Sawmill Drive Extension

Traffic Impact Analysis
Final Report

June 17, 2010

Prepared for:

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ACRONYMS AND ABBREVIATIONS

<u>These acronyms and abbreviations may be used throughout this document.</u>

AADT, ADT	Average Annual Daily Traffic, Annual Daily Traffic
AAC	Alaska Administrative Code
AASHTO	American Association of State Highway and Transportation Officials
ADOT/ (&)PF, or DOT(/)(&)PF	Alaska Department of Transportation and Public Facilities
DD, DDHV	Directional Distribution, DD Hourly Volume
EB, EBL,EBLT	Eastbound, eastbound left turn
FT.,ft.,Ft.	Feet or foot (length)
GDHS	Geometric Design of Highways and Streets (Reference)
HCM, HCM2000	Highway Capacity Manual 2000 (Reference)
HCS, HCS2000	Highway Capacity Software
Hr.,hr.,H., h.	Hour(s)
Hwy	Highway
ISD	Intersection Sight Distance
ITE	Institute of Transportation Engineers
LOS	Level of Service (performance grade)
LRTP	Long Range Transportation Plan
LT, L	Left turn(s)
MEV, MVM	Million Entering Vehicles, Million Vehicle Miles
Mph, MPH, mph	Miles Per Hour
MUTCD	Manual of Uniform Traffic Control Devices
NB, NBL, NBLT	Northbound, northbound left turn
NCHRP	National Cooperative Highway Research Program
NPS	National Park Service
pcu	Passenger car unit(s)
PHF	Peak Hour Factor
Ped	Pedestrian
Pkwy	Parkway
PSD	Pedestrian Sight Distance
PTR	Permanent Traffic Recorder
Rd, RD	Road
RT, R	Right turn(s)
SB, SBL, SBLT	Southbound, southbound left turn
S, Sec	Second
Sf, SF	Square feet
SSD	Stopping Sight Distance
St, ST	Street
T, Th, Thru	Through
TRB	Transportation Research Board

These acronyms and abbreviations may be used throughout this document.

TIA	Traffic Impact Analysis
TWSC	Two-way-stop-control (2 stopped approaches)
UCL	Upper Control Limit
v/c,V/C	Volume to Capacity Ratio
Veh,v	Vehicle(s)
Vol	Volume
WB, WBL, WBLT	Westbound, westbound left turn

EXECUTIVE SUMMARY

This traffic impact study is being prepared for the City of Valdez to assess the impacts of the Sawmill Drive extension on area roadways and intersections. The proposed extension of Sawmill Drive will create a new intersection with Salcha Way, and will provide new circulation routes for existing and future developments. The street has an expected construction year of 2010, and an assumed design year of 2020. The work in the study included:

- Traffic data collection (volume, speed, sight distance).
- Review and assimilation of historic traffic volume information.
- Review and analysis of historic crash data.
- Review of the area planning background.
- Analysis of projected development including trip generation and distribution.
- Operational and capacity analyses of background conditions and conditions with future developments.

Two options were considered for the connection of the Sawmill Drive extension to Salcha Way/Atigun Drive. The first option would connect the extension at the existing corner where Atigun Drive turns into Salcha Way. The second option would be a new connection further to the north along Salcha Way across from the ball field. Based on volumes and sight distance it is recommended that the connection of the extension be made at the existing intersection of Salcha Way and Atigun Drive, with Salcha Way the stop controlled approach.

Sawmill Drive is currently functions local road with an unpaved surface and no outlet at its eastern terminus. The proposed extension will create a connection with the local road network at Salcha Way that will likely change the functional class of Sawmill Drive. After the connection, Sawmill Drive will likely function as a Minor Collector carrying traffic from local roads to the Richardson Highway which is a Principal Arterial. This connection to the Richardson Highway is expected to shift local traffic that currently uses Salcha Way to access the Richardson Highway via Valdez Airport Road, to the new extended Sawmill Drive.

Historical data was gathered from both population based and traffic based sources to determine what the background growth rate should be. It was found that data sources show an unchanging or declining growth rate. As such no background growth is applied, only a redistribution of traffic within the study area in the opening year.

In addition to the redistribution of area traffic, there are several developments expected to increase traffic generated along Atigun Drive and in the vicinity of Sawmill Drive. Expected development includes expansion of two mobile home parks, and the new development of an approximately 70 acre industrial subdivision on Atigun Drive. In the vicinity of Sawmill Drive there is potential for the addition of a number of single family residences. Kinney

Engineering, LLC (KELLC) estimated the potential trips generated by the new developments. These additional trips were then added to area roadway traffic to arrive at 2020 volumes.

The 2020 traffic numbers were then used to assess the levels of service at area intersections. Based on predicted turning movement volumes at these intersections KELLC checked whether signal warrants were met and evaluated whether auxiliary turn lanes would be required. It was found that none of the area intersections are likely to meet signal warrants in the future according to the Cal Trans method of evaluating intersections for warrants based on future volumes. Approaches to the intersections were evaluated based on NCHRP guidelines, operational levels of service, and crash history to determine whether turn lanes were recommended. Several auxiliary lanes are recommended and those lanes are presented in Exhibit 1.

The Salcha Way segment had a high crash rate when compared to similar roadways during the 10 year study period. Out of the 4 crashes that occurred on this segment, 3 of them were under snow and ice conditions. It can be concluded that Salcha Way's high crash rate may overstate the safety problems on the street, especially since all were property damage only, and most occurred on poor road conditions. Severity was overrepresented during the study period for the intersection of the Richardson Highway and Valdez Airport Road. The rear end and overtaking sideswipe crash rates will benefit from the construction of auxiliary lanes to remove turning vehicles from the through traffic.

As part of the construction of the new connection between the Sawmill Drive extension and the Atigun Drive/Salcha Way intersection, the existing pathway on the east side of Salcha Way will need to be reconfigured to accommodate the intersection improvements. In addition, since there is a substantial residential development planned in this area, a new pathway along the Sawmill Drive extension should be considered for future implementation to provide a direct corridor to the Richardson Highway facilities and promote non-motorized travel.

If the future development is limited to residential dwellings, without the industrial development, then the roadway would function as a residential or neighborhood collector. The Municipality of Anchorage has developed standards for collector streets which could be applied to this roadway. Under that methodology, Anchorage would require 10-foot lanes and 3.5-foot shoulders and curb/gutters for the collector. This strip paved section could be adapted to provide striped 10-foot lanes, 2-foot paved shoulders, and 4-foot gravel shoulders.

Upon development of the industrial area, both the volumes and percentage of trucks would increase, and the above described section may be inadequate. With increased trucks and volume, 12-foot lanes with additional paved shoulder width (4 to 8 feet) would be recommended.

It is recommended that the surface of Sawmill Drive should be paved along its entirety. It is also recommended that lighting should be provided for the project intersections (on the

Richardson Highway) that have left turn lanes, to illuminate the left turn lanes from the beginning of the widening taper.

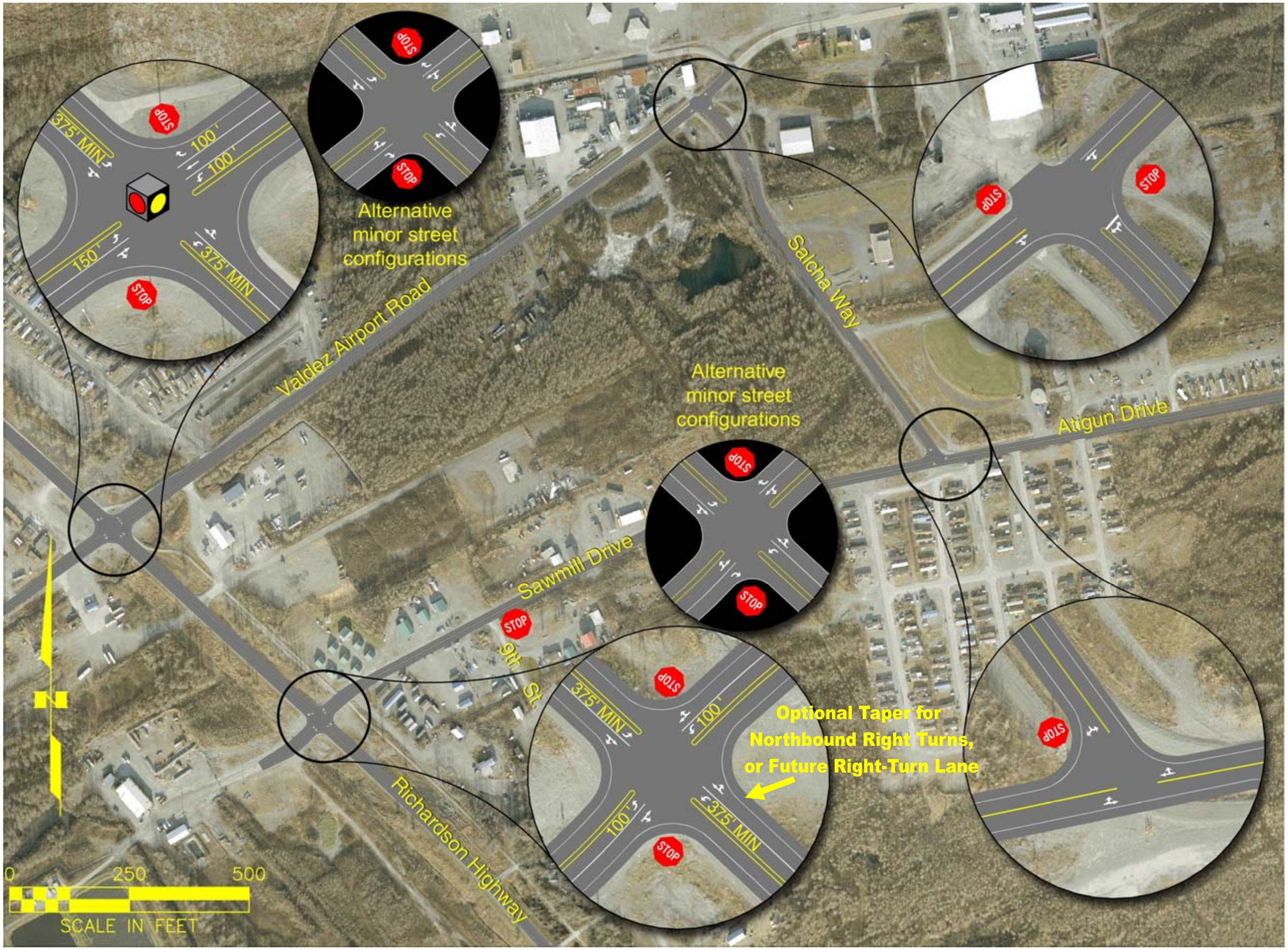


Exhibit 1- Recommended Alternative

1 INTRODUCTION

1.1 Project Description and Location

This Traffic Impact Analysis addresses the future extension of Sawmill Drive to connect with Salcha Way in Valdez Alaska. The project location is presented in the following figure followed by a graphic of the traffic impact study area.

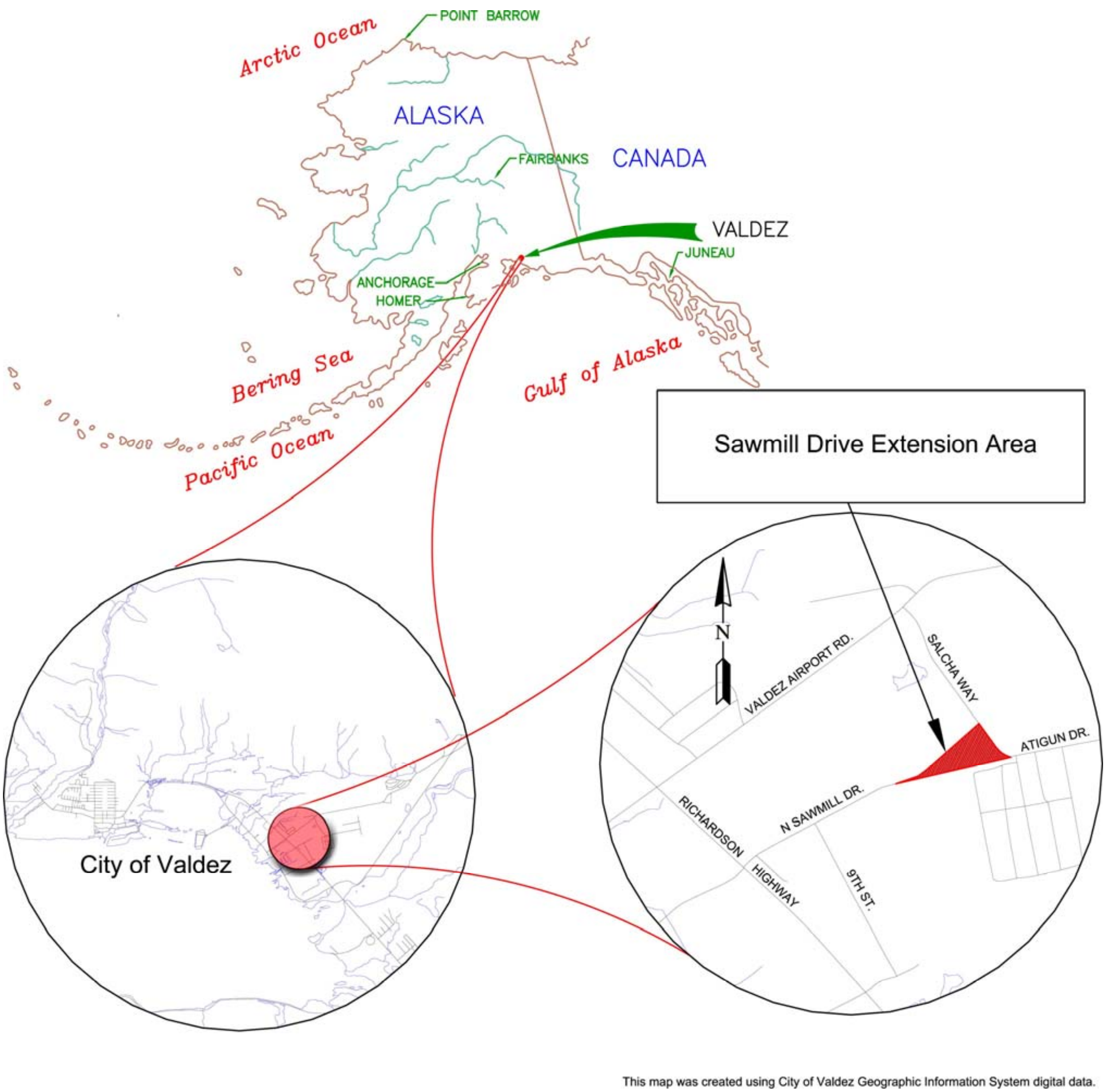


Figure 1- Location and Vicinity Map

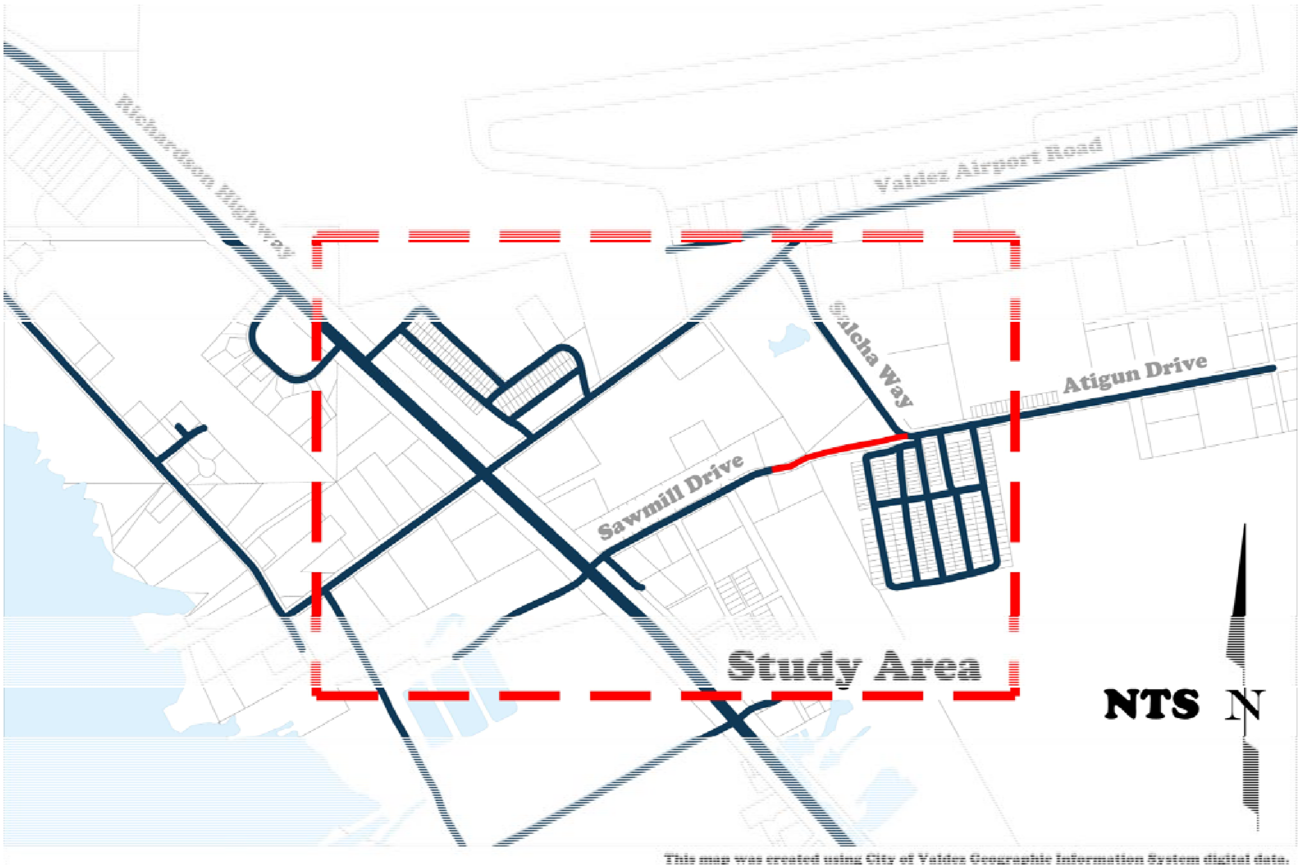


Figure 2- Traffic Impact Study Area

1.2 Proposed Development Overview

The City of Valdez proposes to extend Sawmill Drive to Salcha Way/Atigun Drive. This traffic impact study is being prepared for the City of Valdez to assess the impacts of the extension on area roadways and intersections. The proposed extension of Sawmill Drive will create a new "T" intersection with Salcha Way. This intersection will be stop controlled on the Salcha Way approach and uncontrolled on the eastbound and westbound approaches.

The area of the proposed extension is conceptually presented in the following figure.

Figure 3- Proposed Location of the Sawmill Drive Extension

1.3 Traffic Impact Analysis Issues

The extension of Sawmill Drive to Salcha way will redistribute existing traffic patterns and increase connectivity. It is likely that trips generated from the existing mobile home parks on the north and south side of Atigun Drive will use the new connection to access the Richardson Highway. This traffic is currently required to go north on Salcha Way and make a left at Valdez Airport Road to access the Richardson Highway. This change in circulation is likely to impact the Richardson Highway intersections with Sawmill Drive and with Valdez Airport Road as well. There are also several developments planned for the study area that are expected to increase area traffic. This traffic impact analysis will study the projected impacts of these changes to area intersections.

1.4 Analysis and Design Years

For purposes of this analysis, the study area's roadway network will be evaluated for the following years.

- **2008**- This year represents the current traffic conditions (base year) prior to any Sawmill Drive Extension Construction.
- **2010**- Sawmill Drive Extension Opening year
- **2020**- Sawmill Drive Extension Design Year.

2 INVENTORY OF EXISTING CONDITIONS

2.1 Streets and Intersections

2.1.1 Functional Classification of Existing Roadways

The American Association of State Highways and Transportation Officials (AASHTO) *A Policy on the Geometric Design of Highways and Streets (GDHS)* is a primary reference for roadway design. AASHTO and other agencies generally classify streets under one of three functional classes. Arterials are designed to carry large volumes at an efficient speed. Local streets serve the terminal ends of a trip. Collector streets gather and distribute trips between local streets and arterials. AASHTO and other agencies further provide sub-categories of the broad classes. For example arterials may be classified as freeways, expressways, principal or minor arterials and collectors may include major and minor collectors.

