Center of Open Systems and High Technologies
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Nissan Traffic Simulator Verification Report

Dolgoprudny, February 2011

Abstract

This document contains results of verification process (1) was carried out for Nissan Traffic Simulator (NTS) the microscopic road traffic simulator.

Notes

About Reaction Time

NTS uses IDM model (2) for car-following aspects of vehicle behavior. Authors of original IDM model published article about affecting reaction time delay of IDM commands on stability of traffic flows (3). In the NTS simulator with enabled driver reaction time parameter there is observed quite instable and not realistic traffic flow even with minimal values of reaction time (0.3-0.4 seconds). While this effect is under investigation it was decided to turn off support of Reaction Time Parameter by the NTS Simulation Engine. Actually, we have thoughts that nature of IDM model do not allows introducing such parameter as reaction time because model generally describes planned driver behavior, at least reaction time shouldn't be applied in so straight manner like simple constant time delay between road situation and driver command.

Therefore, in spite of NTS Road Editor allows setting reaction time driver parameter for vehicles behavior NTS Simulation Engine do not applies it for real vehicles. All vehicles have reaction time equal to zero and according to (3), because of NTS update time interval is 0.1 seconds, effective vehicles reaction time is equal to 0.1 seconds.

Measurements methodology

Concept of steady state

Verification Manual from Japan Society of Traffic Engineers defines concept of steady state as a condition in which the volume observed at any point in the section concerned shows an equivalent flow rate within a given tolerance range regardless of the time span. Also manual describes simplified criteria to determine steady state of traffic flow.

For verification of NTS it were determined that traffic flow on the link section is in a steady state when instant average speed of all vehicles in the section do not changes during more than 10 minutes (NTS Instant Average Speed Meter is very useful in this case) and difference between traffic volumes at points on the start and end of the section is constant.

Average measured value in time period

For different verification steps it is required to determine average value of some measurement in period of time (average density and flow in 10 minutes). Because of NTS comes with convenient meters set and meters allow to record history of any measured value average value in a period was calculated as average height of curvilinear trapezoid formed by time interval and history curve of measured value. Meters Information Windows comes with gadget that allows calculating such average value with few mouse clicks.

Typical parameter sets

Table 1. Static vehicle's parameters.

Parameter	Car	Truck
Length, m	4	6
Mass, kg	2500	5000

Table 2. Base Parameter Set.

Index	Name	Standard	Minimum	Maximum
a	**Vehicle and Driver Parameters**			
a-2	Desired Headway, seconds	2.0	1.7	3.0
a-3	Maximum acceleration for cars, m/s2	2.5	1.8	2.5
a-3	Maximum acceleration for trucks, m/s2	1.4	0.8	2.0
a-4	Maximum speed, km/h	60	40	100
b	**Demand Flow Parameters**			
b-2	Percentage of heavy vehicles	15 %	0 %	30 %
c	**Road Network Parameters**			
c-1	Speed Limit, km/h	60	40	100
c-2	Gradient, %	0	-6	6
c-3	Lane Width, m	3.5	2.75	3.5

Table 3. Default vehicle's parameters

Parameter	Car	Truck
Maximum Speed, m/s	16,67	16,67
Max Acceleration, m/s^2	2,5	2,5
Max Deceleration, m/s^2	7	4
Lane change time, s	2	4
Time headway, s	2	2
Min distance, m	2	2
Politeness	0,5	0,5
Abruptness	4	4
Reaction time, s	0	0
Intersection gap time, s	3,0	3,0

Table 4. Default model parameters.

Parameter	Value
Random Seed	1
Immutable vehicles, %	100
Communication Vehicles, %	0
Communication Period, s	300

Vehicles Generation

Generation pattern verification

For verification vehicles generation pattern following dynamics of demand traffic flow was used.

Table 5. Dynamics of demand traffic flow

Period	Total vehicles number	Demand flow, veh/hour
0 – 20 minutes	167	500
20 – 40 minutes	333	1000
40 – 60 minutes	667	2000

Vehicle appearance headway at upstream of the link was measured. NTS doesn't use random number generator to produce demand traffic flow, therefore simulation was carried only once.

NTS implements vehicles generation behavior on fixed time periods, length of the period depends on demand flow rate. In case when vehicles are not accumulated in the origin (no traffic jam on the road), it is possible to calculate appearance headway it equals to inverse value of demand traffic flow rate.

Table 6. Theoretical values of vehicles appearance headway depending on demand flow rate

Demand flow, veh/hour	Average headway, seconds
500	7,2
1000	3,6
2000	0,6

Measured values of vehicles appearance headway is presented on Figure 1.

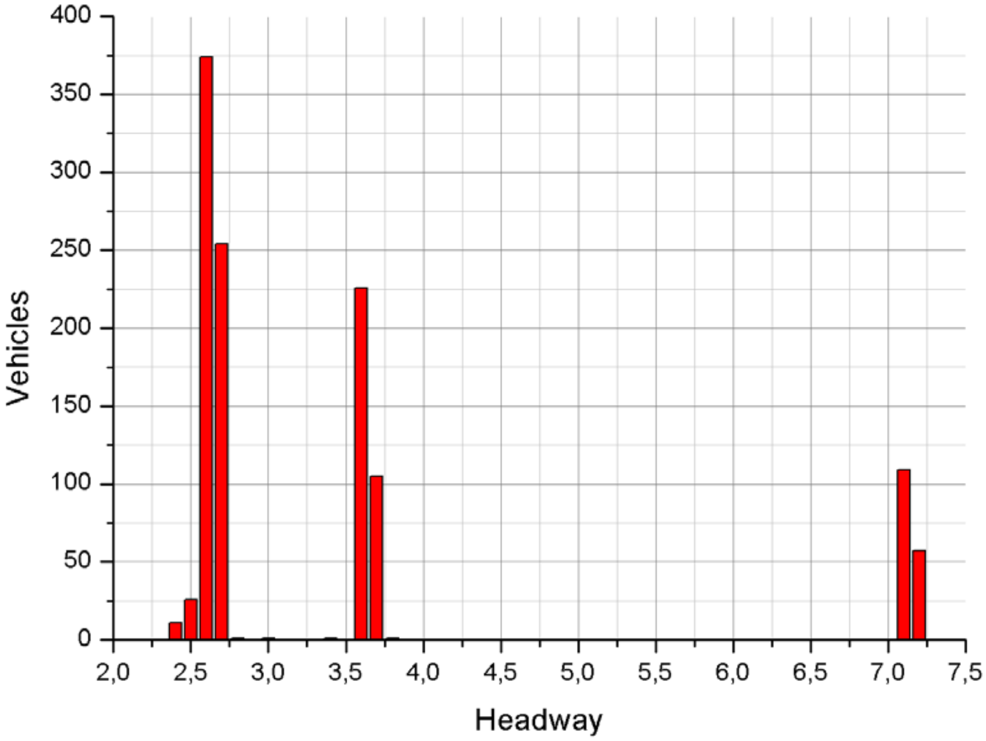


Figure 1. Vehicle appearance headway histogram.

Two peaks near values 7.2 and 3.6 seconds are explained by two first periods of vehicles generation with demand flow rate 500 and 1000 veh/hr, these peaks are in complete compliance with calculated values.

Third peak of measured headways is near value of 2.6 seconds. It not complies with theoretical value of 0.6 seconds, it is explained by the fact that demand flow rate exceeds capacity of link with default vehicles parameters and vehicles retain inside origin in the queue. Link capacity equals to 1360 veh/hr and it is inverse value of 2.6 seconds headway.

Verification of vehicles accumulation inside origin was carried out with demand traffic flow rate of 4400 veh/hr and period of one hour. On the Figure 3 dynamics of traffic volumes for demand and real traffic flow are presented, as it can be seen all demand 4400 vehicles are successfully generated but with a delay of 2 hours due to small link capacity.

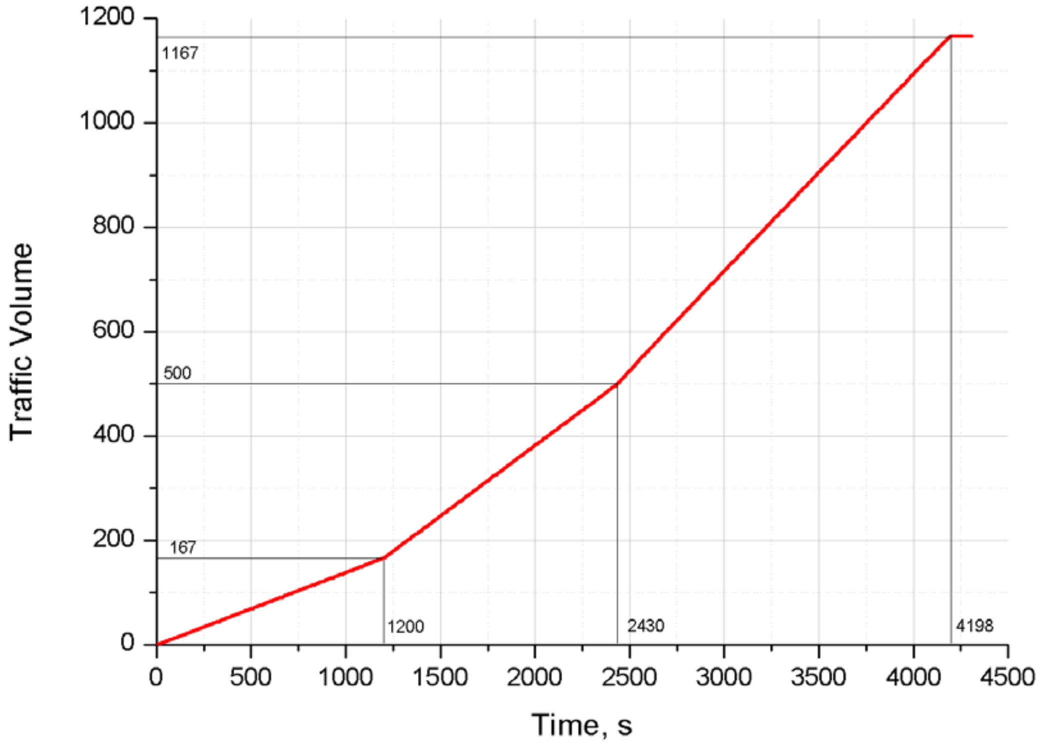


Figure 2. Dynamics of traffic volume near origin with three periods of vehicles generation.

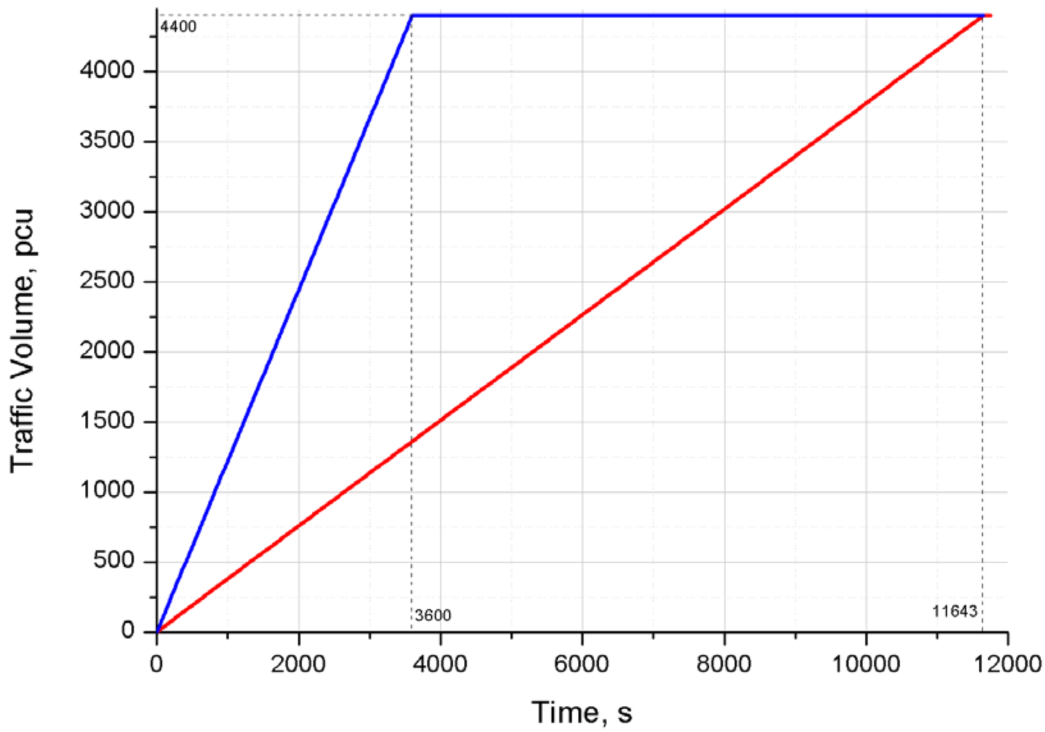


Figure 3. Traffic volume dynamics of real traffic flow and demand exceeds link capacity.

Relationship between model parameters, flow characteristics and bottleneck capacity

Free Flow Characteristics Verification

For verification free flow characteristics parameter sets listed in Table 7 were used. Parameter sets Free-2,4 are not used because they affect on driver reaction time which is turned off in NTS Simulation Engine under verification.

Table 7. Parameter sets for free flow.

Parameter Set	Modification
Free-1	All standard values
Free-4	a-2 minimum
Free-5	a-2 maximum
Free-6	a-3 minimum
Free-7	a-3 maximum
Free-8	a-4 minimum
Free-9	a-4 maximum
Free-10	b-2 minimum
Free-11	b-2 maximum

For each parameter set there were carried out measurements of traffic flow in steady state with different rates of demand traffic flow. Maximum traffic flow rates or link capacities for each parameter set were determined. Results of measurements are presented in Table 8.

Table 8. Free flow verification results.

Parameter Set	Capacity, veh/h	Density, veh/km	Average Speed, km/h
Free-1	1342	37,5	35,8
Free-4	1523	41,3	36,9
Free-5	960	28,5	33,7
Free-6	1341	37,5	35,8
Free-7	1341	37,5	35,8
Free-8	1199	46,9	25,6
Free-9	1492	27,1	55,1
Free-10	1354	38,1	35,5
Free-11	1327	36,8	36,1

Fundamental traffic flow diagrams of free flow for each parameter set are presented overlapped on Figure 4. As we can see most influence on characteristics of traffic flow have desired headway parameter (curves Free-4 and Free-5) and parameter of maximum speed (curves Free-9 and Free-10). Other parameters have almost no effect on flow characteristics.

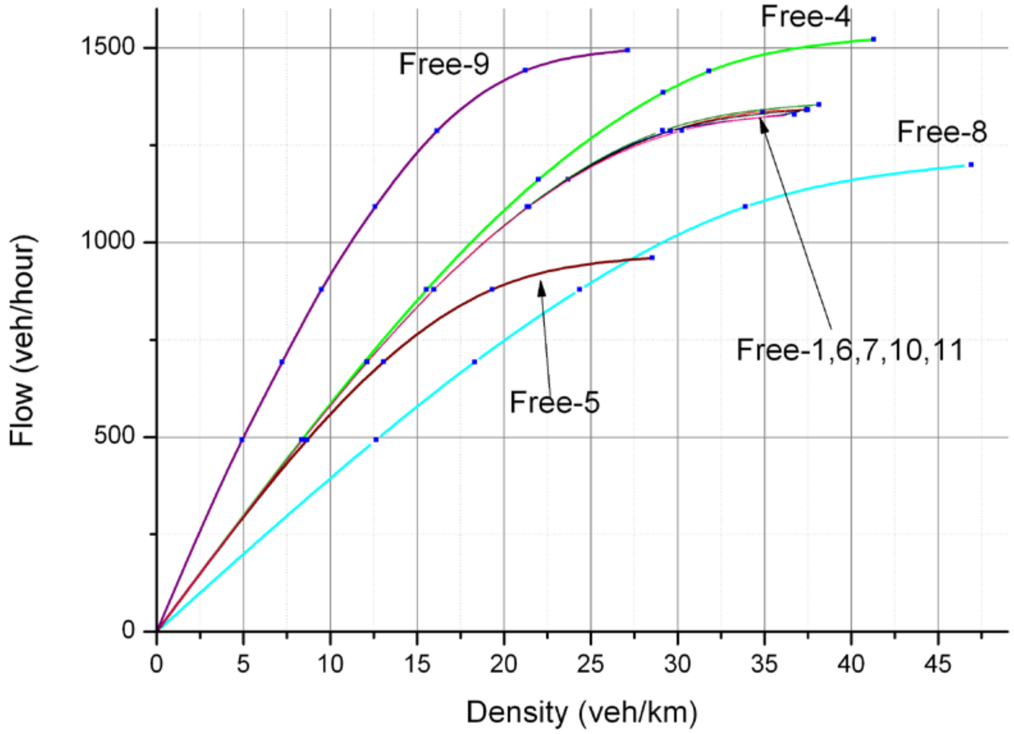


Figure 4. Overview of free flow traffic diagrams.

Congested Flow Characteristics Verification

For verification flow characteristics in congested state simple maps with bottlenecks were created according to verification manual. Parameter sets listed in Table 9 were used.

Table 9. Parameter sets for congested traffic flow

Parameter Set	Modification
Jam-1	Change Free-1 + c-1) of downstream link from standard to lower limit
Jam-2	Change Free-1 + c-2) of downstream link from standard to upper limit
Jam-3	Change Free-1 + c-3) of downstream link from standard to lower limit
Jam-13	Change Free-4 + c-1) of downstream link from standard to lower limit
Jam-14	Change Free-4 + c-2) of downstream link from standard to upper limit
Jam-15	Change Free-4 + c-3) of downstream link from standard to lower limit
Jam-17	Change Free-5 + c-1) of downstream link from standard to lower limit
Jam-18	Change Free-5 + c-2) of downstream link from standard to upper limit
Jam-19	Change Free-5 + c-3) of downstream link from standard to lower limit
Jam-21	Change Free-6 + c-1) of downstream link from standard to lower limit
Jam-22	Change Free-6 + c-2) of downstream link from standard to upper limit
Jam-23	Change Free-6 + c-3) of downstream link from standard to lower limit
Jam-25	Change Free-7 + c-1) of downstream link from standard to lower limit
Jam-26	Change Free-7 + c-2) of downstream link from standard to upper limit
Jam-27	Change Free-7 + c-3) of downstream link from standard to lower limit
Jam-29	Change Free-8 + c-1) of downstream link from standard to lower limit
Jam-30	Change Free-8 + c-2) of downstream link from standard to upper limit
Jam-31	Change Free-8 + c-3) of downstream link from standard to lower limit
Jam-33	Change Free-9 + c-1) of downstream link from standard to lower limit
Jam-34	Change Free-9 + c-2) of downstream link from standard to upper limit
Jam-35	Change Free-9 + c-3) of downstream link from standard to lower limit
Jam-37	Change Free-10 + c-1) of downstream link from standard to lower limit
Jam-38	Change Free-10 + c-2) of downstream link from standard to upper limit
Jam-39	Change Free-10 + c-3) of downstream link from standard to lower limit
Jam-41	Change Free-11 + c-1) of downstream link from standard to lower limit
Jam-42	Change Free-11 + c-2) of downstream link from standard to upper limit
Jam-43	Change Free-11 + c-3) of downstream link from standard to lower limit

For each parameter set corresponded property of bottleneck section was changed to vary bottleneck capacity. Observations shown that is standard range of bottleneck properties there are no any influence on the characteristics of flow in steady state. Significant influence on flow characteristics is observed only with parameter set Jam-33 and Jam-35.

Absence of any affect from existence of potential bottleneck on the road for most parameter sets is explained by a fact that all bottleneck properties eventually effects on maximum speed of vehicles (desired speed in IDM equation). Speed limitation sign effects explicitly, gradient decreases maximum speed because of decreasing effective engine power, lane width force driver to decrease its maximum speed because of safety reason.

In other hand saturated flow on a link in a steady state has speed that differs from vehicles maximum speed. Table 8 contains measured saturated flow speed in a steady state.

Analyzing IDM Model equation it can be derived formula for maximum flow rate depending on flow speed in a steady state:

$$F = \upsilon \frac{\sqrt{1-(\upsilon/\upsilon_0)^{\partial}}}{s_0 + \upsilon T}$$

Plot of this equation for Free-1 parameter set is presented on Figure 5. As it can be seen maximum flow is achieved with speeds near 30 km/h and this fact is confirmed by data from Table 8.

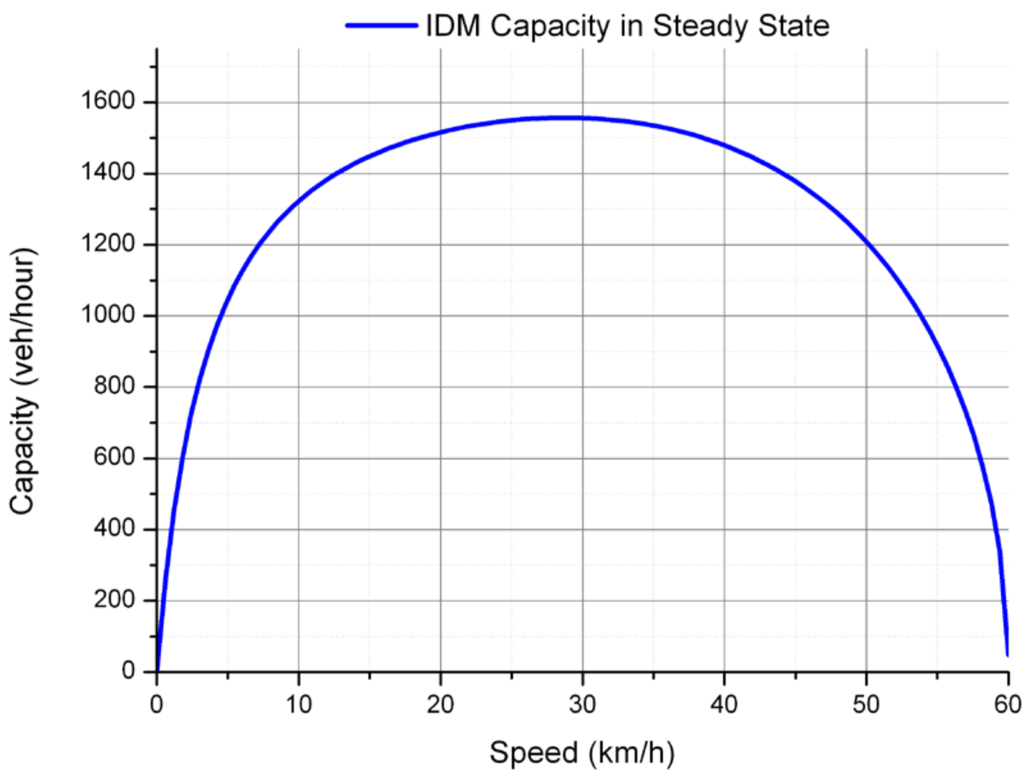


Figure 5. IDM link capacity in a steady state.

While bottleneck properties changed in standard ranges (ranges from Verification Manual) influence of bottleneck on a maximum speed is not enough to decrease speed less than speed of saturated steady flow. Only with parameter set Free-9 saturated flow speed is high enough to allow bottleneck effect on flow characteristics. On Figure 6 traffic diagram for Jam-33 parameter set is presented.

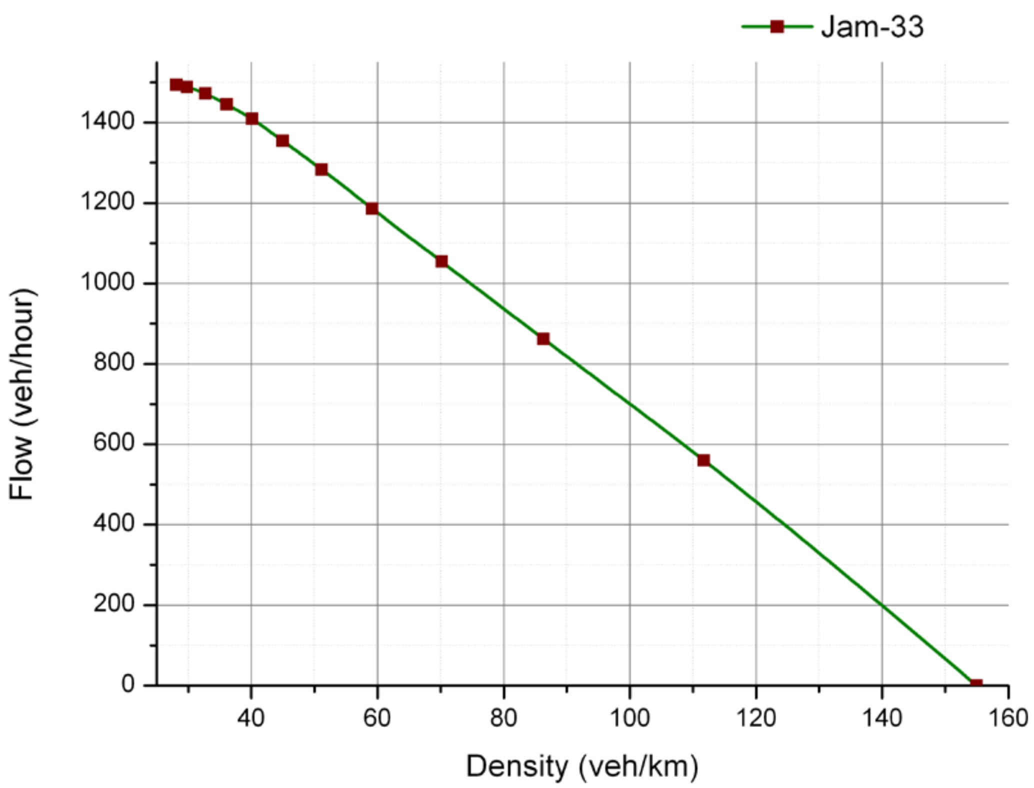


Figure 6. Jam-33 traffic diagram.

For verification flow characteristics in congested state standard ranges of bottleneck properties were changed. In Table 10 modified parameter sets are listed. Only sign limitation sign was used to create bottleneck for modified parameter sets because it is hard to define such gradient and lane width that will effect on characteristics of traffic flow in a steady state.

Table 10. Modified Jam parameter sets.

Parameter Set	Modification
Jam-1	Change Free-1 + c-1) from 30 to 5 km/h with step 5km/h
Jam-13	Change Free-4 + c-1) from 30 to 5 km/h with step 5km/h
Jam-17	Change Free-5 + c-1) from 30 to 5 km/h with step 5km/h
Jam-21	Change Free-6 + c-1) from 30 to 5 km/h with step 5km/h
Jam-25	Change Free-7 + c-1) from 30 to 5 km/h with step 5km/h
Jam-29	Change Free-8 + c-1) from 30 to 5 km/h with step 5km/h
Jam-33	Change Free-9 + c-1) from 55 to 5 km/h with step 5km/h
Jam-37	Change Free-10 + c-1) from 30 to 5 km/h with step 5km/h
Jam-41	Change Free-11 + c-1) from 30 to 5 km/h with step 5km/h

For each parameter set from Table 10 simulations were carried out according to Verification Manual, traffic flow rate and density were measured and plotted on a traffic diagram. Traffic diagrams and measured data for each parameter set are presented in the Appendix.

