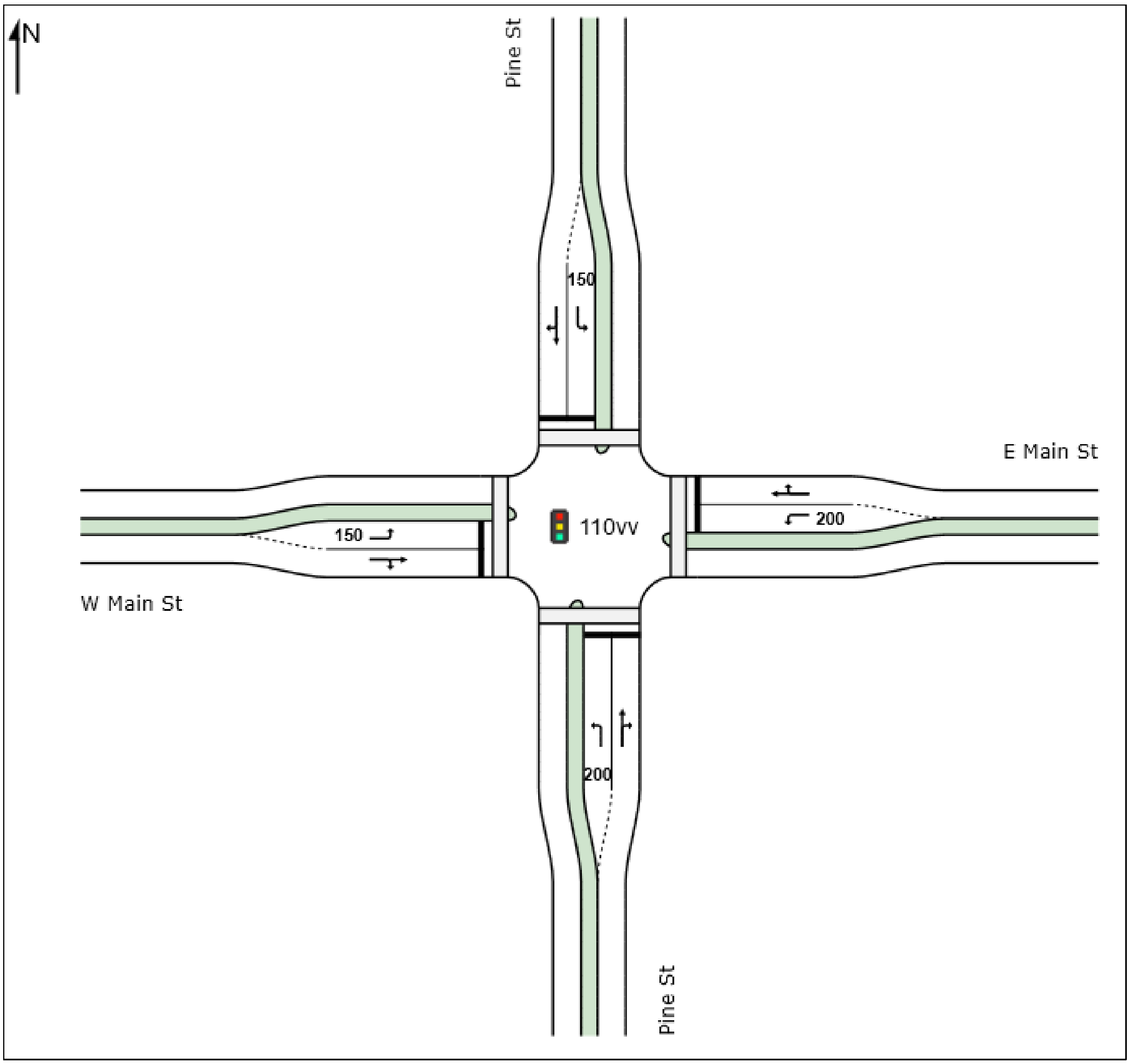
Total Lost time = 16 sec

The critical intersection v/c ratio is then calculated using the HCM 6 equation:

$$Xc = \text{Sum of critical flow ratios} * C/(C-L) = 0.87 * 110/(110-16) = 1.02$$

Example 13-4 Calculating Critical Intersection v/c Ratio in SIDRA Intersection

This example shows the calculations for the intersection v/c ratio for a four-leg three-phase intersection in SIDRA (Version 8). The phase rotation is given as east-west protected lefts, east-west through-rights, and then north-south permitted movements. The cycle time is 85 seconds with 4 seconds of lost time assumed for each phase. The signalized intersection layout is shown below.



The critical movements are shown under the Signal Timing (Movement Timing Information) in the "Critical Movements and Cycle Time" report section of the Detailed Output. The movements can be identified by noting the approach and destination of each movement. For example, the

E-S movement (Movement #1HV), goes from the east leg to the south leg and is the westbound left turn movement. Other critical movements identified are the westbound through-right movement (Movement #6HV) and the northbound left turn (Movement #3HV). The report will also tell if the critical movement occurs during the second green period, which is not the case in in this example, as the column is blank. The overall cycle length used by SIDRA or specified by the analyst is shown as 85 seconds.

```
CRITICAL MOVEMENTS AND CYCLE TIME

----------------------------------------------------------------------------------
  Crit          App    Green   Phases    Adjusted    Adjusted   Required   Required
  Mov           and    Period  ------      Lost        Flow     Grn Time   Movement
  ID     Turn   Dest           Fr  To      Time        Ratio     Ratio       Time
----------------------------------------------------------------------------------
   1HV   L2     E_S            A    B        4         0.237     0.340       32.9
   6HV   T1     E_W            B    D       29           -         -         29.0Max
   3HV   L2     S_W            D    A       29           -         -         29.0Max
                                           ----       ------    ------     ------
                                   Total:   62         0.237     0.340       90.9
----------------------------------------------------------------------------------
   -  Flow ratio not used for cycle time calculations and
      the adjusted lost time equals the required movement time
      (=Min or Max as shown in Movement Timing Information)

  Cycle Time:
    Minimum   Maximum   Practical   Chosen
       52        NA         94         85
    (Network Cycle Time - User-Given)
```

The next step is to identify the critical flow ratios for each phase from the "Lane Flow and Capacity Information" report on the next page. Note that Lane 1 is the left turn lane and Lane 2 is the through-right lane. Earlier versions of SIDRA gave the flow ratios directly, but Version 8 will requre that they be calculated separately. Flow ratios are calculated by dividing the total arriving flow by the saturation flow accounting for lane blockage. Note that while this example does not have any lane blockage effects, the values in the two saturation flow columns can be substantially different when there are lane blockages.

East-west protected left turn phase

The critical flow ratio = WBL Volume / WBL saturation flow = 371 vph / 1568 vph = **0.237**

East-west through phase

For this phase, there are two saturation flow rates given for the different green periods (this is a reflection of a varying saturation flow rate caused by turning vehicles or overflow effects from the adjacent turn lane. Review the Lane Saturation Flow Rates report (available in the report section of the left viewing pane in SIDRA) and note the saturation flow rate for the subject lane under the Flow Factors section of the report. In this case, this is 1563 vph. Trace the saturation flow across the reduction sections to determine which green period applies, which is the first green period. The critical flow ratio can then be calculated as:

WBT flow ratio = WBT Volume / WBT saturation flow = 332 vph / 1563 vph = **<u>0.212</u>**

```
LANE FLOW AND CAPACITY INFORMATION

---------------------------------------------------------------------------------
                              Saturation Flow Rate
                         -------------------------
                               W/O Lane      With Lane
 Lane     Total      Lane   Adj.  Blockage      Blockage      2nd   Tot   Deg.  Lane
 No.    Arv Flow   Width  Basic  1st   2nd      1st   2nd     Cap   Cap   Satn  Util
          veh/h      ft   tcu/h  veh/h veh/h    veh/h veh/h   veh/h veh/h   x     %
---------------------------------------------------------------------------------
 South: Pine St
 1         146      12.0  1750   1439           1439           83   254  0.574  100
 2         182      12.0  1750    257  1695      257   1695    46   502  0.363  100
---------------------------------------------------------------------------------
 East: E Main St
 1         371      12.0  1750   1568<          1568<           0   443  0.838  100
 2         332      12.0  1750   1563  1074     1563   1074    75   492  0.676  100
---------------------------------------------------------------------------------
 North: Pine St
 1         120      12.0  1750   1425           1425           83   319  0.377  100
 2         250      12.0  1750    686  1668      686   1668    51   499  0.501  100
---------------------------------------------------------------------------------
 West: W Main St
 1          40      12.0  1750   1634           1634            0   461  0.087  100
 2         245      12.0  1750   1551   854     1551    854    78   498  0.492  100
---------------------------------------------------------------------------------
  <  Reduced saturation flow due to a short lane effect
  Some upstream delays at entry to short lanes are not included.

  Basic Saturation Flow in this table is adjusted for area type factor, lane width,
  approach grade, parking manoeuvres and number of buses stopping.
  Saturation flow scale (Demand & Sensitivity dialog) applies if specified.

  Saturation Flow rates Without (W/O) Lane Blockage are used for
  signal timing purposes when the signal timing option to exclude
  downstream lane blockage effects is selected in Network analysis.
```

Lane Saturation Flow Rates	Basic Satn Flow [1]	Flow Factors [MCs & Turns	Satn Flow [3]]	Other Model Elements [4] [1st Grn	2nd Grn]	Lane Block. [5] [1st Grn	2nd Grn]	Short Lane [6] [1st Grn	2nd Grn]
	tcu/h		veh/h	veh/h	veh/h	veh/h	veh/h	veh/h	veh/h
South: Pine St									
Lane 1	1750	0.934	1634	1439	-	1439	-	1439	-
Lane 2	1750	0.969	1695	257	1695	257	1695	257	1695
East: E Main St									
Lane 1	1750	0.934	1634	1634	-	1634	-	1568[8]	-
Lane 2	1750	0.893	1563	1563	1074	1563	1074	1563	1074
North: Pine St									
Lane 1	1750	0.934	1634	1425	-	1425	-	1425	-
Lane 2	1750	0.953	1668	686	1668	686	1668	686	1668
West: W Main St									
Lane 1	1750	0.934	1634	1634	-	1634	-	1634	-
Lane 2	1750	0.886	1551	1551	854	1551	854	1551	854

North-south permitted phase

The northbound left was identified as a critical movement in this phase. This movement is part of a critical pair with the southbound through-right. Like what was done with the east-west through phase, the Lane Saturation Flow Rates report needs to be checked for this movement for the appropiate green period. In this case, the second period was identified. The critical flow ratios can then be calculated as :

NBL flow ratio = NBL Volume / NBL saturation flow = 146 vph / 1439 vph = 0.102

SBT flow ratio = SBT Volume / SBT saturation flow = 250 vph / 1668 vph = 0.150

These are then summed to obtain the critical pair flow ratio = 0.102 + 0.150 = **0.252**

The critical v/c ratio (X_c) is then calculated by dividing the cycle length by the cycle length minus the lost time per cycle and then multiplying this times the sum of the criticla flow ratios for each phase. As stated prevously, there are 4 seconds of lost time per phase and a 85 second cycle time was used. This is a three-phase intersection, so 12 seconds of total lost time is used.

X_c = [C / (C-L)] * (E-W L + E-W T + N-S) = [85s / (85s-12s)] * (0.237 + 0.212 + 0.252) = **0.82**

13.4.5 Analysis Procedures Regarding Signal Timing

Capacity analysis of signalized intersections should be performed in accordance with the methods and default parameters contained in this manual. ODOT has established the following criteria for traffic impact studies with regard to the timing chosen for the capacity analysis of signalized intersections. ODOT reserves the right to reject any operational improvements that in its judgment would compromise the safety and efficiency of the facility.

Phase Splits

Thirteen seconds is the lowest total split that should be used including yellow and all-red time. Clear documentation of the selected maximum splits for each phase must be provided in the analysis. The total side street splits should not be greater than the highway splits. Except in cases where the analyst is directed otherwise by ODOT staff, the splits are considered optimized when they yield the lowest overall intersection v/c ratio. This optimization should be done for each capacity analysis.

Non-Coordinated Signals

Cycle lengths and phase splits should be optimized to meet an ideal level of service, queuing and/or volume to capacity ratio for a non-coordinated traffic signal intersection. If simulation is going to be needed, existing signal timing will be necessary for the calibration process. For a new signal, the cycle length for the analysis should not exceed 60 seconds for a two-phased traffic signal, 90 seconds for a three-phased traffic signal (e.g., protected highway left turns and permissive side streets left turns) or 120 seconds for a four or more phased traffic signal. The signal cycle length should cover the pedestrian clearance time for all crosswalks. For information on pedestrian crossings, see ODOT Traffic Signal Policy and Guidelines.

Signals in Coordinated Signal System

At the start of a project, ODOT staff will determine whether the analysts should use the existing signal timings for all analysis scenarios or develop optimized timings for the coordinated system. The existing timings may need to be used to calibrate a simulation model. If the existing timings are to be used in the analysis, Region traffic shall provide timing files, timing sheets or Synchro files of the existing settings. If optimized timings are to be developed, those settings are subject to approval by ODOT and those conditions become the baseline for all comparisons.

The following settings should be optimized for each analysis scenario when the analyst is asked to use optimum coordination settings.

- Cycle Length
- Phase Length (Splits)
- Phase Sequence (Lead/Lag Left Turns)
- Intersection Offsets

The optimum settings must meet the criteria established in OAR 734-020-0480 as it relates to progression analysis while also attempting to find the lowest v/c ratio for each intersection. This OAR only applies when modifications are proposed to a signal which would affect the settings of the coordination plans. Examples of these modifications are changes in cycle length, decreased green time for mainline, additional phases, longer crosswalks and intersection relocation. For specific software setting requirements refer to Appendix 12/13.

<u>Adaptive Signal Timing</u>

In non-adaptive/responsive control, signalized intersections operate off of set timing plans that are programmed into the signal controllers. Adaptive Signal Timing (AST) technology allows the signal controller to continuously vary the signal timing (green time or splits) based on detection of real time traffic flows. AST is normally installed as a system with multiple signals in coordination and focuses on progression. AST can better progress traffic when the signal system is under-saturated as compared to set timing plans. When these signals operate under fully saturated or oversaturated conditions the timing can be more consistent since the splits are maxed out. There are different types of AST platforms that are currently available and installed on ODOT facilities and those of local jurisdictions (i.e. SCOOT, SCATS, RHODES, OPAC, Insync, Synchro Green, etc).

Traditional capacity analysis methods based on the Highway Capacity Manual (i.e. Synchro, SimTraffic, HCS, Traffix, etc) analyze signalized intersections assuming a set timing scheme and do not model AST behavior. Multiple analysis methods are possible. The simplest method to analyze AST is to assume all intersections are actuated and coordinated, and to optimize the signal timing even for Existing conditions.

Other possible methods include

- Run different scenarios over a full range of cycle lengths and splits and take the average of the results.
- Some adaptive signal controller data can be input directly into Vissim.
- Use Vissim's custom adaptive signal timing

Future Signals

For future signals, left turns should be assumed to have the appropriate phasing (i.e., permitted, protected-permitted or protected only) according to the criteria for left turn treatment contained in the current ODOT Traffic Signal Policy and Guidelines. The Region Traffic Section and the Traffic-Roadway Section should be consulted any time a new signal is proposed. It should always be considered that while new traffic signals provide a benefit to some users, the capacity of the mainline is typically cut in half by new signal installations and improper or unjustified signals can increase the frequency of rear-end collisions, delays, disobedience of signal indications and the use of less adequate routes.

