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Road Diet Feasibility Analysis for Nebraska

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ROAD DIET FEASIBILITY ANALYSIS FOR NEBRASKA

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Many rural towns and cities throughout Nebraska have experienced consistent population decline over the past 50 years, and the highway system once built to accommodate hoped-for growth is not serving the population as well as it could. These towns and cities would benefit from implementing a road diet conversion on their main highways. Within rural communities, road diets are an increasingly popular method of improving safety along major arterials through the reduction of excess capacity in favor of increasing refuge areas for turning vehicles. A typical application might be the restriping of a four-lane undivided highway into a three-lane highway comprised of two lanes of through movement and a two-way left-turn lane. Deciding when or if to implement a road diet conversion involves the consideration of many factors. The consideration of numerous factors can often lead to explanations on the feasibility of road improvement projects saturated with technical language. Since support for road improvement projects such as road diets lies in the public sector, the decision making process needs to be made easy to understand.

Public edification of the decision making process involves streamlining the process as well as reducing criteria which are technically sound yet abstract to the public. To streamline the decision making process, a case study and sensitivity analysis is conducted to determine best practices, evaluation methodology, and decision making processes. A before and after simulation analysis is performed using VISSIM, examining delay. Existing literature on road diets is used to establish broad guidelines and determine long-term effects, such as changes in crash rates. Existing literature is also used to help measure the effects of road diet improvements on roadway performance as economic benefits and costs, metrics which are more easily understood by the public. Existing volume conditions at the case study locations are found to be well-below capacity, and the facility performs equally well when modeling with and without the road diet improvements. Subsequently, sensitivity analysis is conducted to determine the impact of volume demands on the bottom line costs and benefits of a road diet conversion. This information is then used to create guidelines, with easily understood criteria, for making decisions on whether or not a road diet improvement should be implemented.

TABLE OF CONTENTS

CHAPTER 1 INTRODUCTION 9
1.1 Background 9
1.2 Research Questions 10
1.3 Methodology 11
1.4 Document Layout 12
CHAPTER 2 LITERATURE REVIEW 14
2.1 Traffic Volume Impacts on Road Diet Feasibility 14
2.2 Safety Impacts of Road Diets 18
2.3 Operational Impacts of Road Diets – Speed, Travel Time, and Average Delay 23
2.4 Additional Feasibility Factors 30
CHAPTER 3 ANALYSIS METHODOLOGY 31
3.1 Data Collection and Reduction 31
3.2 Operational Analysis of Roadway Conditions 32
3.2.1 Synchro (HCM) Analysis of Existing and Road Diet Conditions 33
3.2.2 VISSIM Analysis of Existing and Road Diet Conditions 35
3.3 Economic Analysis of Road Diet Impacts 36
3.3.1 Economic Analysis Procedure 36
3.3.2 Economic Analysis Methodology 37
CHAPTER 4 CASE STUDY SELECTION 42
4.1 Case Study Site Location – Alliance, Nebraska 42
4.2 Case Study Site Location – Aurora, Nebraska 44

4.3 Case Study Site Location – David City, Nebraska ... 46

4.4 Case Study Site Location – Ord, Nebraska ... 48

CHAPTER 5 DATA COLLECTION AND REDUCTION ... 51

5.1 Alliance Data Collection ... 51

5.2 Aurora Data Collection ... 54

5.3 David City Data Collection ... 58

5.4 Ord Data Collection ... 63

5.5 Data Reduction ... 65

CHAPTER 6 RESULTS ... 67

6.1 Operational Impacts of Road Diet Implementations ... 67

6.1.1 Results of Existing Conditions Analysis ... 67

6.1.2 Results of Road Diet Analysis ... 71

6.1.3 Results of Road Diet Impact Analysis ... 74

6.1.4 Impact of Percentage of Turning Vehicles on Road Diet Operations ... 78

6.2 Assessment of User Cost Impacts from Road Diets ... 80

CHAPTER 7 ROAD DIET DECISION MATRIX ... 86

7.1 Analysis of Road Diet Limitations ... 86

7.2 Decision Matrix for Consideration of Road Diet Implementation ... 89

CHAPTER 8 LIMITATIONS ... 92

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS ... 93

REFERENCES 95

LIST OF MULTIMEDIA OBJECTS

Table 2.1 Results of road diet conversion in Seattle, Washington [4] ... 15
Table 2.2 Results of 2001 road diet conversion study by Knapp and Giese [5] ... 16
Table 2.3 Results of 2010 road diet study by HSIS [6] ... 17
Figure 2.1 Results of 2012 road diet study by Stamatiadis and Kirk [8] ... 18
Table 2.4 Collision results for 1999 study by Welch [4] ... 20
Table 2.5 Results of 2003 road diet study by Huang, Stewart, Zegeer, and Esse [12] ... 21
Table 2.6 Results of 2003 road diet study by Huang, Stewart, Zegeer, and Esse [12] ... 22
Table 2.7 Results of 2014 Knapp et al. study in FHWA Road Diet Guidebook [13] ... 23
Table 2.8 Speed reduction results from 1999 study on road diets by Welch [4] ... 24
Table 2.9 Speed impact results from 2001 road diet study by Knapp and Giese [5] ... 24
Table 2.10 Speed impact results from 2001 road diet study by Knapp and Giese [5] ... 25
Table 2.11 Speed impact results from 2001 road diet study by Knapp and Giese [5] ... 25
Table 2.12 Motorized vehicle LOS criteria for signalized intersections [2] ... 27
Table 2.13 Results of LOS analysis from Welch's 1999 road diet study [4] ... 28
Table 2.14 Results of road diet LOS analysis from study by Li and Tian [16] ... 29
Table 2.15 Results of road diets LOS analysis from study by Li and Tian [16] ... 29
Figure 3.1 Setup of MioVision Scout cameras ... 32
Figure 3.2 Typical section for existing (top) and road diet conditions (bottom) [19] ... 33
Table 3.1 Unit value of time and occupancy rates for case study locations ... 38
Table 3.2 Fuel consumption (gallons) per minute of delay [3] ... 39
Table 3.3 Crash data reported by Nebraska DOT [22]–[24] ... 40
Figure 4.1 Site location in Alliance, Nebraska [26] ... 43
Figure 4.2 Typical street cross-section – Alliance, Nebraska [26] ... 44
Figure 4.3 Site location in Aurora, Nebraska [26] ... 45
Figure 4.4 Typical street cross-section – Aurora, Nebraska [26] ... 46
Figure 4.5 Site location at David City, Nebraska [26] ... 47
Figure 4.6 Typical street cross-section – David City, Nebraska [26] ... 48
Figure 4.7 Site location at Ord, Nebraska [26] ... 49
Figure 4.8 Typical street cross-section – Ord, Nebraska [26] ... 50

Figure 5.1 Camera view at Flack Avenue and Highway 2 52
Figure 5.2 Camera view at Mississippi Avenue and Highway 2 52
Figure 5.3 Camera view at Niobrara Avenue and Highway 2 53
Figure 5.4 Camera view at Big Horn Avenue and Highway 2 53
Figure 5.5 Camera view at Black Hills Avenue and Highway 2 53
Figure 5.6 Camera view at 1st Street and Highway 34.. 55
Figure 5.7 Camera view at 3rd Street and Highway 34 55
Figure 5.8 Camera view at 5th Street and Highway 34 56
Figure 5.9 Camera view at 7th Street and Highway 34 56
Figure 5.10 Camera view at 9th Street and Highway 34 56
Figure 5.11 Camera view at 10th Street and Highway 34 57
Figure 5.12 Camera view at 12th Street and Highway 34 57
Figure 5.13 Camera view at 13th Street and Highway 34 57
Figure 5.14 Camera view at 14th Street and Highway 34 58
Figure 5.15 Camera view at 16th Street and Highway 34 58
Figure 5.16 Camera view at Lakeside Drive and Highway 15 59
Figure 5.17 Camera view at Iowa Street and Highway 15 60
Figure 5.18 Camera view at A Street and Highway 15 60
Figure 5.19 Camera view at C Street and Highway 15.. 60
Figure 5.20 Camera view at E Street and Highway 15.. 61
Figure 5.21 Camera view at G Street and Highway 15 61
Figure 5.22 Camera view at I Street and Highway 15 61
Figure 5.23 Camera view at K Street and Highway 15 62
Figure 5.24 Camera view at M Street and Highway 15... 62
Figure 5.25 Camera view at O Street and Highway 15 62
Figure 5.26 Camera view at 17th Street and Highway 11/70 63
Figure 5.27 Camera view at 20th Street and Highway 11/70 64
Figure 5.28 Camera view at 22nd Street and Highway 11/70... 64
Figure 5.29 Camera view at 24th Street and Highway 11/70 64
Figure 5.30 Camera view at 24th Street and Highway 11/70 65
Table 6.1 Results of existing condition analysis for all case study locations 68

Figure 6.1 Results of existing condition analysis from Synchro 69
Figure 6.2 Results of existing condition analysis from VISSIM 69
Table 6.2 Results of road diet condition analysis for all case study locations................. 71
Figure 6.3 Results of road diet analysis from Synchro 72
Figure 6.4 Results of road diet analysis from VISSIM... 73
Table 6.3 Impacts of road diet improvement on delay (Synchro results) 75
Figure 6.5 Increase in average delay at varying levels of AADT................................... 76
Figure 6.6 Road diet impact on existing average delay 77
Table 6.4 Right turn percentages corresponding to delay and AADT (Synchro)............ 79
Table 6.5 Breakdown of user costs for road diets at case study locations 81
Table 6.6 Expansion of total user costs from peak-hour to yearly.................................. 82
Figure 6.7 Total annual net user cost of road diets across levels of AADT 83
Figure 6.8 Effect of average delay on annual net user costs for road diets 84
Figure 6.9 Effect of change in average delay on annual net user costs for road diets..... 84
Figure 7.1 Road diet conversion decision tree 89

CHAPTER 1 INTRODUCTION

This chapter provides an introduction to the research presented as part of this study. The background section introduces the general concepts of road diet implementations, and the factors that impact whether they should be pursued. The research questions and analysis methodology are presented in the second and third sections of this chapter. Finally, a description of the document layout is provided.

1.1 Background

In many rural towns, a two-lane undivided highway expands to a four-lane undivided highway as it arrives at the edge of town, and narrows back to two lanes as it exits the other side. However, these rural towns rarely have the traffic volumes to warrant four lanes. Transportation engineers have been looking for different methods of improved sustainability for rural roadways. In recent years, there has been an increase in the implementation of road diets. A road diet is the reduction in the number of lanes of a roadway. Typically for rural applications, this would be a four lane undivided highway that is restriped to three lanes, one being a two-way left-turn lane (TWLTL). The remaining lane space can then be used for a variety of possibilities; most frequently it is left as additional shoulder space, but in some cases it may alternatively be used as on-street parking or landscaping. Some of the major benefits of a road diet include better mobility and access, and an enhancement to the livability and walkability of the roadway, and some improvement to safety where conflicts between turning and through movements are reduced. [1]

1.2 Research Questions

This research seeks to address the feasibility of road diet conversions in Nebraska by identfying expected impacts to operations and other performance metrics that result from a road diet implementation. The primary objective is to develop the framework to determine road diet feasibility using inputs which are readily available to regional and local decision makers. There are two main questions examined to develop this framework: (1) because the information is readily available, how accurate of a predictor is AADT regarding the feasibility of a road diet conversion, and (2) beyond AADT, what are other readily available factors which are predictive of the feasibility of road diet conversions.

To answer the first question of AADT as an indicator for road diet feasibility, four roadway corridors from separate sites throughout Nebraska are assessed with before-and-after analysis for road diet conversions. This analysis is performed using both Synchro, implementing the methodology from the Highway Capacity Manual [2], and VISSIM, implementing traffic microsimulation analysis. The sites selected exhibit different levels of congestion, indicated by the AADT for the corridor, to establish a baseline for the range of AADT for which a road diet conversion is expected to be feasible. Then, congestion sensitivity analysis is conducted by running traffic operations analysis on multiple volume scenarios at each location. The volume conditions will be used to establish the ability of AADT to predict road diet feasibility across differing roadway geometries, seeking to make broader assumptions about the feasibility of road diet implementations in Nebraska. Recommendations are made regarding the range of AADT for which road diets are expected to be universally feasible, the range over which further study is needed, and the range of AADT for which road diets are expected to be infeasible.

To investigate the second question regarding other indicator variables for the feasibility of road diet implementations, a cost-benefit analysis will be conducted for all sites and volume conditions. Results of the cost-benefit analysis will be examined, relative to a variety of parameters such as average delay and change in average delay, to determine which factors have the greatest influence on the economic feasibility of road diet conversions.

1.3 Methodology

This research study involves the analysis of predicted impacts of road diet implementations converting four-lane roadways to two-lane roadways with a two-way left-turn lane (TWLTL). The primary inputs for the study are the existing roadway geometry and the observed peak-hour traffic at the case study sites selected. The primary performance metric for the roadway facilities is the operational analysis of both existing and proposed conditions, using the average delay per vehicle as a service measure. Secondary output measures include other measures of mobility such as average stops per vehicle, as well as cost-benefit analysis of the case study sites. A set of volume scenarios will be examined which increase the traffic demands until failure of the facility, to aid in investigating a broader range of AADT than what is able to be observed through field data collection. To collect the necessary data for this study, five MioVision Scout cameras were placed at major intersections along each study corridor.

The data was analyzed using two separate software applications, Synchro and VISSIM. Synchro is a traffic modeling software based on the Highway Capacity Manual (HCM) [2] methodology, utilizing macroscopic flow parameters to analyze traffic performance. VISSIM is a micro-simulation traffic modeling software, modeling the flow of each car through a road network and aggregating the individual results to generate performance measures. These programs were used for a "before and after" analysis by comparing the movement of vehicles for the current four-lane roadway cross-section to the movement of vehicles for the reduced capacity two-lane cross-section.

To establish an analysis baseline for the case study sites, the existing network performance is analyzed in Synchro, which is based on the Highway Capacity Manual. VISSIM analysis of the case study corridors was conducted by creating two models for each site. One model for the existing condition and one model for the road diet condition. An economic analysis of the effects of road diet improvements is conducted, performed in accordance with guidelines and procedures set forth by the American Association of State Highway and Transportation Officials (AASHTO) in the *User and Non-User Benefit Analysis for Highways*. [3]

1.4 Document Layout

This thesis is divided into nine chapters. Chapter 1 is the introduction. Chapter 2 reviews road diets: examining factors affecting feasibility and post-implementation impacts, and the current state of practice in cost-benefit economic analysis. Chapter 3 reviews the methodology for the data collection, operational analysis, and economic analysis required by the research. Chapter 4 reviews the case study sites selected for analysis as well as the methodology for selecting those sites. Chapter 5 reviews the collection and reduction of traffic data used as input in the research. Chapter 6 discusses the results of the simulations performed during the research. Chapter 7 discusses the limitations of the work performed during the course of the research. Chapter 8 presents the decision matrix created from the results of the simulations and analysis. Chapter 9 is the conclusion and recommendations for further research.

CHAPTER 2 LITERATURE REVIEW

The literature review for this study is broken out into four sections. The first section examines the current literature on the use of AADT to make determinations about the feasibility of reducing capacity through the repurposing of lanes as part of a road diet implementation. The second section reviews the literature on the safety improvements experienced as a result of road diet implementations; ideally a road diet should be implemented in a location where the existing facility has excess capacity, such that a reduction in lanes can provide opportunities for safer travel without reductions to operational performance. The third section of the literature review examines the inter-related operational performance metrics of speed, travel time, and average delay. The fourth and final section of the literature examines which additional feasibility factors have been utilized by other studies when assessing road diet implemenations.

2.1 Traffic Volume Impacts on Road Diet Feasibility

Many studies have conducted research on the effectiveness of road diet conversions in relation to the AADT of the roadway corridor on which the road diet is being implemented.

Welch conducted a study with sites in Seattle, Washington, and found that road diets to be feasible on roadways with AADT up to 20,000+ vehicles per day, with results shown below in **Table 2.1**. [4] The primary concern of Welch's investigation was to look into changes to crash rates and ADT; the author found that ADT increased on all locations despite the constrained capacity, while reported crash rates reduced or were held constant.

