Final Report

Development of Crash Modification Factors for Speed Management of Traffic Signal Progression

Project Number

BDV25-977-73

Final Report

Prepared For

Florida Department of Transportation

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2022 CUTR CENTER FOR URBAN TRANSPORTATION RESEARCH CUTR.USF.EDU UNIVERSITY OF SOUTH FLORIDA

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Prepared for:

Florida Department of Transportation

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February 2022

Disclaimer

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the State of Florida Department of Transportation.

Metric Conversion

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	M
yd	yards	0.914	meters	M
mi	miles	1.61	kilometers	km
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	ounces	28.35	grams	G
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
ºF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	ºC

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This research project investigated the impacts of traffic signal progression strategy on the safety of pedestrians and bicyclists. The CUTR team used historical (crash, traffic, and roadway) data, cross-sectional study designs, and random parameter negative binomial models to estimate the effects of signal progression design on pedestrian and bicycle crash frequency and severity. Crash modification factors (CMFs) were developed to assess the safety effects of three countermeasures: (1) progression speed, (2) progression speed management, and (3) progression quality (smooth traffic flow). The study findings show the pronounced impacts of well-designed progression speed management with good progression quality on reducing the frequency and severity of pedestrian and bicycle crashes. Improving poor progression speed management to good-to-great progression speed management can reduce pedestrian and bicycle crash frequency by 55% and severe pedestrian and bicycle crash frequency by 75%. Improving poor progression quality to good progression quality from signal retiming can decrease pedestrian and bicycle crash frequency by 52% and severe pedestrian and bicycle crash frequency by 72%. With traffic signal retiming to increase progression speed and improve speed management and progression quality, both driver mobility and pedestrian and bicycle safety can be achieved simultaneously.</td></tr>
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Executive Summary

Elimination of pedestrian fatalities and serious injuries on Florida public roadways is the critical mission of the Florida Department of Transportation (FDOT). With anticipated population growth in Florida, efforts to improve pedestrian safety and mobility are increasingly important, and speeding is one of major causes of pedestrian injuries and fatalities. According to the Federal Highway Administration (FHWA), speeding accounts for 26% of all fatalities in the U.S. (FHWA, 2017). Although less than 10% of fatal crashes involving pedestrians and bicyclists are related to speeding, an increase in travel speed can increase the severity of pedestrian crashes (Neuner et al., 2016). Based on data from the Florida Department of Highway Safety and Motor Vehicles (FL DHSMV), Florida had 720 pedestrian fatalities and 9,356 pedestrian crashes in 2018. Similar to roadways in other states in the past, Florida arterials in urban and suburban areas favor high progression speeds to improve or enhance mobility, which could increase the risk of pedestrians being struck and killed by motor vehicles. In recent years, FDOT has invested significant funding and effort to make Florida roadways safer for all road users, not just drivers.

The escalation of speeding issues in Florida has influenced FDOT to identify more effective strategies to mitigate these problems. Speed control has been identified as an important treatment to prevent pedestrian fatalities and injuries on urban roadways, especially urban arterials. The 3 E's (Engineering, Enforcement, and Education) are common types of speed management countermeasures implemented by FDOT to ensure safety and reduce pedestrian crashes, injuries, and fatalities. Traffic signal progression that synchronizes traffic signal timing along a corridor to enable platoons of vehicles to pass through coordinated intersections without stopping at a red light is a cost-effective and commonly-used traffic control strategy for improving traffic mobility and safety.

Despite speed management having high potential and benefits to improve pedestrian and bicycle safety, limited studies and documents exist on its safety impacts, especially for pedestrians and bicyclists. This impedes FDOT and local transportation agencies from implementing traffic signal control strategies to improve pedestrian and bicycle safety while improving mobility for all modes on Florida arterials. The goal of this project was to evaluate the effectiveness of signal progression design via speed management and traffic flow patterns to improve pedestrian and bicycle safety. The team collected massive coordinated signal timing data and associated pedestrian and bicycle crash and roadway data and evaluated the relationships between (1) traffic signal progression with speed management and (2) frequency and the severity of pedestrian and bicycle crashes. Based on detailed quantitative analysis, researchers developed random parameter negative binomial models and crash modification factors (CMFs) to identify the benefits of signal progression speed management and signal progression quality on improving pedestrian and bicycle safety. Case studies are used to demonstrate the benefits.

Study findings show the pronounced impacts of well-designed progression speed management on reducing the frequency pedestrian and bicycle crashes and the frequency of severe injuries and fatalities. Improving poor progression speed management to good-to-great progression speed management can reduce pedestrian and bicycle crash frequency by 55% and severe pedestrian and bicycle crash frequency by 75%.

Similarly, the findings prove that good progression quality (smooth traffic flow) can enhance the safety of pedestrians and bicyclists on urban roadways. Improving poor progression quality to good progression quality from signal retiming can reduce pedestrian and bicycle crash frequency by 52% and severe pedestrian and bicycle crash frequency by 72%.

Research results indicate that an increase in progression speed (improvement of mobility) after signal retiming will slightly increase overall pedestrian and bicycle crash frequency. Findings show that good speed management and good progression quality can significantly reduce pedestrian and bicycle crash frequency and severity (improvement of safety). When a traffic signal retiming project can increase progression speed and improve speed management and progression quality, both driver mobility and pedestrian and bicycle safety can be achieved simultaneously. This is an important finding and conclusion based on intensive data analysis and modeling.

Lower progression speeds and lower posted speed limits are not necessarily the key to pedestrian and bicycle safety. If there are large speed differentials among vehicles on urban arterials, a slower average vehicle speed does not ensure safety for pedestrians and bicyclists. A higher progression speed is not always unsafe for pedestrians and bicyclists if appropriate designs are used with good speed management and good progression quality.

Data analysis results show that an increase in Annual Average Daily Traffic (AADT) is highly correlated with an increase in pedestrian and bicycle crash frequency. Therefore, for similar urban arterials, an arterial with a higher AADT likely has a larger number of pedestrian and bicycle crashes. Good progression speed management combined with progression quality becomes essential for arterials with moderate to high traffic volumes.

Research results based on cross-sectional data analysis show that an increase in posted speed limits is correlated to a decrease in pedestrian and bicycle crash frequency. The result is likely due to less pedestrian and bicycle exposure on roadways with higher posted speed limits; therefore, there are fewer pedestrian and bicycle crashes.

Findings from this research project also confirm that arterials with wider medians likely reduce pedestrian and bicycle crashes. An increase in access density can lead to a decrease in pedestrian and bicycle safety, indicating that access management on urban arterials is important.

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1 Introduction

The safety of pedestrians and bicyclists on Florida roadways is among the top priorities of the Florida Department of Transportation (FDOT), and speeding is a major cause of pedestrian injuries and fatalities. According to the Federal Highway Administration (FHWA), speeding accounts for 26% of all fatalities in the U.S. (FHWA, 2017), a trend that has been persistent during the last decade (Neuner et al., 2016). For example, in 2011, one third of traffic fatalities in the U.S. were related to speeding, and 90% of those fatalities occurred on non-highways.

According to the National Highway Traffic Safety Administration (NHTSA), in 2019, when roadway function class was known, 86% of speeding-related fatalities occurred on non- interstate roadways (NHTSA, 2021). Similarly, in 2018, about 9,378 of 36,560 fatalities were speeding crashes, a statistic that was 3.5% lower in 2018 compared to 2017 (FHWA, 2017). Speeding is an issue for all types of roadways; more than 35% of speeding fatalities occur on collector and local roadways rather than highways (FHWA, 2017).

FHWA encourages efforts to reduce speeding-related crashes for roadway departures and at intersections and speeding crashes involving pedestrians and bicycles (Neuner et al., 2016). As a result, many state departments of transportation (DOTs) evaluate the safety of their systems relative to these main areas. Roadway departure, intersection, and pedestrian and bicycle crashes represent 90% of traffic fatalities in the U.S. Additionally, about 40% of fatal crashes related to roadway departure and 20% of fatal crashes at intersections are caused by speeding (Neuner et al., 2016). Although less than 10% of fatal crashes involving pedestrians and bicyclists are related to speeding, their numbers are still high. An increase in travel speed can significantly increase the severity of pedestrian crashes (Neuner et al., 2016) (Figure 1).