Signal Timing Sheets

If it is desired to closely match the current traffic operations, the timing parameters installed in the signal controller need to be used in the analysis. The field timing parameters are recorded on the signal timing sheets located in the signal cabinet. Signal timing sheets should be obtained from the Region Traffic office as they generally have the most recent copies from the signal cabinet. Signal timing changes frequently, so the analyst should make sure to have the most recent version. For the analyst, not all of the included sheets are necessary, but it is important that all of the needed sheets are obtained. The following shows the important sheets (Exhibit 13-4 through 13-10, Sheets 2, 3, 6, 7 and 8. Sheets 4 and 5 are required if multiple timing plans exist) and what to look for on each sheet. The example signal timing sheet used to illustrate this section is the intersection of US 97 (Bend Parkway) and Pinebrook Boulevard in Bend.

<u>Sheet 2 – Phase Rotation Diagram</u>

The phase rotation diagram shows how the signal operates through its cycle. This diagram is needed so the signal is entered correctly into Synchro or other program. For complicated phasing, the diagram is an invaluable source. Exhibit 13-4 shows a phase rotation diagram for US 97 and Pinebrook Boulevard, which is a two-phase signal. Many timing sheets, especially the electronic ones, are missing the phase rotation diagram. Contact the appropriate Region Traffic section to obtain.

Exhibit 13-4 Signal Timing Sheet 2

SHEET 2

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

Table Numbers refer to Trafficview & Translink

TABLE 3

Clock, EV and Misc. (C + Key)									
Function	Key	Clock Location C + 3 uses Call / Active Display Sunday = 1							
Year	0								
Month	1								
Date	2								
Day of Week	3								
Hour	4								
Minute	5								
Second	6								
1/10 Second	7								
		Phase Number							
		1	2	3	4	5	6	7	8
	8								
Handicap Ped	E								

TABLE 6 (also see sheet 6)

Miscellaneous (D + Code)			
Function	Code	Value	Notes
Floating Ped	2E		0 = Off 1 = On (Ph. 7 & 8 Not permitted)
ID Number	2F	061	Range 0 to 253 (1)
Coordination Ped Recalls	3E	1	0 = Recall 1 = No Recall
Rest in WALK	3F		0 = Off 1 = ON
Advance Warning End of Green	4E		Extend time for green after sign turns on (2) (5)
Advance Warning	4F		Delay time for sign after yellow (2) (5)
			red flash
			1 = Flash Flash Red
NEMA Inputs	66		Non zero value reassigns C1 inputs. (3)
Bus Delay	6D		Delay time before preemption (4)
Bus Timer #1	6E		Extension of max green for phases 2 & 6 (Free operation)
Bus Timer #3	6F		Force off time for Ph 4 & 8 (only Free operation)
JHK Protocol	76		0 = no 0.1 = yes
JHK Area No. & 1st digit local	7D		Area No. 0 - 7 and Local 001 - 510 (1)
EV minimum timed Start / end of call	7E		0 = at start of call 1 = at end of call
EV On Indicators	7F		0 = Off, 1 = Flash, 5 = solid indication (5)

Phase Rotation Diagram

TABLE 3

Preemption Data (E+Key)			
Function	Key	Parameter	Timing
EVA	0	Delay	0
	1	Minimum	1
EVB	2	Delay	0
	3	Minimum	1
EVC	4	Delay	0
	5	Minimum	1
EVD	6	Delay	0
	7	Minimum	1
Overlaps	8	Red Revert	5.0
Railroad	9	Delay	
	A	Minimum	

		Phase Number							
		1	2	3	4	5	6	7	8
RR Clear Ph	B								
RR Permit	C								
RR OL Permit	D								
Nema Hold Ph	E								
	F								

Notes

(1) JHK ID no. is formed by Area no. (0 to 7) and 3 digit Local no. (001-510). Left most digits entered as x.x in location 7D and rightmost as xx in location 2F

(2) See Sheet 6, Location B+0+E

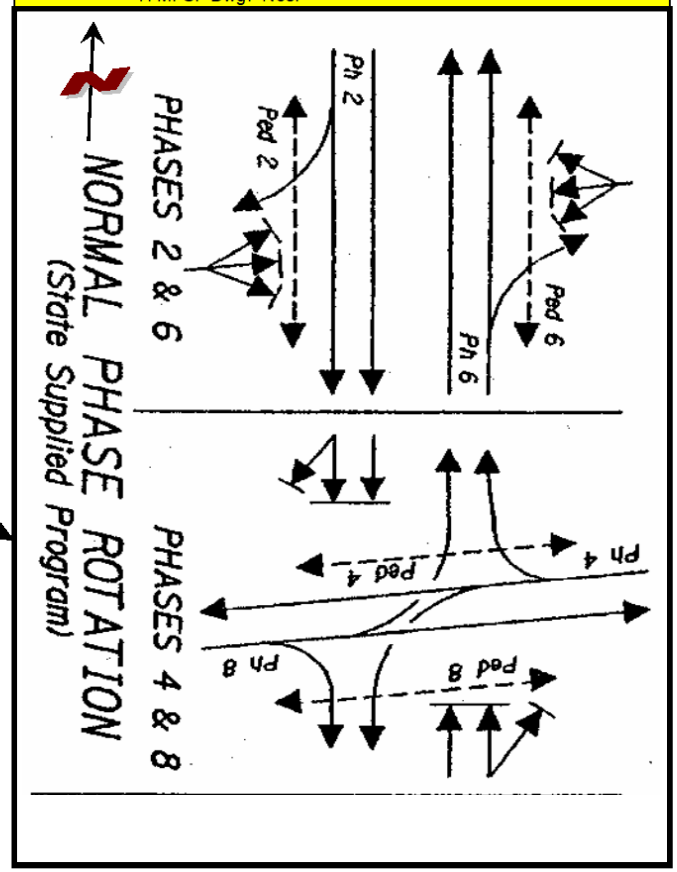
(3) C1 pins 54, 63, 64, 75, 76, and 77. See sheet 6, Location B+0+D.

(4) Entering 25.5 in this location is the only way of disabling bus preempt.

(5) Ped yellow outputs, C1-35, 36, 37, and 38 are used by Rt. Turn Overlaps, EV on indicators, TOD/DOW programmable outputs, Fiber Optic sign for RR flash yellow clearance, and Advance Warning sign operation.

Phase Rotation Diagram

T. M. S. Dwg. Nos:



Sheet 3 – Table 1 Phase Functions

Table 1 (Exhibit 13-5) shows the basic phasing properties and Exhibit 13-6 shows the pedestrian timings and the advanced actuated phasing properties needed for signalized analysis and simulation programs. Vehicle Recall (Key =0) shows what phases will appear for at least a minimum amount of time in each cycle the signal would return to if there is no demand on the side street. Permitted Phase (Key=4) shows what phases are present at this intersection. Overlap A-D (Key A-D) shows what phases operate together on each of the overlap outputs on the controller. If there are no checked boxes in this section, then there are no overlapping phases, but there may be signal heads displaying outputs from two phases such as the common vertical five-section right-turn signal head.

Sheet 3 – Table 1 Phase Timing

For non-coordinated signals, the cycle length and phase splits can be determined from the Phase Timing portion of Table 1. If multiple timing plans exist then they will be listed on Sheet 4 and/or Sheet 5. The only values that are needed to determine splits and cycle lengths from this portion of Table 1 are the maximum greens (Key = ph + 0), max 2 greens (Key = ph +1), yellow time (Key = ph + C) and all-red time or red clear (Key = ph + D).

The cycle length of actuated signals will vary from cycle to cycle depending on the vehicle demand. Synchro's phase splits include yellow and all-red, which is different from the maximum green on the timing sheet. Synchro also forces the maximum greens to add up perfectly to the cycle length. Therefore, the maximum cycle length needs to be proportionally adjusted down to match with Synchro's cycle length (the cycle length that is entered into the program). The maximum cycle length can be determined by summing the maximum greens (or max 2 greens if those are used in the analysis hour) and the yellow/all-red for each phase. The max green values on Sheet 3 are just that, i.e., maximum green times. The total maximum split used in Synchro will be the sum of the max green (or max 2 green), yellow and all-red. To convert the Sheet 3 timing into Synchro-compatible timing, the following is done.