Table 2.1 Results of road diet conversion in Seattle, Washington [4]

Data on Street Conversions - Seattle, Washington					
ROADWAY SECTION	DATE CHANGE	ADT (BEFORE)	ADT (AFTER)	CHANGE	COLLISION REDUCTION
Greenwood Ave. N, from N 80th St. to N 50th St.	April 1995	11872	12427	4 lanes to 2 lanes plus TWLTL plus bike lanes	24 to 10 58%
N 45th Street in Wallingford Area	December 1972	19421	20274	4 lanes to 2 lanes plus TWLTL	45 to 23 49%
8th Ave. NW in Ballard Area	January 1994	10549	11858	4 lanes to 2 lanes plus planted median with turn pockets as needed	18 to 7 61%
Martin Luther King Jr. Way, north of I-90	January 1994	12336	13161	4 lanes to 2 lanes plus TWLTL plus bike lanes	15 to 6 60%
Dexter Ave. N, East side of Queen Anne Area	June 1991	13606	14949	4 lanes to 2 lanes plus TWLTL plus bike lanes	19 to 16 59%
24th Ave. NW, from NW 85th St. to NW 65th St.	October 1995	9727	9754	4 lanes to 2 lanes plus TWLTL	14 to 10 28%
Madison St., from 7th Ave. to Broadway	July 1994	16969	18075	4 lanes to 2 lanes plus TWLTL	28 to 28 0%
W Government Way/Gilman Ave. W, from W Ruffner St. to 31st. Ave. W	June 1991	12916	14286	4 lanes to 2 lanes plus TWLTL plus bike lanes	6 to 6 0%
12th Ave., from Yesler Way to John St.	March 1995	11751	12557	4 lanes to 2 lanes plus TWLTL plus bike lanes	16 to 16 0%
				Total	185 to 122 34%

Studying road diet conversions across the United States, Knapp and Giese found successful road diet conversions for roadways with AADT ranging from 8,400 to 24,000 vehicles per day, based on improvements to safety with negligible impacts to traffic operations, shown below in **Table 2.2**. [5]

Table 2.2 Results of 2001 road diet conversion study by Knapp and Giese [5]

Location	Approx. ADT	Safety	Operations
Montana			
Billings—17th Street West	9,200–10,000	62 percent total crash reduction (20 months of data)	No Notable Decrease**
Helena—U.S. 12	18,000	Improved**	No Notable Decrease**
Minnesota			
Duluth—21st Avenue East	17,000	Improved**	No Notable Decrease**
Ramsey County—Rice Street	18,700 Before 16,400 After	28 percent total crash reduction (3 years of data)	NA
Iowa			
Storm Lake—Flindt Drive	8,500	Improved**	No Notable Decrease**
Muscatine—Clay Street	8,400	Improved**	NA
Osceola—U.S. 34	11,000	Improved**	No Notable Decrease**
Sioux Center—U.S. 75	14,500	57 percent total crash reduction (1 year of data)	Overall travel speed decreased from 28–29 mph to 21 mph, and free-flow speed from 35 to 32 mph. There was a 70 percent decrease in speeds greater than 5 mph over the posted speed limit.
Blue Grass	9,200–10,600	NA	85th percentile speed reduction up to 4 mph (two locations increased 1 to 2 mph in one direction). The change in percent vehicles speeding depended upon location and direction (see discussion).
Des Moines (Note: This was a conversion from multiple cross sections to a three-lane)	14,000	NA	Average travel speed increased from 21 to 25 mph
California			
Oakland—High Street	22,000–24,000	17 percent in total crash reduction (1 year of data)	No notable change in vehicle speed
San Leandro—East 14th Street	16,000–19,300 Before 14,000–19,300 After	52 percent in total crash reduction (2 years of data)	Maximum of 3 to 4 mph spot speed reduction
Washington			
Seattle—Nine Locations	9,400–19,400 Before 9,800–20,300 After	34 percent average total crash reduction (1 year of data)	NA

*NA = Not Available. Safety data duration is for before/after conversion.
**Summarized results based on anecdotal information.

The Highway Safety Information System (HSIS) evaluated and found road diets to be successful on sites in Iowa with AADT volumes ranging from 3,718 to 26,376 vehicles per day in a 2010 study. [6] This study is notable for the large number of sites investigated, both of road diet implementations as well as control locations not undergoing geometric changes, as shown below in **Table 2.3**.

Table 2.3 Results of 2010 road diet study by HSIS [6]

DATABASE/SITE TYPE	CHARACTERISTIC	MEAN	MINIMUM	MAXIMUM
Iowa Treatment (15 sites)	Years before	17.53	11.00	21.00
	Years after	4.47	1.00	11.00
	Crashes/mile-year before	23.74	4.91	56.15
	Crashes/mile-year after	12.19	2.27	30.48
	AADT before	7,987	4,854	11,846
	AADT after	9,212	3,718	13,908
	Average length (mi)	1.02	0.24	1.72
Iowa Reference (296 sites)	Years	21.8	5	23
	Crashes/mile-year	26.8	0.2	173.7
	AADT	8,621	296	27,530
	Average length (mi)	0.99	0.27	3.38
HSIS Treatment (30 sites)	Years before	4.7	1.8	8.5
	Years after	3.5	0.6	8.8
	Crashes/mile-year before	28.57	0.00	111.10
	Crashes/mile-year after	24.07	0.00	107.62
	AADT before	11,928	5,500	24,000
	AADT after	12,790	6,194	26,376
	Average length (mi)	0.84	0.08	2.54
HSIS Reference (51 sites)	Years	7.82	4.50	12.17
	Crashes/mile-year	42.19	5.96	169.73
	AADT	15,208	1,933	26,100
	Average length (mi)	0.95	0.10	3.31

A report published by the Kentucky Transportation Center, conducted by Stamatiadis, Kirk, Wang, and Cull, concluded that road diets may be a feasible alternative on roadways with AADT as high as 23,000 vehicles per day but may also fail on roadways with low ADT depending upon side street volume. [7] Expanding on their previous research, Stamatiadis and Kirk refined their finding to recommend combined ranges of ADT for the major and minor roadways at which a road diet implementation would be recommended, not recommended, or requiring additional evaluation, as shown below in **Figure 2.1**. [8]

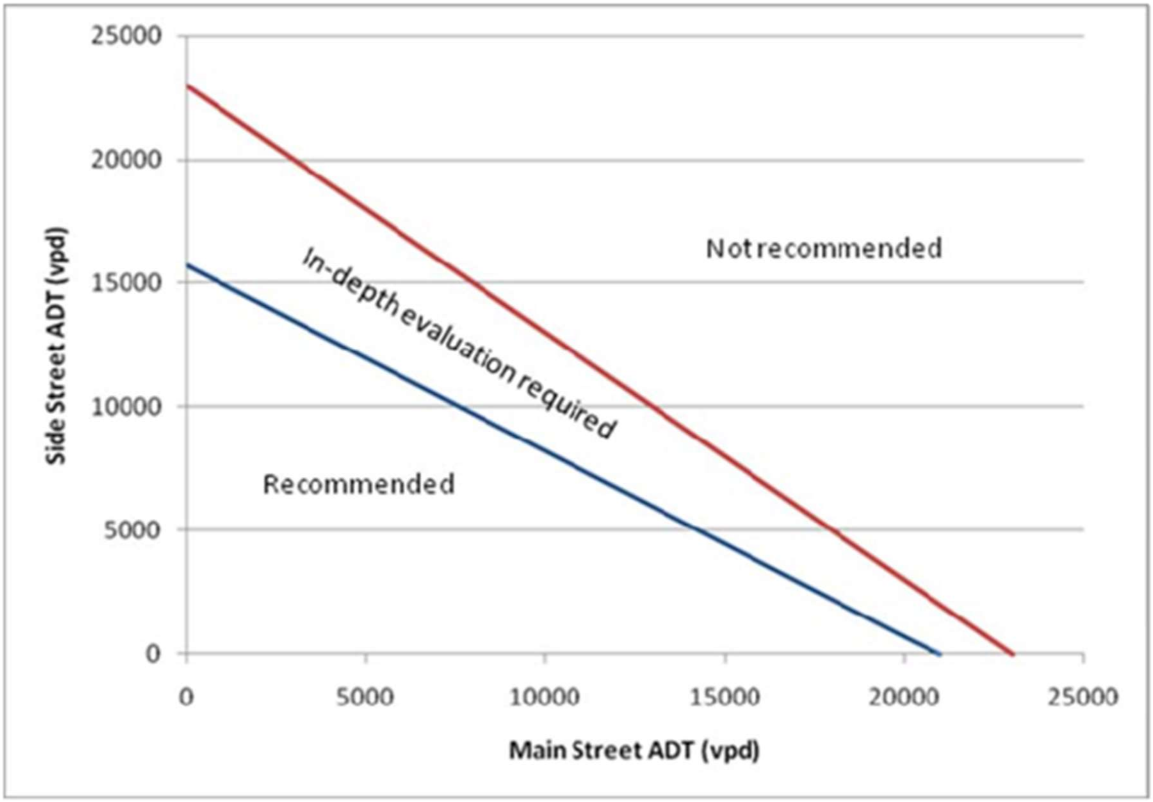

Figure 2.1 Results of 2012 road diet study by Stamatiadis and Kirk [8]

Pawlovich, Li, Carriquirry, and Welch found that AADT volumes for effective road diet conversions range from 2,030 to 15,350 vehicles per day. [9] Tan reported successfully implemented road diet conversions on sites with AADT ranging from 12,000 to 20,000 vehicles per day. [10]

2.2 Safety Impacts of Road Diets

So long as the demand flow-rate on the roadway is less than the reduced capacity, there is negligible impact on traffic operations with the implementation of a road diet, with the primary impact being an increase in safety on the facility. With ranges of appropriate

AADT values for road diets fairly well established, a study by Gates, Noyce, Talada, and Hill concluded that road diets are a recommended option within a given range of average daily traffic values if the roadway is experiencing safety problems related to the conflict of left-turn traffic with through traffic vehicles. [11]

Data previous presented in the context of AADT analysis of road diet implementations shed some light on the safety benefits experienced. **Table 2.1**, previously shown, reflects data from several street conversions in Seattle, Washington. It appears a 20 to 30% reduction in crashes would be a reasonable estimate of the potential safety improvement of a four- to three-lane conversion. [4] Research conducted by Knapp and Giese studied 13 road diet sites from Montana, Minnesota, California, Washington, and Iowa. The authors found that road diet conversions resulted in total crash reductions ranging from 17 to 62 percent, as previously shown in **Table 2.2**. [5]

A study by Pawlovich et al. compared 15 road diet sites against 15 comparison sites in Iowa to assess crash reduction due to road diets. Utilizing data provided over 23 years, the results of this research indicate a 25.2 percent reduction in crash frequency and an 18.8 percent reduction in crash rate. [9] Gates et al. examined nine road diet sites in Minnesota in 2007. Of the nine sites studies, seven sites had crash data available and it was determined that the road diets resulted in an overall crash reduction of 44.2 percent. [11] In an attempt to better consolidate road diet impacts, Persuad and Lyon reanalyzed previous studies with consistent analysis methods. This included a study with data from Washington and California as well as a more recent data from Iowa in an attempt to reduce differences in results by standardizing methodology. The combined study with revised methodology found a 47 percent reduction in total crashes for Iowa (25 percent originally) and a 19 percent reduction in total crashes for the HSIS study (6 percent originally). When all of the data sets are combined, crashes are reduced on average by 29 percent. [6] The results of this study are shown previously in **Table 2.3**.

In a study on the safety implications of road diet conversions, Welch assessed the change in collision rates for nine sites in Iowa. [4] The research identified collision rate reductions ranging from 0 percent to 61 percent for the individual sites and established an average, collision rate reduction of 34 percent across all sites. **Table 2.4** shows the three-year before- and after- midblock and non-signalized intersection crash information for a four to three-lane conversion project on Minnesota Trunk Highway 49 (Rice Street) in Ramsey County, Minnesota. The ADT on Rice Street during the after period was 16,400 vehicles per day.

Table 2.4 Collision results for 1999 study by Welch [4]

MTH-49 (Rice Street), Hoyt Avenue to Demont Avenue Ramsey County, MN			
Collision Type	Number of Collisions		Percent Change
	Before	After	
Rear End	68	39	-43
Sideswipe Passing	16	10	-38
Left Turn	23	20	-13
Right Angle	36	31	-14
Right Turn	2	2	0
Head On	5	0	-100
Sideswipe Opposing	2	1	-50
Off Road Left	1	2	+100
Off Road Right	4	1	-75
Other	5	8	+120
Total	162	117	-28

A study by Huang et al. analyzed 30 road diet sites from California and Washington using 50 additional control sites to determine the reduction in crash rates. The research found a 6 percent reduction in total crashes on average for road diet sites. [12] The results are reported in **Table 2.5** and **Table 2.6**, below.

Table 2.5 Results of 2003 road diet study by Huang, Stewart, Zegeer, and Esse [12]

Group Number	Site Type (Note 1)	Months of Data		Crashes		Percent		P-value
		Before	After	Before	After	After	χ^2_{1df}	
1	R	40	106	63	164	72.3	.009	.9255
1	C	40	106	347	917	72.6		
2	R	91	25	102	32	23.9	.039	.8444
2	C	91	26	231	76	24.8		
3	Note 2							
4	R	56	56	82	74	47.4	.014	.9048
4	C	56	56	583	537	48.0		
5	R	35	75	152	252	62.4	2.995	.0835
5	C	35	75	95	208	68.7		
6	R	50	60	85	97	53.0	.538	.4632
6	C	50	60	793	1005	55.8		
7	R	74	19	44	8	15.4	.015	.9030
7	C	74	19	188	36	16.1		
8	R	42	48	16	4	20.0	8.275	.0040
8	C	42	48	61	73	54.5		
9	R	66	12	255	28	9.9	3.479	.0621
9	C	66	12	661	110	14.3		
10	R	53	25	121	39	24.4	4.180	.0409
10	C	53	25	877	419	32.3		
11	R	61	8	407	43	9.6	.002	.9610
11	C	61	8	1210	129	9.6		
Total	R			1327	741	35.8	Note 3	Note 3
Total	C			5045	3510	41.0		

Table 2.6 Results of 2003 road diet study by Huang, Stewart, Zegeer, and Esse [12]

ANALYSIS CATEGORY	COMPARISON			
	Road Diets Before vs. After	Comparison Sites Before vs. After	"Before" Period Road Diets vs. Comparison Sites	"After" Period Road Diets vs. Comparison Sites
Crash Frequency	Reduction in "After" Period	No Change	No Difference	Road Diets Lower
Crash Rates	No Change	No Change	Road Diets Lower	Road Diets Lower
Crash Severity	No Change	No Change	No Difference	No Difference
Crash Type	No Change	No Change	Difference: 1. Road diets had a higher percentage of angle crashes 2. Road diets had a lower percentage of rear-end crashes	Difference: 1. Road diets had a higher percentage of angle crashes 2. Road diets had a lower percentage of rear-end crashes

Stamatiadis et al. studied the impacts of road diets at four sites in Kentucky. This 2011 study found that total crashes were reduced an average of 36 percent across the four sites. [7] Also conducted in 2011, a study by Tan analyzed road diets from Florida, Georgia, Iowa, and Washington. From the study sites, crash rates were reduced anywhere from 14 percent to 53 percent. [10] In 2014, Knapp et al. documented the results of 10 separate road diet studies while compiling the FHWA Road Diet Informational Guidebook. The compilation of this data noted that road diets resulted in a reduction in total crashes ranging from 6 percent to 53 percent. [13] The results of this study are shown in **Table 2.7**, below. Finally, a 2015 road diet cost-benefit analysis by Noland et al. in New Jersey found that road diet conversions result in a 19 percent crash reduction. [14]

Table 2.7 Results of 2014 Knapp et al. study in FHWA Road Diet Guidebook [13]

Reference	Treatment Sites	ADT	Key Safety Results
FHWA, 2010	45 sites in California, Iowa, and Washington	3,718 to 26,376	Iowa data: 47% reduction in total crashes California and Washington data: 19% reduction in total crashes Combined data: 29% reduction in total crashes
Noyce et al., 2006	7 treatment sites throughout Minnesota	8,900 to 17,400	Traditional before-after approach: 42-43% reduction in crashes. Yoked/group comparison analysis: 37% reduction in total crashes and 47% reduction in crash rates. EB approach: 44% reduction in total crashes.
Pawlovich et al., 2006	15 treatment sites throughout Iowa	4,766 to 13,695	25.2% reduction in crash frequency per mile; 18.8% reduction in crash rate.
Li and Carriquiry, 2005	15 treatment sites throughout Iowa	3,007 to 15,333	29% reduction in the frequency of crashes per mile; 18% reduction in the crash rate.
Huang et al., 2003	12 treatment sites in California and Washington	10,179 to 16,070	6% reduction in total crashes relative to control; no reduction in crash rate.
Lyles et al., 2012	24 treatment sites throughout Michigan	3,510 to 17,020	9% reduction in total crashes (non-significant).
Stout, 2005 Stout et al., 2005 Stout (year unknown)	11 to 15 treatment sites in various Iowa cities	2,000 to 17,400	21 to 38 percent reduction in total crashes; similar reduction in crash rates.
Clark, 2001	One treatment site in Athens-Clarke County, GA	18,000 to 20,000	52.9% reduction in total crashes; 51.1% reduction in crash rate (first 6 months).
City of Orlando, 2002	One treatment site in Orlando, FL	18,000 to 20,000	34% reduction in crash rate; 68% reduction in injury rate (first 4 months).
Preston, 1999	Minnesota	Not Provided	27% lower crash rate on three-lane roads than on four-lane undivided roadways (cross-sectional comparison – not a before-after study)

2.3 Operational Impacts of Road Diets – Speed, Travel Time, and Average Delay

Aside from impacts on collision rates, road diet conversions have a potential impact on travel time. This can be due to a reduction in free flow speed due to the loss of a lane, or the increase in control delay at intersections. Welch found road diet conversions to reduce average travel speed by 1.7 miles per hour. [4] The results of this study's speed reduction analysis are shown in **Table 2.8**.

Table 2.8 Speed reduction results from 1999 study on road diets by Welch [4]

Arterial LOS
US Highway 75 Corridor from 1st Street to N. 4th Street

Cross Section	Total Corridor Travel Delay	Average Travel Speed	LOS
Four lane undivided	20.5 secs	16.0 mph	C
Three lane alternative	29.4 secs	14.3 mph	C
Five lane alternative	15.8 secs	17.1 mph	C

Knapp and Giese found that road diet conversions reduce the average arterial or 85th percentile speed by 5 miles per hour or less. Results from the 2001 study also noted that road diet conversions reduce the number of drivers traveling 5 miles per hour or more above the posted speed limit, this reduction typically ranged from 60 to 70 percent. [5] See **Tables 2.9, 2.10,** and **2.11** for more detailed information on speed reduction effects of road diets from this study.