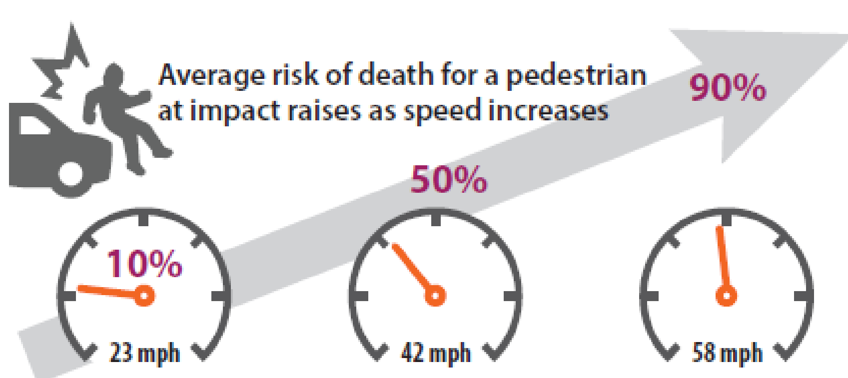


Figure 1. Pedestrian risk of death related to impact speed

Source: (Neuner et al., 2016)

A study by the American Automobile Association (AAA) elaborates more on speeding issues. Findings indicate that the average risk of severe injury for a pedestrian struck by a vehicle reaches 10% at an impact speed of 16 mph, 25% at 23 mph, 50% at 31 mph, 75% at 39 mph, and

90% at 46 mph, and the average risk of death reaches 10% at an impact speed of 23 mph, 25% at 32 mph, 50% at 42 mph, 75% at 50 mph, and 90% at 58 mph. Risks also vary significantly by age; for example, the average risk of severe injury or death for a pedestrian age 70 who is struck by a car traveling at 25 mph is similar to the risk for a pedestrian age 30 hit at 35 mph.

A closer look at Florida data reveals similar trends when compared to the U.S. Based on data from the Florida Department of Highway Safety and Motor Vehicles (FL DHSMV), Florida had 720 pedestrian fatalities and 9,356 pedestrian crashes in 2018. Preliminary data from the FL DHSMV showed that there were 716 pedestrian fatalities and 9,685 pedestrian crashes in 2019. Similar to roadways in other states in the past, Florida arterials in urban and suburban areas, mainly designed for vehicle mobility, favor high speeds to improve or enhance mobility, which could increase the risk of pedestrians being struck and killed by motor vehicles. In recent years, FDOT invested significant funding and efforts to make Florida roadways safe for all road users, not just for drivers.

The escalation of speeding issues in Florida is influencing FDOT to identify effective strategies to mitigate these problems. Speed control has been identified as an important treatment to prevent pedestrian fatalities and injuries on urban roadways, especially urban arterials. The 3 Es (Engineering, Enforcement, and Education) are common types of speed management countermeasures employed to ensure safety (Neuner et al., 2016). Traffic signal progression is an engineering speed management strategy that links traffic signals along a corridor to enable platoons of vehicles to pass through coordinated intersections without stopping at a red light. This effective traffic control strategy for reducing delays and the number of stops is widely used on Florida urban arterials. On the other hand, it can increase average speed by 6%, leading to higher pedestrian injuries and fatalities.

Elimination of pedestrian fatalities and serious injuries on Florida public roadways is the critical mission of FDOT. With anticipated population growth in Florida, efforts to improve pedestrian safety are increasingly important. FDOT has explored and implemented various countermeasures, including the 3 Es and emergency services, to reduce pedestrian crashes and prevent pedestrian injuries and fatalities. However, existing documents and studies do not address the safety impacts of traffic signal progression, especially for pedestrians. The absence of a full understanding of crashes caused by traffic signal progression impedes FDOT from implementing effective traffic signal control strategies to improve pedestrian safety while improving mobility for all modes on Florida arterials. Adequate design of traffic signal progression could effectively manage vehicle speeds and maintain good vehicle progression through a series of traffic signals.

Through this research project sponsored by FDOT. the Center for Urban Transportation Research (CUTR) at the University of South Florida (USF) assessed the effectiveness of using traffic signal progression for speed management to improve pedestrian and bicycle safety, and developed Crash Modification Factors (CMFs) to assess the benefits of using this traffic signal progression strategy. The goal of this project was to research, evaluate and demonstrate how to effectively use the traffic signal progression techniques not only improve mobility on urban arterials via the reduction of vehicle delays, stops and travel time, but also improve the safety of pedestrians and bicyclists via proper progression speed management and smooth progression.

1.1 Project Objectives

This research project had four main objectives and aims: 1) understand signal progression-related factors contributing to or alleviating pedestrian crashes and injuries, 2) investigate and quantify the impacts of traffic signal progression on pedestrian and bicycle crash frequency and severity, 3) develop CMFs for speed management via traffic signal progression, and 4) provide guidelines for implementing effective and adequate traffic signal progression strategies to manage vehicle speeds to reduce pedestrian and bicycle crash frequency and their severity. Specific research objectives included the following:

1. Understand traffic signal progression-related factors contributing to or alleviating pedestrian and bicycle crashes and injuries, such as progression speed, progression quality, traffic volumes, roadway characteristics, and others.
2. Investigate and quantify the impacts of traffic signal progression design on pedestrian and bicycle crash frequency and severity on Florida urban arterials based on detailed data analysis and modeling.
3. Develop CMFs to quantify the effects and assess the benefits of speed management via traffic signal progression to mitigate pedestrian and bicycle crash frequency and severity.
4. Develop guidelines and recommendations for transportation professionals to apply traffic signal control strategies as a tool for speed management on urban arterials for improving pedestrian and bicycle safety while maintaining vehicle mobility.

1.2 Organization of Report

The rest of this report is organized as follows: Chapter 2 summarizes a literature review on various topics related to safety and mobility, traffic signal progression, speed management, the CMF development process, and data. Chapter 3 elaborates on the experimental design, data collection plan, and analysis methodologies. Chapter 4 synthesizes the data collection process. Chapter 5 expands on the data analysis and development of CMFs. Chapter 6 describes case studies, guidelines, and recommendations. Chapter 7 provides conclusions.

2 Literature Review

This chapter summarizes the major findings of a comprehensive literature review on various topics to support this research, including: 1) practice of traffic signal progression for speed management, 2) existing safety and mobility evaluation, 3) existing safety performance functions (SPFs), CMFs, and crash predictive models, 4) methodologies for CMF and SPF development, and 5) potential data sources in Florida and other states.

2.1 Practice of Traffic Signal Progression for Speed Management

Although traffic signal progression is mainly used to optimize traffic flow and reduce delays, it is also employed to manage speed by providing a progressive green band to cars moving at a designated speed. For a better outcome, it is often important to associate the strategy with speed signs educating motorists about the signal being timed with a particular speed (Figure 2). These educational signs can advise drivers to drive at the coordinated speed to avoid multiple stops—for example, traffic synchronization for speed management is used in downtown Portland, Oregon, and in France. The strategy has proved to reduce average speed by 10–20% and the 85th percentile speed by 15–25% in France (Kimley-Horn and Associates, Inc, 2009). Interviews with experts on speed management reveal that it is crucial to maintain speeds in the range of 5–10 mph below posted speeds when synchronizing signals for speed management (Kimley-Horn and Associates, Inc, 2009).

Figure 2. Example of speed sign educating motorists about the signal being timed

Source: (Tampa Bay Times, tampabay.com)

Signals can be coordinated on arterials to improve long-term progression and uniform speed at intersections (PennDOT, 2016) (Table 1). Ensuring that the coordination is done on the correct type of roadway is noted as important for effective speed management (Neuner et al., 2016). A study done for the City of Pasadena listed signal coordination to a target speed (of at least the posted speed limit) among effective arterial speed management practices for streets with both fewer and greater than 20,000 vehicles per day (Kimley-Horn and Associates, Inc, 2009).