On Figure 9 overview of whole fundamental traffic diagrams for typical parameter sets is presented. Diagrams of parameter sets Free-6,7,10 and 11 are not presented because they all are almost match to diagram of Free-1 parameter set.

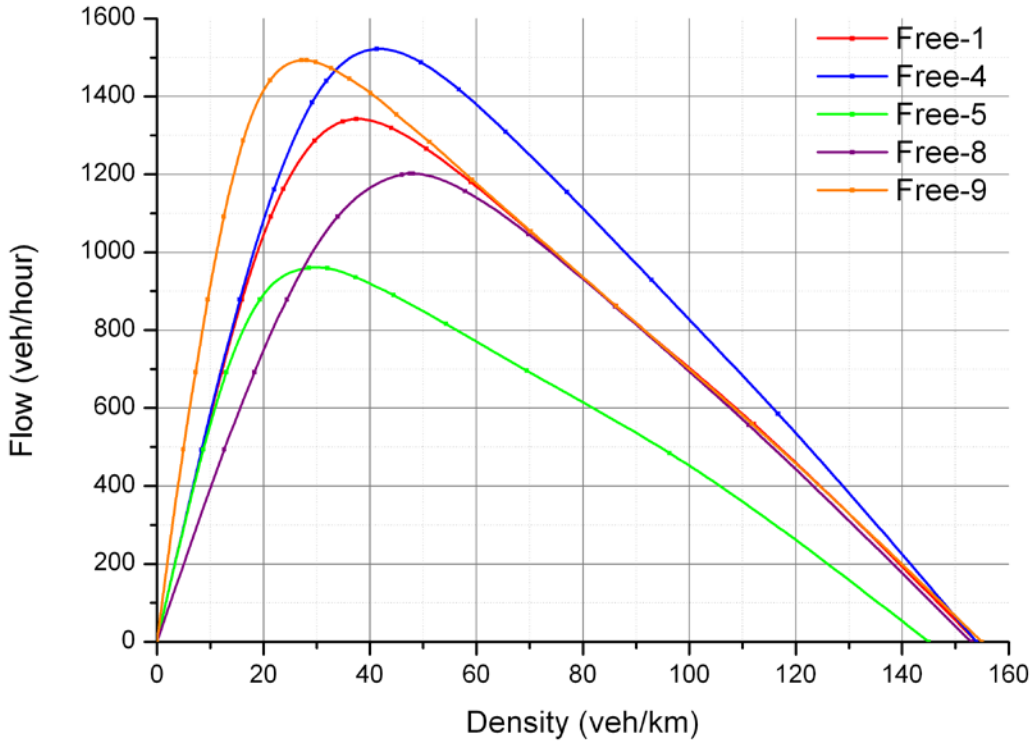


Figure 7. Overview of Traffic Diagrams

Relationship between model parameter and saturation flow rate

For verification of relationship between model parameters and saturation flow rate there were used standard parameter sets and road network configurations from verification manual. Parameters indexed as a), b) and c) stand for base parameters listed in the beginning of the report.

Table 11. Saturation Flow Parameter Sets

Parameter Set	Modification
SFR-1	All standard values
SFR-4	a-2) minimum
SFR-5	a-2) maximum
SFR-6	a-3) minimum
SFR-7	a-3) maximum
SFR-8	a-4) minimum
SFR-9	a-4) maximum
SFR-10	b-2) minimum
SFR-11	b-2) maximum
SFR-12	c-1) minimum
SFR-13	c-1) maximum
SFR-14	c-2) minimum
SFR-15	c-2) maximum
SFR-16	c-3) minimum

For each parameter set simulation process was carried out according to verification manual. Saturation flow rates appear to be stable for each parameter set. Table 12 contains information about measured saturation flow rates for each parameter. Graphs with overlapped traffic volume curves for ten cycles for each parameter set are given in the Appendix.

Data from Table 12 presented on Figure 10. As it can be seen, again, parameter sets Free-4,5 anв Free-8,9 have significant influence on saturation flow rate, while over vehicle and driver parameters and even different characteristics of link like speed limitation signs, gradient and lane width have almost no impact on saturation flow rate.

Also it is interesting to compare saturation flow rate and link capacities for the same vehicle and driver parameter sets. Saturation flow rate always greater than link capacity because of flow that originated by opening traffic light signal is not in a steady state and vehicles have more freedom to accelerate speed and moving forward.

Table 12. Saturation Flow Rates

Parameter Set	Saturation Flow, veh/hr	Capacity, veh/hr
SFR-1	1370	1342
SFR-4	1533	1523
SFR-5	1030	960
SFR-6	1360	1341
SFR-7	1386	1341
SFR-8	1276	1199
SFR-9	1498	1492
SFR-10	1395	1355
SFR-11	1368	1326
SFR-12	1360	1342
SFR-13	1364	1342
SFR-14	1383	1342
SFR-15	1357	1342
SFR-16	1357	1342

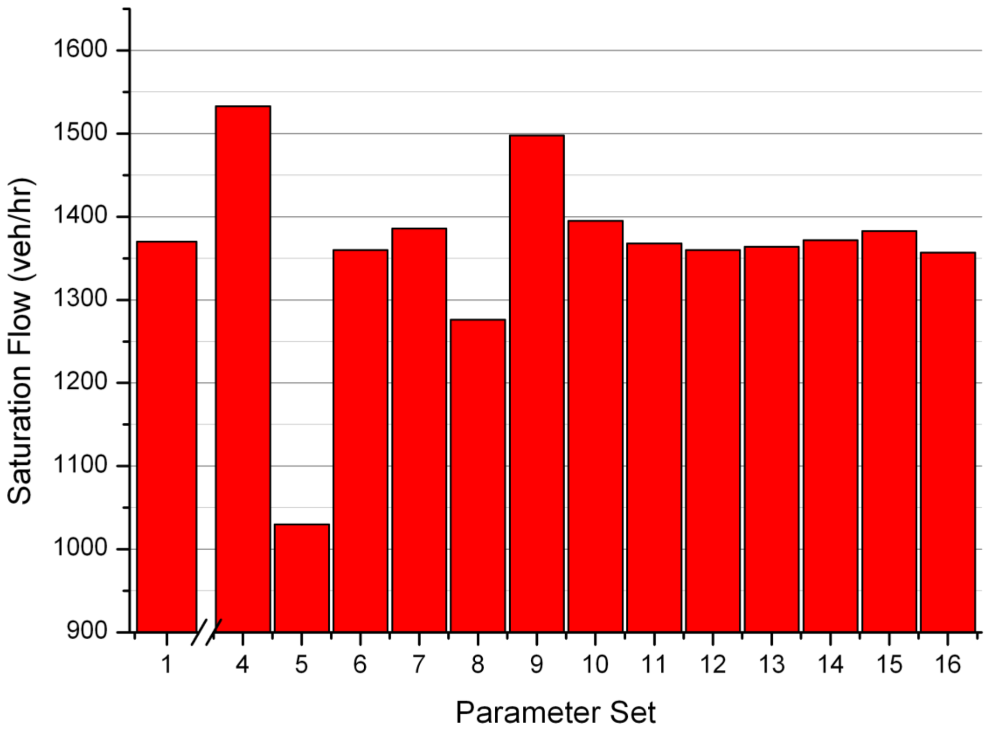


Figure 8. Saturation flow rates overview

Shock Waves

Shock waves with bottleneck

For verification of shock waves propagation there was created SWT-4 map with 2km length of ordinary full sized one lane link and with bottleneck on the end of this link. Bottleneck was created by means of speed limitation sign with limit to 10 km/h. Traffic Volume Meters were established on each 250 meter length section from bottleneck downstream to the beginning of link, so, total number of measured points were 9. There was used Free-4 parameters set for driver behavior parameters.

Shock waves propagation rates were calculated from traffic diagram for Free-4 parameter set as shown on Figure 9.

Dynamics of demand traffic flow was set according to Table 13.

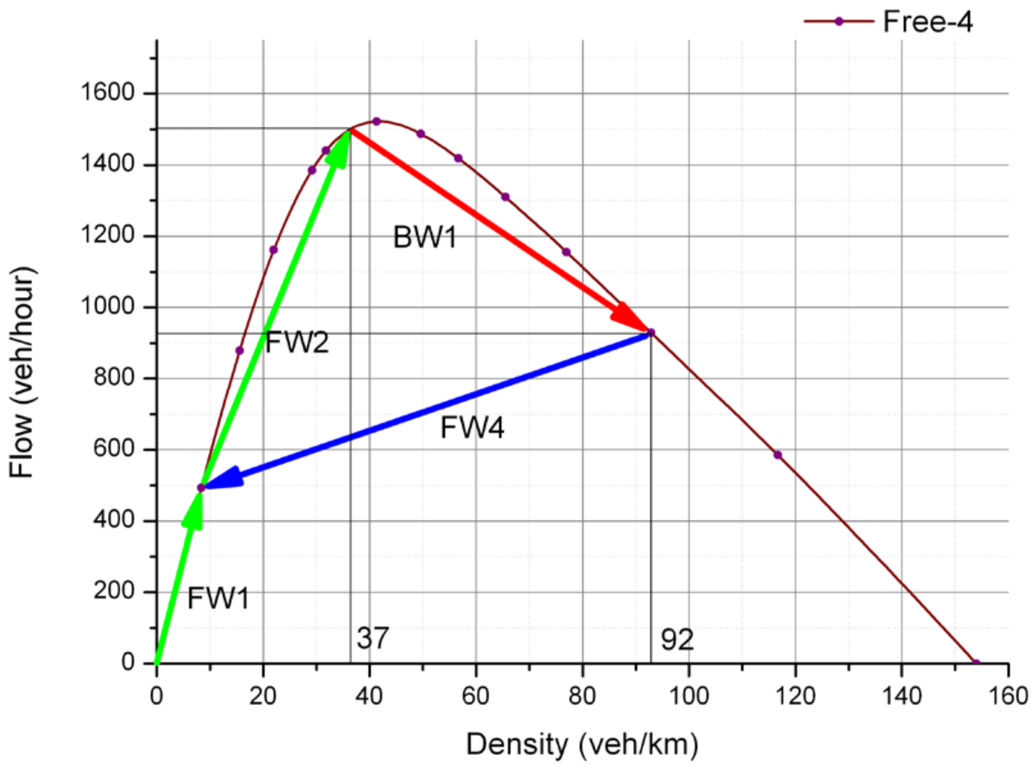


Figure 9. Determining shock wave propagation rates for Free-4 parameter set.

Table 13. Dynamics of demand traffic flow.

Period	Demand, veh/hour
0-300 seconds	500
300-1000 seconds	1500
1000-2400 seconds	500

Simulation was made for described above road network and dynamics of demand flow. Data from each of established Traffic Volume Meters were recorded and Traffic Condition Transition Diagram was created, it presented on Figure 10. Actual shock wave propagation rates were measured using Traffic Condition Transition Diagram. Table 14 contains comparison of theoretical and actual shock waves propagation speeds.

Table 14. Shock Waves Speed

Wave	Theoretical Speed, km/h	Observed Speed, km/h
FW1	59	59
FW2	40	49
BW1	-10,2	-9,6
FW3	40	36
FW4	5,2	5,1

Difference between theoretical and actual values of FW2 propagation speed can be explained by fact that when FW2 starts propagation from origin its traffic flow in not in a steady state and first vehicles are moved very fast almost with the speed of vehicles from FW1 wave, i.e. their speed is larger and propagation rate is higher. All other shock wave speeds seems to be equals taking into account measurement inaccuracy.

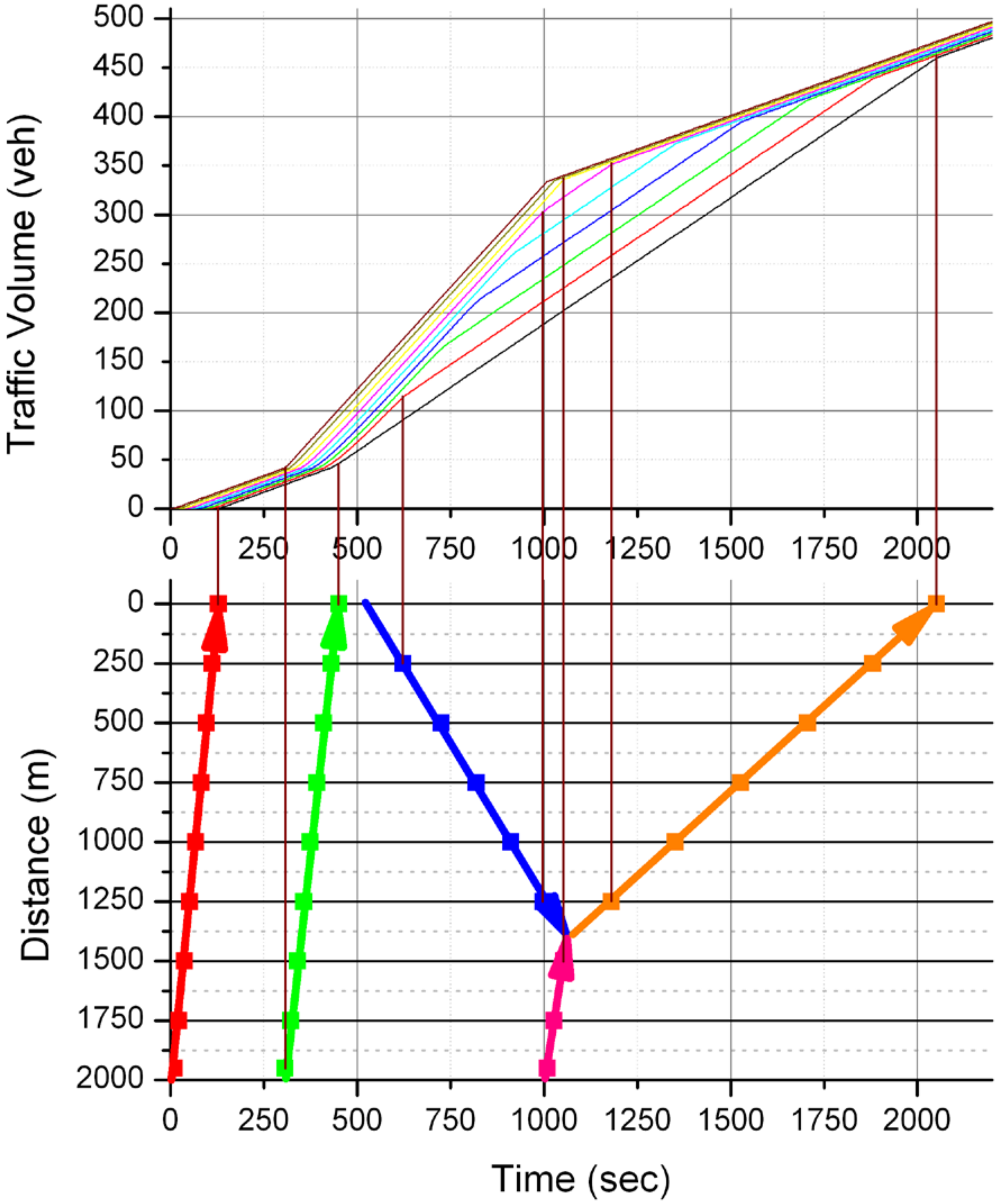


Figure 10. Traffic Condition Transition Diagram for SWT-4 map.

Shock waves with signalized intersection

For verification of shock wave propagation in case of model with signalized intersection there was used Free-1 parameter set.

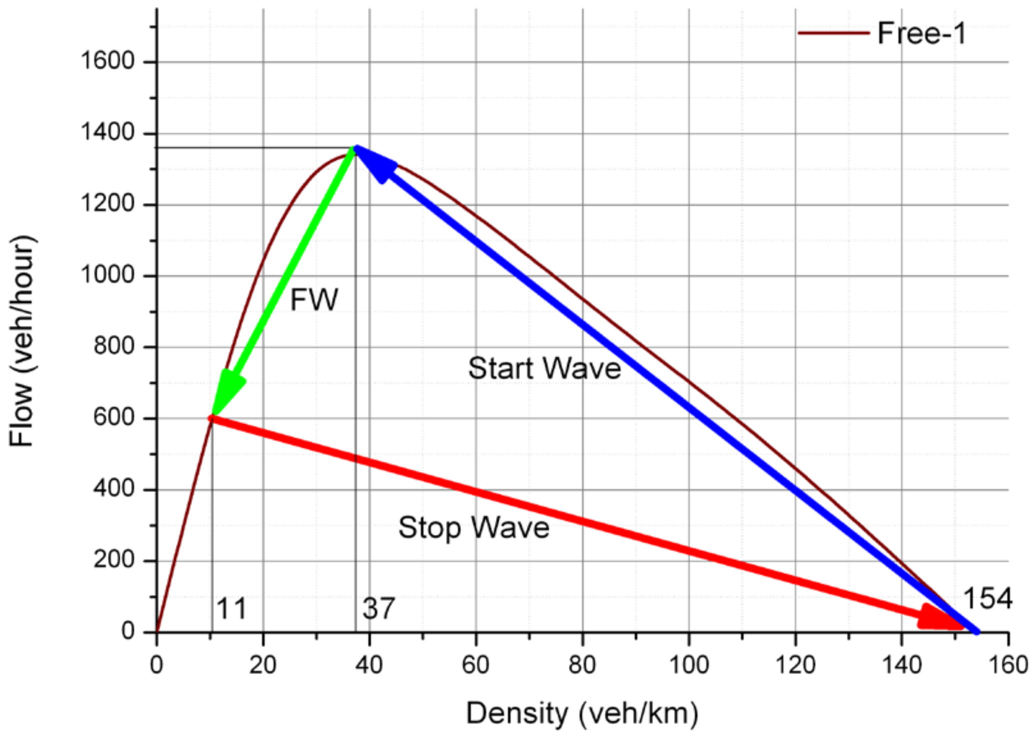


Figure 11. Determining shock waves propagation rates for Free-1 parameter set in case of model with signalized intersection.

According to verification manual theoretical values of shock wave propagation rates were calculated on the base of traffic diagram for Free-1 parameter set.

Actual waves propagation rates were measured from Traffic Condition Transition Diagram (presented on Figure 12) that was constructed on the base of measured data.

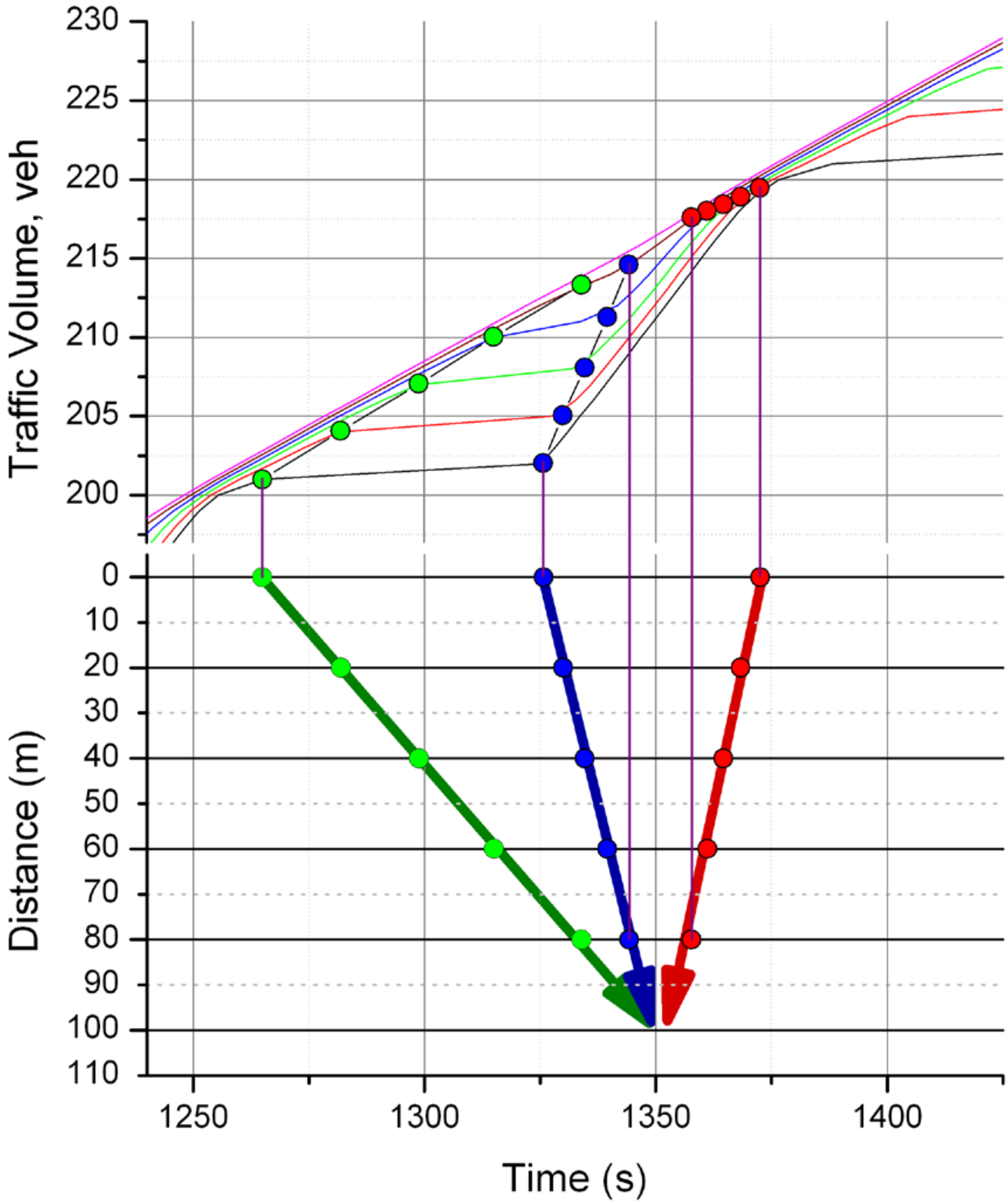
Bellow comparison table for theoretical and experimental shock waves propagation rates is presented.

Table 15. Shock waves speeds near signalized intersection.

Wave	Theoretical rate (km/h)	Experimental rate (km/h)
Stop Wave	-4,2	-4,2
Start Wave	-11,7	-15,5
Forward Wave	29,6	19,5

As it can be seen, shock wave theory precisely predict propagation rate of stop wave, but theoretical values differs a lot for start and forward waves. This can be explained by the fact that traffic flow on start is not in steady state (even saturation flow rate exceeds capacity of link in a steady state). At the same time traffic flow is close to steady state when stop wave propagates backward, this fact explains high precision of theoretical prediction.

Figure 12. Traffic Condition Transition Diagram for signalized intersection traffic waves



Merge Ratio Verification

For verification of merge ratios principal parameters listed in the Table 16 were used. Because of NTS employs MOBIL lane change model and vehicles have finite lane change speed additional principal parameters d-1 and d-2 were introduced for verification.

On the base of individual principal parameters there was list of parameter sets created, it presented in Table 17.

Table 16. Principal input items and parameters affecting merge behavior.

Name		A	B	C
d) For driving behavior of vehicles				
d-1)	Politeness	0,5	0	1
d-2)	Lane Change Time	2	1	4
e) For the merge section				
e-1)	Merge section length	100m	0m	200m
f) Merge branch and main lane demand ratio				
f-1)	Total 2000[veh/hr]	3:7	1:9	5:5

Table 17. Parameter sets used for verification of merge area.

Parameter Set	Modification
Merge-1	«Free-1» + «d-1-A» + «d-2-A» + «e-1-A» + «f-1-A»
Merge-2	«Free-1» + «d-1-B» + «d-2-A» + «e-1-A» + «f-1-A»
Merge-3	«Free-1» + «d-1-C» + «d-2-A» + «e-1-A» + «f-1-A»
Merge-4	«Free-1» + «d-1-A» + «d-2-B» + «e-1-A» + «f-1-A»
Merge-5	«Free-1» + «d-1-A» + «d-2-C» + «e-1-A» + «f-1-A»
Merge-6	«Free-1» + «d-1-A» + «d-2-A» + «e-1-B» + «f-1-A»
Merge-7	«Free-1» + «d-1-A» + «d-2-A» + «e-1-C» + «f-1-A»
Merge-8	«Free-1» + «d-1-A» + «d-2-A» + «e-1-A» + «f-1-B»
Merge-9	«Free-1» + «d-1-A» + «d-2-A» + «e-1-A» + «f-1-C»

Simulation process was carried out for each of parameter set in Table 17. Traffic flows on main line and merge branch were measured and merge ratio was calculated. Results of measurements are presented in Table 18. Overview of merge ratios is presented by diagram on Figure 13. Plots of traffic volume curves for each parameter set are presented in the Appendix.

Let’s explain merge ratios results for each parameter set.

Merge-1 is merge ratio that will be used as standard because it is all standard parameters set. Branch merge ratio of Merge-1 is quite small only 10% and as it can be seen from its traffic volume curve it is quite unstable. It can be explained by fact that traffic flow on main line is close to saturation and even exceeds it. For vehicles on branch link it is quite hard to find gap in such saturated flow.

Merge-2 is parameter set with zero politeness driver parameter. Drivers are more aggressive on the road and therefore merge ratio becomes larger than Merge-1, also Merge-2 merge dynamic expose enough stability as it can be seen from branch traffic volume curve.

Merge-3 with politeness factor equals to 1 exposes the same merge ratio as standard Merge-1 parameter set, as well as stability of traffic flow is very low. Drivers with politeness 1 are not aggressive and merge ratio is smaller than Merge-2.

Merge-4 exposes larger merger ratio because of higher lane change speed. Vehicles can accept smaller gaps on main line traffic flow.

Merge-5 merge ratio is equal to zero. Lane change time for this parameter set is equal to 4 seconds. Vehicles on branch line are too slow to find gaps in flow on main line.

Merge-6 exposes largest merge ratio. It is because merge of links with zero merge section length are treated by vehicles as merge of links with equal priority and vehicle on both link tries to respect each over and capacity of merge area is shared uniformly between traffic flows. Therefore, capacity of each of merged link is equal to 1350/2=675 veh/hr, i.e. branch flow doesn't form traffic jam because its rate is 600 veh/hr, but flow on main line uses all over capacity 1350-600=750 veh/hr. This evaluation is in precise compliance with measured values.

Merge-7 exposes large merge ratio against standard Merge-1 because merge section length is longer and more vehicles are able to find gaps in the main line flow.

Merge-8 for exposes a little bit larger branch merge ratio than standard parameter set, it can be explained by the fact that demand floe ratio is equal to merge ratio of standard parameter set and vehicles from branch link have more ability to find gaps in the flow on main line.

Merge-9 exposes large merge ratio because flow on main line is not so saturated and there are a lot of gaps in the main line flow.

