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Analysis of Isolated Traffic Signal Control Systems

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ANALYSIS OF ISOLATED TRAFFIC SIGNAL CONTROL SYSTEMS

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ANALYSIS OF ISOLATED TRAFFIC SIGNAL CONTROL SYSTEMS

by

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THESIS

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Abstract

Intelligent Transportation Systems (ITS) are implemented for increase of efficiency and safety in transportation. This thesis focuses on the area of traffic signal timing operations which is one of ITS components. The first objective is to compare the Czech and the U.S. methods for isolated fixed time traffic signal control, using an intersection in El Paso, Texas, as the test site. The second objective of this thesis is to evaluate the U.S. actuated isolated timing plan and compare it against the Czech and U.S. fixed time controls. A microscopic traffic simulation model of the selected intersection is coded in VISSIM to perform the comparative evaluation of average delay at the intersection. Ring Barrier Controller (RBC) is an additional module in VISSIM helping to create and evaluate the actuated timing plan. This module also allows the creation and evaluation of Transit Signal Priority (TSP). As the third objective, a few different scenarios with different bus arrival times and bus headways are evaluated in RBC to show its impact on bus delay and average delay of other vehicles at the intersection.

Table of Contents

List of Tables

List of Figures

Chapter 1: Introduction

1.1 Background

Transportation development is characterized by the continuous growth of number of vehicles in large cities and increasing demands for people or freight transportation. Growing traffic volume brings necessarily increasing a number of undesirable situations, such as increase of travel time, fuel consumption and congestion of transportation infrastructure.

Intelligent Transportation Systems (ITS) are implemented for increase of transport efficiency and safety. These systems integrate information and telecommunication technologies.ITS vary in technologies applied from basic management systems to monitor applications and to more advanced applications that integrate real-time data and feedback from a number of other sources. (Edgeict.com, 2011)

This thesis focuses on the area of traffic signal timing at intersections and its design and also on the area of transit signal priority, which are two components of ITS.Traffic signals are devices for regulating, directing or warning motorists or pedestrians. They help to separate competing flows of traffic, improve safety, reduce average delay through an intersection and increase intersection capacity. Signal timing involves deciding how much green time the traffic lights shall provide at an intersection approach, how long the pedestrian walk signal should be, and many other factors. Many intersections have some mechanism for detecting vehicles as they approach the intersection, and respond to the demands for right-of-ways in real-time. If so, an

intersection is said to be under actuated signal control, otherwise, its signal is said to be under fixed time control.

Actuated operation is especially good where traffic demands on the various approaches fluctuate over a wide range during the course of a day(Hubbell, 2001). The advantage of actuation is that green time is given based on traffic demand and the green signal indication can end as soon as the demand is gone. During low volume periods, the cycle length will be reduced and the green times will be relatively short. During heavy volume periods, the cycle lengths will increase and the green times will be longer. This provides for maximum traffic throughput with a minimum of delay to stopped vehicles(Hubbell, 2001).

Transit Signal Priority (TSP) is an operational strategy that is applied to reduce the delay experienced by transit vehicles at traffic signals. TSP system at an intersection involves communication between buses and the traffic signals so that the controller can alter its signal to give priority to transit vehicles. Transit signal priority has the potential to improve transit reliability, efficiency, and mobility.

Here are two basic methods to adjust the signal timing at an intersection for an approaching bus when implementing TSP: reducing the red time (red truncation) or extending the green time (green extension). Transit signal priority has a limited effect on signal timing because it adjusts to normal timing and logic to serve a specific vehicle type. The priority algorithm modifies the green allocation and may work within the constraint of signal coordination(FHWA, 2009).

1.2 Objective and Scope

The first objective of this thesis is to compare the Czech and the U.S. methods of isolatedfixed time traffic signal control, using an intersection in El Paso, Texas, as the test site. The second objective is to evaluatethe U.S. actuated isolated timing plan and compare it against the Czech and U.S. methods of isolated fixed time control using the same intersection in El Paso, Texas.A microscopic traffic simulation model of the selected intersection will be coded in VISSIM to perform the comparative evaluations. The third objective is to evaluate TSP using the RBC add-on module in VISSIM. Two TSP strategies, early green and extended green, are used at the intersection in El Paso, Texas.

1.3 Organization of Thesis

Chapter 1 of the thesis introduces a need to study traffic signal timing design methods in fixed or actuated form for optimization of traffic flow in the intersection and for the topic of the TSP. Chapter 2 covers the literature review which aims to study differences between traffic signal timing methods used in the U.S. and in the Czech Republic. Methods and research of TSP are examined in this chapter as well as the need to use the VISSIM software for the microscopic simulation. Chapter 3 details the site selection and the data collection used for the purpose of this thesis. Chapter 4 covers model development in VISSIM software and Chapter 5 addresses different design methods for the Czech and the U.S. fixed time control plan as well as for the U.S. actuated timing plan. Chapter 6 is the first of the two evaluation chapters. This chapter evaluates isolated signal timing operations. Chapter 7 evaluates the TSP operations. Chapter

8summarizes the results achieved in the thesis and makes conclusions and recommendations for the future research.

Chapter 2: Literature review

2.1 Traffic Signal Design in the U.S.

This section reviews the fixed time traffic signal control method of designing and implementing the U.S. signal timing plan. Mannering et al.(2009) describes the design procedure.In this thesis, an intersection in El Paso was chosen with a certain number of lanes on each approach. Therefore, some related parameters were already used as fixed inputs for calculations needed for this method. The last part of this section describes principles of the actuated traffic signal timing plan used in the U.S.

2.1.1 Fixed Time Control Plan Calculations

Signal phases, their sequence and saturation flow rates are pre-determined as inputs in the U.S. method of fixed time signal design. Research has found that a typical maximum saturation flow rate of 1900 passenger cars per hour per lane (cars/hr/lane) is possible at signalized intersections, and this is referred to as the base saturation flow rate. However, traffic factors include lane widths, grades, curbside parking maneuvers, the distribution of traffic among multiple approach lanes, the level of roadside development, bus stops, and the influence of pedestrians, bicycles, and heavy vehicles alter the base saturation flow rate(Mannering et al., 2009). It means that real saturation flow rate is usually less than the maximum saturation flow rate. For the purpose of this thesis, values of saturation flow rate were taken from the default

settings in the SYNCHRO model(Trafficware). This approach is based on the practice of the City of El Paso Transportation Department.

Lost time isdue to the traffic signal's function of continuously alternating the right-of-way between conflicting movements, resulting in traffic streams continuously start and stop. The total lost time for a movement during a cycle consists of start-up lost time and clearance lost time. (Mannering et al., 2009) It is conventional to use a value of 2.0 seconds for start-up lost time and clearance lost time respectively and this value was used in this thesis as well.

The next step of the design is to determine critical lane groups and total cycle lost time. The critical lane group for each phase is the lane group with the highest ratio of vehicle arrival rate to vehicle departure rate and is designated asv/s (traffic volume per lane$v$ divided by saturation flow rates). In addition, the sum of the flow ratios for the critical lane groups Y_c can be used to calculate a suitable cycle length, which is discussed furthermore. This is given by

$$Y_c = \sum_{i=1}^{n} \left(\frac{v}{s}\right)_{ci} \tag{2.1}$$

where$(v/s)_{ci}$ is flow ratio for critical lane group i, and n is a number of critical lane groups.

Total cycle lost time L is given as

$$L = \sum_{i=1}^{n} (t_L)_{ci} \tag{2.2}$$

where$(t_L)_{ci}$ is total lost time for critical lane group i in seconds.

The cycle length is the sum of the individual phase lengths. A practical equation for the calculation of the cycle length that seeks to minimize vehicle delay was developed by Webster. Webster's optimum cycle length formula is

$$C_{opt} = \frac{1.5L+5}{1-\sum_{i=1}^{n}\left(\frac{v}{s}\right)_{ci}} \tag{2.3}$$

The calculated cycle length is rounded up to nearest 5 seconds (as in the practice of City of El Paso Transportation Department). Therefore, this rounded cycle length is used in the green-time allocation calculation. Equation 2.4 can be arranged to solve for X_c as follows:

$$X_c = \frac{\sum_{i=1}^{n}(\frac{v}{s})_i C}{C-L} \tag{2.4}$$

The allocated green time for each phase is then calculated as

$$g_i = \left(\frac{v}{s}\right)_{ci}\left(\frac{C}{X_c}\right) \tag{2.5}$$

The minimum green time required for pedestrian crossing is checked against the apportioned green time for the phase. If there is not enough time for a pedestrian to safely cross the street, the apportioned green time is increased to meet the pedestrian's need. The minimum pedestrian green time is given by

$$G_{\boldsymbol{p}} = 3.2 + \frac{L}{S_p} + \left(0.27 \times N_{ped}\right) \; for \; W_E \leq 3.05m \tag{2.6}$$

or

$$G_{\boldsymbol{p}} = 3.2 + \frac{L}{S_p} + \left(2.7 \times \frac{N_{ped}}{W_E}\right) \; for \; W_E > 3.05m \tag{2.7}$$

where

G_P = minimum pedestrian green time in seconds,

3.2 = pedestrian start-up time in seconds,

L = crosswalk length in meters,

S_p = walking speed of pedestrians, usually taken as 1.2 m/s,

N_{ped} = number of pedestrians crossing during an interval, and

W_E = effective crosswalk width in meters.

Green time adjustment for each phase is made if there is not enough green time for pedestrians to cross the street within their phase.

$$g_{i(adj.)} = G_{pi} - Y_i - AR_i \tag{2.8}$$

U.S. DOT (2009) observes a few restrictions that have to be kept and they are:

- Minimum "intergreen" time between the end of a green time for vehicles and beginning of a green time for pedestrians consists of amber time plus all red time for each phase
- Minimum WALK time for pedestrians is in the range of 4 to 7 seconds. When there are less than 10 pedestrians during the cycle, minimum WALK time is only 4 seconds (which is the case for the intersection in El Paso in this thesis)
- Minimum green time for vehicles is 5 seconds.

The total cycle length C can be then calculated as

$$C = \sum_{i=1}^{n} g_i + \sum_{i=1}^{n} AR_i + \sum_{i=1}^{n} Y_i \tag{2.9}$$

where

i = phase,

n = number of phases,

g_i = green time for phase i,

AR_i = all red time for phase i,

Y_i = amber time for phase i.

The calculated cycle length is rounded up to a nearest 5 seconds again. Therefore, green times for the phases should be adjusted according to Equations (2.4) and (2.5) once again.

2.1.2 Fixed Time Control Plan Design

After the design calculations described above have been completed, the signal plan diagram can be drawn.An example of the signal plan is shown below.

Figure 2.1: Example of signal plan based on the U.S. design method

In the signal plan chart above, all green, yellow, all-red times are clearly visualized as well as pedestrian WALK and DON'T WALK times. The phase sequence and traffic signal timing are easily recognized as well.

2.1.3 Actuated Timing Plan in the U.S.

Roess et al. (2001)provide a review of the design and operations of the U.S. actuated traffic signal timing plan. Actuated signal controllers are manufactured in accordance with one of two sets of standards. The most common is the standards of the National Electronic Manufacturer's Association (NEMA). The NEMA standards specify all features, functions, and timing intervals. The second set of standards is for the Type 170 class of controllers. Due to the common usage and the compatibility with the Ring Barrier Controller (RBC) controller simulation in the VISSIM software, NEMA was chosen as an example to follow in this thesis.

Basically, there are few actuated controller features that need to be understood. They are summarized in the following table.

Table 2.1: Actuated controller features

Feature	Symbol	Description
Minimum green time	G_{min}	The smallest amount of green time for phase
Unit/Vehicle extension	U	The amount of time added to the green phase when an additional actuation is received within the unit extension
Maximum green time	G_{max}	Time that limits the length of a green phase, even if there are continued actuations that would normally retain the green
Recall switches	Recall	Recall switches determine what happens to the signal when there is no demand (Normally, one recall switch is placed on the "on" position, while all other are turned "off")
Yellow & All-red intervals	Y, AR	Times providing safe transition from "green" to "red" (They are fixed times)
Pedestrian WALK, Clearance & DON'T WALK	Walk, Ped Clearance	Pedestrian intervals which are set in accordance with the minimum green time for each phase

Knowledge on the operations of the actuated control is essential to the evaluation. For better visualization, the following figure is provided as an illustration.

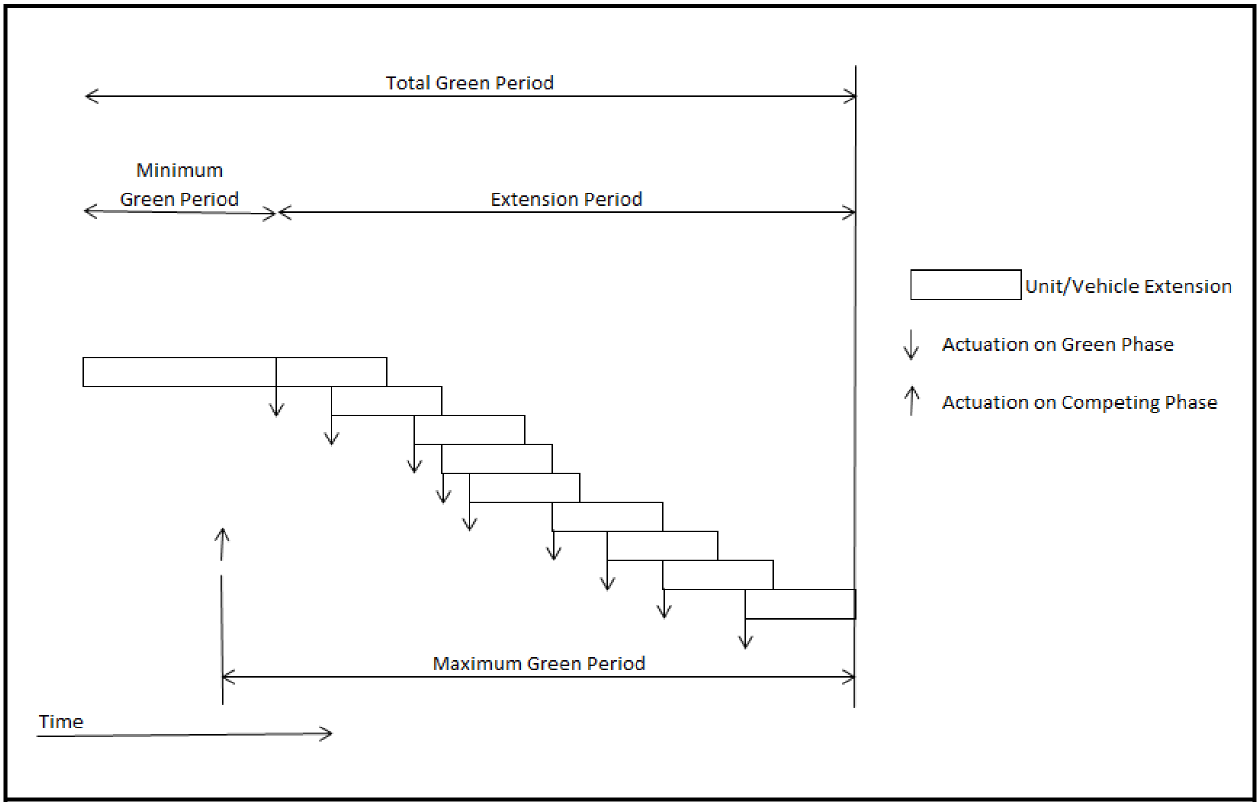


Figure 2.2: Process of actuations

There are three critical parameters in the figure: minimum green, maximum green, and unit/vehicle extension. When the green is initiated for a phase, it will be at least as long as the minimum green period G_{min}. The controller divides the minimum green into an initial portion and a portion equal to multiples of one unit extensions. If an additional "call" (demand for service by a vehicle) is received during the initial portion of the minimum green, no time is added to the phase, as there is sufficient time within the minimum green to cross the STOP line. If a "call" is received during the last *U* seconds of the minimum green, *U* seconds of green are added to the phase. Thereafter, every time an additional "call" is received during a unit extension of *U* seconds, an additional period of *U* seconds is added to the green. The "green" is terminated in one of two ways: a unit extension of *U* seconds expires without an additional actuation or "call",

or the maximum green has been reached. The maximum green begins timing out when a “call” on a competing phase is noted(Roess).

Roess (2001) calculates all parameters needed for actuated time traffic signal control design. However, for the purpose of this thesis, all parameters with its values for selected intersectionwere provided by the City of El Paso Transportation Department and therefore there is no need for any calculation.

2.2 Traffic Signal Design in the Czech Republic

This section reviews the method of designing and implementing of the Czech fixed time traffic signal control plan. According toEDIP(2008)and for the purposes of signal plan calculations and design, the Webster method was chosen as a typical method used in the Czech Republic. The Czech design method covers the whole design process including the determination of number of lanes needed based on known traffic volumes.

2.2.1 Implementation of the Czech Signal Timing Plan

In this thesis, an intersection in El Paso was already chosen with certain number of lanes on each approach. Therefore, some related parameters are fixed inputs for calculations in this method. In addition to the existing number of lanes and existing phase sequence, there is a need to measure the geometry of the intersection as precisely as possible to calculate the split intervals. Split intervals calculation is the most time consuming part of the Czech method but on

the other hand, this is the most beneficial part which leads to saving in overall delay at the intersection.

2.2.2 Split Intervals Calculation

After the signal phase sequence is determined for an intersection, split intervals for all possible conflicting movements are calculated. Asplit interval is defined as the time interval from the end of a green time for one movement to the start of the green time for the next movement in order to avoid a collision. The following three equations are needed to calculate the split interval:

Time for vehicles entering

$$t_e = \frac{D_e}{v_e} \tag{2.10}$$

Time for vehicles clearing

$$t_c = \frac{D_c + L_{veh}}{v_c} \tag{2.11}$$

Split interval

$$t_{si} = t_c - t_e + t_s \tag{2.12}$$

where

t_{si} = split interval (rounded up to the next integer value),

t_c= time for vehicles clearing,

t_e = time for vehicles entering,

t_s = safety time (due to amber signal),

D_c = clearing distance,

D_e= entering distance,

L_{veh} = vehicle length,

v_c= clearing velocity,

v_e= entering velocity.

For the split interval calculation, the values from Table 2.2 are used.

Table 2.2: Values for split interval calculation

CLEARING AND ENTERING VELOCITIES	v
Vehicles in thru movement	35 km/h or 9.7 m/s
Vehicles in turning movement	25 km/h or 7.0 m/s
Pedestrians	5 km/h or 1.4 m/s
CLEARING VEHICLE LENGTH	L_{veh}
Vehicle	5 m
Pedestrian	0 m
SAFETY TIME	t_s
Vehicle	2 s
Pedestrian	0 s

In the split interval calculation, it is necessary to measure entering and clearing distances at the intersection. Drawing in AutoCAD software may be used for this purpose. The entering and clearing distances are calculated from the stop line of the movement to the possible conflict point. The most likely routes of vehicles are also considered here. It is necessary to recognize

which vehicle is entering and which one is clearing the intersection. For better understanding and illustration, an example from an AutoCAD drawing is shown below.