Table 1. Speed Management Strategies and Their Impact and Relative Cost

Strategy	Impact Area			Relative Implementation Time			Relative Cost			Relative Impact		
	RwD	Intersections	Ped/Bike	Immediate	Short Term	Long Term	Low	Midrange	High	High	Midrange	Project Specific
Examine ways to include implications on bicyclists and pedestrians for different locations and facilities within setting of speeds. Balance multimodal interests within the context of the facility, considering the different users and uses.			X		X			X				X
Review locations that transition from higher speeds to lower speeds to evaluate the speed limits and the location of the speed limit signs.	X	X	X			X	X			X		
Traffic Signals												
Develop a plan to systematically review all signal timings to ensure yellow and all-red clearance intervals are appropriate for the speed limit and the intersection geometry.		X		X				X			X	
Review flashing traffic light operations		X		X			X				X	
Improve signal hardware for pedestrians, bicyclists, and people with disabilities.												
Coordinate signals on arterials to promote progression and uniform speed.		X				X		X		X		
Targeted Enforcement												
Determine specific corridors with a high speeding-related roadway departure or intersections crash history and conduct high visibility enforcement and education efforts.	X	X				X			X	X		
Enforce speed limits along high speeding-related crash locations where data indicates is increased risk of pedestrian or bicyclist involvement, such as schools, busy urban areas, etc.			X			X			X	X		
Internal Training												
Conduct a training workshop for internal planning, design, and traffic staff (and others as appropriate) devoted to speed and speed management, including functional classification, choosing design speed, measuring operating speeds, setting speed limits, choosing speed management countermeasures, designing safe roadsides, and transitioning between high/low speed areas.	X	X	X		X		X				X	

Source: (PennDOT, 2016)

The Pinellas County Pedestrian Safety Action Plan (Tindale Oliver & Associates, Inc., 2009) also discusses how signal progression could be used to reduce speeds and improve pedestrian safety on arterials, especially when physical changes or modifications to infrastructure are not feasible. In the plan, the importance of associating the strategy with speed limit signs, other Intelligent Transportation System (ITS) messaging devices, proper signal spacing, and cycle lengths is emphasized.

2.1.1 Small Coordination Zones

A research study confirmed that speeding could be discouraged on arterial roadways by using short cycle lengths and lowering progressing speeds (Table 2) (Furth et al., 2018). The authors used case studies to illustrate that speeding could be reduced on an arterial with minimal or no vehicular delay when lower cycle length and progression speed are used together with small "coordination zones." In this case, each coordination zone needs to have its own cycle length (Furth et al., 2018). This strategy is also recommended for long arterials comprising many intersections (Hao et al., 2018). Two corridors in Boston were used for the case studies—Massachusetts Avenue and Melnea Cass Boulevard. VISSIM software was used to generate a simulation model using AM and PM peak-hour volumes and 15-minute periods. A lower degree of saturation, closer intersection spacing, recall, and minimum green parameters that make the length of the arterial through phases less variable were indicated as factors that could lead to speeding (Furth et al., 2018). Adaptive control approaches without a common cycle may also reduce speeding (Furth et al., 2018).

Table 2. Timing Plan Performance for Different Cycle Lengths and Progression Speeds

				Off-peak			Peak		
Cycle length (s)	Progression speed (mph)	Ideal cluster size	Pedestrian delay (s)	Vehicular delay (s)	% unconstrained vehicles	% speeders	Vehicular delay (s)	% unconstrained vehicles	% speeders
70	15	1.3	28	57	12%	2.7%			
	20	1.7	28	33	16%	4.1%			
	25	2.1	28	32	16%	4.1%			
	30	2.6	28	31	22%	5.5%			
	35	3.0	28	33	22%	5.4%			
80	15	1.5	33	54	16%	3.6%	80	9%	1.2%
	20	2.0	33	44	17%	3.7%	46	10%	1.8%
	25	2.4	33	36	22%	5.2%	45	10%	2.1%
	30	2.9	33	30	24%	5.7%	43	11%	2.3%
	35	3.4	33	41	24%	5.7%	50	14%	2.8%
100	15	1.8	43	46	23%	4.7%	57	12%	2.1%
	20	2.4	43	38	23%	4.7%	45	15%	2.9%
	25	3.1	43	37	25%	4.9%	43	15%	3.0%
	30	3.7	43	33	28%	6.0%	43	16%	3.3%
	35	4.3	43	35	32%	7.0%	58	16%	3.3%
120	15	2.2	53	62	23%	5.2%	74	12%	2.1%
	20	2.9	53	48	25%	5.5%	54	15%	3.2%
	25	3.7	53	35	35%	8.5%	54	17%	3.5%
	30	4.4	53	35	36%	8.6%	49	20%	4.0%
	35	5.1	53	35	36%	8.3%	49	20%	4.0%

Source: (Furth et al., 2018)

2.1.2 Busch Boulevard Signal Retiming Speed Management Techniques

Iteris, Inc. (2020) assisted FDOT in retiming 17 intersections along Busch Boulevard (SR-580) (Figure 3) in Hillsborough County, Florida. The main objective of the retiming project was to "improve progression while providing speed management on Busch Boulevard (SR-580) from end to end for all road users" (Iteris, Inc, 2020). Results of the before-after analysis of the retiming project showed significant improvement in delays and number of stops after implementation (Table 3, 4, and 5). Although safety effects of the retiming project were not assessed, the project is anticipated to reduce crashes at the intersections (Iteris, Inc, 2020).

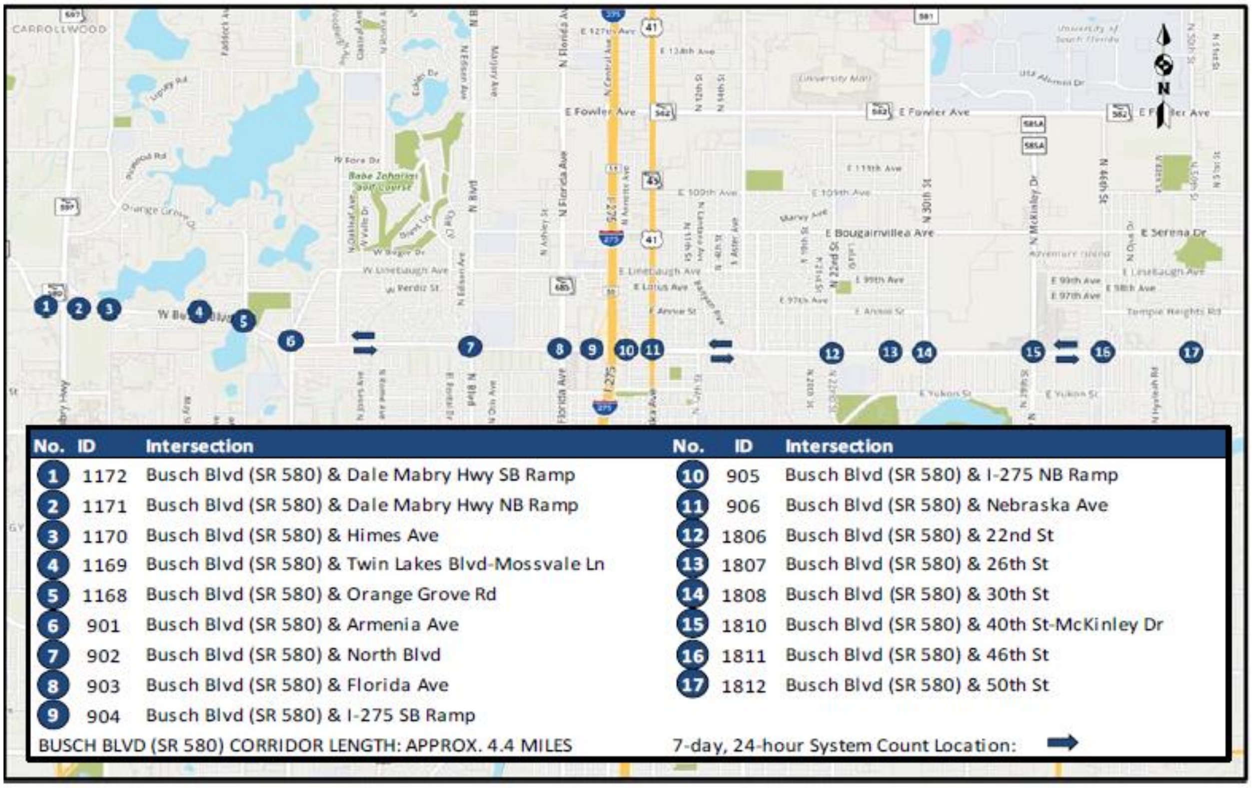


Figure 3. Traffic signals included in Busch Blvd retiming project

Source: (Iteris, Inc., 2020)

Table 3. Network Performance Measures before and after Signal Retiming and Implementation of Speed Management (Synchro)