Table 2.9 Speed impact results from 2001 road diet study by Knapp and Giese [5]

Sensitivity Analysis Factor	Undivided Four-Lane Average Arterial Travel Speed (mph)	Three-Lane Average Arterial Travel Speed (mph)	Average Arterial Travel Speed Difference (mph)*	Min. Diff.* (mph)	Max. Diff.* (mph)
Total Entering Volume (vphpd)**					
500	22.0	20.7	1.3	0.8	1.7
750	20.3	19.6	0.7	0.0	2.0
875	20.5	18.7	1.8	1.5	2.7
1000	19.4	16.3	3.1	1.8	3.8
Access Point Density (ppm)					
0	22.1	19.6	2.5	1.5	3.9
10	21.5	19.6	1.9	0.8	3.5
20	21.2	19.2	2.0	0.7	3.2
30	20.3	18.6	1.8	0.4	3.5
40	19.8	18.1	1.7	0.0	3.8
50	20.1	18.6	1.4	0.2	3.6
Access Point Left-Turn Volumes (percent of total entering volume)					
10	20.6	18.7	1.9	0.0	3.6
20	20.6	18.8	1.7	0.2	3.8
30	20.6	18.9	1.7	0.4	3.7

*Difference = Average Arterial Travel Speed with the Four-Lane Undivided Cross Section - Average Arterial Travel Speed with the Three-Lane Cross Section; vphpd = vehicles per hour per direction; ppm = access points per mile per roadway side.
**Differences for total volumes do not include those for the corridor with no access points.

Table 2.10 Speed impact results from 2001 road diet study by Knapp and Giese [5]

Total Entering Volume (vphpd)*	Access Point Left-Turn Volume (percent of total entering volume)	Four-Lane Undivided Average Arterial Travel Speed (mph)*	Three-Lane Average Arterial Travel Speed (mph)*	Average Arterial Travel Speed Difference (mph)*
500	10	22.1	20.6	1.5
	20	22.0	20.6	1.3
	30	21.9	20.8	1.1
Avg.	—	22.0	20.7	1.3
750	10	20.5	19.6	0.9
	20	20.3	19.7	0.7
	30	20.3	19.7	0.6
Avg.	—	20.3	19.6	0.8
875	10	20.6	18.6	2.0
	20	20.5	18.7	1.8
	30	20.5	18.7	1.8
Avg.	—	20.5	18.7	1.9
1,000	10	19.3	16.2	3.1
	20	19.5	16.3	3.1
	30	19.6	16.4	3.3
Avg.	—	19.4	16.3	3.2

* Difference = Average Arterial Travel Speed with the Four-Lane Undivided Cross Section - Average Arterial Travel Speed with the Three-Lane Cross Section; vphpd = vehicles per hour per direction.
**Any discrepancies in table are due to round-off error.

Table 2.11 Speed impact results from 2001 road diet study by Knapp and Giese [5]

Access Point Density (ppm)*	Left-Turn Volume (percent of total entering volume)*	Four-Lane Undivided Average Arterial Travel Speed (mph)*	Three-Lane Average Arterial Travel Speed (mph)*	Average Arterial Travel Speed Difference (mph)*
0	0	22.1	19.6	2.5
10	10	21.6	19.6	2.0
	20	21.5	19.6	1.9
	30	21.3	19.7	1.6
Avg.	—	21.5	19.6	1.9
20	10	21.5	19.1	2.4
	20	21.1	19.2	1.9
	30	20.9	19.2	1.7
Avg.	—	21.2	19.2	2.0
30	10	20.3	18.4	2.0
	20	20.4	18.7	1.7
	30	20.3	18.7	1.7
Avg.	—	20.3	18.6	1.8
40	10	19.8	18.1	1.7
	20	19.8	18.1	1.8
	30	19.9	18.2	1.8
Avg.	—	19.8	18.1	1.7
50	10	19.9	18.6	1.3
	20	20.0	18.7	1.3
	30	20.3	18.6	1.7
Avg.	—	20.1	18.6	1.4

* Difference = Average Arterial Travel Speed with the Four-Lane Undivided Cross Section - Average Arterial Travel Speed with the Three-Lane Cross Section; vphpd = vehicles per hour per direction, ppm = access points per mile per roadway side.
**Any discrepancies in table are due to round-off error.

A 2007 study conducted by Gates et al. observed a median reduction of 2 miles per hour each in both the mean speed and the 85th percentile speed. [11] In 2008, a study evaluating road diets in New Jersey found that average speeds were reduced between 1 and 9 miles per hour. [15] The 2010 HSIS study to standardize evaluation methods also examined road diet impacts on speed. Iowa sites experienced a 4 to 5 mph reduction in 85th percentile free-flow speed and a 30 percent reduction in the percentage of vehicles traveling in excess of 5 mph above the speed limit. [6] In a 2008 report, Tan documented speed reductions ranging from 18 percent to 52 percent across multiple sites. The total number of drivers traveling above the speed limit experienced a reduction of 8 to 10 percent. [10] Li and Tian conducted a road diet case study in Reno, Nevada that found average speeds to be reduced by 2 to 3 miles per hour. [16]

The change in travel time as the result of road diet conversions can have an impact on the Level-of-Service (LOS) of the roadway corridor. However, each LOS regime has a range of values of delay under which that particular grade is constant, based on the criteria established in the Highway Capacity Manual. [2] Therefore, small increases in travel time may or may not impact the LOS calculated for a facility, based on how close the original delay value is to the threshold between LOS regimes. An example of how average delay per vehicle and LOS regimes are related is shown below, in **Table 2.12**.

Table 2.12 Motorized vehicle LOS criteria for signalized intersections [2]

Control Delay (s/veh)	LOS by Volume-to-Capacity Ratio[a] ≤1.0	>1.0
≤10	A	F
>10–20	B	F
>20–35	C	F
>35–55	D	F
>55–80	E	F
>80	F	F

Note: [a] For approach-based and intersectionwide assessments, LOS is defined solely by control delay.

In his 1999 study on road diet conversions, Welch concluded that although total delay increases for both the corridor and individual intersections, the delay is not significant enough to affect LOS. [4] Results from the LOS analysis for this study are shown in **Tables 2.8**, above and **2.13**, below.

Table 2.13 Results of LOS analysis from Welch's 1999 road diet study [4]

Intersection Performance Summary
US 65 / Brooks Rd. Intersection

Existing 4 lane undivided:							
	Lane Mvmts	v/c Ratio	g/C Ratio	Mvmt: Delay	LOS	Approach: Delay	LOS
EB	LTR	0.356	0.314	12.2	B	12.2	B
WB	LTR	0.379	0.314	12.4	B	12.4	B
NB	LTR	0.342	0.600	4.6	A	4.6	A
SB	LTR	0.293	0.600	4.4	A	4.6	A
Intersection Delay = 6.2 sec/veh					Intersection LOS = B		

Proposed 3-lane with TWLT Lane:							
	Lane Mvmts	v/c Ratio	g/C Ratio	Mvmt: Delay	LOS	Approach: Delay	LOS
EB	LTR	0.356	0.134	12.2	B	12.2	B
WB	LTR	0.379	0.314	12.4	B	12.4	B
NB	L	0.234	0.600	4.3	A	5.1	B
	TR	0.457	0.600	5.2	B		
SB	L	0.139	0.600	4.0	A	5.0	A
	TR	0.438	0.600	5.1	B		
Intersection Delay = 6.7 sec/veh					Intersection LOS = B		

L-Left, T-Through, R-Right, LOS-Level of Service

When analyzing the effects of road diet conversions in their 2001 study, Knapp and Giese found the LOS of both the existing and road diet cross sections was either LOS B or LOS C. Therefore, the cross sections primarily had the same LOS, except for a small number of input variable combinations. Average intersection stop delay was also used to determine the LOS at the signalized intersections along the simulated corridor. Overall, it was determined that the signalized intersections in the simulations operated at LOS B for all sensitivity analysis factor combinations. [5] **Tables 2.2, 2.9, 2.10** and **2.11** show the results of examining the impact of road diet conversion on operations for this study. A study by Li and Tian found that road diet conversions increase the LOS of intersections along the corridor by one level at most. [16] **Tables 2.14** and **2.15** show the changes in LOS as a result of road diet conversion for this study.

Table 2.14 Results of road diet LOS analysis from study by Li and Tian [16]

Intersections	Existing			Future		
	Int. LOS	Critical LOS	V/C	Int. LOS	Critical LOS	V/C
Mayberry Drive/McCarran Boulevard	B	D (NBL)	0.47	B	C (NBL)	0.46
Mayberry Drive/Hunter Lake Drive/Charles Drive	B	C (EBL)	0.09	B	C (EBL)	0.13
California Avenue/Booth Street	B	C (NEL)	0.77	C	D (NEL)	0.84
California Avenue/Keystone Avenue	B	C (NEL)	0.76	C	D (SWT)	0.88
California Avenue/South Arlington Avenue/Clay Street	B	C (WBL)	0.11	C	D (WBL)	0.13
California Avenue/South Sierra Street	A	B (EBT)	0.49	A	B (EBT)	0.58
California Avenue/South Virginia Street	A	B (SBT)	0.37	A	B (SBT)	0.37

Table 2.15 Results of road diets LOS analysis from study by Li and Tian [16]

Intersections	Existing			Future		
	Int. LOS	Critical LOS	V/C	Int. LOS	Critical LOS	V/C
Mayberry Drive/McCarran Boulevard	D	D (EBL)	0.84	C	D (SBL)	0.87
Mayberry Drive/Hunter Lake Drive/Charles Drive	B	C (WBL)	0.67	C	D (EBL)	0.70
California Avenue/Booth Street	B	C (NEL)	0.30	C	C (NEL)	0.43
California Avenue/Keystone Avenue	B	C (NEL)	0.53	C	D (NEL)	0.69
California Avenue/South Arlington Avenue/Clay Street	C	D (EBL)	0.53	D	D (NBL)	0.87
California Avenue/South Sierra Street	A	B (WBT)	0.58	B	B (WBT)	0.73
California Avenue/South Virginia Street	B	C (EBL)	0.43	B	C (EBL)	0.43

2.4 Additional Feasibility Factors

Several studies looked at factors to determine the feasibility of implementing road diet improvements on a roadway corridor. Knapp, Welch, and Witmer examined feasibility determination factors that include: roadway function; total traffic volume; turning volumes and patters; weaving, speed, and queues; accident type and patters; pedestrian and bike activity; right-of-way availability and cost. [17]

In a follow-up study, Knapp et al. identify several feasibility factors. Factors considered to determine feasibility include the need for improved safety for all road users; a desire to incorporate context sensitive solutions and complete street features; roadway function/operation; vehicle speed; ADT; multimodal LOS; peak hour volumes and directions; turning volumes and patters; presence of slow-moving or frequently stopping vehicles; a desire to better accommodate bicycles, pedestrians, and transit service; right-of-way availability and cost; existence of parallel roadways, parallel parking, and at-grade railroad crossings; public outreach, public relations, and political considerations. [13]

CHAPTER 3 ANALYSIS METHODOLOGY

This research study involves the analysis of predicted impacts from converting four-lane roadways to two-lane roadways with a two-way left-turn lane (TWLTL). The primary inputs for the study are the existing roadway geometry and the observed peak-hour traffic at the sites selected for study. The primary output of the study is the operational analysis of the roadway with both existing and proposed conditions, using the average delay per vehicle as a service measure. Secondary output measures include other measures of mobility, such as average stops per vehicle, operations relative to various levels of average annual daily traffic, and cost-benefit analysis of sample road diet implementations.

3.1 Data Collection and Reduction

To collect the necessary data for this study, five MioVision Scout cameras were placed at major intersections along each study corridor. An example of the equipment setup is shown in **Figure 3.1**. Given that the number of intersection of interest exceeded the number of cameras, peak hour video data collections were conducted on multiple days, alternating the intersections being studied to provide a more comprehensive coverage for turning movement data.

Figure 3.1 Setup of MioVision Scout cameras

The collected video data was uploaded for processing by MioVision, which analyzes the video data and provides traffic volume measurements as turn movement counts for each fifteen-minute period for the duration of the video recording.

3.2 Operational Analysis of Roadway Conditions

The data was analyzed using two separate software applications: Synchro and VISSIM. Syncrho is a traffic modeling software based on the Highway Capacity Manual (HCM) [2] methodology, which utilizes macroscopic flow parameters to analyze traffic performance. VISSIM is a micro-simulation traffic modeling software, modeling the flow of each car through a road network. [18]

These programs were used for a "before and after" analysis by comparing the movement of vehicles for the current highway configuration to the movement of vehicles for the reduced-capacity configuration. Currently, the four-lane highway is comprised of two through lanes in each direction without any median. The road diet configuration would include one through lane in each direction and a two-way left-turn lane, similar to the configuration shown in **Figure 3.2**.

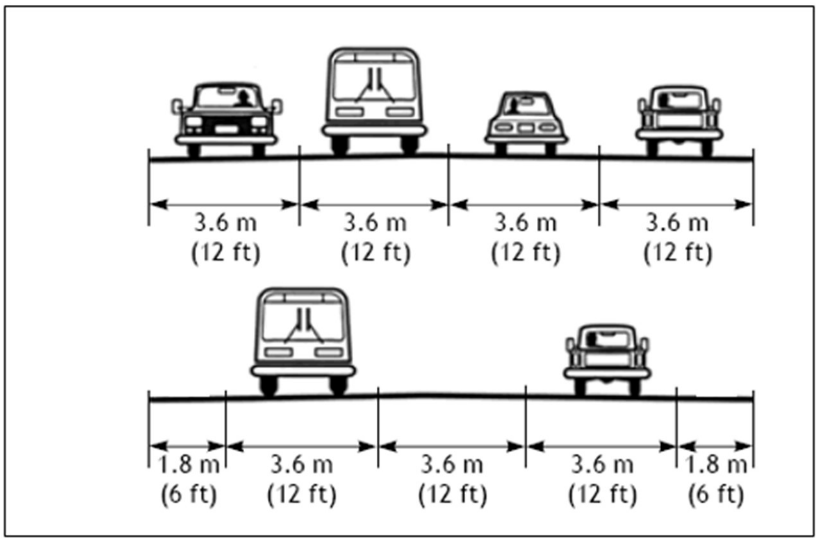

Figure 3.2 Typical section for existing (top) and road diet conditions (bottom) [19]

3.2.1 Synchro (HCM) Analysis of Existing and Road Diet Conditions

To establish an analysis baseline for the case study sites, the existing network performance is analyzed in Synchro, which implements analysis based on the HCM methodology. [2] Using an HCM-based analysis software is beneficial to further analysis of the case study sites since three of the sites have at least one signalized intersection along the corridor of interest: analysis performed using Synchro can determine the optimal cycle lengths and phase splits for signalized intersections. The website Google Maps was utilized to measure the roadway geometrics along the corridors. Across the corridors of interest during the two-hour data collection window, recorded heavy vehicle percentage ranged from 1.73% to 2.16%. Therefore, the heavy vehcile percentage used in analysis was standardized at 2% across each corridor. The lane configuration at each intersection was set to match the existing intersection geometry, including both number of lanes as well as storage length of auxiliary turn lanes. Because Synchro does not have a built in option for modeling a two-way left-turn lane, the functionality of the road diet cross section was modeled using a dedicated left turn lane and single lane shared by thru and right turn movements at every intersection. Speed limits along the corridor were entered in the model based on field observations of posted signage. Each intersection along the study corridors was assigned a unique numerical identifier. This allowed the turning movement volumes to be imported from a UDTF input file.

After turning movement volumes were imported from the input file, signal timing for signalized intersections was optimized using Synchro's internal signal optimization tool. The information on the performance reports includes control delay and traffic volume for each movement. This information was then used to calculate the total network delay using **Equation 3-1**:

$$D = \frac{\sum d_i V_i}{\sum V_i} \qquad \mathbf{(3-1)}$$

Where D is the average delay per vehicle for the network in seconds per vehicle, d_i is the control delay for turning movement i in seconds per vehicle, and V_i is the vehicle volume for turning movement i.

3.2.2 VISSIM Analysis of Existing and Road Diet Conditions

VISSIM analysis of the case study corridors was conducted by creating two models for each site. One model for the existing condition and one model for the road diet condition. To measure vehicle travel times and determine delay, “Vehicle Travel Time Measurement” objects were created in the network. These devices were placed 5 units from the beginning of any link and at the terminating location of any previous measurement device. This placement ensures that there is no position along the modeled corridor at which a simulated vehicle is not having travel time recorded. To determine network delay, an extra delay measurement was added to the model, which included the data collected by all of the vehicle travel time measurements and generated an aggregate delay for the whole network. VISSIM does not have a specific setting to allow for the creation and modeling of a two-way left-turn lane. To work around this and model the road diet condition, a dedicated left turn lane was added along the corridor of interest at every intersection. These dedicated lanes were then extended upstream of the intersection to allow for separate queuing as would be expected in the field. In the models, the simulation parameters were set to allow for loading, recording, and unloading times.

Because of the queuing issues associated with high volumes the side streets were extended to provide room for queuing and recording of queued vehicles. The first five minutes of simulation time was used for loading the network, recording the data of all vehicles scheduled to enter the network between 5 minutes and 20 minutes from the start of the simulation, allowing each run of the simulated scenario to last for 25 minutes to clear all vehicles through the system. The sensitivity analysis is performed by varying the input demand flow-rates for the models. The input volumes include 100, 150, 200, 250, 300, 350, and 400 percent of the observed demand flow-rate.