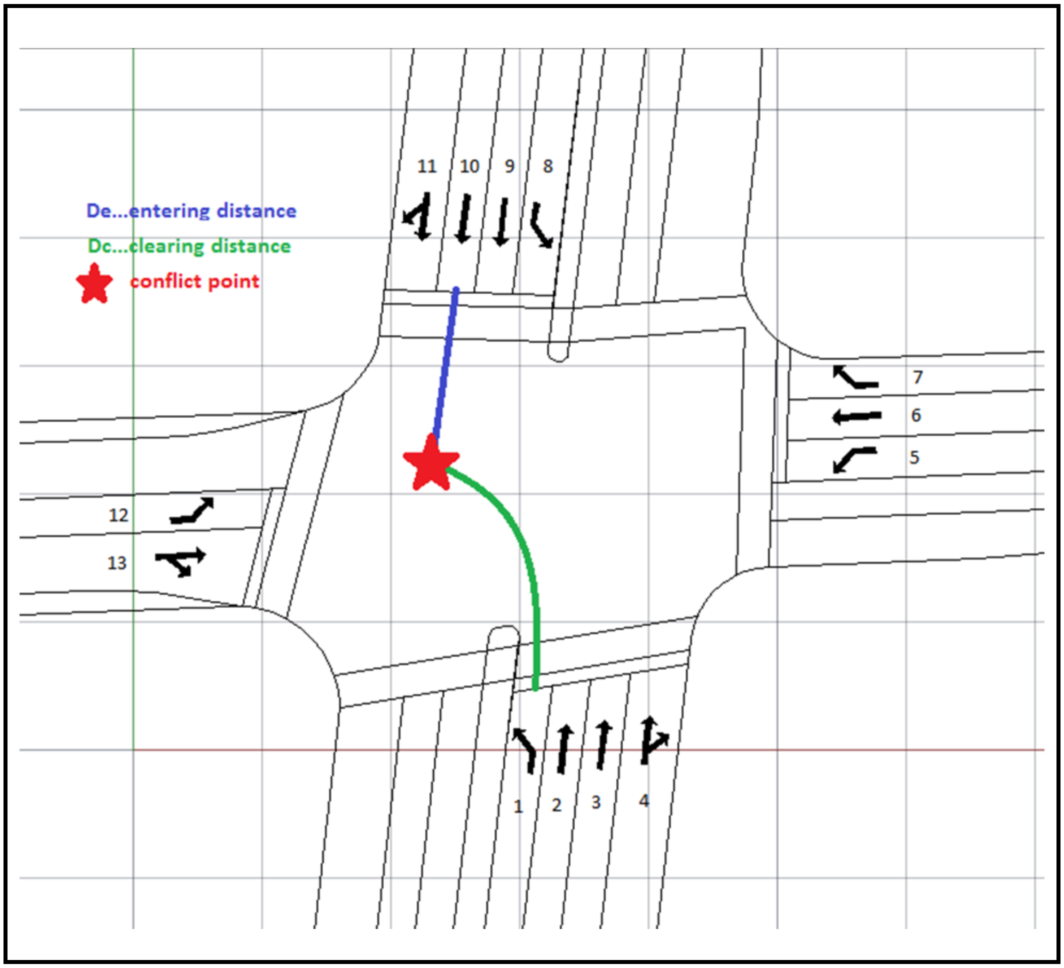


Figure2.3: Conflicting movements

In the figure, a vehicle from lane number 10 is entering the intersection (when the green signal starts) while vehicle from lane number 1 is clearing the intersection (when the green signal turns to yellow). It is possible to choose a vehicle from lane 9 as the entering vehicle but from Equation (2.3) it is obvious that the maximum value of split interval is for vehicles in lane 10. The split intervals for all possible conflicting movements in the intersection are calculated and

their values are recorded in the split intervals table. Table 2.3 is an example of the split interval table.

Table 2.3: Example of the split intervals table

			entering											
	Signal group		phase A		phase C		phase B				phase C		phase B	
		movements	1	5	2	6	3	8	4	7	P38	P47	P16	P25
clearing	phase A	1		x	5	x	3	3	4	2	2	x	x	x
		5	x		x	5	1	x	3	4	x	3	x	x
	phase C	2	2	x		x	4	4	1	1	1	x	4	x
		6	x	2	x		1	x	5	4	x	3	x	2
	phase B	3	3	6	4	5		x	5	x	x	x	x	x
		8	4	x	3	6	x		x	1	x	5	3	x
		4	2	4	5	2	1	x		x	4	x	x	1
		7	5	3	7	4	x	5	x		x	x	x	x
	phase C	P38	7	x	11	x	10	15	9	x		x	x	x
		P47	x	10	x	16	x	12	17	12	x		x	x
	phase B	P16	13	x	8	13	6	x	9	x	x	x		x
		P25	x	12	13	10	x	12	x	8	x	x	x	

Note: x means no conflict point was found for that movement and all values are in seconds.

2.2.3 Phase Sequence Optimization

From all possible phase sequences, the sequence with the minimal sum of relevant split intervals is selected. The relevant split interval is the splitinterval with the maximum value in that phasechange. This part is the biggest difference in the U.S. and Czech design methods.

2.2.4 Signal Plan Calculations

For signal timing plan calculations, the method of saturated flow (Webster method) was chosen. Saturated flow may be defined as the maximum number of vehicles which can pass the

stop line per unit of time under ideal traffic conditions. It is calculated from basic saturated flow S_b (veh/hr/lane), which is form

$$S_b = 1800 + 100 \times (w - 3.5) \tag{2.13}$$

where$w$ is the lane width in meters (subject to a maximum of 4 m).

ThisS_b value is calculated for all the lanes separately. Then, there is a calculation of the saturation flow of the entry lane.

$$S = S_b \times c_g \times c_a \tag{2.14}$$

where$c_g$is called the coefficient of gradient, c_a is the coefficient of arc. The coefficient of gradient is calculated for all lanes as

$$c_g = 1 - 0.2 \times a \tag{2.15}$$

The value of a depends on the real gradient of the approach lane. For gradients up to 10%, a is equal to 0(EDIP). This value is used for all lanes in the selected intersection in El Paso.

The coefficient of arc is calculated for all lanes as

$$c_a = \frac{R}{R+1.5\times f} \tag{2.16}$$

where

R = real radius of an arc [m],

f = ratio of traffic volume of turning vehicles / traffic volume of all vehicles.

If there is no exclusive left turn lane and the turn is permissive, then R = 1.5 m (fictitious radius). In case of an exclusive lane for the turning movement, the value of f is logically equal to 1.

The saturation entry per lane is calculated as

$$y = \frac{I}{S} \tag{2.17}$$

where I is the intensity (or traffic volume) of vehicles per lane. This parameter is calculated for all lanes.

Next, the maximum value of y is determined for each phase and that is denoted as y_{max}. The overall saturation level is determined as the sum of these maximal values.

$$Y = \sum_{i=1}^{n} y_{max,i} \tag{2.18}$$

where

i = phase,

n = number of phases.

The lost time for each phase is calculated next as

$$l_i = t_{rsi,i} - 1 \tag{2.19}$$

where $t_{rsi,i}$ is the relevant split interval for phase i. For better understanding, Figure 2.4 is added to help in this explanation.

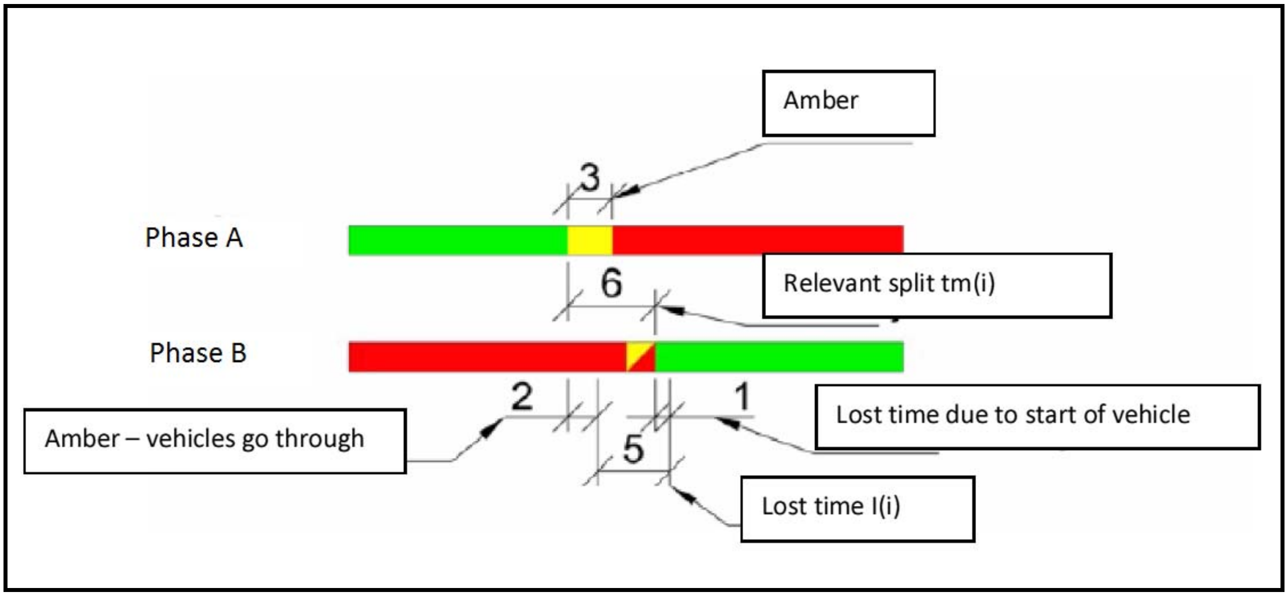


Figure 2.4: Calculated parameters in the signal phase change

The amber signal in the Czech Republic is 3 seconds. There is also 2 seconds of amber-red signal used before start of the green time for vehicles. This is common in the Czech Republic and this indication should tell the driver be ready for the green signal. Then, the total lost time is calculated as

$$L = \sum_{i=1}^{n} l_i \tag{2.20}$$

The calculation of optimal cycle length is based on the Webster equation:

$$C_{opt} = \frac{(1.5 \times L + 5)}{(I - Y)} \tag{2.21}$$

The real cycle length C must fall into the following range: $0.75 \times C_{opt} < C < 1.5 \times C_{opt}$. Within this range, the overall delay does not vary significantly. The proposed cycle length is rounded to the nearest 10 seconds, for example 40, 50, 60, 70 or 80 seconds. There are also some constraints which have to be considered. The minimal cycle length is 30 seconds, while the

maximal cycle length is 100 seconds. Only in special cases the cycle length can be extended up to 120 seconds.

After the determination of the cycle length, the next step is to calculate green time per each phase. This is calculated just for lanes where y_{max} is present.

$$g_{opt} = \left[y \times \frac{C-L}{Y}\right] - 1 \tag{2.22}$$

where

C = proposed cycle length,

Y = maximal level of saturation for certain phase.

After obtaining g_{opt}, it is converted to g, the actual value of green time which is close to g_{opt}.There are some restrictions on the value of g. These restrictions are:

- Minimal green time for vehicles and pedestrians is 5 seconds
- Minimal amber time is 3 seconds
- Amber-red signal is 2 seconds

The final cycle length then is a sum of all relevant split intervals plus the sum of all green times.

$$C = \sum_{i=1}^{n} t_{rsi,i} + \sum_{i=1}^{n} g_i \tag{2.23}$$

where

i = phase i,

n = number of phases.

The green times for pedestrians are added on from the split intervals table.

2.2.5 Signal Plan Evaluation

To know if the number of lanes in the intersection is designed well, there is a reserve capacity calculation. To determine the needed lane lengths, there is a lane length calculation as well.The first step is to calculate minimal green time assigned to a lane.

$$g_{min} = \left(I \times \frac{C}{S}\right) - 1 \tag{2.24}$$

Thecondition $g_{min} \leq g$ has to be kept.The lane capacity in number of vehicles per hour is calculated for all lanes.

$$K = S \times \frac{g+1}{C} \tag{2.25}$$

The reserve capacity is calculated then as

$$Reserve = \frac{I}{K} \times 100 \tag{2.26}$$

where*I* is intensity or traffic volume per lane.

These values should be more than 10%. If not, the number of lanes for the same movement should be increased and the signal timing plan redesigned.

The lane length evaluation is the last calculation and it is done for all the lanes again.

$$l = \frac{7 \times I \times (C - g)}{3600} \tag{2.27}$$

where$l$ is a minimum lane length needed in meters: and *7* means vehicle length plus gap between standing vehicles in meters.

The suggested lane length is 30 meters. If a higher value of l is calculated, then this new value is assigned to the parameter l_{real} which is the real lane length needed.

For the example used as illustration in this Chapter, the results of all calculations mentioned above are summarized in following table.

Table 2.4: Signal plan calculations

Phase	Lane	I	S(bas)	S	y	y(max)	Y	C(opt)	C	L	g(opt)	g	g(min)	K	Reserve	l	l(real)
1	1	23	1825	1659.09	0.014									165.909	13.863	2.460	30
2	2	152	1825	1825.00	0.083									547.500	27.763	12.709	30
2	3	152	1825	1825.00	0.083									547.500	27.763	12.709	30
2	4	151	1850	1707.52	0.088									512.256	29.477	12.625	30
3	5	464	1850	1681.82	0.276									728.788	63.667	31.578	32
3	6	8	1850	1850.00	0.004	0.276					25.989	25	3.534	801.667	0.998	0.544	30
3	7	101	1825	1586.96	0.064		0.511	40.910	60	10				687.681	14.687	6.874	30
1	8	77	1850	1681.82	0.046	0.046					3.479	5	0.796	168.182	45.784	8.235	30
2	9	341	1850	1850.00	0.184									555.000	61.441	28.511	30
2	10	341	1800	1800.00	0.189	0.189					17.532	17	7.173	540.000	63.148	28.511	30
2	11	340	1850	1835.42	0.185									550.627	61.748	28.428	30
3	12	29	1825	1659.09	0.017									718.939	4.034	1.974	30
3	13	35	1850	1651.79	0.021									715.774	4.890	2.382	30

In table 2.4, there are three cells highlighted by yellow color. These are the values for reserve capacity calculations and these values should be more than 10 according to condition(EDIP). It means lanes 6, 12 and 13 are not used as they should be and fewer lanes for the movement should be considered. However, these are lanes on existing intersection and no change will be done.

2.2.6 Signal Plan Final Design

All input parameters and design calculations described above are for passenger cars. Afterwards, other modes of transportation are considered.

There is one condition that has to be kept. Split interval between the end of green time for vehicles and the start of green time for pedestrians has to be at least 4 seconds (3 seconds for amber plus 1 extra second). So, if after calculation, there is somewhere split interval less than 4 seconds for these kinds of movements, that split interval has to be extended in final signal plan design to 4 seconds.The figure of the signal plan is below including at least 4 seconds for all the split intervals.

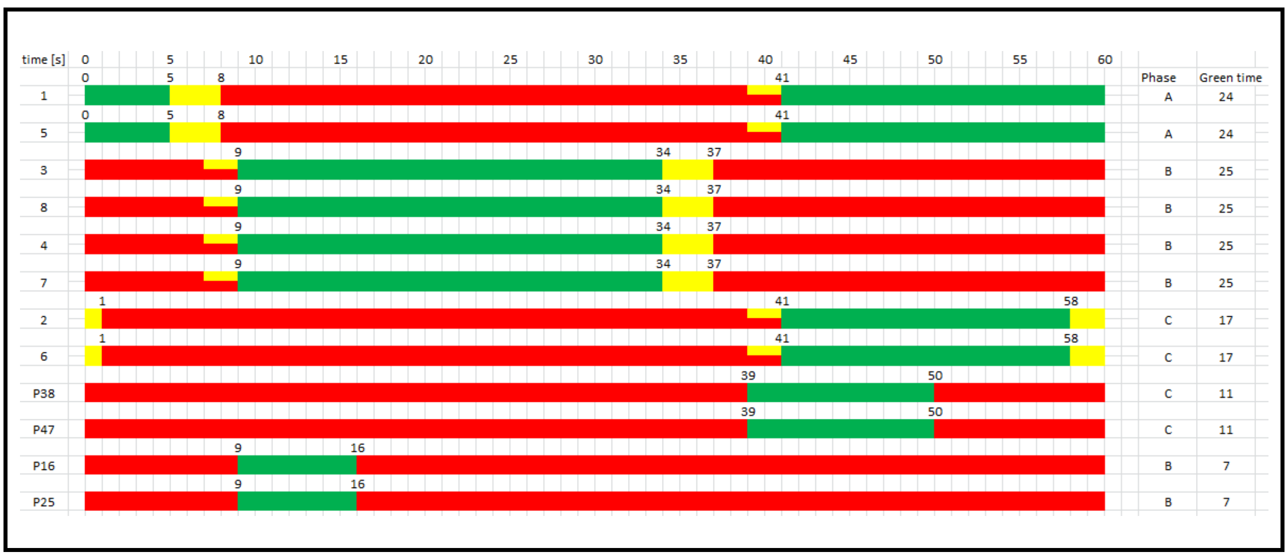


Figure 2.5: Example of signal plan based on the Czech design method

2.3 Transit Signal Priority

There are generally two classes of TSP strategies, passive and active. Passive priority strategies give priority to transit vehicles without the need for transit vehicle detection. Conversely, active priority strategies provide priority to transit vehicles after a transit vehicle is detected and priority conditions are met (Ova et al., 2001).

Active priority strategy is chosen in this thesis due to use of the intersection of High Ridge & N Resler Dr in El Paso where detectors are used for traffic flow management and monitoring.

Active priority strategies are dynamic signal timing enhancements, where the signal phases are modified upon the detection of a transit vehicle. This strategy provides for an efficient operation of the signal by responding to the transit call and then returning to normal operations after the call has expired or serviced. Active TSP strategies include four different types: early green, extended green, phase insert and phase suppression. This thesis examines two TSP strategies which are mostly used: early green and extended green(Ova et al., 2001).

Early green strategy is the process indicating a green light prior to the normal start of a priority movement or phase. This process is done by shortening the green time of the opposing phase, without violating the minimum green time, pedestrian movements, or clearance intervals, and returning to the priority phase. Extended green is similar to early green in the sense that the opposing phases are shortened after the priority phase was extended. Both methods are intended to allow for the passage of the transit vehicle in the most efficient manner, dependent upon the arrival time within the cycle(Ova et al., 2001).

Figure 2.6 provides a graphical comparison between early green and extended green strategies.

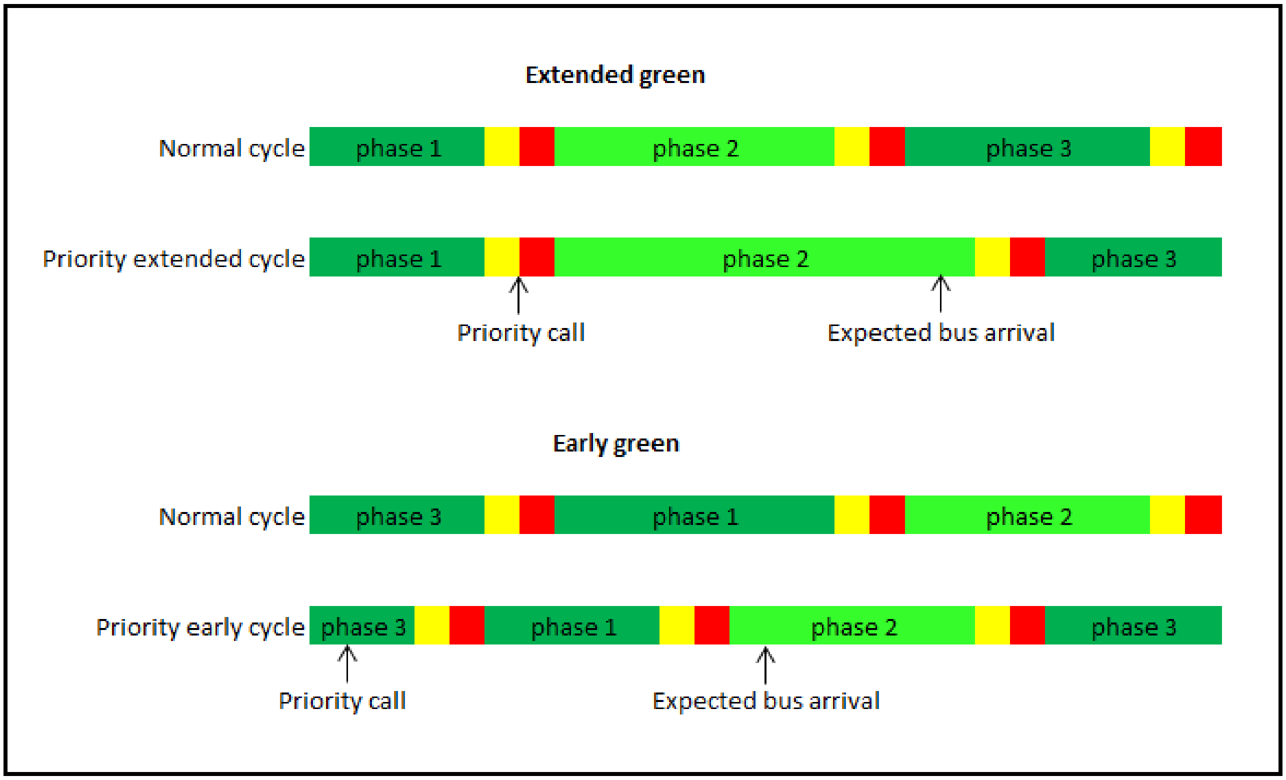


Figure 2.6: Example of extended green and early green strategy

Expected bus arrival needs to be calculated after a priority "call". There could be more factors influencing the calculation such as bus velocity and acceleration, presence/absence of bus lane, bus stop location, detector location, congestion level, etc.

Ideal conditions for use of TSP are following:

- A detector for a bus "call" is placed in sufficient distance from the intersection ensuring that there is enough time for traffic signals to adjust green time in each phase
- There is no obstacle in the way (bus lane) and no congestions expected so that calculation of expected bus arrival is precise

2.4 VISSIM Microscopic Traffic Simulation

VISSIM is the leading microscopic simulation program for multi-modal traffic flow modeling. With its unique high level of detail it accurately simulates urban and highway traffic, including cyclists and motorized vehicles(PTV, PTV AG: VISSIM - Multi-Modal Traffic Flow Modeling).