	AM Peak			MD Peak			PM Peak			PM Off-peak		
	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference
Total Delay (hr)	1,252	1,023	-18.3%	541	431	-20.3%	1,314	1,001	-23.8%	374	350	-6.4%
Total Stops	48,811	47,762	-2.1%	35,678	31,733	-11.1%	53,424	52,841	-1.1%	26,105	27,152	4.0%
Total Travel Time (hr)	2,148	1,920	-10.6%	1,205	1,096	-9.0%	2,257	1,944	-13.9%	964	940	-2.5%
Fuel Consumed (gal)	2,781	2,602	-6.4%	1,778	1,648	-7.3%	2,956	2,721	-7.9%	1,431	1,427	-0.3%
	Weekend AM Peak			Weekend MD Peak			Weekend PM Peak					
	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference			
Total Delay (hr)	312	227	-27.2%	671	489	-27.1%	454	345	-24.0%			
Total Stops	21,299	18,482	-13.2%	35,385	34,887	-1.4%	30,702	28,514	-7.1%			
Total Travel Time (hr)	778	693	-10.9%	1,390	1,208	-13.1%	1,059	950	-10.3%			
Fuel Consumed (gal)	1,156	1,060	-8.3%	1,949	1,811	-7.1%	1,573	1,469	-6.6%			

Source: (Iteris, Inc., 2020)

Table 4. Network Performance Measures before and after Signal Retiming and Implementation of Speed Management (SimTraffic)

	AM Peak			MD Peak			PM Peak			PM Off-peak		
	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference
Total Delay (hr)	1,843	1,715	-6.9%	630	501	-20.4%	1,800	1,493	-17.0%	396	377	-4.9%
Total Stops	53,087	54,371	2.4%	29,608	26,068	-12.0%	59,656	55,129	-7.6%	21,175	21,004	-0.8%
Total Travel Time (hr)	3,065	2,932	-4.4%	1,566	1,439	-8.1%	3,060	2,789	-8.9%	1,232	1,211	-1.7%
Fuel Consumed (gal)	1,842	1,826	-0.9%	1,265	1,235	-2.4%	1,906	1,908	0.1%	1,080	1,073	-0.6%
	Weekend AM Peak			Weekend MD Peak			Weekend PM Peak					
	Existing	Implemented	Difference	Existing	Implemented	Difference	Existing	Implemented	Difference			
Total Delay (hr)	298	240	-19.4%	911	647	-29.0%	593	382	-35.6%			
Total Stops	16,777	14,890	-11.2%	34,655	30,516	-11.9%	26,291	22,094	-16.0%			
Total Travel Time (hr)	951	891	-6.2%	1,927	1,656	-14.1%	1,457	1,225	-15.9%			
Fuel Consumed (gal)	840	819	-2.5%	1,408	1,358	-3.6%	1,149	1,097	-4.5%			

Source: (Iteris, Inc., 2020)

Table 5. Busch Boulevard Signal Retiming Summary Results

Average Total Travel Time & Delay — Busch Blvd (SR 580): 1.4 miles

		AM Peak		MD Peak		PM Peak		PM Off-peak		Weekend AM Peak		Weekend MD Peak		Weekend PM Peak	
		Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)
Eastbound	Existing	254	140	168	54	411	297	177	62	167	52	219	104	179	65
Eastbound	Implemented	231	116	145	30	243	129	145	31	145	30	185	71	153	38
Eastbound	Difference	-23		-23		-168		-32		-22		-34		-26	
Eastbound	% Difference	-9.1%	-16.4%	-13.7%	-42.6%	-40.9%	-56.6%	-18.1%	-51.6%	-13.2%	-42.3%	-15.5%	-32.7%	-14.5%	-40.0%
Westbound	Existing	145	30	161	47	181	66	157	42	156	42	156	41	166	52
Westbound	Implemented	138	24	148	34	126	12	131	17	121	7	122	8	136	22
Westbound	Difference	-7		-13		-55		-26		-35		-34		-30	
Westbound	% Difference	-4.8%	-23.3%	-8.1%	-27.7%	-30.4%	-83.3%	-16.6%	-61.9%	-22.4%	-83.3%	-21.8%	-82.9%	-18.1%	-57.7%

Eastbound : Dale Mabry Hwy SB Ramp to Armenia Ave
Westbound : Armenia Ave to Dale Mabry Hwy SB Ramp

Average Total Travel Time & Delay — Busch Blvd (SR 580): 1 miles

		AM Peak		MD Peak		PM Peak		PM Off-peak		Weekend AM Peak		Weekend MD Peak		Weekend PM Peak	
		Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)
Eastbound	Existing	234	142	171	79	221	129	159	67	174	82	186	94	165	74
Eastbound	Implemented	191	99	133	41	231	139	115	24	127	35	140	48	97	5
Eastbound	Difference	-43		-38		10		-44		-47		-46		-68	
Eastbound	% Difference	-18.4%	-30.3%	-22.2%	-48.1%	4.5%	7.8%	-27.7%	-65.7%	-27.0%	-57.3%	-24.7%	-48.9%	-41.2%	-91.9%
Westbound	Existing	263	171	142	50	206	114	153	62	147	56	275	183	211	119
Westbound	Implemented	234	142	116	24	177	85	120	28	108	16	124	32	103	12
Westbound	Difference	-29		-26		-29		-33		-39		-151		-108	
Westbound	% Difference	-11.0%	-17.0%	-18.3%	-52.0%	-14.1%	-25.4%	-21.6%	-53.2%	-26.5%	-69.6%	-54.9%	-82.5%	-51.2%	-90.8%

Eastbound : North Blvd to Nebraska Ave
Westbound : Nebraska Ave to North Blvd

Average Total Travel Time & Delay — Busch Blvd (SR 580): 2 miles

		AM Peak		MD Peak		PM Peak		PM Off-peak		Weekend AM Peak		Weekend MD Peak		Weekend PM Peak	
		Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)
Eastbound	Existing	250	89	280	118	259	97	220	58	237	76	296	134	353	192
Eastbound	Implemented	208	46	204	42	208	47	182	20	169	7	188	27	212	50
Eastbound	Difference	-42		-76		-51		-38		-68		-108		-141	
Eastbound	% Difference	-16.8%	-47.2%	-27.1%	-64.4%	-19.7%	-52.6%	-17.3%	-65.5%	-28.7%	-89.5%	-36.5%	-80.6%	-39.9%	-73.4%
Westbound	Existing	276	114	249	87	293	131	276	114	256	94	300	138	287	126
Westbound	Implemented	238	76	218	57	277	115	217	55	245	83	232	70	216	54
Westbound	Difference	-38		-31		-16		-59		-11		-68		-71	
Westbound	% Difference	-13.8%	-33.3%	-12.4%	-35.6%	-5.5%	-12.2%	-21.4%	-51.8%	-4.3%	-11.7%	-22.7%	-49.3%	-24.7%	-56.3%

Eastbound : 22nd St to 50th St
Westbound : 50th St to 22nd St

Source: (Iteris, Inc., 2020)

2.2 Existing Safety and Mobility Evaluation

Studies evaluating the impacts of traffic signal progression on speed and safety are limited; this section summarizes examples of available evaluations. Delaware DOT summarized the advantages and disadvantages of traffic signal progression in its traffic design manual (Table 6), with benefits and drawbacks related to safety, operations, multimodal, and others highlighted. Overall, signal coordination can reduce rear-end and left turn crashes and improve traffic flow and can also decrease conflicts between pedestrians and motor vehicles (Delaware DOT, 2015). However, traffic signal coordination also can encourage speeding (Delaware DOT, 2015).

Table 6. Benefits and Drawbacks of Signal Coordination

Table IV-14
Summary of Issues for Providing Signal Coordination

Characteristic	Potential benefits	Potential Liabilities
Safety	Fewer rear-end and left-turn collisions.	May promote higher speeds
Operations	Improves traffic flow.	Usually longer cycle lengths.
Multimodal	May reduce pedestrian-vehicle conflicts.	May result in longer pedestrian delays due to longer cycle lengths.
Physical	No physical needs.	None identified.
Socioeconomic	Reduces fuel consumption, noise, and air pollution	None identified.
Enforcement, Education, and Maintenance	May result in less need for speed enforcement.	Signal timing plans need periodic updating.