1. Add up the Synchro cycle lengths from Sheet 3 by summing the maximum greens.
2. Add the yellow time and all-red time to the cycle length calculated in Step 1 to obtain the maximum cycle length.
3. The Synchro phase lengths are calculated by dividing the green + yellow + all-red time for a phase by the maximum cycle length. This ratio is then multiplied by the Step 1 Synchro cycle length.
4. Repeat for each phase.

The sum of the Synchro phases should add up to the Step 1 cycle length.

Exhibit 13-5 Signal Timing Sheet 3 – Basic Phase Settings

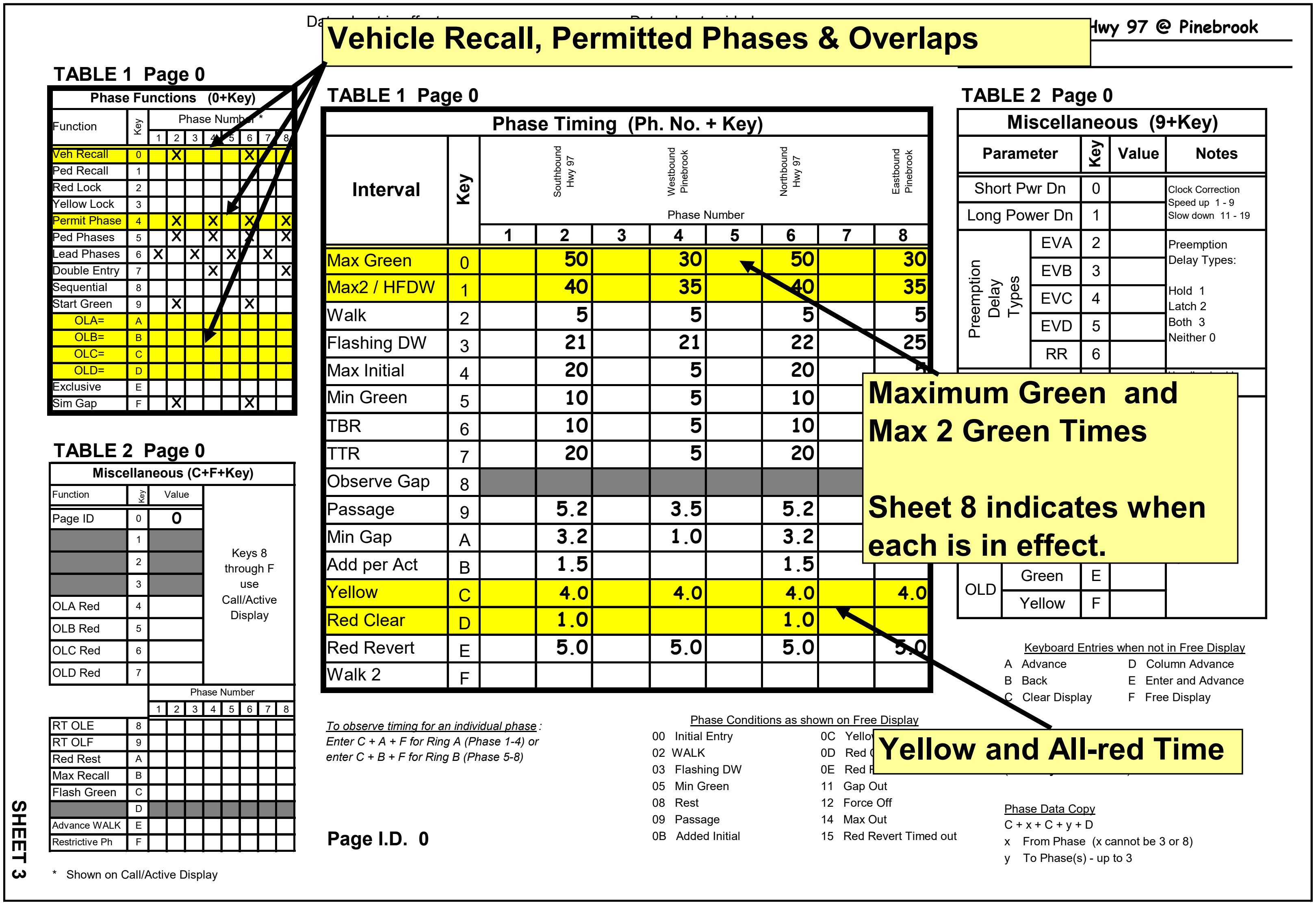


TABLE 1 Page 0

Phase Functions (0+Key)									
Function	Key	Phase Number * 1	2	3	4	5	6	7	8
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)		
Function	Key	Value
Page ID	0	0
	1	
	2	
	3	
OLA Red	4	
OLB Red	5	
OLC Red	6	
OLD Red	7	

Keys 8 through F use Call/Active Display

Function	Key	Phase Number 1	2	3	4	5	6	7	8
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)									
Interval	Key	1	2 Southbound Hwy 97	3	4 Westbound Pinebrook	5	6 Northbound Hwy 97	7	8 Eastbound Pinebrook
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		
Min Green	5		10		5		10		
TBR	6		10		5		10		
TTR	7		20		5		20		
Observe Gap	8								
Passage	9		5.2		3.5		5.2		
Min Gap	A		3.2		1.0		3.2		
Add per Act	B		1.5				1.5		
Yellow	C		4.0		4.0		4.0		4.0
Red Clear	D		1.0				1.0		
Red Revert	E		5.0		5.0		5.0		5.0
Walk 2	F								

To observe timing for an individual phase: Enter C + A + F for Ring A (Phase 1-4) or enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

Phase Conditions as shown on Free Display

00 Initial Entry
02 WALK
03 Flashing DW
05 Min Green
08 Rest
09 Passage
0B Added Initial
0C Yello
0D Red
0E Red
11 Gap Out
12 Force Off
14 Max Out
15 Red Revert Timed out

TABLE 2 Page 0

Miscellaneous (9+Key)				
Parameter		Key	Value	Notes
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types: Hold 1 Latch 2 Both 3 Neither 0
	EVB	3		
	EVC	4		
	EVD	5		
	RR	6		
OLD	Green	E		
	Yellow	F		

Keyboard Entries when not in Free Display

A Advance
B Back
C Clear Display
D Column Advance
E Enter and Advance
F Free Display

Phase Data Copy

C + x + C + y + D
x From Phase (x cannot be 3 or 8)
y To Phase(s) - up to 3

Exhibit 13-6 Signal Timing Sheet 3 - Advanced Phase Settings

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

SHEET 3

TABLE 1 Page 0

Phase Functions (0+Key)									
Function	Key	Phase Number *							
		1	2	3	4	5	6	7	8
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)		
Function	Key	Value
Page ID	0	0
	1	
	2	

Keys 8 through F

		1	2	3	4	5	6	7	8
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

Actuated Phasing Settings for Timing Plans and Simulation

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)									
Interval	Key		Southbound Hwy 97		Westbound Pinebrook		Northbound Hwy 97		Eastbound
		Phase Number							
		1	2	3	4	5	6	7	8
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		5
Min Green	5		10		5		10		5
TBR	6		10		5		10		5
TTR	7		20		5		20		5
Observe Gap	8								
Passage	9		5.2		3.5		5.2		3.5
Min Gap	A		3.2		1.0		3.2		1.0
Add per Act	B		1.5				1.5		
	C		4.0		4.0		4.0		4.0
	D		1.0				1.0		
	E		5.0		5.0		5.0		5.0
	F								

Walk and Flashing Don't Walk times

To observe timing for an individual phase :
Enter C + A + F for Ring A (Phase 1-4) or
enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

Phase Conditions as shown on Free Display

00	Initial Entry	0C	Yellow
02	WALK	0D	Red Clear
03	Flashing DW	0E	Red Revert
05	Min Green	11	Gap Out
08	Rest	12	Force Off
09	Passage	14	Max Out
0B	Added Initial	15	Red Revert Timed out