Table 18. Measured Merge Ratios.

Parameter Set	Traffic Flow, veh/hr			Merge Ratios
	Main Line	Branch	Total Capacity	
Merge-1	1250	128	1378	0,10 / 0,90
Merge-2	1188	174	1362	0,13 / 0,87
Merge-3	1249	130	1379	0,10 / 0,90
Merge-4	1167	192	1359	0,14 / 0,86
Merge-5	1366	0	1366	0,00 / 1,00
Merge-6	739	590	1329	0,44 / 0,56
Merge-7	1187	210	1397	0,15 / 0,85
Merge-8	1222	163	1385	0,12 / 0,88
Merge-9	972	397	1369	0,29 / 0,71

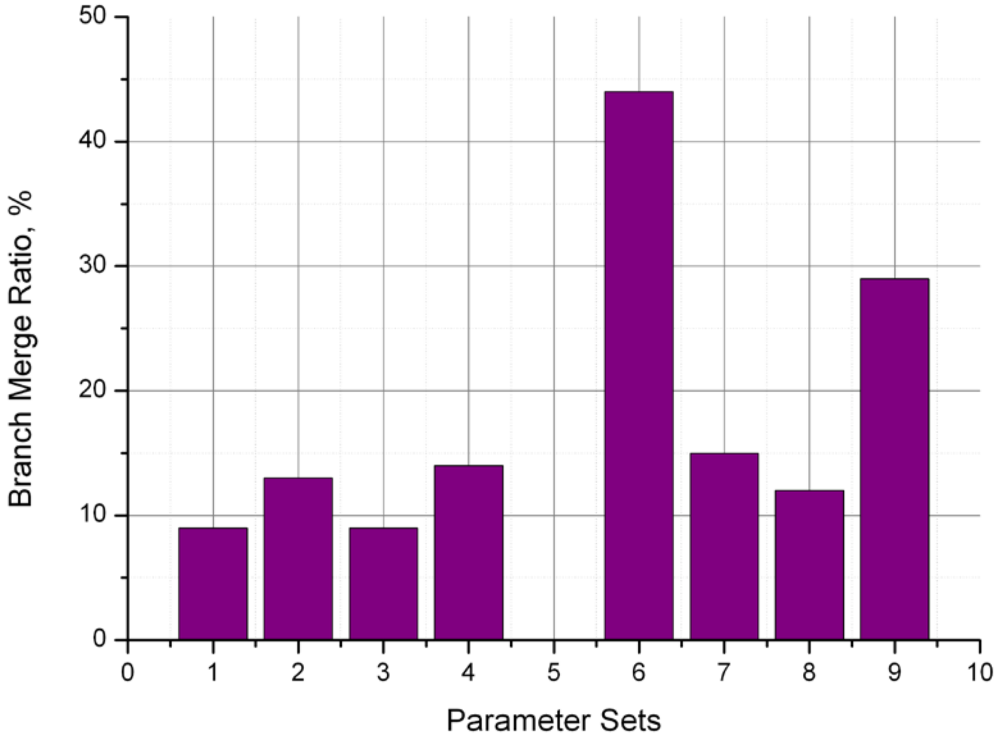


Figure 13. Merge Ratios Diagram.

Right Turn Capacity Verification

For verification of right turn capacity in the signalized intersection road network with configuration described in verification manual was created. There were used principal input items from Table 19 and parameter sets from Table 20.

Verification procedure was carried out according to manual, right-turn capacities for each parameter set was measured and plotted on the graph near with empirical equation from Japan Society of Traffic Engineers. All graphs can be found in Appendix to this document. On Figure 7 graph for Rturn-6 parameter set is shown, this parameter set is most complied to empirical equation.

Table 19. Principal input items and parameters affecting right-turn capacity.

Name		A	B	C
g) For driving behavior of vehicles				
g-1)	Opposing straight through gap acceptance threshold	3.0 sec	5.0 sec	8.0 sec
h) For intersection structure				
h-1)	No. of vehicles retained in the intersection	1	2	4
h-2)	Min. right-turn headway	1,7	2	3
i) Signal parameters				
i-1)	Cycle	120sec (common to all cases)		
i-2)	Effective green time	Three type of 40, 60 and 80sec		
j) For demand				
j-1)	Opposing through demand	Variable in 200 steps from 200-1200[veh/hr]		
j-2)	Right-turn demand	2000[veh/hr] (common to all cases)		

Table 20. Parameter sets for right-turn capacity verification

Rturn-1	“SFR-1” + “g-1-A” + “h-1-A” + “h-2-A”
Rturn-2	“SFR-1” + “g-1-B” + “h-1-A” + “h-2-A”
Rturn-3	“SFR-1” + “g-1-C” + “h-1-A” + “h-2-A”
Rturn-4	“SFR-1” + “g-1-A” + “h-1-B” + “h-2-A”
Rturn-5	“SFR-1” + “g-1-A” + “h-1-C” + “h-2-A”
Rturn-6	“SFR-1” + “g-1-A” + “h-1-A” + “h-2-B”
Rturn-7	“SFR-1” + “g-1-A” + “h-1-A” + “h-2-C”

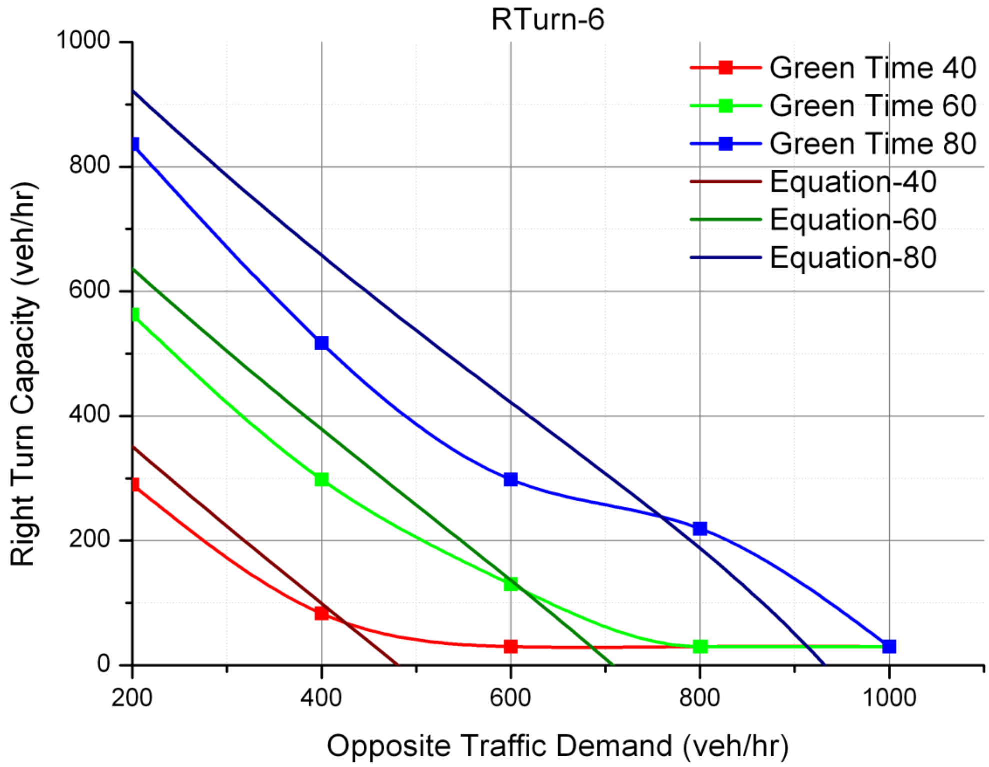


Figure 14. Graph for RTurn-6 parameter set.

Route Selection Behavior

Verification procedure for route selection behavior was not carried out for NTS, because NTS uses quite simple logic to select routes – all vehicles use Dijkstra's algorithm to choose shortest route from origin to destination in the road network.

References

1. *Verification process and its application to network traffic simulation models.* **Ryota Horiguchi, Masao Kuwahara.** 3, Autumn (Fall) 2002 г., Journal of Advanced Transportation, Т. 36. pages 243–264.

2. *Congested Traffic States in Empirical Observations and Microscopic Simulations.* **Martin Treiber, Ansgar Hennecke, Dirk Helbing.** 2000, Physical Review, Vol. E 62. 1805-1824.

3. *How Reaction Time, Update Time, and Adaptation Time Influence the Stability of Traffic Flow.* **Arne Kesting, Martin Treiber.** 2008, Computer-Aided Civil and Infrastructure Engineering, Vol. 23. 125–137.

Appendix

Free Flow Traffic Diagrams and Data

Figure 15. Free-1 Flow Traffic Diagram

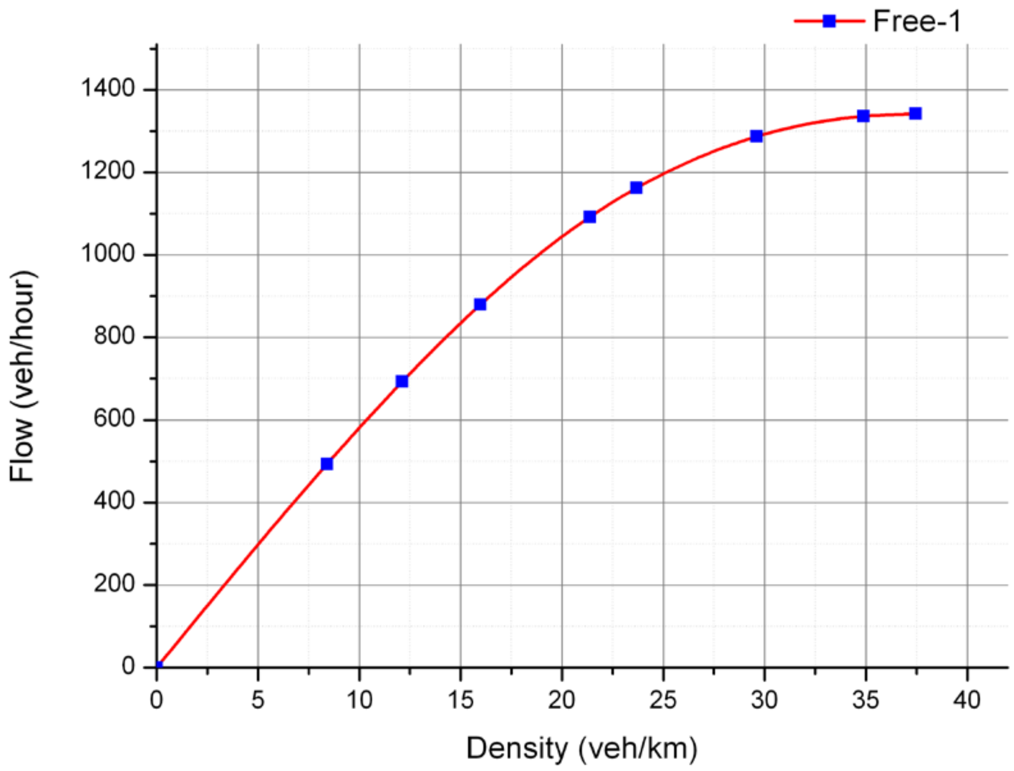


Table 21. Free-1 Flow Data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,41	493,5
700	12,12	692,5
900	15,97	879
1100	21,37	1091,7
1200	23,69	1162,1
1300	29,6	1286,6
1350	34,88	1336
1400	37,46	1342

Figure 16. Free-4 Flow Traffic Diagram.

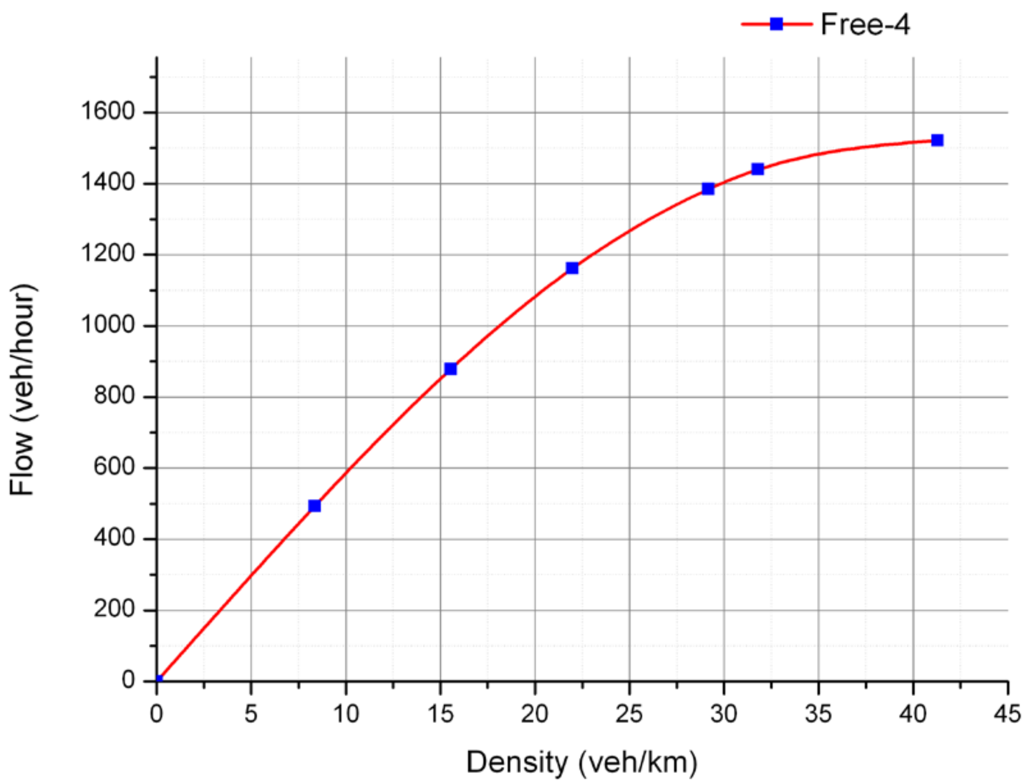
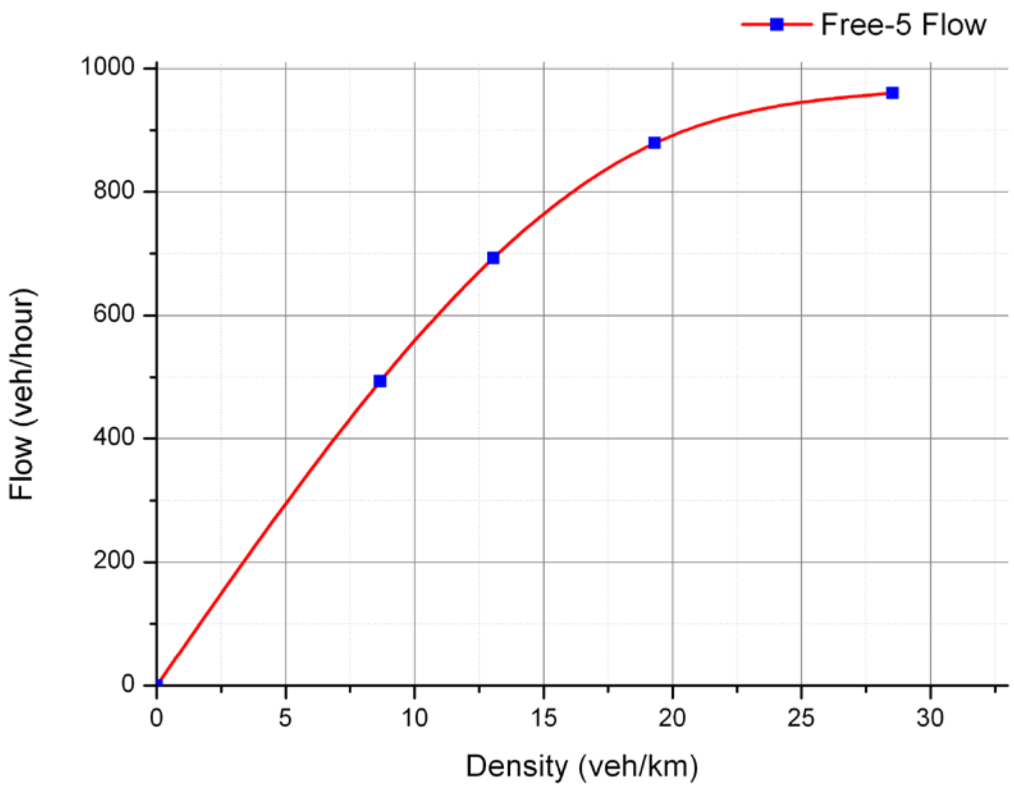


Table 22. Free-4 Flow Data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,35	492,8
900	15,55	878,7
1200	21,98	1161,5
1400	29,17	1384,8
1500	31,8	1440
1600	41,3	1522
1650	41,3	1523

Figure 17. Free-5 Flow Traffic Diagram.



.

Table 23. Free-5 Flow Data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,67	493,3
700	13,06	693
900	19,32	879
1100	28,54	962
1200	28,58	961
1000	28,54	960

Figure 18. Free-6 Flow Traffic Diagram.

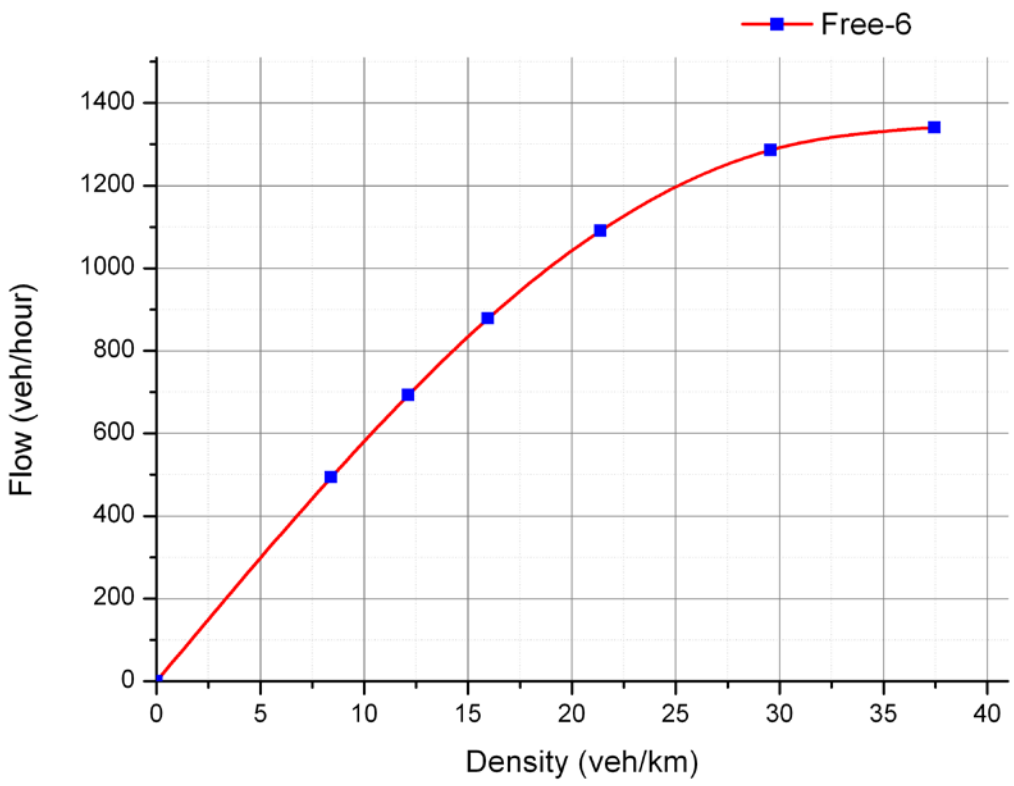


Table 24. Free-6 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,41	493,4
700	12,12	692,7
900	15,97	878,7
1100	21,38	1090,8
1300	29,57	1286
1400	37,45	1341

Figure 19. Free-4 Flow Traffic Diagram.

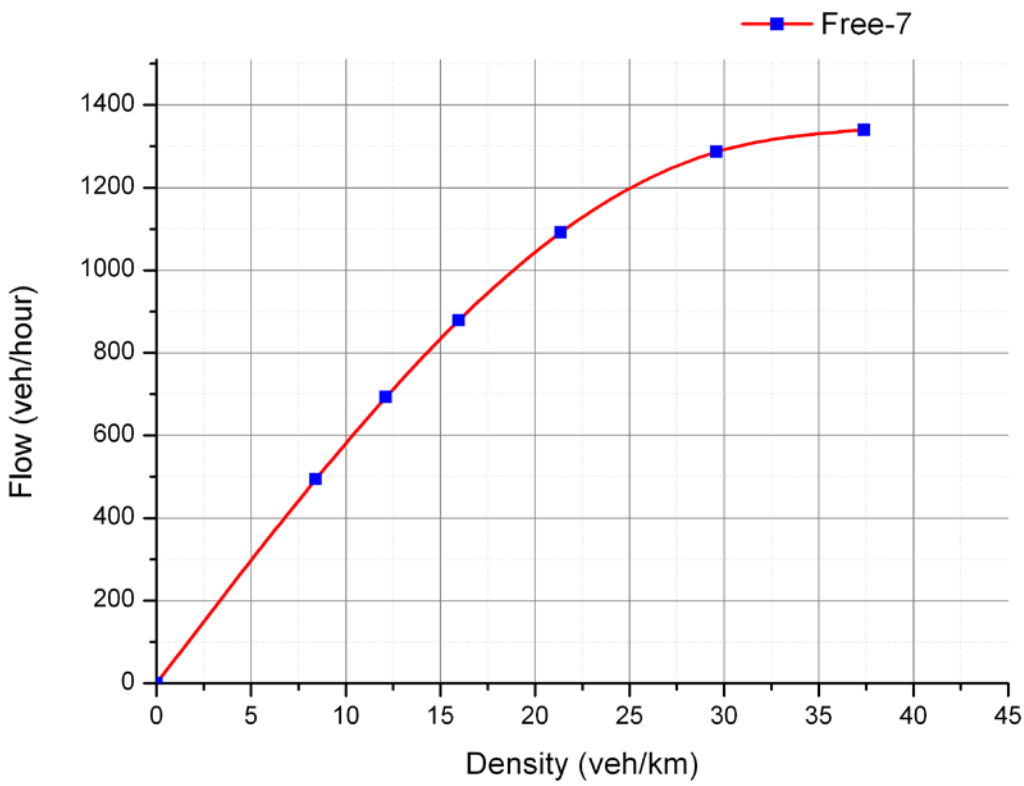


Table 25. Free-7 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,41	493,4
700	12,12	692,7
900	15,97	878,6
1100	21,37	1091,7
1300	29,6	1286,3
1400	37,4	1340
1500	37,5	1341

Figure 20. Free-10 Flow Traffic Diagram.

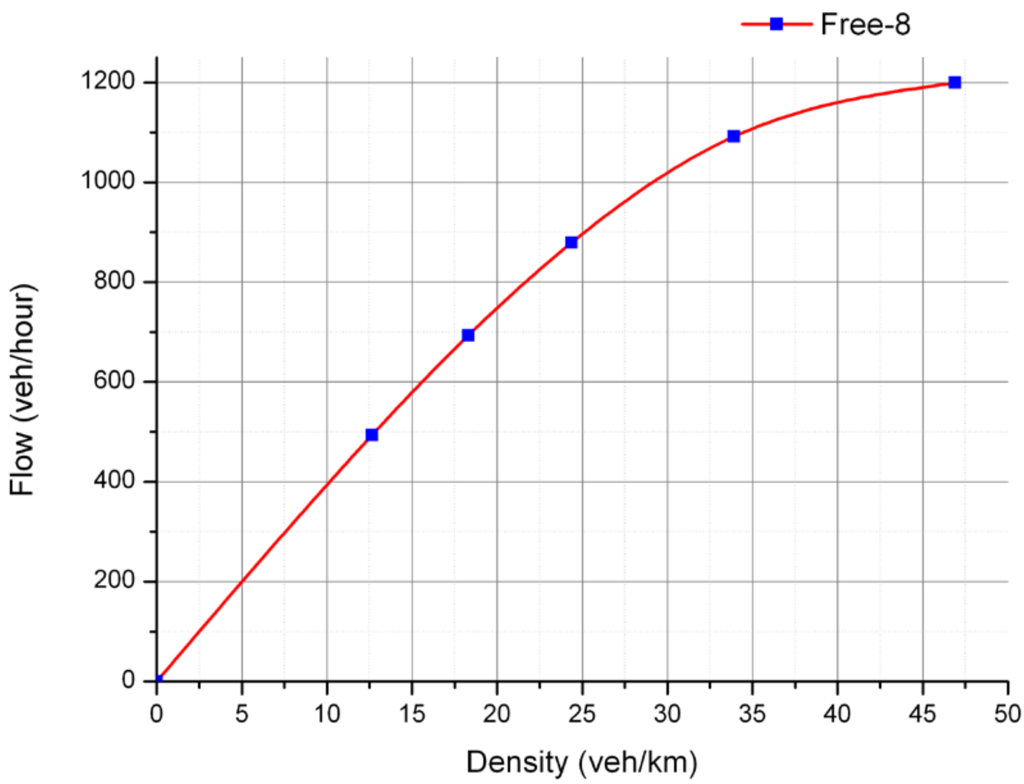


Table 26 Free-8 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	12,65	493,41
700	18,31	692,94
900	24,37	879,02
1100	33,91	1091,67
1300	46,91	1199,24

Figure 21. Free-9 Flow Traffic Diagram.

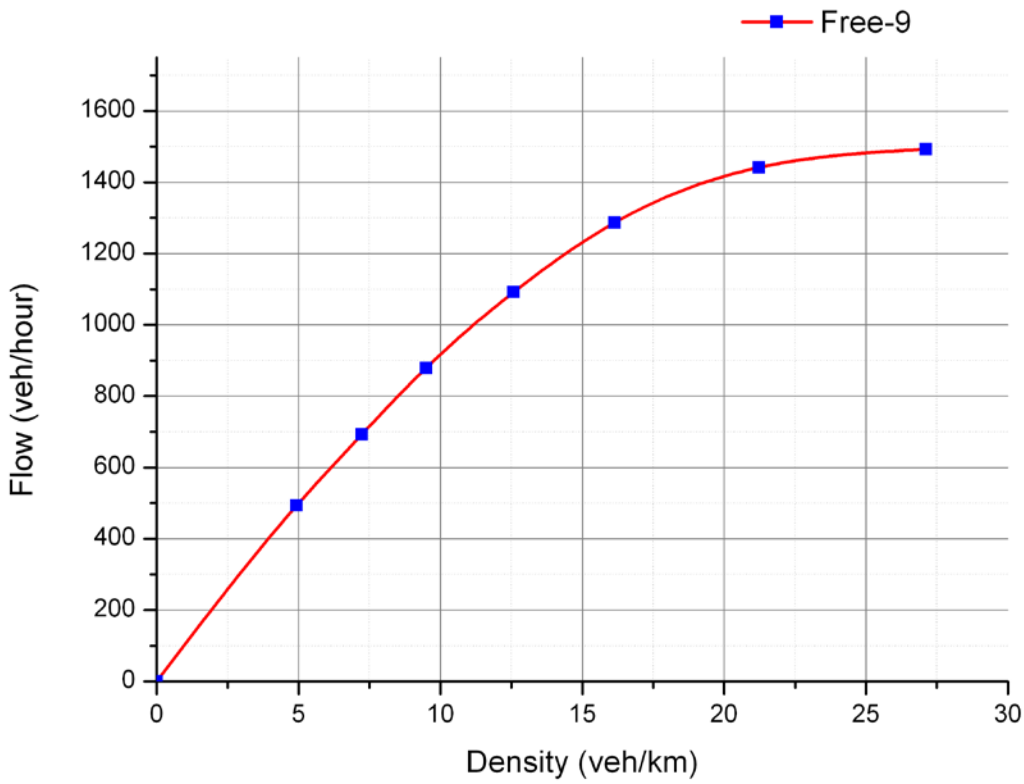


Table 27. Free-9 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	4,93	493,36
700	7,23	692,82
900	9,49	878,79
1100	12,58	1091,75
1300	16,14	1286,86
1500	21,23	1441,71
1700	27,12	1493
1900	27,1	1492,17
1600	27,13	1492,89

Figure 22. Free-10 Flow Traffic Diagram.