Source: (Delaware DOT, 2015)

2.2.1 Effects of Traffic Signal Progression on Crash Frequency and Severity

In the U.S., there is growing interest in improving the safety of pedestrians/bicyclists on various types of roadways. As speed increases, the probability of pedestrian/bicyclist injuries and fatalities increases. Traffic calming strategies may be appropriate for low-speed roadways (e.g., collectors) but not for urban arterials. Signal progression is a design strategy often considered for speed management and to improve safety, especially on urban arterials. A summary of selected past studies that demonstrated the benefits of traffic coordination and progression in reducing crashes is provided in Table 7 (Delaware DOT, 2015). One study indicated that signal coordination can reduce overall crashes 3–18% and rear-end crashes 14–43%. Another study highlighted that signal coordination that provides progression can reduce all crashes 10–20%. Progression speed higher than the posted speed limit can cause safety concerns (Yue, 2020). Some engineers recommend setting progression speeds lower than posted speed for safety reasons (Yue, 2020). “Under multiple driveway conditions, progression speed should be adjusted to the platoon’s travel speed” (Yue, 2020).

Table 7. Safety Effects of Signal Coordination on Progression

Treatment	Finding
Signal Coordination	• 3 to 18% estimated reduction in all collisions along corridor • 14 to 43% estimated reduction in rear-end collisions along corridor
Provide Signal Progression	• 10 to 20% estimated reduction in all collisions along corridor

Source: (Delaware DOT, 2015; Chandler et al., 2013)

Signal progression is also used as a strategy to enforce a slow speed zone and safety along signalized roadways. The strategy can be combined with other speed enforcement types to encourage slow speed and promote safety. For example, New York City employed signal progression to reduce speeds along 25 corridors with high crash rates as part of its Vision Zero initiative. Signal timing was changed for those corridors, and the speed was reduced to 25 mph. A speed limit sign and police enforcement were used to supplement the signal progression strategy and to ensure that vehicles yielded to pedestrians (Porter et al., 2016; Lin et al., 2017) (Figure 4).

Figure 4. New York City slow-zone sign

Source: (Porter et al., 2016)

Signal retiming can reduce crashes related to multiple stops. For example, "if progression along an arterial corridor is improved so that the number of times that vehicles must stop is decreased, the number of rear-end crashes can be expected to decrease" (CTRE, 2018; Antonucci et al. 2004). Improved signal progression can be a short-term and low-cost strategy for reducing rear-end collisions on corridors (Table 8).

Wei and Tarko evaluated the impact of coordination on safety (2011) by assessing the relationship between arterial signal coordination and rear-end and right-angle crash frequencies. Rear-end and right-angle crash types were selected because they are predominant at signalized coordinated intersections. The authors employed multinomial logit models to evaluate the probability and severity of crashes on coordinated arterials using 15-minute intervals. The research found that platoons of vehicles approaching coordinated intersections during the second half of a green phase are less likely to be involved in severe and less-severe crashes than for non-coordinated intersections. Other important factors that can decrease crash rates at coordinated intersections include short distances between intersections, short cycle lengths, and right-turn bays. However, coordination can increase crash levels for other parts of the traffic flow (Wei and Tarko, 2011). Signal coordination was also found to reduce overall crashes by10–20% (FHWA, 2015).

Table 8. US-250 Corridor Study Recommended Improvements

Safety Issue		Opportunities for Improvement
1	Eastbound Rear End	• Adjust signal timings to improve progression of traffic through the intersection.
2	Eastbound Angle	• Re-configure signal timings to include additional all-red timing. • Redirect northbound left-turning traffic and optimize signal phasing. • Install angled visors on the eastbound signal heads at the US 250 / US 29 NB intersection. • Implement red-light running enforcement strategies at this location.
3	Other	• Install elongated route shield pavement marking for US 29 in the westbound left-turn lane. ❖ Widen westbound approach to two through lanes. ❖ Permanently close the NB left-turn lane by a raised median.

• Short-term, low-cost
○ Intermediate, medium-cost
❖ Long-term, high-cost

Source: (Kimley-Horn & Associates, 2018)

2.2.2 Effects of Traffic Signal Progression on Average Speed and Speed Variance on Corridors

The primary objective of signal progression is to enable mobility and reduce delays on high-priority lanes for vehicles (Wang, 2020). Signal coordination can encourage speeding (Delaware DOT, 2015). The Corridor Synchronization Performance Index (CSPI) is a measure created in 2009 by the Orange County Transportation Authority (OCTA), the California Department of Transportation (Caltrans), and local agencies in the county for assessing the performance of signalized arterials (Wang, 2020). This index is based on three scores (Figure 5): “1) average speed, with the highest possible score of 36; 2) the ratio of the number of greens versus reds through signalized intersections, with the highest possible score of 40; and 3) the average number of stops per mile, with the highest possible score of 33.” The relationship between a CSPI score and the quality of the progression is shown in Table 9. It can be seen that with the CSPI score, the quality of progression is directly related to average speed—the higher the average speed, the better the progression.

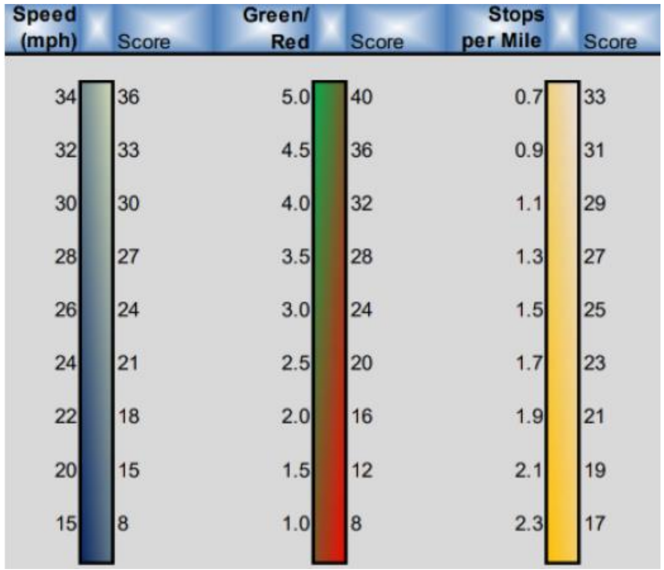


Figure 5. CSPI scoring methodology based on three measures

Source: (Wang, 2020)

Table 9. CSPI Corridor Synchronization Performance Criteria

CSPI Score	Signal Synchronization Description	Level
>=80	Very good progression – traveling through signalized intersections with minimal stops and favorable travel speeds.	Tier 1
70-80	Good progression – traveling through signalized intersections with few stops and good travel speeds.	Tier 2
60-70	Fair progression – traveling through signalized intersections with moderate stops and fair travel speeds.	Tier 3
50-60	Limited progression* – traveling through signalized intersections with moderately high stops and slower travel speeds.	Tier 4
< 50	Very limited progression* – traveling through signalized intersections with frequent stops and slow travel speeds.	Tier 5

Source: (Wang, 2020)

Wang (2020) created a new performance measure called attainability of ideal progression (AIP). The equation of the new measure is estimated as a function of average speed and progression speed, as illustrated below:

$$AIP = \frac{Average\ Speed}{Ideal\ Progressive\ Speed} \times 100\% \quad (1)$$

Another recent study recommended a progression speed slightly higher (3 mph) than the posted speed or using the posted speed limit (Table 10 and Table 11). The author suggested using 85th percentile traffic flow speed when traffic flow speed is different from posted speed limit (Yue, 2020). However, to regulate speeds, the progression speed needs to be set at or below the speed limit based on the operational objective in North Carolina Department of Transportation (NCDOT) *Signal System Timing Philosophy Manual* (NCDOT, 2017) (Table 12).