AASHTO indicates that the arterial's main system function is mobility, the local street's main function is landside access, and that collectors offer a balance of access and mobility. The following figure illustrates mobility and access by classification, and is adapted from AASHTO's Exhibit 1-5 in the *Policy*. It also conceptually annotates street sub-categories upon the continuum of the access-mobility function.

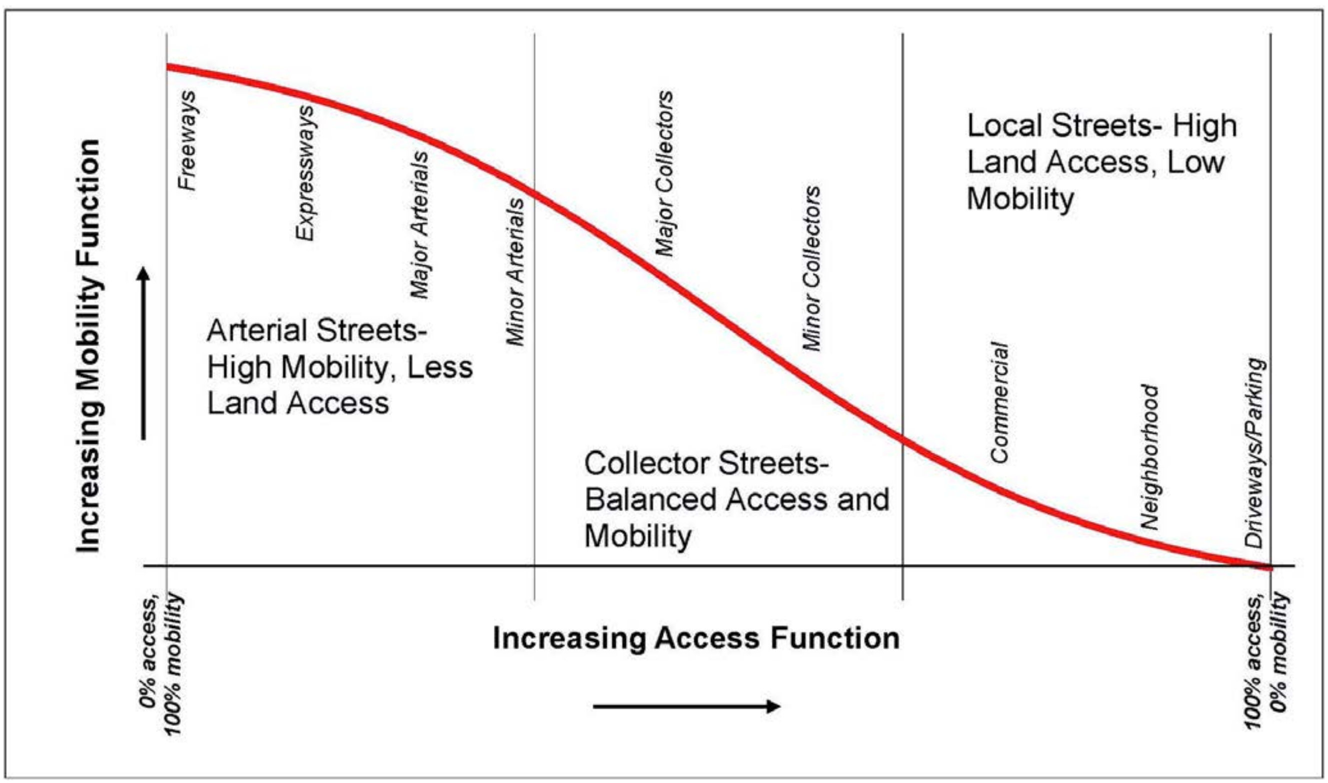


Figure 4- Functional Classification Mobility and Access Relationship

A good street system encourages a hierarchy of movement. The local street is at the bottom of the hierarchy and the arterial is at the top. For the most efficient travel, the motorist moves up and down the street hierarchy to spend as little time as possible in the lower levels and the most time possible at the arterial level. As such, the most desirable hierarchical movement would be from driveway or parking lot to local street to collector to arterial, and vice-versa upon nearing the destination. It is less desirable, although somewhat common, to connect local streets directly to arterial streets because the hierarchical movement is violated, and in doing so it can result in operational and safety issues.

The Alaska Department of Transportation and Public Facilities (ADOT&PF) provides functional classifications for their roadways, as well as many local jurisdiction roadways. The functional classification of the Richardson Highway is a Rural Principal Arterial (similar to Major Arterial in Figure 4). The functional classification of Valdez Airport Road is Rural Major Collector west of Salcha Way and Rural Minor Collector east of Salcha Way. All other roads in the study area are functionally classified as Local Roads.

The area label “rural” has the consequence of higher levels of service (performance measure) than roadway in an “urban” or “suburban” setting. From a practical standpoint, these facilities are in a small urban area, and therefore less restrictive performance measures should apply.

2.1.2 Street Typical Sections

The following table presents the lane and shoulder width for the roadways within the study area.

Roadway	Lane Width	Shoulder Width
Richardson Highway	12	4
Sawmill Drive	Gravel Approximately 24 Foot Top	
Airport Road	12	4
Salcha Way	12	1
Atigun Drive	12	1
9th Street	Gravel Approximately 24 Foot Top	

Table 1- Roadway Lane and Shoulder Dimensions

2.1.3 Intersections

The existing conditions are presented in the following figure including the existing intersection lane configurations and intersection controls.

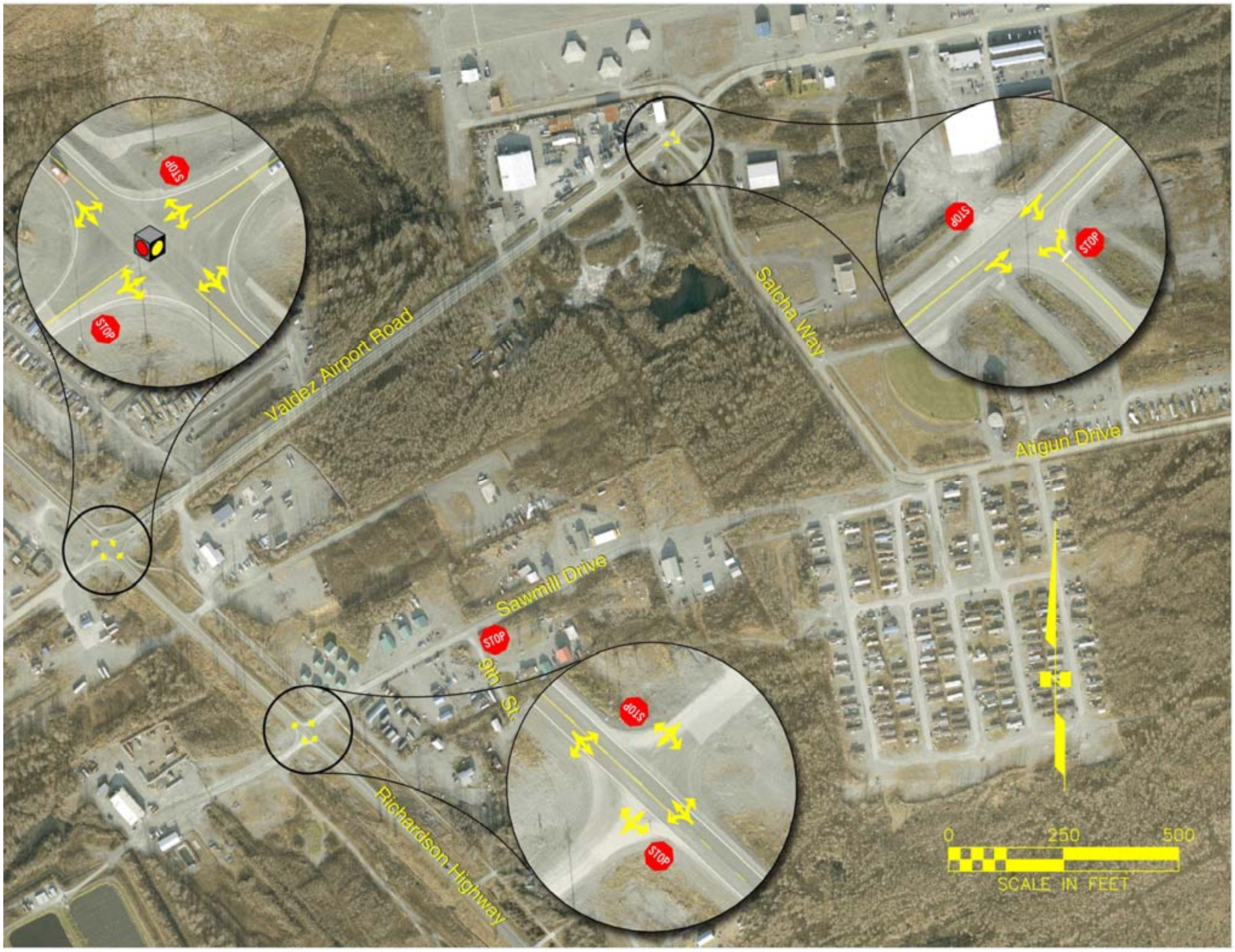


Figure 5- Existing Conditions

2.1.4 Public Transit

There are no modes of public transit in the study area.

2.1.5 Pedestrian and Bicycle Facilities

There is a bike path on the east side of the Richardson Highway, the north side of Valdez Airport Road, and the west side of Salcha Way within the study area. There are no pedestrian or bicycle facilities currently along Sawmill Drive or Atigun Drive other than sharing the roadway with motorists.

3 EXISTING TRAFFIC CONDITIONS

Kinney Engineering collected volume and speed data in Valdez during March 2010, and compiled existing volume data from ADOT&PF references. The locations of field studies are presented in the following figure. Hand held radar collectors on the Richardson Highway indicate zones of speed studies. Radar recorder symbols indicate the location of radar traffic data collectors that are capable of continuous collection of speed, volume, vehicle type and gap data.

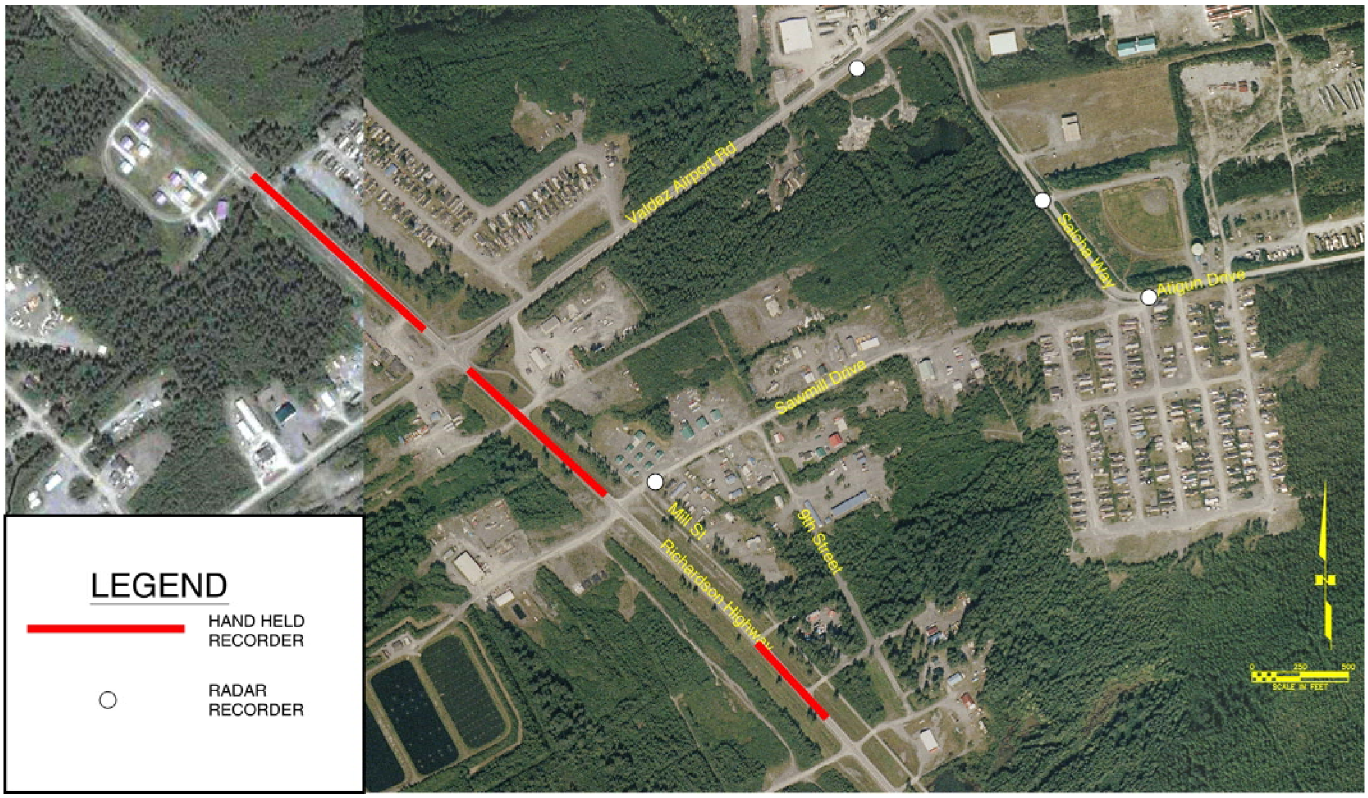


Figure 6- Collection Locations (March 2010)

3.1 <u>Turning Movement Volumes</u>

Figure 7 presents the existing turning movement volumes counted at area intersections in March 2010.

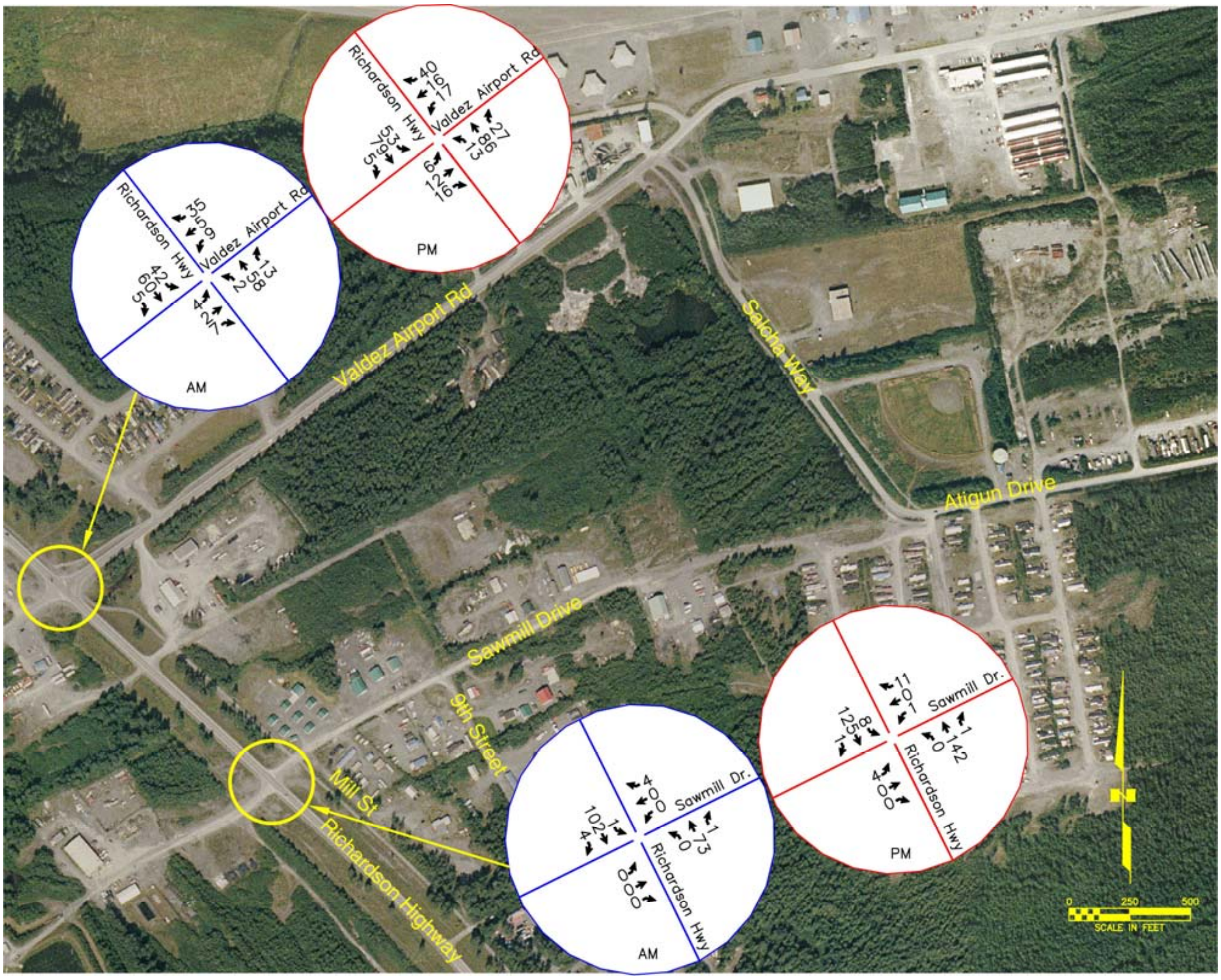


Figure 7- March 2010 Turning Movement Counts

This turning movement data also yield truck counts and peak hour factors which are used to convert hourly volumes to peak 15-minute traffic flow rates. The average truck percentage at the Richardson/Airport Road intersection was about 2% of total entering traffic. The peak hour factor varied by movement but was about 0.82 for the entire intersection. These parameters were applied to capacity analysis models.

3.2 Speed Study

3.2.1 Richardson Highway

There were spot speed studies (with radar gun) at three locations on the Richardson Highway. One was located south of the Sawmill Road intersection, one was located between the Sawmill Drive intersection and the Valdez Airport Road intersection and one was located north of the Valdez Airport Road intersection. Figure 6 on page 8**Error! Reference source not found.** presents the data collection locations. The following table summarizes speed information by direction for the time that the machines were deployed.

Richardson Highway Speed Data	Location/Direction					
	South of the Sawmill Drive Intersection		Between the Sawmill Drive Intersection and the Valdez Airport Road Intersection		North of the Valdez Airport Road Intersection	
	Northbound	Southbound	Northbound	Southbound	Northbound	Southbound
Posted	55 MPH	55 MPH	55 MPH	55 MPH	55 MPH	55 MPH
Average	55 MPH	53 MPH	53 MPH	51 MPH	54 MPH	53 MPH
85th Percentile	59 MPH	59 MPH	57 MPH	56 MPH	57 MPH	57 MPH
Observations	51	54	100	115	110	79

Table 2- Richardson Highway Speed Study Results

The 85th percentile speeds vary along the Richardson Highway. The posted speed, 55 MPH, is a good representation of the 85th percentile speed and should be applied to existing conditions analysis.

3.2.2 Sawmill Drive

A radar traffic data collector was deployed on Sawmill Drive (see Figure 6 on page 8). The counter was in place for two days and continuously collected traffic volume, speed, gap, and type information. The following table summarizes speed information by direction for the time that the machine was deployed.

Sawmill Drive	Eastbound	Westbound
Posted	20 MPH	20 MPH
Average	22 MPH	22 MPH
85th Percentile	25 MPH	27 MPH
Observations	56	60

Table 3- Sawmill Drive Speed Study Results

The 85th percentile speed is between 25-27 mph for Sawmill Drive, which substantially exceeds the posted speed limit. The 85th percentile speed, say 30 mph should be applied to existing conditions analysis in place of the existing posted speed.

3.2.1 Salcha Way

A radar traffic data collector was deployed on Salcha Way (see Figure 6 on page 8). The counter was in place for two days and continuously collected traffic volume, speed, gap, and type information. The following table summarizes speed information by direction for the time that the machine was deployed.

Salcha Way	Southbound	Northbound
Posted	30 MPH	30 MPH
Average	24 MPH	26 MPH
85th Percentile	28 MPH	31 MPH
Observations	277	235

Table 4- Salcha Way Speed Study Results

The 85^{th} percentile speed is between 25-27 mph for Sawmill Drive. The 30 MPH posted speed is a good representation of the 85^{th} percentile speed and should be applied to existing conditions analysis.