3.3 Economic Analysis of Road Diet Impacts

3.3.1 Economic Analysis Procedure

As it is an easily understood metric, assigning a dollar value to more abstract quantities allows the benefits and costs of a project to be laid out in a simpler manner. In 1991, Zeeger et. al published a study assessing cost-effective geometric improvements to horizontal curves for safety considerations. The general methodology used in the study involved determining the impact of geometric improvements on accident severity and rates. Changing severity and rates were then assigned dollar values based on the cost of individual accident severity types and the difference between the current and expected frequency of accidents. This established the general economic benefit of the geometric improvement, which could then be compared against the cost of implementation to determine the net benefit or cost. [20] Valuating changes in travel times has been notably assessed by the Transportation Research Board in 1999. The research laid out processes of valuing travel time on the basis of differing incomes as well as assigning additional value for metrics such as the time of day and the predictability of travel times. [21] The primary source for methodology on economic analysis of roadway improvement projects is AASHTO's *User and Non-User Benefit Analysis for Highways*. [3] In this guidebook, a general procedure is laid out for identifying and assigning costs to factors such as delay. The effects of delay on costs are shown to be quantified in multiple ways: from the change in travel time cost to changes in operating costs from fuel consumption.

3.3.2 Economic Analysis Methodology

While delay is quantifiable, as is the change in delay that's anticipated to occur due to a road diet improvement, it is a more complicated analysis to measure and quantify the impact of this change in terms of cost or value. To address this issue, an economic analysis of the effects of road diet improvements is conducted. The effects of a roadway improvement are identified as affecting three types of costs: travel time costs (H_c), operating costs (OC_c), and accident costs (AC_c) for each vehicle class, c. These individual cost categories are then used to determine the total cost (B_c), or benefit, imposed on the user by the roadway improvement project using **Equation 3-2**:

$$B_c = \Delta U_c \left(\frac{V_{c,0} + V_{c,1}}{2}\right) L = (\Delta H_c + \Delta OC_c + \Delta AC_c) \left(\frac{V_{c,0} + V_{c,1}}{2}\right) L \qquad \mathbf{(3-2)}$$

Where B_c is the user benefit, ΔU_c is the change in user cost per vehicle mile traveled (VMT), ΔH is the change in the value of travel time per VMT (or per-user), ΔOC is the change in operating costs per VMT (or per-user), ΔAC is the change in unreimbursed accident costs per-VMT (or per-user), V_0 is the demand flow rate of vehicles (or users) without the improvement, V_1 is the demand flow rate of vehicles (or users) with the improvement, and L is the segment or corridor length in miles. [3] The change in individual costs are calculated using **Equation 3-3**.

$$\Delta H_c = \frac{100 M_c O_c \Delta D_c}{3600} \qquad \mathbf{(3-3)}$$

In **Equation 3-3**, ΔH_c is the value of travel time due to delay in cents per vehicle, M_c is the unit value of time for users in dollars per hour, O_c is the occupancy rate of vehicles, and ΔD_c is the change in delay in seconds. [3] For the case study locations, information collected by the United States Census Bureau was used to determine the unit value of time, in 2015 dollars, and occupancy rate. The values used in this analysis are shown in **Table 3.1.**

Table 3.1 Unit value of time and occupancy rates for case study locations

Site	Mc ($/hr)	Oc
Alliance	10.84	1
Aurora	10.98	1
David City	10.24	1
Ord	10.23	1

$$\Delta OC_c = (gal_{c,min})(D_0 - D_1)P_c \qquad \mathbf{(3-4)}$$

In **Equation 3-4**, ΔOC_c is the change in operating costs in cents per vehicle, $gal_{c,min}$ is the fuel consumption in gallons per minute, D_0 is the average delay per vehicle (in minutes) without the improvement, D_1 is the average delay per vehicle (in minutes) with the improvement, and P_c is the fuel price in cents per gallon. [3] Inferring from **Table 3.2**, shown below, by assuming a free flow speed of 35 miles per hour and an equal mix of small and big cars, $gal_{c,min}$ is taken to be 0.026 gallons per minute for this analysis. P_c is taken to be $2.51 per gallon based on information provided by the Official Nebraska Government Website for 2017. [22]

Table 3.2 Fuel consumption (gallons) per minute of delay [3]

Free Flow Speed	Small Car	Big Car	SUV	2-Axle SU	3-Axle SU	Combo
20	0.011	0.022	0.023	0.074	0.102	0.198
25	0.013	0.026	0.027	0.097	0.133	0.242
30	0.015	0.030	0.032	0.122	0.167	0.284
35	0.018	0.034	0.037	0.149	0.203	0.327
40	0.021	0.038	0.043	0.177	0.241	0.369
45	0.025	0.043	0.049	0.206	0.280	0.411
50	0.028	0.048	0.057	0.235	0.321	0.453
55	0.032	0.054	0.065	0.266	0.362	0.495
60	0.037	0.060	0.073	0.297	0.404	0.537
65	0.042	0.066	0.083	0.328	0.447	0.578
70	0.047	0.073	0.094	0.360	0.490	0.620
75	0.053	0.080	0.105	0.392	0.534	0.661

Source: ECONorthwest calculations based on HERS model equations.

$$\Delta AC_c = v_D \Delta D + v_I \Delta I + v_P \Delta P \qquad \textbf{(3 - 5)}$$

In **Equation 3-5**, ΔAC_c is the change in accident costs in cents per vehicle mile, v_D is the perceived cost associated with a fatal accident in cents, v_I is the perceived cost associated with an injury accident in cents, v_P is the perceived cost associated with a property damage accident in cents, ΔD is the change in expected number of fatal accidents per mile, ΔI is the change in expected number of injury accidents per mile, and ΔP is the change in expected number of property damage accidents per mile. [3] From existing literature, a road diet improvement is estimated to reduce accident rates of all types by 33 percent. [4], [6], [7], [9]–[14] Because the Nebraska Department of Transportation does not report crash data for three of the case study sites specifically, but rather reports the data by county, the crash rates used in analysis will be the county average. The specific data and rates used are shown in **Table 3.3**. Note that the values for perceived cost are shown in year 2000 dollar values.

Table 3.3 Crash data reported by Nebraska DOT [22]–[24]

Site	VMT (millions)	Fatality Accidents Reported	Injury Accidents Reported	Property Damage Accidents Reported	DR (acc/million VMT)	IR (acc/million VMT)	PR (acc/million VMT)	v_D	v_I	v_P
Alliance	64.636	1	52	137	0.015	0.805	2.120	$ 3,723,700	$ 108,600	$ 200
Aurora	76.021	1	53	166	0.013	0.697	2.184	$ 3,723,700	$ 108,600	$ 200
David City	106.505	1	35	60	0.009	0.329	0.563	$ 3,723,700	$ 108,600	$ 200
Ord	39.087	1	9	39	0.026	0.230	0.998	$ 3,723,700	$ 108,600	$ 200

Since the traffic data collected and delay results all correspond to the peak hour, the user benefit calculated (B_c) must corresponds to one peak hour. However, peak hour benefits can be estimated as equivalent values for longer periods of time, as shown in **Equation 3-6**, below. To expand a peak hour benefit to a daily benefit:

$$B_{daily,total} = \left[\frac{1}{(-0.057b^2 + 1.636b + 1.34)\left(\frac{V_h}{V_{daily}}\right) + 0.004b^2 - 0.098b - 0.03}\right] B_{h,total} = \eta B_{h,total} \qquad \mathbf{(3-6)}$$

Where $B_{daily,total}$ is the benefits to all traffic in a weekday, $B_{h,total}$ is the peak hour benefits to all traffic, V_h is the peak hour traffic volume, V_{daily} is the AADT, b is the exponent in the BPR-type volume-delay formula (b is also β), and η is the peak-to-daily benefit expansion factor (1<η<24). [3] Because the case study sites are small in population and traffic volume, b is assumed to be one. Daily benefits are expanded to monthly benefits using **Equation 3-7**, below.

$$B_{monthly} = \mu B_{weekly} = \mu\kappa B_{daily} \qquad \mathbf{(3-7)}$$

Where $B_{monthly}$ is the monthly benefits to all traffic, B_{weekly} is the weekly benefits to all traffic, B_{daily} is the benefits to all traffic in a weekday, μ is the weekly-to-monthly expansion factor (4.345 typically), and κ is the daily-to-weekly expansion factor (5.0 to

5.5 typically). [3] For this analysis, μ is assumed to be 4.345 and κ is assumed to be 5.0. Finally, monthly benefits are expanded to yearly benefits using **Equation 3-8**, below.

$$\mathrm{B_{yearly}} = \left[\frac{1}{(0.076\mathrm{b} + 0.206)\left(\frac{\mathrm{ADT}}{\mathrm{AADT}} - 1\right) + \frac{1}{12}}\right]\mathrm{B_{monthly}} \qquad \mathbf{(3-8)}$$

Where B_{yearly} is the yearly benefits to all traffic, B_{montly} is the monthly benefits to all traffic, b (β) is the exponent in the BPR-type volume-delay formula, and ADT/AADT is the ratio of monthly average daily traffic to annual average daily traffic. [3] Since traffic counts were collected in different months for different case study sites, ADT/AADT differs from site to site. David City traffic data was collected in May: for May in Nebraska, ADT/AADT equals 1.060. Aurora traffic data was collected in June: for June in Nebraska, ADT/AADT equals 1.098. Alliance and Ord traffic data was collected in August: for August in Nebraska, ADT/AADT equals 1.052. [24]

For the economic analysis to be conducted, dollar values must be converted to the same year. The data collected used dollar values from the years 2000, for perceived accident costs, 2015, for value of trip maker's time, and 2017 for gas prices. For the analysis, these values will all be converted to year 2017 dollar values. Converting the dollar value year is performed by multiplying the known dollar value by the ratio of consumer price index, hereafter CPI, for the desired year to CPI for the known year. The average CPI's for 2000, 2015, and 2017 are 172.2, 237.0, and 245.1 respectively.

CHAPTER 4 CASE STUDY SELECTION

In determining case study site locations, several criteria were used. Sites were considered desirable for the study based on the presence of a four-lane cross section, the perceived receptiveness of local decision makers to a possible road diet conversion, and a desire to have a variety of each for the location within the state, the population, and the AADT. Based on these criteria, four case study site locations were selected with the input of NDOT. The four locations selected are Alliance, Aurora, David City, and Ord. These sites were chosen because they represented a range of locations throughout the state as well as a range of AADT. Aurora and David City were identified as being receptive towards the idea of road diet conversions and Alliance and Ord were identified as being resistant towards the idea of road diet conversions.

4.1 Case Study Site Location – Alliance, Nebraska

The first location selected for the case study is in Alliance, Nebraska. A segment of road along Highway 2 from Flack Avenue to Black Hills Avenue was selected, approximately one mile in length, and encompassing 17 intersections, as shown in **Figure 4.1**. Alliance is a town in northwestern Nebraska with a population of approximately 8,500 people. The population in 1970 was 6,862 people, indicating slight growth over the past 50 years. While many towns in rural Nebraska have experienced similar growth, there are many others that have experienced a decrease in population over the past few decades [25].

Figure 4.1 Site location in Alliance, Nebraska [26]

Although the population has increased, the amount by which it has done so is not sufficient to necessitate the expansion to four lanes on Highway 2. Although the traffic volume on this corridor is the highest volume of all the case study sites, with an AADT of 10,040 on record with the Nebraska Department of Transportation (NDOT), the volume is toward the middle of the effective road diet AADT ranges established by existing literature. [27]

Figure 4.2 shows the typical street cross-section for Highway 2 in Alliance, Nebraska. The cross-section shown looks east along Highway 2 between Sweetwater Avenue and Yellowstone Avenue. The overall width of the traveled way is 68 feet. This includes four 12-foot lanes for traffic, two in each direction, and a 10-foot shoulder along the outer sides of the street. Separated from the roadway by a 10-foot buffer, a 4-foot sidewalk runs along both the north and south sides of Highway 2. The land-use surrounding the highway is primarily commercial, and a majority of the lots have driveways connecting directly to Highway 2, creating the potential for a high degree of midblock friction depending on the trip generation characteristics of each lot. A road diet implemented in Alliance would result in a typical cross-section consisting of a 12-foot, two-way left-turn lane, two 12-foot, through lanes, and a 16-foot shoulder on each side.

Figure 4.2 Typical street cross-section – Alliance, Nebraska [26]

4.2 Case Study Site Location – Aurora, Nebraska

The second location selected for the case study is in Aurora, Nebraska. A segment of road along Highway 34 from 1st Street to 16th Street was selected, approximately one mile in length, and encompassing 12 intersections, as shown in **Figure 4.3**. Aurora is a town in central-southeastern Nebraska with a population of approximately 4,500 people. The population in 1970 was 3,180 people, indicating slight growth over the past 50 years. Of the four case study sites, Aurora has experienced the most significant population growth at approximately 40 percent growth whereas Alliance and David City have both registered approximately 20 percent growth, and Ord has experienced negative growth.

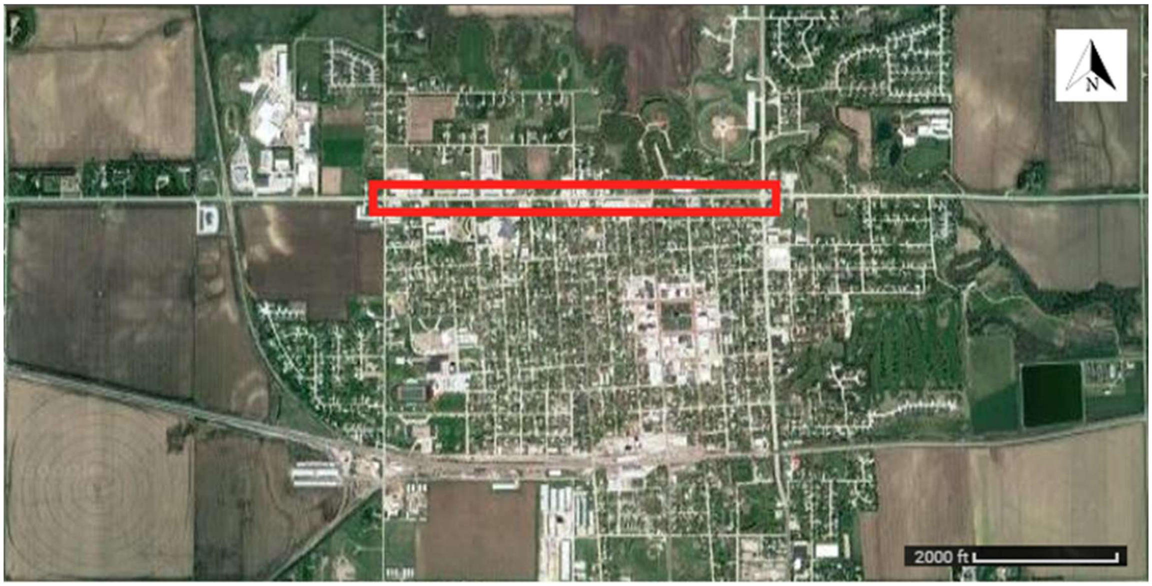

Figure 4.3 Site location in Aurora, Nebraska [26]

Although the population has increased, the amount by which it has done so is not sufficient to necessitate the expansion to four lanes on Highway 34. With an AADT of 5,850 on record with NDOT, Aurora's traffic volumes are in the lower end of the range of effective road diet volumes established in existing literature. [27]

Figure 4.4 shows the typical street cross-section for Highway 34 in Aurora, Nebraska. The cross-section shown looks east along Highway 34 between 8th Street and 9th Street. The overall width of the traveled way is 48 feet. This includes four 12-foot lanes for traffic, two in each direction, and no shoulders. Separated from the roadway by a 6-foot buffer, a 4-foot sidewalk runs along both the north and south sides of Highway 34. The land-use surrounding the highway is a mix between residential and commercial, and just under half of the lots have driveways connecting directly to Highway 34. A road diet implemented in Aurora would result in a typical cross-section consisting of a 12-foot, two-way left-turn lane, two 12-foot, through lanes, and a 6-foot shoulder on each side.

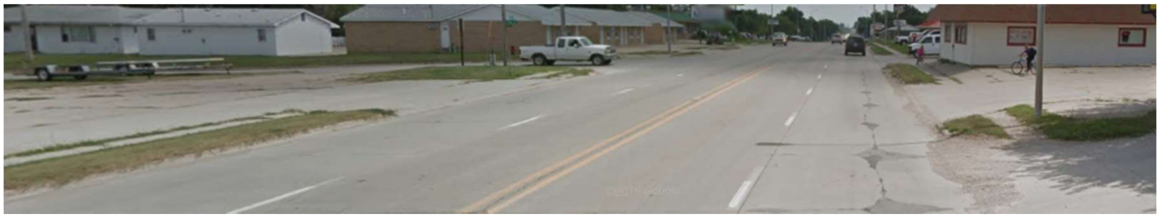

Figure 4.4 Typical street cross-section – Aurora, Nebraska [26]

A specific feature of the Highway 34 in Aurora that makes it a good candidate for a road diet is a crosswalk that connects a community swimming pool and a convenience store, with high pedestrian volume passing between them during the summer months. A desire for an improvement in the safety along this highway, specifically at the crosswalk mentioned, as well as the lower traffic volumes currently present at this location made it a good option for a case study site location. Passing through Aurora, there is one signalized intersection on Highway 34 at the 16th Street crossing.

4.3 Case Study Site Location – David City, Nebraska

The third location selected for the case study is in David City, Nebraska. A segment along Highway 15 from Lakeside Road to O Street was selected, approximately one and a half miles long, and encompassing 21 intersections, as shown in **Figure 4.5**. David City is a town in east-central Nebraska with a population of approximately 2,815 people. The population in 1970 was 2,380 people, indicating slight growth over the past 50 years.