Table 10. Progression Speed Design of Frequent Conditions

Adjusted Progression Speed (mph)		Speed Limit (mph)					
		30	35	40	45	50	55
Volume	Low	35	39	45	49	55	60
	Medium	32	37	42	47	53	57
	High	30	35	40	45	50	55
	Capacity	30	35	40	45	50	55
	Overcapacity	30	35	40	45	50	55

Source: (Yue, 2020)

Table 11. Conservative Progression Speed Design of Frequent Conditions

Adjusted Progression Speed (mph)		Speed Limit (mph)					
		30	35	40	45	50	55
Volume	Low	30	35	40	45	50	55
	Medium	30	35	40	45	50	55
	High	30	35	40	45	50	55
	Capacity	30	35	40	45	50	55
	Overcapacity	30	35	40	45	50	55

Source: (Yue, 2020)

Table 12. Operational Objectives of North Carolina Department of Transportation

Operational Objective	Ways Operational Objectives Are Used
Improve Efficiency of Free Run Ops	Modify an undersaturated individual traffic signal's timing parameters to minimize delay.
Regulate Speeds/Traffic Calming	Restrain vehicles to a desired speed. Examples: Set the progression speed at or below the speed limit. Set offsets to cause planned stops restraining buildup of platoon speeds.
Minimize Cycle Failures	Modify to avoid vehicles not being served due to inadvertent gap-outs.
Improve Travel Times/Minimize Delay	Modification such that the average after travel time run is faster than the average before travel time run from one end of the corridor to the other end of the corridor.
Maximize Throughput	Modification to allow the maximum number of vehicles to traverse the corridor.
Improve Progression	Minimize number of stops.
Improve Safety	Modify corridor/network to reduce vehicle on vehicle, or vehicle on pedestrian/bicycle accidents.
Reduce Citizen Complaints	Address issues raised by the public or politicians.
Reduce Congestion	Minimize the accumulation of vehicles in a specific area, or to minimize the time length of the accumulation.
Minimize Environmental Impacts	Maximize fuel economy, minimize fuel consumption, or minimize pollution (carbon monoxide, mono-nitrogen oxides, and volatile organic compounds) emissions.

Source: (NCDOT, 2017)

2.2.3 Effects of Traffic Signal Progression on Operational Performance of Corridors and Intersections

Levels of progression and delay are commonly used to assess the performance of intersections (CTRE, 2018). Traffic coordination is employed to reduce travel time (speed), number of stops, and delays and to improve platoon progression between signals on signalized corridors (Robinson et al., 2000; Delaware DOT, 2015; Prassas and He, 2017; Wang, 2020). The relationships between signal coordination and three common level of service (LOS) factors—travel time/travel speed, number of stops per mile, and vehicle delay—are summarized in Table 13. Traffic signal progression is a beneficial strategy to relieve congestion when the option of changing the physical network is unavailable (Bhattachary, 2004).

Table 13. Performance Metrics Used in Current Practices

Travel Time /Travel Speed	Good quality of progression can be demonstrated by the reduced travel time or the increased travel speed.	Travel-run trajectories	Travel time or average speed can be an intuitive performance measure to the public, operators, planners, and maintenance staff.	It can be influenced by non-signal-timing factors such as congestion levels along arterials. It does not reveal travelers' perceptions.
Number of Stops per Mile	The fewer stops per mile, the smoother platoon operation achieved, which indicates a better quality of signal coordination.	Travel-run trajectories	It closely relates to fuel consumption, polluting emissions, and the underlying feelings of travelers	It is sensitive to signal density, and the number of stops is not differentiated regarding stop duration
Vehicle Delay	Good quality of signal coordination typically can reduce the average delay of the vehicles in the system. Some measures were developed based on vehicle delay, e.g., Performance Index [21] which is a combination of cumulative delay and the number of stops incurred on the trip (usually one stop equals to 20-second delay time)	Simulation studies or mathematic calculations	Average vehicle delay is a network-level metric, which covers traffic on the side streets	It is challenging to measure vehicle delay time in the real world.

Source: (Wang, 2020) (*partial table*)

Signal progression is often used to improve the travel experience and mobility of pedestrians, bicyclists, and transit users (NCDOT, 2017; Kittelson & Associates, Inc., 2012) (Table 14). For example, to improve the flow of bicyclists through a corridor, it is useful to set up the progression speed near the typical cycling speed. During the process, it is important to balance progression for both bicyclists and motorists on the main and cross streets (Kittelson & Associates, Inc., 2012). Progression is noted as one of the factors that influences cyclist signal compliance. For high-volume bicycle routes, it is crucial to consider bicycle progression. Similarly, Bhattachary (2004) explained that it is critical to consider both pedestrian and vehicle progression in signal timing design to effectively address the needs of both vehicles and pedestrians. This is especially important for roadways with high pedestrian volumes.

Table 14. Signal Progression Strategies and Associated Action Plan

Strategies	Action Plan
Optimize Mainline Progression	Develop a timing plan for an undersaturated or saturated corridor/network that allows vehicle platoons to travel along a corridor in the primary direction of traffic flow with a minimum of stops/delays at intersections.
Bi-directional Progression	Develop a timing plan for an undersaturated or saturated corridor that allows vehicle platoons to travel along a corridor in both directions with a minimum of stops/delays at intersections.
Transit Progression	Use of equipment, software and/or timing plans that allow busses priority travel along a corridor with a minimum of stops/delays at intersections.
Pedestrian Progression	Develop a timing plan that allow pedestrians to walk at 3.5 feet per second along the corridor/network with a minimum of waiting at the intersections.
Bicycle progression	Use of equipment or development of timing plans that allow bicyclists to ride along the corridor/network at an average bicycle speed with a minimum of delay at the intersections.
Maximize Mainline Capacity	Develop a timing plan for an undersaturated or saturated corridor that provides as much green time as possible to the mainline through movements.
Reduce Vehicle Delays	Develop a timing plan for an undersaturated or saturated corridor/network to minimize the amount of time vehicles are stopped or traveling below the speed limit.
Reduce Transit Delay	Use of equipment, software and/or timing plans that allow busses to minimize the amount of time they are stopped or traveling below the speed limit.
Reduce Pedestrian Delay	Use of equipment or development of timing plans on a corridor/network to minimize pedestrian wait times at intersections.
Reduce Bike Delay	Use of equipment or development of timing plans on a corridor/network that minimize bicyclists' wait times at intersections.
Reduce Max Outs (free run)	Provide adequate Max Green values for individual traffic signals during undersaturated actuated-uncoordinated operation.
Reduce Cycle and Split Failures	Develop a timing plan for an undersaturated corridor/network to ensure all vehicles are served each cycle.
Reduce Progression ("cut through traffic")	Develop a timing plan for an undersaturated corridor/network that limits vehicle platoon progression speed to the speed limit or below.
Fine-tune Individual Signal Parameters	Modify individual traffic signal timing parameters such as Min Green, gap/extension, Max Green.

Source: (NCDOT, 2017)

As noted, progression is an important measure to evaluate signalized intersections. Signal progression has been directly linked to LOS. For example, Table 15 shows the relationship between LOS and the quality of progression. A case study in Pima County also illustrates how a good progression design can reduce the number of arrivals on red and increase the number of arrivals on green (Movision, 2018) (Figure 6).

Table 15. Signal Progression and LOS

LOS	Seconds of Delay	Description
A	Less than 10	Very low vehicle delays, free traffic flow, signal progression extremely favorable, most vehicles arrive during given signal phase.
B	10- 20 sec	Good signal progression, more vehicles stop and experience higher delays than for LOS A.
C	20 to 35 sec	Stable traffic flow, fair signal progression, significant number of vehicles stop at signals.
D	35 to 55 sec	Noticeable traffic congestion, longer delays and unfavorable signal progression, many vehicles stop at signals.
E	55 to 80 sec	Limit of acceptable vehicle delay, unstable traffic flow, poor signal progression, traffic near roadway capacity, frequent cycle failures.
F	More than 80.0	Unacceptable delay, extremely unstable flow, heavy congestion, traffic exceeds roadway capacity, stop-and-go conditions.

Source: Highway Capacity Manual, Transportation Research Board, 2000.