The program can analyze private and public transport operations under constraints such as lane configuration, traffic composition, traffic signals, public transportation stops, etc., thus making it a useful tool for the evaluation of various alternatives based on transportation engineering and planning measures of effectiveness. VISSIM can be applied as a useful tool in a variety of transportation problem settings(PTV, VISSIM 5.20 User Manual).

Zlatkovic(2009) presents a use of the VISSIM traffic simulation software in evaluating TSP strategies using two types of emulated signal controllers: the Ring Barrier Controller (RBC) emulator, and the ASC/3 Software-in-the-loop traffic controller. Those signal control types were evaluated for their abilities to provide TSP and the impacts it causes on the overall traffic. The results have shown benefits of the evaluated traffic controllers within VISSIM. This study served as an example how to use a RBC controller for VISSIM traffic simulation. This controller was chosen for TSP evaluation in this thesis due to its ability to simulate TSP strategies described in section 2.3.

Chapter 3: Site Selection and Data Collection

3.1 Site Selection

A signalized intersection site in El Paso, Texas was selected according to specific conditions. They were a following:

- The intersection is operated under actuated control
- The intersection is not coordinated with neighboring signalized intersections, i.e. isolated

After meeting with Mr. Tony Do, Transportation Engineer at the City of El Paso, the intersection of High Ridge and N ReslerDr was selected. Figure 3.1 shows the overall view of the intersection

Figure 3.1: High Ridge and N ReslerDr intersection (source: www.google.com)

3.2 Data Collection

Several different data were collected for the signal design calculations and simulation model development. The following four sub-sections describe the data collection effort.

3.2.1 Traffic Volume

Three different videos at the intersection of High Ridge and N ReslerDr were recorded. The purpose was to obtain hourly traffic volumes for three different periods – morning peak hour, noon off-peak hour and afternoon peak hour during a typical working day. After observations of the video recordings, specific start and end times for all three periods were determined.

Table 3.1: Specific time periods used for traffic volume evaluation

Day period	Hour period	Day	Date
Morning peak hour	7:15 am – 8.15 am	Tuesday	4th October 2011
Noon saddle hour	11:45 am – 12:45 pm	Wednesday	12th October 2011
Afternoon peak hour	4:45 pm – 5:45 pm	Thursday	6th October 2011

After determining the above periods, traffic volumes were then counted from the videos. These counts were done for three different vehicle/user categories – cars (including vans and small trucks for personal use), trucks and pedestrians. Specific movements at the intersection were also distinguished. Traffic counts were calculated with the help of MicroTallydevices(VehicleCounts, 2012).The results for all three different day times are listed in a following three tables.

Table 3.2: Morning peak hour counts

Date Day	Time	Type	SB L	SB T	SB R	NB L	NB T	NB R	WB L	WB T	WB R	EB L	EB T	EB R	Total
4.10.2011 Tu	7:00 AM	Cars	7	167	2	0	56	15	107	3	16	5	0	9	387
4.10.2011 Tu	7:15 AM	Cars	15	205	4	4	91	19	117	1	21	7	1	4	489
4.10.2011 Tu	7:30 AM	Cars	15	277	4	7	122	24	108	1	41	12	4	6	621
4.10.2011 Tu	7:45 AM	Cars	19	274	3	5	100	19	118	3	21	2	0	11	575
4.10.2011 Tu	8:00 AM	Cars	25	244	6	6	52	18	117	3	15	8	1	7	502
4.10.2011 Tu	8:15 AM	Cars	17	197	5	4	104	31	108	1	11	2	4	10	494
4.10.2011 Tu	7:00 AM	Pedestrians		2			0			0			0		2
4.10.2011 Tu	7:15 AM	Pedestrians		1			3			0			0		4
4.10.2011 Tu	7:30 AM	Pedestrians		1			1			0			0		2
4.10.2011 Tu	7:45 AM	Pedestrians		2			1			0			2		5
4.10.2011 Tu	8:00 AM	Pedestrians		6			0			0			1		7
4.10.2011 Tu	8:15 AM	Pedestrians		0			0			7			0		7
4.10.2011 Tu	7:00 AM	Trucks	1	0	0	0	4	1	0	1	0	0	0	0	7
4.10.2011 Tu	7:15 AM	Trucks	0	2	1	1	4	0	1	0	0	0	1	0	10
4.10.2011 Tu	7:30 AM	Trucks	0	1	0	0	0	1	0	0	2	0	0	0	4
4.10.2011 Tu	7:45 AM	Trucks	1	0	0	0	1	3	1	0	1	0	0	0	7
4.10.2011 Tu	8:00 AM	Trucks	2	1	0	0	1	0	2	0	0	0	0	0	6
4.10.2011 Tu	8:15 AM	Trucks	0	2	1	0	1	1	1	1	0	0	0	0	7

Vehicle counts were made for longer period than just one hour. In the table, there are vehicle counts for a time period from 7:00 am until 8:30 am. Maximum hourly traffic volume was selected and therefore, uncolored rows were excluded from the one hour period needed for the simulation model. The same is done for other two day time periods.

Table 3.3: Noon off-peak hour counts

Date Day	Time	Type	SB L	SB T	SB R	NB L	NB T	NB R	WB L	WB T	WB R	EB L	EB T	EB R	Total
12.10.2011 We	11:45 AM	Cars	6	129	4	6	81	29	23	2	7	6	0	6	299
12.10.2011 We	12:00 PM	Cars	10	149	9	6	93	23	27	3	10	3	1	8	342
12.10.2011 We	12:15 PM	Cars	9	129	3	2	73	25	24	1	9	3	3	4	285
12.10.2011 We	12:30 PM	Cars	17	159	1	6	71	15	37	0	9	4	3	9	331
12.10.2011 We	12:45 PM	Cars	13	136	3	12	127	28	46	2	12	6	0	10	395
12.10.2011 We	11:45 AM	Pedestrians		3			0			0			3		6
12.10.2011 We	12:00 PM	Pedestrians		0			0			3			1		4
12.10.2011 We	12:15 PM	Pedestrians		1			0			2			2		5
12.10.2011 We	12:30 PM	Pedestrians		0			0			5			1		6
12.10.2011 We	12:45 PM	Pedestrians		1			0			7			0		8
12.10.2011 We	11:45 AM	Trucks	0	0	0	0	3	2	1	0	2	1	0	0	9
12.10.2011 We	12:00 PM	Trucks	0	2	0	0	7	0	1	0	0	0	0	0	10
12.10.2011 We	12:15 PM	Trucks	0	3	0	0	2	0	0	0	0	0	0	0	5
12.10.2011 We	12:30 PM	Trucks	0	3	0	1	2	1	0	0	0	0	1	0	8
12.10.2011 We	12:45 PM	Trucks	0	1	0	0	3	0	0	1	0	0	0	0	5

Table 3.4: Afternoon peak hour counts

Date Day	Time	Type	SB L	SB T	SB R	NB L	NB T	NB R	WB L	WB T	WB R	EB L	EB T	EB R	Total
6.10.2011 Th	4:45 PM	Cars	11	212	8	21	137	31	41	5	12	5	2	18	503
6.10.2011 Th	5:00 PM	Cars	20	249	6	22	171	33	46	11	8	7	9	13	595
6.10.2011 Th	5:15 PM	Cars	24	223	3	20	138	39	49	10	12	6	1	17	542
6.10.2011 Th	5:30 PM	Cars	29	227	8	20	124	42	58	6	12	3	3	13	545
6.10.2011 Th	5:45 PM	Cars	16	192	11	28	103	25	46	8	13	5	3	10	460
6.10.2011 Th	4:45 PM	Pedestrians		1			0			2			1		4
6.10.2011 Th	5:00 PM	Pedestrians		0			0			0			0		0
6.10.2011 Th	5:15 PM	Pedestrians		0			0			0			0		0
6.10.2011 Th	5:30 PM	Pedestrians		2			2			1			0		5
6.10.2011 Th	5:45 PM	Pedestrians		1			1			2			0		4
6.10.2011 Th	4:45 PM	Trucks	0	1	0	0	1	0	0	0	0	0	0	1	3
6.10.2011 Th	5:00 PM	Trucks	0	0	0	0	2	1	0	0	0	0	0	0	3
6.10.2011 Th	5:15 PM	Trucks	0	0	0	0	0	0	0	0	0	0	0	0	0
6.10.2011 Th	5:30 PM	Trucks	0	1	0	0	1	0	1	0	0	0	0	0	3
6.10.2011 Th	5:45 PM	Trucks	0	1	0	0	1	1	0	0	0	0	0	0	3

These counts were used as an input to the VISSIM model.

3.2.2 Site Geometry

For the purpose of the Czech fixed time control plan method, site geometry had to be captured. The most important elements were: lane widths, turning bay lengths, median widths, radii between the approaches of the intersection, pedestrian crosswalk lengths, widths and slope on the approaches of the intersection.

These measurements were done with the help of geodetic apparatus. A drawing in AutoCAD software was made and it was used to calculate split intervals in the Czech fixed time control plan. The drawing is shown on the figure below.

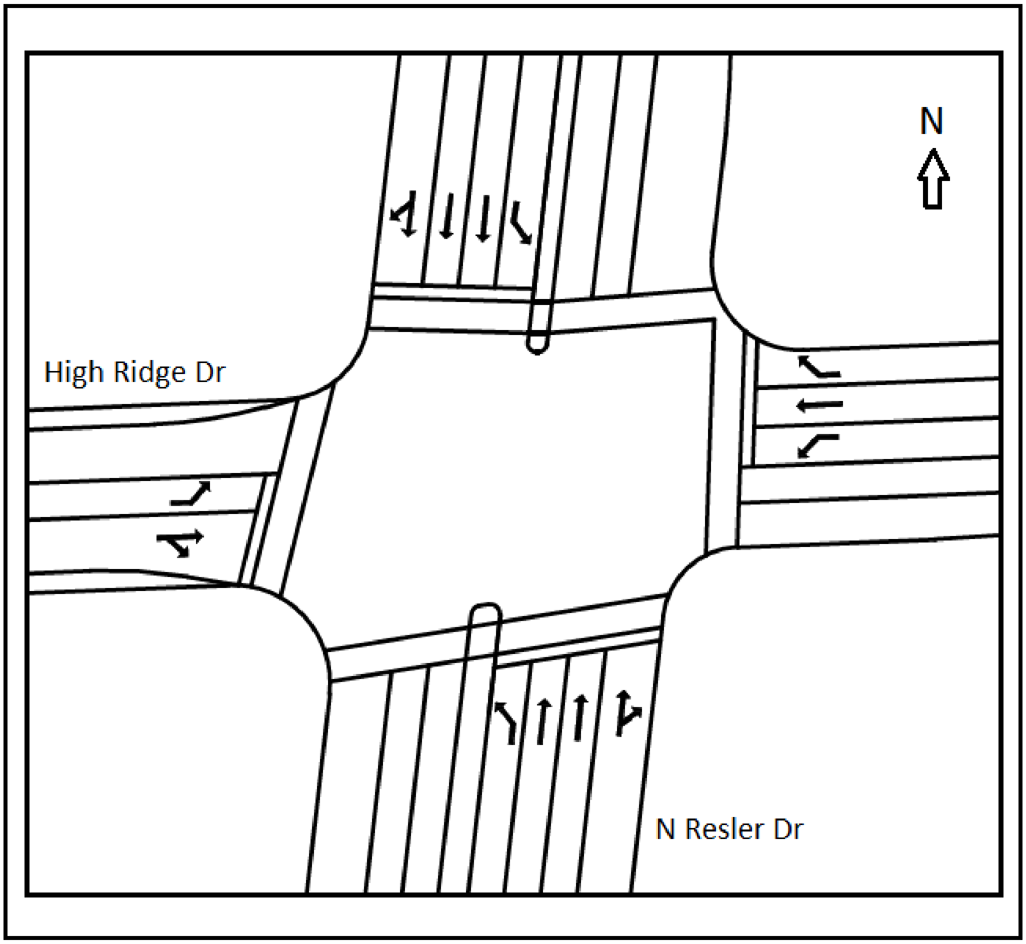


Figure 3.2: AutoCAD drawing of the selected intersection

3.2.3 Existing Signal Timing Plan

Mr. Tony Do, Transportation Engineer from the City of El Paso Transportation Department, provided existing traffic signal timing sheet for the intersection of High Ridge and N Resler Dr. This timing sheet contains information about existing phases, movements in each of the phases, minimum and maximum green times, passage times, vehicle clearance times, all red times, pedestrian clearance times and others. The timing sheet is attached in theAppendix. From the data in the timing sheet, the cycle length, phase sequence, yellow time and splits were deduced.

3.2.4 Bus Routes

Two different bus routes pass the intersection – routes 14 (red arrows) and 19 (blue arrows). Routes of these buses are shown in the illustration below in Figure 3.3.

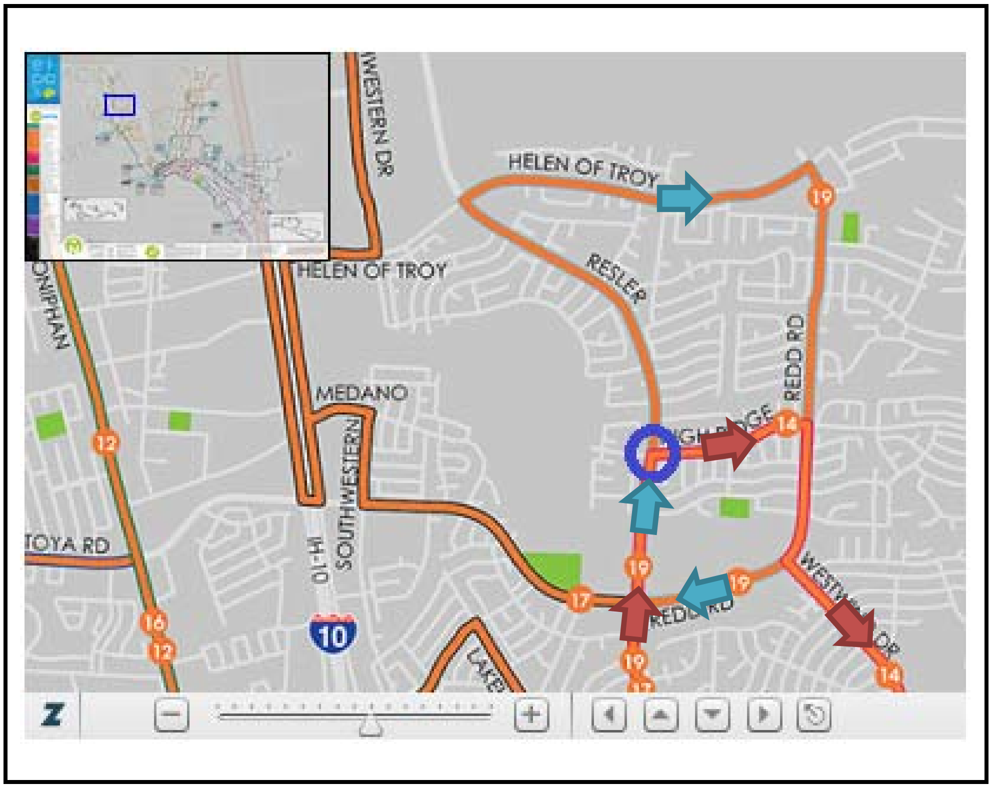


Figure 3.3: Bus routes (source: www.elpasotexas.gov/sunmetro)

Route 14 passes the intersection from the south to the east only and the average headway in a working days is 35 minutes. Route 19 passes the intersection from the south to the north only and the average headway in a working day is 45 minutes.

3.2.5 Detectors for the Actuated Control

For the purpose of actuated traffic signal control evaluation, detectors at the intersection were measured in terms of its width, length and position. These attributes were later coded into the VISSIM simulation model.

Chapter 4: Model Development in VISSIM

The VISSIM software (Version 5.20) was used to develop the microscopic simulation model for the intersection of High Ridge and N Resler Dr. Each step of the model development is described in this chapter.

4.1 Network and Infrastructure

The first step in the VISSIM software is to create a roadway infrastructure according to the purpose of the model. To create the model as realistic as possible, the geometry of the intersection was measured and with the help of the image from the Google Earth, the intersection was drawn into the VISSIM software. Links with its parameters, connectors and pavement markers were created according to measured data at the intersection of High Ridge and N Resler Dr. Links representing real roads have a length over 200 meters. The following figure shows a user interface in the VISSIM where roadways are created with the possibility to specify number of lanes, length, width, slope and other attributes.

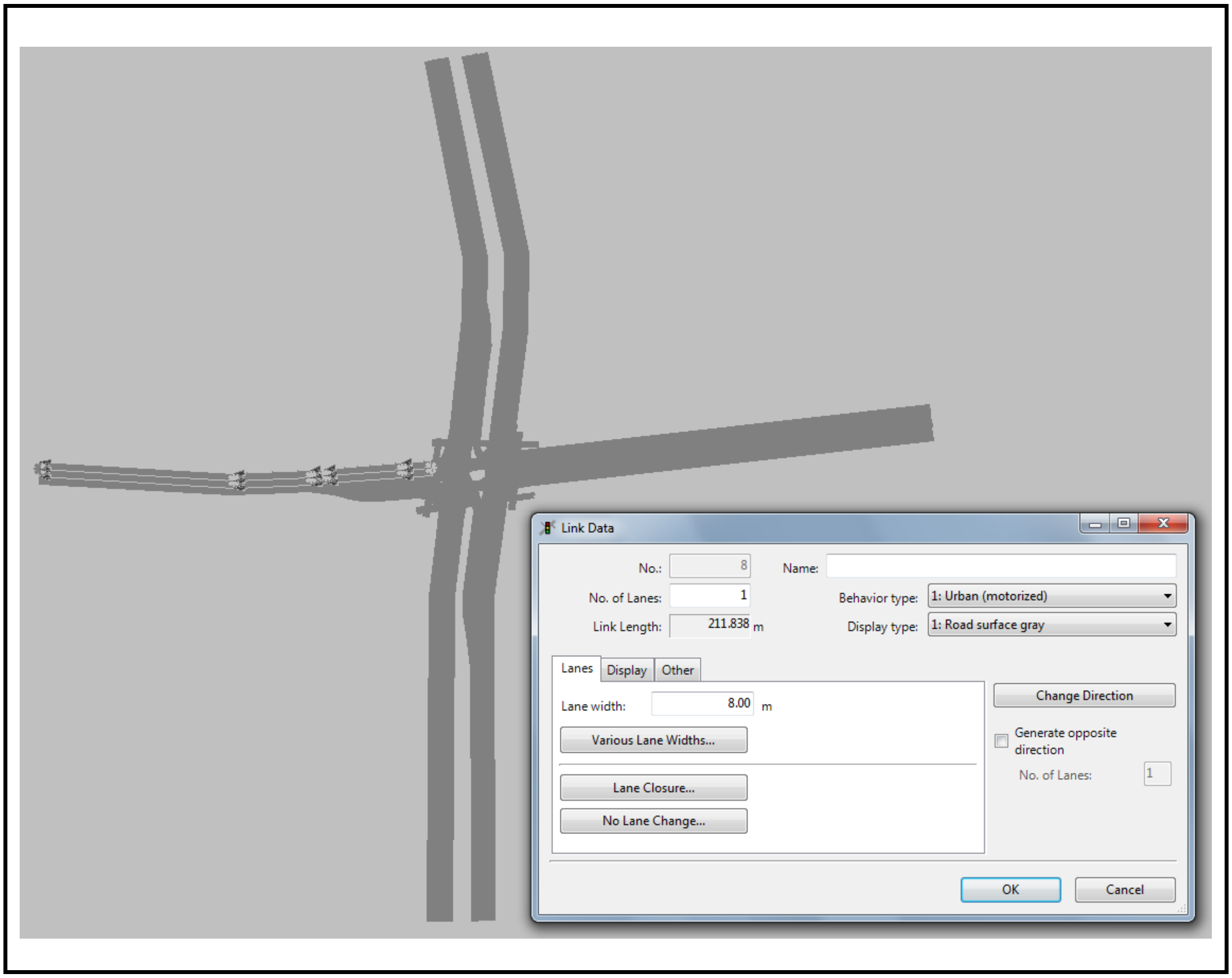


Figure 4.1: Network in VISSIM

4.2 Speed Reduction and Conflict Areas

The VISSIM software allows a user to specify desired speed decisions, speed reduction areas and conflict areas. From the known speed limits on the approaches of the intersection, desired speed decisions were inserted into the model. On High Ridge, the speed limit is 30 mph and the value of 50 km/h was inserted. To enable vehicles to have random desired speeds, VISSIM internally uses the speed range from 48.0 km/h to 58.0 km/h for the average value 50 km/h. On N ReslerDr, the speed limit is 35 mph and the value of 60 km/h was inserted with a default range from 58.0 km/h to 68.0 km/h. Figure 4.2

shows how a user can insert desired speed decision into the model so vehicles will keep certain speed for some specific road section.