TABLE 2 Page 0

		Key		
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types: Hold 1 Latch 2 Both 3 Neither 0
	EVB	3		
	EVC	4		
	EVD	5		
	RR	6		
Ped Inhibit		7		Usually should be 0
OLA	Green	8		Overlap Yellow Time should always be specified
	Yellow	9		
OLB	Green	A		
	Yellow	B		
OLC	Green	C		
	Yellow	D		
OLD	Green	E		
	Yellow	F		

Keyboard Entries when not in Free Display

A	Advance	D	Column Advance
B	Back	E	Enter and Advance
C	Clear Display	F	Free Display

Reinitialization
D + 1 + F + 1 + E
(Use **only** when in flash)

Phase Data Copy
C + x + C + y + D
x From Phase (x cannot be 3 or 8)
y To Phase(s) - up to 3

Example 13-5 Signal Phase Splits

Example values for Sheet 3 are (Exhibit 13-6):

- Vehicle Recall = Phases 2 and 6 (US 97)
- Permitted Phases = 2, 4, 6 and 8. From the phase rotation diagram in Exhibit 13-4 it is seen that Phase 2 and 6 on US 97 go together and Phase 4 and 8 on Pinebrook go together.
- Overlaps = No overlapping phases

If this signal was not coordinated (it isn't) then the maximum cycle length would be the maximum greens plus the yellow times plus the all-red times. In checking Sheet 8 (Exhibit 13-10), it is found that the max 2 green time is in effect starting at 4:30 PM, so the max 2 green time will be used to calculate the cycle length.

Maximum Cycle length = Max 2 green for Phase 2 and 6 + Max 2 green for Phase 4 and 8 + yellow x 2 phases + all-red x 1 phase = 40 + 35 + (4 x 2) + 1= 84 seconds.

Synchro phase split conversion:

1. Synchro Cycle length = 40 + 35 = 75 s
2. Maximum cycle length = 75 + 4(2) +1 = 84 s
3. Synchro Phase 2&6 = ((40 + 4 + 1) / 84) x 75 = 40 s
4. Synchro Phase 4&8 = ((35 + 4) / 84) x 75 = 35 s
5. Check = 40 +35 = 75 s = Step 1 cycle length

In the above example the differences in the phase splits are small, resulting in Synchro splits that are the same as the timing sheet splits. The splits are different if the maximum greens were used instead of the max 2 greens, as shown below.

1. Synchro Cycle length = 50 + 30 = 80 s
2. Maximum cycle length = 80 + 4(2) +1 = 89 s
3. Synchro Phase 2&6 = ((50 + 4 + 1) / 89) x 80 = 49 s
4. Synchro Phase 4&8 = ((30 + 4) / 89) x 80 = 31 s
5. Check = 49 +31 = 80 s = Step 1 cycle length

For most new actuated signals, additional settings need to be pulled from Table 1. Pedestrian settings can have a large impact on signal operation and the resulting intersection v/c especially if there are a large number of pedestrian calls per hour on an approach. For creating a calibrated simulation, the actual pedestrian timing should be used as shown in Table 1 (Key= ph + 2 and Key = ph + 3) If the timing is not known, the ODOT standard walk time is 7.0 seconds with the curb-to-curb flashing don't walk time based on a 4.0 ft/s walk time.

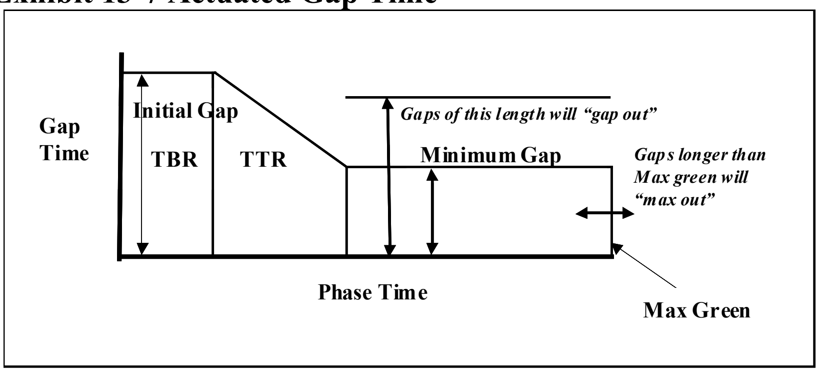
Table 1 also covers the actuated signal phasing parameters that are needed for creating timing plans and calibrated simulations. These five parameters are:

- **Minimum Green** (Key= ph + 5) - Minimum green time that a signal indication will occur for once the phase is served..

- **Time Before Reduce** (TBR) (Key= ph + 6) – Time elapsed before gap time is reduced
- **Time To Reduce** (TTR)(Key = ph + 7) - Time elapsed during gap time reduction to minimum.
- **Passage** (Key = ph +9) – This is the time that a phase is initially extended after a call is placed on a vehicle approach. Also known as initial gap.
- **Minimum Gap** (Key = ph + A) – Gap time after reduction until end of phase.

Exhibit 13-7 shows the progression of the gap time from when a green indication starts at the initial gap in the TBR period down to the minimum gap time. During the TTR period, the initial gap time is reduced down to the minimum gap time as specified on the timing sheet. If during the minimum gap time, the minimum gap is exceeded, then the signal will turn yellow (also known as a “gap out”). If vehicles keep approaching, the passage time will extend the green time to the maximum green time and then turn yellow (also known as a “max out”). Having a signal gap out is preferable, as dilemma vehicles (vehicles that either quickly accelerate or decelerate under yellow) can occur under max out conditions.

Exhibit 13-7 Actuated Gap Time



Sheet 6 – Table 6 Operation

Table 6 indicates whether or not the signal is ever coordinated over the course of a day or week. If Mode (Key = B+0+4) is a non-zero value, then the intersection is coordinated. The intersection may or may not be in coordination during the analysis periods. The actual times that coordination plans are in effect are entered on Sheet 8 of the local controller or on Table 5 of the On-Street Master Controller. Exhibit 13-8 shows that the example intersection is coordinated, but is not the master.

Exhibit 13-8 Signal Timing Sheet 6

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

SHEET 6

TABLE 6 (Also see sheet 2)

Key	Operation (B + 0 + Key) Parameter	Value
0	Present Plan	
1	Time of Day Plan	
2	Hardwire Plan	
3	MODEM Plan	
4	Mode (0 - 4 see right)	3
5	Master (0 - 4 see right)	0
6	Master Cycle Clock	
7	Local Cycle Clock	
8	Local Timer	
9		
A		
B		

Key		Phase Number 1	2	3	4	5	6	7	8
C									
D	NEMA CNA								
E	Adv. Warn.								
F	MRI Phases		X		X		X		X

Mode: *0 = Free, 1 = TBC, 2 = Hardwire, 3 = Modem, 4 = TM System*

Master: *0 = Off, 1 = Modem Master, 2 = Hardwire Master, 3 = 1 + 2, 4 = TM Master*

	Y	N
OSM ?		X

OSM Location

Powers Rd.