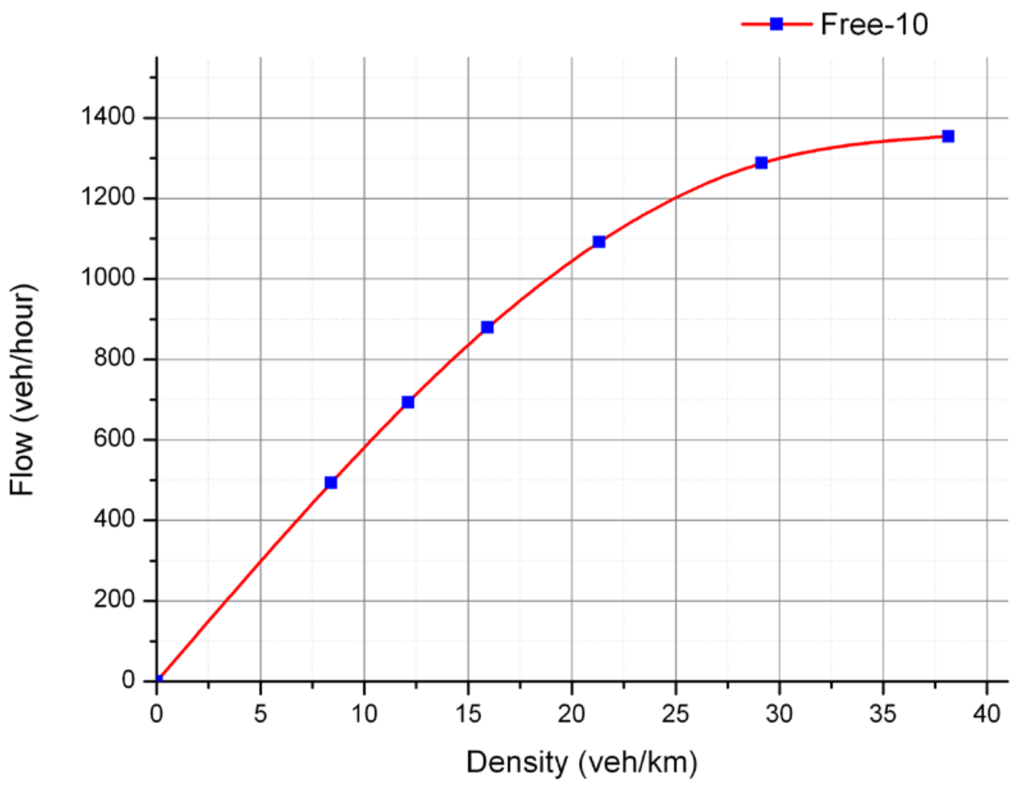


Table 28. Free-10 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,41	493,56
700	12,11	693
900	15,95	878,7
1100	21,33	1091,52
1300	29,14	1287,06
1400	38,14	1354,09
1500	38,18	1355,19

Figure 23. Free-11 Flow Traffic Diagram.

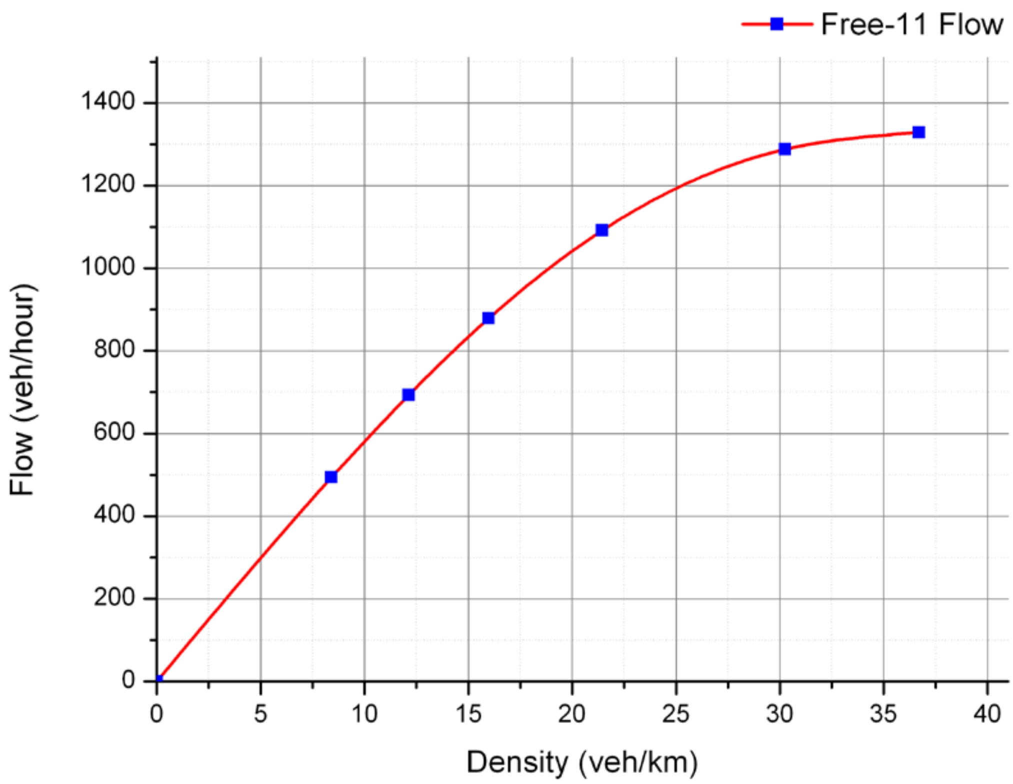


Table 29. Free-11 Flow data.

Demand(veh/h)	Density(veh/km)	Flow(veh/h)
0	0	0
500	8,41	493,74
700	12,13	692,64
900	15,99	878,6
1100	21,45	1091,67
1300	30,25	1287,78
1400	36,71	1328,67
1500	36,79	1326,74

Congested Traffic Flow Diagrams and Data

Figure 24. Jam-1 Traffic Diagram.

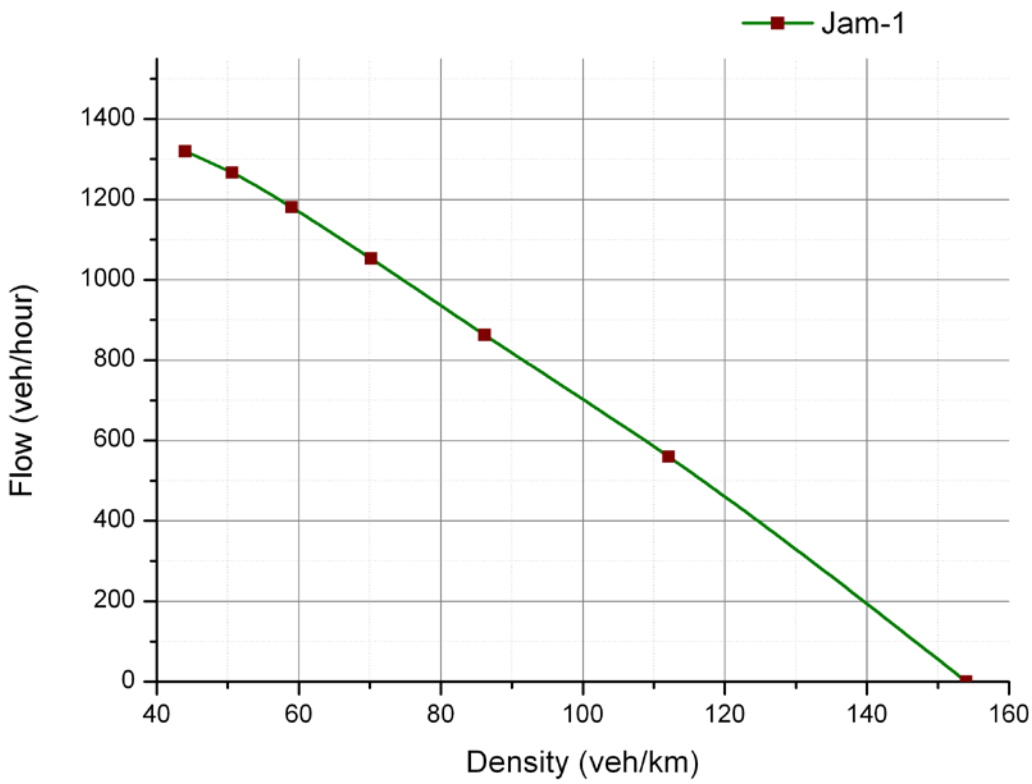


Table 30. Jam-1 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	44	1319
25	50,6	1266
20	59	1180
15	70,2	1052
10	86,2	863
5	112,1	559
0	154	0

Figure 25. Jam-13 Traffic Diagram.

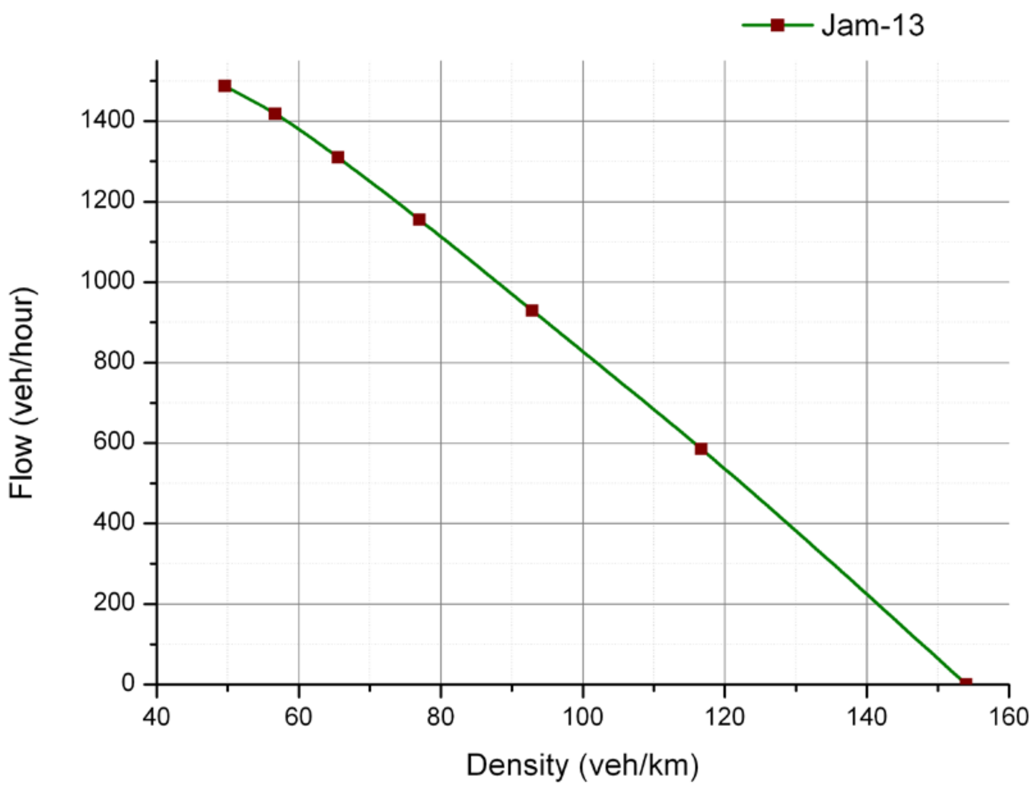


Table 31. Jam-13 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	49,6	1487
25	56,7	1418
20	65,5	1310
15	77	1155
10	92,9	929
5	116,7	585
0	154	0

Figure 26. Jam-17 Traffic Diagram.

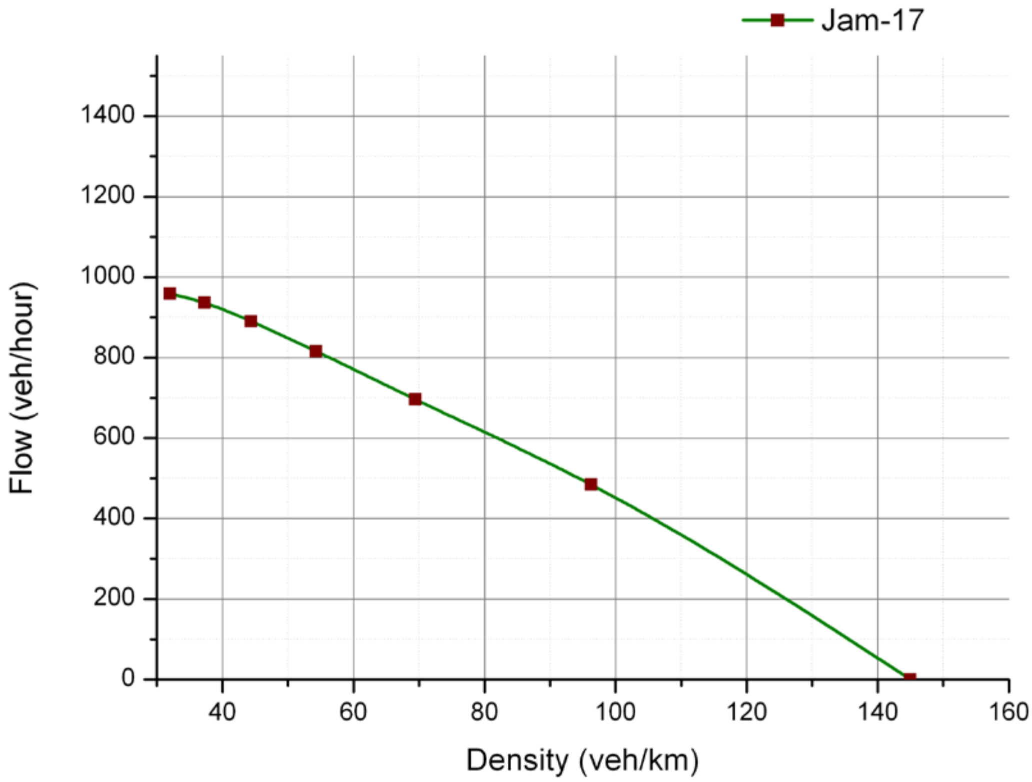


Table 32. Jam-17 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	32	959
25	37,3	936
20	44,4	890
15	54,3	816
10	69,5	696
5	96,3	484
0	145	0

Figure 27. Jam-21 Traffic Diagram.

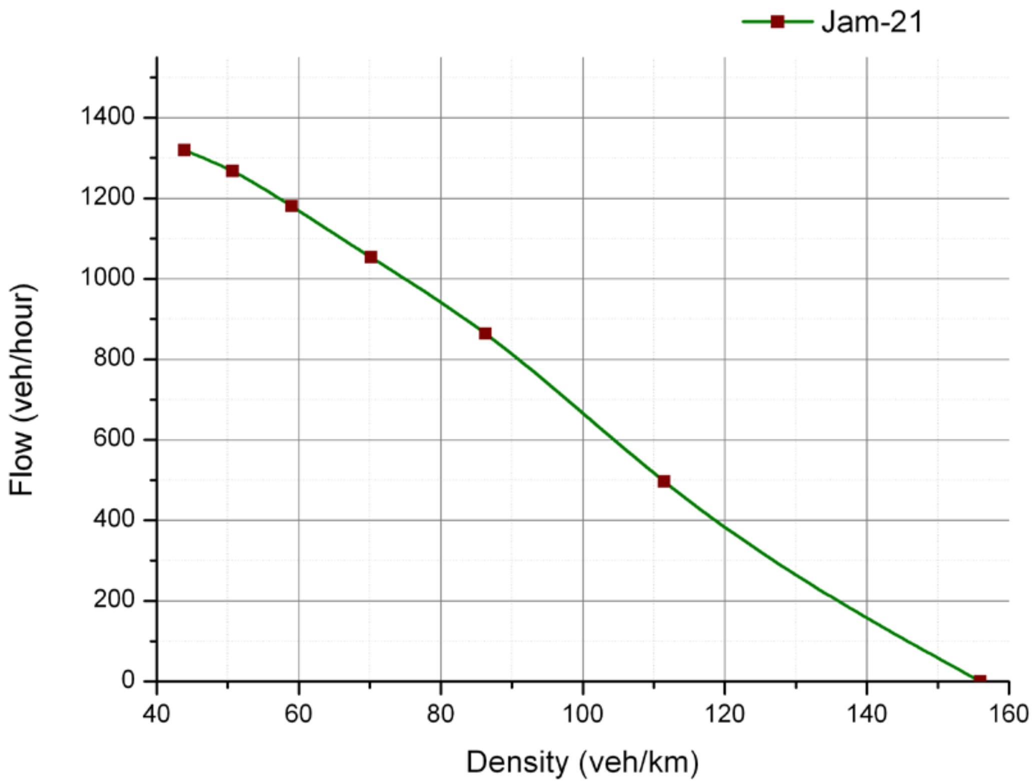


Table 33. Jam-21 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	43,9	1319
25	50,7	1267
20	59	1180
15	70,2	1053
10	86,3	864
5	111,5	496
0	156	0

Figure 28. Jam-25 Traffic Diagram.

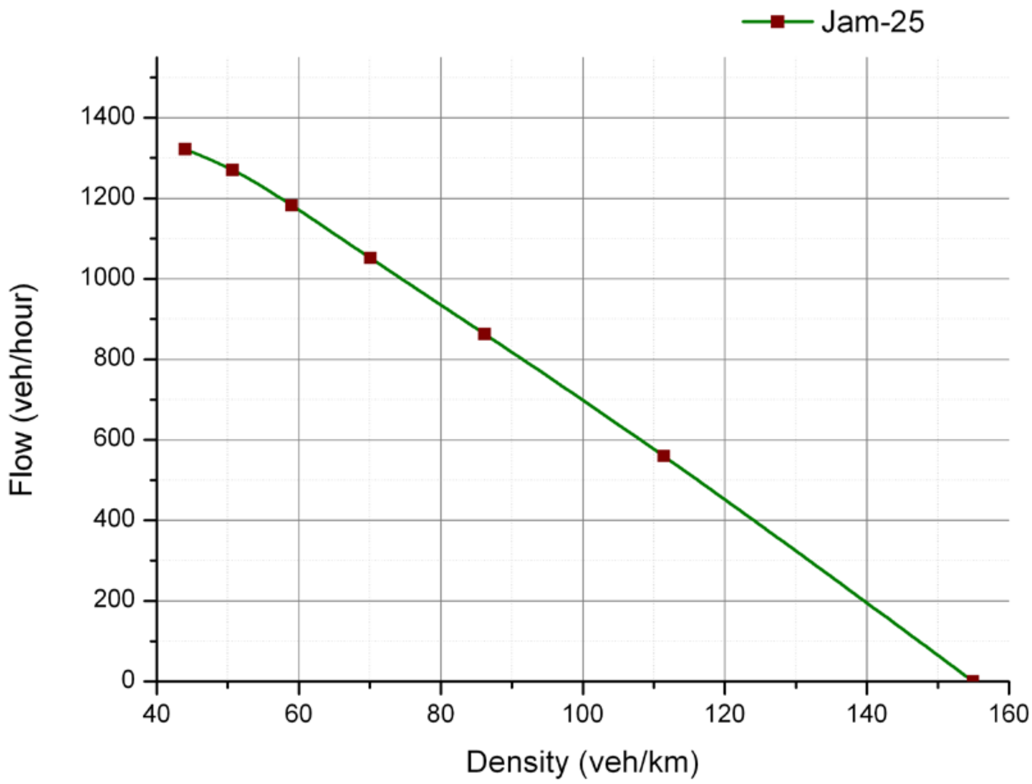


Table 34. Jam-25 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	44	1322
25	50,7	1270
20	59	1182
15	70,1	1051
10	86,2	863
5	111,4	559
0	155	0

Figure 29. Jam-29 Traffic Diagram.

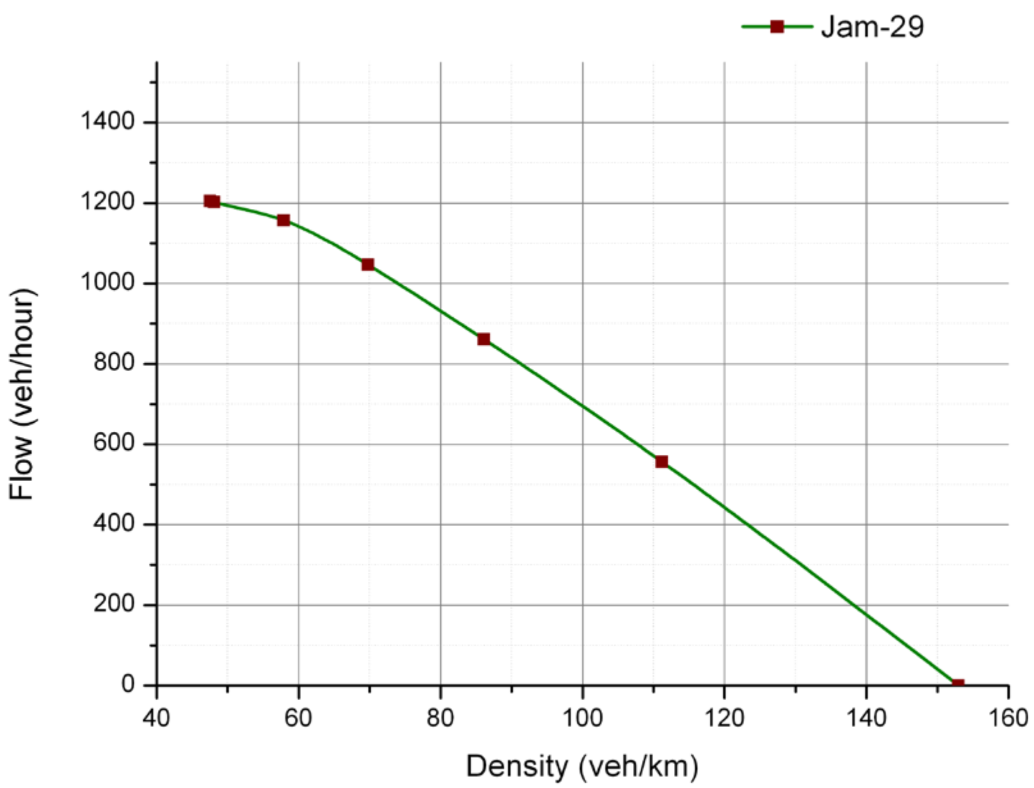


Table 35. Jam-29 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	47,5	1205
25	48,1	1202
20	57,9	1157
15	69,8	1046
10	86,1	861
5	111,2	556
0	153	0

Figure 30. Jam-33 Traffic Diagram.

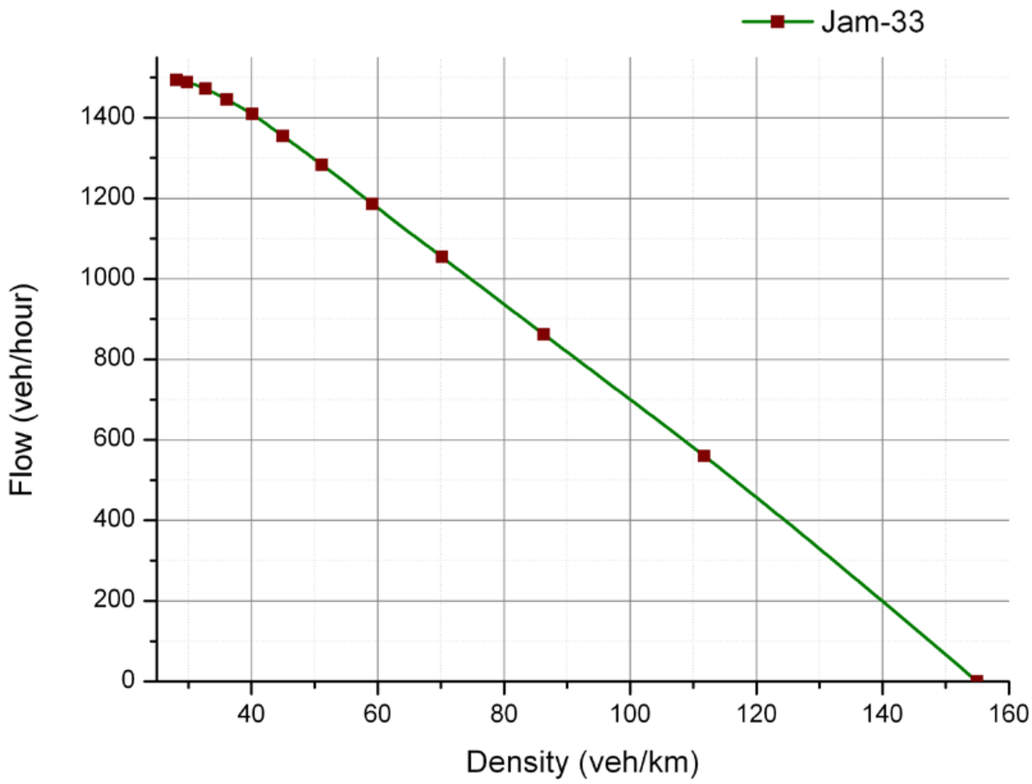


Table 36. Jam-33 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
55	28,1	1493
50	29,8	1488
45	32,7	1472
40	36,1	1445
35	40,1	1409
30	45	1354
25	51,2	1283
20	59,2	1186
15	70,2	1054
10	86,3	862
5	111,8	559
0	155	0

Figure 31. Jam-37 Traffic Diagram.