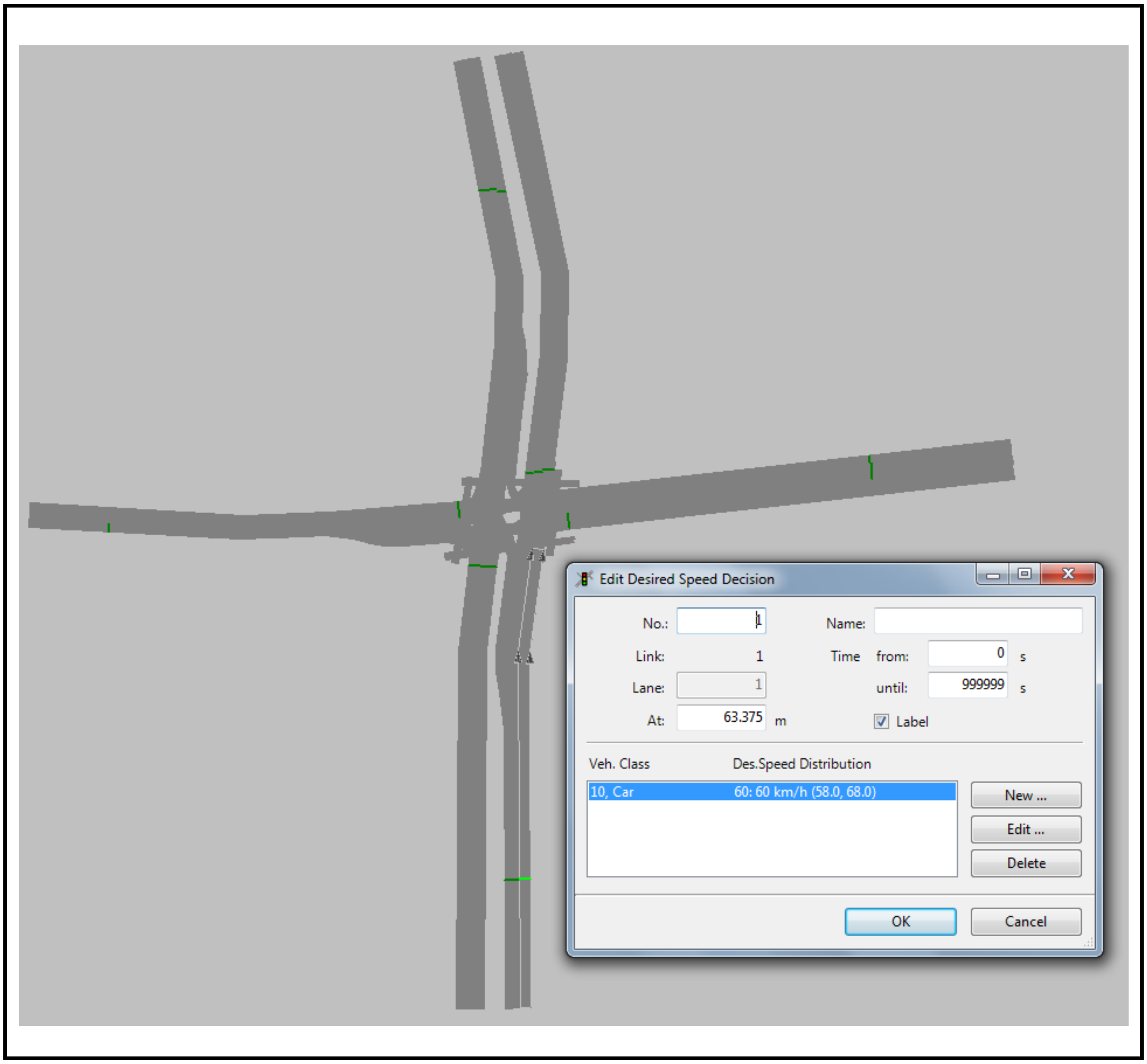


Figure 4.2: Desired Speed Decisions in VISSIM

Other than the desired speed decisions, speed reduction areas can be created as well. It is obvious that while turning at the intersection, vehicles have to slow down. On those areas, speed was reduced to 16 mph (25 km/h). Figure 4.3 shows the speed reduction areas in the model.

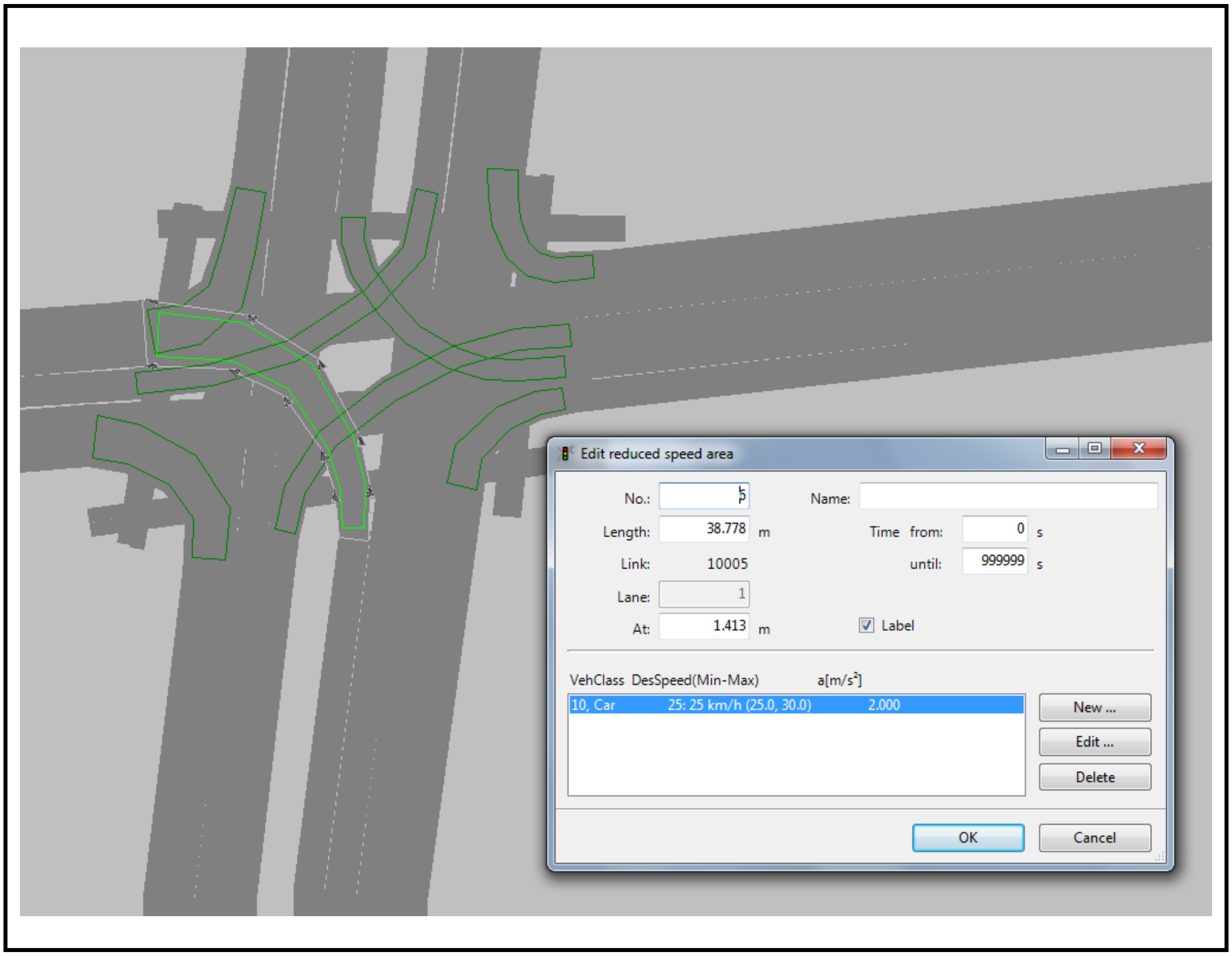


Figure 4.3: Reduced Speed Areas in VISSIM

Another step is to create conflict areas in the intersection. In cases when there is a permissive movement or in case of pedestrian involvment, conflict areas are created to specify logically which movement has a right of way. Figure 4.4. shows the conflict areas created in the model.

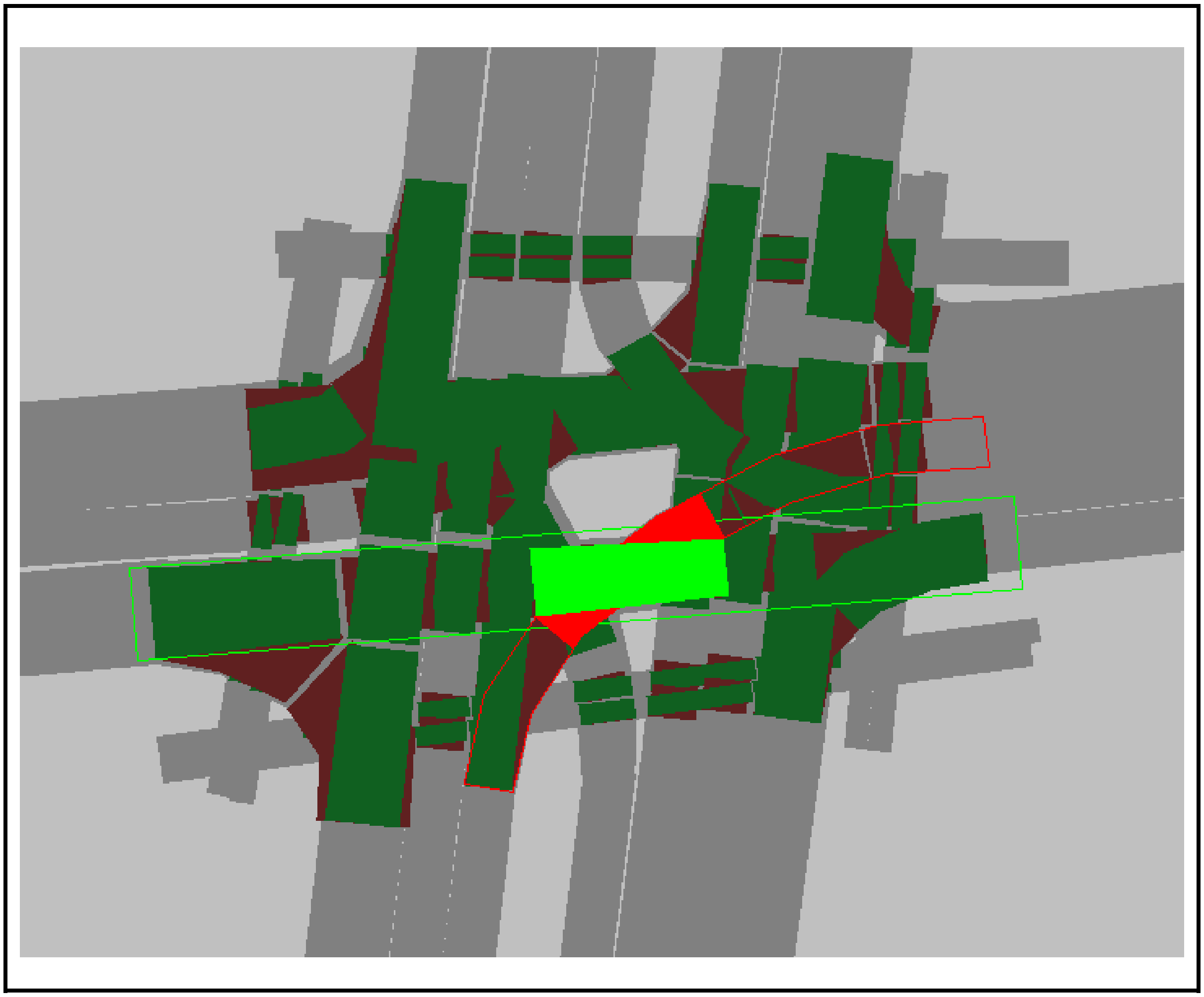


Figure 4.4: Conflict Areas in VISSIM

4.3 Vehicle Inputs

Traffic volume has to be inserted into the model. From known traffic volumes, a user can input these values at the upstream points of the approaches into the model. It is also possible to insert not only the average hourly traffic volumes but volumes at user specified intervals within the hour. For this model, four 15-minute intervals were chosen. Figure 4.5 shows the coding of eastbound-left volume at 15-minute intervals.

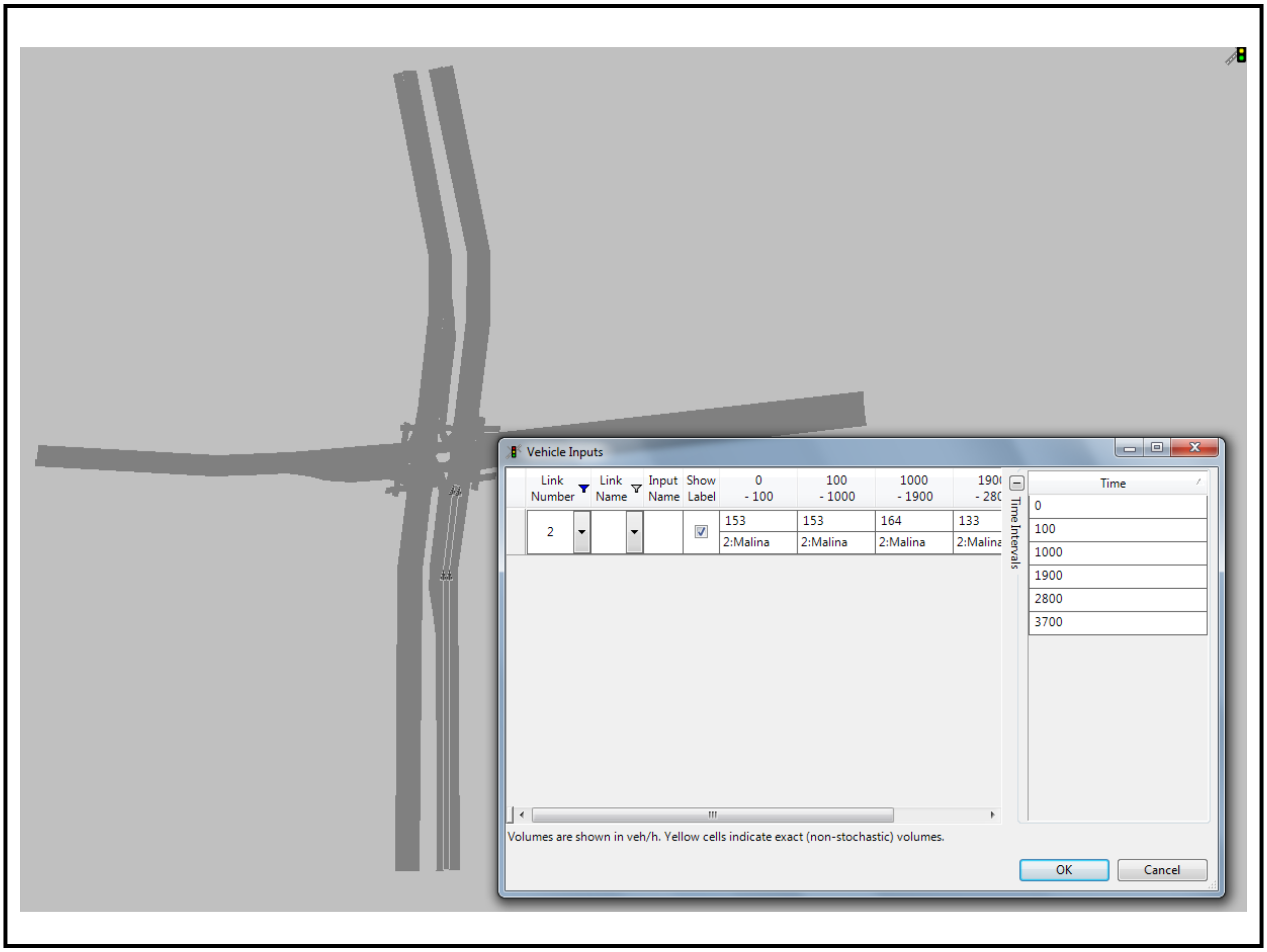
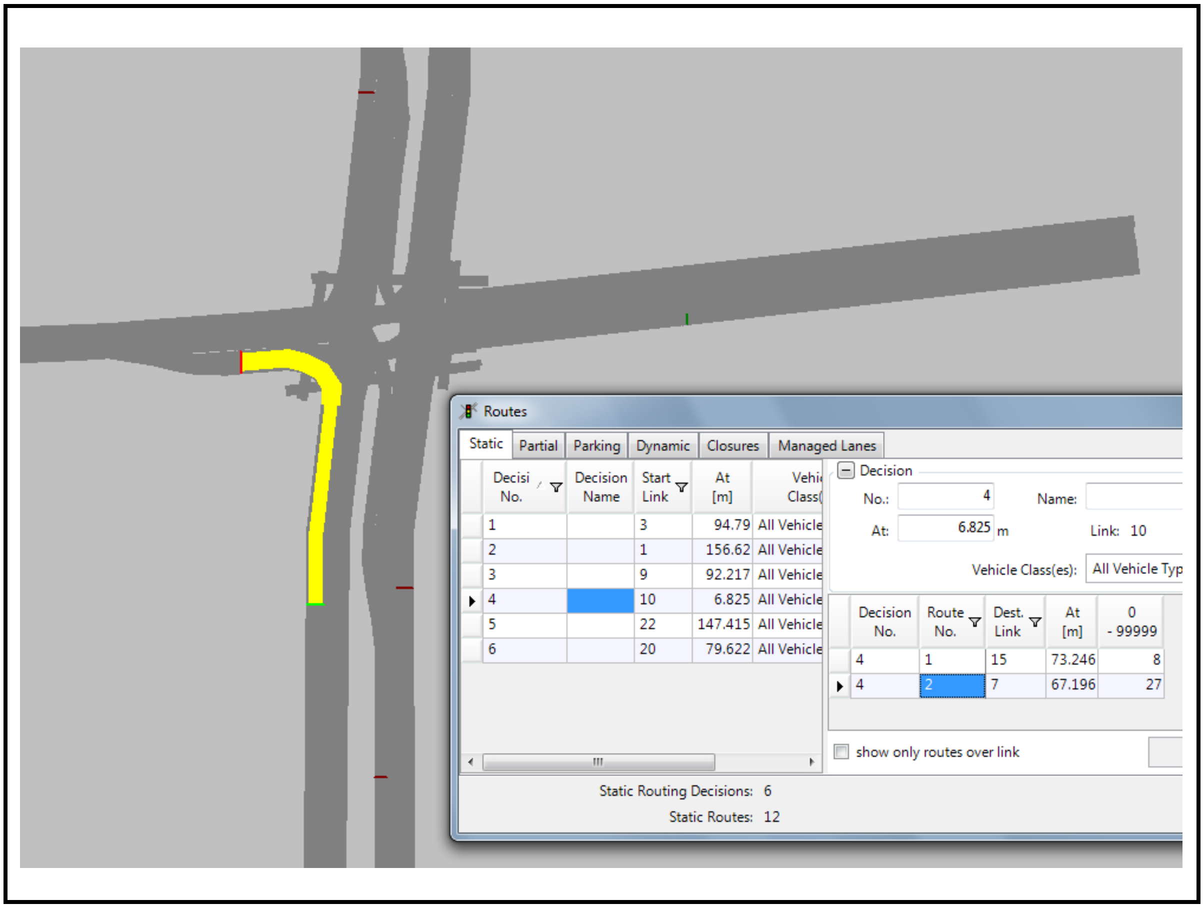


Figure 4.5: Vehicle inputs in VISSIM

VISSIM uses the Routes function to specify the lane use and turning decisions of vehicles. Users can specify what amount of vehicles turn left, right or continue through from approaches of one direction (e.g. eastbound). This is how all the traffic volumes in one approach are assigned to their respective lanes and movements. Figure 4.6 shows the specification for the route for eastbound-right, such that vehicles turning right from the eastbound approach can only turn from the rightmost lane into the rightmost lane.