Coordination Mode and Master Type

Function		Time Clock On	Time Clock Off	Manual On	Manual Off	Notes
Outputs	A	71	81			*Phase 2 ped yellow (C1-35) (1)*
Outputs	B	72	82			*Phase 6 ped yellow (C1-36) (1)*
Outputs	C	73	83			*Phase 4 ped yellow (C1-37) (1)*
Outputs	D	74	84			*Phase 8 ped yellow (C1-38) (1)*
TOD Red Rest		25	24			*See Sheet 10 at B + C + D to set phases*
TOD Max Recall		27	26			*See Sheet 10 at B + A + E to set phases*
TOD Ped Recall		29	28			*See Sheet 10 at B + B + E to set phases*
WALK 2		55	54			*Use WALK 2 times set on Sheets 3, 4, 5*
Plan No.		1 - 18		1 - 18	0	*Sets operation to coordination plans on Sheet 7*
Free		20		20		*Sets operation to fully actuated*
Flash		19 or 33	32	19 or 33	0	*Sets operation to flash*
Max 2		129	128	129	0	*Use Max 2 times set on Sheets 3, 4, 5*
Det. Count 15		131	130			*Log Detector Counts - 15 min. intervals*
Det. Count 60		132	130			*Log Detector Counts - 60 min. intervals*
[illegible]		[illegible]				*Clear Detector Count Log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostics and log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostics without log*
Clear Det Diag.		138				*Clear Detector Diagnostic Log*
Send Real Time		199		199		*Modem master only*
Time Transfer		100		100		*Implements Page 0*
Time Transfer		101		101		*Implements Page 1*
Time Transfer		102		102		*Implements Page 2*
Page Copy				93	---	*Copies Page 0 data to Pages 1 & 2. Make sure Page 0 is the active Page*
Burn EEPROM				94	---	*Places active timing data into backup timing (Use reinitialization to place backup into active)*
Print Out				96	0	*Conect printer to C2 connector*

Note

(1) These C1 pins are used for other functions. See note (5) on Sheet 2.

Manual (D + 1 + E)	

TABLE 13 (Also see Sheet 10)

Miscellaneous (E + F + Key) Function	Key	Time
Railroad Max 2	0	
Ped Permissive Plan 1	1	
Ped Permissive Plan 2	2	
Ped Permissive Plan 3	3	
Ped Permissive Plan 4	4	
Ped Permissive Plan 5	5	
Ped Permissive Plan 6	6	
Ped Permissive Plan 7	7	
Ped Permissive Plan 8	8	
Ped Permissive Plan 9	9	
Number of Long Powerouts	A	
Number of Short Powerouts	B	
Failed Detector Number	C	
Max 2 On	D	
No Daylight Savings	E	
Revision Level	F	

TABLE 10 (Also see Sheet 12)

Sample Detectors (0 = off, 1 = on) (A + 3 + 9) [ ]

Sampling detectors are assigned using extended input codes on Sheet 11

For Protected / Permissive Left Turns

Left Turn Type (0, 1, or 2) (A + 3 + A) [ ]

0 = Off

1 = Left turn places call on cross street

2 = Left turn is omitted until cross street is serviced

Note: This feature works only with leading left turn phases 1, 3, 5, or 7. It is used to prohibit a green arrow from immediately following a green ball.

Sheet 7 – Table 7 Coordination Timing

If a signal operates in coordinated mode, then the timing shows up in Table 7. Timing values such as lead-lag settings on Sheet 7 override the values on Sheet 3. A signal controller will not exceed the max greens from Sheet 3 nor the force-offs (when the phase is forced "off" by the clock) on Sheet 7. The cycle length shown on Sheet 7 can be directly entered into Synchro. Using the force-offs the actual phase splits can be calculated. These values can also be directly entered into Synchro.

Exhibit 13-9 shows Table 7 for the example. In this case, Plan 2 with the 80 second cycle length is in operation during the afternoon peak. Read down the column. At 0 seconds Phases 2 and 6 are forced off. At 35 seconds Phases 4 and 8 are forced "off." Phases 2 and 6 operate from 35 seconds around to 0 seconds on the clock (80 – 35 = 45 seconds). In this case Phase 2 and 6 are 45 seconds and Phase 4 and 8 are 35 seconds. Note how this is would be different if this intersection was not coordinated, as shown under Sheet 3.

Exhibit 13-9 Signal Timing Sheet 7

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 7 (1 of 2)

Hardwire Conversion	Dial	1			2			3		
	Offset	1	2	3	1	2	3	1	2	3

Parameter	Key	Coordination Timing (B + Plan No. + Key)									
		1	2	3	4	5	6	7	8	9	Plan Number
Cycle Length	0	70	80								
Forceoffs for Phase Indicated by Key No.	1										
	2	0	0								
	3										
	4	31	35								
	5										
	6	0	0								
	7										
	8	31	35								
Offset	9	45	48								
Permissive	A	2	2								
Max. Dwell	B	30	35								

Coordination Timing Plans

Plan #2 is used in the example.

Sheet 8 of the master controller shows when each plan is in effect.

Plan			1	2	3	4	5	6	7	8
1	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
2	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
3	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
4	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
5	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
6	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
8	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
9	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								

TABLE 7 (2 of 2)

Parameter	Key 2	Coordination Timing (B + D + Key 1 + Key 2)									
		10	11	12	13	14	15	16	17	18	Plan Number
		7	8	9	A	B	C	D	E	F	Key 1
Cycle Length	0										
Forceoffs for Phase Indicated by Key No.	1										
	2										
	3										
	4										
	5										
	6										
	7										
	8										
Offset	9										
Permissive	A										
Max. Dwell	B										

Plan			1	2	3	4	5	6	7	8
10	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
11	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
12	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
13	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
14	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
15	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
16	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
17	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
18	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								