3.2.2 Atigun Drive

A radar traffic data collector was deployed on Atigun Drive (see Figure 6 on page 8). The counter was in place for two days and continuously collected traffic volume, speed, gap, and type information. The following table summarizes speed information by direction for the time that the machine was deployed.

Atigun Drive	Eastbound	Westbound
Posted	20 MPH	20 MPH
Average	21 MPH	21 MPH
85th Percentile	27 MPH	27 MPH
Observations	224	236

Table 5- Atigun Drive Speed Study Results

The 85^{th} percentile speed is 27 mph for both directions on Atigun Drive, which substantially exceeds the posted speed limit. For analyses purposes, a 30 MPH speed should be applied to existing conditions analysis in place of the existing posted speed.

3.2.3 Valdez Airport Road

A radar traffic data collector was deployed on Valdez Airport Road (see Figure 6 on page 8). The counter was in place for two days and continuously collected traffic volume, speed, gap, and type information. The following table summarizes speed information by direction for the time that the machine was deployed.

Valdez Airport Road	Eastbound	Westbound
Posted	35 MPH	35 MPH
Average	37 MPH	37 MPH
85th Percentile	40 MPH	40 MPH
Observations	739	760

Table 6- Valdez Airport Road Speed Study Results

The 85th percentile speed is 40 mph for both directions of Valdez Airport Road, which exceeds the posted speed limit. The 85th percentile speed should be applied to existing conditions analysis in place of the existing posted speed.

3.3 Traffic Volumes

3.3.1 Average Annual Daily Traffic

The following table presents average annual daily traffic (AADT) that has been recorded by the State of Alaska Department of Transportation and Public Facilities, and published in the annual *Northern Region Traffic Volume Report*.

Location	1998	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	Average
190000 Richardson Highway												
Mineral Creek Loop	6,050	5,350	5,500	5,713	5,925	5,375	5,275	4,650	5,080	4,550	4,095	5,233
Valdez Airport Road	4,650	3,725	3,725	3,950	4,175	3,750	3,000	3,725	3,400	3,745	3,240	3,735
191000 Valdez Airport Road												
Richardson Highway	3,350	2,700	2,700	2,588	2,475	2,525	2,550	2,625	2,560	1,950	1,845	2,533
Salcha Way	725	575	575	575	575	575	575	600	575	500	500	577

Table 7- Study Area Roadways AADT History

3.3.2 2009 State Counts

There is a permanent traffic recorder (PTR) on the Richardson Highway at mile point 0.543, approximately 3 miles from the study area (Valdez Airport Road is at MP 3.426). The mile point 0.543 PTR data was used to determine the peak daily traffic volumes. This volume of vehicles is used as the basis for future volume predictions. The data used to determine the peak volume is included in Appendix E- PTR Data, Richardson Highway. The historical data indicates that AADT has been trending downward over the ten years of data. The average rate of decline over all roadways has been approximately -2%.

3.3.3 March 2009 Daily Volume Counts

Daily volume data for Salcha Way and Atigun Drive was collected by radar traffic data recorders located as shown in Figure 6 on page 8. The 2010 daily volume for these streets are summarized in Figure 12 on page 26 as 2010 “no-build” volumes.

4 TRAFFIC SAFETY ANALYSIS

4.1 Substantive Safety Evaluation, Crash History

Crash data was collected from ADOT&PF for the 10 most recent years that are available, 1999 to 2008. Three intersections and two segments were analyzed to determine if they had statistically higher crash rates than state populations during the study period. Table 9 below summarizes the crash rates for the intersections and segments evaluated.

Rate analysis is especially useful when there is a population of facilities to which we can compare the study area. ADOT&PF develops and distributes statewide populations for segments and intersections. A method known as the Rate Quality Control Method establishes an upper control limit (UCL) to determine if the facility's accident rate is significantly higher than accident rates in facilities with similar characteristics. If the UCL is exceeded, we would conclude that the high crash rate is not solely due to chance, and that there are truly crash issues at the location. Appendix B- Crash Evaluation Methodology discusses crash evaluation methods and the UCL computation further.

Segment	Segment Crashes 1999 to 2008	Segment Length (Miles)	Ave AADT 1999 to 2008	Million Vehicle Miles (MVM)	Crashes / MVM	State Population	UCL @ 95.00 % Conf	Above Average ?	Above Critical ?
Airport Rd, Richardson Hwy to Salcha Wy	1	0.472	2452	4.224	0.237	1.436	2.513	no	no
Salcha Wy, Valdez Airport Rd to Atigun Dr	4	0.362	500 (esti-mated)	0.661	6.055	0.934	3.647	yes	yes

Table 8- Segment Crashes and Crash Rates, 1999 to 2008

Intersection	Crashes	Crashes / MEV	State Populations Average Rate for Similar Intersections Crashes / MEV	UCL @ 95.00% Confidence Crashes / MEV	Above Average ?	Above Critical ?	Safety Index
Valdez Airport Rd and Richardson Hwy	20	.927	0.736	1.063	yes	no	0.87
Sawmill Dr and Richardson Hwy	0	0.000	0.736	1.161	no	no	0.00
Valdez Airport Rd and Salcha Wy	1	0.182	0.582	1.208	no	no	0.15

Table 9- Intersection Crashes and Crash Rates, 1999 to 2008

4.1.1 Salcha Way Segment from Valdez Airport Road to Atigun Drive

As shown in Table 8 and the segment of Salcha Way from its intersection with Valdez Airport Road to Atigun Drive is the only segment or intersection with a crash rate above the upper control limit (UCL) of the State populations for similar roadway segments. The following is an analysis of the crash severity along the segment of Salcha Way from Valdez Airport Road to Atigun Drive and the intersection of the Richardson Highway with Valdez Airport Road.

Severity	Number	% of Total	Population %	Statistical Significance at an α=0.05
Property Damage Only	4	100%	71.62%	Not significant.

Table 10- Salcha Way Segment from Valdez Airport Road to Atigun Drive Crash Severity Overview

All four crashes during the study period resulted in property damage only. Though the percentage of crashes is above population proportions, it is not statistically overrepresented. There is not a severity issue along this segment.

The following table summarizes crash types on this segment.

Crash Type	Number
Ditch	1
Head On	1
Parked	1
Snowberm	1
Total for Intersection	4

Table 11- Salcha Way Segment from the Intersection with Airport Road to Atigun Drive Overrepresented Crashes, 1999 to 2008

One of these crashes was within the 180-foot radius curve area where Salcha Way sweeps into Atigun Drive and it occurred under icy road conditions. The normal design speed for this curve is about 22 to 25 mph, which is less than the recorded 85th percentile speeds presented in Table 4 and Table 5. The other three were on the north end of the project, also within curves, and two of these three occurred under snow and ice conditions. Severity of these collisions is limited to property damage only.

It can be concluded that Salcha Way's very high crash rate may overstate the safety problems on the street, especially since all were property damage only, and most occurred on poor road conditions.

4.1.2 Richardson Highway Intersection with Valdez Airport Road

Table 9 indicates that the Richardson Highway Intersection with Valdez Airport Road is higher than average, with about 2 crashes per year, but does not exceed the UCL for the intersection. The following table summarizes the Richardson Highway/Valdez Airport Road severity history.

Severity	Number	%	SOA Population %	Statistical Significance at an α=0.05
Major Injury	3	15%	2.85%	Major Injury crashes are significant
Minor Injury	3	15%	24.96%	Not Significant
Property Damage Only	14	70%	71.62%	Not Significant

Table 12- Richardson Highway Intersection with Valdez Airport Road Crash Severity Overview

The following table summarizes crash types.

Crash Type	Major Injury	Minor Injury	Property Damage Only	Total
Ditch			1	1
Head On			2	2
Parked			1	1
Rear End		2	5	7
Right Angle	3	1	2	6
Sideswipe			2	2
Utility Post			1	1
Grand Total	3	3	14	20

Table 13- Richardson Highway Intersection with Valdez Airport Road Crash Type and Severity

Major injury crashes are statistically overrepresented at the Richardson Highway/Valdez Airport Road when compared to State of Alaska severity trends. Out of the 3 major injury crashes the driver of vehicle one was cited with negligent driving. Vehicle one is normally coded as the vehicle causing the crash. No ticket was issued in the third crash but driver inattention was cited as the human circumstance. All three of these crashes were right angle crashes which may be mitigated by reducing the number of conflicts at the intersection.

Rear-end and same-direction sideswipe crashes often have the same contributing factors. These types accounted for 9 of the 20 crashes at this intersection. Of those, seven occurred on the Richardson Highway northbound and southbound approaches. The following table summarizes involved vehicles' pre-actions on the Richardson Highway approaches.

	Vehicle 2 Action Or Pre-Action					
Vehicle 1 Action Or Pre-Action	**Changing Lanes**	**Slowing**	**Stopped**	**Straight Ahead**	**Turning Right**	**Row Total**
Entering Traffic Lane				1		1
Skidding	1		1			2
Slowing		1				1
Stopped				1		1
Straight Ahead					1	1
Turning Right				1		1
Column Total	1	1	1	3	1	**7**

Table 14- Richardson Highway Intersection with Valdez Airport Road Pre-Actions of Rear-End and Sideswipe Involved Drivers

As shown in this table, one or both of the involved drivers on the Richardson Highway approaches were stopping or slowing. These pre-actions on a highway's free flow approaches indicates that rear-end and sideswipe crashes may be caused by through vehicles colliding with decelerating left- or right-turning target vehicles. In fact, two crashes explicitly involved right-turning vehicles, and both of these crashes involved northbound vehicles. Left-turns are often not recorded as pre-actions because the police report only notes the actions at instant of the collision, without regard of the intentions.

Six of the intersection crashes were right-angle types of crashes involving vehicles from the orthogonal approaches of the intersection. Five of these collisions had citations of failure to yield or negligent driver. Sight distances and high volumes may contribute to minor approach driver errors in selecting suitable gaps to enter the main highway. At this location, though, the drivers have adequate sight distance, and the volumes are not excessively high.

4.2 Compliance with Sight Distance Standards (Nominal Safety)

American Association State Highway Transportation Officials' (AASHTO) G*eometric Design of Highway and Streets* (GDHS) Chapter 9 discusses intersection sight distance (ISD) in which a sight triangle is formed by conflicting approach vehicles. Minimum ISD for the stop-controlled approach is the stopping sight distance (SSD) along the major, uncontrolled street. This would allow major street vehicles time to adjust speeds or stop in the case where an egress movement from the minor street fails to yield properly.

A more conservative and desirable design condition would provide ISD to allow the minor-approach vehicle to view main road vehicles and select safe gaps for egress maneuvers. The vehicles on the stop sign controlled approach are under Case B ISD, which is the most restrictive condition, and generally controls for when the main street is two-way traffic flow. The minor approach vertex of ISD sight triangle is 15 feet from the travel way, at a height of 3.5 feet. The major approach vertex of the sight triangle is at the center of the approach lane at a 3.5-foot height and the sight distance is the minimum ISD or desirable Case B distance. The following table presents the field measured sight distance, the required sight distance for stopping and the required sight distance for the Case B1 (left turn from stop) condition at the intersections within the study area.

		Sight Distance in Feet		
Roadway	**85th Percentile Speed in MPH**	**Stopping (Minimum Intersection Sight Distance)**	**Case B1 (left turn from side street)**	**Measured Sight Distance from Intersection**
Richardson Highway SB	55	495	610	>1000
Richardson Highway NB	55	495	610	>1000
Valdez Airport Road at Salcha Way EB	40	305	445	335
Valdez Airport Road at Salcha Way WB	40	305	445	>1000
Atigun Drive at Salcha Way	30	200	335	>1000

Table 15- Sight Distance Summary

The field measured sight distance exceeds both minimum and desirable sight distance for all cases except for the intersection of Valdez Airport Road and Salcha Way which meets minimum sight distance.

5 PLANNING

5.1 Land Use and Zoning

The following graphic presents the current land use and zoning for the study area. Much of the study area is zoned light industrial with mixed land use largely consisting of government, single family residence, and commercial.

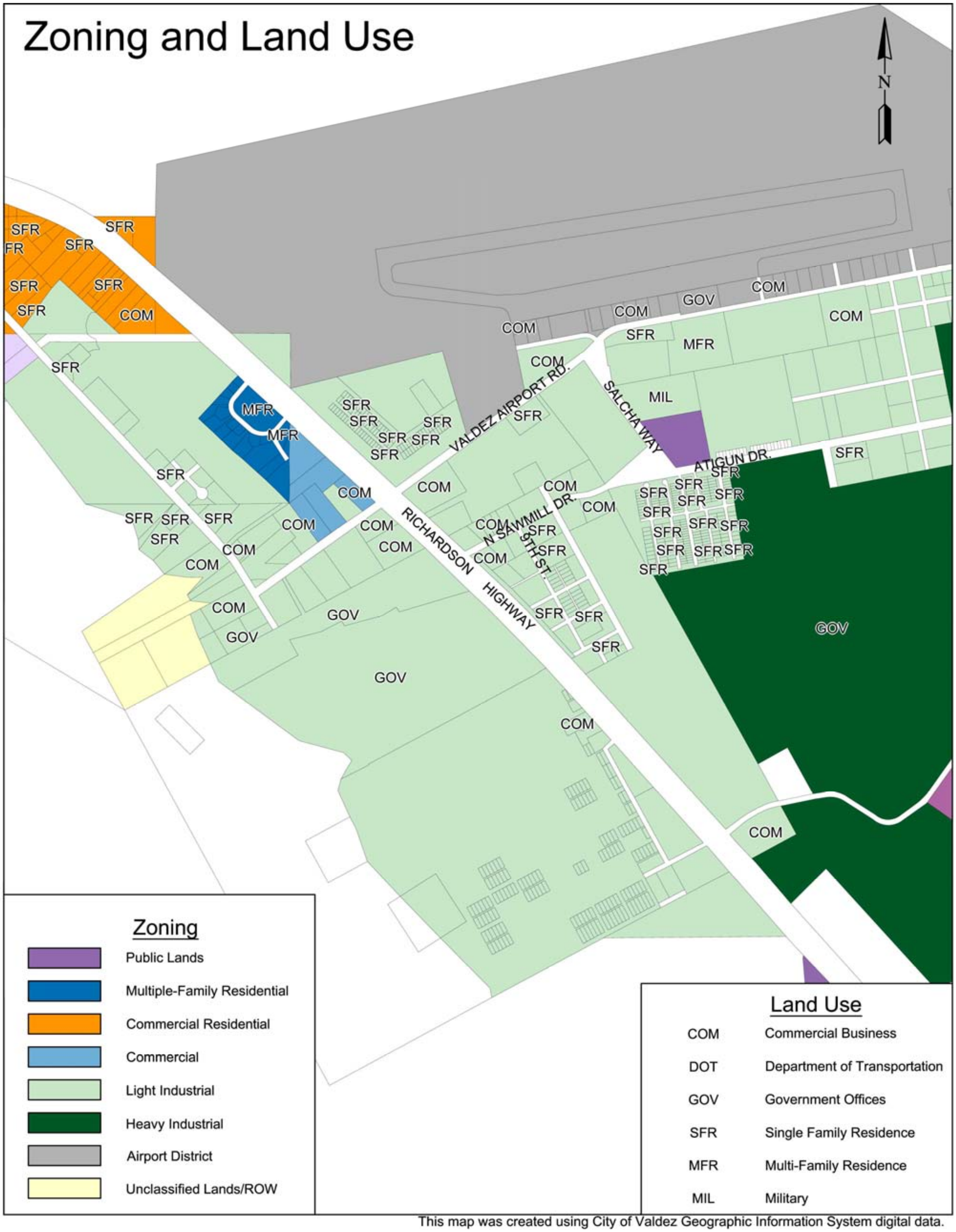


Figure 8- Study Area Land Use and Zoning

5.2 Potential Airport Industrial Subdivision

The following graphic presents the potential development of the Airport Industrial Subdivision.

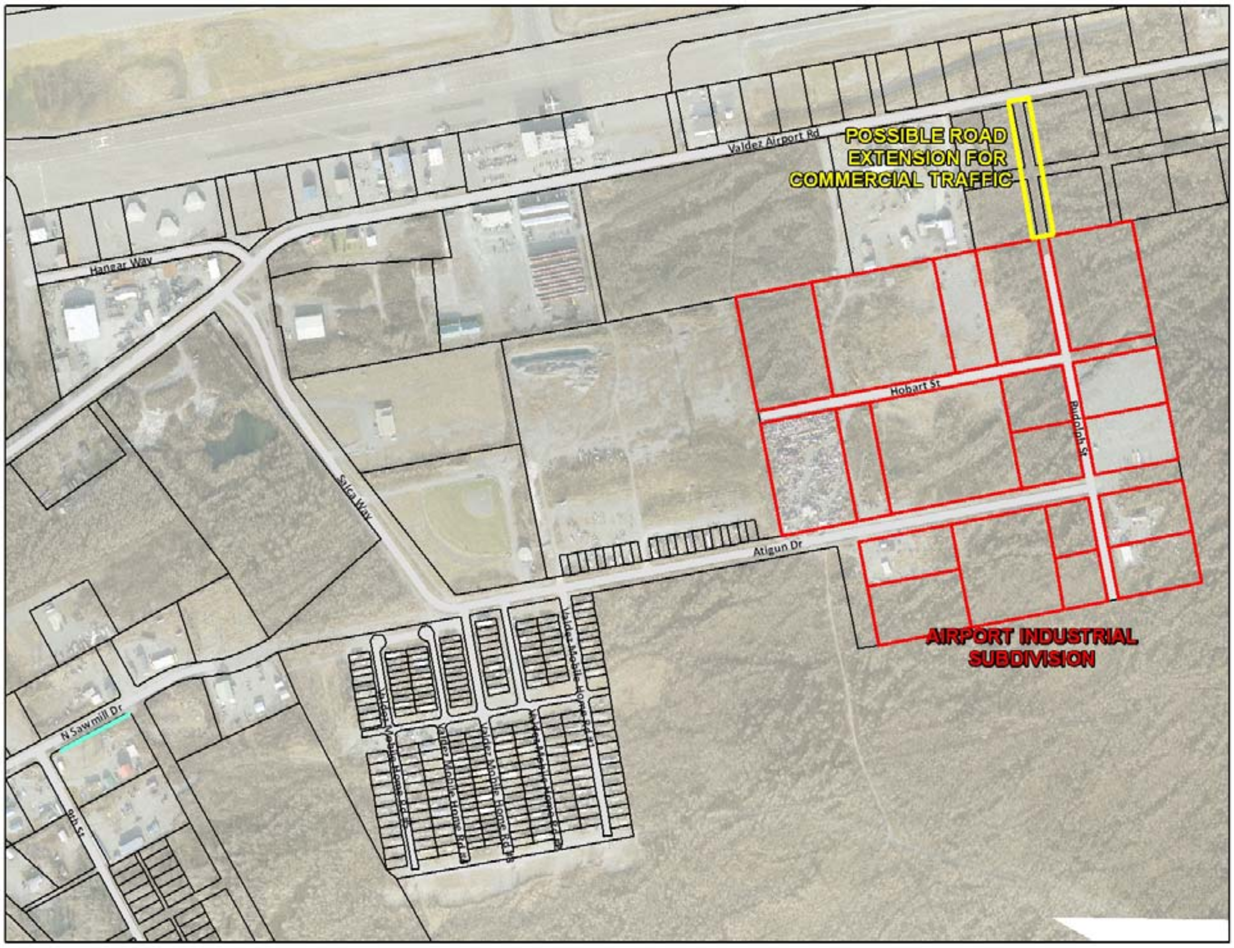


Figure 9- Airport Industrial Subdivision

Traffic generated by this development will be included in the design year model as the connection to the Valdez Airport road is not guaranteed and this traffic may need to use Atigun Drive to Salcha Way. Discussion of the trips expected to be generated by this development and their distribution to area roadways is included in Section 6.1.3.

5.3 Other Potential Development

The following graphic presents the study area with potential trip generating future development areas labeled.