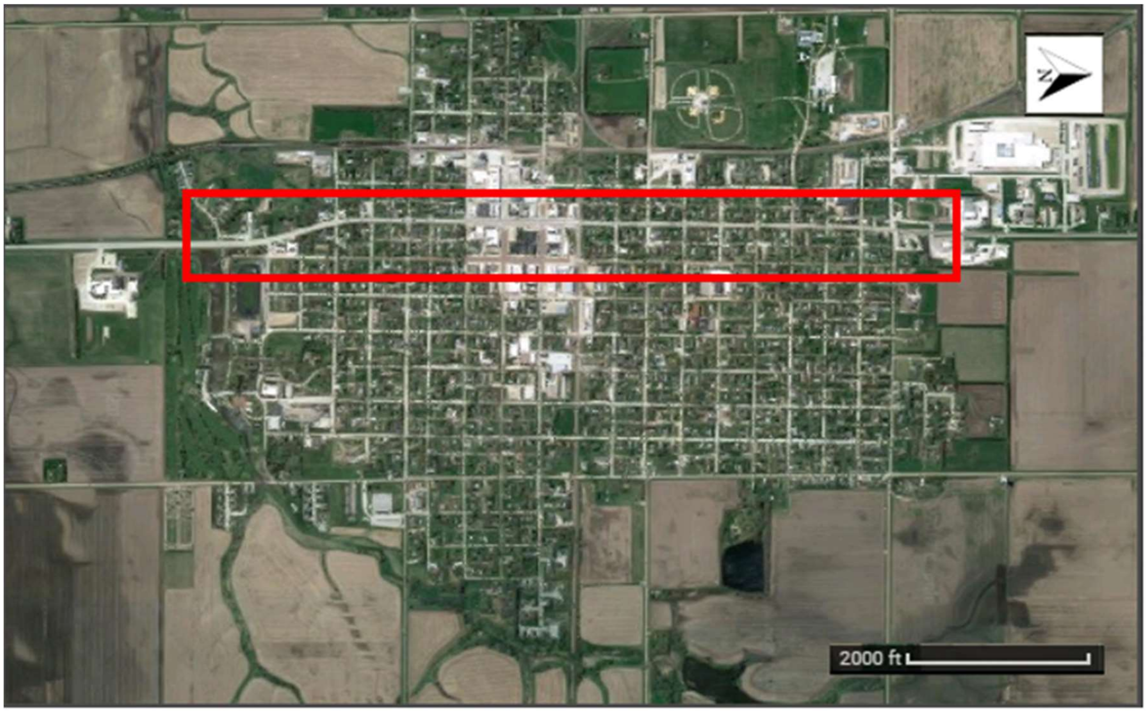

Figure 4.5 Site location at David City, Nebraska [26]

Although the population has increased, the amount by which it has done so is not sufficient to necessitate the expansion to four lanes on Highway 15. With an AADT of 4,510 on record with NDOT, David City's traffic volumes are in the lower end of the range of effective road diet volumes established in existing literature. [27]

Figure 4.6 shows the typical street cross-section for Highway 15 in David City, Nebraska. The cross-section shown looks north along Highway 15 between H Street and I Street. The overall width of the traveled way is 48 feet. This includes four 12-foot lanes for traffic, two in each direction, and no shoulders. Separated from the roadway by a 30-foot buffer, a 4-foot sidewalk runs along both the east and west sides of Highway 15. With the exception of the three blocks between C Street and F Street representing the downtown business district, the land-use surrounding the highway is residential, and approximately two-thirds of the lots have driveways connecting directly to Highway 15.

A road diet implemented in David City would result in a typical cross-section consisting of a 12-foot, two-way left-turn lane, two 12-foot, through lanes, and a 6-foot shoulder on each side.

Figure 4.6 Typical street cross-section – David City, Nebraska [26]

A specific feature of the roadway that makes it a good candidate for a road diet are the crosswalks traversing the highway within the main business district of the town. Additionally, David City began a resurfacing project in June 2017, stated to culminate in restriping the street as a three-lane roadway as opposed to its current four-lane state. This allows true before-and-after analysis to validate the results of the study simulations. Passing through David City, there is one signalized intersection on Highway 15 at the D Street crossing.

4.4 Case Study Site Location – Ord, Nebraska

The final location selected for the case study is in Ord, Nebraska. A segment of road along Highway 11/70 from 17th Street to 27th Street was selected, approximately three-quarters of a mile long, and encompassing 10 intersections, as shown in **Figure 4.7**. Ord is a town in central Nebraska with a population of approximately 2,078 people. The population in 1970 was 2,658 people, indicating slight negative growth over the past 50 years. Of the four case-study sites, Ord is the only location that has not experienced at least marginal population growth over the past 50 years.

Figure 4.7 Site location at Ord, Nebraska [26]

With an AADT of 3,800 along Highway 11/70, the corridor in Ord has the lowest traffic volume of the four case-study sites, and like Aurora and David City, Ord's traffic volumes are in the lower end of the range of effective road diet volumes established in existing literature. [27]

Figure 4.8 shows the typical street cross-section for Highway 11/70 in Ord, Nebraska. The cross-section shown looks east along Highway 11/70 between 18th Street and 19th Street. The overall width of the traveled way is 48 feet. This includes four 12-foot lanes for traffic, two in each direction, and no shoulders. Separated from the roadway by a 10-foot buffer, a 4-foot sidewalk runs along both the north and south sides of Highway 11/70. The land-use surrounding the highway is overwhelmingly residential, and approximately two-thirds of the lots have driveways connecting directly to Highway 11/70. One and a half blocks north of the highway is Ord High School. Crosswalks have been placed on the highway at 18th and 19th Streets in service of the pedestrian volume generated by the high school. A road diet implemented in Ord would result in a typical cross-section consisting of a 12-foot, two-way left-turn lane, two 12-foot, through lanes, and a 6-foot shoulder on each side.

Figure 4.8 Typical street cross-section – Ord, Nebraska [26]

CHAPTER 5 DATA COLLECTION AND REDUCTION

For each case study site location, traffic data and roadway geometry data was collected for inputs in simulation and analysis. Video data was obtained during the peak hour of the different car movements at each intersection. These video recordings were uploaded to MioVision for data processing to acquire traffic counts and turn movements for each intersection and pedestrian counts where relevant. Roadway geometry data was collected using Google Earth Pro, with measurments taken for the various roadway geometry aspects of interest.

5.1 Alliance Data Collection

In Alliance, technical issues with the camera units limited data collection to 5 of 17 intersections; video was recorded and recovered at Flack, Mississippi, Niobrara, Big Horn, and Black Hills Avenues. The recorded intersections were spread evenly across the corridor ensuring that the data collected provides a complete picture of the traffic conditions in Alliance. The intersection coverage is shown in **Figure 5.1** through **Figure 5.5**. The view of the camera recording at the Flack Avenue intersection is shown in **Figure 5.1**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the Mississippi Avenue intersection is shown in **Figure 5.2**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the Niobrara Avenue intersection is shown in **Figure 5.3**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the Big Horn Avenue intersection is shown in **Figure 5.4**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the Black Hills Avenue intersection is shown in **Figure 5.5**; this camera was positioned on the north-west quadrant of the intersection.

Figure 5.1 Camera view at Flack Avenue and Highway 2

Figure 5.2 Camera view at Mississippi Avenue and Highway 2

Figure 5.3 Camera view at Niobrara Avenue and Highway 2

Figure 5.4 Camera view at Big Horn Avenue and Highway 2

Figure 5.5 Camera view at Black Hills Avenue and Highway 2

5.2 Aurora Data Collection

In Aurora, 10 of 12 intersections were recorded; the studied intersections include 1st, 3rd, 5th, 7th, 9th, 10th, 12th, 13th, 14th, and 16th Streets. The cameras were set up on two consecutive days on alternating intersections to record video on 5 streets each day. The intersection coverage is shown in **Figure 5.6** through **Figure 5.15**. The view of the camera recording at the 1st Street intersection is shown in **Figure 5.6**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the 3rd Street intersection is shown in **Figure 5.7**; this camera was positioned on the south-west quadrant of the intersection. The view of the camera recording at the 5th Street intersection is shown in **Figure 5.8**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 7th Street intersection is shown in **Figure 5.9**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 9th Street intersection is shown in **Figure 5.10**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 10th Street intersection is shown in **Figure 5.11**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the 12th Street intersection is shown in **Figure 5.12**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 13th Street intersection is shown in **Figure 5.13**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 14th Street intersection is shown in **Figure 5.14**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the 16th Street intersection is shown in **Figure 5.15**; this camera was positioned on the north-east quadrant of the intersection.

Figure 5.6 Camera view at 1st Street and Highway 34

Figure 5.7 Camera view at 3rd Street and Highway 34

Figure 5.8 Camera view at 5th Street and Highway 34

Figure 5.9 Camera view at 7th Street and Highway 34

Figure 5.10 Camera view at 9th Street and Highway 34

Figure 5.11 Camera view at 10th Street and Highway 34

Figure 5.12 Camera view at 12th Street and Highway 34

Figure 5.13 Camera view at 13th Street and Highway 34

Figure 5.14 Camera view at 14th Street and Highway 34

Figure 5.15 Camera view at 16th Street and Highway 34

5.3 David City Data Collection

In David City, 10 of 20 intersections were recorded; the studied intersections include Lakeside Drive, Iowa, A, C, E, G, I, K, M, and O Streets. This placement method allowed for every other intersection along the corridor of interest to be recorded. The intersection coverage is shown in **Figure 5.16** through **Figure 5.25**. The view of the camera recording at the Lakeside drive intersection is shown in **Figure 5.16**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the Iowa Street intersection is shown in **Figure 5.17**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the A Street intersection is shown in **Figure 5.18**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the C Street intersection is shown in **Figure 5.19**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the E Street intersection is shown in **Figure 5.20**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the G Street intersection is shown in **Figure 5.21**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the I Street intersection is shown in **Figure 5.22**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the K Street intersection is shown in **Figure 5.23**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the M Street intersection is shown in **Figure 5.24**; this camera was positioned on the north-west quadrant of the intersection. The view of the camera recording at the O Street intersection is shown in **Figure 5.25**; this camera was positioned on the north-east quadrant of the intersection.

Figure 5.16 Camera view at Lakeside Drive and Highway 15

Figure 5.17 Camera view at Iowa Street and Highway 15

Figure 5.18 Camera view at A Street and Highway 15

Figure 5.19 Camera view at C Street and Highway 15

Figure 5.20 Camera view at E Street and Highway 15

Figure 5.21 Camera view at G Street and Highway 15

Figure 5.22 Camera view at I Street and Highway 15

Figure 5.23 Camera view at K Street and Highway 15

Figure 5.24 Camera view at M Street and Highway 15

Figure 5.25 Camera view at O Street and Highway 15

5.4 Ord Data Collection

Similar to Alliance, technical issues limited data collection in Ord; of the 10 intersections along the corridor of interest, video was recorded and recovered at 17th, 20th, 22nd, 23rd, and 24th Streets. The resulting collection points were spread across the corridor such that the data collected is still comprehensive for the corridor. The intersection coverage is shown in **Figure 5.26** through **Figure 5.30**. The view of the camera recording at the 17th Street intersection is shown in **Figure 5.26**; this camera was positioned on the south-west quadrant of the intersection. The view of the camera recording at the 20th Street intersection is shown in **Figure 5.27**; this camera was positioned on the south-east quadrant of the intersection. The view of the camera recording at the 22nd Street intersection is shown in **Figure 5.28**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the 23rd Street intersection is shown in **Figure 5.29**; this camera was positioned on the north-east quadrant of the intersection. The view of the camera recording at the 24th Street intersection is shown in **Figure 5.30**; this camera was positioned on the north-east quadrant of the intersection.

Figure 5.26 Camera view at 17th Street and Highway 11/70

Figure 5.27 Camera view at 20th Street and Highway 11/70

Figure 5.28 Camera view at 22nd Street and Highway 11/70

Figure 5.29 Camera view at 24th Street and Highway 11/70

Figure 5.30 Camera view at 24th Street and Highway 11/70

5.5 Data Reduction

Because the number of cameras and data collection periods was insufficient to collect video data at all intersections, the turning movement counts at several intersections were inferred. The turning movements for non-recorded intersections were determined using the movements for the recorded intersections immediately adjacent to the intersection of interest. Along the direction of the highway, traffic volumes entering and exiting are calculated from the exiting volumes of the previous intersection and entering volume of the subsequent intersection. For example, to calculate northbound turning movements at the non-recorded intersection, the entering volume is equal to the sum of NBT, EBL, and WBR movements at the recorded intersection immediately to the south, and the exiting volume is equal to the sum of NBL, NBT, and NBR movements at the recorded intersection immediately to the north. These volume balancing equations are shown as **Equation 5-1** and **Equation 5-2**, below.

$$(\mathrm{NBT}+\mathrm{EBL}+\mathrm{WBR})_{\mathrm{south}} = \mathrm{Entering}_{\mathrm{non-recorded}} \qquad \mathbf{(5-1)}$$

$$(\mathrm{NBL} + \mathrm{NBT} + \mathrm{NBR})_{\mathrm{north}} = \mathrm{Exiting}_{\mathrm{non-recorded}} \qquad \mathbf{(5-2)}$$

The difference between the entering and exiting volumes is then used to determine whether more traffic turns into or out of the primary directional flow. If the entering volume is greater than the exiting volume, more traffic turns out of the flow than into the flow and vice versa. Non-thru turning movements are then calculated by distributing the difference between entering and exiting traffic between the remaining turning movements (NBL, NBR, EBL, and WBR continuing from the previous example). This distribution is performed using multiples of 1/9. If entering volumes are greater than exiting volumes, 12/9 of the difference is assigned to movements turning out of the flow and 3/9 of the difference is assigned to movements turning into the flow. Inversely, if entering volumes are less than exiting volumes, 3/9 of the difference is assigned to movements turning out of the flow and 12/9 of the difference is assigned to movements turning into the flow.

To account for any impact driveway traffic of houses or businesses might have on the traffic flow, counts were obtained from the video data for any driveways that had a significant number (more than 10) of vehicles entering or exiting during the peak hour. For the driveways that did not have a significant number of vehicles, the Institute of Transportation Engineers Trip Generation Manual was used to determine the rate at which vehicles could be assumed to be entering or exiting these driveways during the peak hour. [28]

CHAPTER 6 RESULTS

Because two unique traffic modeling software applications were used in analysis, the results of each application can be compared as a form of validation, to assess the likelihood of success for a road diet implementation for a given level of traffic. The results indicate the level of traffic at which the conventional facility will fail, the level of traffic at which a road diet implementation for a given facility will fail, and difference between the two, representing the impact of the road diet conversion. Because this study is concerned primarily with feasibility and factors deciding implementation, the average delay per vehicle experienced by the reduced-capacity (road diet) facility at varying volume input levels is the primary result of interest.

6.1 Operational Impacts of Road Diet Implementations

6.1.1 Results of Existing Conditions Analysis

The results of the existing condition analysis are shown here both in tabular and graphical form. The compilation of existing condition analysis results for all case study locations is shown in **Table 6.1**.

Table 6.1 Results of existing condition analysis for all case study locations

Site	Roadway Condition	Volume Condition	AADT	Average Delay Synchro	Average Delay VISSIM
Alliance	Existing	100%	10,040	2.75	2.01
Alliance	Existing	150%	15,060	3.79	2.45
Alliance	Existing	200%	20,080	10.31	3.06
Alliance	Existing	250%	25,100	52.27	3.92
Alliance	Existing	300%	30,120	197.78	4.92
Alliance	Existing	350%	35,140	N/A	8.90
Alliance	Existing	400%	40,160	N/A	26.87
Aurora	Existing	100%	5,850	2.79	2.19
Aurora	Existing	150%	8,775	11.48	2.73
Aurora	Existing	200%	11,700	45.00	3.57
Aurora	Existing	250%	14,625	418.93	11.01
Aurora	Existing	300%	17,550	N/A	41.84
Aurora	Existing	350%	20,475	N/A	59.95
Aurora	Existing	400%	23,400	N/A	80.95
David City	Existing	100%	4,510	2.18	1.32
David City	Existing	150%	6,765	3.49	1.58
David City	Existing	200%	9,020	7.47	2.24
David City	Existing	250%	11,275	26.81	3.49
David City	Existing	300%	13,530	74.99	4.73
David City	Existing	350%	15,785	247.19	5.81
David City	Existing	400%	18,040	N/A	6.73
Ord	Existing	100%	3800	0.95	0.67
Ord	Existing	150%	5700	1.29	0.95
Ord	Existing	200%	7600	2.07	1.11
Ord	Existing	250%	9500	6.13	1.42
Ord	Existing	300%	11400	23.55	1.63
Ord	Existing	350%	13300	97.47	2.12
Ord	Existing	400%	15200	N/A	2.79

This table shows the results from both VISSIM and Synchro (HCM methodology) analysis for each site and volume condition. In Synchro analysis, some of the volume conditions exceed the model capacity, which invalidates the results of the analysis condition. For these cases, "N/A" replaces the results in **Table 6.1**. From the table, the relationship between volume and delay is observed to be more sensitive in Synchro analysis than in VISSIM analysis. Graphical results of existing roadway geometry from the Synchro (HCM methodology) analysis is shown below in **Figure 6.1**, with results of the VISSIM (micro-simulation) analysis shown below in **Figure 6.2**.