SHEET 7

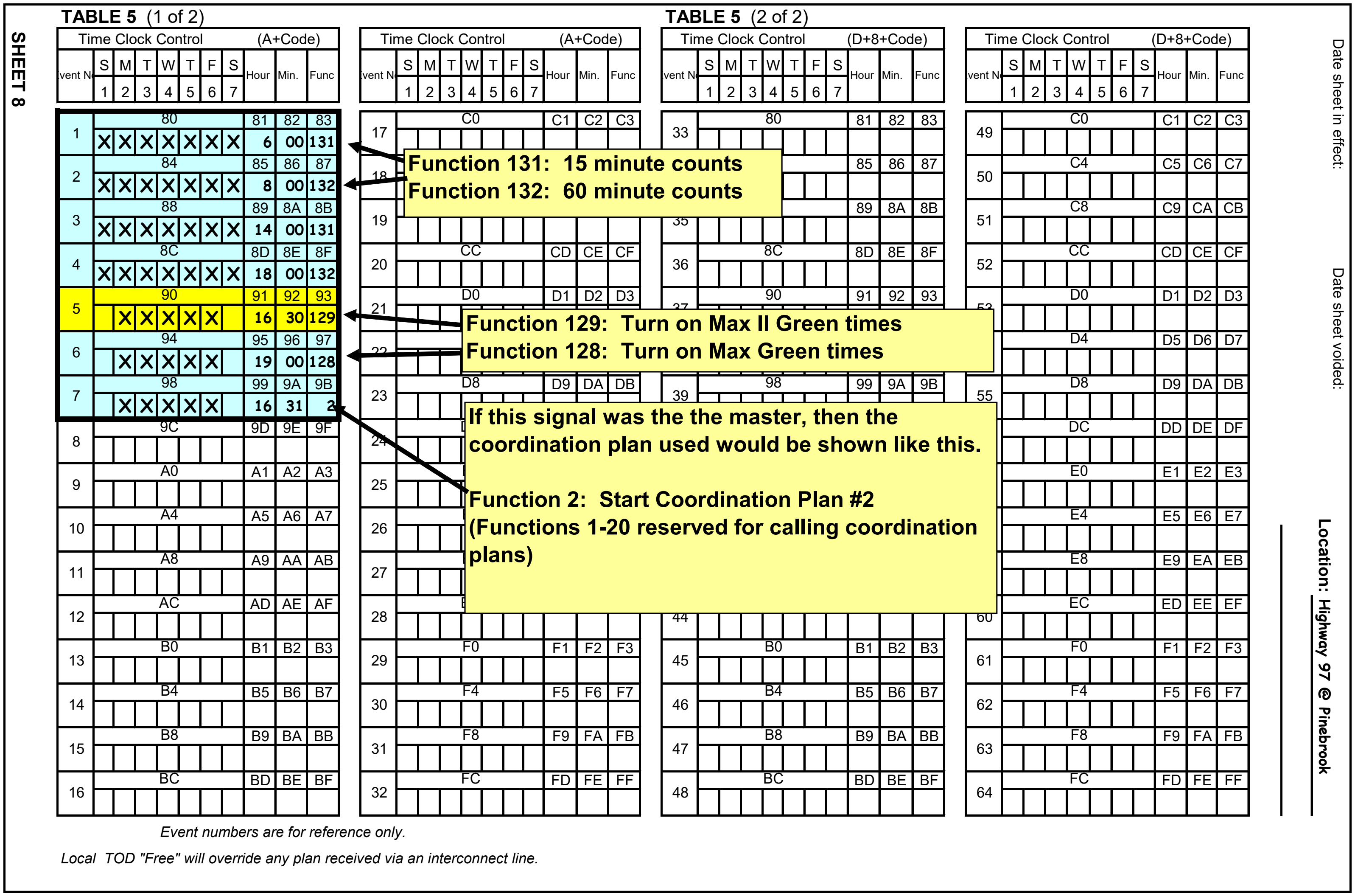
Sheet 8 – Table 5 Time Clock Control

Table 5 shows the times that various timing plans and max greens are in effect for a particular intersection. In the absence of timing sheets from an on-street master controller (noted as "OSM" on the front of the timing sheet), the analyst will have to contact Region Traffic to verify which timing plan on Sheet 7 is in effect during the desired analysis period. Generally, during the PM peak plan #2 is in effect. The master controller would indicate in Table 5 which coordination plan shown on Sheet 7 would be operating at any given time. The function codes in the right-hand column in Table 5 can tell the analyst what maximum green applies. Code 128 is for the maximum green while Code 129 is for the max 2 green. Codes 100, 101 and 102 apply to Page 0, 1, 2 (on Sheets 3, 4 or 5) respectively, so the analyst can determine what phase timing is in effect. Codes 131 and 132 are just to tell the controller to count the traffic volume data in 15-minute intervals or 60-minute intervals, respectively.

Exhibit 13-10 shows the timing plans in effect for the example intersection. The controller for this intersection is coordinated, but is not the master. If this signal was not coordinated, Code 129 would be indicated starting at 4:30 PM, in which case the max 2 green would be used for calculating the cycle length and phase splits.

If this controller was the master controller, an event would be listed showing when each plan went into effect. Event 7 has been added to the table to illustrate this.

Exhibit 13-10 Signal Timing Sheet 8



13.4.6 Progression Analysis

This section pertains to planning analyses as provided for traffic signal engineering investigations, corridor studies and other planning efforts. Oregon Administrative Rule (OAR) 734-020-0480 stipulates that a progression analysis is required for the approval of new or revised traffic signal systems if the proposed location is within ½ mile of an existing or possible future traffic signal. The roadway segment analyzed, to the extent possible, shall include all traffic signals in the existing or future traffic signal system. The purpose of a planning progression analysis is to ensure that a new signal or revised traffic signal will function acceptably with other nearby signals.

At the start of a project, ODOT traffic operations staff will determine whether the analyst should use the existing signal timings for all analysis scenarios or develop optimized timings for the coordinated system. If the existing timings are to be used in the analysis, Region traffic shall provide timing files, timing sheets or Synchro files of the existing settings. If optimized timings are to be developed, those settings are subject to approval by ODOT and those conditions become the baseline for all comparisons. The following settings should be optimized for each analysis scenario when the analyst is asked to use optimum coordination settings:

- Cycle Length;
- Side Street Phase Lengths (Splits);
- Phase Sequence (Lead/Lag Left Turns);
- Intersection Offsets; and
- Link Speed or Progression Speed

The optimum settings must meet the criteria established in OAR 734-020-0480 as it relates to progression analysis while also attempting to find the lowest intersection v/c ratio and minimizing queue lengths. This OAR only applies when modifications are proposed to a signal which would affect the settings of the coordination plans. Examples of these modifications are changes in cycle length, decreased green time for mainline, additional phases, longer crosswalks and intersection relocation.

Requirements for Signal Progression Analysis

For planning analysis, the following requirements must be met:

- Demonstrate acceptable existing and future traffic signal system operation during commute peak hours
- Provide for a progressed traffic band speed within 5 mph of the existing posted speed for both directions of travel during the off-peak periods and within 10 mph of the existing posted speed during peak periods. Approval by the State Traffic Engineer or designated representative shall be required where speeds deviate more than the above.
- Demonstrate sufficient vehicle storage is available at all locations within the traffic signal system without encroaching on the functional boundaries of adjacent lanes and signalized intersections. The functional boundary of an intersection

shall be determined using procedures specified by the ODOT Access Management Unit.

- Provide a common cycle length with adequate pedestrian crossing times at all signalized intersections.

The analysis must demonstrate that the additional or revised signal still allows the signal system to have a progression bandwidth as large as that required or as presently exists, for through traffic on the state highway at the most critical intersection within the roadway segment. The most critical intersection is the intersection carrying the highest through volume per lane on the state highway. Unless directed otherwise by ODOT traffic signal operations staff, the analysis should use optimized timing settings. The carrying capacity of the progression bandwidth should be estimated with the following equation:

$$\text{Bandwidth Capacity (veh/cycle)} = \frac{(\text{Bandwidth(sec)} - 4) \times (\text{Adj. Sat. Flow Rate})}{3600}$$

This capacity should be compared with the average platoon size expected to arrive at the most critical intersection for both directions of travel. The average platoon size may be found by the following simplified calculation.

$$\text{Average Platoon Size} = \frac{C * V}{3600}$$

where:

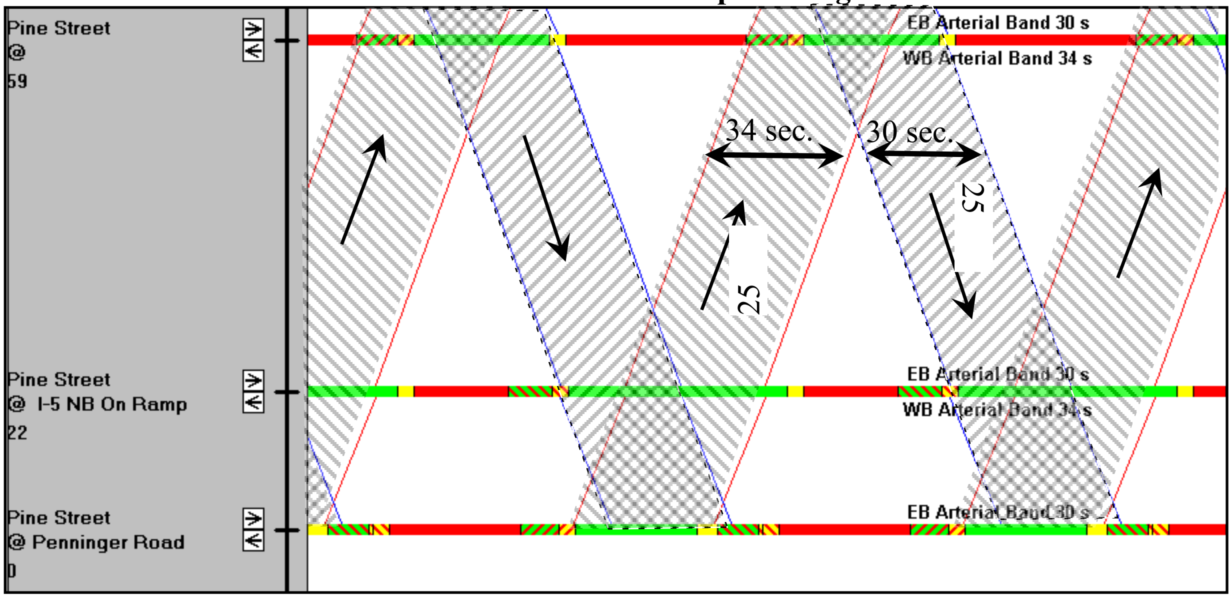
C = cycle length
V = volume (adjusted for PHF)

Complete time-space diagrams are required for each of the analysis scenarios, including the existing coordinated system. They should indicate the offsets, phasing and split times for each of the signals in the system. If using Synchro, the bandwidth shall be reported for the maximum green times or the 90^{th} percentile arrival rates. The reported bandwidth may include green and yellow clearance times. An example time-space diagram is shown in Exhibit 13-11.