Decisi No.	Decision Name	Start Link	At [m]	Vehi Class(
1		3	94.79	All Vehicle
2		1	156.62	All Vehicle
3		9	92.217	All Vehicle
4		10	6.825	All Vehicle
5		22	147.415	All Vehicle
6		20	79.622	All Vehicle

Decision No.	Route No.	Dest. Link	At [m]	0 - 99999
4	1	15	73.246	8
4	2	7	67.196	27

Figure 4.6: Routes in VISSIM

4.4 Traffic Signals

The first step of the traffic signals implementation in VISSIM is to insert signal heads on all necessary approaches and lanes. Then, signal timing plan can be set up. VISSIM offers many different ways to enter the timing plan data. Basically, fixed or actuated controls are the options. In the case of the actuated traffic control, methods such as Ring Barrier Control, NEMA, VAP or SIEMENS VA can be chosen. These could however require some kind of special applications such as Vehicle Actuated Programming (VAP). Therefore, the actuated control for the U.S. method was simulated with the help of the NEMA option. For fixed time control, users can create signal groups and signal programs (plans)

according to signal plan calculations. Figure 4.7 shows user interface for creating signal groups and signal programs where green, yellow and all red times are inserted.

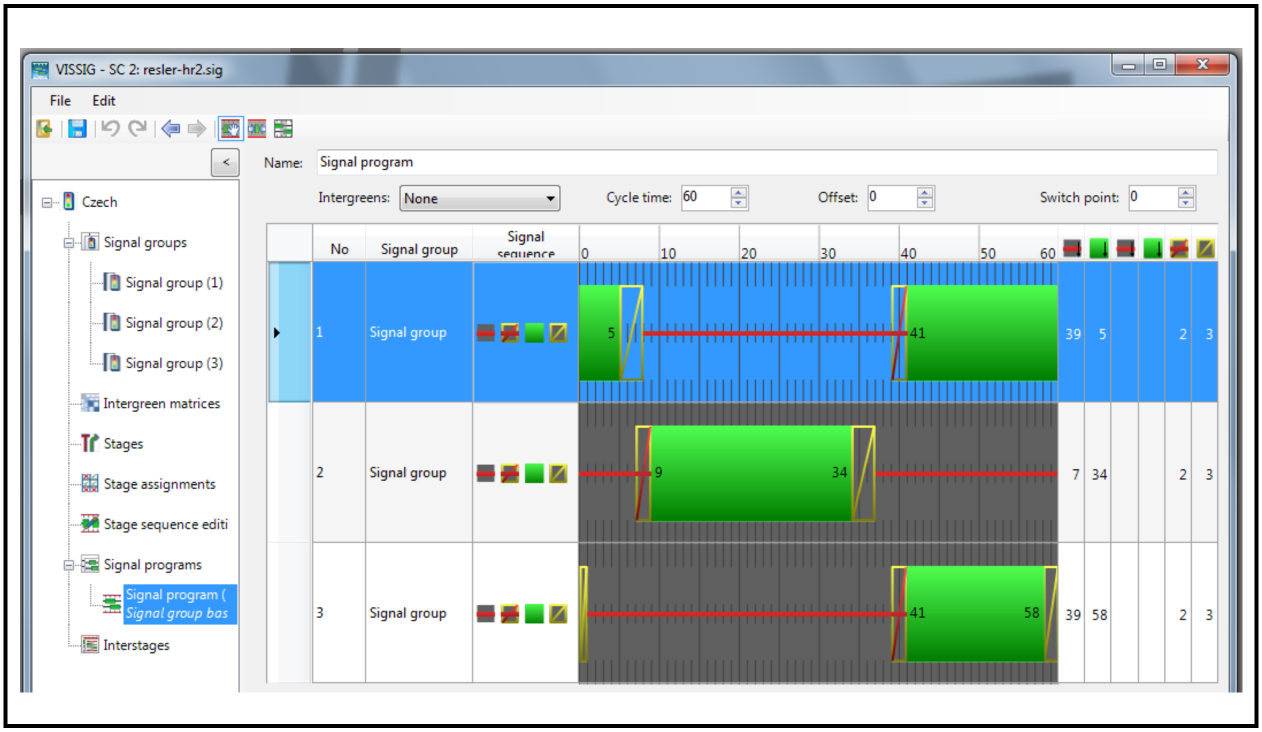


Figure 4.7: Signal groups and Signal programs for fixed control in VISSIM

Actuated signal control using the NEMA convention is created with the help of an add-on module called Ring Barrier Controller (RBC). This controller allows a user to insert input values such as minimum green time, vehicle extension, maximum green time, vehicle clearance or all red time. The RBC also allows users to specify recall switches used at the intersection. The VISSIM Reference Manual(PTV, Ring Barrier Controller User Manual)explains all the necessary steps to create functioning actuated signal control using RBC add-on module. Figure 4.8 shows RBC user interface.

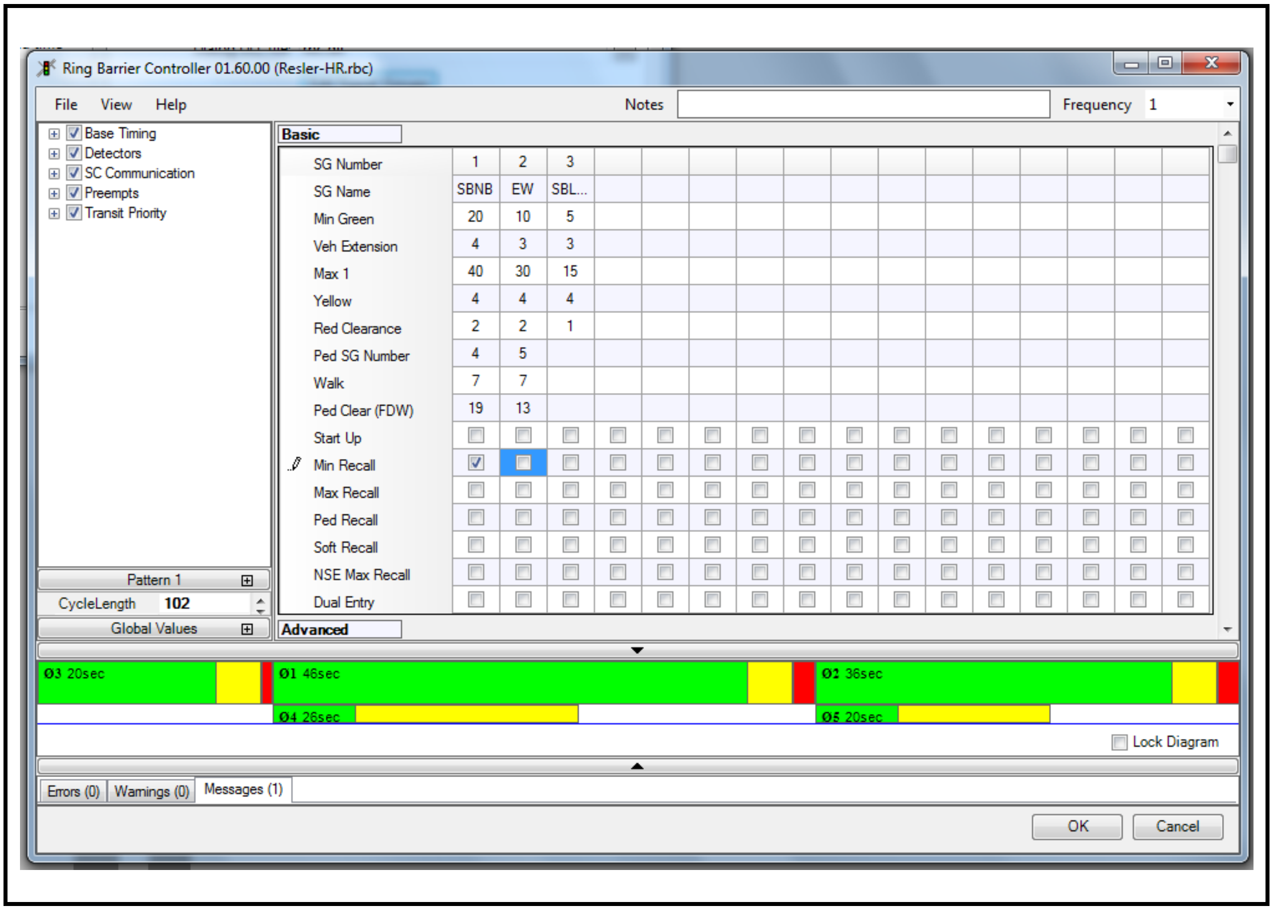


Figure 4.8: RBC user interface in VISSIM

4.5 Public Transportation

The first step of specifying bus routes in VISSIM is to create Public Transport (PT) stops. There are two types of PT stops: street and lay-by. In case of the street type, the PT stop is created on the existing mixed use travel lane so buses just stop in the lane to alight and board passengers. In case of lay-by PT stop, bus leaves the lane and stops in the bus bay so that other traffic is not obstructed by this boarding process. In this thesis, one bus stop belonging to the street type was created on N ReslerDr northbound approach of the intersection. Figure 4.9 shows the location of the bus stop.

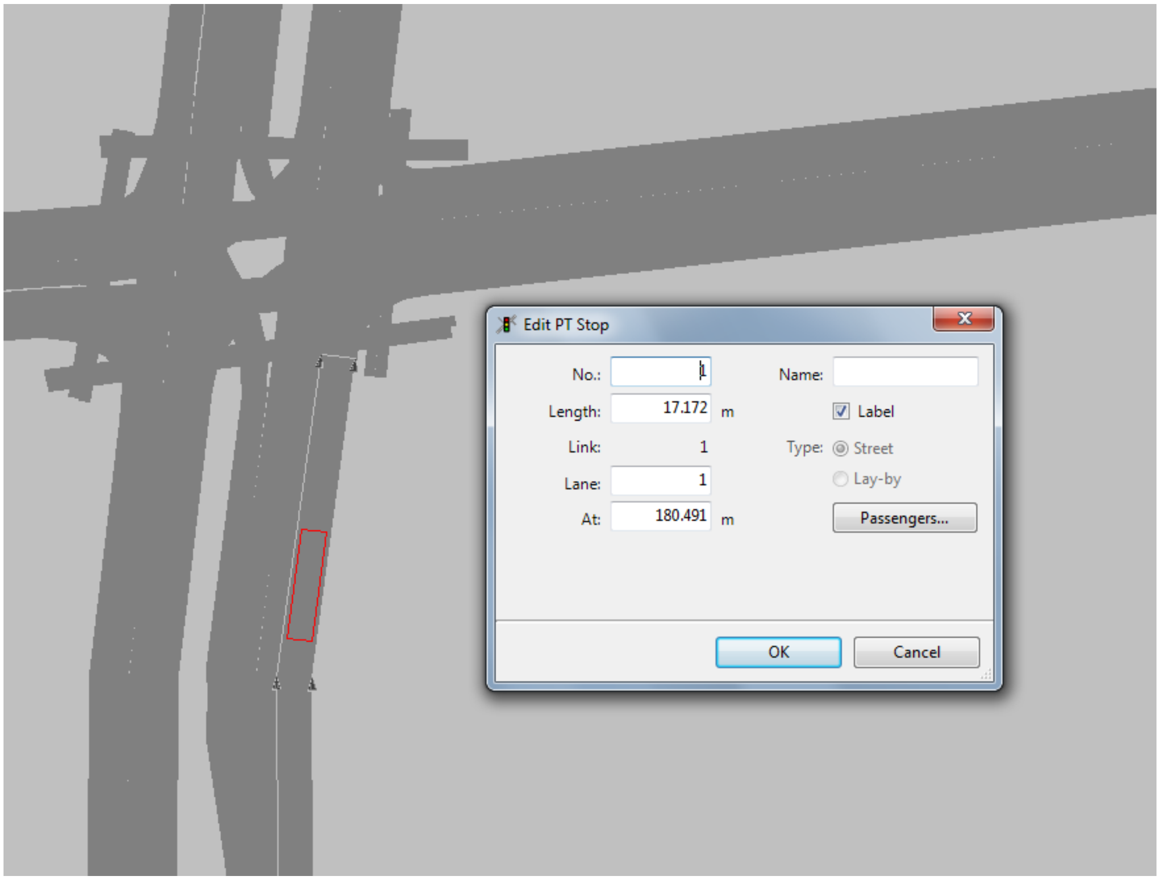


Figure 4.9: PT Stops in VISSIM

4.6 Detectors and Travel Time Sections

For the purpose of signal timing plan evaluation, detectors, data collection points and travel time sections must be created in the model. Detectors play the same role like at real intersections. Therefore, their usage and positions can be codedaccording to reality. There are three different types of detectors in the VISSIM and they differ in size and usage: pulse, presence and PT calling detectors. In this thesis, pulse detectors are used on all lanes from all approaches plus four presence detectors for all possible left turns. Also, for the purpose of TSP evaluation,one checking in and one checking out detector is used in the right lane on the northbound approach. The positions of detectors are coded according to the real locations mentioned in section 3.2.5. Figure 4.10 shows the location of the detectors in the model.

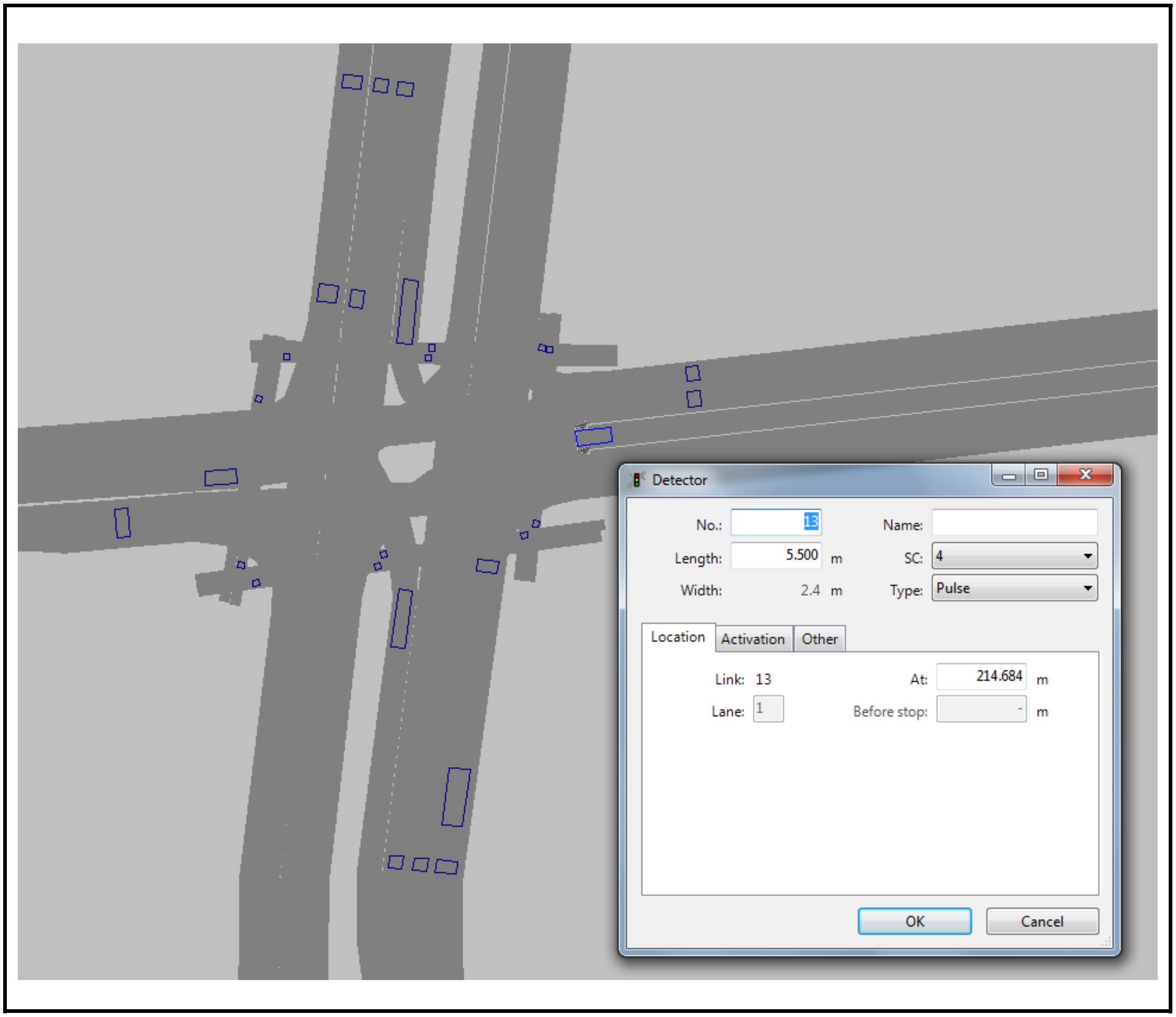


Figure 4.10: Detectors coded in VISSIM model

Travel time sections are another tool which helps with evaluation. These sections usually start at the beginning of routes in the model and terminate at the end of routes. They are particularly used for evaluation of delay. Figure 4.11 shows several travel time sections in the model. The beginning of asection is indicated by a red bar across the link while the end of the section is indicated by a green bar across the link.

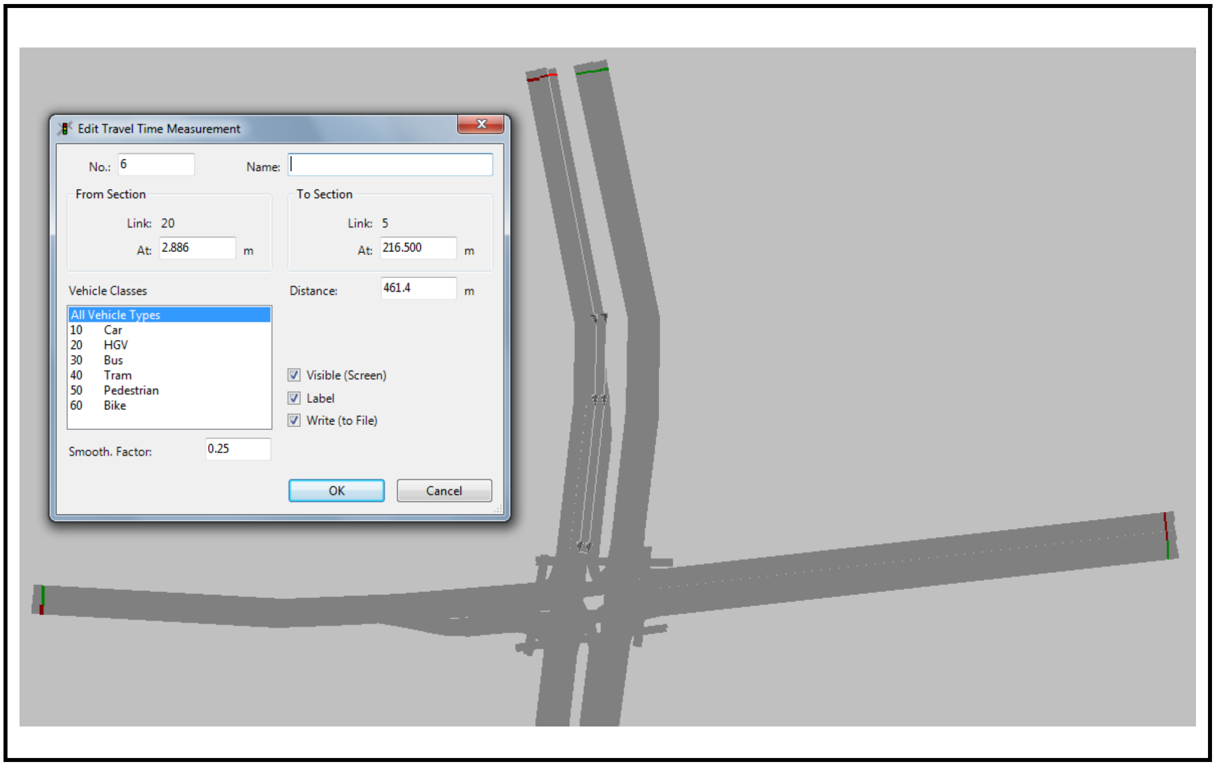


Figure 4.11: Travel Time Sections in VISSIM

Chapter 5: Traffic Signal Plan Designs

5.1. Chapter Introduction

This chapter shows the results of the different traffic signal plan design calculations starting with the U.S. traffic signal plan under fixed time control, followed by the U.S. traffic signal plan under actuated control and finally the Czech traffic signal plan under fixed time control.

5.2. U.S. Traffic Signal Plan under Fixed Control

The phases and phase sequence at the High Ridge and N ReslerDr intersection was determined according to reality. The following figure shows all the vehicle movements (in continuous lines) and pedestrian movements (in dashed lines) that are organized into three phases.