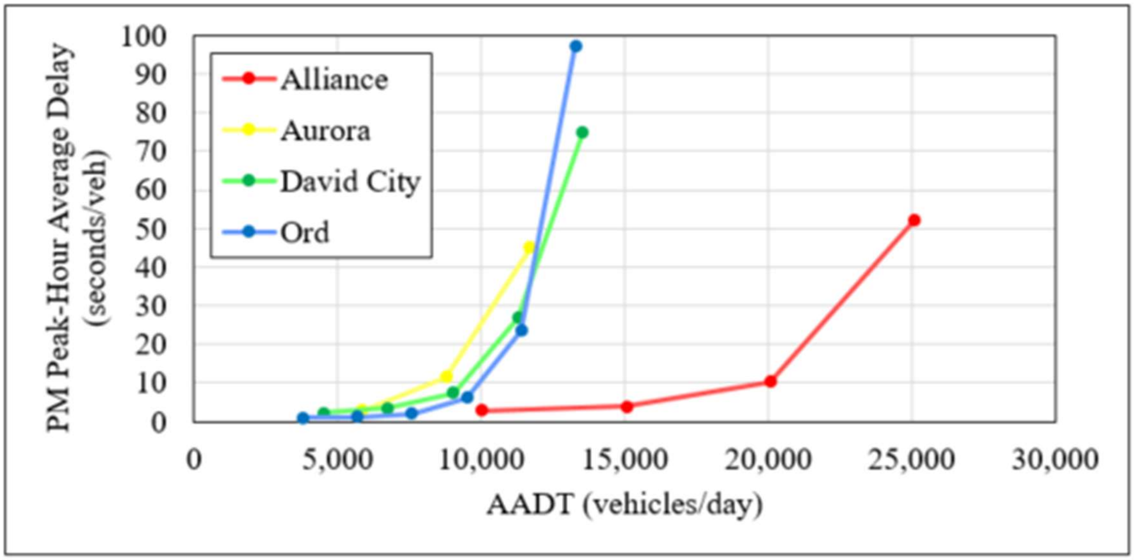

Figure 6.1 Results of existing condition analysis from Synchro

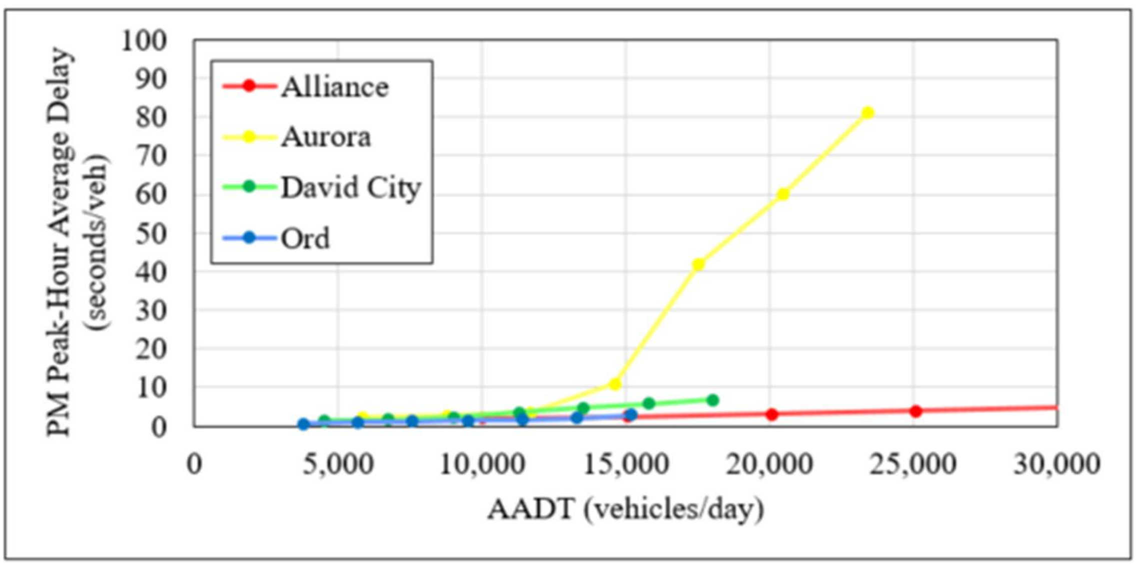

Figure 6.2 Results of existing condition analysis from VISSIM

Examining the results graphically reveals a difference between the Synchro (HCM methodology) and VISSIM (micro-simulation) methodologies. These results indicate that

Synchro is more conservative, in predicting that the capacity of the roadway will be reached faster than the VISSIM results indicate. Seeking to maintain the average delay per vehicle at less than 35 seconds, consistent with a level of service (LOS) C or better for a given signalized intersection, the results are analyzed to determine threshold AADT levels for which the existing geometry is anticipated to sufficiently accommodate traffic.

The Synchro results, being the more conservative of the two, indicate that a four-lane section is sufficient for all locations up to an AADT of around 10,000 vehicles/day. The Synchro results for Alliance indicate a much higher threshold of more than 22,000 vehicles/day. The difference between the performance of the Alliance facility and the other facilities can likely be attributed to the additional turn lanes provided at the signalized intersections at this location, further identifying the signalized intersections as the limiting factor for the four-lane cross section roadways.

The VISSIM results predict acceptable operational performance at significantly higher volumes than did the Synchro results. In the case of the micro-simulation analysis, Aurora was predicted to experience LOS D or worse conditions beyond 16,000 AADT, with Alliance predicted to operation sufficiently up to and beyond 30,000 AADT.

The findings for the existing condition will be governed by the more-conservative Synchro results, as no chances should be taken with regard to a reduction in capacity of a roadway. The four lane cross-section is found to be universally sufficient up until around 10,000 AADT for a corridor, with further study necessary between the range of 10,000 and 25,000 AADT. The design of the signalized intersection is expected to be the limiting condition for roadways in the 10,000 to 25,000 AADT range, with the four lane cross section sufficient once traffic is able to pass beyond the signalized intersections.

6.1.2 Results of Road Diet Analysis

The results of the road diet condition analysis are shown here both in tabular and graphical form. The compilation of road diet condition analysis results for all case study locations is shown in **Table 6.2**.

Table 6.2 Results of road diet condition analysis for all case study locations

Site	Roadway Condition	Volume Condition	AADT	Average Delay Synchro	Average Delay VISSIM
Alliance	Road Diet	100%	10,040	2.70	2.62
Alliance	Road Diet	150%	15,060	4.12	3.35
Alliance	Road Diet	200%	20,080	16.33	5.46
Alliance	Road Diet	250%	25,100	71.76	21.70
Alliance	Road Diet	300%	30,120	268.40	46.98
Alliance	Road Diet	350%	35,140	594.41	68.51
Alliance	Road Diet	400%	40,160	N/A	72.40
Aurora	Road Diet	100%	5,850	2.92	3.36
Aurora	Road Diet	150%	8,775	10.60	4.29
Aurora	Road Diet	200%	11,700	44.42	8.14
Aurora	Road Diet	250%	14,625	174.09	25.54
Aurora	Road Diet	300%	17,550	919.54	65.70
Aurora	Road Diet	350%	20,475	N/A	85.12
Aurora	Road Diet	400%	23,400	N/A	99.14
David City	Road Diet	100%	4,510	2.31	1.74
David City	Road Diet	150%	6,765	3.37	2.29
David City	Road Diet	200%	9,020	9.20	3.09
David City	Road Diet	250%	11,275	36.86	5.04
David City	Road Diet	300%	13,530	96.66	6.83
David City	Road Diet	350%	15,785	290.60	8.71
David City	Road Diet	400%	18,040	346.85	10.10
Ord	Road Diet	100%	3800	1.00	1.36
Ord	Road Diet	150%	5700	1.42	1.70
Ord	Road Diet	200%	7600	2.41	2.09
Ord	Road Diet	250%	9500	8.22	2.56
Ord	Road Diet	300%	11400	34.43	3.07
Ord	Road Diet	350%	13300	118.73	5.03
Ord	Road Diet	400%	15200	123.80	10.86

This table shows the results from both VISSIM and Synchro (HCM methodology) analysis for each site and volume condition. In Synchro analysis, some of the volume conditions exceed the model capacity, which invalidates the results of the analysis condition. For these cases, “N/A” replaces the results in **Table 6.2**. Similar to the results of the existing roadway geometry, the relationship between volume and delay is observed to be more sensitive in Synchro analysis than in VISSIM analysis. Graphical results of reduced-capacity road diet geometry from the Synchro (HCM methodology) analysis is shown below in **Figure 6.3**, with results of the VISSIM (micro-simulation) analysis shown below in **Figure 6.4**.

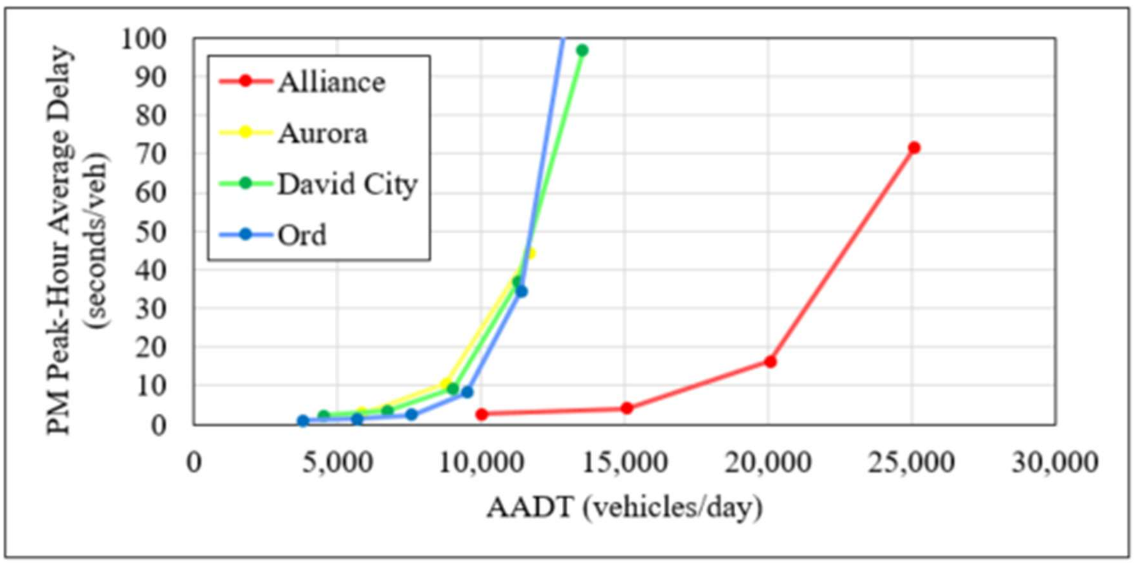

Figure 6.3 Results of road diet analysis from Synchro

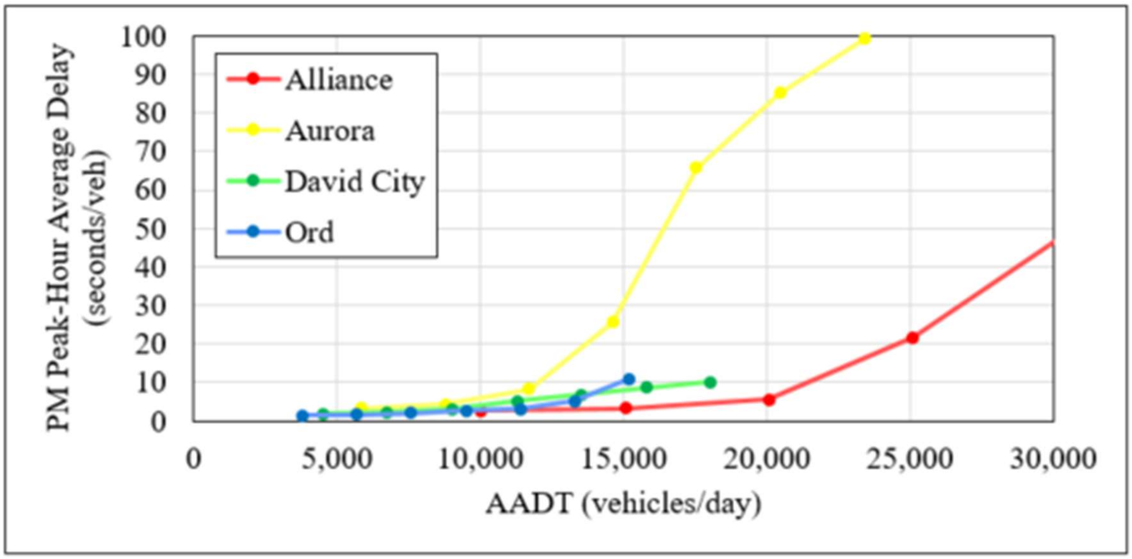

Figure 6.4 Results of road diet analysis from VISSIM

As with the existing roadway geometry, examining the results of the road diet analysis graphically reveals a difference between the Synchro (HCM methodology) and VISSIM (micro-simulation) methodologies. These results indicate that Synchro is more conservative, in predicting that the capacity of the road diet geometry capacity will be reached faster than the VISSIM results indicate. Seeking to maintain the average delay per vehicle at less than 35 seconds, consistent with a level of service (LOS) C or better for a given signalized intersection, the results are analyzed to determine threshold AADT levels for which the road diet geometry is anticipated to sufficiently accommodate traffic.

The Synchro results, being the more conservative of the two, indicate that a two-lane section is sufficient for all locations up to an AADT of around 10,000 vehicles/day, no change from the four-lane cross section. The Synchro results for Alliance indicate a much higher threshold of more than 22,000 vehicles/day, again, no change from the four-lane cross section. This highlights the significance of the auxiliary lanes at the signalized intersections, that reducing the capacity of the unimpeded traffic lanes by half has marginal impact on the travel times experienced by drivers, while additional lanes at signalized intersections can greatly reduce the average delay per vehicle for the entire corridor.

The VISSIM results predict acceptable operational performance at significantly higher volumes than did the Synchro results. In the case of the micro-simulation analysis, Aurora was predicted to experience LOS D or worse conditions beyond 15,000 AADT, with Alliance predicted to operation sufficiently up to around 27,000 AADT, both slightly decreased from the original four-lane cross section.

6.1.3 Results of Road Diet Impact Analysis

Further analysis of the results will be limited to only the results from Synchro (HCM methodology), as this has been shown to be the more conservative method of the two. Comparing the performance of existing condition models against road diet condition allows the impact of road diet to be assessed, with results shown in **Table 6.3**, below.

Table 6.3 Impacts of road diet improvement on delay (Synchro results)

Site	Roadway Condition	Volume Condition	AADT	Existing Condition Average Delay	Road Diet Average Delay	Increase in Average Delay
Alliance	Road Diet	100%	10,040	2.8	2.7	0.0
Alliance	Road Diet	150%	15,060	3.8	4.1	0.3
Alliance	Road Diet	200%	20,080	10.3	16.3	6.0
Alliance	Road Diet	250%	25,100	52.3	71.8	19.5
Alliance	Road Diet	300%	30,120	197.8	268.4	70.6
Alliance	Road Diet	350%	35,140	N/A	594.4	N/A
Alliance	Road Diet	400%	40,160	N/A	N/A	N/A
Aurora	Road Diet	100%	5,850	2.8	2.9	0.1
Aurora	Road Diet	150%	8,775	11.5	10.6	-0.9
Aurora	Road Diet	200%	11,700	45.0	44.4	-0.6
Aurora	Road Diet	250%	14,625	N/A	174.1	N/A
Aurora	Road Diet	300%	17,550	N/A	919.5	N/A
Aurora	Road Diet	350%	20,475	N/A	N/A	N/A
Aurora	Road Diet	400%	23,400	N/A	N/A	N/A
David City	Road Diet	100%	4,510	2.2	2.3	0.1
David City	Road Diet	150%	6,765	3.5	3.4	-0.1
David City	Road Diet	200%	9,020	7.5	9.2	1.7
David City	Road Diet	250%	11,275	26.8	36.9	10.1
David City	Road Diet	300%	13,530	75.0	96.7	21.7
David City	Road Diet	350%	15,785	247.2	290.6	43.4
David City	Road Diet	400%	18,040	N/A	346.9	N/A
Ord	Road Diet	100%	3800	1.0	1.0	0.0
Ord	Road Diet	150%	5700	1.3	1.4	0.1
Ord	Road Diet	200%	7600	2.1	2.4	0.3
Ord	Road Diet	250%	9500	6.1	8.2	2.1
Ord	Road Diet	300%	11400	23.6	34.4	10.9
Ord	Road Diet	350%	13300	97.5	118.7	21.3
Ord	Road Diet	400%	15200	N/A	123.8	N/A

Values within plus or minus one second when examining the column displaying the Increase in Average Delay indicate values that have negligible change from existing roadway cross section to road diet geometry, and are within the margin of error for the analysis methodology.

The primary concern with implementing a road diet on a facility is that the existing travel times will increase significantly due to congestion experienced by the traffic currently utilizing two lanes being forced to utilize one lane. The results in **Figure 6.5**, below, indicate that the change in average delay is strongly influenced by the degree to which the existing facility is approaching capacity.