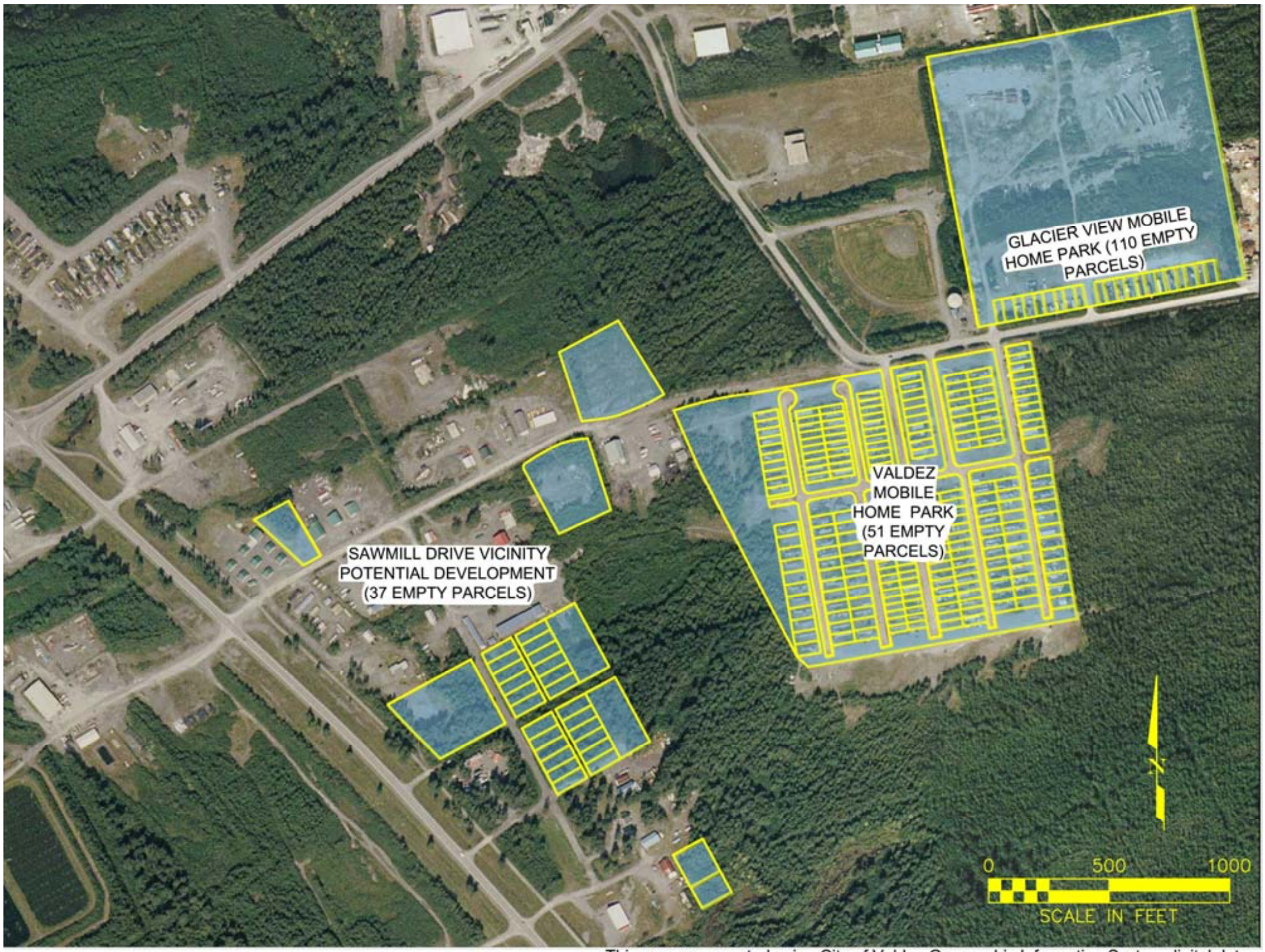


Figure 10- Potential Residential Development

5.3.1 Glacier View Mobile Home Park

As shown in Figure 10 the glacier View Mobile Home Park has the potential to develop 110 existing unoccupied parcels. This potential development and the related trips that will be generated are discussed further in Section 6.1.3.

5.3.2 Valdez Mobile Home Park

The Valdez Mobile Home Park shown in Figure 10 above currently has 51 unoccupied parcels which may develop into trip generating residences within the design horizon. These trips are discussed further in Section 6.1.3.

5.3.3 Sawmill Drive Vicinity

As shown in Figure 10, the vicinity of Sawmill Drive between Salcha Way and the Richardson Highway has the potential to develop multiple unoccupied lots. There are 37 lots that may develop into single family residences within the design horizon of this study. These lots and their contribution to future traffic volumes are discussed further in Section 6.1.3.

5.4 Valdez Population Based Studies

There are no travel demand models for Valdez. As such, this analysis uses population forecasts as a surrogate for general background traffic forecasts.

5.4.1 Alaska Economic Trends, October 2007

The Alaska Department of Labor and Workforce Development prepared a population projection for the state of Alaska and each of its major regions in October of 2007. This document predicts an average annual growth rate of 0.00% in the Valdez-Cordova census area for the period from 2007 to 2030.

5.4.2 Census History

The Valdez population was reported as 4068 in 1990 and 4036 in 2000. The population over this ten year period declined by 0.79% according to the census data.

6 TRAFFIC MODELS AND VOLUME PREDICTIONS

6.1 AADT

6.1.1 Background Traffic AADT Volumes

Kinney Engineering collected field data including turning movement counts and radar volume data to supplement the historical AADT and PTR data provided by the State ADOT&PF.

The permanent traffic recorder (PTR) on the Richardson Highway was used to establish peak hour volumes from AADT. Conversely, Kinney Engineering also used this data to derive conversions of peak hour volumes to AADT. The collected field data and historical data were then used to calculate a 2010 base volume for the existing condition. This base model is the model used to redistribute traffic for the 2010 build model and to add future development volumes for the 2020 build models.

6.1.2 Background Traffic Growth Rate

As discussed previously the population and historic AADT have shown a 0.79% negative growth rate for Valdez in the past years. As such, it is assumed that the background traffic may continue on this trend, and overall the community populations will remain reasonably close to current populations. Therefore, the base volumes in the model are not increased over time throughout the model. The build option simply redistributes the background 2010 AADT from another area of town to reflect the addition of the new connection between Sawmill Drive and Salcha Way.

6.1.3 Trip Generation and 2020 Traffic Models

As stated previously the background traffic is not expected to increase due to increases in population. However, several areas affecting the study area road network are expected to shift the location of trip generation sources through development. As shown in Section 5 a light industrial subdivision is expected to be developed at the eastern end of Atigun Drive. Also, the existing trailer courts on Atigun Drive and some of the properties along Sawmill Drive have development potential. The 2020 design year model accounts for these potential developments and assumes that they will develop within the planning horizon for this project. As opposed to the expected population growth for the entire area of Valdez, this potential for development is expected to be confined to the study area. The following sections discuss the trip generation methodology and results for these developments.

The Institute of Transportation Engineers (ITE) publishes the Trip Generation Manual. The version used for this study is the 8th Edition. Historical data was gathered by ITE and categorized according to land use such as single family residence or industrial park. For each of these categories data was accumulated and plotted on graphs with differing parameters such as acreage, population, or number of parcels. A regression line was then created to represent the data and its corresponding equation presented in the charts. These equations are then used to predict the number of trips expected by a future development.

The following table summarizes the units used, number of units, and expected trips generated by those units based on the trip generation manual charts for each of the expected trip generators.

				Trips Generated				
Trip Generator	**Category**	**Units**	**Potential Expansion Units**	**AADT**	**AM Peak Entering**	**AM Peak Exiting**	**PM Peak Entering**	**PM Peak Exiting**
Airport Industrial Subdivision	Industrial Park	Acres	74	4114	429	110	102	383
Valdez Mobile Home Park	Mobile Home Park	Parcels	51	458	6	26	21	13
Glacier View Mobile Home Park	Mobile Home Park	Parcels	110	664	11	42	41	27
Sawmill Drive Area	Single Family Detached Housing	Parcels	37	416	9	26	10	5

Table 16- AADT, AM, and PM Peak Hour Trips Generated by Expected Development

The additional trips were then added to the 2010 build model to determine the future AADT numbers. The following graphic depicts the distribution of new traffic generated by the above trip generators per hundred vehicles to illustrate how the new traffic filtered through the roadway network to and from the trip generators.

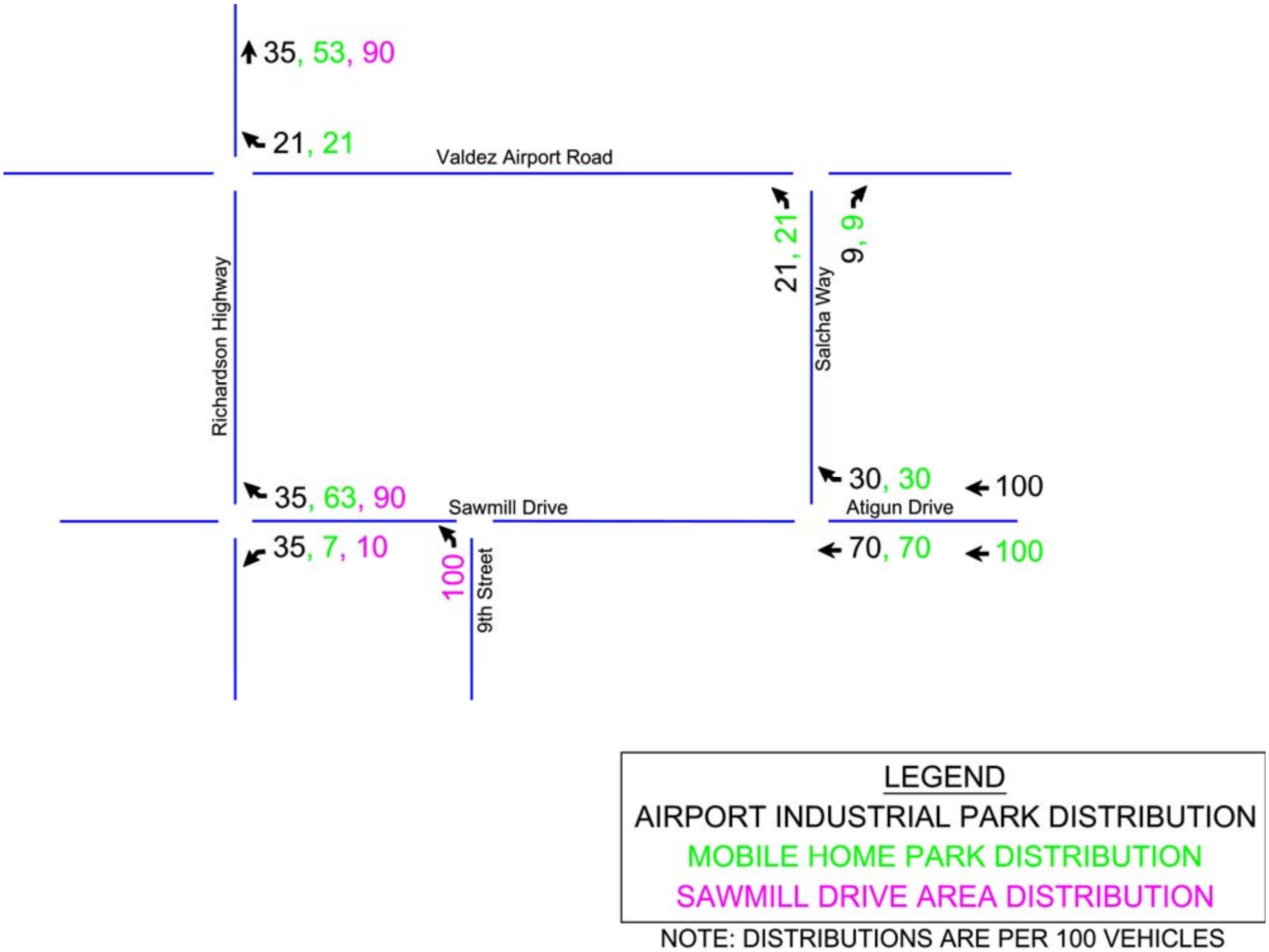


Figure 11- Development Traffic Distribution

Based on the additional traffic numbers the following graphic was prepared to illustrate the expected AADT for the 2010 base model, the 2010 build model, the 2020 build model and a modification of the 2020 build model that does not include trips generated by the Airport Industrial park subdivision.

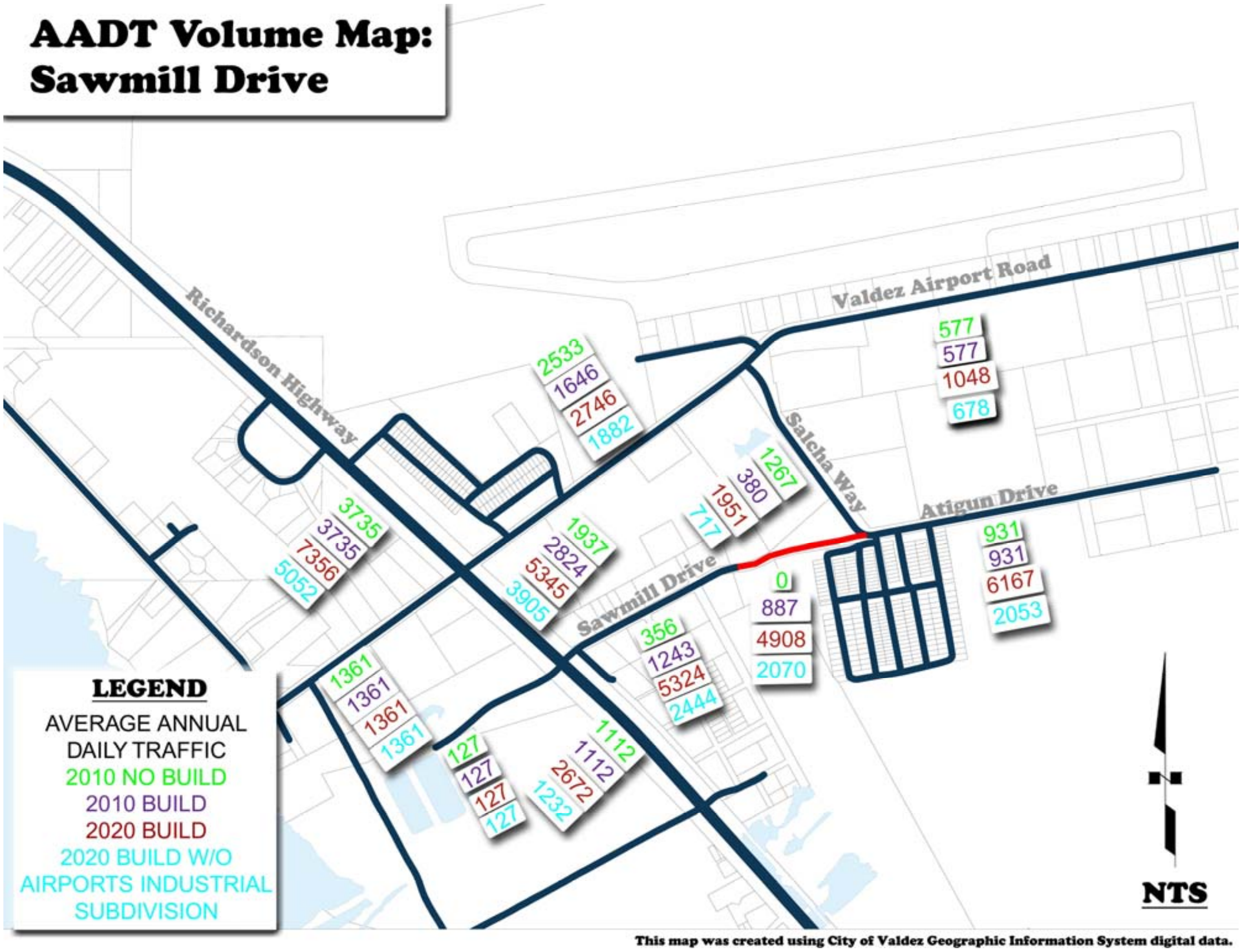


Figure 12- Base and Future Traffic Model AADT

The following section discusses the operational impact analysis that Kinney Engineering performed to assess the impacts of the above traffic volumes on the roadway network.

7 ALTERNATIVE ANALYSIS AND RECOMMENDATIONS

7.1 Sawmill Drive Extension Functional Classification

Sawmill Drive will collect and distribute traffic from the intersecting local streets and individual properties to the east of Richardson Highway. As such, the street is in a collector position within the area's street network in which the hierarchical system of traffic flow; local to collector to arterial and vice-versa; would be attained. As shown in Figure 12 on page 26, the 2020 AADT on Salcha Drive would be between 2,500 and 5,300, depending upon the development level, which is within the collector road volume range (Anchorage collectors range from 2,000 to 10,000 AADT). In recognition of these factors, Sawmill Drive will likely function as a minor collector and it is recommended that this project use that functional classification.

7.2 Sawmill Drive Typical Section

It has been proposed by the City that Sawmill Drive be constructed with a 24-foot driving surface. As will be discussed under Section 7.3 below, pavement is recommended for this street.

If the future development is limited to residential dwellings, without the industrial development, then the roadway would function as a residential or neighborhood collector. The Municipality of Anchorage has developed standards for collector streets which could be applied to this roadway. Under that methodology, with an AADT of 2,500 (rounded), 2% trucks (observed), 50/50 directional split, and 10% design hour volume (estimated), Anchorage would require 10-foot lanes and 3.5-foot shoulders and curb/gutters for the collector. This strip paved section could be adapted to provide striped 10-foot lanes, 2-foot paved shoulders, and 4-foot gravel shoulders.

Upon development of the industrial area, both the volumes and % trucks would increase, and the above described section would be inadequate. With increased trucks and volume, 12-foot lanes with additional paved shoulder width (4 to 8 feet) would be recommended.

7.3 Sawmill Drive Surface

The existing surface of Sawmill Drive is gravel. It has been shown that the new extension is expected to significantly increase the traffic using Sawmill Drive. The 2010 build AADT is estimated to be 1243 AADT with 5345 AADT in the 2020 design year. Guidance concerning the decision to pave or not to pave the roadway can be found in FHWA publication number FHWA-CFL/TD-05-004, Context Sensitive Roadway Surfacing

Selection Guide. This publication is intended to aid designers in making decisions concerning what type of surface treatment to apply on a given roadway based on the roadway use, setting, traffic parameters etc. Pertinent tables from this publication are shown below.

Design Volume (Vehicles/day)	Suggested Descriptive Term	Design Speed (mph)	
		Preferred	Minimum
< 200	Very Low	40	30
200 – 400	Low	50	40
400 – 1000	Medium	50	40
1000 – 4000	High	55	45
4000 – 8000		60	50
>8000		60	50

Table 17- Volume Classifications

Road Surfacing Type	Traffic			
	Very Low	Low	Medium	High
Asphalt Surfacing (non-structural)				
Cape Seal	A	A	A	B
Chip Seal	A	A	A	B
Chip Seal over Geotextile	A	A	A	B
Fog Seal	A	A	B	C
Microsurfacing	A	A	A	A
Multiple Surface Treatments (Seals)	A	A	A	B
Open Graded Friction Course	A	A	A	A
Otta Seal	A	A	B	C
Sand Seal	A	A	B	C
Scrub Seal	A	A	A	C
Slurry Seal	A	A	A	B
Ultrathin Friction Course	A	A	A	A
Asphalt Surfacing (structural)				
Cold Mix Asphalt Concrete Pavement	A	A	A	B
Hot Asphalt Concrete Pavement (HACP)	A	A	A	A
Exposed Aggregate HACP	A	A	B	C
Imprinted / Embossed HACP	A	A	B	C
Pigmented HACP	A	A	A	A
Porous HACP	A	A	C	X
Resin Modified Pavement	A	A	A	A
Synthetic Binder Concrete Pavement	A	A	A	A
Portland Cement Concrete (PCC) Surfacings				
Cellular PCC	A	A	B	X
Portland Cement Concrete Pavement (PCCP)	A	A	A	A
Exposed Aggregate PCCP	A	A	A	B
Pigmented PCCP	A	A	A	B
Porous PCCP	A	A	A	C
Stamped PCCP	A	A	B	C
Roller Compacted Concrete	A	A	A	B
Whitetopping	A	A	A	A
Unbound & Mechanically Stabilized Surfacings				
Cellular Confinement	B	B	C	X
Fiber Reinforcement	B	C	X	X
Geotextile/Geogrid Reinforcement	B	C	C	X
Gravel (crushed or uncrushed)	B	C	X	X
Sand	C	X	X	X
Other Stabilized Surfacings				
Chlorides	B	C	X	X
Clay Additives	B	C	X	X
Electrolyte Emulsions	B	C	X	X
Enzymatic Emulsions	B	C	X	X
Lignosulfonates	B	C	X	X
Organic Petroleum Based Emulsions	B	C	C	X
Synthetic Polymer Emulsions	A	B	C	X
Tree Resin Emulsions	A	B	C	X
Unit Surfaces				
Brick Pavers	A	A	B	C
Natural Stone Cobbles	B	B	C	X
Unit Pavers	A	A	A	A
Porous Unit Pavers	B	B	B	C
Recycling Alternatives				
Hot In-Place Recycling	A	A	A	A
Recycled HACP	A	A	A	A

A:	Highly suitable
B:	Acceptable for use
C:	Not ideal, but can be used
X:	Not suitable
	Not applicable

Table 18- Suggested Suitability Designations for Screening Stage

These AADT values place Sawmill Drive in the “high” category according to this guidance. Based on this information, these volumes limit the surfacing choices to the following list assuming that only “highly suitable” treatments are acceptable.