Source: (City of Austin Transportation Department, 2016)

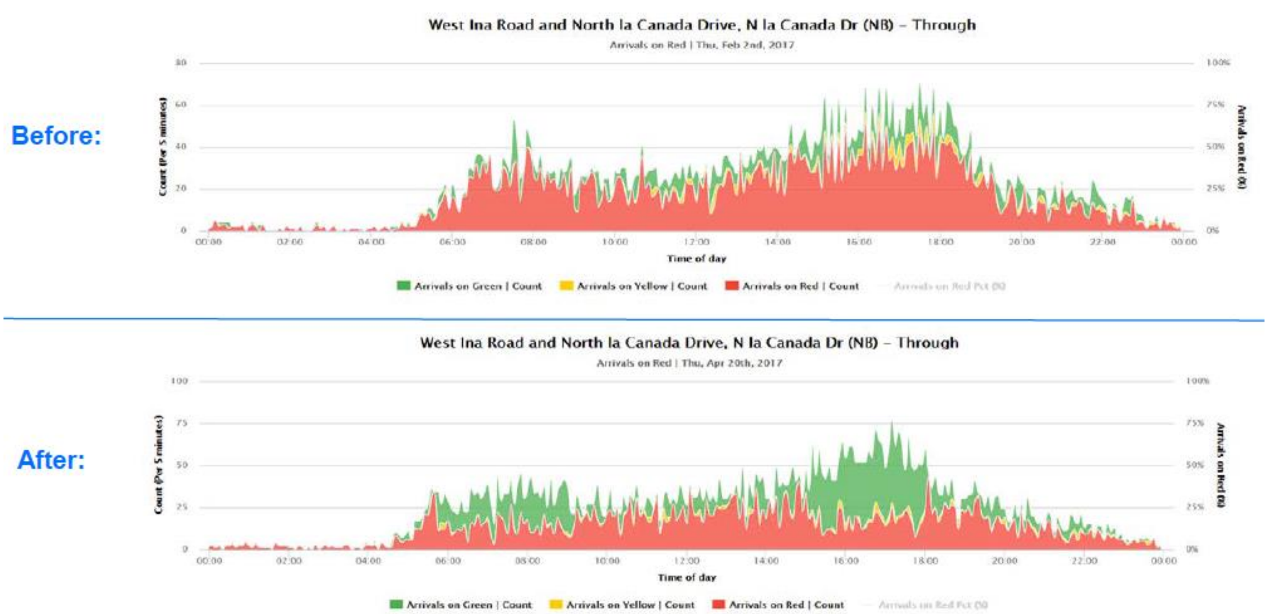


Figure 6. Effects of progression on arrivals on red

Source: (Movision, 2018)

Several operational factors can influence traffic progression and performance, including vehicle delay, queue length, percent arrival on green, and travel time (Table 16). For example, during the

US-250 corridor study, reconfiguring traffic signal timings was indicated as a good strategy to help improve progression of eastbound and westbound through vehicles, which also reduces congestion and delays at intersections. The specific improvements were as follows (Kimley-Horn & Associates, 2018):

- In the AM peak hour, overall intersection delay decreased from 22.8 to 20.0 sec.
- In the PM peak hour, overall intersection delay decreased from 11.3 to 8.5 sec.

Table 16. Objective, Category, and Performance Measure of a Traffic Signal System

Objective	Category	Performance Measure	
		Name	Purpose
Sustain System Health	Communication	Percentage of communication loss	Health index of communication
		Data completeness	
	Detection	Frequency of detector failure alerts	Health index of detection
Improve Operational Efficiency	Capacity Allocation	Traffic throughput/volume	Opportunity index for capacity reallocation
		Frequency of split failures	
		Degree of saturation/green-time utilization	
	Traffic Progression	Vehicle delay	Level of service Progression quality index
		Queue length	
		Percent arrival on green	
		Travel time	

Source: (Arizona DOT, 2018)

2.2.4 Effects of Other Factors Influencing Safety and Operation Performance on Corridors with Traffic Signal Progression

Several factors influence the safety and operation of corridors with traffic signal progression. For example, the City of Tucson, Arizona, acknowledged that uniform spacing of traffic signals is central to good progression, and good quality progression also depends on cycle length and vehicle speed (City of Tucson, 2011). Shorter cycle length, for example, can improve progression for two-phase signal control (FHWA, n.d.). The relationships between speed of progression, cycle length, and signalized intersection spacing are exemplified in greater detail in Figure 7, and recommended spacing values based on cycle length and posted speed are provided in Table 17. The effects of spacing on signal progression may depend on location type; for example, for intersections outside of Central Business Districts (CBDs), uniform and long traffic signal spacing is highlighted as critical to facilitate bi-directional progression on arterial roadways (Murtha, 2009; Gluck et al., 1999).

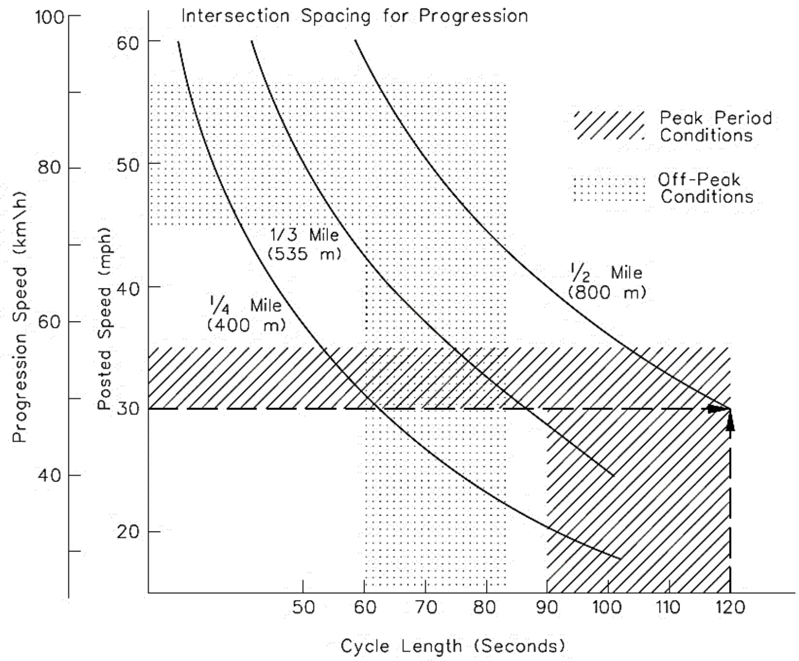


Figure 7. Relationship between speed of progression, cycle length, and signalized intersection spacing

Source: (Illinois DOT, 2020)

Table 17. Signalized Intersection Spacing Guidelines

US Customary							
Cycle Length (sec)	Posted Speed (mph)						
	25	30	35	40	45	50	55
	Intersection Spacing for Progression (2)						
60	1,100 ft	1,320 ft	1,540 ft	1,760 ft	1,980 ft	2,200 ft	2,430 ft
70	1,280 ft	1,540 ft	1,800 ft	2,050 ft	2,310 ft	2,500 ft	2,640 ft
80	1,470 ft	1,760 ft	2,050 ft	2,350 ft	2,640 ft	2,640 ft	2,640 ft
90	1,630 ft	1,980 ft	2,310 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft
120	2,200 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft
150(1)	2,640 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft	2,640 ft
Metric							
Cycle Length (sec)	Posted Speed (mph)						
	25	30	35	40	45	50	55
	Intersection Spacing for Progression (2)						
60	335 m	400 m	470 m	535 m	605 m	670 m	730 m
70	390 m	470 m	550 m	625 m	705 m	760 m	800 m
80	450 m	535 m	625 m	715 m	800 m	800 m	800 m
90	495 m	605 m	705 m	800 m	800 m	800 m	800 m
120	670 m	800 m	800 m	800 m	800 m	800 m	800 m
150(1)	800 m	800 m	800 m	800 m	800 m	800 m	800 m

Notes:

1. *Represents maximum cycle length for actuated signal if all phases are used.*
2. *From a practical standpoint when considering progression, the distance between signalized intersections will usually be 2640 ft (800 m) or less. Therefore, the values in the table have been limited to 2640 ft (800 m)*

Source: (Illinois DOT, 2020)

Phase sequences, turning movement types, and signal location are other factors that affect traffic progression. For example, serving left-turn and through movement concurrently can result in better progression and raises overall vehicular throughput (Illinois DOT, 2020). Protected left-turn mode is recommended for synchronized intersections (Oregon DOT, 2017). Although protected left-turn phases can be favorable for safety reasons, a previous study noted that they can hinder progression and induce more delays at intersections (Delaware DOT, 2015). A lagging left turn at the middle intersection can improve progression of vehicle platoons in both directions; this configuration enables two platoons to arrive at different times in the cycle (Koonce et al., 2008).

Additionally, overflows of turn bays affect progression by blocking through traffic and preventing vehicles from proceeding to downstream intersections. The location of signals is listed among factors that impact platoon progression (Murtha, 2009) (Figure 8). A previous study also indicated that a median U-turn intersection treatment can improve progression and decrease delay for through traffic on major arterials (FHWA, n.d.).