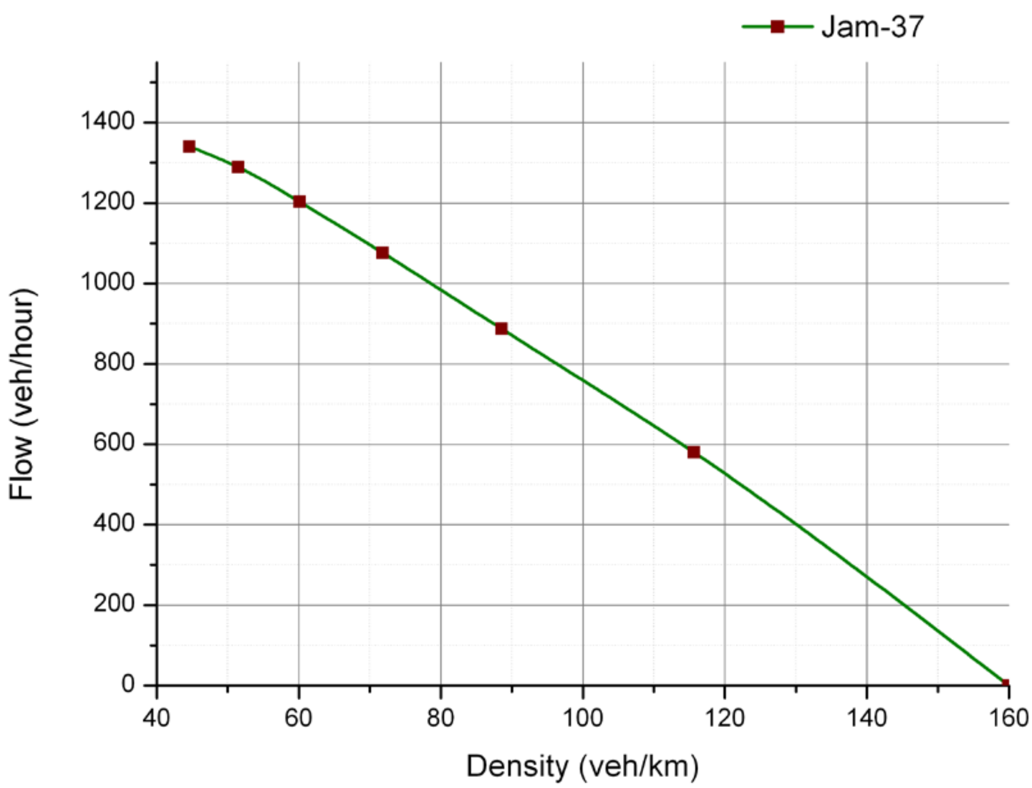


Table 37. Jam-37 Flow Data.

Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	44,6	1340
25	51,5	1289
20	60,1	1203
15	71,8	1076
10	88,6	887
5	115,7	579
0	160	0

Figure 32. Jam-41 Traffic Diagram.

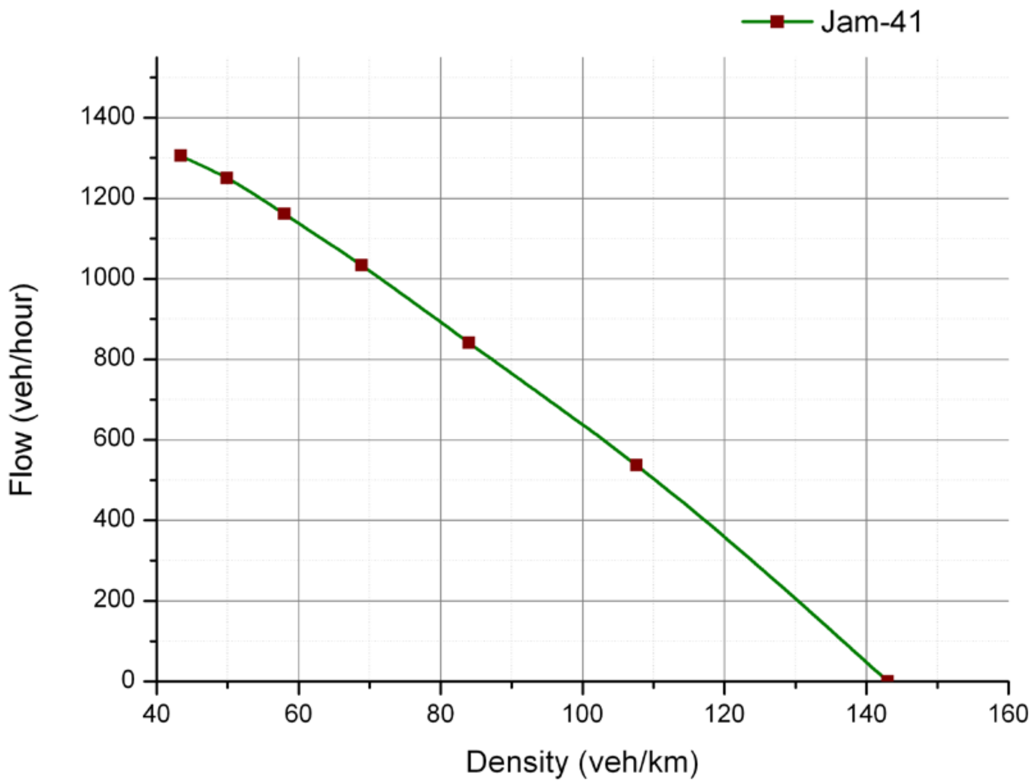
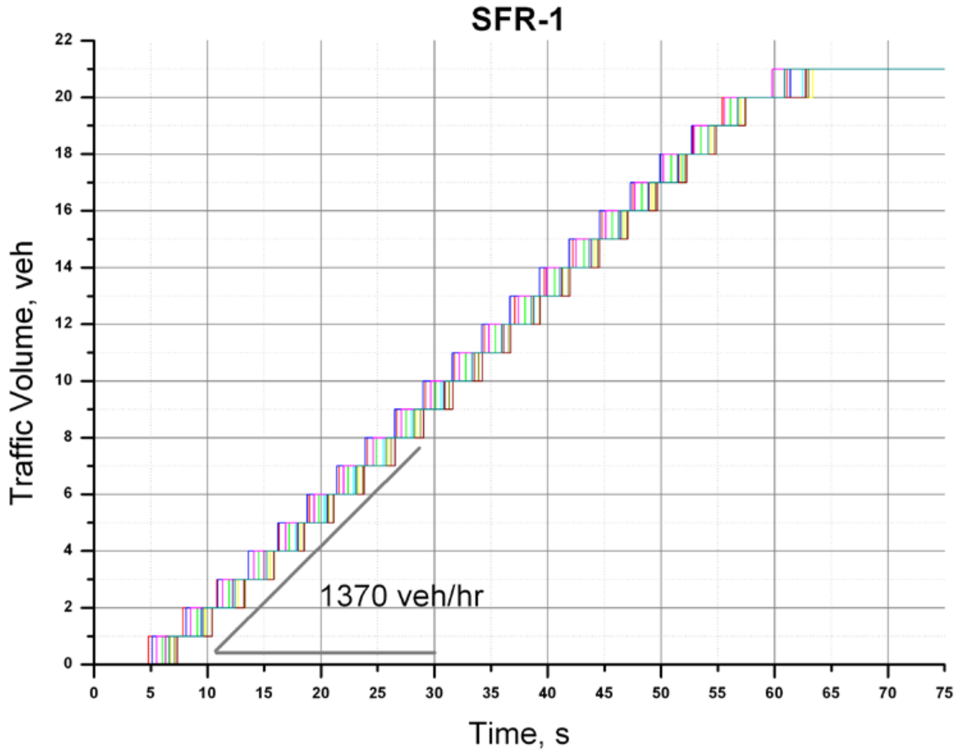
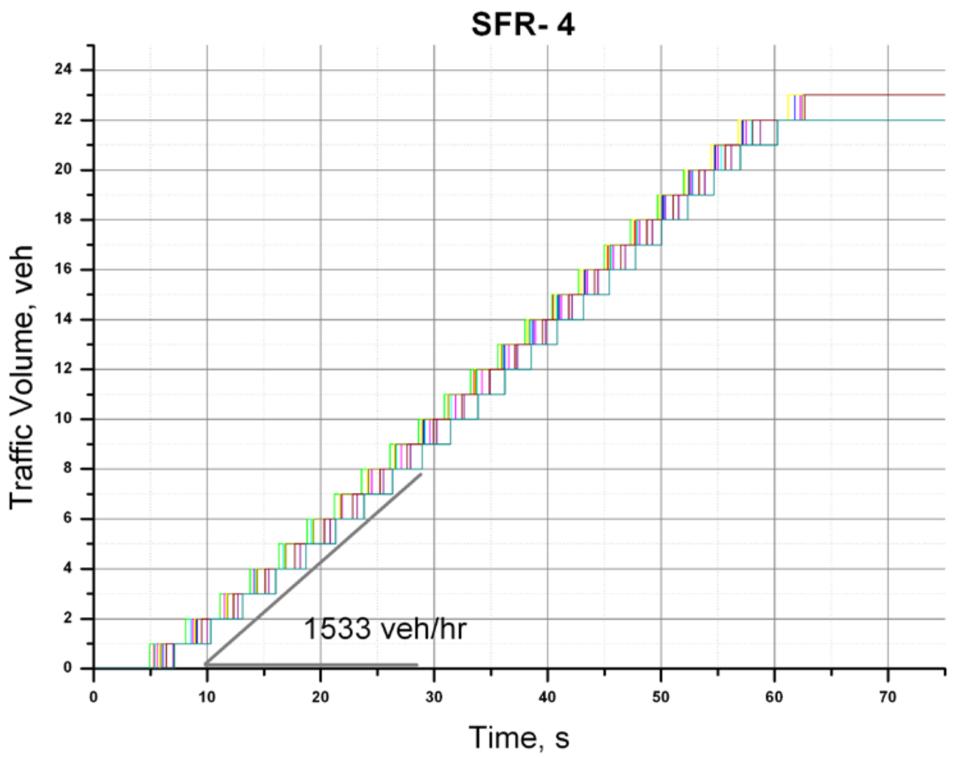


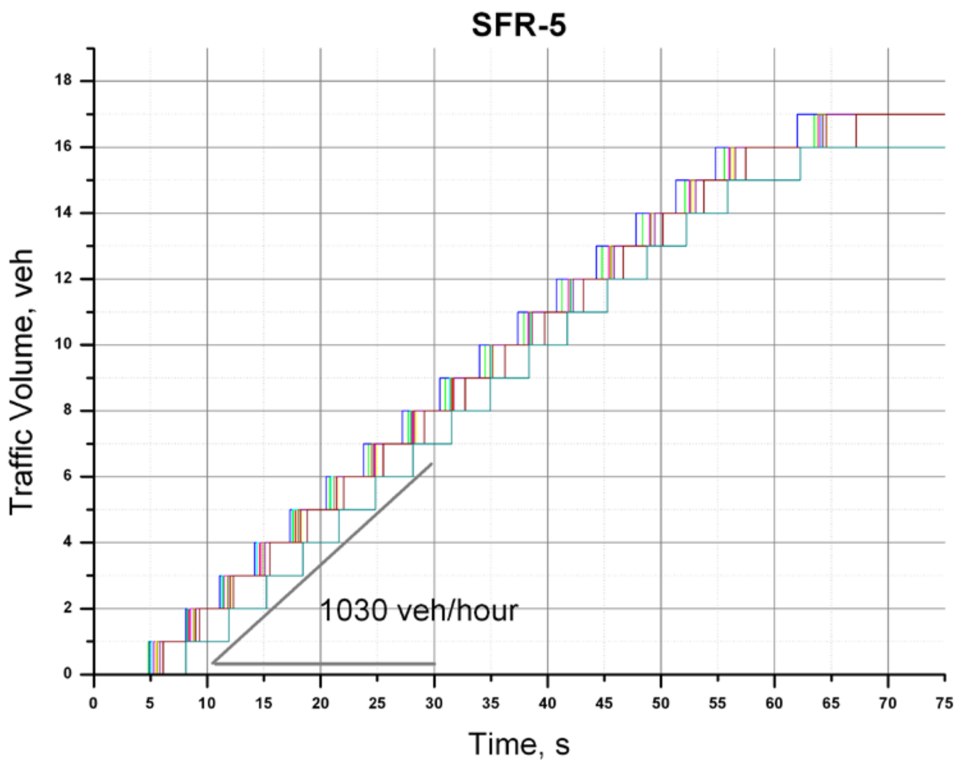
Table 38. Jam-41 Flow Data.

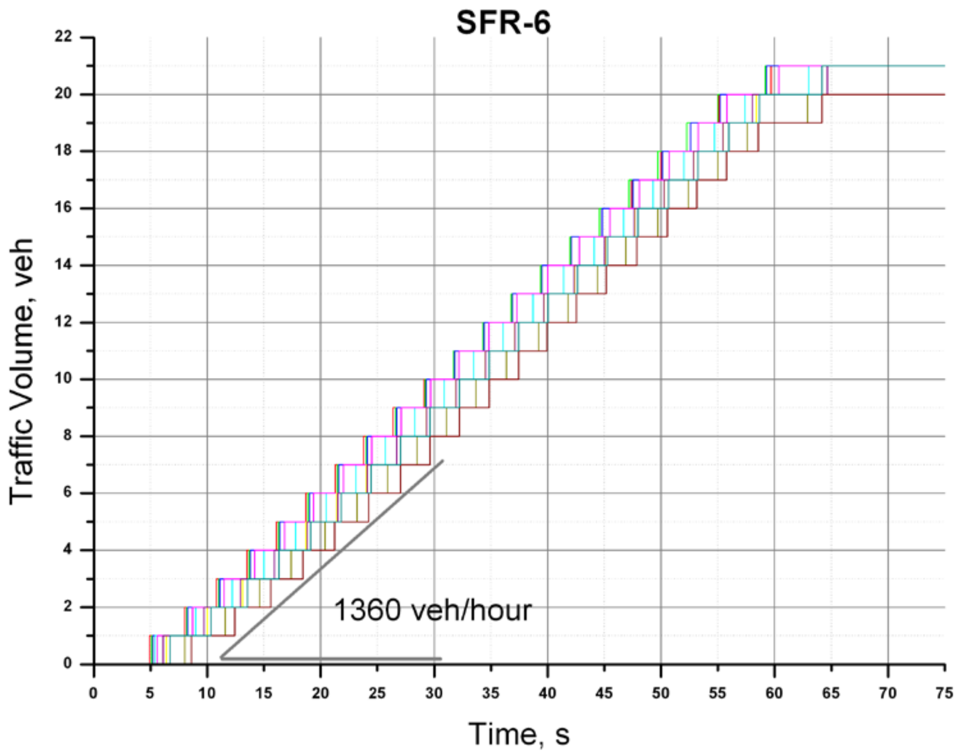
Speed Limit (km/h)	Density (veh/km)	Flow (veh/h)
30	43,4	1305
25	49,9	1250
20	58	1161
15	68,9	1033
10	84	841
5	107,6	537
0	143	0

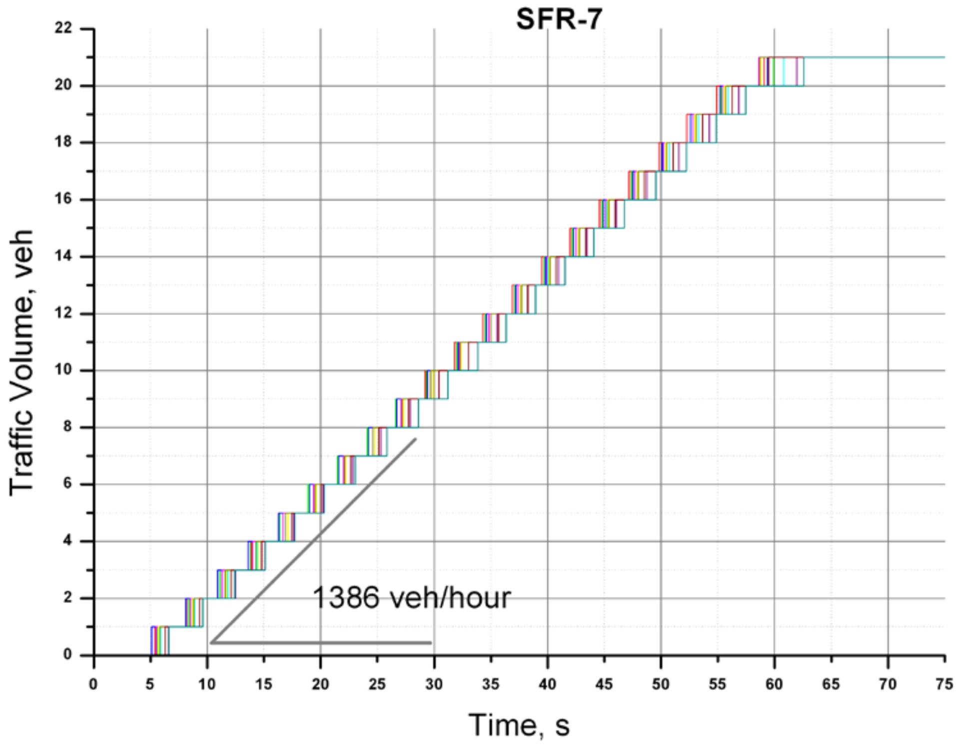
Saturation Flow Rate Verification Graphs

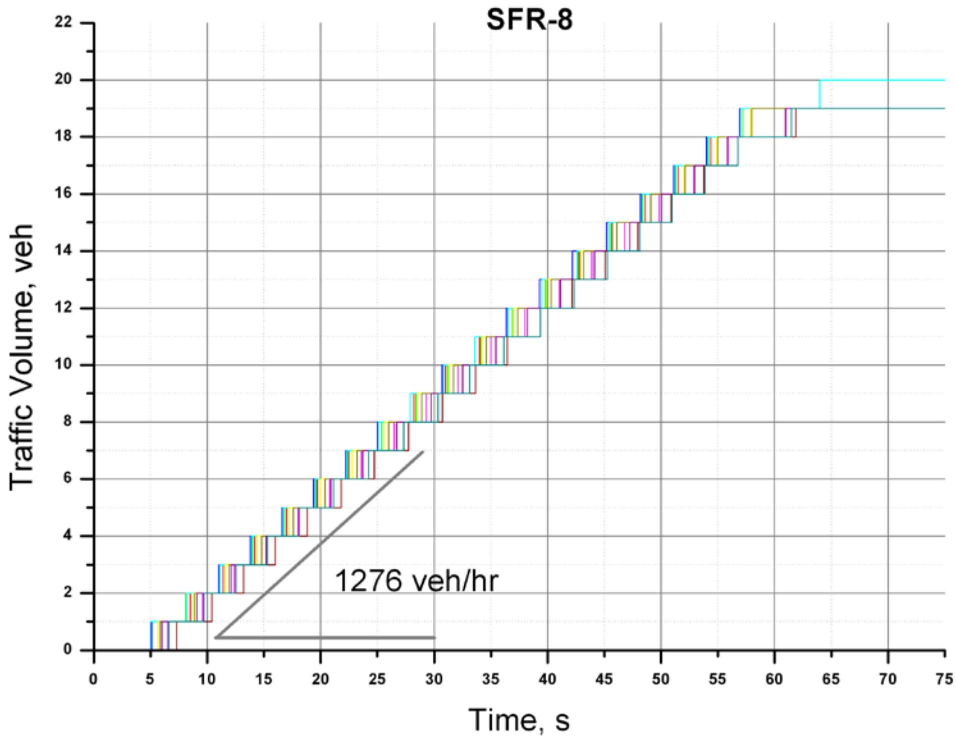





SFR-5
Traffic Volume, veh
Time, s
1030 veh/hour
0
2
4
6
8
10
12
14
16
18
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-6
Traffic Volume, veh
Time, s
1360 veh/hour
0
2
4
6
8
10
12
14
16
18
20
22
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-7
Traffic Volume, veh
Time, s
1386 veh/hour
0
2
4
6
8
10
12
14
16
18
20
22
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-8
Traffic Volume, veh
Time, s
1276 veh/hr
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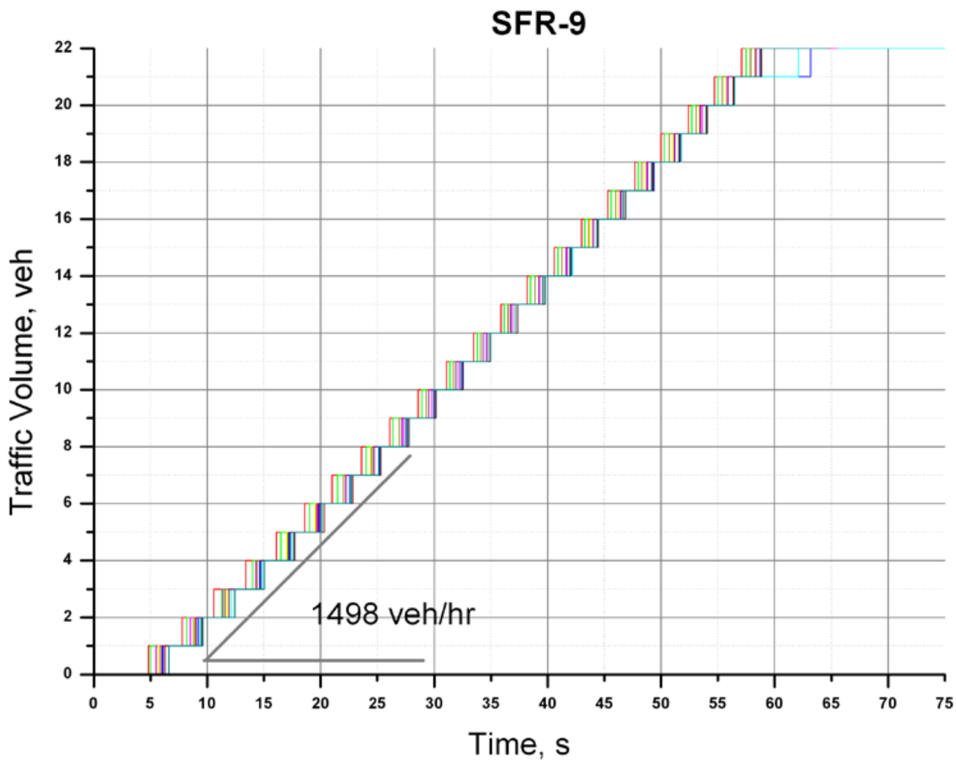
2
4
6
8
10
12
14
16
18
20
22
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-9
Traffic Volume, veh
Time, s
1498 veh/hr
0
2
4
6
8
10
12
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16
18
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22
0
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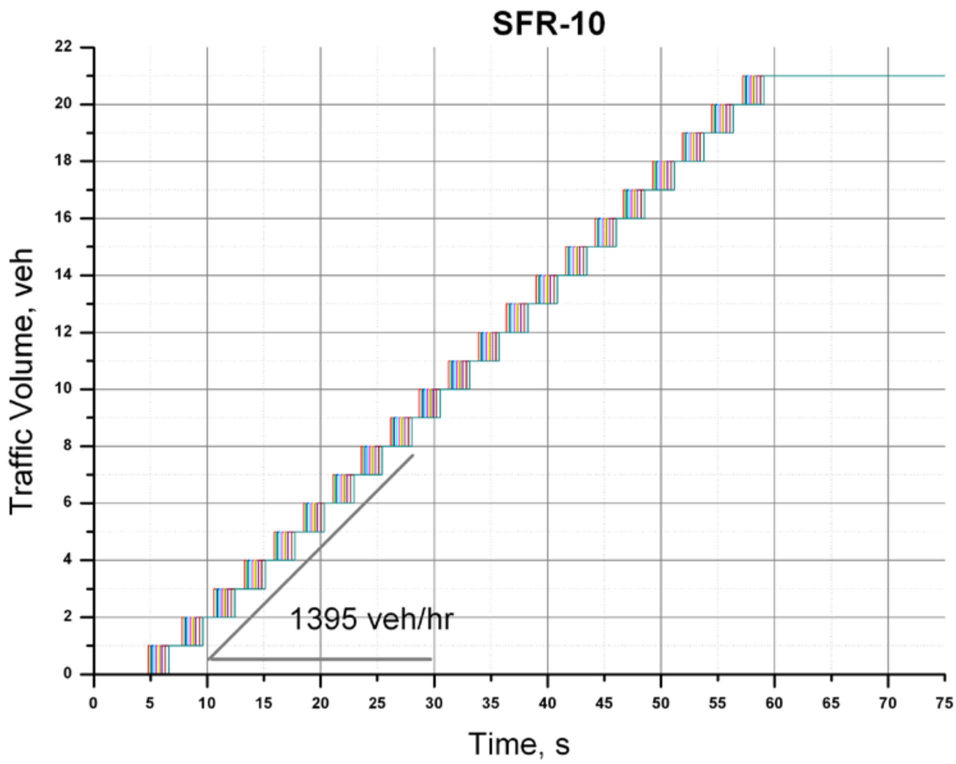
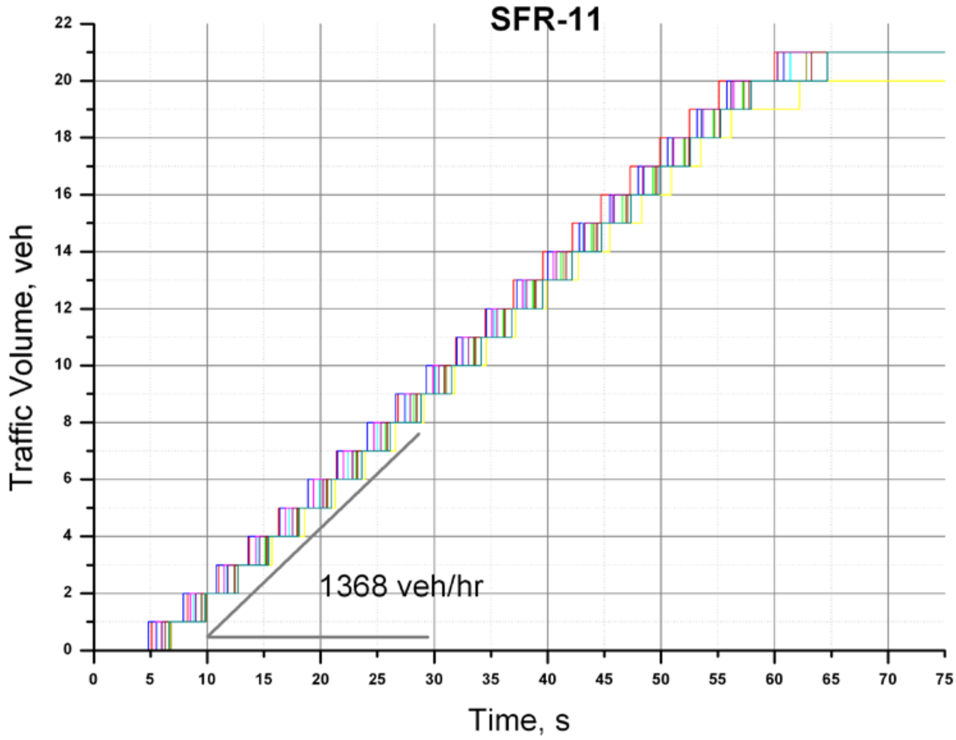
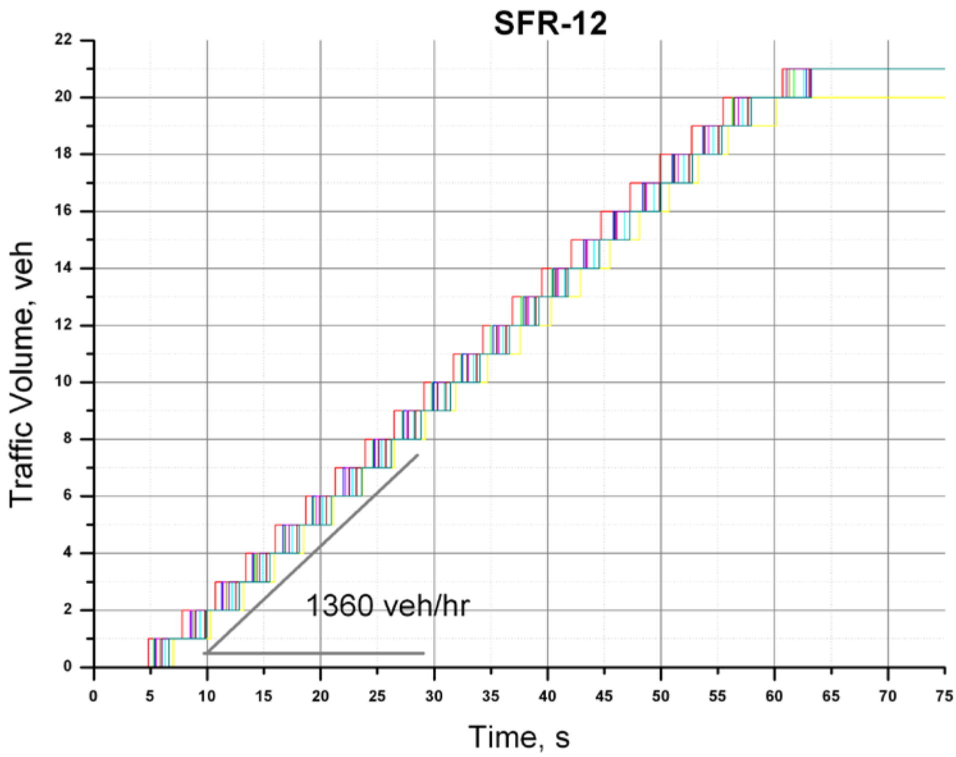
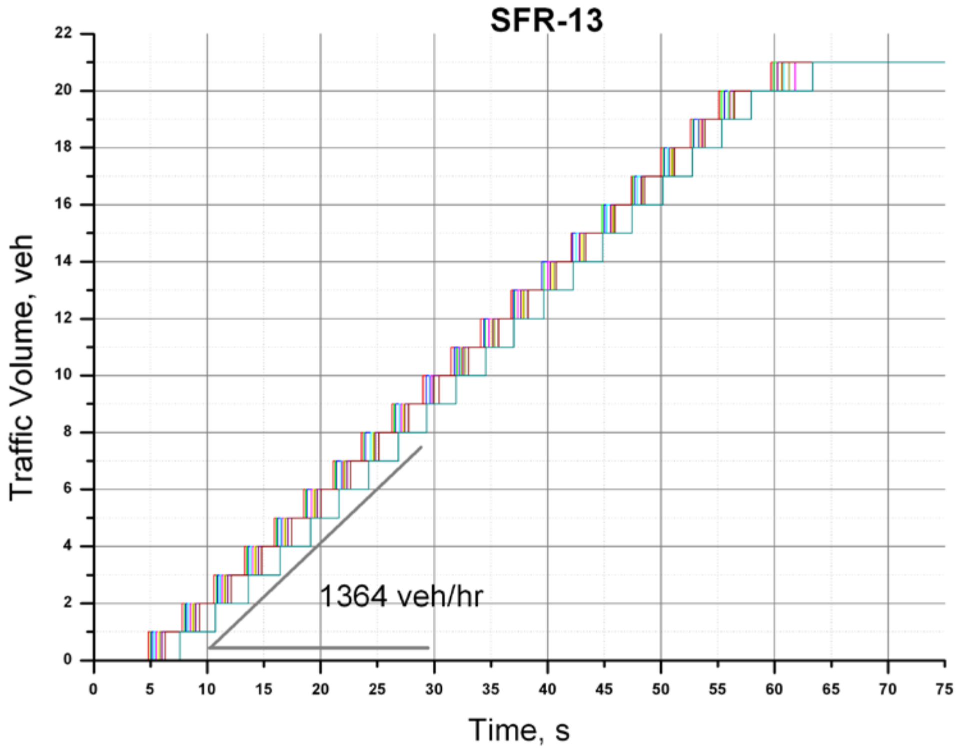
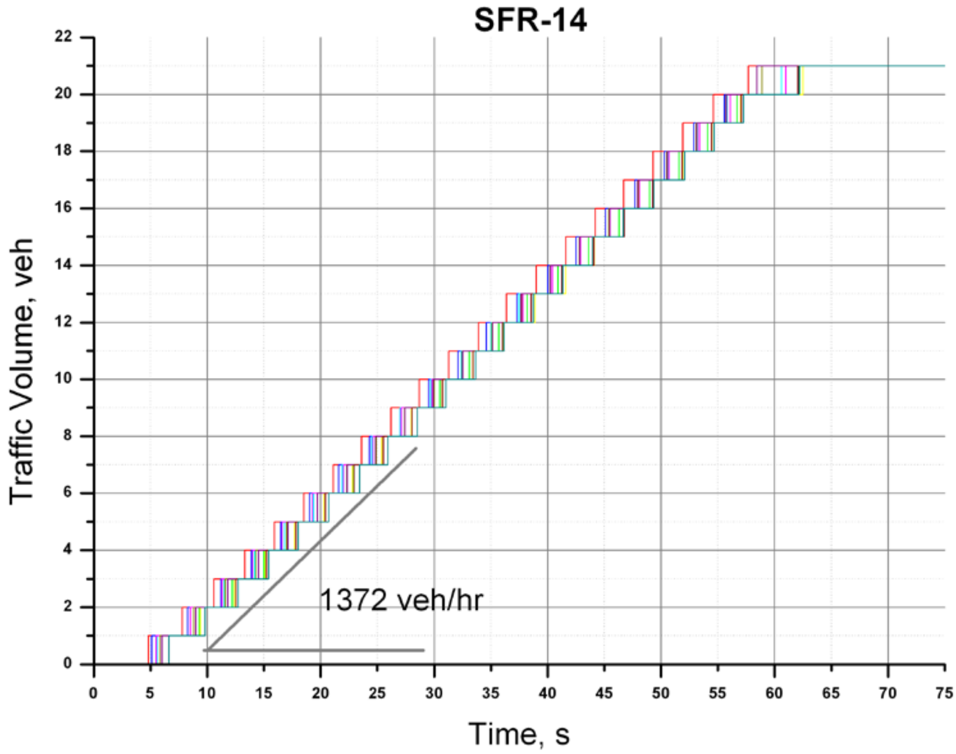
20
25
30
35
40
45
50
55
60
65
70
75