- Microsurfacing
- Ultrathin Friction Course
- Hot Asphalt Concrete Pavement(HACP)
- Pigmented HACP
- Resin Modified Pavement
- Synthetic Binder Concrete Pavement
- Portland Cement Concrete Pavement
- Whitetopping
- Unit Pavers
- Hot In-Place Recycling
- Recycled HACP

Some of these surface treatments may be eliminated early in the selection process based on cost or practicality such as unit pavers or hot in-place recycling. All of the “highly suitable” treatments for Sawmill Drive involve some form of paved surface treatment. Unbound, mechanically stabilized, and other stabilized surfaces are not considered “highly suitable” for this volume of traffic. It is recommended to construct a paved surface treatment on Sawmill Drive throughout its length. The selection of the final design surface is beyond the scope of this document and should be determined during final design.

7.4 Pedestrian and Bicycle Facilities

As part of the construction of the new connection between the Sawmill Drive extension and the Atigun Drive/Salcha Way intersection, the existing pathway on the east side of Salcha will need to be reconfigured to accommodate the intersection improvements. In addition, since there is a substantial residential development planned in this area, a new pathway along the Sawmill Drive extension should be considered for future implementation to provide a direct corridor to the Richardson Highway facilities and promote non-motorized travel.

7.5 Intersection Configuration and Control

7.5.1 Peak Hour Turning Movement Volumes

The following graphics present the peak turning movement volumes calculated for the 2010 no-build, 2010 build, 2020 build, and 2020 build alternative without the Industrial Airport subdivision development.

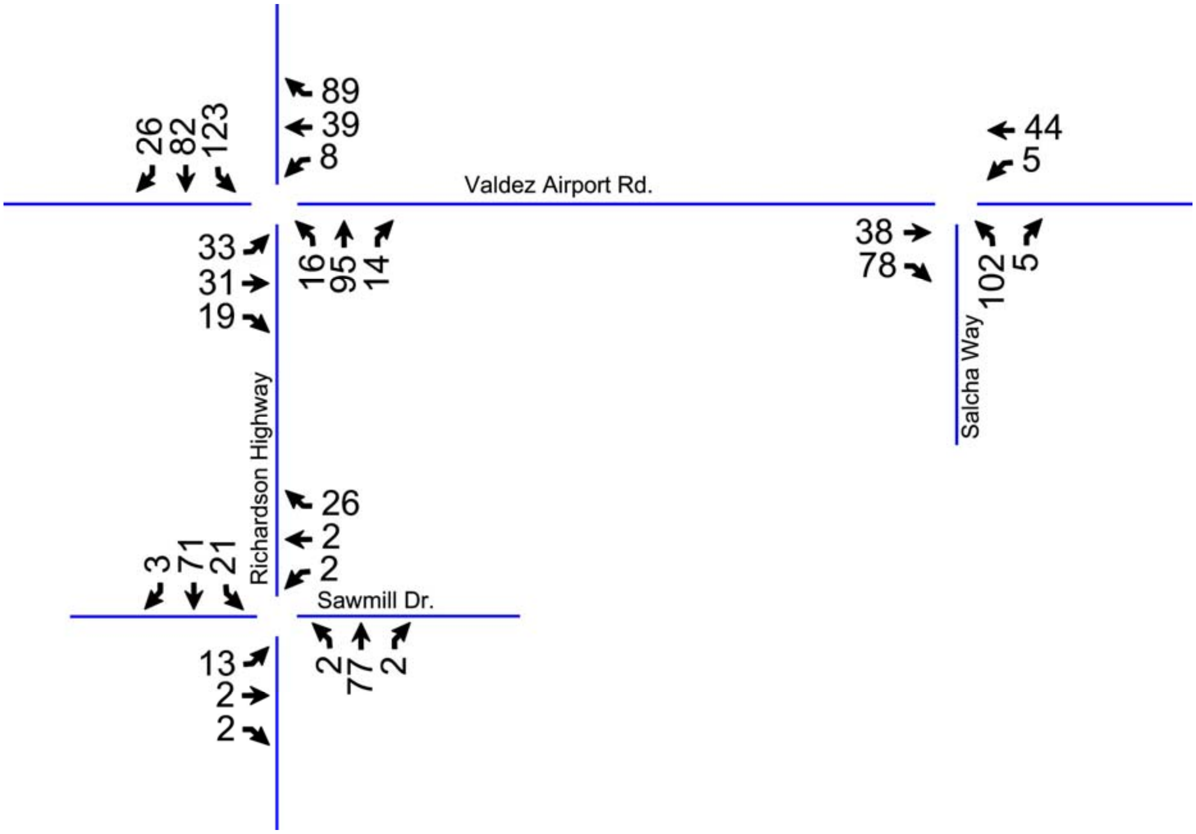


Figure 13- Peak Hour Turning Movements, 2010 Base

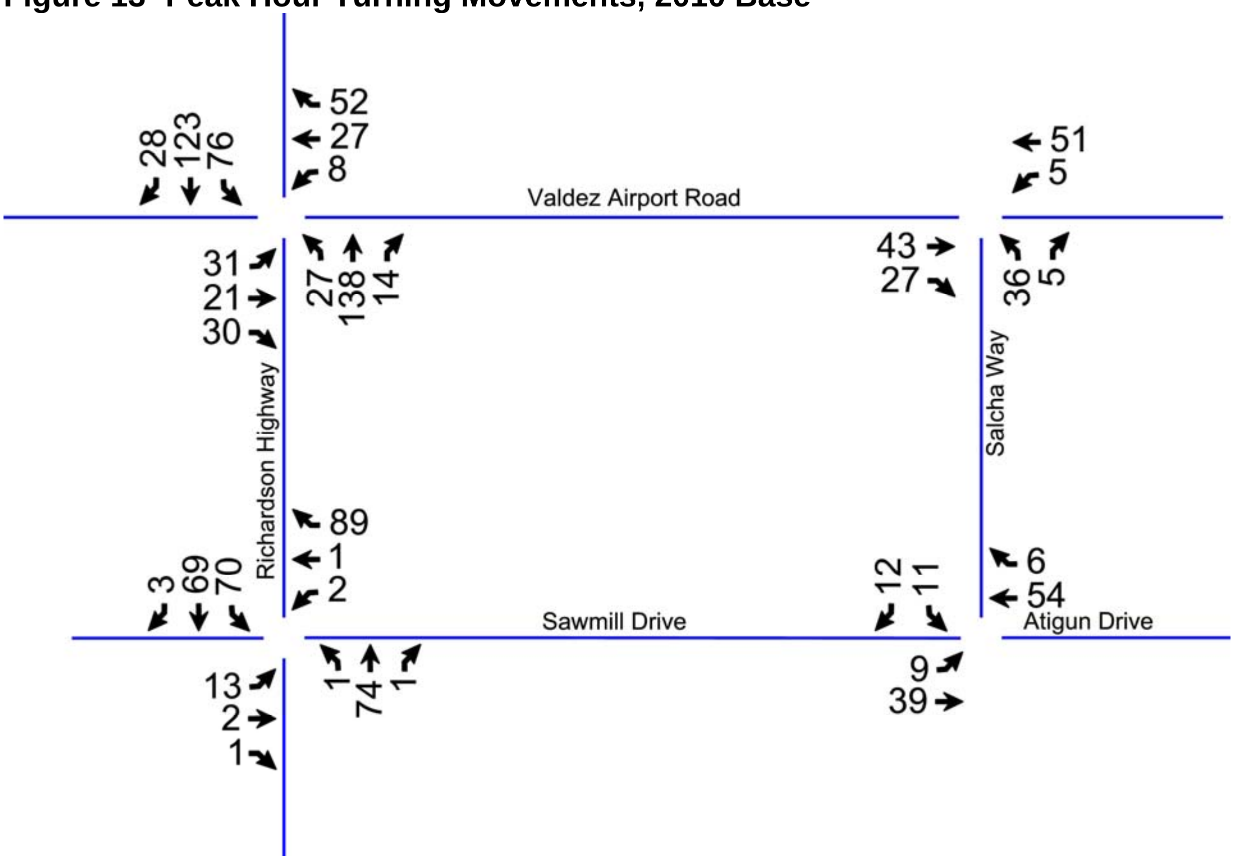


Figure 14- Peak Hour Turning Movements, 2010 Build

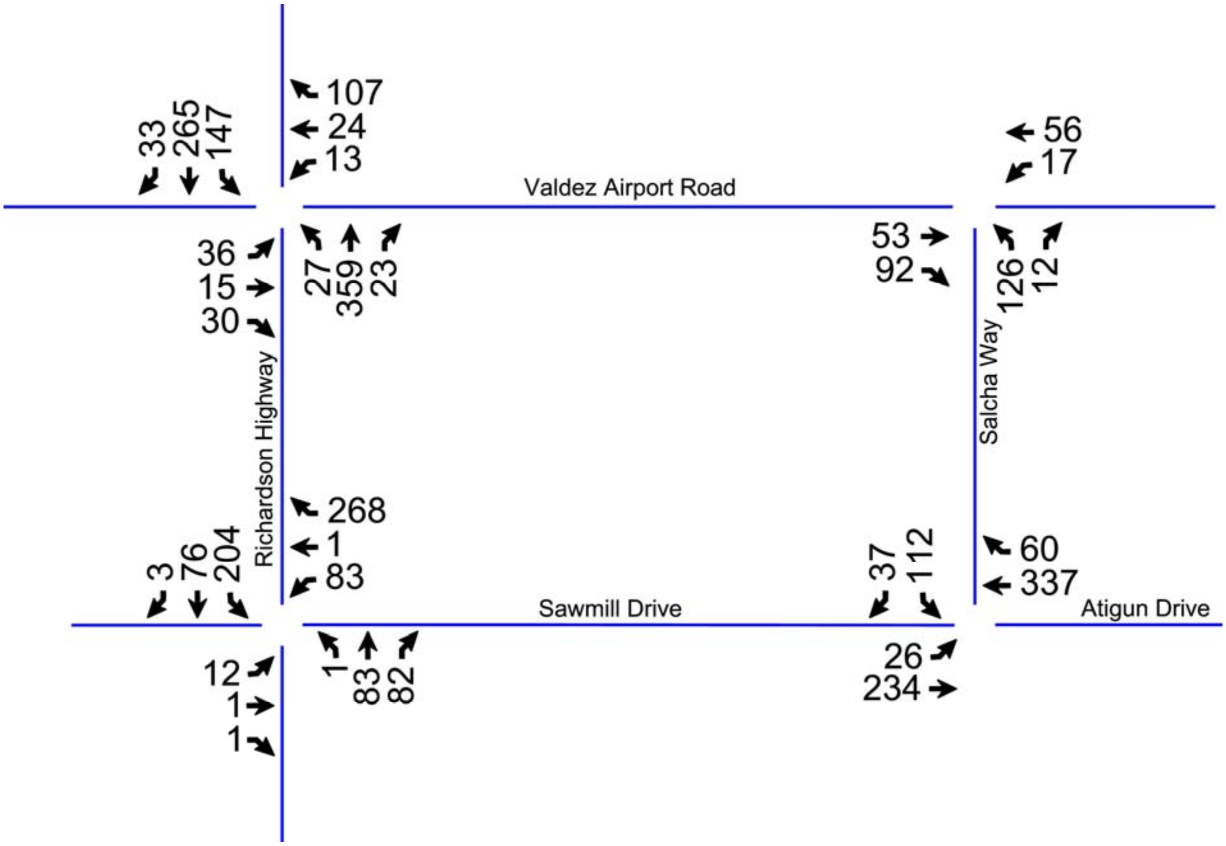


Figure 15- Peak Hour Turning Movements, 2020 Build

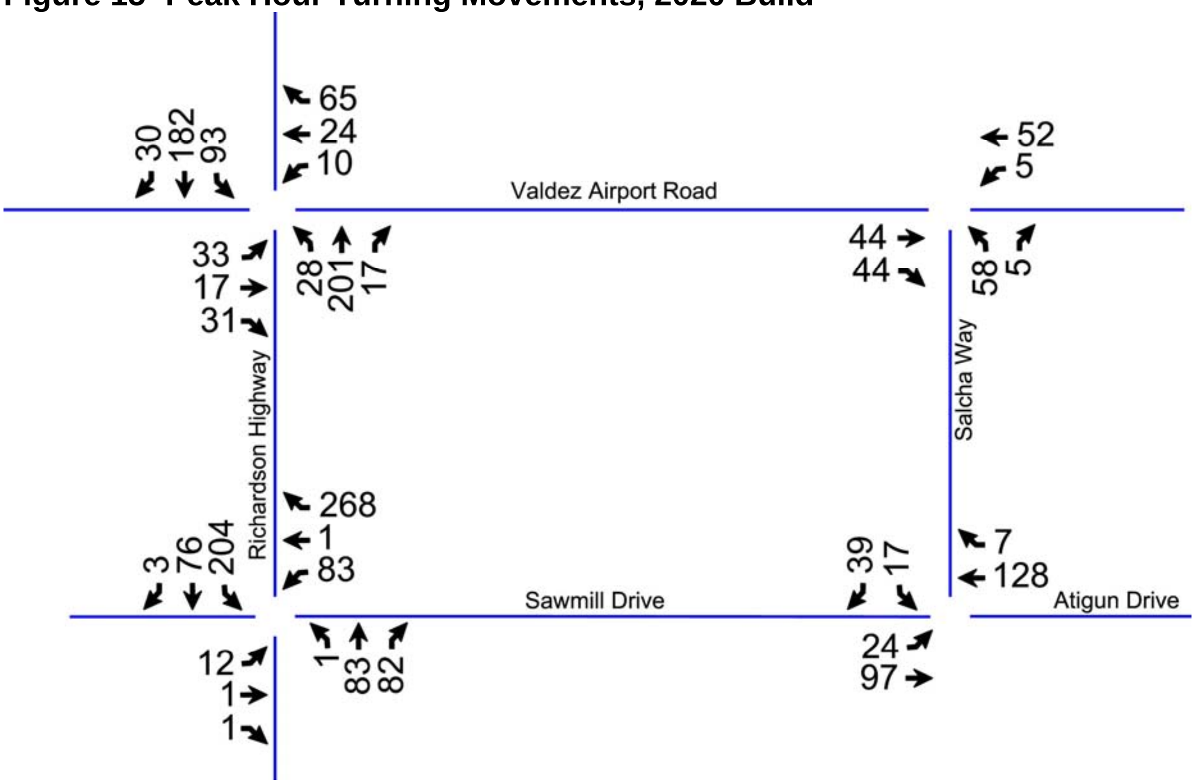


Figure 16- Peak Hour Turning Movements, 2020 build - No Industrial Airport Subdivision

7.5.2 Future Signal Warrants

An option for intersection control other than the two way stop controlled condition described above is to install a traffic signal. Although a traffic signal may provide a higher level of service for the minor approaches, it would impose delay to the major approach traffic.

Intersections must meet specific thresholds to warrant a traffic signal. Appendix C- Intersection Signal Warrants presents signal warrant analysis methodology for evaluating whether an intersection warrants a signal.

A future signal warrant analysis was conducted using the Cal-Trans method of evaluating whether a signal will likely meet future warrants based on predicted traffic volumes for the intersections of the Richardson Highway with both Sawmill Drive and Valdez Airport Road. No warrants are likely to be met based on predicted traffic volumes.

7.5.3 Intersection Auxiliary Lanes

7.5.3.1 <u>Left Turn Lanes on Major Approaches of Unsignalized Intersections</u>

Major approaches of unsignalized intersections are free-flow, but the left-turning vehicles within a shared movement lane must stop and yield to oncoming through traffic and thus become targets for rear-end collisions. Left-turn auxiliary lanes will separate conflicts between stopped or slowing left-turning vehicles and the through traffic stream; which decreases potential rear-end crashes, as well as reduces delay for through traffic. As discussed under Section 4.1.2 on page 15, most, at least 5 rear-end and sideswipe collisions during the study period were attributed to left-turning vehicles on the major highway approaches at the Richardson/Valdez Airport intersection; all of which would have been correctable by left-turn lanes. The ADOT&PF's Highway Safety Improvement Program Handbook lists a crash reduction factor of 50% for left-turn lanes on the major street approach of unsignalized urban intersections.

Exhibit 9-75 on page 685 of the AASHTO A Policy on the Geometric Design of Highways and Streets, provides a guide for left turn lanes on two way highways based on volumes. This methodology is also presented in NCHRP 457 by a spreadsheet tool (Figure 2-5). The following table presents the left turn lane analysis based on the AASHTO procedure. The computations of the NCHRP 457 spreadsheet tool are presented in Appendix A- Auxiliary Turn Lane Analysis.

Approach	**Operating Speed (mph)**	**Opposing Volume (veh/h)**	**Advancing Volume (veh/h)**	**Left Turn Volume (veh/h)**	**% of Left Turns**	**Left Turn Lane Recommended? See note 1**
Southbound Richardson Highway at Valdez Airport Road	55	313	445	147	33.03%	Yes
Northbound Richardson Highway at Valdez Airport Road	55	298	338	25	7.40%	No
Southbound Richardson Highway at Sawmill Drive	55	165	283	204	72.08%	Yes
Northbound Richardson Highway at Sawmill Drive	55	79	166	1	0.60%	No
Westbound Valdez Airport Road at Salcha Way	40	145	73	17	23.29%	No
Eastbound Sawmill Drive at Salcha Way	30	397	260	26	10.00%	No
Note 1: Decision based on using the data with AASHTO Exhibit 9-75, page 685 of the Policy on the Geometric Design of Highways and Streets (also known as the "Green Book")						

Table 19 - Left Turn Lane Analysis (Based Upon 2020 Full Build Scenario)

Both southbound approaches to the intersections of the Richardson highway with Valdez Airport Road and with Sawmill Drive should have left turn lanes according to AASHTO. AASHTO recommends that whenever practical left-turning traffic should be removed from the through lanes. When a left turn lane is required on an approach but not the opposing approach it is practical to install a left turn lane on the opposing approach since the pavement width is provided and a left turn lane can be installed on the opposing approach with less costs and less impacts than if the lane were installed separately. As such a left turn lane is recommended for the northbound Richardson Highway Approaches to the intersections with Sawmill Drive and with Valdez Airport Road.

7.5.3.2 Right-Turn Lanes on Major Approaches of Unsignalized Intersections

Right-turning vehicles on the major approaches of unsignalized intersections slow to complete the turn, and create a differential speed conflict with following through vehicles. If volumes and speeds are high enough to create significant conflict frequencies, then rear end crashes may result. More importantly, if the turns are unexpected, as could be the case on a rural or suburban two-lane highway, the potential would increase as well. As discussed under Section 4.1.2 on page 15, at least 2 rear-end and sideswipe collisions during the study period were attributed to right-turning vehicles on the major highway northbound approach at the Richardson/Valdez Airport intersection; all of which are correctable by right-turn lanes. Northbound right-turners may be more susceptible to collisions, because northbound drivers may not be fully aware that they've entered in the suburban part of Valdez and turning vehicles are not expected. The ADOT&PF's Highway Safety Improvement Program Handbook lists a crash reduction factor of 24% for right-turn lanes on the major street approach of unsignalized rural intersections (no listing for urban or suburban intersections).

Both NCHRP 457 (Figure 2-6) and NCHRP's Report 279, *Intersection Channelization Design Guide*, Figure 4-23 present guidelines for installation of right-turn lanes on two-lane highways. Both of these guidelines compare general cost of right-turn lanes and However, the NCHRP 457 method is founded on right-turns in rural environments, and may not apply to this section of the Richardson Highway. As such, the NCHRP 279 methodology was applied to the major approaches right-turn 2020 design volumes for the Richardson Highway.