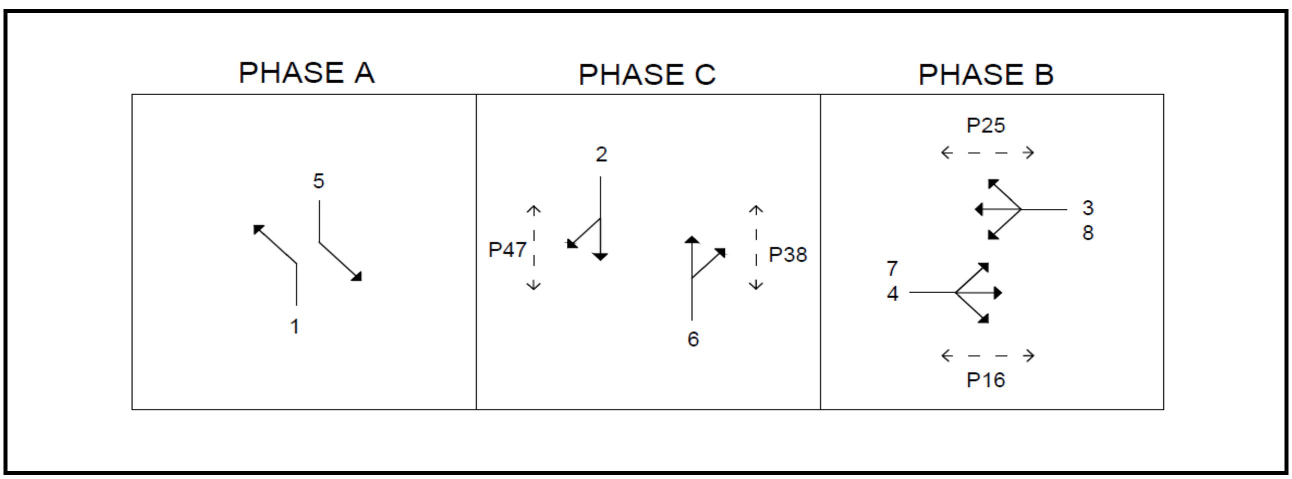


Figure 5.1: Phase sequence for the U.S. traffic signal plan

According to the steps described in section 2.1, the values of all calculated parameters are summarized in the following table for the morning, noon and afternoon day times, respectively.

Table 5.1: Results of signal plan calculations for U.S. fixed time control plan

Morning												
phase i	movement	v [veh/hr]	s [veh/hr/lane]	v/s ratio	(v/s)ci	Yc	Copt	Xc	gi [s]	gi(ped) [s]	gi(adj.) [s]	C [s]
A	SB L	77	1789	0.0430	0.0430	0.5783	54.5375	0.7414	3.17	-	5	65
A	NB L	23	1789	0.0129								
C	SB T/R	1022	5142	0.1988	0.1988				14.62	18	16	
C	NB T/R	455	5142	0.0885								
B	EB L	29	1411	0.0206	0.3365				24.75	24	27	
B	WB L	464	1379	0.3365								
B	EB T/R	35	1883	0.0186								
B	WB T	8	1883	0.0042								
B	WB R	101	1601	0.0631								

Noon												
phase i	movement	v [veh/hr]	s [veh/hr/lane]	v/s ratio	(v/s)ci	Yc	Copt	Xc	gi [s]	gi(ped) [s]	gi(adj.) [s]	C [s]
A	SB L	42	1789	0.0235	0.0235	0.2204	29.5006	0.3715	1.86	-	5	55
A	NB L	21	1789	0.0117								
C	SB T/R	591	5142	0.1149	0.1149				9.13	18	15	
C	NB T/R	427	5142	0.0830								
B	EB L	17	1413	0.0120	0.0819				6.51	24	18	
B	WB L	113	1379	0.0819								
B	EB T/R	35	1883	0.0186								
B	WB T	6	1883	0.0032								
B	WB R	37	1601	0.0231								

Afternoon												
phase i	movement	v [veh/hr]	s [veh/hr/lane]	v/s ratio	(v/s)ci	Yc	Copt	Xc	gi [s]	gi(ped) [s]	gi(adj.) [s]	C [s]
A	SB L	84	1789	0.0470	0.0470	0.3769	36.9108	0.5584	3.10	-	5	55
A	NB L	83	1789	0.0464								
C	SB T/R	938	5142	0.1824	0.1824				12.06	18	15	
C	NB T/R	720	5142	0.1400								
B	EB L	21	1377	0.0153	0.1475				9.75	24	18	
B	WB L	195	1322	0.1475								
B	EB T/R	77	1883	0.0409								
B	WB T	32	1883	0.0170								
B	WB R	44	1601	0.0275								

Comparing different values of calculated green times to adjusted green times for vehicles, it was clear that some green times had to be adjusted to satisfy the minimum green times for pedestrians. The final cycle length is logically adjusted as well in those cases.

From the adjusted timing plans in the previous table, signal plan charts were drawn. The first figure (Figure 5.2) is the timing plan designed for the morning peak hour.

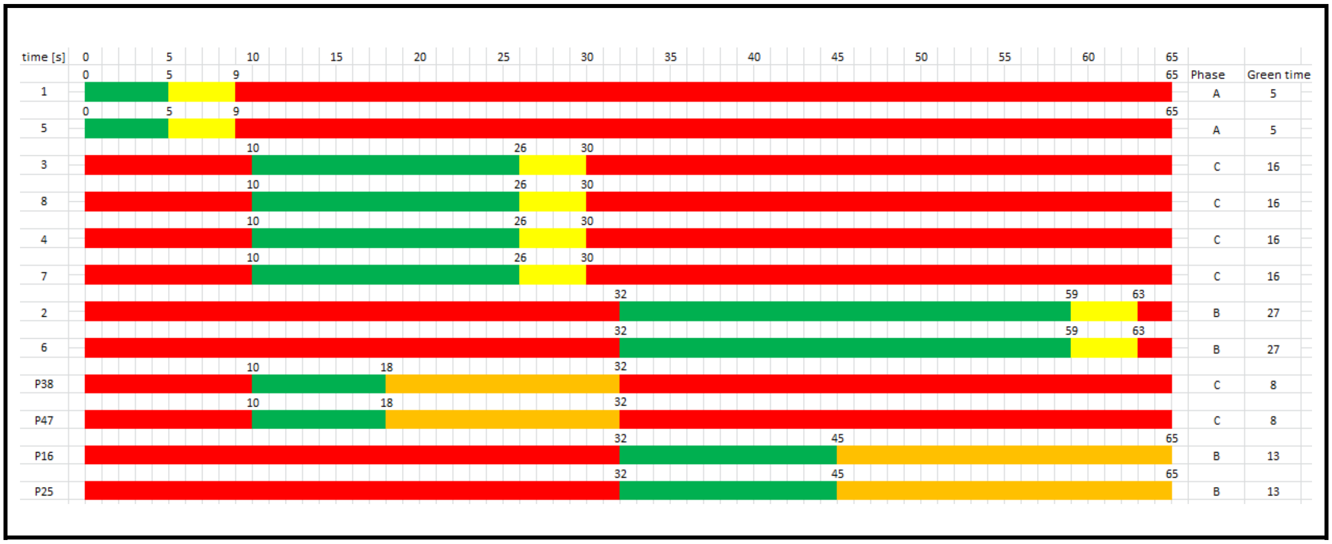


Figure 5.2: Morning signal plan for the U.S. fixed time control

Each row represents different vehicle movement coded according to the phase sequence in Figure 5.1including pedestrian movements in the last four rows. Yellow and all red times were taken from the Traffic signal timing sheet (see Appendix) provided by the City of El Paso and they are visualized in the figure above. The calculated minimum pedestrian green time was 18 seconds for phase C (the crosswalk is split into two sections with pedestrian holding area and push buttons in the median) and 24 seconds for phase A. These amounts of time consist of WALK (green color) time and DON'T WALK (orange) time. The calculated time for crossing the crosswalk was 14 seconds for phase C and 20 seconds for phase B. Therefore, the DON'T WALK time was considered as 14 seconds for phase C and 20 seconds for phase B with respect that there is no reason for delay caused by large number of pedestrians crossing in the same time at this intersection (third part of Equation 2.7 is very close to 0 second). The value of WALK time is the difference between minimum pedestrian time and DON'T WALK time. This assumption was used for other two design periods.

The second signal plan design was for the noon time.

Figure 5.3: Noon signal plan for the U.S. fixed time control

The third signal plan chart is added below (Figure 5.4) and this is forthe afternoon peak hour.

Figure 5.4: Afternoon signal plan for the U.S. fixed time control

All these signal plan designs were implemented to the VISSIM model for further evaluation.

5.3. U.S. Traffic Signal Plan under Actuated Control

The traffic signal timing sheet provided by the City of El Paso was the reference for coding the actuated control design in the simulation model. Values of all the necessary parameters were taken from this timing sheet. Section 2.1.3 describes these features in more detail. Therefore, there was no need for further calculation. The used values are summarized in the following table.

Table 5.2: Values of basic features used for the U.S. actuated timing plan

Phase	A	C	B
Street	Resler	Resler	High Ridge
Movements	SBLT/NBLT	NBTR/SBTR	E/W
Min Green	5	20	10
Passage	3	4	3
Max Green	15	40	30
Veh Clearance	4	4	4
All Red	1	2	2
Walk		7	7
Ped Clearance		19	13
Recall	Off	Min	Off
Detector	Presence		Pulse

The phase sequence for actuated control is the same as for the case of the U.S. fixed time control plan. That is, the phases are in the sequence of A – C – B. All values in Table 5.2 were implemented in the model using the RBC controller option.

5.4. Czech Traffic Signal Plan under Fixed Control

For the Czech method of fixed time control, the signal phases were taken from the real situation at the selected intersection (as in Figure 5.1).

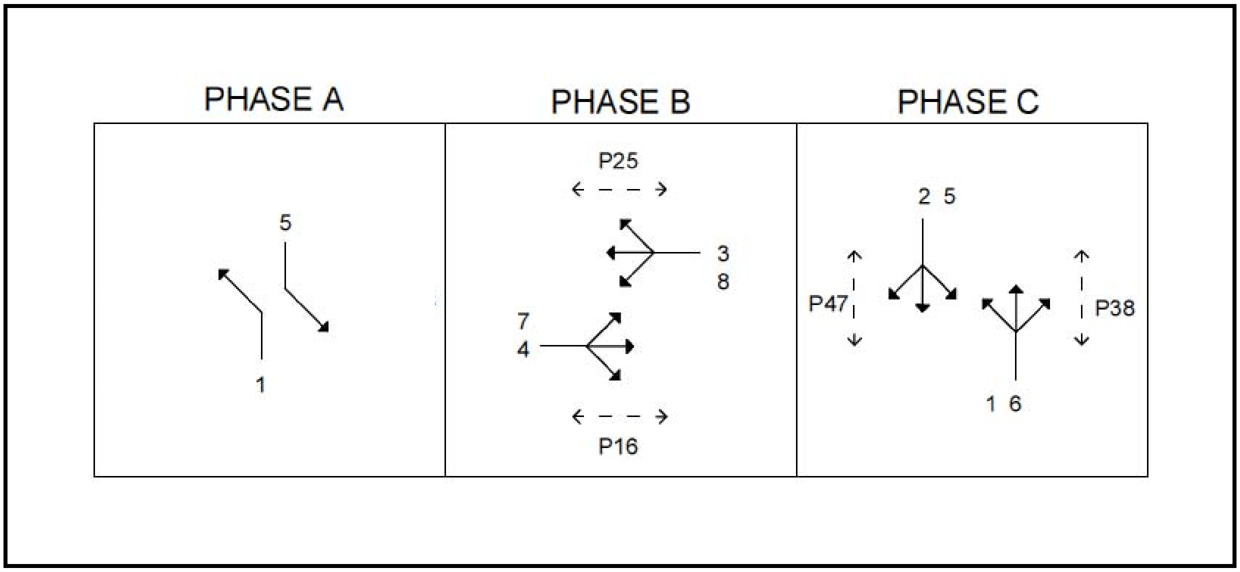


Figure 5.5: Phase sequence for the Czech traffic signal control design

As a very important part of the calculation procedures leading up to the final signal timing plan, the split interval table (Table 5.3) was created following all steps discussed in section 2.2.The intersection geometry measurements and AutoCAD drawing helped with the calculations of real travel distances at the intersection. In the table, all the split intervals are in seconds.

Table 5.3: Split intervals table used for the intersection

			entering											
	Signal group		phase A		phase C		phase B				phase C		phase B	
		movements	1	5	2	6	3	8	4	7	P38	P47	P16	P25
clearing	phase A	1		x	5	x	3	3	4	2	2	x	x	x
		5	x		x	5	1	x	3	4	x	3	x	x
	phase C	2	2	x		x	4	4	1	1	1	x	4	x
		6	x	2	x		1	x	5	4	x	3	x	2
	phase B	3	3	6	4	5		x	5	x	x	x	x	x
		8	4	x	3	6	x		x	1	x	5	3	x
		4	2	4	5	2	1	x		x	4	x	x	1
		7	5	3	7	4	x	5	x		x	x	x	x
	phase C	P38	7	x	11	x	10	15	9	x		x	x	x
		P47	x	10	x	16	x	12	17	12	x		x	x
	phase B	P16	13	x	8	13	6	x	9	x	x	x		x
		P25	x	12	13	10	x	12	x	8	x	x	x	

Note: x means no conflict point was found for that movement or the value was less than 1 and all values are in seconds.

For the selected intersection, there are only two possible phase sequences: A – C – B (used in reality) or A – B – C. For example, if phase A is clearing and the phase C is entering, the split intervals from the Table 5.3 are {5, x, x, 5} seconds. The maximum split interval value for this phase change is therefore 5 seconds. The maximum split intervals for other phase changes are done in the same way. The results of this phase sequence order are added below.

A – C – B order: A is clearing, C is entering: 5 seconds,

C is clearing, B is entering: 5 seconds,

B is clearing, A is entering: 6 seconds.

Total sum of this phase order is therefore 16 seconds.

A – B – C order: A is clearing, B is entering: 4 seconds,

B is clearing, C is entering: 7 seconds,

C is clearing, A is entering: 2 seconds.

A total sum of this phase order is therefore 13 seconds.

It is obvious that the A – B – C phase sequence saves 3 seconds of total split interval in comparison with phase sequence A – C – B. Therefore, the A – B – C phase sequence was chosen. This is different from the phase sequence implemented at the site.

After adopting the A-B-C phase sequence, all other calculations described in section 2.2 are summarized in Tables 5.4 to 5.6 for the morning, noon and afternoon time periods, respectively.

Table 5.4: Results of calculations forthe Czech fixed time control plan for the morning

Phase	Lane	I	S(bas)	S	y	y(max)	Y	C(opt)	C	L	g(opt)	g	g(min)	K	Reserve	I	I(real)
A	1	23	1825	1659.09	0.014									199.091	11.553	2.013	30
C	2	152	1825	1825.00	0.083									511.000	29.746	10.936	30
C	3	152	1825	1825.00	0.083									511.000	29.746	10.936	30
C	4	151	1850	1707.52	0.088									478.105	31.583	10.864	30
B	5	464	1850	1681.82	0.276									672.727	68.973	27.969	30
B	6	8	1850	1850.00	0.004	0.276					20.591	19	2.778	740.000	1.081	0.482	30
B	7	101	1825	1586.96	0.064		0.511	40.910	50	10				634.783	15.911	6.088	30
A	8	77	1850	1681.82	0.046	0.046					2.583	5	0.497	201.818	38.153	6.738	30
C	9	341	1850	1850.00	0.184									518.000	65.830	24.533	30
C	10	341	1800	1800.00	0.189	0.189					13.826	13	5.811	504.000	67.659	24.533	30
C	11	340	1850	1835.42	0.185									513.919	66.158	24.461	30
B	12	29	1825	1659.09	0.017									663.636	4.370	1.748	30
B	13	35	1850	1651.79	0.021									660.714	5.297	2.110	30

Table 5.5: Results of calculations for the Czech fixed time control plan for the noon

Phase	Lane	I	S(bas)	S	y	y(max)	Y	C(opt)	C	L	g(opt)	g	g(min)	K	Reserve	I	I(real)
A	1	21	1825	1659.09	0.013									199.091	10.548	1.838	30
C	2	142	1825	1825.00	0.078									766.500	18.526	8.283	30
C	3	143	1825	1825.00	0.078									766.500	18.656	8.342	30
C	4	142	1850	1699.22	0.084									713.674	19.897	8.283	30
B	5	113	1850	1681.82	0.067									437.273	25.842	8.349	30
B	6	6	1850	1850.00	0.003	0.067					12.331	12	0.234	481.000	1.247	0.443	30
B	7	37	1825	1586.96	0.023		0.202	25.050	50	10				412.609	8.967	2.734	30
A	8	42	1850	1681.82	0.025	0.025					3.955	5	-0.057	201.818	20.811	3.675	30
C	9	197	1850	1850.00	0.106									777.000	25.354	11.492	30
C	10	197	1800	1800.00	0.109	0.109					20.714	20	3.702	756.000	26.058	11.492	30
C	11	197	1850	1824.99	0.108									766.495	25.701	11.492	30
B	12	17	1825	1659.09	0.010									431.364	3.941	1.256	30
B	13	35	1850	1651.79	0.021									429.464	8.150	2.586	30

Table 5.6: Results of calculations for the Czech fixed time control plan for the afternoon

Phase	Lane	I	S(bas)	S	y	y(max)	Y	C(opt)	C	L	g(opt)	g	g(min)	K	Reserve	I	I(real)
A	1	83	1825	1659.09	0.050									199.091	41.689	7.263	30
C	2	240	1825	1825.00	0.132									730.000	32.877	14.467	30
C	3	240	1825	1825.00	0.132									730.000	32.877	14.467	30
C	4	240	1850	1757.72	0.137									703.088	34.135	14.467	30
B	5	195	1850	1681.82	0.116									470.909	41.409	14.029	30
B	6	32	1850	1850.00	0.017	0.116					12.650	13	1.162	518.000	6.178	2.302	30
B	7	44	1825	1586.96	0.028		0.340	30.293	50	10				444.348	9.902	3.166	30
A	8	84	1850	1681.82	0.050	0.050					4.880	5	1.499	201.818	41.622	7.350	30
C	9	313	1850	1850.00	0.169									740.000	42.297	18.867	30
C	10	313	1800	1800.00	0.174	0.174					19.471	19	6.611	720.000	43.472	18.867	30
C	11	312	1850	1834.13	0.170									733.651	42.527	18.807	30
B	12	21	1825	1659.09	0.013									464.545	4.521	1.511	30
B	13	77	1850	1754.31	0.044									491.207	15.676	5.540	30

In the above three tables, there are cells highlighted by yellow color. These are the values for reserve capacity calculations less than 10 and in theory these lanes should be grouped with others. However, these are the existing lanes at the intersection and therefore no change was made.

EDIP (2008) specifies that the real cycle length$C$ should fall into the following range: $0.75 \times C_{opt} < C < 1.5 \times C_{opt}$ to ensure that the overall delay does not deviate significantly. It is clear from the tables that this condition is not satisfied in two cases out of three. For the morning period, this condition is satisfied (C_{opt} = 40.910, C = 50 = 1.22 x C_{opt}), but for the noon period (C_{opt} = 25.050, C = 50 = 2 x C_{opt}) and afternoon (C_{opt} = 30.293, C = 50 = 1.65 x C_{opt}) it is not. This deficiency appears because thegreen times had to be adjusted to meet the minimum green time for pedestrians. The final cycle length is logically adjusted as well in all cases.

From the previous tables, it is easy to draw signal plan charts as a result of the Czech design procedure. The first figure (Figure 5.6) is the timing plan designed for the morning peak hour.

Figure 5.6: Morning signal plan for the U.S. fixed time control

In the first column, there are all possible vehicle movements followed by pedestrian movements. All yellow-red times, green times, yellow times and calculated relevant split intervals have an influence on the final signal timing plan. In the Czech Republic, there is no DON'T WALK signal indication for pedestrians. The pedestrians are only presented with a green indication. Additionally, for vehicular movements 1 and 5, it is possible to extend 5 seconds of green time in phase A and add green time of these movements also in phase C. Therefore, a green time of 20 seconds is assigned to these movements. This is also done for the other two Czech signal plans.

The following figure shows the signal plan for the noon time period.