If the analysis shows that the proposed signal will not meet the requirements of OAR 734-020-480, other alternatives should be evaluated. These may include:

- Moving the new/revised intersection;
- Reducing phases on one or more signals;
- Providing additional lanes to reduce side street green or increase mainline capacity
- Decrease side street demands through TDM measures or construction of alternative routes.

Exhibit 13-11 Illustration of Bandwidths on a Time-Space Diagram



To implement the requirements of OAR 734-020-480, analysts may use the coordinated system software program of their choice (see Section 13.5). Hand calculations and time-space diagrams are also acceptable. Refer to Appendix 13A for settings for each of these tools.

Microsimulation programs such as SimTraffic, CORSIM and VISSIM do not produce signal progression timing. They can model signal progression timing as an input. SimTraffic automatically models progression timing developed in Synchro. Refer to Chapter 15 for simulation guidance.

13.5 Estimating Queue Lengths for Signalized Intersections

For signalized movements, queue length estimates are most often recommended to be calculated using traffic analysis software. The use of software in estimating vehicle queue lengths can often be conducted simultaneously with capacity analysis, which can make it a very convenient method. There are many different software programs available that provide queue length estimates. However, caution should be used in selecting one as results may vary significantly between programs. As an example, the HCS has been found to produce consistently poor queue length estimates as compared to field measurements and should not be used for this purpose.

The minimum storage length for urban or rural left turn lanes at signalized intersections on state highways is 100 feet. Left Turn Lane layouts/dimensions are available in [HDM]() Chapter 8 Figure 8-9 and [Traffic Line Manual]() (TLM) Section 310.

Whether queue lengths have been calculated through manual methods or computer software, as a general rule-of-thumb the installation of signalized turn lanes with more than 350-feet of storage should be reconsidered through discussions with Region Traffic. In some cases, it may be preferable to install dual turn lanes with shorter storage bays.

For the estimation of queues at intersections belonging to a coordinated signal system, over-capacity conditions and areas where queue spill-back may be a problem, it is recommended that simulation software be used to report the 95th percentile queues. Refer to Chapter 15 for further information.

However, manual methods are also available that can offer acceptable estimates without requiring access to a computer. In either case, engineering judgment should be used to discern whether the results obtained are reasonable.

13.5.1 Manual Methods

Manual methods offer a practical means of estimating queue lengths with little equipment or data required. While they can produce reasonable results, unless otherwise noted, they are generally recommended for planning-level analysis, with the use of specialized software preferred for design purposes.

13.5.2 Left Turn Movement Queue Estimation Technique

A "rule of thumb" equation[2] that can be used to manually estimate queue lengths for single-lane left turn movements is shown below.

Storage Length = (Volume/Number of Cycles Per Hour) x (t) x (25-feet)

Where "t" is a variable, the value of which is selected based on the minimum acceptable likelihood that the storage length will be adequate to store the longest expected queue. Suggested values are listed in Exhibit 13-12. Typically, transportation analysis uses the 95th percentile queue.

Exhibit 13-12 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

[2] *Discussion Paper No. 10: Left-Turn Bays*, Transportation Research Institute, Oregon State University, 1996, p. 17.

It should also be noted that the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased, as shown in Exhibit 13-13. This adjustment is only for the manual methods; software packages may require a different adjustment.

Exhibit 13-13 Storage Length Adjustments for Trucks

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

While the rule of thumb equation is intended for use in estimating vehicle queue lengths for single-lane left turn movements, the vehicle queue lengths for double left turn lanes can be estimated by dividing the results of this method by 1.8. This value represents the assumption that queued vehicles will not be evenly distributed between the turn lanes.

13.5.3 Right Turn Movement Queue Estimation Techniques

A similar rule of thumb equation, sometimes referred to as the "red time" formula[3], is also available for signalized single-lane right turn queue estimates. It is represented by the following equation.

Storage Length = (1-G/C) (V) (K) (25-feet) / (Number of Cycles Per Hour) (N_L)

where:

G = Green time provided for the right turn movement
C = cycle length
V = right turning volume
K = random arrival factor
N_L = number of right turn lanes

A value of 2 should be used for the random arrival factor (K) where right-turn-on-red is prohibited. Where right-turn-on-red is allowed, a value of 1.5 should be used.

As with the equation for left turn queue estimates, the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased in the same manner recommended for the left turn queue estimate using Exhibit 13-13.

[3]Koepke, F. J., Levinson, H. S., *Access Management Guidelines for Activity Centers*, NCHRP Report 348, TRB, Washington, D.C., 1992, p. 99.

Another, less accurate, method for manually estimating vehicle queue lengths is using the assumption that “V” vehicles per hour per lane entering a signalized lane with a cycle length of 90 seconds will produce a “V”-foot-long queue per lane. For example, if the volume turning left from a dual left turn lane is 400 vehicles per hour, a ballpark queue length estimate would be 400/2 = 200 feet per lane.

13.6 Available Analysis Tools

A few of the computer software programs capable of performing operational, progression, and queuing analysis of signalized intersections include:

Synchro is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets and cycle lengths for individual intersections, an arterial or a complete network. Synchro performs capacity analysis using current HCM methods. Synchro provides detailed time space diagrams that can show vehicle paths or bandwidths. Synchro can be used for creating data files for SimTraffic and other third-party traffic software packages. The software supports the Universal Traffic Data Format (UTDF) for exchanging data with signal controller systems and other software packages. Synchro is used in conjunction with SimTraffic for microsimulation analysis (refer to Chapter 15).

Vistro is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets, and cycle lengths for individual intersections, an arterial or a complete network. Vistro performs capacity analysis using current HCM methods. Has embedded graphics to create customized reports including volume figures. Can create HCM 7th edition critical intersection v/c ratios without extra calculations or use of HCM 2000. Works well for multiple scenarios for a single intersection in the same file, such as all-way stop, two-way stop, roundabout, and signalized intersection. Can be used as a starting point to create a Vissim simulation network, or to detail a network from Visum. Refer to Appendix 8B PTV network setup guide. Good for lot of scenario management. Vistro is used in conjunction with Vissim for microsimulation analysis (refer to Chapter 15).

SIDRA is a software application for optimizing traffic signal timing and performing capacity analysis. The software optimizes splits, offsets, and cycle lengths for individual intersections, an arterial or a complete network. SIDRA performs capacity analysis using current HCM methods and offers enhancements through extensions. SIDRA will also reduce lane capacities in a network based on oversaturated upstream or downstream segments. Full flexibility to handle non-standard intersections easily (e.g. three-way stops), multiple modes (e.g. bicycles, streetcars) and related facilities (e.g. bus lane).

[Appendix 12A/13A – Software and Settings for Intersection Analysis]

References

1 Nevers, B., H. Steyn, Y. Mereszczak, Z. Clark, N. Rouphail, J. Hummer, B. Schroeder, Z. Bugg, J. Bonneson, and D. Rhodes. *NCHRP Report 707: Guidelines on the Use of Auxiliary Through Lanes at Signalized Intersections.* Transportation Research Board of the National Academies, Washington, D.C., 2011.

2 Thomas Creasey, F & Stamatiadis, Nick & Viele, Kert. (2011). Right-Turn-on-Red Volume Estimation and Incremental Capacity Models for Shared Lanes at Signalized Intersections. Transportation Research Record: Journal of the Transportation Research Board. 2257. 31-39. 10.3141/2257-04.

3 Right-Turn-on-Red Volume Estimation and Incremental Capacity Models for Shared Lanes at Signalized Intersections, F. Creasey, Nikiforos Stamatiadis, Kert Viele, Transportation Research Record: Journal of the Transportation Research Board Dec 2011, Vol. 2257, pp. 31-39

4 *Right Turn on Red Study Minnesota*, Finkelstein, Jonah et al, Spack Consulting, 2017