Since Valdez Airport Road, Sawmill Drive, and Salcha Way are lower speed roads, this analysis would not apply, and instead, capacity requirements determine right-turn lane needs.

The Richardson Highway approach analyses are presented in Appendix A- Auxiliary Turn Lane Analysis. No full width right turn lanes are recommended by the application of these guidelines. The procedure recommends a tapered right-turn treatment for the northbound approach for the Richardson Highway/Sawmill intersection.

7.5.4 Unsignalized Intersection Operational Analysis

The intersections within the study area are modeled as two-way stop controlled intersections where the minor approaches are stop controlled. With regards to vehicular operational quality, the primary performance measure is level of service, with levels A (best, free-flow) through F (failed, long delays). The methodology for unsignalized intersections only computes LOS for the minor movements of the intersection, which include the minor street approaches under sign control, or major movements that must yield to oncoming traffic, such as left-turning traffic. Unsignalized LOS is defined as follows (HCM Exhibit 17-2):

- LOS A: ≤10 seconds of control delay per vehicle
- LOS B: >10 and ≤15 seconds of control delay per vehicle
- LOS C: >15 and ≤25 seconds of control delay per vehicle
- LOS D: >25 and ≤35 seconds of control delay per vehicle
- LOS E: >35 and ≤50 seconds of control delay per vehicle
- LOS F: >50 seconds of control delay per vehicle

AASHTO's GDHS 2004, Exhibit 2-32 provides guidelines for design levels of service of functionally classed facilities which indicate that urban or suburban collectors may have a LOS of D. Therefore, all minor streets should have a LOS D or better through the design year. Richardson Highway, an arterial that is in a suburban setting within Valdez, should have a LOS of C or better to comply with AASHTO's guidelines.

An HCM unsignalized intersection analysis was performed to estimate the level of service (LOS) for each of the stop controlled approaches. Auxiliary turning lanes are added to the major approaches of the Richardson Highway intersections based on the recommendations discussed under Sections 7.5.3.2 and 7.5.3.1 above. In addition, auxiliary lanes were added to the cross street intersections to optimize approach levels of service to the extent feasible for the forecasted traffic under the 2020 full development scenario. Figure 17 on page 37 presents the future lane configurations that would be recommended for 2020 full development.

The approach LOS for each of the minor approaches within the study area is summarized in Table 20 on page 38. These LOS are for the minor street lane intersection approach configuration alternatives shown where a minor approach left turn lane is not shared. If the secondary alternative were to be implemented, that is the approach would have a shared through/left and right-turn lane configuration for the stopped approach, the right-turn LOS improves but the through/left LOS declines.

The HCM unsignalized intersection analysis using Synchro is presented in Appendix D-Capacity Analysis Reports

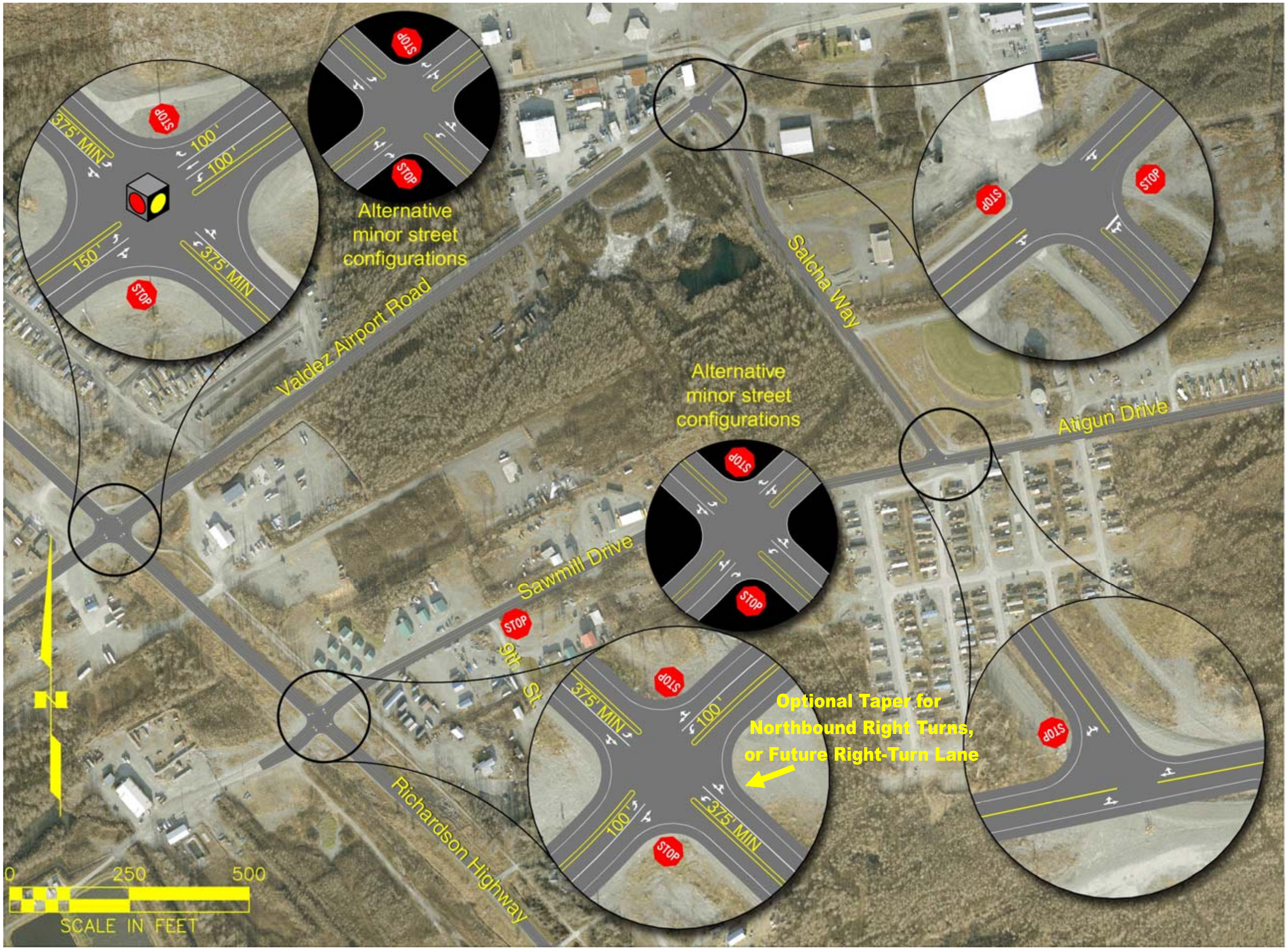


Figure 17- Recommended Lane Configurations, for 2020 Full Development

Intersection	Approach/Movement	2010	2010 Build, No Improvements to Existing Intersections	2020 Full Development (Design Case, Figure 17)	2020 Without Industrial Park No Improvements to Existing Intersections	2021 Without Industrial Park, with Improved Intersections
Richardson Highway at Valdez Airport Road	Westbound Left	C (1 Lane)	C (1 Lane)	F *	C (1 Lane)	D
	Westbound Through			E *		C (Thru/Rt in 1 lane)
	Westbound Right			B *		[redacted]
	Eastbound Left	D (1 Lane)	C (1 Lane)	F **	E (1 Lane)	E
	Eastbound Through & Right			C **		B
Richardson Highway at Sawmill Drive	Westbound Left	A (1 Lane)	A (1 Lane)	C	A (1 Lane)	B
	Westbound Through & Right			B		A
	Eastbound Left	B (1 Lane)	B (1 Lane)	E ***	C (1 Lane)	C
	Eastbound Through & Right			B ***		B
Valdez Airport Road at Salcha Way	Northbound (all movements)	A	A	B	A	A
Salcha Way at Sawmill Drive	Southbound (all movements)	[redacted]	A	C	A	A

* Approach LOS is C
**Approach LOS is F
***Approach LOS is E

Table 20- Unsignalized Intersections Level of Service by Approach Lane Group

For the design horizon year of 2020 two alternative scenarios were analyzed to assess the impacts of full development of the planned Airport Industrial Park. Table 20 illustrates that upon full development of the Airport Industrial Park, the Richardson Highway intersection minor approach level of service is below desirable minimums even with the installation of additional turn lanes on the approaches to these intersections.

7.5.4.1 Auxiliary Turn Lane Geometry

Auxiliary lanes on major road, free-flow approaches should accommodate queues, and in-lane deceleration for design speeds that are greater than 35 mph (Highway Preconstruction Manual, Section 1150). Where design speeds are 35 mph or less, Section 1150 only requires the lane to be long enough to store queues.

The desirable length of an auxiliary turn lane on a free-flow approach would allow a vehicle to enter the back of the lane at the design speed, and decelerate to a stop behind the 95th percentile queue. A minimum lane length is developed by assuming that the vehicle enters the bay taper at a speed that is 10 mph slower than the design speed, and then begins deceleration at about 2/3 through the bay taper. The auxiliary lane length, then, is the sum of the lane needed for reduced deceleration length and the 95th percentile queue. The minimum lane length is 100 feet.

For approaches that are under stop sign control, auxiliary lanes are only required to be long enough to store queues.

The following table summarizes the recommended components of right and left turn lanes for the 2020 full development configuration that is presented in Figure 17 on page 37.

	NBL	SBL	EBL	WBL	WBR
Richardson Highway & Valdez Airport Road					
Queues (ft.)	25	25	155	25	30
Minimum Lane Length (ft.)	375	375	150	100	100
Desirable Lane Length (ft.)	600	600	150	100	100
Bay Taper Rate	15:1	15:1	6:1	6:1	6:1
Richardson Highway & Sawmill Drive					
Queues (ft.)	25	25	25	40	
Minimum Lane Length (ft.)	375	375	100	100	
Desirable Lane Length (ft.)	600	600	100	100	
Bay Taper Rate	15:1	15:1	6:1	6:1	

Table 21- Auxiliary Lanes Components

7.5.4.2 Sawmill Drive/ Salcha Way/Atigun Drive Intersection Configuration

Two options were considered for the connection of the Sawmill Drive extension to Salcha Way/Atigun Drive. The first option would connect the extension at the existing corner where Atigun Drive turns into Salcha Way. The second option would be a new connection further to the north along Salcha Way across from the ball field.

The expected volumes using the extended Sawmill Drive are high when compared to the volumes expected on Salcha Way. Particularly for the 2020 build assuming full development of the trailer parks and airport subdivision. This volume condition supports connecting at the existing corner since the higher volumes using the extension would be under a free-flow condition with Salcha Way as the stopped approach. Connecting at the existing corner also provides unobstructed sight lines for the stop controlled Salcha Way approach. Connecting further to the north would mean that the existing corner was within the sight lines of a vehicle stopped on Sawmill Drive which may restrict sight distance particularly during the winter when snow is banked on the side of the road. As such it is recommended to connect the Sawmill Drive extension at the existing corner where Atigun Drive turns into Salcha Way.

7.6 Lighting

According to “The Traffic and Safety Features Design Guide (preferred design practices), version 1” prepared by ADOT in April of 2005….the preferred practices concerning intersection illumination is to “*Use a minimum of 2 fixtures to provide silhouette lighting at any intersection with auxiliary turn lanes” and “All LT pockets from the beginning point of the lane shift and widening taper to the intersection.”* As such, lighting should be provided for the project intersections to illuminate the left turn lanes from the beginning of the widening taper.

APPENDIX A- AUXILIARY TURN LANE ANALYSIS

<u>Left-Turn Lane Guidelines with NCHRP 457 Spreadsheet Tools</u>

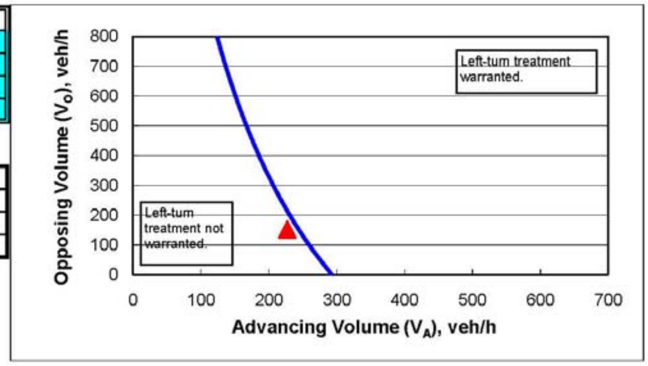
Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

2-lane roadway (English)
INPUT

Variable	Value
85th percentile speed, mph:	55
Percent of left-turns in advancing volume (V_A), %:	33%
Advancing volume (V_A), veh/h:	227
Opposing volume (V_O), veh/h:	152

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	244
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane , s:	1.9

Figure 18- Left Turn Lane Analysis, Southbound Richardson Highway at Valdez Airport Road, Design Year (2020)

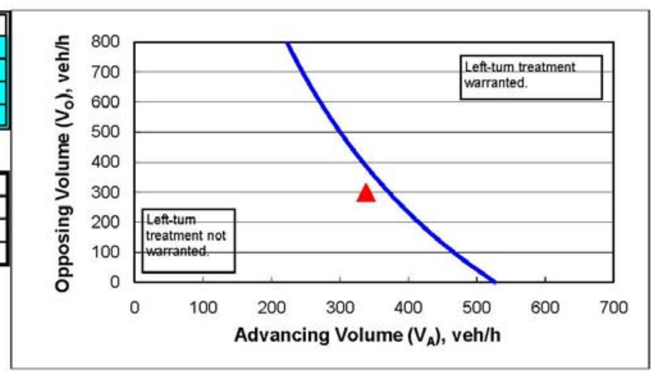
Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

2-lane roadway (English)
INPUT

Variable	Value
85th percentile speed, mph:	55
Percent of left-turns in advancing volume (V_A), %:	7%
Advancing volume (V_A), veh/h:	338
Opposing volume (V_O), veh/h:	298

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	372
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane , s:	1.9

Figure 19- Left Turn Lane Analysis, Northbound Richardson Highway at Valdez Airport Road, Design Year (2020)

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

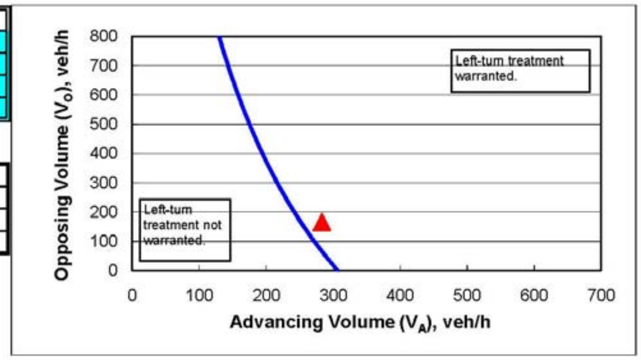
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	55
Percent of left-turns in advancing volume (V_A), %:	72%
Advancing volume (V_A), veh/h:	283
Opposing volume (V_O), veh/h:	165

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	252
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment warranted.	



CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane , s:	1.9

Figure 20- Left Turn Lane Analysis, Southbound Richardson Highway at Sawmill Drive, Design Year (2020)

Figure 2 - 5. Guideline for determining the need for a major-road left-turn bay at a two-way stop-controlled intersection.

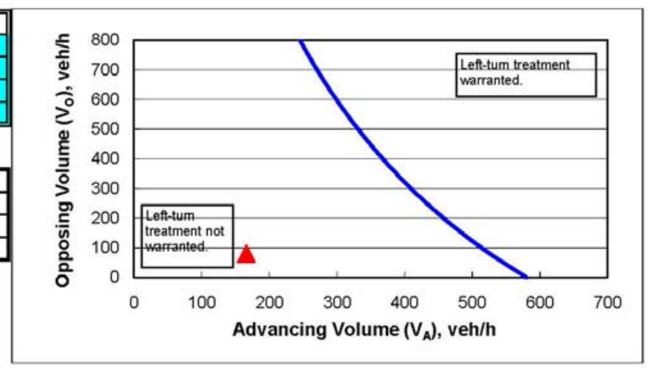
2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	55
Percent of left-turns in advancing volume (V_A), %:	6%
Advancing volume (V_A), veh/h:	166
Opposing volume (V_O), veh/h:	79

OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	526
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	



CALIBRATION CONSTANTS

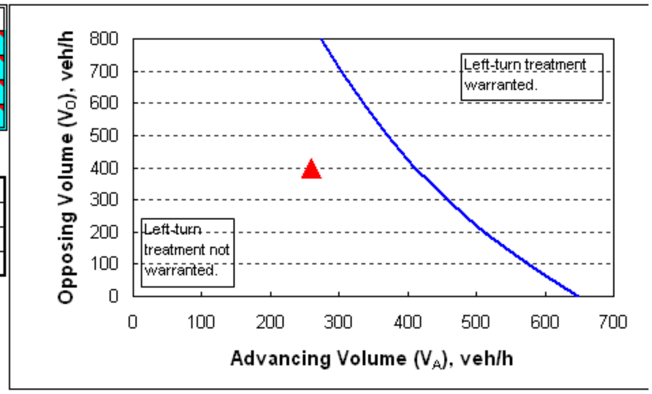
Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane , s:	1.9

Figure 21- Left Turn Lane Analysis, Northbound Richardson Highway at Sawmill Drive, Design Year (2020)

2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	30
Percent of left-turns in advancing volume (V_A), %:	10%
Advancing volume (V_A), veh/h:	260
Opposing volume (V_O), veh/h:	400



OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	411
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	

CALIBRATION CONSTANTS

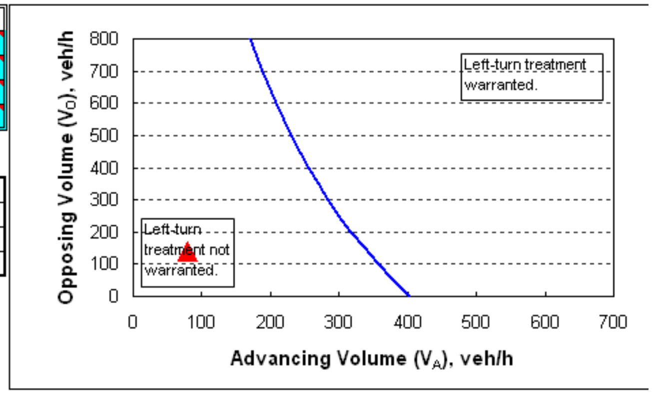
Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 22- Left Turn Lane Analysis, Eastbound Sawmill Drive at Salcha Way, Design Year (2020)

2-lane roadway (English)

INPUT

Variable	Value
85th percentile speed, mph:	40
Percent of left-turns in advancing volume (V_A), %:	25%
Advancing volume (V_A), veh/h:	80
Opposing volume (V_O), veh/h:	140



OUTPUT

Variable	Value
Limiting advancing volume (V_A), veh/h:	340
Guidance for determining the need for a major-road left-turn bay:	
Left-turn treatment NOT warranted.	