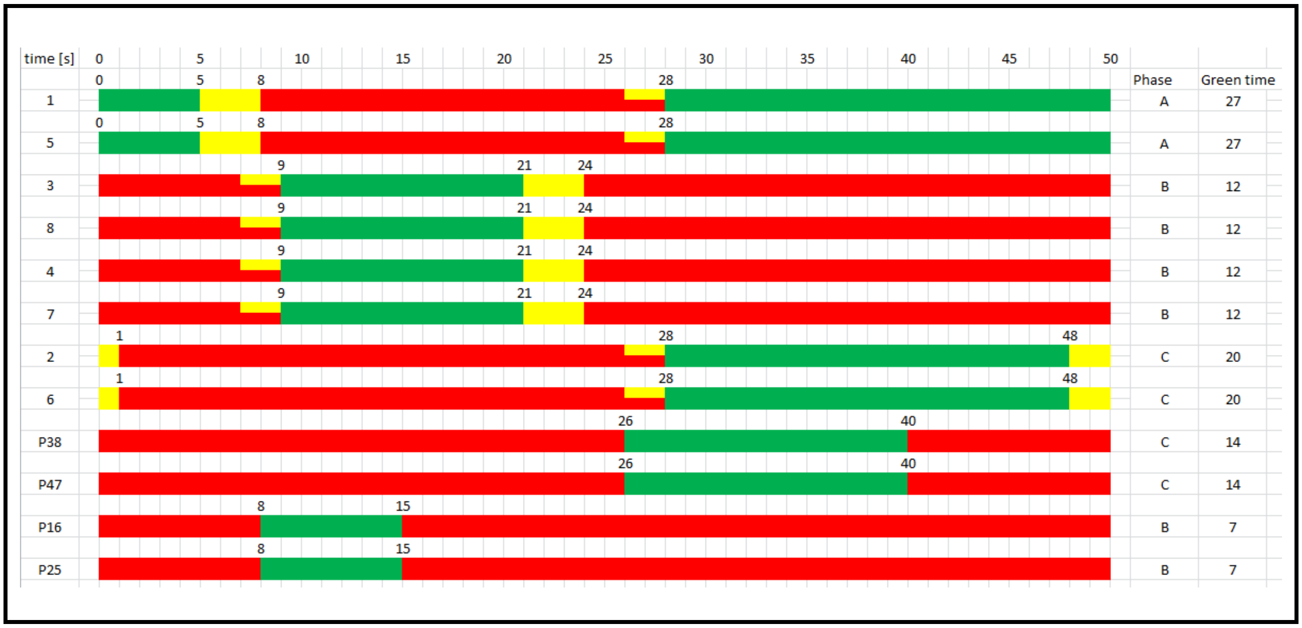


Figure 5.7: Noon signal plan for the U.S. fixed time control

The third signal plan chart is added below and concerns afternoon peak hour.

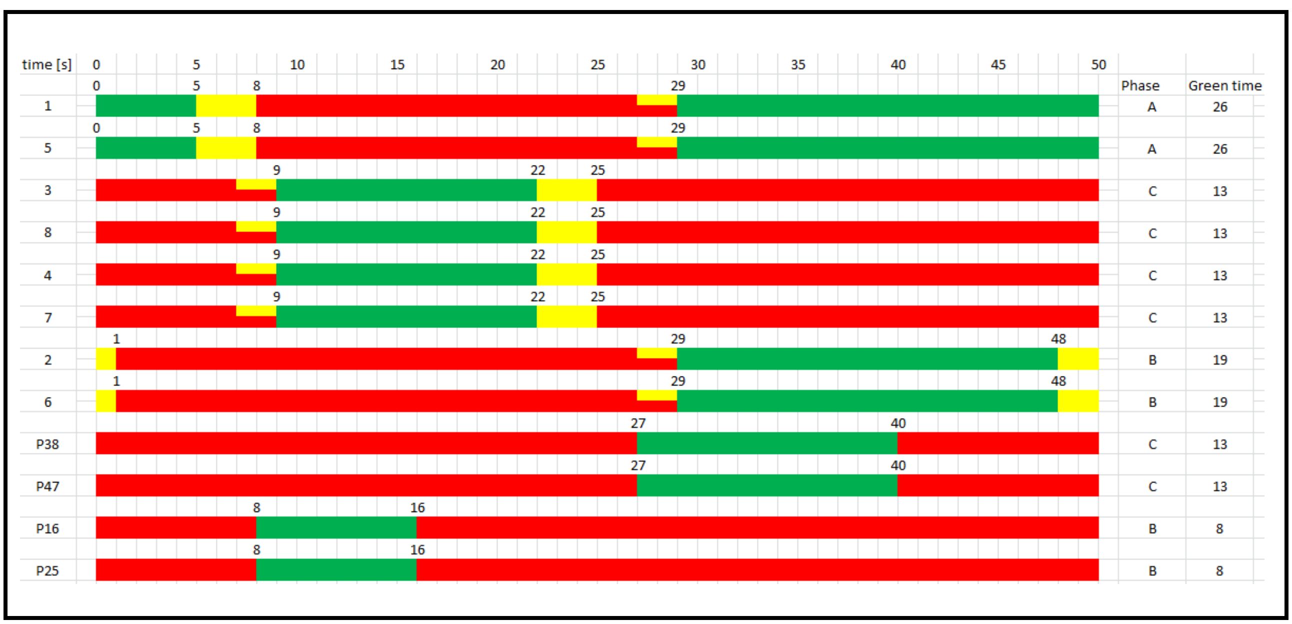


Figure 5.8: Afternoon signal plan for the U.S. fixed time control

All these signal plan designs were implemented to the VISSIM model for further evaluation.

5.5. Traffic Signal Control Designs Comparison

In this section, all the designs described in the previous sections are compared and the main differences discussed.The comparisons are made in three tables (Tables 5.7 to 5.9), one for each time period of a day.

Table 5.7 compares the designed timing plans for the morning peak period.

Table 5.7: Traffic signal plans comparison for the morning day time

Parameters/Designs	US actuated	US fixed	CZ fixed
Phase A green time	5 (15) s	5 s	5 s
Phase B green time	10 (30) s	27 s	19 s
Phase C green time	20 (40) s	16 s	13 s
Phase B ped green time	7 s	13 s	14 s
Phase C ped green time	7 s	8 s	7 s
Amber time	4 s	4 s	3 s
Amber-red time	-	-	2 s
Cycle length	52 (102) s	65 s	50 s

Due to thenature of actuated traffic signal control, all green times as well as cycle length vary depending on actual traffic flow. An interesting observation is that due to the heavy traffic for the eastbound left turn movement in the morning, green time for phase B is higher than that for phase C in both the U.S. and Czech fixed time control plans. The U.S. actuated timing plan is designed in the manner that phase C usually gets longer green time than phase B but this can also vary depending on vehicle "calls". The difference in amber time and amber-red time in the U.S. and Czech plan generally was discussed in Chapter 2 and it can affect the total delay at the intersection. Another obvious difference could be found in the last row of the table, for both fixed time designs. Here, the U.S. fixed time design gives a cycle time of 65 seconds while the Czech design method gives a cycle length of only 50 seconds.This could be affected by the different phase sequence.

The comparisons of the timing plans for the noon period are summarized in the following table.

Table 5.8: Traffic signal plans comparison for noon day time

Parameters/Designs	US actuated	US fixed	CZ fixed
Phase A green time	5 (15) s	5 s	5 s
Phase B green time	10 (30) s	18 s	12 s
Phase C green time	20 (40) s	15 s	20 s
Phase B ped green time	7 s	4 s	7 s
Phase C ped green time	7 s	7 s	14 s
Amber time	4 s	4 s	3 s
Amber-red time	-	-	2 s
Cycle length	52 (102) s	55 s	50 s

The U.S. actuated timing plan parameters remain the same for all three design or evaluation-periods in a day. Due to lower traffic at noon, cycle length in the U.S. fixed time control plan is expected

to decrease by 10 seconds to 55 seconds. There is a significant difference in green time allocation in fixed time plans. For the U.S. plan, more green time is assigned to phase B while for the Czech plan, there is more green time assigned to phase C.

The third comparison table is compiled for the afternoon peak period in Table 5.9.

Table 5.9: Traffic signal plans comparison for the afternoon day time

Parameters/Designs	US actuated	US fixed	CZ fixed
Phase A green time	5 (15) s	5 s	5 s
Phase B green time	10 (30) s	18 s	13 s
Phase C green time	20 (40) s	15 s	19 s
Phase B ped green time	7 s	4 s	8 s
Phase C ped green time	7 s	7 s	13 s
Amber time	4 s	4 s	3 s
Amber-red time	-	-	2 s
Cycle length	52 (102) s	55 s	50 s

The timing plans for thenoon off-peak hour and afternoon peak hour are very similar.

Chapter 6: Evaluation of Isolated Signal Timing Operations

6.1 Introduction

In this chapter, the three different traffic signal design methods, namely the U.S. fixed time traffic signal control plan, the actuatedtraffic signal timing plan and the Czech fixed time traffic signal control plan, as designed in Chapter 5, are evaluated for the intersection of High Ridge and N ReslerDr in El Paso. The comparative evaluation was conducted by means of microscopic traffic simulation using a model coded in the VISSIM software. The model development has been described in Chapter 4.

6.2 Traffic Signal Plans Evaluation

Theaverage delay(in seconds) at the intersection is evaluated for all three signal timing plans mentioned in Chapter 5. This evaluation is done for the morning peak hour, noon off-peak hour and afternoon peak hour. For this purpose, the model in VISSIM was run and, using the coded travel time sections, the delay of every vehicle during simulationwas measured. The delays of all the vehicleswere analyzed according to the movements to identify the movement that causes the biggest delay.TRB (2000) specifiesthe Level of Service (LOS) criteria for the signalized intersections and they are:

Table 6.1: Level-of-service criteria for signalized intersections

LOS	Average delay [s]
A	0 - 10
B	10 - 20
C	20 - 35
D	35 - 55
E	55 - 80
F	80 more

These colors are used in all the evaluation tables, from Tables 6.2 to 6.4.

Table 6.2: Average delay evaluation for the morning peak hour

Travel Time Section		5	3	2	1	4	7	6	8	9	10	11	12	13	16	15	14	Total # of vehicles	Average delay [s]
Movement		Left	Thru	Thru	Thru	Right	Left	Thru	Thru	Thru	Right	Left	Thru	Right	Left	Thru	Right		
Method\Approach		Northbound					Southbound					Westbound			Eastbound				
US_act	Av.delay [s]	24.33	20.67	17.64	19.40	21.26	30.02	18.91	22.53	21.56	19.15	32.99	13.06	18.42	14.93	0.00	13.78	2206	23.35
	# of vehicles	19	149	147	63	90	80	343	344	311	20	481	8	94	27	0	30		
	Av.delay [s]	19.81					21.61					30.36			14.32				
US_fix	Av.delay [s]	28.31	16.78	18.55	19.60	15.83	26.06	20.37	24.71	20.99	19.48	26.98	14.96	12.47	11.63	0.00	12.04	2208	21.72
	# of vehicles	20	146	148	63	91	79	341	343	312	20	487	8	93	27	0	30		
	Av.delay [s]	18.03					22.30					24.52			11.84				
CZ_fix	Av.delay [s]	9.62	14.90	13.46	15.63	17.96	14.24	17.87	20.33	18.12	17.54	15.85	10.13	8.73	8.93	0.00	8.56	2206	16.44
	# of vehicles	20	146	147	63	91	79	341	345	313	20	483	8	93	27	0	30		
	Av.delay [s]	14.92					18.45					14.64			8.74				

Table 6.2 shows that the best results are given by the Czech fixed time control plan (CZ fix). The average delay of this method is almost 5.3 seconds less than the U.S. fixed time control plan method (US_fix) which is the second best for the morning peak hour. The U.S. actuated timing plan (US_act) has an average delay of 1.6 secondslonger than the U.S. fixed time control plan. The reasons for this average delay order are:

- The decision on the phase sequence for the Czech method (which relies on split intervals) lowers the average delay significantly.

- Both fixed time plans were calculated using measured traffic volume and the same traffic volume was inserted into the model. Therefore, these plans should work better than actuated control.
- Both U.S. fixed time and actuated control plans have difficulties to serve left turning vehicles efficiently, especially for the westbound left turn movement where very high traffic volume is present in the morning.
- The U.S. actuated timing plan has additional difficulty to serve the eastbound and westbound traffic in comparison to the other two designs.The maximum green time for westbound approach is not enough if the traffic is very heavy in the morning peak hour there.

Table 6.3 evaluates all three timing plans for the noon time off-peak hour.

Table 6.3: Average delay evaluation for the noon-time day time

Travel Time Section		5	3	2	1	4	7	6	8	9	10	11	12	13	16	15	14	Total # of vehicles	Average delay [s]
Movement		Left	Thru	Thru	Thru	Right	Left	Thru	Thru	Thru	Right	Left	Thru	Right	Left	Thru	Right		
Method\Approach		Northbound					Southbound					Westbound			Eastbound				
US_act	Av.delay [s]	22.35	12.31	12.66	16.93	10.29	32.78	13.36	13.83	12.04	12.73	18.71	9.75	18.17	25.07	15.89	16.67	1253	14.65
	# of vehicles	15	134	143	59	80	44	192	197	170	24	111	2	28	19	7	28		
	Av.delay [s]	13.03					14.49					18.48			19.52				
US_fix	Av.delay [s]	23.77	16.63	17.41	16.01	16.12	25.18	18.32	16.11	18.35	17.78	13.03	0.00	14.38	13.03	13.87	12.54	1255	16.92
	# of vehicles	16	134	143	59	81	44	192	196	170	24	112	2	28	19	7	28		
	Av.delay [s]	16.97					18.10					13.12			12.89				
CZ_fix	Av.delay [s]	9.69	9.47	11.21	10.71	10.53	9.13	10.15	11.07	9.92	9.53	15.60	16.70	14.14	17.44	11.26	10.50	1259	11.01
	# of vehicles	16	134	144	59	81	44	194	198	171	24	110	2	28	19	7	28		
	Av.delay [s]	10.42					10.28					15.33			13.04				

Due to the lower traffic volume at noon, vehicle delays are significantly lower in comparison to the morning peak hour. The Czech fixed time control plan gives the average delay of 11.0 seconds which is 3.6 seconds less than the U.S. actuated timing plan. The worst is the U.S. fixed time control

plan which has an average delay of 2.3 seconds higher than the U.S. actuated timing plan. The reasons for these results are summarized here:

- The selection of phase sequence based on split intervals in the Czech signal control plan lowers the average delay significantly.
- With the lower traffic volume, the U.S. actuated timing plan serves the traffic more efficiently than the U.S. fixed time control plan even though the exact traffic volume used for the fixed time design calculation was used in the simulation.
- Both U.S. fixed time and actuated control plans have difficulties to serve left turning vehicles efficiently. This deficiency concerns mainly with the U.S. actuated timing plan for the southbound left-turn movements, which has a delay of 32.8 seconds.
- The U.S. fixed time control plan has additional difficulties to serve thenorthbound and southbound traffic which carry heavy volumes. On the other hand, the U.S. actuated timing plan does not serve theeastbound traffic well.

The table 6.4 evaluates the traffic operations during the afternoon peak hour.

Table 6.4: Average delay evaluation for the afternoon peak hour

Travel Time Section		5	3	2	1	4	7	6	8	9	10	11	12	13	16	15	14	Total # of vehicles	Average delay [s]
Movement		Thru	Thru	Thru	Right	Left	Thru	Left	Thru	Thru	Right	Left	Thru	Right	Right	Thru	Left		
Method\Approach		Northbound					Southbound					Westbound			Eastbound				
US_act	Av.delay [s]	30.35	17.29	18.03	14.37	15.21	30.87	18.43	16.97	17.19	15.93	26.42	28.04	21.53	18.69	17.58	19.26	2168	19.32
	# of vehicles	73	226	238	106	136	81	320	305	270	32	210	31	37	16	16	71		
	Av.delay [s]	17.98					18.58					25.95			18.91				
US_fix	Av.delay [s]	26.13	17.36	19.17	19.15	17.82	24.09	21.51	21.15	20.90	19.94	17.15	9.54	10.71	12.08	13.79	13.11	2167	19.39
	# of vehicles	73	227	238	104	136	81	321	305	268	32	210	31	38	17	15	71		
	Av.delay [s]	19.06					21.40					15.43			13.04				
CZ_fix	Av.delay [s]	16.18	12.34	12.10	14.49	11.89	14.73	13.52	12.04	13.70	12.43	21.61	8.60	13.94	16.48	12.45	15.65	2173	13.92
	# of vehicles	72	227	238	105	136	80	322	306	273	32	210	31	38	17	15	71		
	Av.delay [s]	12.83					13.18					19.12			15.32				

Again, the Czech fixed time traffic signal control plan gives the best results with the average delay of less than 14 seconds. The U.S. fixed time and the U.S. actuated timing plans have similar results. Both have average delays of5.4 seconds higher than the Czech signal control plan. The reasons for these results are basically the same as for the morning peak hour and the noon time off-peak period. In addition, it is observed that the U.S. actuated timing plan has difficulty to serve the westbound traffic compared to other two plans while the U.S. fixed time control plan does not serve the southbound traffic well.

6.5 Results and Discussions

All significant results from the previous section are summarized in the following table.

Table 6.5: Average delay comparison

Day time	Design	Total delay [s]	Difference [s]	Level-of-Service
Morning	U.S. actuated	23.35	+ 6.91	C
	U.S. fixed	21.72	+ 5.28	C
	CZ fixed	16.44	0.00	B
Noon	U.S. actuated	14.65	+ 3.64	B
	U.S. fixed	16.92	+ 5.91	B
	CZ fixed	11.01	0.00	B
Afternoon	U.S. actuated	19.32	+ 5.40	B
	U.S. fixed	19.39	+ 5.47	B
	CZ fixed	13.92	0.00	B

In all the three time periods of a weekday, the Czech fixed time traffic signal control plans give the lowest values of average delay at the intersection. In addition, the improvementsin comparison to

both U.S. plans are very significant. The LOS for the Czech design is B for all three different time periods of a day. For the noon time off-peak hour, theaverage delay was not so far from LOS A. Both the U.S. plans have almost the same LOS for all three different periods. In the morning peak hour, the LOS is C, in the noon time off-peak hour, the LOS is B and in the afternoon peak hour, the LOS is B but not very far from C.It is necessary to mention that the U.S. actuated timing plan is using the same parameter values of green time throughout the day, regardless of the traffic volume. Still, it is more efficient to use the U.S. actuated timing plan over the U.S. fixed time control plan. The main advantage of the Czech fixed time control plan lies in the determination of split intervals. The Czech method of fixed time signal timing plan design has a unique way of calculating split intervals that determine the "intergreen" times between phases and also the phase sequence.

6.6 Chapter Conclusions

The Czech fixed time traffic signal control plan was evaluated as the most efficient at the intersection of Resler and High Ridge in El Paso. This designsaves 5.28, 5.91 and 5.47 seconds compared to the U.S. fixed time control plan for the morning, noon and afternoon period respectively in terms of average delayand 6.91, 3.64 and 5.40 seconds compared to the U.S. actuated timing plan for the same periods of a day. The main reason for this difference isin the split interval calculation of the Czech fixed time design method which is a base for phase sequence selection.

Chapter 7: Evaluation of Transit Signal Priority

7.1 Chapter Introduction

This chapter focuses on TSP implementation and evaluation. The RBC add-on module in VISSIM allows the creation of TSP strategies into the model.The early green and extended green strategies discussed in section 2.3 were chosen to be evaluated for the test site. The average bus delay was compared for the scenario of actuated timing plan with TSP and for the scenario of actuated timing plan without TSP. Different bus time arrival times and bus headways were tested for the morning peak period. Besides the impact of TSP on the average delay of the buses, there is a need to evaluate the overall impact to other vehicles at the intersection. Therefore, the average delay of other vehicles with the use of TSP was also observed and compared to the average delay of other vehicles without the use of TSP for the morning, noon and afternoon period, respectively.

7.2 Design and Implementation of Transit Signal Priority

According to PTV(2010) and Zlatkovic (2009), TSP for the intersection of High Ridge and N ReslerDr in El Paso was implemented into the RBC logic in VISSIM. There are a few relevant features in the controller that need to be explained. They are summarized in the following table.

Table 7.1: Basic features with its values used for the TSP

Feature	Description	Value
Parent SGs	Signal groups linked with transit signal groups	1
Priority Mode	Defines a strategy of the transit signal group	Early/Extend
Travel Time	Estimated time for a bus to arrive at the intersection once it passes check-in detector	25 s
Travel Time Slack	Amount of time identifying the uncertainty of the actual travel time of the bus	10 s
Check in	Detector for check-in “call”	311
Check out	Detector for check-out “call”	321

“Parent SGs” are the signal groups that are assignedfor the transit priority. The value 1 was inserted for this feature indicating that the transit signal is linked to signal group 1 (which is part of phase C) in the model.