SFR-10
Traffic Volume, veh
Time, s
1395 veh/hr
0
2
4
6
8
10
12
14
16
18
20
22
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-11
Traffic Volume, veh
Time, s
1368 veh/hr


SFR-12
Traffic Volume, veh
Time, s
1360 veh/hr


SFR-13
Traffic Volume, veh
Time, s
1364 veh/hr
0
2
4
6
8
10
12
14
16
18
20
22
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-14
Traffic Volume, veh
Time, s
1372 veh/hr
0
2
4
6
8
10
12
14
16
18
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22
0
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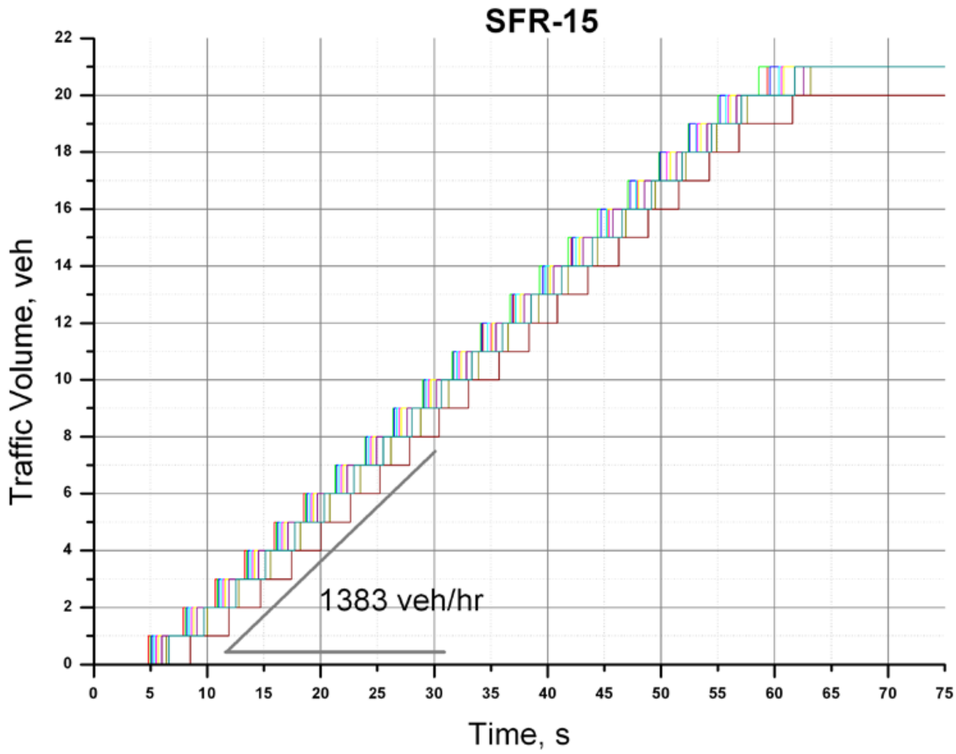
10
15
20
25
30
35
40
45
50
55
60
65
70
75


SFR-15
Traffic Volume, veh
Time, s
1383 veh/hr
0
2
4
6
8
10
12
14
16
18
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22
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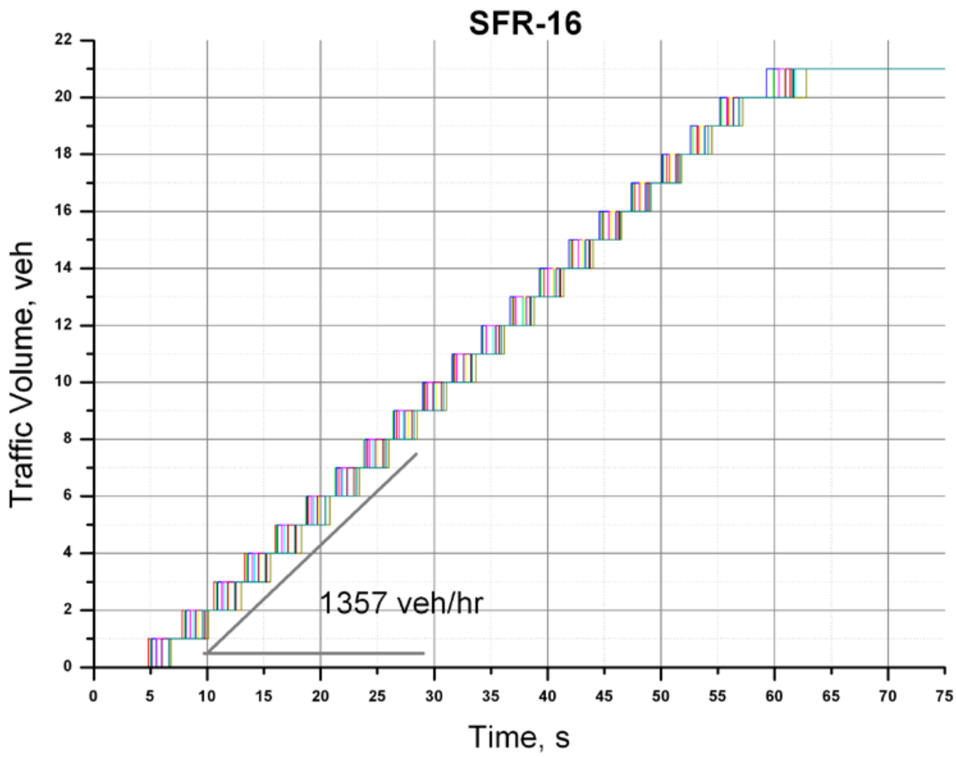
25
30
35
40
45
50
55
60
65
70
75


SFR-16
Traffic Volume, veh
Time, s
1357 veh/hr
0
2
4
6
8
10
12
14
16
18
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22
0
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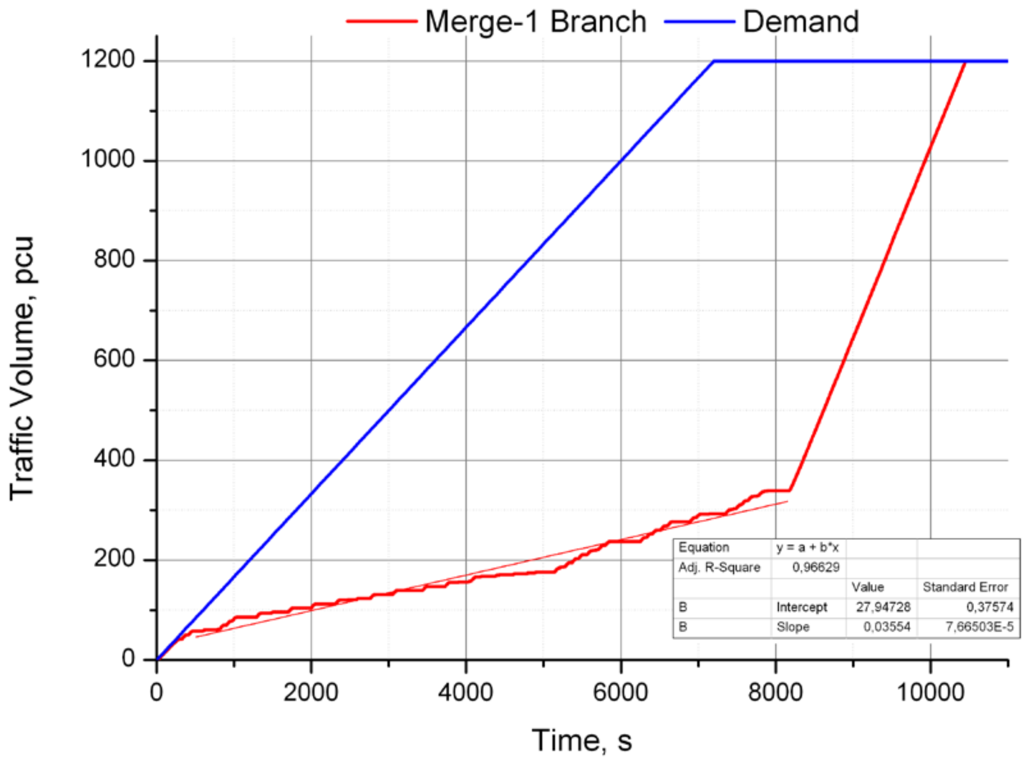
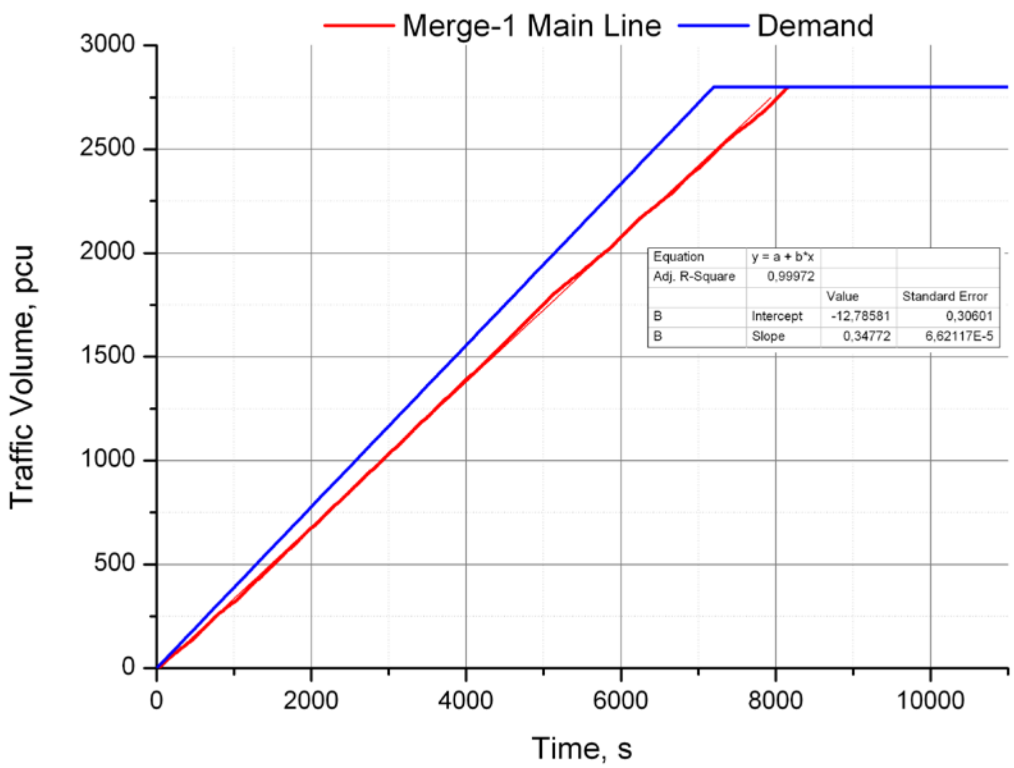
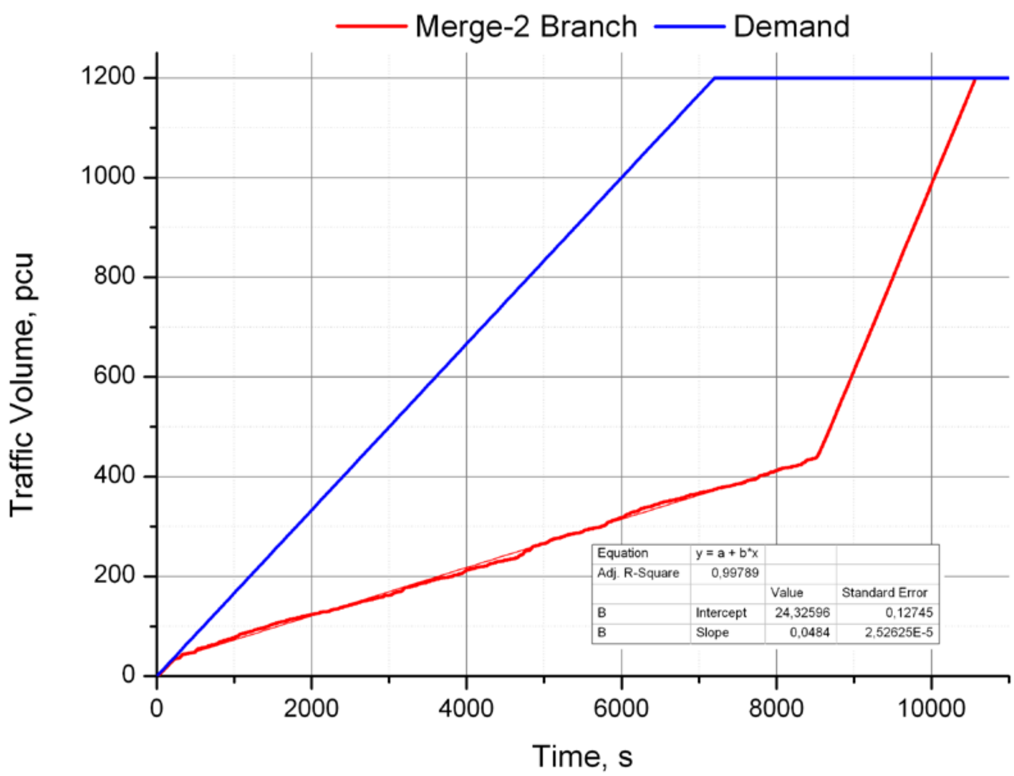
15
20
25
30
35
40
45
50
55
60
65
70
75

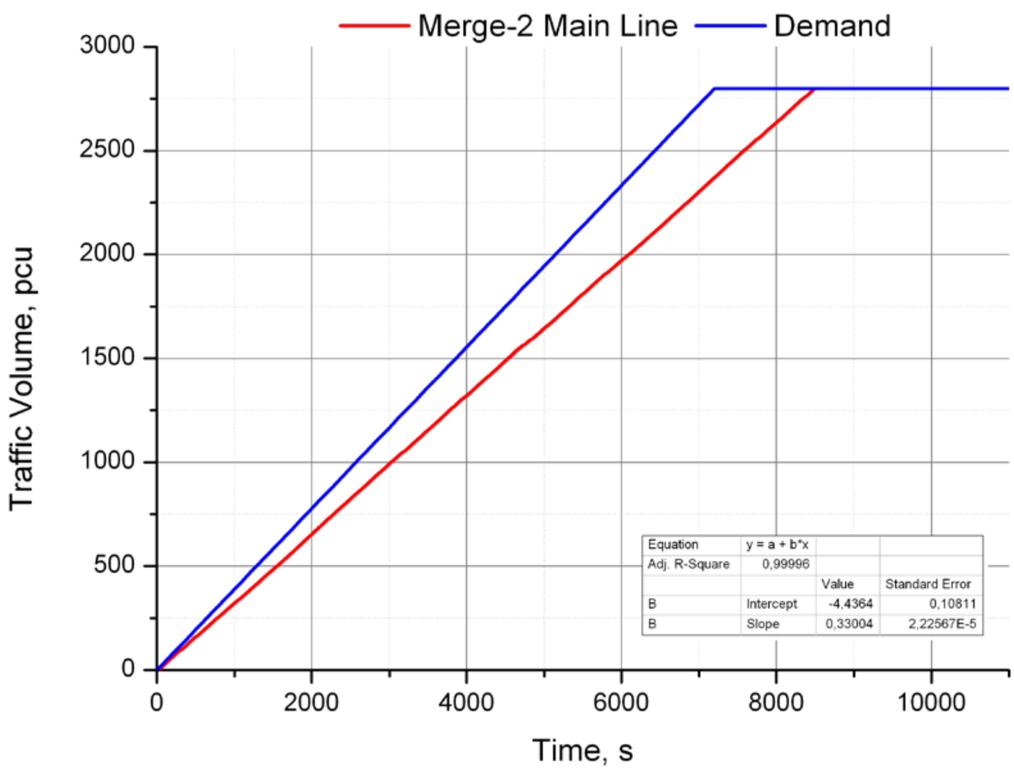
Merge Ratio Verification Graphs



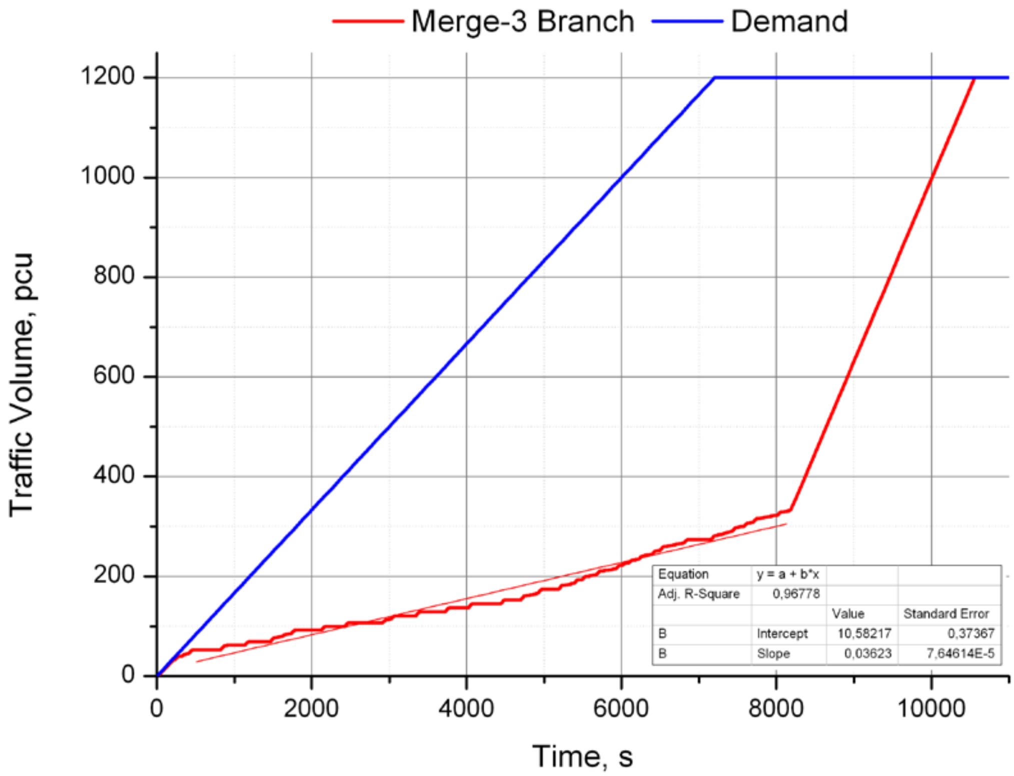




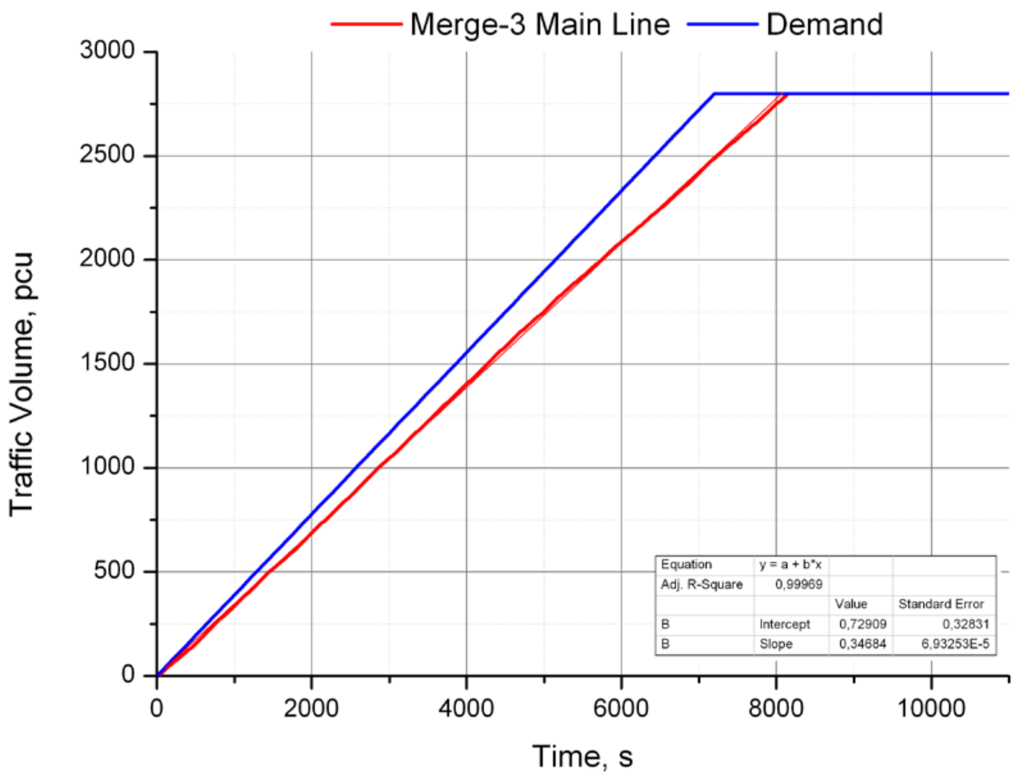
Merge-2 Branch
Demand
Traffic Volume, pcu
Time, s
0
200
400
600
800
1000
1200
0
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,99789
Value
Standard Error
B
Intercept
24,32596
0,12745
B
Slope
0,0484
2,52625E-5