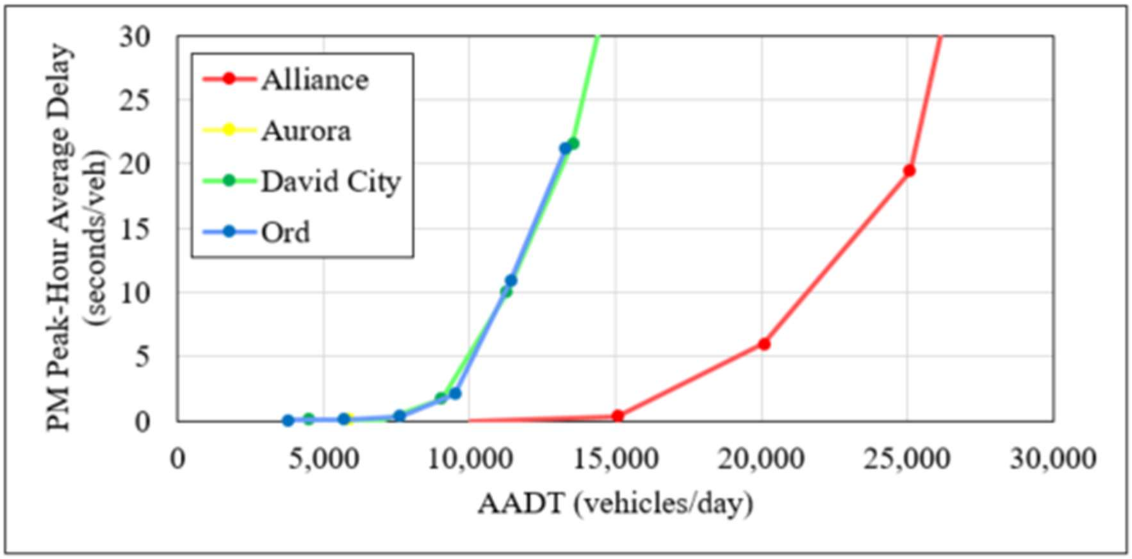

Figure 6.5 Increase in average delay at varying levels of AADT

Implementing a road diet conversion for facilities similar to Aurora, David City, and Ord will result in less than a 5-second increase in average travel time at values of AADT less than 10,000. At facilities similar to Alliance, where additional capacity is built into the signalized intersections along the corridor, implementing a road diet at volumes less than 20,000 is expected to result in a similar 5-second increase in average travel time of all vehicles in the network. Examining the relationship between existing congestion and the implementation of a road diet, **Figure 6.6**, below, plots the increase in delay caused by implementing a road diet against the existing condition average delay for the facility.

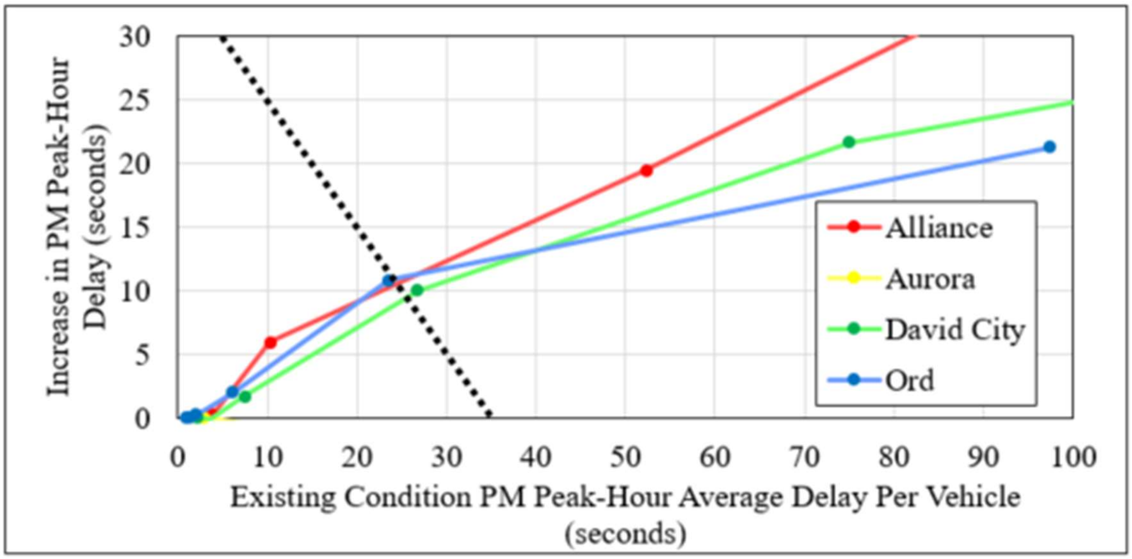

Figure 6.6 Road diet impact on existing average delay

The results displayed above in **Figure 6.6** show that the existing average delay per vehicle on a roadway network is a much better indicator of expected road diet impact than is AADT. The dashed black line defines acceptable traffic operational conditions, with delay to the left and below the dashed line having an average delay of less than 35 seconds per vehicle, indicating that an induced delay of up to 10 seconds due to a road diet implementation should be acceptable. Examining the general trend that is indicated by the results at all four locations, it is seen that a road diet increases existing delay by approximately 50% for existing conditions delay values of 30 seconds/vehicle or less, and at rates less than that for higher levels of existing delay.

Referring back to the literature on road diet implementations, the results for the Nebraska case-study locations are consistent with the national findings, including both simulation and field observation results. Maintaining an average delay per vehicle in the network of 35 seconds or less is predicted to be consistently possible at AADT values of less than 12,000, with much higher values up to 20,000 AADT possible depending on the capacity of the signalized intersections along the corridor. The average delay per vehicle of the existing facility was found to be a more accurate predictor of road diet functionality than was AADT, with a value of 25 seconds of delay per vehicle for the existing condition found to be the threshold at which the road diet facility results in a delay of 35 seconds per vehicle.

6.1.4 Impact of Percentage of Turning Vehicles on Road Diet Operations

In between 12,000 and 20,000 vehicles per day, the values for delay are disperse which means a road diet would be feasible in some cases and infeasible in others. One hypothesis is that the percentage of turning movements in the corridor is having an impact on the road diet performance, since the removal of the second lane requires all through vehicles to wait for slowing right-turning vehicles. **Table 6.4**, below reports the delay and AADT values, as well as the corresponding right-turn percentage.

Table 6.4 Right turn percentages corresponding to delay and AADT (Synchro)

Site	Roadway Condition	Volume Condition	AADT	% Right Turning	Existing Condition Average Delay	Road Diet Average Delay	Increase in Average Delay
Ord	Road Diet	100%	3,800	2.9	1.0	1.0	0.0
David City	Road Diet	100%	4,510	4.7	2.2	2.3	0.1
Ord	Road Diet	150%	5,700	2.9	1.3	1.4	0.1
Aurora	Road Diet	100%	5,850	6.9	2.8	2.9	0.1
David City	Road Diet	150%	6,765	4.7	3.5	3.4	-0.1
Ord	Road Diet	200%	7,600	2.9	2.1	2.4	0.3
Aurora	Road Diet	150%	8,775	6.9	11.5	10.6	-0.9
David City	Road Diet	200%	9,020	4.7	7.5	9.2	1.7
Ord	Road Diet	250%	9,500	2.9	6.1	8.2	2.1
Alliance	Road Diet	100%	10,040	4.1	2.8	2.7	0.0
David City	Road Diet	250%	11,275	4.7	26.8	36.9	10.1
Ord	Road Diet	300%	11,400	2.9	23.6	34.4	10.9
Aurora	Road Diet	200%	11,700	6.9	45.0	44.4	-0.6
Ord	Road Diet	350%	13,300	2.9	97.5	118.7	21.3
David City	Road Diet	300%	13,530	4.7	75.0	96.7	21.7
Aurora	Road Diet	250%	14,625	6.9	N/A	174.1	N/A
Alliance	Road Diet	150%	15,060	4.1	3.8	4.1	0.3
Ord	Road Diet	400%	15,200	2.9	N/A	123.8	N/A
David City	Road Diet	350%	15,785	4.7	247.2	290.6	43.4
Aurora	Road Diet	300%	17,550	6.9	N/A	919.5	N/A
David City	Road Diet	400%	18,040	4.7	N/A	346.9	N/A
Alliance	Road Diet	200%	20,080	4.1	10.3	16.3	6.0
Aurora	Road Diet	350%	20,475	6.9	N/A	N/A	N/A
Aurora	Road Diet	400%	23,400	6.9	N/A	N/A	N/A
Alliance	Road Diet	250%	25,100	4.1	52.3	71.8	19.5
Alliance	Road Diet	300%	30,120	4.1	197.8	268.4	70.6
Alliance	Road Diet	350%	35,140	4.1	N/A	594.4	N/A
Alliance	Road Diet	400%	40,160	4.1	N/A	N/A	N/A

As previously established, an average delay of less than 35 seconds per vehicle is desirable. All road diet implementations at sites with less than 10,000 AADT were found to be successful, with negligible impacts to average delay per vehicle due to the reduction in capacity. Two sites beyond this threshold were found to have acceptable levels of delay, being Ord at 11,400 AADT, and Alliance at 15,060 AADT. Their right-turn percentages are 2.9 and 4.1 respectively, indicating that a lower percentage of turning vehicles may fair better than a higher percentage of turning vehicles. However, due to the limited number of sites investigated for this study, there is not sufficient evidence to support the hypothesis that turn percentage has a significant impact on the performance of a reduced-capacity roadway, relative to the observed AADT for the facility.

6.2 Assessment of User Cost Impacts from Road Diets

Referring to **Table 6.4**, there are 8 cases with an AADT that falls within the range 12,000 to 20,000 vehicles per day. To understand further the feasibility limits within the AADT range 12,000 to 20,000 vehicles per day, the results of the economic analysis are examined. **Table 6.5**, shown below, includes the values of individual costs (travel time, operating, and accident costs) as well as total user cost per vehicle.

Table 6.5 Breakdown of user costs for road diets at case study locations

Site	Volume Condition	AADT (vpd)	ΔH_c (cents/veh)	ΔOC_c (cents/veh)	ΔAC_c (cents/v-m)	ΔB_c (cents/veh)
Ord	100%	3,800	0.0135	0.0050	-5.7163	-3.4112
David City	100%	4,510	0.0382	0.0141	-3.3577	-5.6556
Ord	150%	5,700	0.0382	0.0141	-5.7163	-3.3774
Aurora	100%	5,850	0.0410	0.0141	-5.9374	-7.0697
David City	150%	6,765	-0.0353	-0.0131	-3.3577	-5.7564
Ord	200%	7,600	0.0999	0.0370	-5.7163	-3.2929
Aurora	150%	8,775	-0.2776	-0.0957	-5.9374	-7.4981
David City	200%	9,020	0.5089	0.1882	-3.3577	-5.0109
Ord	250%	9,500	0.6142	0.2273	-5.7163	-2.5882
Alliance	100%	10,040	-0.0156	-0.0054	-6.8992	-8.9900
David City	250%	11,275	2.9564	1.0931	-3.3577	-1.6585
Ord	300%	11,400	3.1974	1.1834	-5.7163	0.9511
Aurora	200%	11,700	-0.1829	-0.0631	-5.9374	-7.3709
Ord	350%	13,300	6.2479	2.3124	-5.7163	5.1305
David City	300%	13,530	6.3746	2.3570	-3.3577	3.0236
Alliance	150%	15,060	0.1028	0.0359	-6.8992	-8.8303
David City	350%	15,785	12.7699	4.7216	-3.3577	11.7834
Alliance	200%	20,080	1.8747	0.6548	-6.8992	-6.4395
Alliance	250%	25,100	6.0693	2.1199	-6.8992	-0.7798
Alliance	300%	30,120	21.9914	7.6811	-6.8992	20.7036

Table 6.6, shown below, sums the total user cost per vehicle to show total peak hour user cost and then expands the peak hour cost to daily, weekly, monthly, and yearly total user costs. Positive values reflect costs imposed on the user while negative values reflect benefits realized by the user.

Table 6.6 Expansion of total user costs from peak-hour to yearly

Site	Volume Condition	AADT (vpd)	Existing Average Delay (s/veh)	Road Diet Average Delay (s/veh)	ΔD_c (sec)	$\Delta B_{h,total}$ ($/hr)	ΔB_{daily} ($/day)	ΔB_{weekly} ($/week)	$\Delta B_{monthly}$ ($/month)	ΔB_{yearly} ($/year)
Ord	100%	3,800	1.0	1.0	0.05	-$29.47	-$54.61	-$273.06	-$1,186.43	-$12,106.74
David City	100%	4,510	2.2	2.3	0.13	-$106.72	-$106.72	-$533.61	-$2,318.53	-$23,126.69
Ord	150%	5,700	1.3	1.4	0.13	-$43.77	-$81.10	-$405.52	-$1,762.00	-$17,980.05
Aurora	100%	5,850	2.8	2.9	0.13	-$125.98	-$164.65	-$823.23	-$3,576.92	-$32,233.44
David City	150%	6,765	3.5	3.4	-0.12	-$162.93	-$162.93	-$814.67	-$3,539.74	-$35,307.91
Ord	200%	7,600	2.1	2.4	0.34	-$56.90	-$105.43	-$527.16	-$2,290.51	-$23,373.21
Aurora	150%	8,775	11.5	10.6	-0.88	-$200.43	-$261.93	-$1,309.67	-$5,690.54	-$51,280.26
David City	200%	9,020	7.5	9.2	1.73	-$189.11	-$189.11	-$945.56	-$4,108.47	-$40,980.86
Ord	250%	9,500	6.1	8.2	2.09	-$55.91	-$103.59	-$517.94	-$2,250.46	-$22,964.49
Alliance	100%	10,040	2.8	2.7	-0.05	-$174.59	-$396.23	-$1,981.17	-$8,608.19	-$87,841.08
David City	250%	11,275	26.8	36.9	10.05	-$78.24	-$78.24	-$391.20	-$1,699.76	-$16,954.68
Ord	300%	11,400	23.6	34.4	10.88	$24.65	$45.68	$228.39	$992.34	$10,126.18
Aurora	200%	11,700	45.0	44.4	-0.58	-$262.70	-$343.32	-$1,716.60	-$7,458.61	-$67,213.24
Ord	350%	13,300	97.5	118.7	21.26	$155.15	$287.48	$1,437.38	$6,245.43	$63,730.62
David City	300%	13,530	75.0	96.7	21.67	$171.17	$171.17	$855.83	$3,718.59	$37,091.92
Alliance	150%	15,060	3.8	4.1	0.33	-$257.23	-$583.80	-$2,918.98	-$12,682.96	-$129,421.49
David City	350%	15,785	247.2	290.6	43.41	$778.24	$778.24	$3,891.18	$16,907.18	$168,644.60
Alliance	200%	20,080	10.3	16.3	6.02	-$250.11	-$567.65	-$2,838.23	-$12,332.12	-$125,841.38
Alliance	250%	25,100	52.3	71.8	19.49	-$37.86	-$85.93	-$429.63	-$1,866.75	-$19,048.98
Alliance	300%	30,120	197.8	268.4	70.62	$1,206.19	$2,737.54	$13,687.69	$59,473.02	$606,884.02

Plotting the AADT values against the total annual user costs results in **Figure 6.7**. At the $0.00 value, there is a thick black horizontal line. This line represents the break-even point for total user costs. At the break-even point, the road diet improvement, user benefits realized by accident rate reduction are equal to the road diet improvement, user costs incurred by the change (increase) in delay. Thus, any road diet improvement with a positive net user cost (i.e. plots above the thick black horizontal line in **Figure 6.7**) is economically infeasible.

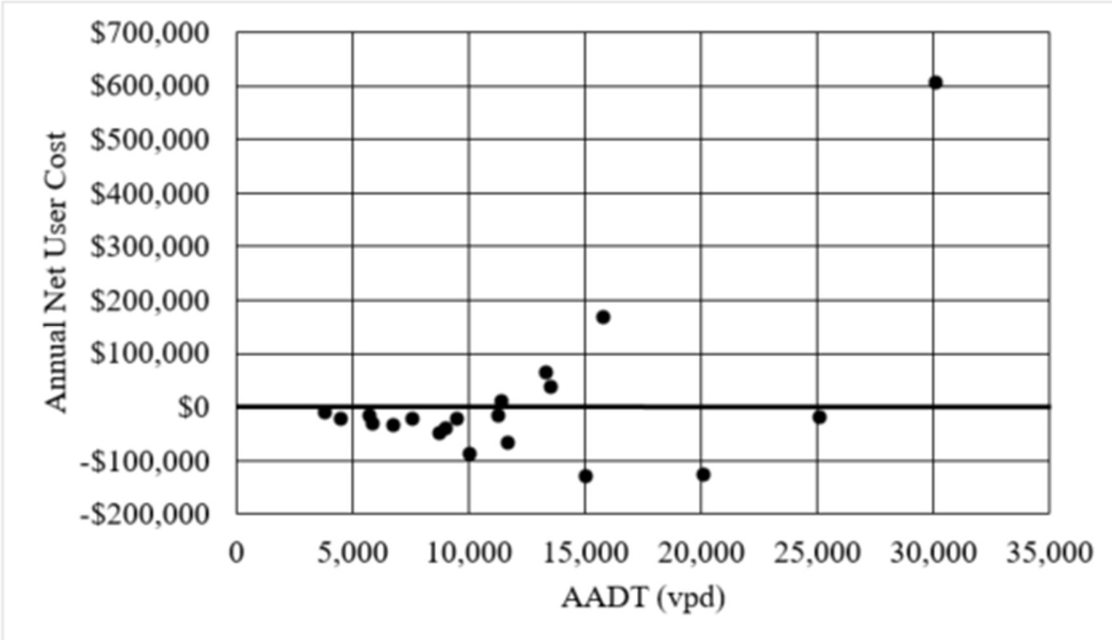

Figure 6.7 Total annual net user cost of road diets across levels of AADT

Looking at **Table 6.6** and **Figure 6.7**, there are five cases with positive annual user cost, four of which fall in the 12,000 to 20,000 vehicles per day range. This means that for the eight cases with AADT falling in the range 12,000 to 20,000 vehicles per day, half have conditions making a road diet improvement infeasible.

To determine the criteria for changing from infeasible to feasible, the relationships between average delay and total annual user cost as well as change in average delay and total annual user cost are examined. The relationship between average delay and annual user cost, shown in **Figure 6.8**, shows that while all of the cases in which annual user cost is positive have an average delay in excess of 30 seconds. However, there does not appear to be a direct correlation between average delay and total annual user cost.