CALIBRATION CONSTANTS

Variable	Value
Average time for making left-turn, s:	3.0
Critical headway, s:	5.0
Average time for left-turn vehicle to clear the advancing lane, s:	1.9

Figure 23- Left Turn Lane Analysis, Westbound Valdez Airport Road at Salcha Way, Design Year (2020)

Right-Turn Lane Guidelines with NCHRP 279 Spreadsheet Tools

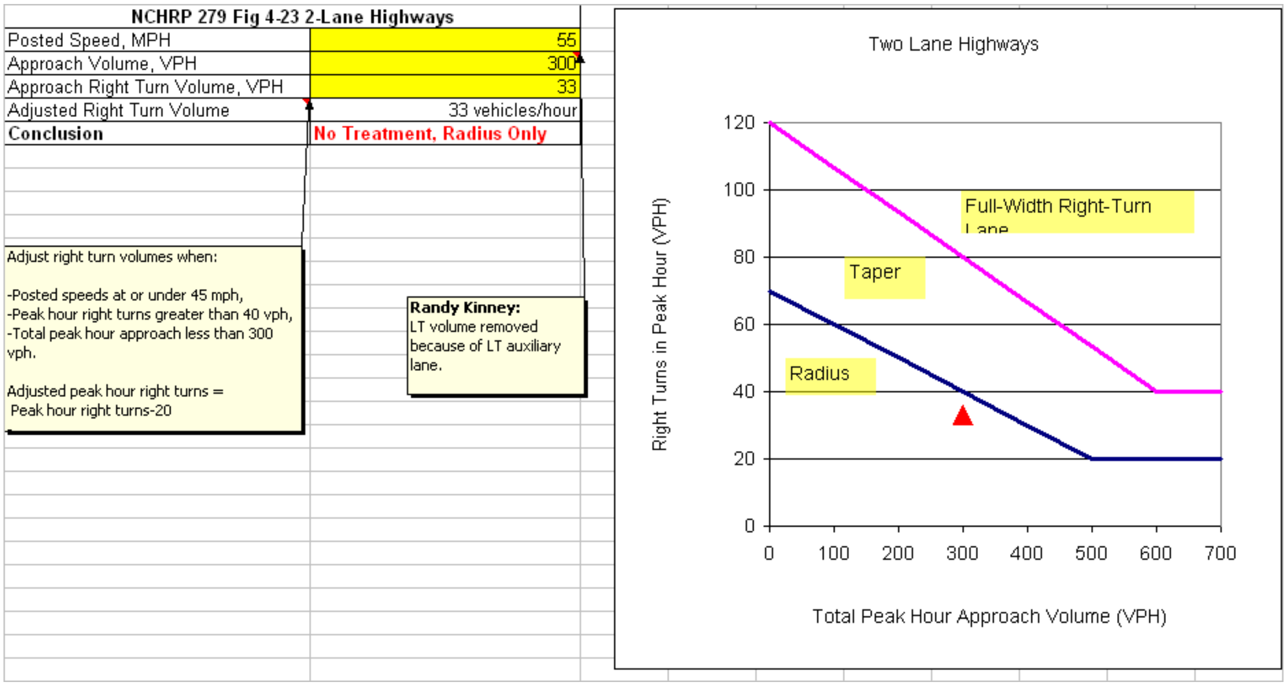


Figure 24- Right Turn Lane Analysis, Southbound Richardson Highway at Valdez Airport Road, Design Year (2020)

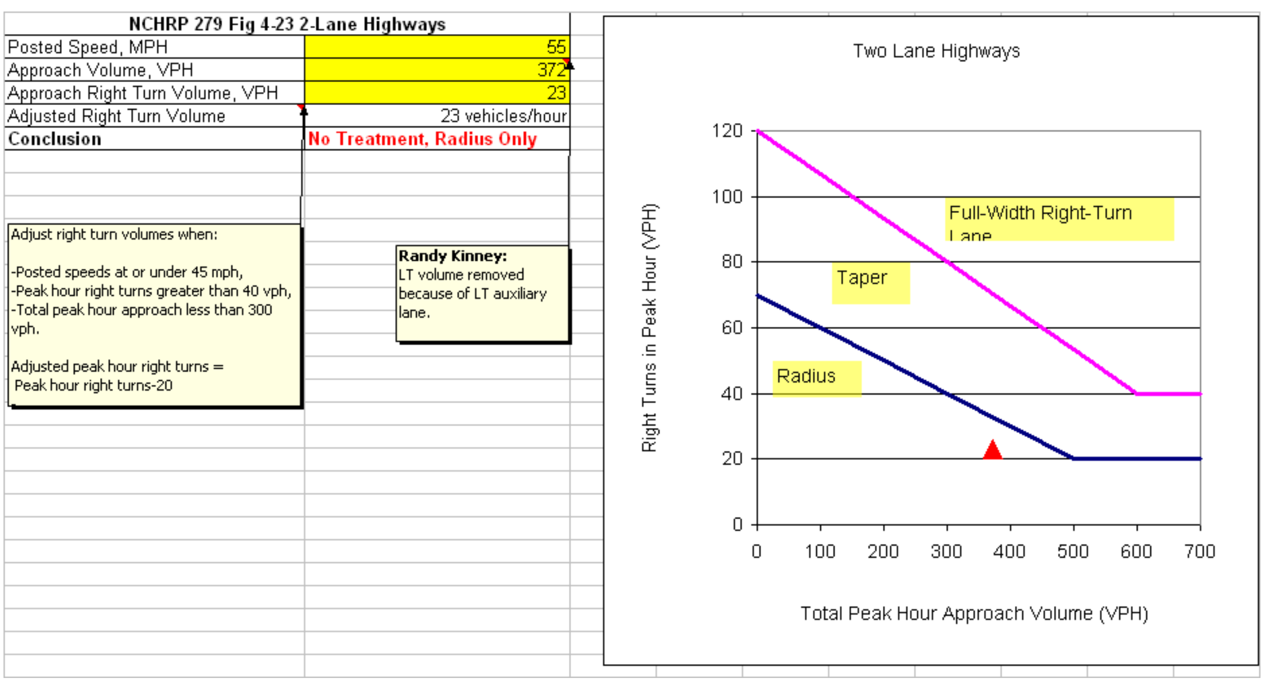


Figure 25- Right Turn Lane Analysis, Northbound Richardson Highway at Valdez Airport Road, Design Year (2020)

NCHRP 279 Fig 4-23 2-Lane Highways	
Posted Speed, MPH	55
Approach Volume, VPH	79
Approach Right Turn Volume, VPH	3
Adjusted Right Turn Volume	3 vehicles/hour
Conclusion	**No Treatment, Radius Only**

Adjust right turn volumes when:

-Posted speeds at or under 45 mph,
-Peak hour right turns greater than 40 vph,
-Total peak hour approach less than 300 vph.

Adjusted peak hour right turns = Peak hour right turns-20

Randy Kinney:
LT volume removed because of LT auxiliary lane.

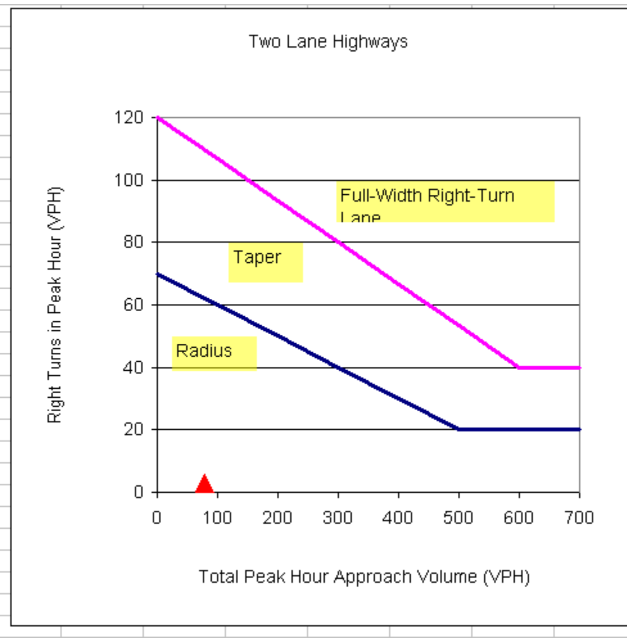


Figure 26- Right Turn Lane Analysis, Southbound Richardson Highway at Sawmill Drive, Design Year (2020)

NCHRP 279 Fig 4-23 2-Lane Highways	
Posted Speed, MPH	55
Approach Volume, VPH	165
Approach Right Turn Volume, VPH	82
Adjusted Right Turn Volume	82 vehicles/hour
Conclusion	**Taper Only**

Adjust right turn volumes when:

-Posted speeds at or under 45 mph,
-Peak hour right turns greater than 40 vph,
-Total peak hour approach less than 300 vph.

Adjusted peak hour right turns = Peak hour right turns-20

Randy Kinney:
LT volume removed because of LT auxiliary lane.

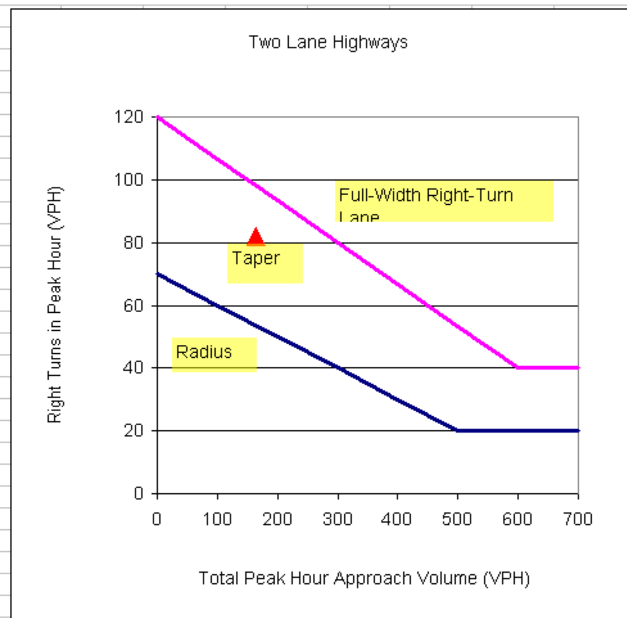


Figure 27- Right Turn Lane Analysis, Northbound Richardson Highway at Sawmill Drive, Design Year (2020)

Turn Lane Geometric Computations (Richardson Highway)

The following figure presents the turn lane geometry template for application of the values shown in the turn lane geometry computation tables.

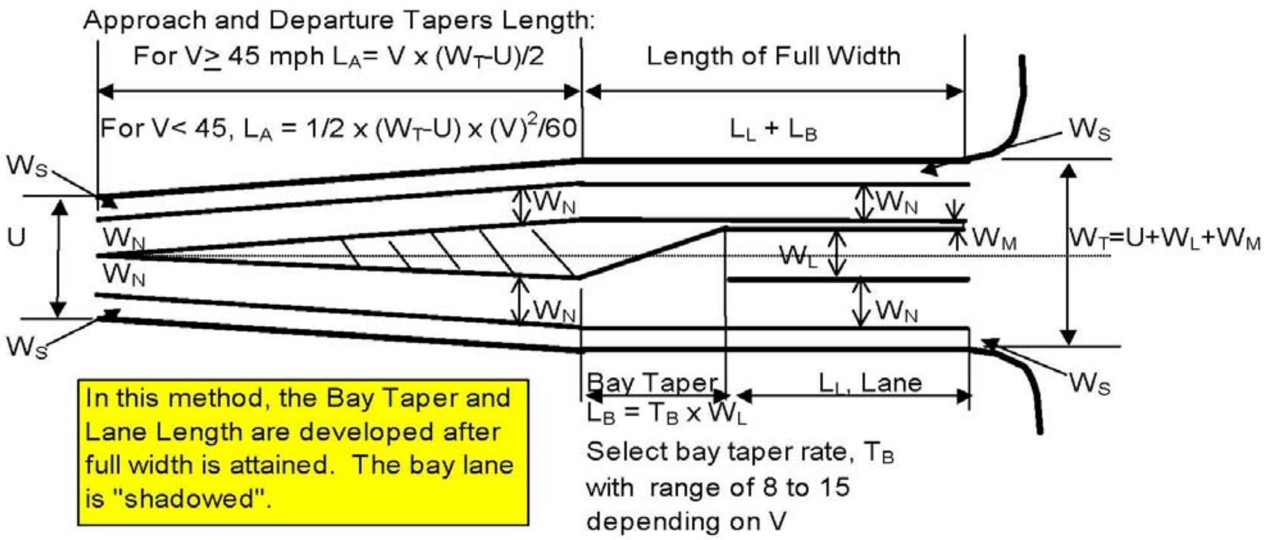


Figure 28- Turn Lane Geometry Template

Input

U=2W_S+2W_N	40.00 feet	
W_M	4.00 feet	
W_L	12.00 feet	
W_S	8.00 feet	
V (posted)	55 mph	
L_L	375.00 feet	See Lane Comps

Output

W_T	56.00 feet
T_B	15.0 : 1
L_A	440.00 feet
T_A	55.0 : 1
Y	5.43 feet
X	298.57 feet
L_B	141.43 feet
L_A + L_L	815.00

.

Table 22- Turn Lane Geometry Calculation, 60 MPH (Minimum Lane)

APPENDIX B- CRASH EVALUATION METHODOLOGY

The accident evaluation methodology uses elements from the *Highway Safety Improvement Program Handbook* by ADOT&PF, and NCHRP Report 162 from Transportation Research Board, *Methods for Evaluating Highway Safety Improvements* by John C. Laughland, *et al.,* National Research Council, Washington, D.C. 1975.

Intersection accident rates are calculated with the following formula:

Equation A1. $$R = \frac{1{,}000{,}000 \times A}{365 \times N \times V}$$

The variables in this equation are:

R= Accident rate for the intersection expressed as accidents per million entering vehicles (MEV),
A= Frequency of accidents in the study period,
N= Number of years of data,
V= Traffic volumes entering the intersection daily, usually ½ of the sum of the Average Annual Daily Traffic (AADT) volumes on the intersection's legs for two way approaches, or the sum of entering AADT volumes on one-way approaches.

Segment rates are defined as:

Equation A2. $$R = \frac{1{,}000{,}000 \times A}{365 \times N \times ADT \times L}$$

R= Accident rate for the intersection expressed as accidents per million vehicle miles (MVM),
A= Frequency of accidents in the study period,
N= Number of years of data,
ADT= Segment Average Annual Daily Traffic (AADT) volumes, both directions.
L= Segment length, miles

Rate analysis is especially useful when there is a population of facilities to which we can compare the study area. ADOT&PF has developed statewide populations for segments and intersections, and provides this data in the HSIPHB and supplements and the annual *Traffic Accident Report*.

We can calculate accident rates using Equation A1 or A2 to compare the facility to the corresponding like State of Alaska accident populations. However, by only comparing the rate of the facility under analysis to an average, we may erroneously infer that those facilities with higher than average rates are problem areas.

Instead, we would like to establish an upper limit for the rate that is our threshold of concern. The Rate Quality Control Method establishes an upper control limit (UCL) to determine if the facility's accident rate, as calculated in Equation 1, is significantly higher than accident rates in facilities with similar characteristics. The UCL is determined statistically as a function of the statewide average accident rate for the facility category (i.e., highway or intersection) and the vehicle exposure at the location being considered. UCL is calculated with the following equation:

Equation A3. $$UCL = Ra + Z \times \sqrt{\frac{Ra}{M}} + \frac{1}{2 \times M},$$

The variables in this equation are:

R_a= Average Accident Rate for the population in accidents per MEV (intersections) or accidents per MVM (road segments);
M= Facility Exposure in MEV for the intersections or MVM for roadway section;
Z= Normal Distribution Transformation Variable (1.64 for 95% confidence)

Intersections or segments with rates that exceed the UCL are considered truly to have an accident rate above average.

APPENDIX C- INTERSECTION SIGNAL WARRANTS

The Manual on Uniform Traffic Control Devices (MUTCD) uses warrants to determine if signal may be used in traffic control. Meeting one or more of the warrants doesn't necessarily mandate a signal, especially where other, less restrictive remedies can be used. The warrants include:

- Warrant 1- Eight-Hour Volume
- Warrant 2- Four-Hour Volume
- Warrant 3- Peak Hour Volume
- Warrant 4- Minimum Pedestrian Volumes
- Warrant 5- School Crossings
- Warrant 6- Coordinated Signal System
- Warrant 7- Crash Experience
- Warrant 8- Roadway Network

The MUTCD warrant system described above only evaluates recent or current conditions. Cal-Trans has a methodology for future signal warrants based that is presented in the Institute of Transportation Engineers (ITE) *Manual of Traffic Signal Design*, Second Edition, by James H. Kell and Iris J. Fullerton. The method uses future estimated average daily traffic (in this case AADT from the demand models) as the input variables and estimates whether the intersection with future estimated average daily traffic would meet the Manual of Uniform Traffic Control Devices signal Warrant 1, Condition A- Minimum Vehicular Volume; Condition B- Interruption of Continuous Traffic; and the combination of warrants allowed in MUTCD procedure.

The method uses future estimated average daily traffic as the input variables and includes the sum of both approach volumes, or AADT for the major road; and highest minor approach entering AADT volume. The following figure provides volume thresholds for the Cal-Trans method from *Manual of Traffic Signal Design*

TRAFFIC SIGNAL WARRANTS

(Based on Estimated Average Daily Traffic—See Note 2)

URBAN ______ RURAL ______	Minimum Requirements EADT			
1. Minimum Vehicular Satisfied ______ Not Satisfied ______	**Vehicles per day on major street (total of both approaches)**		**Vehicles per day on higher-volume minor-street approach (one direction only)**	
Number of lanes for moving traffic on each approach				
Major Street / Minor Street	**Urban**	**Rural**	**Urban**	**Rural**
1 / 1	**8,000**	**5,600**	**2,400**	**1,680**
2 or more / 1	**9,600**	**6,720**	**2,400**	**1,680**
2 or more / 2 or more	**9,600**	**6,720**	**3,200**	**2,240**
1 / 2 or more	**8,000**	**5,600**	**3,200**	**2,240**
2. Interruption of Continuous Traffic Satisfied ______ Not Satisfied ______	**Vehicles per day on major street (total of both approaches)**		**Vehicles per day on higher-volume minor-street approach (one direction only)**	
Number of lanes for moving traffic on each approach				
Major Street / Minor Street	**Urban**	**Rural**	**Urban**	**Rural**
1 / 1	**12,000**	**8,400**	**1,200**	**850**
2 or more / 1	**14,400**	**10,080**	**1,200**	**850**
2 or more / 2 or more	**14,400**	**10,080**	**1,600**	**1,120**
1 / 2 or more	**12,000**	**8,400**	**1,600**	**1,120**
3. Combination Satisfied ______ Not Satisfied ______ No one warrant satisfied but following warrants fulfilled 80% or more ___ 1 ___ 2	**2 Warrants**		**2 Warrants**	

NOTE:
1. Left turn movements from the major street may be included with minor street volumes if a separate signal phase is to be provided for the left-turn movement.
2. To be used only for NEW INTERSECTIONS or other locations where actual traffic volumes cannot be counted.