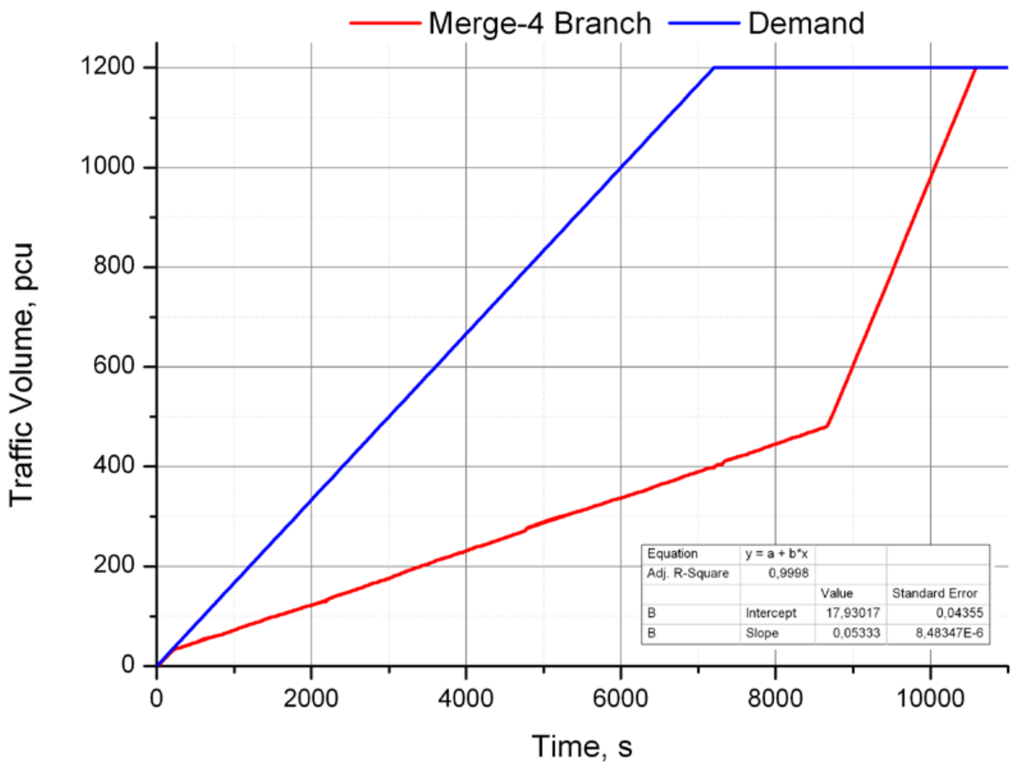
Merge-2 Main Line
Demand
Traffic Volume, pcu
Time, s
0
500
1000
1500
2000
2500
3000
0
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,99996
Value
Standard Error
B
Intercept
-4,4364
0,10811
B
Slope
0,33004
2,22567E-5



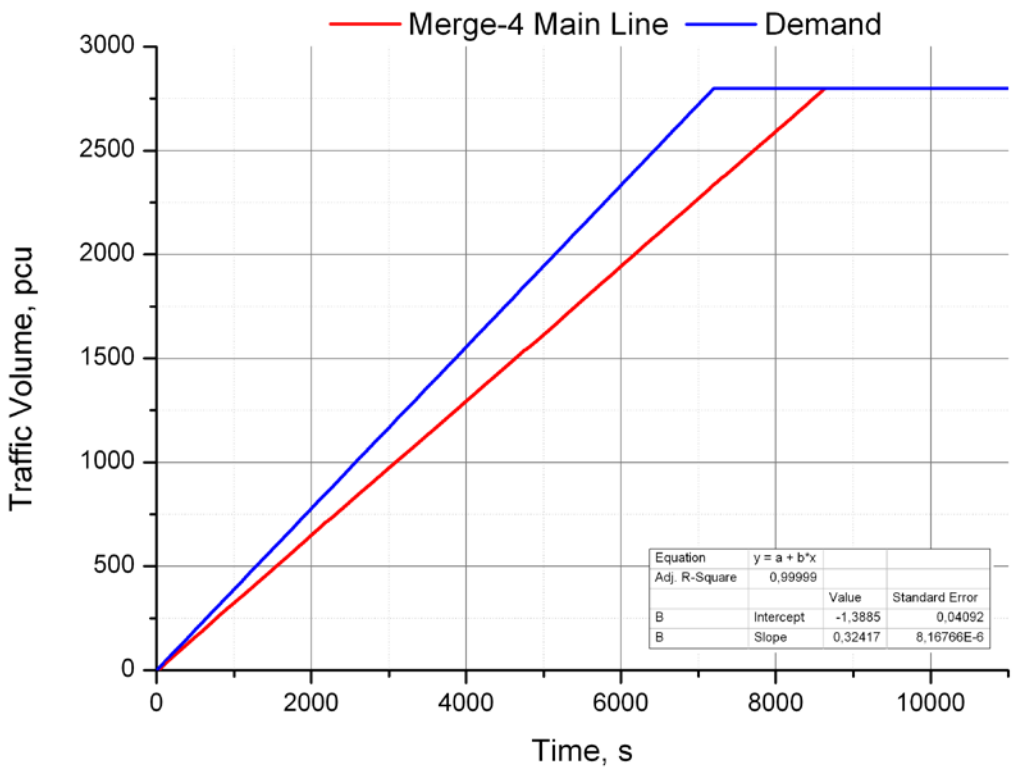
Merge-3 Branch
Demand
Traffic Volume, pcu
Time, s
Equation
y = a + b*x
Adj. R-Square
0,96778
Value
Standard Error
B
Intercept
10,58217
0,37367
B
Slope
0,03623
7,64614E-5



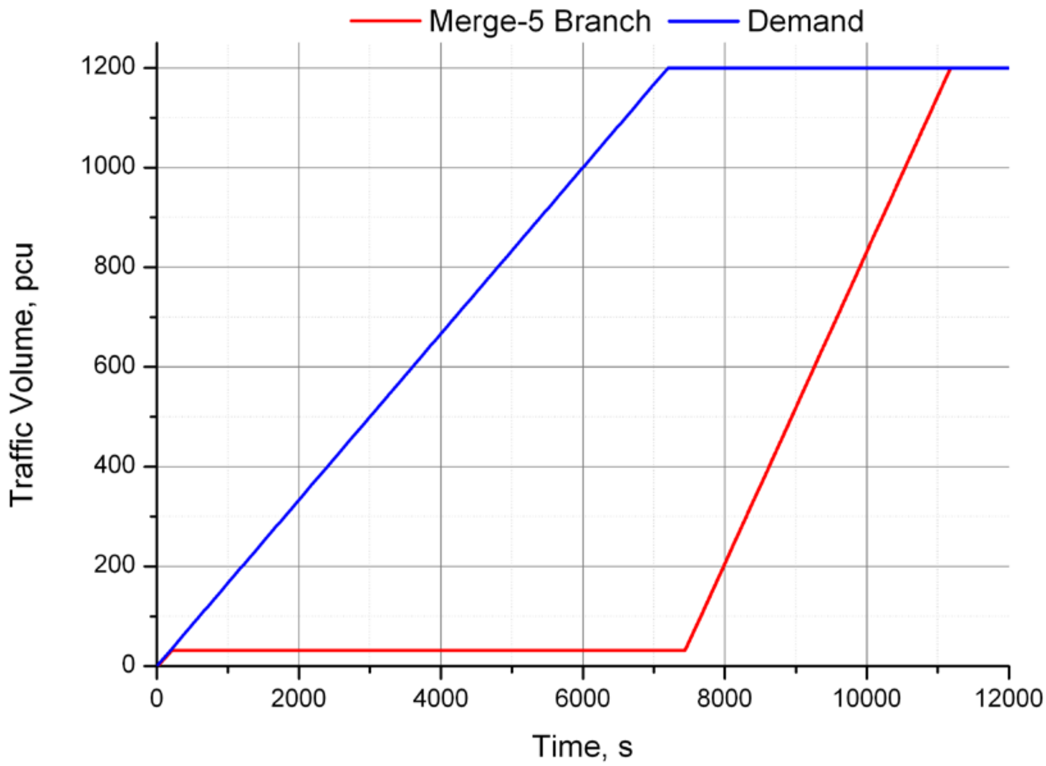
Merge-3 Main Line
Demand
Traffic Volume, pcu
Time, s
Equation
y = a + b*x
Adj. R-Square
0,99969
Value
Standard Error
B
Intercept
0,72909
0,32831
B
Slope
0,34684
6,93253E-5

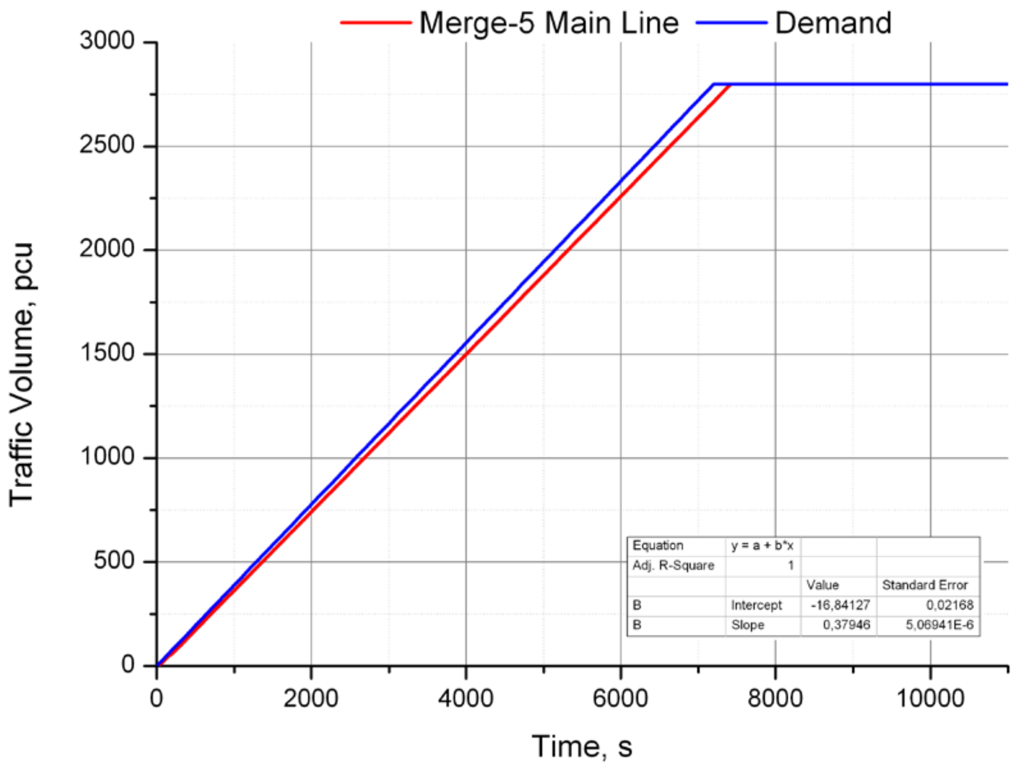


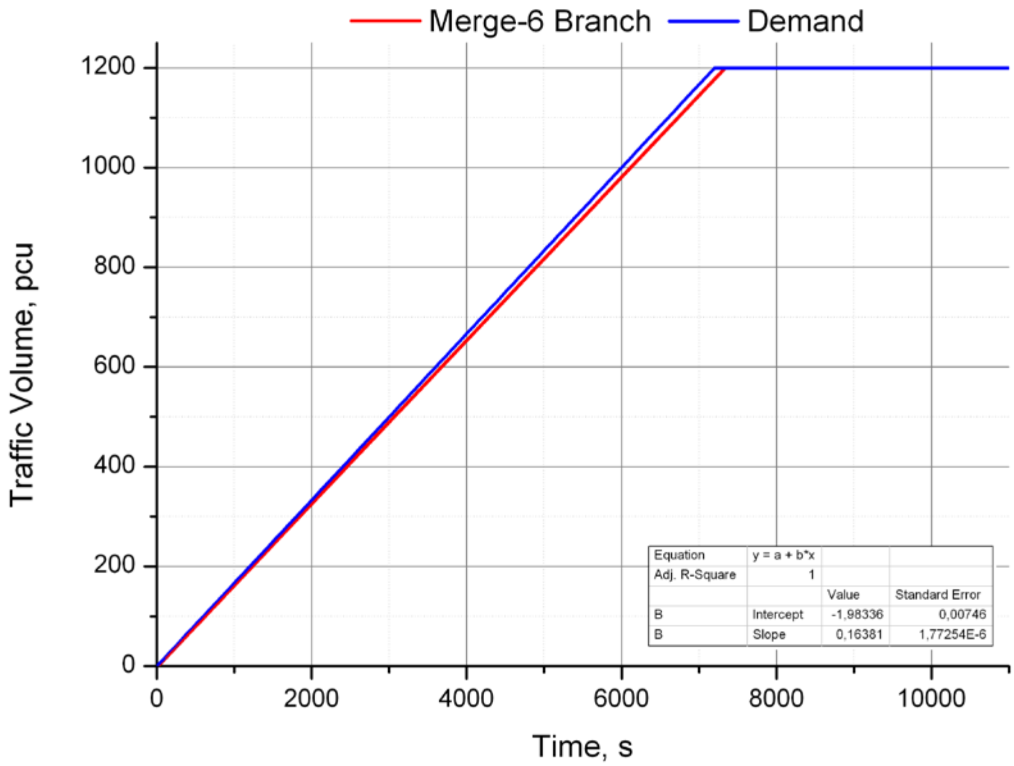
Merge-4 Branch
Demand
Traffic Volume, pcu
Time, s
Equation | y = a + b*x
Adj. R-Square | 0,9998
Value | Standard Error
B | Intercept | 17,93017 | 0,04355
B | Slope | 0,05333 | 8,48347E-6



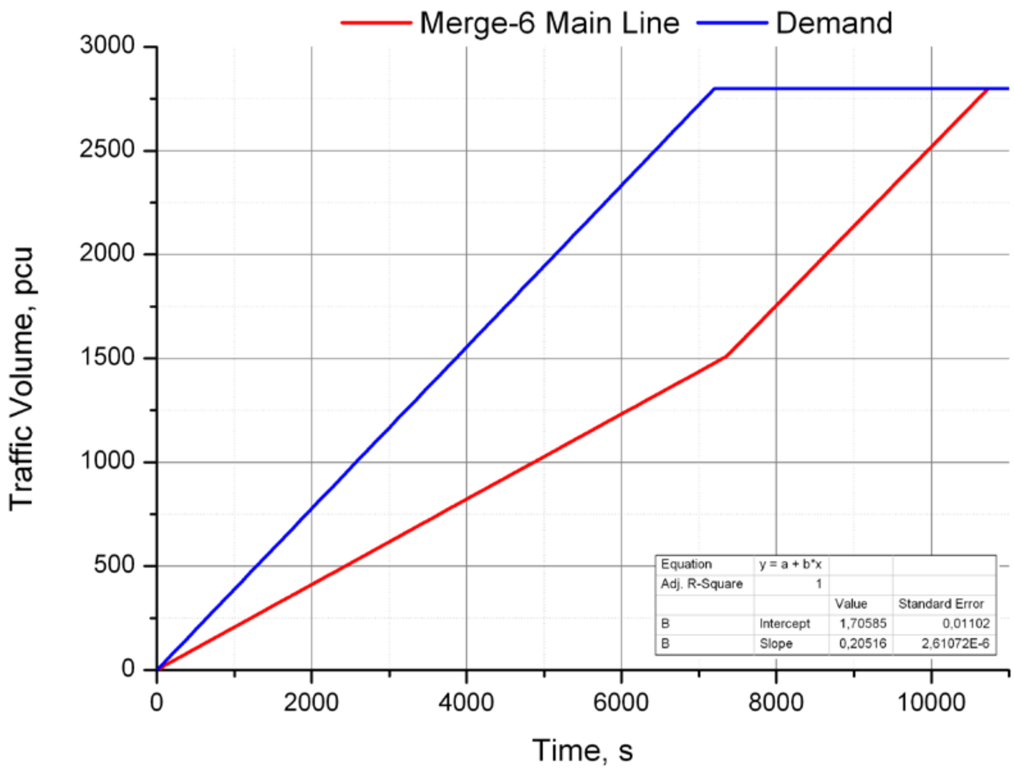
Merge-4 Main Line
Demand
Traffic Volume, pcu
Time, s
Equation | y = a + b*x
Adj. R-Square | 0,99999
Value | Standard Error
B | Intercept | -1,3885 | 0,04092
B | Slope | 0,32417 | 8,16766E-6


Merge-5 Branch
Demand
Traffic Volume, pcu
Time, s
0
200
400
600
800
1000
1200
0
2000
4000
6000
8000
10000
12000

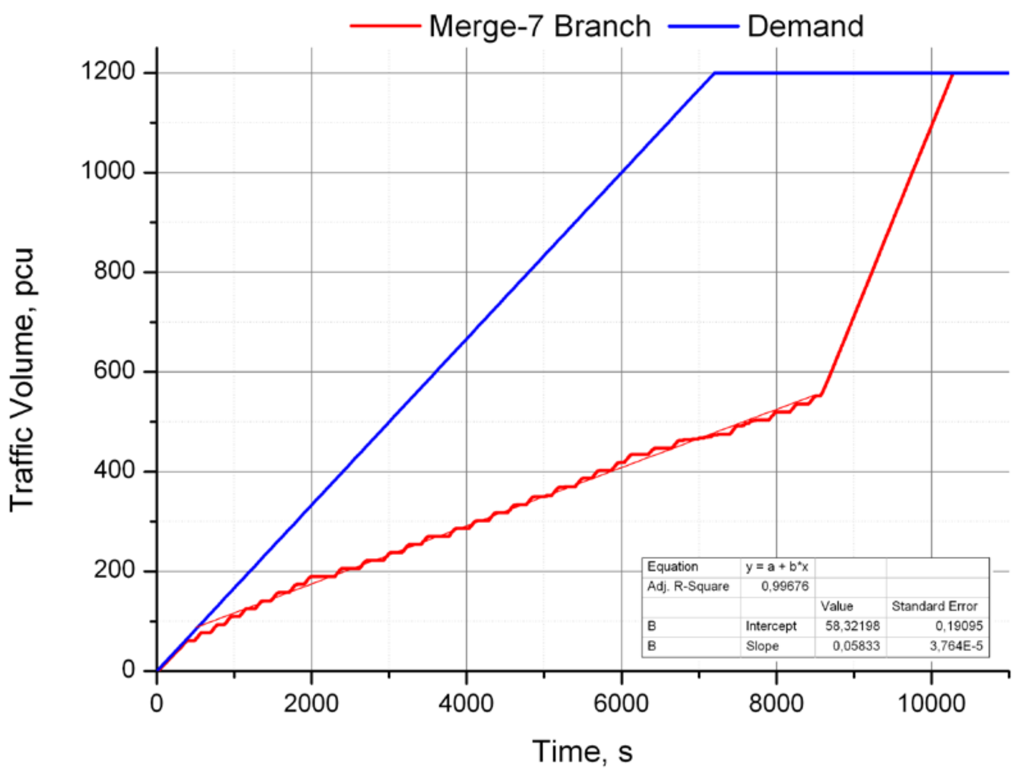

Merge-5 Main Line
Demand
Traffic Volume, pcu
Time, s
0
500
1000
1500
2000
2500
3000
0
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
1
Value
Standard Error
B
Intercept
-16,84127
0,02168
B
Slope
0,37946
5,06941E-6



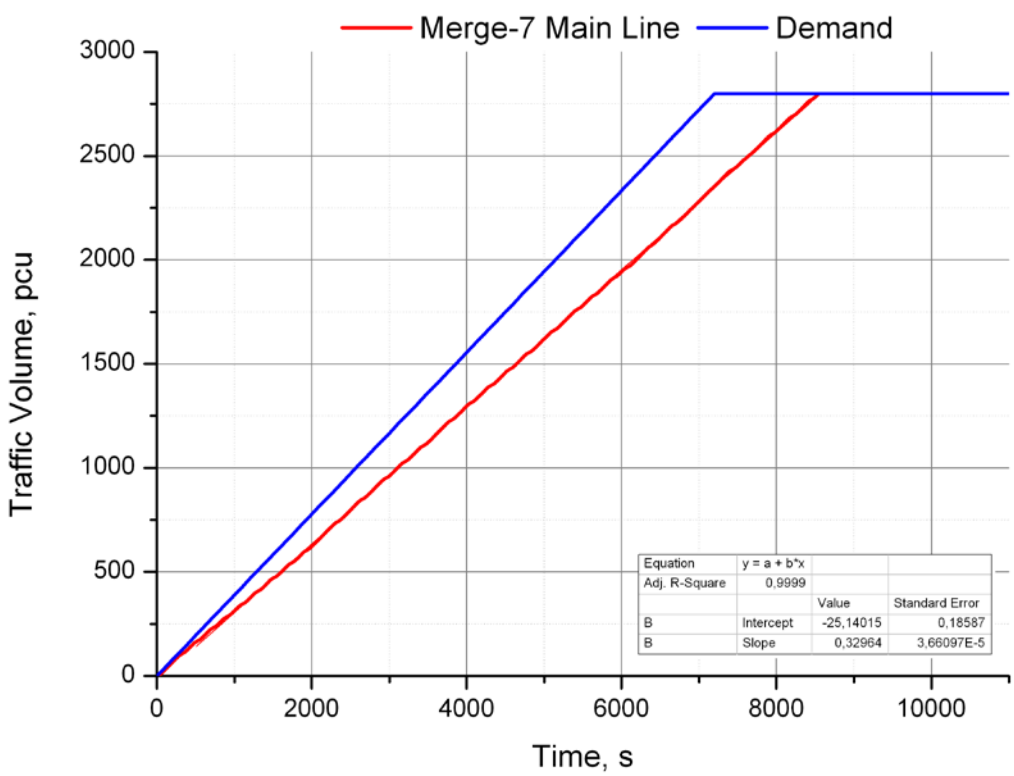
Merge-6 Branch
Demand
Traffic Volume, pcu
0
200
400
600
800
1000
1200
Time, s
0
2000
4000
6000
8000
10000
Equation | y = a + b*x
Adj. R-Square | 1
Value | Standard Error
B | Intercept | -1,98336 | 0,00746
B | Slope | 0,16381 | 1,77254E-6



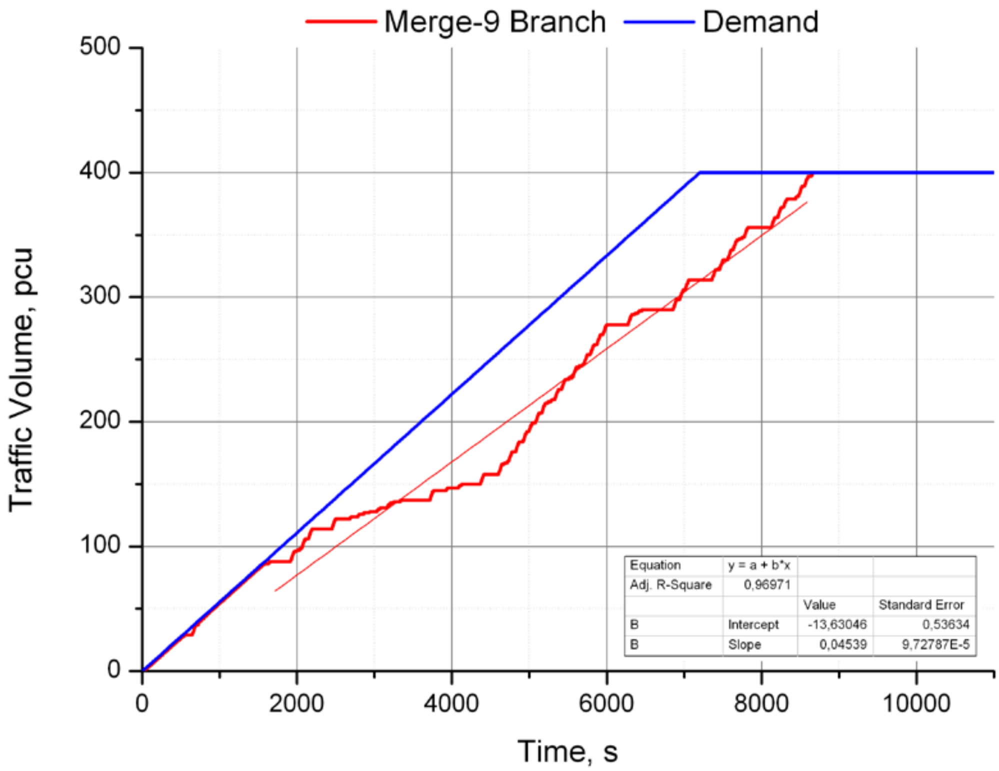
Merge-6 Main Line
Demand
Traffic Volume, pcu
0
500
1000
1500
2000
2500
3000
Time, s
0
2000
4000
6000
8000
10000
Equation | y = a + b*x
Adj. R-Square | 1
Value | Standard Error
B | Intercept | 1,70585 | 0,01102
B | Slope | 0,20516 | 2,61072E-6