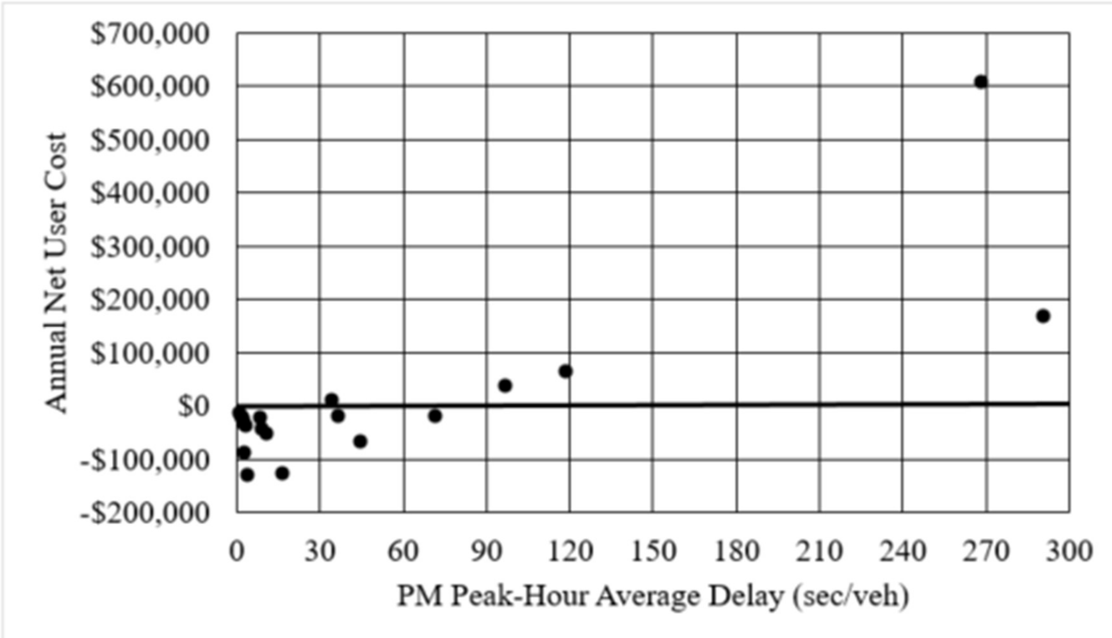

Figure 6.8 Effect of average delay on annual net user costs for road diets

The relationship between change in average delay and total annual user cost, shown in **Figure 6.9**, appears to have a more direct correlation between the two variables; as change in average delay increases, so does the total annual user cost.

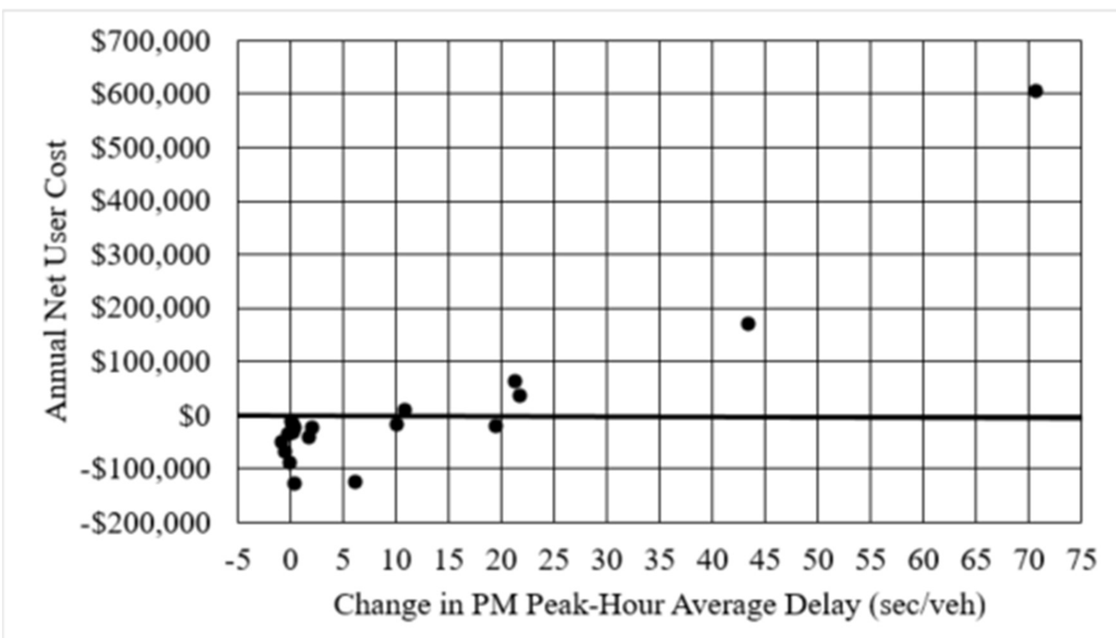

Figure 6.9 Effect of change in average delay on annual net user costs for road diets

From **Figure 6.9**, all road diet improvement cases with less than a 10 seconds per vehicle increase in delay have a negative annual user cost, which is to say that these cases incur a benefit for the users. Conversely, only one road diet improvement cases with an increase in delay in excess of 10 seconds per vehicle have a positive annual user cost. Based on the results of the economic analysis, another criterion for implementing road diet improvements can be determined: if the expected change in average delay from implementing the road diet improvement is less than 10 seconds per vehicle, the road diet improvement is economically feasible. Else, the road diet improvement is most-likely economically infeasible.

CHAPTER 7 ROAD DIET DECISION MATRIX

Assessing the results from **CHAPTER 6**, recommendations can be made regarding when road diets are appropriate in the context of traffic conditions observed in Nebraska.

7.1 Analysis of Road Diet Limitations

The feasibility of road diet conversions is assessed by examining factors influencing operations, logistics, and implementation. Operationally, the results from **CHAPTER 6** show implementing a road diet improvement increases delay in most cases thus, other factors must be considered in determining the feasibility of road diet improvements. These factors affect the extent to which delay is increased. In considering other factors, the relationship between AADT and delay was examined. Information from existing literature shows road diet improvements to be infeasible at AADT levels greater than 25,000 vehicles per day, determining one of the bounds for feasibility. [4]–[6], [8], [9] Sufficiently low and generally uniform delay results at AADT levels less than 14,000 vehicles per day establishes a second bound for feasibility. At this point, observations from the results in **CHAPTER 6** show that road diet implementation is feasible at AADT levels less than 12,000 vehicles per day and infeasible at AADT levels greater than 20,000 vehicles per day. However, if AADT is between 12,000 and 20,000 vehicles per day, the results from examining the relationship between AADT and delay are too disperse to suffice for determining whether or not a road diet conversion is a feasible improvement.

Continuing to look at the relationship between AADT and delay, points which correspond to delay values less than 20 seconds per vehicle are relatively uniform and appear to fit a trend-line established by points at lower volumes. Examining the right-turn percentages for points corresponding to delay values greater than 20 seconds per vehicle, it is determined that road diets are feasible roadway improvement projects between 12,000 and 20,000 vehicles per day if the average delay per vehicle for the existing roadway geometry is less than 20 seconds per vehicle.

To understand further the feasibility limits within the AADT range 12,000 to 20,000 vehicles per day, the results of the economic analysis are examined. Plotting the AADT values against the total annual user costs reveals five cases with positive annual user cost, rendering them economically infeasible, four of which fall in the 12,000 to 20,000 vehicles per day range. According to the feasibility limits just established, four cases in the 12,000 to 20,000 vehicles per day range have conditions making a road diet improvement infeasible. The case that switched from infeasible to feasible with the result of the economic analysis is Aurora, Nebraska, with an AADT of 14,625 vehicles per day, and a right turn percentage of 6.9 percent. Examining the relationship between AADT and delay, this case corresponds to the point falling just outside the bounds established in **CHAPTER 6**. It is determined that the cause of this case changing from infeasible to feasible, is that the increase in average delay is low enough that the accident reduction impacts of road diets offset the decreased operational performance. This is because the results reveal that all road diet improvement cases with less than an 10 seconds per vehicle increase in delay have a negative annual user cost, which is to say that these cases incur a benefit for the users. Conversely, most road diet improvement cases with an increase in delay in excess of 10 seconds per vehicle have a positive annual user cost. Thus, another criterion for implementing road diet improvements can be determined: if the expected change in average delay from implementing the road diet improvement is less than 10 seconds per vehicle, the road diet improvement is economically feasible. Else, the road diet improvement is likely economically infeasible.

Logistically, factors which influence the feasibility of road diet implementation include characteristics of the vehicle and non-vehicle users of the roadway. Existing literature shows road diets to be estimated to reduce vehicle-accident rates of all types by 33 percent. [4], [6], [7], [9]–[14] This accident reduction rate means that a road diet improvement can serve and an effective accident mitigation technique. In cases such as Aurora and Ord that have destinations which may generate significant pedestrian volumes, the impact of road diets on traffic calming is also a factor influencing feasibility. While road diets conversions will not impact the posted speed limit on a road, existing literature does show road diets to be effective at reducing the speed variability of traffic. [4]–[6], [10], [11], [16] The reduction in speed variability is due to road diets decreasing the percentage of vehicles travelling 5 or more miles per hour in excess of the posted speed limit. Reducing speed variability of vehicle traffic makes the roadway safer for pedestrians looking to cross the road.

The primary implementation factor governing the feasibility of road diet conversions is cost. Examining the cost of conversions, there is a critical component in Nebraska for which to account: in Nebraska, townships have to pay half the cost for repaving and surface maintenance of the roadway when it is four lanes, but they do not have to pay for one lane. Therefore, to limit the cost impact of road diet conversions, road diets should be implemented as restriping as part of necessary resurfacing work. This effectively eliminates any cost associated with the actual road diet implementation, as the restriping costs are already accounted for in resurfacing projects.

7.2 Decision Matrix for Consideration of Road Diet Implementation

A primary objective of this research is to develop the framework criteria to guide decision making on the feasibility of implementing road diet improvements. Towards this end, existing condition and road diet corridor traffic analysis was performed utilizing Synchro (HCM methodology) and VISSIM (micro-simulation). The results of the traffic analysis were then used as inputs for an economic analysis on the impacts of road diet conversions in Nebraska. Both the traffic and economic analyses yielded results that helped form criteria to guide decision making on the feasibility of implementing road diet improvements. These criteria, coupled with general criteria from existing literature, create a decision tree for implementing road diet conversions in Nebraska. This decision tree is shown in **Figure 7.1**.

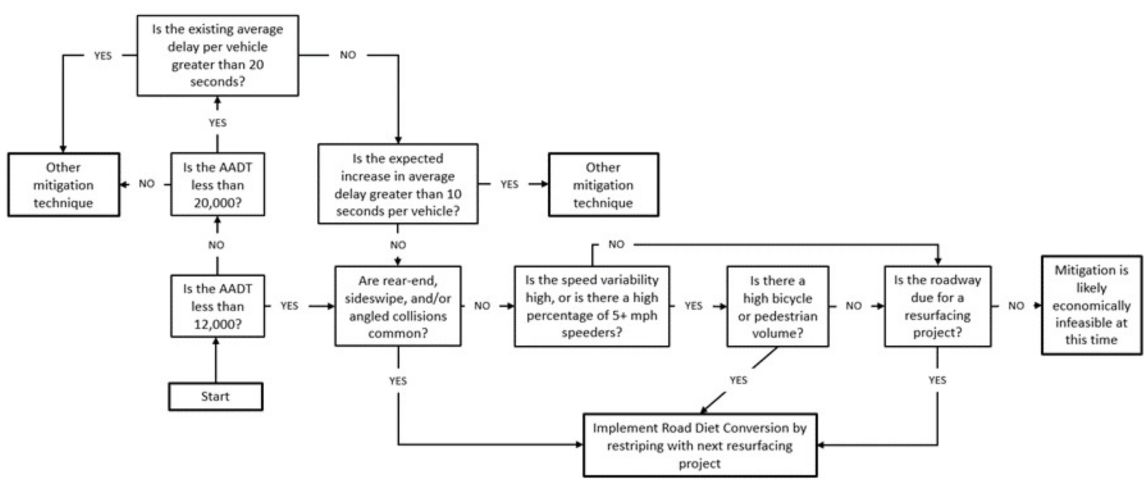

Figure 7.1 Road diet conversion decision tree

The general layout of the decision tree begins with broad criteria involving less intense analysis or collection processes. Thus, the starting criterion for the decision tree is the AADT for the roadway corridor in question. The feasibility ranges previously established are used as the initial pathways on the decision tree. If AADT is less than 12,000 vehicles per day, a road diet conversion is a feasible roadway improvement; if AADT is greater than 20,000 vehicles per day, a road diet conversion is not a feasible roadway improvement; and if AADT is between 12,000 and 20,000 vehicles per day, a road diet conversion may be a feasible improvement. After establishing feasibility from AADT, the next criterion on the decision tree narrows the scope of focus, and thus involves slightly more intense analysis than AADT determination. This second bound is provided by traffic analysis: road diets are feasible roadway improvement projects between 12,000 and 20,000 vehicles per day if the existing facility average delay is less than 20 seconds per vehicle. This criteria point helps bring the decision tree to either result path established by AADT under 12,000 or over 20,000.

Economic analysis provided a criterion to further define the feasibility of road diet improvements within the AADT range 12,000 and 20,000 vehicles per day: if the expected change (increase) in average delay is less than 10 seconds per vehicle, the road diet improvement is feasible. Else, the road diet improvement is infeasible. This is the final step on bringing the decision tree to the binary result paths of a road diet improvement being either feasible or infeasible.

From here, the feasibility of a road diet improvement is more dependent on unique characteristics of the roadway such as crash rates: existing literature shows road diet conversions to be successful in reducing rates of rear-end, sideswipe, and angled collisions. [4], [6], [7], [9]–[14] So if the collision rates on a roadway are a concern, a road diet implementation can be effective in mitigating this. If vehicle-accident rates are not a major concern on the roadway, the presence of pedestrians or bicyclists crossing the street due to social or education destination such as a swimming pool or high school can be a contributing factor to feasibility of a road diet. The significant presence of pedestrians or bicyclists causes the speed variability of vehicle traffic to become a concern. If there is a high speed variability, or a significant percentage of vehicles travelling 5 or more miles per hour over the speed limit, road diets can help calm this variability, without sacrificing operational performance, and make the roadway safer for non-vehicle users such as pedestrians and bicyclists.

The final step in determining the feasibility of road diet implementation is whether the roadway in question is in need of a resurfacing project. Because the costs of a road diet implementation will be absorbed into the cost of a resurfacing project, it is desirable to implement road diets as part of resurfacing rather than as a stand-alone project.

CHAPTER 8 LIMITATIONS

There were a few limitations to the research, which should be mentioned to represent accurately the work that was conducted.

The number of available MioVision Scout cameras for use was a limitation. The cameras performed well in the field, but the low number of units for use meant that certain locations along the study corridor were left unrecorded. While this was partially combatted through multiple data collection days and inferring, being able to use more cameras would allow for a better, or at least more accurate, set of input data for simulation.

When utilizing the software applications to model the road diet configuration of the highway, issues arose when modeling the two-way left-turn lane. VISSIM lacks a function that explicitly adds this into the model. To overcome this, left-turn bays were created for each intersection or driveway. This practice is not exactly accurate since vehicles are able to enter the two-way left-turn lane at any point, rather than one specific entrance point with a left-turn bay. However, the impact of this change is minor for this study since the volumes of vehicles turning left is relatively low.

The case study sites used for this analysis were too small to have reported traffic characteristics (crashes and vehicle miles travelled) with the Nebraska Department of Transportation. In lieu of site-specific information, safety data for the counties in which is case study sites reside were used. While the county data provides a moderately accurate representation of the traffic characteristics of the case study sites, access to site-specific information would allow for results that are more accurate.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

This research sought to address two main questions. The first question: because it is readily available to decision makers, how well does AADT serve as a predictor of the feasibility of a road diet conversion? The second question: outside of AADT, what are some other readily available factors which predict the feasibility of road diet conversions? The results of this research indicate that road diet conversions are feasible roadway improvement projects while AADT is less than 12,000, while AADT ranging from 20,000 and greater renders road diets infeasible as roadway improvement projects. For the AADT between 12,000 and 20,000 this research indicates that the existing facility average delay per vehicle is a primary contributing factor to feasibility. The economic analysis performed found that increasing average delay by more than 10 seconds per vehicle renders road diets economically infeasible. An important non-operational factor to consider is also cost. In Nebraska, townships have to pay half the cost for repaving and surface maintenance of the roadway when it is four lanes, but they do not have to pay for one lane. Therefore, to limit the cost impact of road diet conversions, road diets should be implemented as restriping as part of necessary resurfacing work. This effectively eliminates any cost associated with the actual road diet implementation, as the restriping costs are already accounted for in resurfacing projects.

The results of this study are limited by the data collection process, modeling software, and the nature of the study sites. As discussed previously, the data collection process is effective, but limited by the number of devices available. In future studies, combining the use of more cameras with additional study days will allow for more complete or comprehensive data collection. When utilizing the software application to model the road diet configuration of the highway, issues arise modeling the two-way left-turn lane. This function is not specifically supported in the software. To overcome this, left-turn bays can be created for each intersection or driveway. While this is not completely accurate with the behavior of a true two-way left-turn lane, the impact of this change is minor for this study since the volumes of vehicles turning left is relatively low. Additionally, the population centers and AADT volumes for the case study sites were too low for the Nebraska Department of Transportation to provide detailed traffic characteristics (crashes and vehicle miles travelled) data.

In future studies, sites with higher traffic volumes and population should be chosen to allow access to more complete traffic data. This will allow for a more accurate economic analysis and would result in more finely tuned criteria for the decision tree. Additionally, examining the impacts of midblock friction on road diet feasibility provides an avenue to further refine the research laid out in this document.

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