There are three modes available for the“Priority Mode” feature: None, Early/Extend and Extend only. Early/Extend option was chosen based on the discussion in section 2.3.

“Travel Time” and “Travel Time Slack” are connected to each other in the following manner. Once the transit vehicle passes the transit detector (i.e. check-in), it is expected to arrive at the intersection as soon as the "Travel Time" or as late as the "Travel Time" plus the "Travel Time Slack."To estimate the “Travel Time” and “Travel Time Slack” for input into the TSP logic, bus dwell times in the PT stop in VISSIM were observed. The average dwell time was set to 20 seconds within a range from the observed 18 to 24 seconds. However, within those 20 seconds, there is no delay caused by passengers making fare payments while boarding the bus. Therefore, this model works when there is no boarding passenger or there is no fare payment in the bus made by passengers. Furthermore, it takes 5 seconds for the bus to arrive from the PT stop to this intersection’s stop line. Therefore, “Travel Time”

was calculated as 20 seconds of dwell time plus 5 seconds of time needed to arrive at the intersection. The value of “Travel Time Slack” was chosen as 10 seconds depending on observed deviations of dwell time in PT stop to safely ensure that the bus will be served with TSP.

The detectors are designated as TSP’s “Check in” and “Check out” detectors by the detector numbers in the VISSIM model.

7.3 Results and Discussions

The first part of this section compares the two different scenarios in terms of average bus delay evaluation. The first scenario uses the actuated traffic signal timing plan without TSP, while the second scenario uses the actuated traffic signal timing plan with TSP. Different bus arrival times and bus headways were tested for the morning peak period.

There are two bus routes at the intersection as described in section 3.2.4. However, they both have average headwaysof 35 and 45 minutes in working days. It is obvious that these headways cannot be properly evaluated during one hour simulation due to the small number of bus samples. Therefore, smaller headways were tested for both lines to ensure that any difference in the results can be observed. In the first case, headways of both lines were each set to 300 seconds, creating a combined headway of 150 seconds. For the second case, headways of both lines were each extended to 600 seconds, creating a combined headway of 300 seconds. Different bus arrival times of first buses into the simulation clock (20, 40, 60, 80 and 100 seconds) were tested as well to ensure that buses approach the intersection at different durations in the signal cycle. The results of this evaluation are summarized in the table below.

Table 7.2: Average delay of bus with and without TSP

Combined bus headway = 150 seconds				Combined bus headway = 300 seconds			
Arrival Time	Scenario	Number of buses	Average delay [s]	Arrival Time	Scenario	Number of buses	Average delay [s]
100 s	No TSP	24	31.92	100 s	No TSP	12	38.86
	With TSP	24	16.02		With TSP	12	17.22
80 s	No TSP	24	39.39	80 s	No TSP	12	34.47
	With TSP	24	15.37		With TSP	12	15.70
60 s	No TSP	24	34.37	60 s	No TSP	12	25.93
	With TSP	24	15.33		With TSP	12	15.18
40 s	No TSP	24	26.47	40 s	No TSP	12	30.21
	With TSP	24	16.22		With TSP	12	16.41
20 s	No TSP	24	40.68	20 s	No TSP	11	48.76
	With TSP	24	15.80		With TSP	12	16.36
All	No TSP	120	34.56	All	No TSP	59	35.42
All	With TSP	120	15.75	All	With TSP	60	16.17

Two histograms are added to show the differences of the average delay of buses between the scenario with TSP and the scenario without TSP. The first histogram is drawn for bus headway of 150 seconds and shows the values of average delay for different arrival times of the first buses into the simulation. According to Traffic signal timing sheet (see Apendix), the minimum cycle time is 52 seconds for the actuated timing plan and the maximum cycle time is 102 seconds but can be even longer in the case there is no actuation on competing phases.

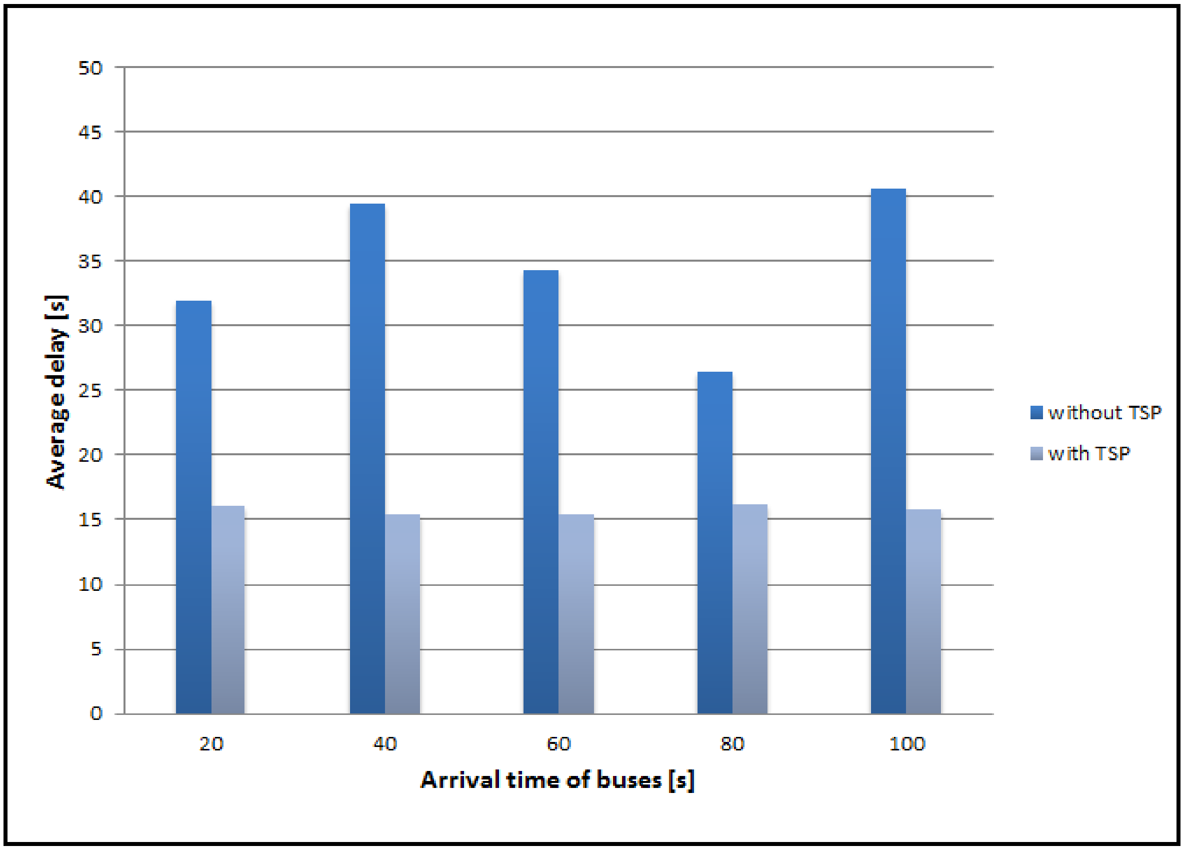


Figure 7.1: Average delay of bus with and without TSP for bus headway of 150 seconds

The second histogram shows the values of average delay for different arrival times of buses into the simulation for bus headway of 300 seconds.

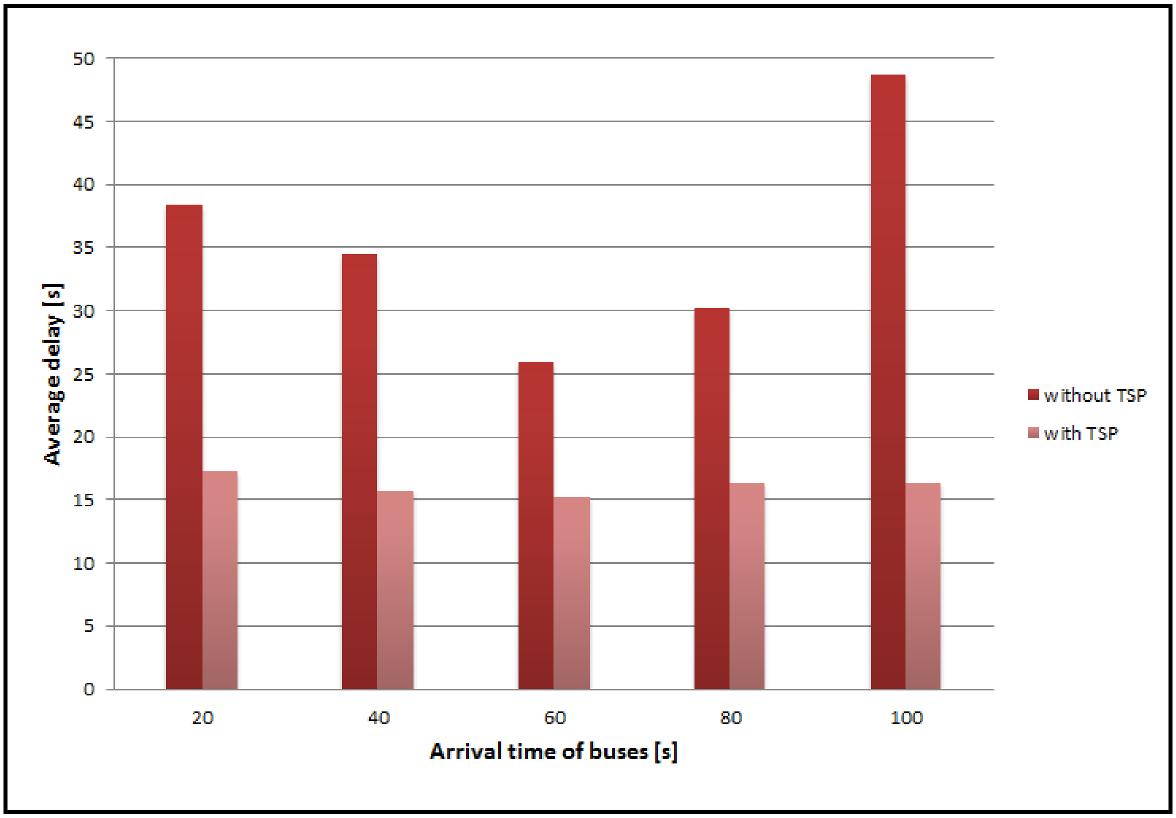


Figure 7.2: Average delay of bus with and without TSP for bus headway of 300 seconds

In VISSIM, the average delay of buses includes the bus dwell time at PT stop. Therefore, the value is always higher than 15 seconds even though the average delay caused by stopping the buses at the intersection stop line in case of the actuated timing plan with TSP is very close to zero. After observing Table 7.2, it is apparent that in all the possible scenarios, the actuated timing plan with TSP decreases the average bus delay significantly compared to the actuated timing plan without TSP. For bus headway of 150 seconds, the TSP decreases the average bus delay by 18.82 seconds. For bus headway of 300 seconds, TSP decreases the average bus delay by 19.25 seconds for the test site.

The second part of this section deals with the overall impact of TSP on the operations of the intersection. The average delay of other vehicles at the intersection for the actuated timing plan with the use of TSP is compared to the average delay of other vehicles at the intersection for the actuated timing

plan without TSP. This evaluation is done for the morning, noon and afternoon period with the same traffic demands as in Chapter 3. Bus headway for each bus line was chosen as 300 seconds, creating a combined headway of 150 seconds and the arrival time of first bus was set to 100 seconds.Results are summarized in the following table. Colored cells in Table 7.3 denote the LOS for the average approach and intersection delays. The colors and LOS criteria are defined according to Table 6.1.

Table 7.3: Average delay of other vehicles with and without TSP

Morning peak hour						
Scenario\Approach		Northbound	Southbound	Westbound	Eastbound	Total
No TSP	# of vehicles	465	1096	586	27	2174
	Av. delay [s]	20.33	22.12	21.15	18.24	21.43
With TSP	# of vehicles	465	1102	583	26	2176
	Av. delay [s]	15.62	16.18	43.97	14.83	23.49
Noon off-peak hour						
Scenario\Approach		Northbound	Southbound	Westbound	Eastbound	Total
No TSP	# of vehicles	419	630	125	19	1193
	Av. delay [s]	14.46	13.77	13.03	16.26	13.98
With TSP	# of vehicles	418	630	139	19	1206
	Av. delay [s]	12.54	12.23	18.32	18.09	13.13
Afternoon peak hour						
Scenario\Approach		Northbound	Southbound	Westbound	Eastbound	Total
No TSP	# of vehicles	777	982	278	16	2053
	Av. delay [s]	20.26	17.30	20.24	16.50	18.81
With TSP	# of vehicles	777	983	277	16	2053
	Av. delay [s]	15.61	14.99	25.01	20.69	16.62

For the morning peak period, the actuated timing plan scenario with TSP has an average delay of 2.06 seconds higher than the actuated timing plan scenario without TSP. This is caused by significant delay (LOS D) for the westbound approach in the scenario with TSP. The westbound approach has high traffic volume in the morning peak hour, but suffers by the insufficient maximum green time, especially for the left-turn movement. Therefore, there are many westbound left-turn vehicles that have to wait for two cycles before being served. On the other hand, theremaining three approaches are served with the lower average delay than in the case of the scenario without TSP.

In the noon period, the average delay for the scenario with TSP is lower than for the scenario without TSP by 0.85 seconds. This is caused by the relatively higher traffic volumes in the northbound and southbound approaches that are served with the lower average delay in the scenario with the use of TSP. With TSP in the same phase as the northbound and southbound movements, these directions are provided with more green time in phase C. Traffic volumes in the remaining two approaches is not high and the average delay in the scenario with TSP is not very high either (LOS B).

The similar case holds for the afternoon period even though the overall traffic volume is significantly higher. The eastbound and westbound approaches have higher average delays in the scenario with TSP. However, the northbound and southbound approaches experienced lower average delay with use of TSP. This is caused by TSP which also benefits other vehicles served within the same phase (phase C). Because there is very significant traffic volume on the northbound and southbound approaches, the average delay for the scenario with TSP has the value of 2.19 seconds lower than the scenario without the use of TSP.

7.4 Chapter Conclusions

The results of this chapter showed the impacts of the implementation of TSP strategies. The average delay for buses was clearlyreduced with the implementation of TSP. After testing different bus arrival times in signal cycles, the average delay for the scenario with TSP using bus headway of 150 seconds was of 18.82 seconds lower than for the scenario without TSP. Almost similarly, the average delay for the scenario with TSP using bus headway of 300 seconds was of 19.25 seconds lower than for the scenario without TSP. At the same time, it is important to evaluate the average delay for other vehicles at the intersection. In the morning period, the average delay for other vehicles in the case of the scenario with the use of TSP was of more than 2 seconds higher than for the scenario without TSP. On the other hand, in the noon off-peak hour and the afternoon peak period, there is a reduction in the average delay for buses as well as for other vehicles. Therefore, the use of TSP appears beneficial for all vehicles at the intersection. The above results are based on bus headways of 150 seconds and 300 seconds respectively.

Chapter 8: Conclusions and Recommendations for Future Research

8.1 Conclusions

There were two objectives of this thesis. The first objective was to compare the Czech and the U.S. methods of isolated fixed time traffic signal control, using an intersection in El Paso, Texas, as a test site. The model of the intersection in VISSIM was developed and both methods were tested as the average delay at the intersection was observed.

Additionally, as the second objective, the U.S. actuated isolated timing plan was evaluated as well and compared against the U.S. and Czech methods of isolated fixed time control, using the same intersection in El Paso, Texas. The same model in VISSIM was used for that purpose. The Czech fixed time traffic signal control plan was evaluated as the most efficient at the intersection of N ReslerDr and High Ridge in El Paso. This design saves 5.28, 5.91 and 5.47 seconds of average delay compared to the U.S. fixed time control plan for the morning, noon and afternoon period respectively and 6.91, 3.64 and 5.40 seconds compared to the U.S. actuated timing plan for the same periods of a day. The main reason for this difference lies in the split interval calculation of the Czech fixed time design method which led to a more optimal phase sequence selection. In addition, the evaluation of the U.S. actuated timing plan in the morning peak, noon off-peak and after peak periods of a working day showed their benefits of this plan compared to the U.S. fixed time control plans.

The third objective was to evaluate the TSP at the same intersection in El Paso, Texas. This evaluation was done viathe RBC add-on module in VISSIM using the same model as for the previous two objectives. The results showed that the average delay for buses was clearly reduced with the

implementation of TSP. After testing different bus arrival times in signal cycles, the average delay for the scenario with TSP using bus headway of 150 seconds was of 18.82 seconds lower than for the scenario without TSP. Almost similarly, the average delay for the scenario with TSP using bus headway of 300 seconds was of 19.25 seconds lower than for the scenario without TSP. In the morning period, the average delay for other vehicles in case of the scenario with the use of TSP was more than 2 seconds higher than for the scenario without TSP. On the other hand, in the noon off-peak hour and the afternoon peak period, there is a reduction in the average delay for buses as well as for other vehicles. Therefore, the use of TSP for the intersection in El Paso, Texas was clearly beneficial.

8.2 Future Research

The isolated intersection of High Ridge & N ReslerDr should be tested in terms of its organization such as geometry or different number of lanes. It was observed that there are unbalanced queue lengths in each lane causing vehicle delays. Furthermore, change of phase sequence should be considered. Results of the Czech fixed time control plan using different phase sequence showed clear decrease of average delay at the intersection.

The signal coordination is recently widely used for intersection sequence in cities but only isolated intersection was considered. The add-on module RBC allows to account the signal coordination. Therefore, the Czech fixed time control plan could be evaluated and compared to the U.S. fixed time control plan using traffic signal coordination.

For the case of this thesis, a bus stop very close to stop line was considered causing possibly inaccurate predictions of bus arrival time to stop line at the intersection. Different bus stop locations and its different distances from the intersection's stop bar should be tested.

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Appendix

CITY OF EL PASO
ENGINEERING DEPARTMENT / TRAFFIC DIVISION
TRAFFIC SIGNAL TIMING SHEET

LOCATION: RESLER &HIGH RIDGE

SYSTEM ID: 230

ADDRESS: 24-0-230

Table A. Controller Timing Information

	Phase Description							
Interval Name	Phase 1 RESLER SBLT	Phase 2 RESLER NBND	Phase 3	Phase 4 HIGH RIDGE E/W	Phase 5 RESLER NBLT	Phase 6 RESLER SBND	Phase 7	Phase 8
Min Green	5	20		10	5	20		
Passage	3	4		3	3	4		
Maximum I	15	40		30	15	40		
Maximum II	15	40		30	15	40		
Veh Clearance	4	4		4	4	4		
All Red	1	2		2	1	2		
Walk		7		7		7		
Ped Clearance		19		13		19		
Recall	OFF	MIN		OFF	OFF	MIN		
Memory	OFF	ON/YELL LK		ON/RED LK	OFF	ON/YELL LK		
Detector	PRESENCE			PULSE	PRESENCE			
Flash Operation	RED	RED		RED	RED	RED		
Prot/P&P Left	P & P				P & P			

Table B. Coordinator Settings (Sec, FO)

Coordinator Timing	Average Cycle 1	Inbound Cycle 2	Outbound Cycle 3
Cycle Length			
Coord.Phase- Resler			
Splits RESLER N/S HIGH RIDGE RESLER LT			
Cycle Check	0	0	0

Table C. Schedule

	Day of Week	
Cycle	Mon-Fri	Sat-Sun
1	NA	NA
2	NA	NA
3	NA	NA
Free	ALL DAY	ALL DAY
Flash	NA	NA

NOTE:

See attached 170 Extra Setting for TOD function

Table D. Revisions

Initials	WO #	Date	Modification
LC	FY04-0080	9/15/03	NEW SIGNAL TIMING
LC	NA	10/13/03	Signal Operational
LC	04-0977	1/22/04	Controller Settings

Vita

PetrMalina was born in Přerov, Czech Republic. After completing his high school education, he moved to Prague, Czech Republic, where he started his higher education at the Czech Technical University (CTU) in Prague. He graduated from the Czech Technical University in Prague in 2010 with a Bachelor Degree at Faculty of Transportation Sciences. During his Bachelor's Degree study he worked part-time in SUDOP Praha in Transport Concept Centre.He was selected for Transatlantic Master's dual-degree program Transport and Logistics Systems**.**This program is jointly offered by the Czech Technical University in Prague(CTU),University of Žilina(UNIZA) and The University of Texas at El Paso(UTEP).Mr. Malina spent first year of his Master's study in EU and the second year in the U.S. at UTEP. While in school at UTEP, he participated in the Institute of Transportation Engineers (ITE) Student Chapter.

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