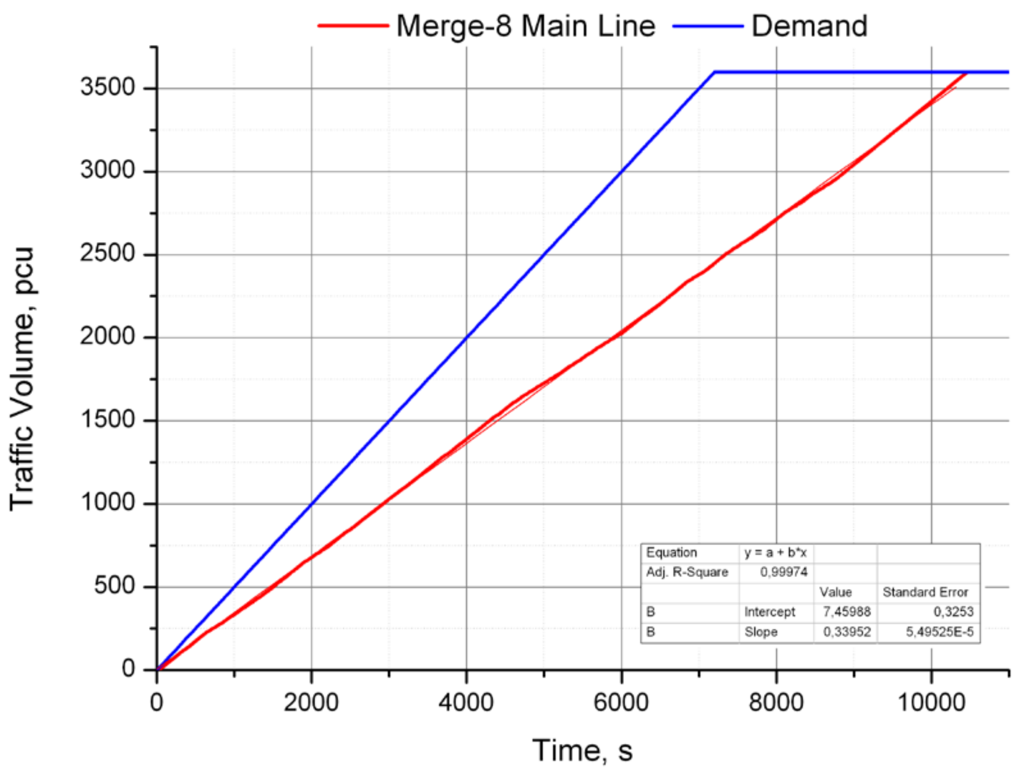
Merge-7 Branch
Demand
Traffic Volume, pcu
Time, s
0
200
400
600
800
1000
1200
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,99676
Value
Standard Error
B
Intercept
58,32198
0,19095
B
Slope
0,05833
3,764E-5



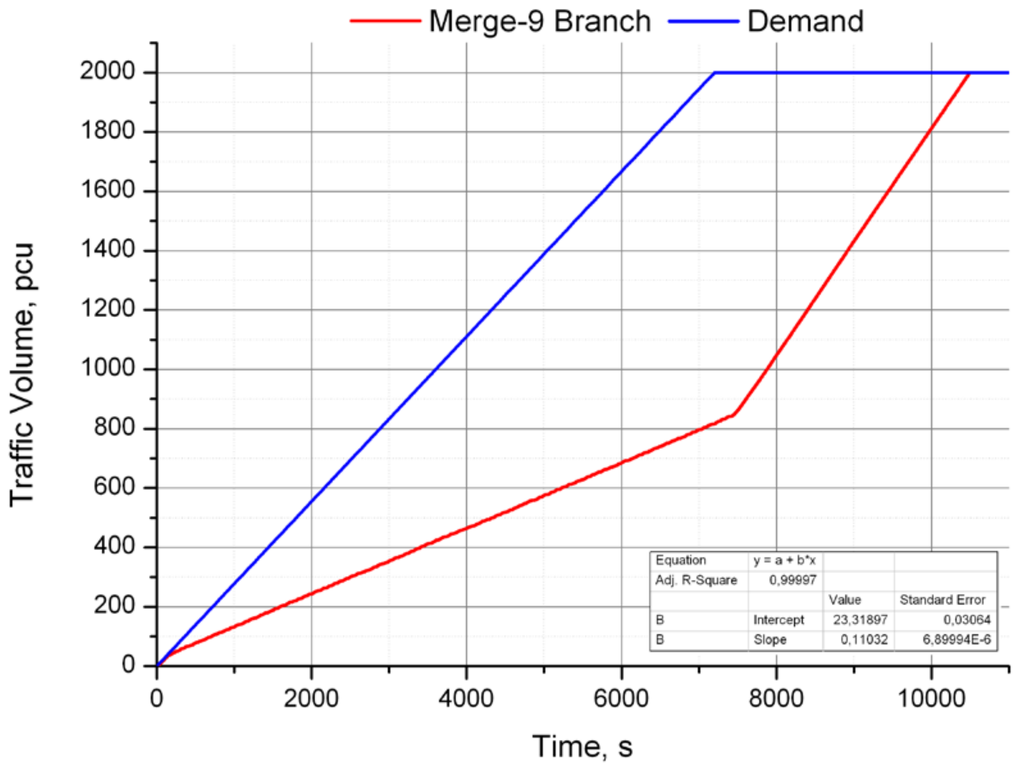
Merge-7 Main Line
Demand
Traffic Volume, pcu
Time, s
0
500
1000
1500
2000
2500
3000
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,9999
Value
Standard Error
B
Intercept
-25,14015
0,18587
B
Slope
0,32964
3,66097E-5

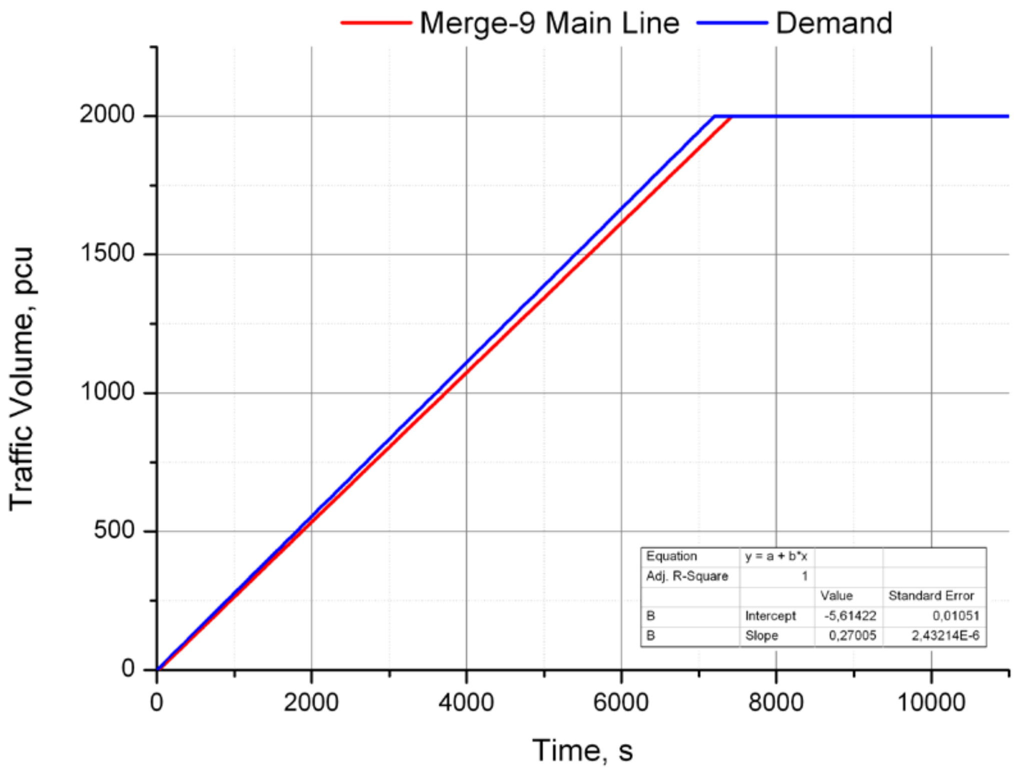


Merge-9 Branch
Demand
Traffic Volume, pcu
Time, s
0
100
200
300
400
500
0
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,96971
Value
Standard Error
B
Intercept
-13,63046
0,53634
B
Slope
0,04539
9,72787E-5

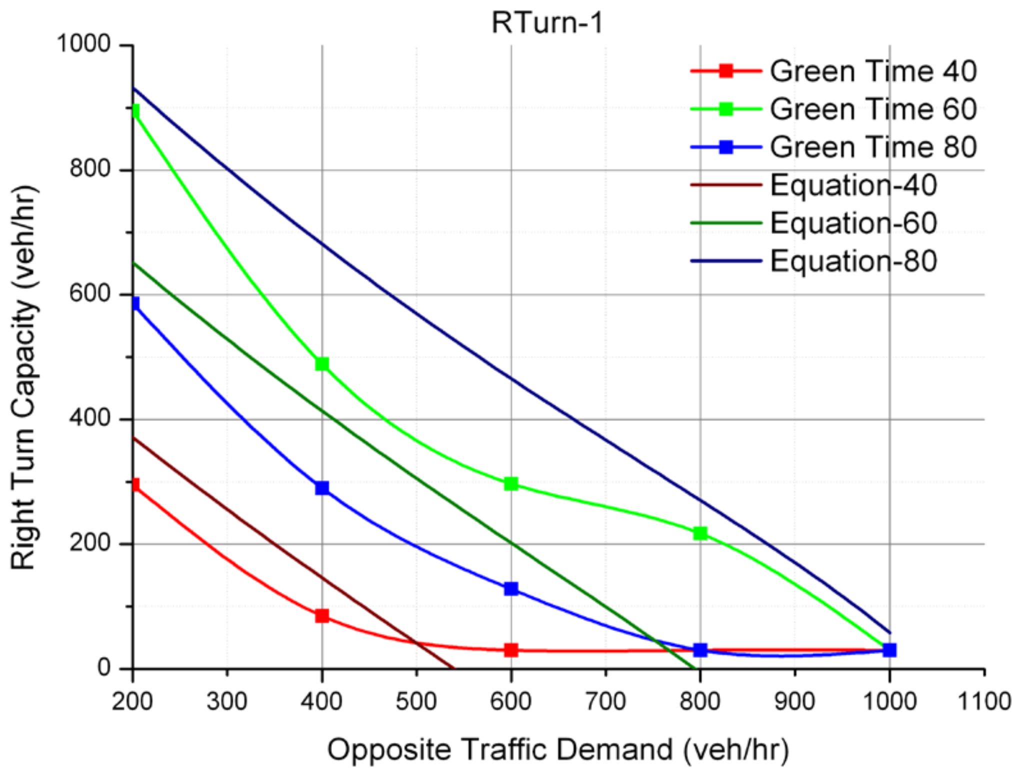


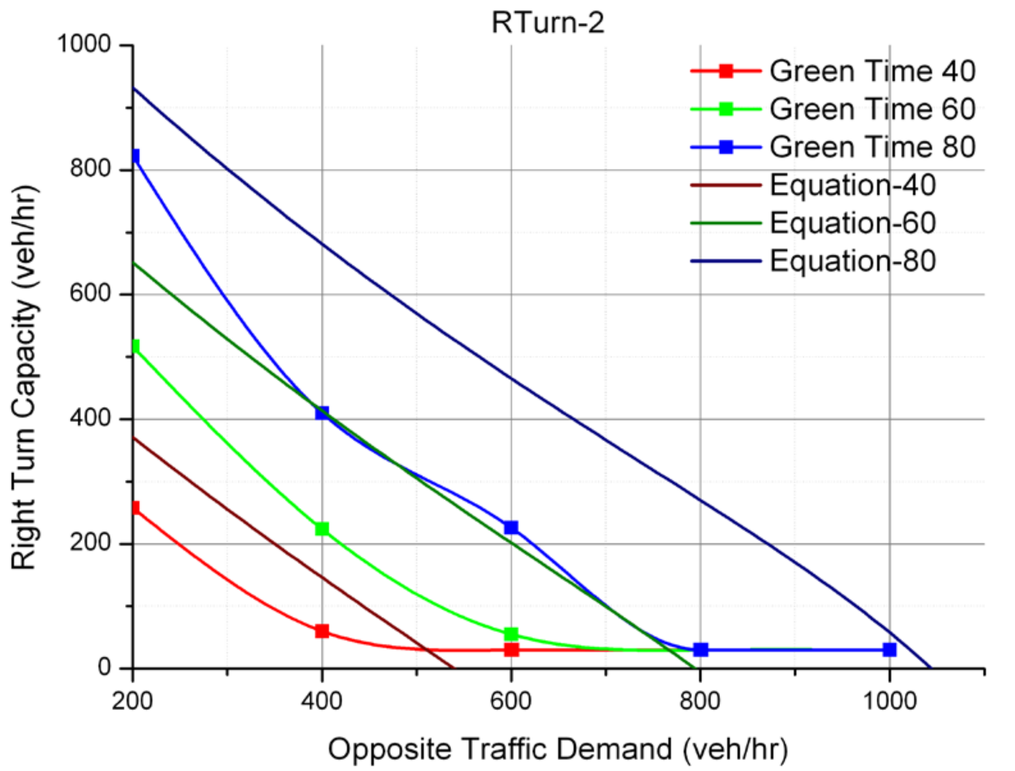
Merge-8 Main Line
Demand
Traffic Volume, pcu
Time, s
0
500
1000
1500
2000
2500
3000
3500
0
2000
4000
6000
8000
10000
Equation
y = a + b*x
Adj. R-Square
0,99974
Value
Standard Error
B
Intercept
7,45988
0,3253
B
Slope
0,33952
5,49525E-5

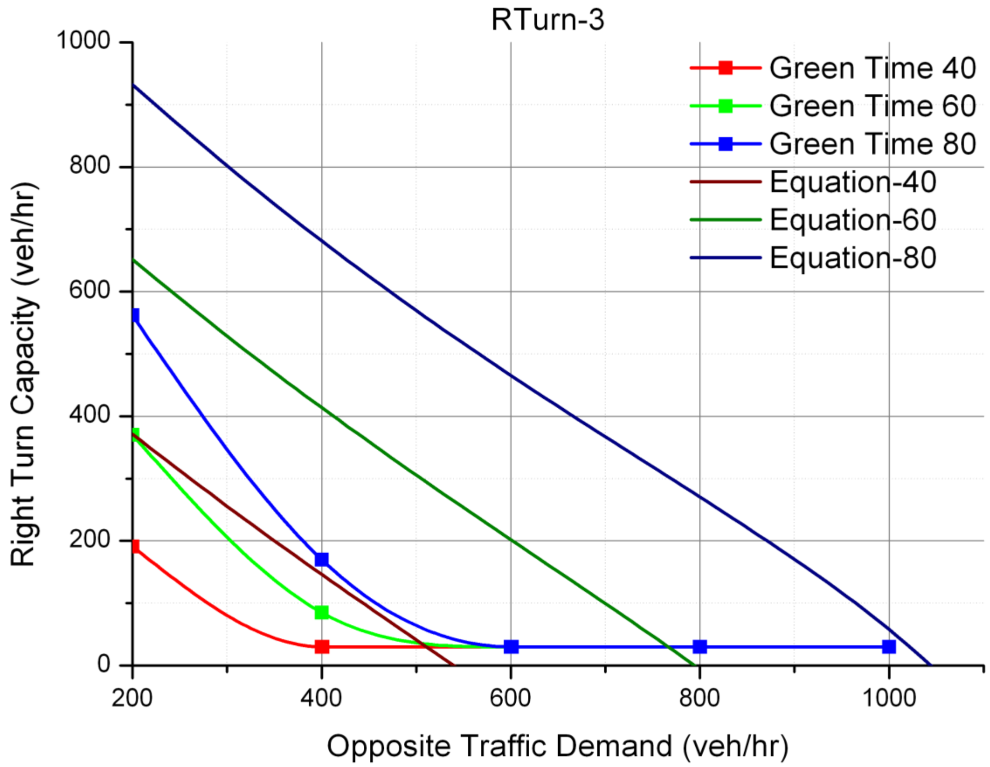

Merge-9 Branch
Demand
Traffic Volume, pcu
Time, s
Equation | y = a + b*x
Adj. R-Square | 0,99997
Value | Standard Error
B | Intercept | 23,31897 | 0,03064
B | Slope | 0,11032 | 6,89994E-6

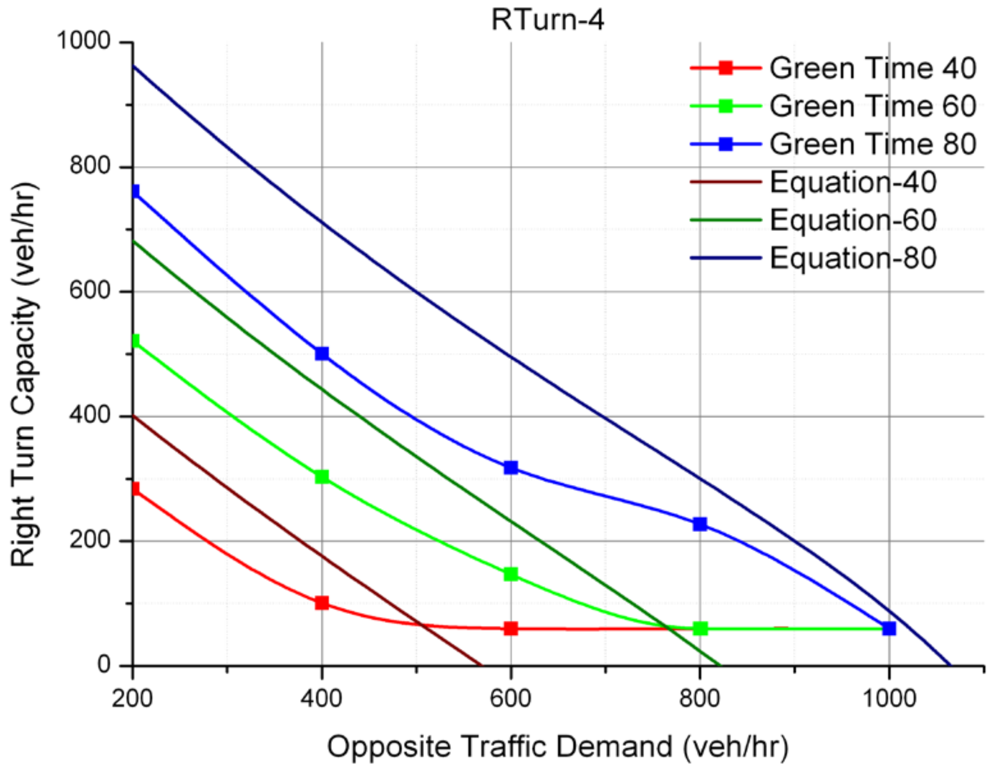

Merge-9 Main Line
Demand
Traffic Volume, pcu
Time, s
Equation | y = a + b*x
Adj. R-Square | 1
Value | Standard Error
B | Intercept | -5,61422 | 0,01051
B | Slope | 0,27005 | 2,43214E-6

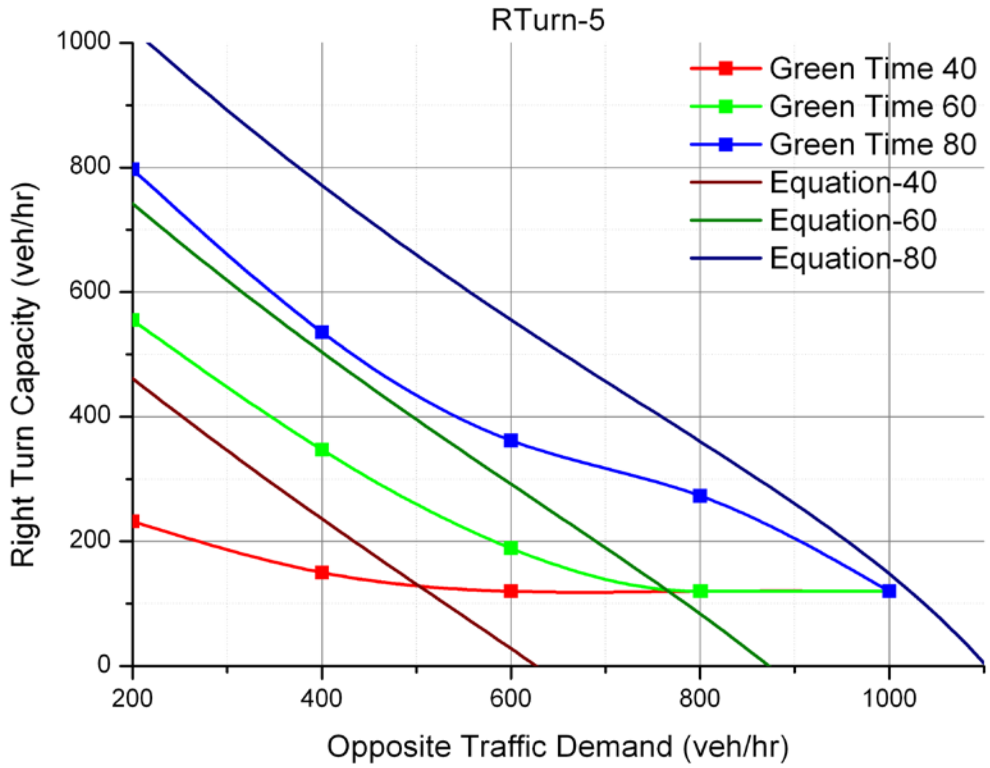
Right Turn Capacity Verification Graphs

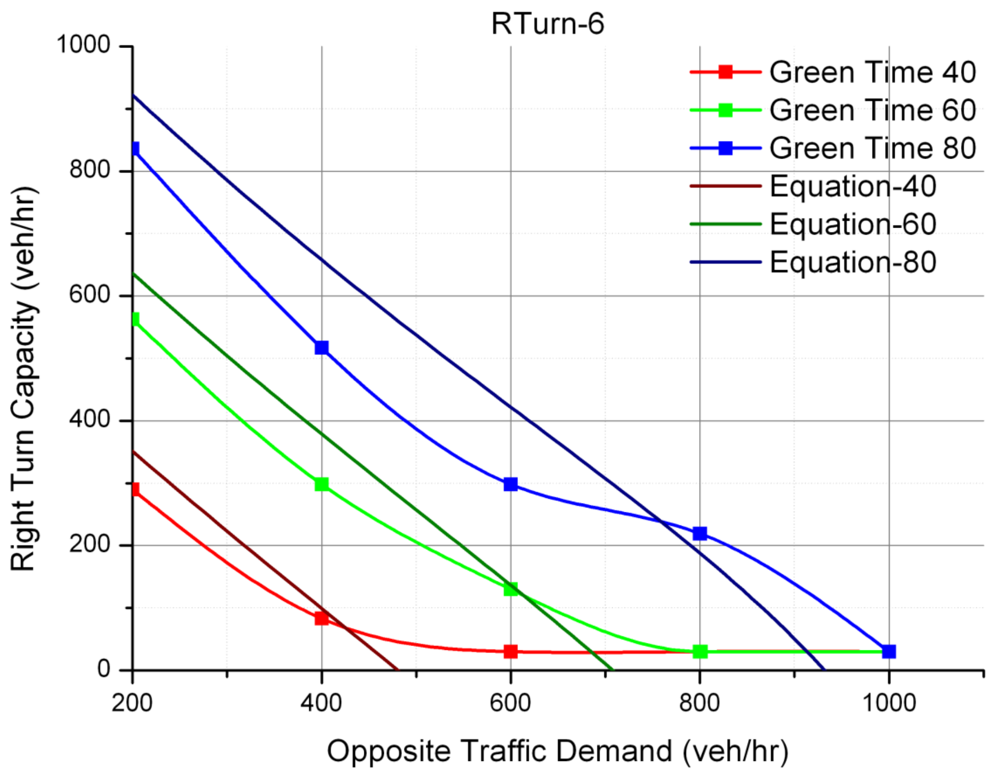


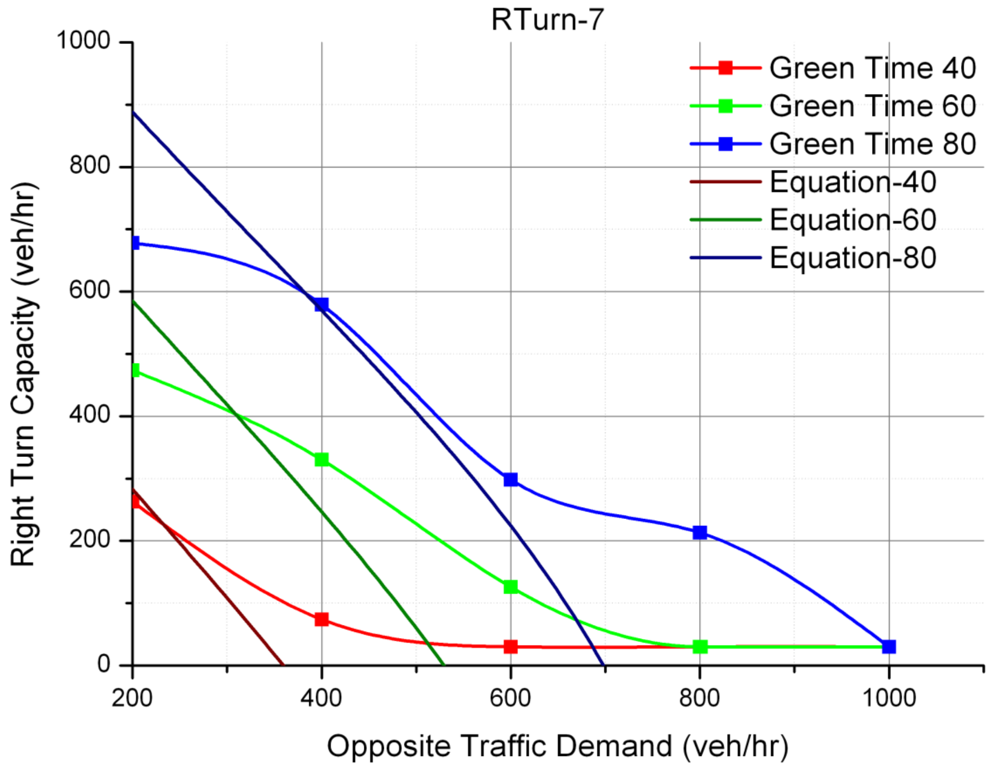



RTurn-3
Green Time 40
Green Time 60
Green Time 80
Equation-40
Equation-60
Equation-80
Right Turn Capacity (veh/hr)
Opposite Traffic Demand (veh/hr)
0
200
400
600
800
1000
200
400
600
800
1000


RTurn-4
Green Time 40
Green Time 60
Green Time 80
Equation-40
Equation-60
Equation-80
Right Turn Capacity (veh/hr)
Opposite Traffic Demand (veh/hr)
0
200
400
600
800
1000
200
400
600
800
1000


RTurn-5
Green Time 40
Green Time 60
Green Time 80
Equation-40
Equation-60
Equation-80
Right Turn Capacity (veh/hr)
Opposite Traffic Demand (veh/hr)
0
200
400
600
800
1000


RTurn-6
Green Time 40
Green Time 60
Green Time 80
Equation-40
Equation-60
Equation-80
Right Turn Capacity (veh/hr)
Opposite Traffic Demand (veh/hr)
0
200
400
600
800
1000


RTurn-7
Green Time 40
Green Time 60
Green Time 80
Equation-40
Equation-60
Equation-80
Right Turn Capacity (veh/hr)
Opposite Traffic Demand (veh/hr)
1000
800
600
400
200
0
200
400
600
800
1000