Figure 29- Appendix E: CALTRANS Future EADT Signal Warrant Method

Appendix D- Capacity Analysis Reports

3: Valdez Airport Road & Salcha Way	Direction, Lane #		EB 1 -T&R	WB 1 -L&T	NB 1 -L&R	
	Volume Total	flow rate, (Vol/PHF)	85	68	50	
	Volume Left	flow rate, (Vol/PHF)	0	6	44	
	Volume Right	flow rate, (Vol/PHF)	33	0	6	
	cSH		1700	1511	862	
	Volume to Capacity		0.05	0	0.06	
	Queue Length 95th (ft)		0	0	5	
	Control Delay (s)		0	0.7	9.4	
	Lane LOS			A	A	
	Approach Delay (s)		0	0.7	9.4	
	Approach LOS				A	
4: Atigun Drive & Salcha & Sawmill	Direction, Lane #		EB 1 -L&T	WB 1 -T&R	SB 1 -L&R	
	Volume Total	flow rate, (Vol/PHF)	59	73	28	
	Volume Left	flow rate, (Vol/PHF)	11	0	13	
	Volume Right	flow rate, (Vol/PHF)	0	7	15	
	cSH		1527	1700	918	
	Volume to Capacity		0.01	0.04	0.03	
	Queue Length 95th (ft)		1	0	2	
	Control Delay (s)		1.4	0	9	
	Lane LOS		A		A	
	Approach Delay (s)		1.4	0	9	
	Approach LOS				A	
5: Valdez Airport Road & RichardsonHigh	Direction, Lane #		EB 1 -L&T&R	WB 1 -L&T&R	NB 1 -L&T&F	SB 1 -L&T&
	Volume Total	flow rate, (Vol/PHF)	143	125	292	315
	Volume Left	flow rate, (Vol/PHF)	62	11	42	97
	Volume Right	flow rate, (Vol/PHF)	53	73	31	44
	cSH		338	473	1352	1315
	Volume to Capacity		0.42	0.26	0.03	0.07
	Queue Length 95th (ft)		51	26	2	6
	Control Delay (s)		23.2	15.3	1.3	2.9
	Lane LOS		C	C	A	A
	Approach Delay (s)		23.2	15.3	1.3	2.9
	Approach LOS		C	C		
10: Sawmill Drive & RichardsonHighway	Direction, Lane #		EB 1 -L&T&R	WB 1 -L&T&R	NB 1 -L&T&F	SB 1 -L&T&
	Volume Total	flow rate, (Vol/PHF)	20	112	93	173
	Volume Left	flow rate, (Vol/PHF)	16	2	1	85
	Volume Right	flow rate, (Vol/PHF)	1	109	1	4
	cSH		461	945	1508	1503
	Volume to Capacity		0.04	0.12	0	0.06
	Queue Length 95th (ft)		3	10	0	5
	Control Delay (s)		13.1	9.3	0.1	3.9
	Lane LOS		B	A	A	A
	Approach Delay (s)		13.1	9.3	0.1	3.9
	Approach LOS		B	A		

Figure 30- HCM Unsignalized Analysis, Development (2010)

3: Valdez Airport Road & Salcha Way	Direction, Lane #		EB 1 -T&R	WB 1 -L&T	NB 1-R						
	Volume Total	flow rate, (Vol/PHF)	177	89	168						
	Volume Left	flow rate, (Vol/PHF)	0	21	154						
	Volume Right	flow rate, (Vol/PHF)	112	0	15						
	cSH		1700	1399	760						
	Volume to Capacity		0.1	0.01	0.22						
	Queue Length 95th (ft)		0	1	21						
	Control Delay (s)		0	1.9	11.1						
	Lane LOS			A	B						
	Approach Delay (s)		0	1.9	11.1						
	Approach LOS				B						
4: Atigun Drive & Salcha & Sawmill	Direction, Lane #		EB 1 -L&T	WB 1 -T&R	SB 1 -L&R						
	Volume Total	flow rate, (Vol/PHF)	317	484	182						
	Volume Left	flow rate, (Vol/PHF)	32	0	137						
	Volume Right	flow rate, (Vol/PHF)	0	73	45						
	cSH		1079	1700	387						
	Volume to Capacity		0.03	0.28	0.47						
	Queue Length 95th (ft)		2	0	61						
	Control Delay (s)		1.1	0	22.3						
	Lane LOS		A		C						
	Approach Delay (s)		1.1	0	22.3						
	Approach LOS				C						
5: Valdez Airport Road & RichardsonHigh	Direction, Lane #		EB 1-L	EB 2 -T&R	WB 1-L	WB 2 -T	WB 3-R	NB 1-L	NB 2 -T&R	SB 1-L	SB 2 -T&R
	Volume Total	flow rate, (Vol/PHF)	66	72	18	36	151	38	511	188	426
	Volume Left	flow rate, (Vol/PHF)	66	0	18	0	0	38	0	188	0
	Volume Right	flow rate, (Vol/PHF)	0	49	0	0	151	0	49	0	52
	cSH		49	267	82	117	581	1134	1700	1054	1700
	Volume to Capacity		1.34	0.27	0.22	0.31	0.26	0.03	0.3	0.18	0.25
	Queue Length 95th (ft)		153	26	20	30	26	3	0	16	0
	Control Delay (s)		375.9	23.4	61.3	48.9	13.4	8.3	0	9.2	0
	Lane LOS		F	C	F	E	B	A		A	
	Approach Delay (s)		192.2		23.9			0.6		2.8	
	Approach LOS		F		C						
10: Sawmill Drive & RichardsonHighway	Direction, Lane #		EB 1-L	EB 2 -T&R	WB 1-L	WB 2 -T&R	NB 1-L	NB 2 -T&R	SB 1-L	SB 2 -T&R	
	Volume Total	flow rate, (Vol/PHF)	15	2	101	328	1	201	249	96	
	Volume Left	flow rate, (Vol/PHF)	15	0	101	0	1	0	249	0	
	Volume Right	flow rate, (Vol/PHF)	0	1	0	327	0	100	0	4	
	cSH		116	411	282	888	1497	1700	1371	1700	
	Volume to Capacity		0.13	0.01	0.36	0.37	0	0.12	0.18	0.06	
	Queue Length 95th (ft)		10	0	39	43	0	0	17	0	
	Control Delay (s)		40.3	13.8	24.7	11.4	7.4	0	8.2	0	
	Lane LOS		E	B	C	B	A		A		
	Approach Delay (s)		36.5		14.5		0		5.9		
	Approach LOS		E		B						

Figure 31- HCM Unsignalized Analysis, Development (2020)

3: Valdez Airport Road & Salcha Way	Direction, Lane #		EB 1 -T&R	WB 1 -L&T	NB 1 -L&R					
	Volume Total	flow rate, (Vol/PHF)	107	70	77					
	Volume Left	flow rate, (Vol/PHF)	0	6	71					
	Volume Right	flow rate, (Vol/PHF)	54	0	6					
	cSH		1700	1483	842					
	Volume to Capacity		0.06	0	0.09					
	Queue Length 95th (ft)		0	0	8					
	Control Delay (s)		0	0.7	9.7					
	Lane LOS			A	A					
	Approach Delay (s)		0	0.7	9.7					
	Approach LOS				A					
4: Atigun Drive &	Direction, Lane #		EB 1 -L&T	WB 1 -T&R	SB 1-R					
	Volume Total	flow rate, (Vol/PHF)	148	165	68					
	Volume Left	flow rate, (Vol/PHF)	29	0	21					
	Volume Right	flow rate, (Vol/PHF)	0	9	48					
	cSH		1414	1700	795					
	Volume to Capacity		0.02	0.1	0.09					
	Queue Length 95th (ft)		2	0	7					
	Control Delay (s)		1.6	0	10					
	Lane LOS		A		A					
	Approach Delay (s)		1.6	0	10					
	Approach LOS				A					
5: Valdez Airport Road & RichardsonHigh	Direction, Lane #		EB 1-L	EB 2 -T&R	WB 1-L	WB 2 -T&R	NB 1-L	NB 2 -T&R	SB 1-L	SB 2 -T&R
	Volume Total	flow rate, (Vol/PHF)	66	77	14	127	43	357	119	304
	Volume Left	flow rate, (Vol/PHF)	66	0	14	0	43	0	119	0
	Volume Right	flow rate, (Vol/PHF)	0	54	0	92	0	38	0	48
	cSH		145	444	175	436	1257	1700	1202	1700
	Volume to Capacity		0.46	0.17	0.08	0.29	0.03	0.21	0.1	0.18
	Queue Length 95th (ft)		52	16	6	30	3	0	8	0
	Control Delay (s)		49.1	14.8	27.3	16.6	8	0	8.3	0
	Lane LOS		E	B	D	C	A		A	
	Approach Delay (s)		30.6		17.7		0.9		2.3	
	Approach LOS		D		C					
10: Sawmill Drive & RichardsonHighway	Direction, Lane #		EB 1-L	EB 2 -T&R	WB 1-L	WB 2 -T&R	NB 1-L	NB 2 -T&R	SB 1-L	SB 2 -T&R
	Volume Total	flow rate, (Vol/PHF)	15	2	5	202	1	96	160	90
	Volume Left	flow rate, (Vol/PHF)	15	0	5	0	1	0	160	0
	Volume Right	flow rate, (Vol/PHF)	0	1	0	201	0	4	0	4
	cSH		254	584	437	955	1505	1700	1497	1700
	Volume to Capacity		0.06	0	0.01	0.21	0	0.06	0.11	0.05
	Queue Length 95th (ft)		5	0	1	20	0	0	9	0
	Control Delay (s)		20	11.2	13.3	9.8	7.4	0	7.7	0
	Lane LOS		C	B	B	A	A		A	
	Approach Delay (s)		18.8		9.9		0.1		4.9	
	Approach LOS		C		A					

Figure 32- HCM Unsignalized Analysis, Development without Industrial Park (2020)

3: Valdez Airport Road & Salcha Way	Direction, Lane #		EB 1 -T&R	WB 1 -L&T	NB 1 -L&R	
	Volume Total	flow rate, (Vol/PHF)	107	70	77	
	Volume Left	flow rate, (Vol/PHF)	0	6	71	
	Volume Right	flow rate, (Vol/PHF)	54	0	6	
	cSH		1700	1483	842	
	Volume to Capacity		0.06	0	0.09	
	Queue Length 95th (ft)		0	0	8	
	Control Delay (s)		0	0.7	9.7	
	Lane LOS			A	A	
	Approach Delay (s)		0	0.7	9.7	
	Approach LOS				A	
4: Atigun Drive &	Direction, Lane #		EB 1 -L&T	WB 1 -T&R	SB 1-R	
	Volume Total	flow rate, (Vol/PHF)	148	165	68	
	Volume Left	flow rate, (Vol/PHF)	29	0	21	
	Volume Right	flow rate, (Vol/PHF)	0	9	48	
	cSH		1414	1700	795	
	Volume to Capacity		0.02	0.1	0.09	
	Queue Length 95th (ft)		2	0	7	
	Control Delay (s)		1.6	0	10	
	Lane LOS		A		A	
	Approach Delay (s)		1.6	0	10	
	Approach LOS				A	
5: Valdez Airport Road & RichardsonHigh	Direction, Lane #		EB 1 -L&T&R	WB 1 -L&T&R	NB 1 -L&T&F	SB 1 -L&T8
	Volume Total	flow rate, (Vol/PHF)	143	141	400	423
	Volume Left	flow rate, (Vol/PHF)	66	14	43	119
	Volume Right	flow rate, (Vol/PHF)	54	92	38	48
	cSH		223	377	1257	1202
	Volume to Capacity		0.64	0.38	0.03	0.1
	Queue Length 95th (ft)		97	43	3	8
	Control Delay (s)		46.3	20.2	1.2	3.1
	Lane LOS		E	C	A	A
	Approach Delay (s)		46.3	20.2	1.2	3.1
	Approach LOS		E	C		
10: Sawmill Drive & RichardsonHighway	Direction, Lane #		EB 1 -L&T&R	WB 1 -L&T&R	NB 1 -L&T&F	SB 1 -L&T8
	Volume Total	flow rate, (Vol/PHF)	17	207	98	250
	Volume Left	flow rate, (Vol/PHF)	15	5	1	160
	Volume Right	flow rate, (Vol/PHF)	1	201	4	4
	cSH		276	929	1505	1497
	Volume to Capacity		0.06	0.22	0	0.11
	Queue Length 95th (ft)		5	21	0	9
	Control Delay (s)		18.9	10	0.1	5.2
	Lane LOS		C	A	A	A
	Approach Delay (s)		18.9	10	0.1	5.2
	Approach LOS		C	A		

Figure 33- HCM Unsignalized Analysis, Development without Industrial Park, Unimproved Existing Intersections (2020)

APPENDIX E- PTR DATA, RICHARDSON HIGHWAY

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SITE 11100071 ROUTE 190000 MP 0.543

*** VALDEZ *** RICHARDSON HIGHWAY AT VALDEZ (NB)

MONTH	MADT	%	6-10	(10-6)	MON	TUE	WED	THU	FRI	WKDY	SAT	SUN
JAN	1746	81	90	10	106.7	100.8	107.6	111.9	113.5	108.1	86.8	72.5
FEB	1806	83.8	90.6	9.4	102.9	111.4	108.4	112.1	110.1	109	84.1	71.2
MAR	1900	88.2	90.3	9.7	101.8	105.8	107.3	107.2	111.3	106.7	91.7	75.1
APR	2013	93.4	90.6	9.4	107.7	107.5	106.6	107.4	109.7	107.8	86.4	74.5
MAY	2339	108.5	90.5	9.5	98.8	104	104.6	102.2	109.6	103.8	97.9	82.7
JUN	2554	118.5	90.2	9.8	103.4	102.4	104.2	106.1	108.9	105	93.4	81.6
JUL	2849	132.2	89.7	10.3	100.1	102.6	101.7	104.6	104.9	102.8	97.3	88.9
AUG	2850	132.3	90	10	103.8	99	101.6	100.9	107.3	102.5	96.7	90.6
SEP	2284	106	90.6	9.4	108.6	104.7	104.9	104.5	109.2	106.4	91.3	76.7
OCT	2024	93.9	90.9	9.1	105	107.7	110.3	106.5	108.7	107.6	89.1	72.8
NOV	1813	84.1	90.1	9.9	111.1	110.8	111.5	99.7	105.2	107.7	89.2	72.3
DEC	1680	78	89.6	10.4	111.2	110	109.9	96.8	109.1	107.4	88.3	74.5
ANN	2155		90.3	9.7	105.1	105.6	106.6	105	109	106.2	91	77.8

HISTORICAL DATA

2008	2155
2007	2334
2006	2539
2005	2395
2004	2491
2003	2572
2002	2657
2001	2713
2000	2724
1999	2643
1998	2745
1997	2808
1996	2747
1995	2858
1994	2998
1993	2701
1992	2581
1991	2515
1990	2558

HIGHEST DAYS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	AVG
3192	3092	3092	3084	3079	3041	3038	3038	3032	3023	3071
29-Aug	3-Jul	29-Jul	11-Aug	2-Jul	18-Aug	8-Aug	15-Jul	15-Aug	1-Aug	
148.1	143.5	143.5	143.1	142.9	141.1	141	141	140.7	140.3	142.5

HIGHEST HOURS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	20TH	30TH	40TH	50TH	AVG
284	279	276	275	274	273	271	270	269	268	258	250	244	242	274
11	17	18	18	18	18	18	18	18	18	18	19	18	18	
1-Sep	3-Aug	21-May	27-May	17-Jun	23-Jun	10-Jul	24-Jun	22-May	11-Jul	28-Jul	29-Aug	3-Jun	4-Aug	
13.2	12.9	12.8	12.8	12.7	12.7	12.6	12.5	12.5	12.4	12	11.6	11.3	11.2	12.7

PERCENT BY HOUR

100	200	300	400	500	600	700	800	900	1000	1100	1200
0.9	0.6	0.4	0.4	1	3	3.1	4.2	4.5	4.5	5.3	5.9
1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400
7.2	7.1	6.5	7.2	7.9	8.4	6.3	4.8	4.1	3.3	2.1	1.4

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***** VALDEZ *** RICHARDSON HIGHWAY AT VALDEZ (COMBINED)**

MONTH	MADT	%	6-10	(10-6)	MON	TUE	WED	THU	FRI	WKDY	SAT	SUN
JAN	3494	81.2	91.1	8.9	107.3	100.5	107.7	111.3	113.9	108.1	87	72.2
FEB	3611	83.9	91.7	8.3	103.7	111.3	108.5	112.2	109.4	109	84.1	70.8
MAR	3806	88.4	91.3	8.7	102.5	105.6	107.4	106.6	111.6	106.7	92.1	74.2
APR	4024	93.5	91.7	8.3	108.2	107.4	106.7	107.1	109.4	107.8	86.8	74.3
MAY	4628	107.5	91.2	8.8	99.6	104.8	105.8	104	112.2	105.3	91.3	82.3
JUN	5123	119	91	9	103.6	102.5	104.2	105.7	108.9	105	94.1	81.1
JUL	5706	132.5	90.4	9.6	100.1	102.4	101.1	105.6	106.4	103.1	98.3	86.2
AUG	5704	132.5	91	9	102.6	98.8	101.4	102.1	109.3	102.8	98	87.8
SEP	4541	105.5	92.1	7.9	106.9	104.7	105.4	105.1	109.5	106.3	92.2	76.2
OCT	4048	94	92.3	7.7	105.7	107.5	110	106.6	108.3	107.6	89.1	72.8
NOV	3620	84.1	91.4	8.6	112.1	110.6	111.2	99.1	104.9	107.6	89.3	72.9
DEC	3358	78	90.9	9.1	112	110.2	109.8	96.5	108.8	107.5	88.7	74
ANN	4305		91.3	8.7	105.4	105.5	106.6	105.2	109.4	106.4	90.9	77.1

HISTORICAL DATA	
2008	4305
2007	4694
2006	5219
2005	4790
2004	4971
2003	5140
2002	5307
2001	5420
2000	5443
1999	5281
1998	5470
1997	5599
1996	5475
1995	5693
1994	5986
1993	5395
1992	5146
1991	5054
1990	5053
1989	7241
1988	3454
1987	3376
1986	3447
1985	3270
1984	3531

HIGHEST DAYS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	AVG
6533	6387	6263	6224	6186	6148	6127	6119	6102	6090	6218
29-Aug	3-Jul	15-Aug	8-Aug	2-Jul	30-Aug	1-Aug	11-Jul	29-Jul	15-Jul	
151.8	148.4	145.5	144.6	143.7	142.8	142.3	142.1	141.7	141.5	144.4

HIGHEST HOURS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	20TH	30TH	40TH	50TH	AVG
558	544	539	530	528	528	526	525	521	520	513	507	501	497	532
18	18	18	18	18	18	18	19	18	17	18	18	18	18	
10-Jul	28-Aug	18-Jul	25-Jun	11-Jul	9-Jul	8-Jul	1-Aug	19-Aug	29-Aug	17-Jun	8-Aug	25-Jul	2-Jul	
13	12.6	12.5	12.3	12.3	12.3	12.2	12.2	12.1	12.1	11.9	11.8	11.6	11.5	12.4

PERCENT BY HOUR

100	200	300	400	500	600	700	800	900	1000	1100	1200
0.8	0.5	0.4	0.4	0.8	2.4	2.8	4.9	4.8	4.8	5.4	6.2
1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400
6.9	6.8	6.5	7.1	7.7	8.7	7.2	4.8	3.7	3	2	1.3

ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
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***** VALDEZ *** RICHARDSON HIGHWAY AT VALDEZ (SB)**

MONTH	MADT	%	6-10	(10-6)	MON	TUE	WED	THU	FRI	WKDY	SAT	SUN
JAN	1749	81.3	92.2	7.8	107.8	100.2	107.8	110.7	114.3	108.2	87.2	71.8
FEB	1804	83.9	92.8	7.2	104.5	111.2	108.6	112.3	108.9	109.1	84.2	70.5
MAR	1905	88.6	92.2	7.8	103.3	105.5	107.5	106	111.9	106.8	92.7	73.4
APR	2011	93.5	92.8	7.2	108.7	107.3	106.7	106.8	109	107.7	87.2	74.2
MAY	2289	106.5	92	8	100.4	105.6	107	105.8	114.9	106.7	84.6	81.8
JUN	2569	119.5	91.7	8.3	103.8	102.6	104.1	105.2	108.8	104.9	94.8	80.7
JUL	2857	132.9	91.2	8.8	100	102.2	100.4	106.5	107.9	103.4	99.3	83.6
AUG	2854	132.7	92	8	101.3	98.7	101.3	103.2	111.3	103.2	99.3	84.9
SEP	2257	105	93.5	6.5	105.2	104.8	105.8	105.6	109.8	106.2	93.1	75.7
OCT	2023	94.1	93.7	6.3	106.3	107.4	109.7	106.6	108	107.6	89.2	72.9
NOV	1808	84.1	92.8	7.2	113	110.4	110.9	98.5	104.6	107.5	89.4	73.4
DEC	1679	78.1	92.3	7.7	112.8	110.2	109.7	96.1	108.5	107.5	89	73.4
ANN	2150		92.4	7.6	105.6	105.5	106.6	105.3	109.8	106.6	90.8	76.4

HISTORICAL DATA	
2008	2150
2007	2360
2006	2522
2005	2395
2004	2488
2003	2568
2002	2650
2001	2707
2000	2719
1999	2638
1998	2725
1997	2791
1996	2728
1995	2835
1994	2989
1993	2694
1992	2565
1991	2539
1990	2571

HIGHEST DAYS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	AVG
3341	3295	3231	3186	3152	3113	3110	3107	3104	3061	3170
29-Aug	3-Jul	15-Aug	8-Aug	30-Aug	4-Jul	11-Jul	2-Jul	1-Aug	18-Jul	
155.4	153.3	150.3	148.2	146.6	144.8	144.7	144.5	144.4	142.4	147.4

HIGHEST HOURS

1ST	2ND	3RD	4TH	5TH	6TH	7TH	8TH	9TH	10TH	20TH	30TH	40TH	50TH	AVG
303	300	294	294	291	289	288	287	287	284	279	272	268	260	292
18	19	19	19	18	18	19	19	18	18	17	18	19	19	
28-Aug	11-Aug	15-Aug	7-Aug	12-Jul	18-Jul	23-Aug	18-Aug	10-Jul	4-Sep	3-Jul	15-Aug	22-Jul	25-Jul	
14.1	14	13.7	13.7	13.5	13.4	13.4	13.3	13.3	13.2	13	12.7	12.5	12.1	13.6

PERCENT BY HOUR

100	200	300	400	500	600	700	800	900	1000	1100	1200
0.7	0.5	0.4	0.4	0.5	1.8	2.5	5.6	5.2	5	5.4	6.4
1300	1400	1500	1600	1700	1800	1900	2000	2100	2200	2300	2400
6.7	6.5	6.5	7.1	7.6	9	8.2	4.7	3.3	2.8	2	1.2

APPENDIX F- REFERENCES

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