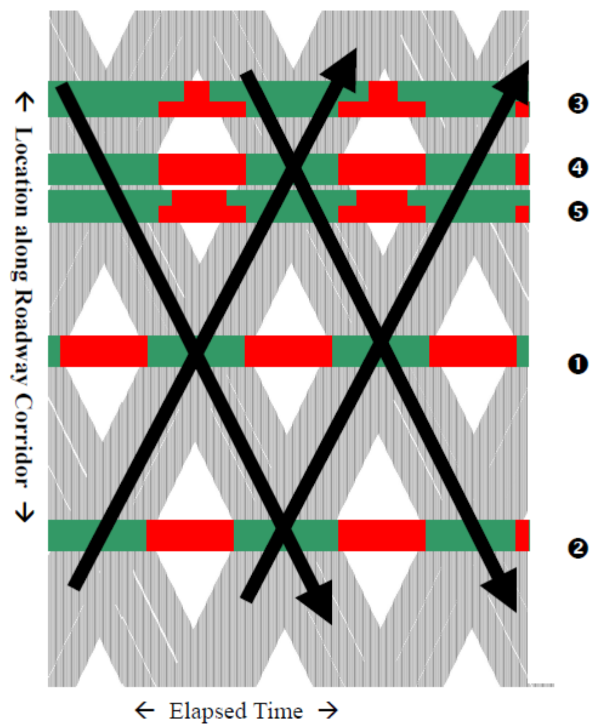


Figure 8. Idealized time-space diagram of arterial corridor illustrating effects of signal placement on platooned traffic progression

Source: (Murtha, 2009)

Other features that affect traffic progression and operation include oversaturation; lane distributions; presence of trucks, buses, and pedestrians; impacts of parking maneuvers; regulation enforcement issues; transit impacts; jaywalking; double-parking; and illegal traffic movements (Boston DOT et al., 2010). Traffic flow characteristics also affect signal progression and coordination (Bhattachary, 2004), including:

- Vehicle movement downstream in an intact platoon
- Higher proportion of through vehicles than turning vehicles (at least 80% of approach volume)

2.3 Existing Safety Performance Functions (SPFs), CMFs, and Crash Predictive Models

This section covers crash predictive models, SPFs, and CMFs of traffic signal progression on pedestrians and bicycle crashes. Crash predictive models estimate the expected average crash frequency of a site; these crashes can be total crashes or grouped by crash severity or collision type. The predictive models are applied to a given time period, traffic volume, and constant geometric design characteristics of the roadway. Chapter 12 of the *Highway Safety Manual* (HSM) (2009) elaborates on predictive models in greater detail. The steps involved in the process of estimating the models are shown in Figure 9. Although predictive models vary by facility and site type, they all comprise the following elements (HSM, 2009):

- Safety Performance Functions (SPFs) – Statistical "base" models are used to estimate the average crash frequency for a facility type with specified base conditions.
- Accident Modification Factors (AMFs) or Crash Modification Factors (CMFs) – AMFs or CMFs (terms used interchangeably throughout the document) are the ratio of the effectiveness of one condition in comparison to another condition. AMFs/CMFs are multiplied with the crash frequency predicted by the SPF to account for the difference between site conditions and specified base conditions.
- Calibration Factor (C) – Multiplied with the crash frequency predicted by the SPF to account for differences between the jurisdiction and time period for which the predictive models were developed and the jurisdiction and time period to which they are applied by HSM users.

The general form of predictive models used for urban and suburban arterials is (HSM, 1009):

$$N_{predicted} = (N_{SPF\ x} \times (AMF_{1x} \times AMF_{2x} \times \ldots \times AMF_{yx}) + N_{pedx} + N_{bikex}) \times C_x \qquad (2)$$

Where,

$N_{predicted}$ = predicted average crash frequency for a specific year on site type x

$N_{SPF\ x}$ = predicted average crash frequency determined for base conditions of the SPF developed for site type x

AMF_{yx} = AMFs specific to site type x and specific geometric design and traffic control features y

N_{pedx} = predicted average number of vehicle-pedestrian collisions per year for site type x

$N_{bike}x$ = predicted average number of vehicle-bicycle collisions per year for site type x

C_x = calibration factor to adjust SPF for local conditions for site type *x*

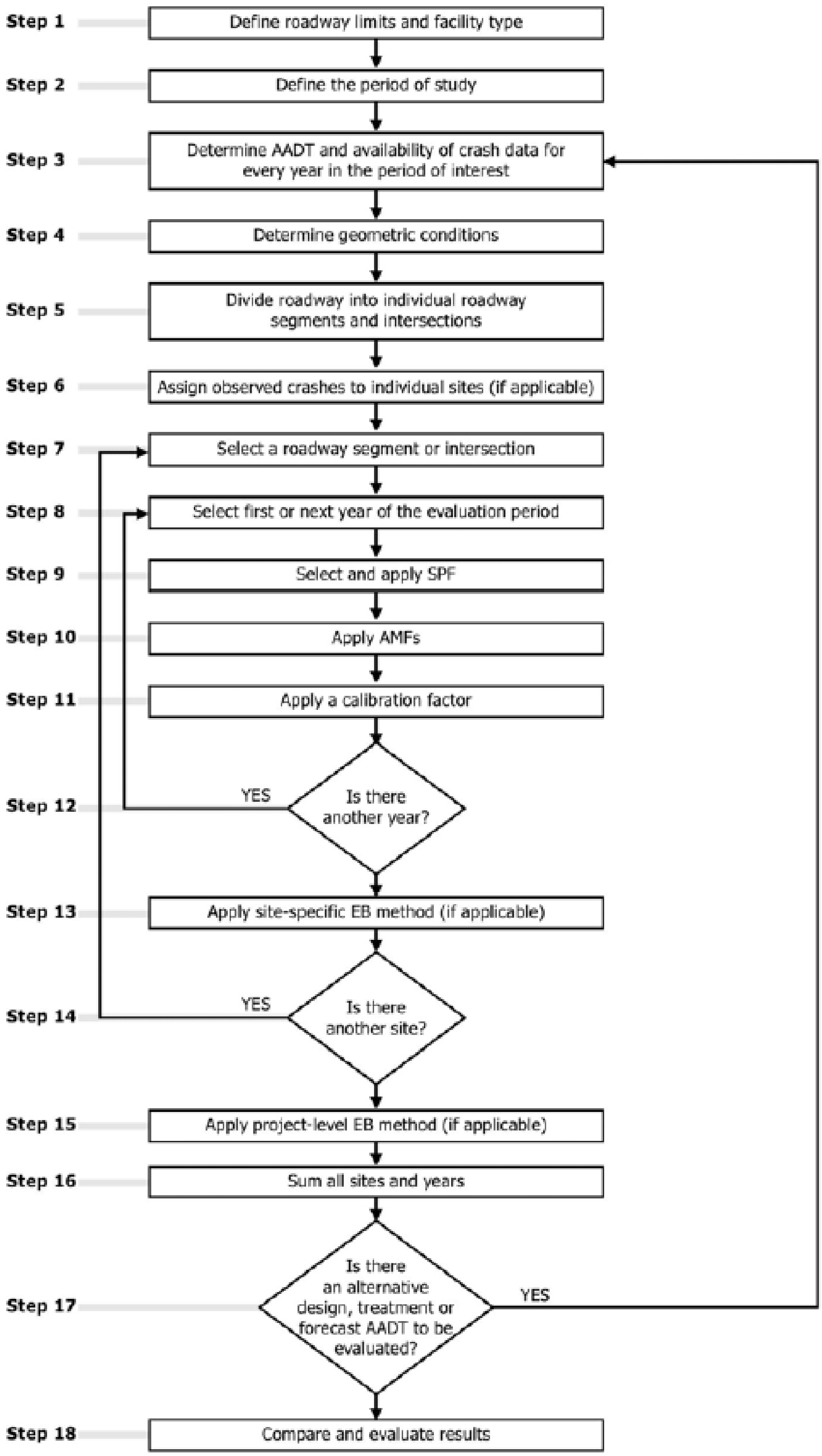


Figure 9. HSM predictive method

Source: (HSM, 2009)

Predictive models specific to intersections for urban and suburban arterials are of the following form:

$$N_{predicted\ int} = C_i \times (N_{bi} + N_{pedi} + N_{bikei}) \tag{3}$$

$$N_{bi} = N_{spf\ int} \times (AMF_{1i} + AMF_{2i} + \ldots + AMF_{6i}) \tag{4}$$

Where,

$N_{predicted\ int}$ = predicted average crash frequency of an intersection for the selected year

N_{bi} = predicted average crash frequency of an intersection (excluding vehicle-pedestrian and vehicle-bicycle collisions)

$N_{spf\ int}$ = predicted total average crash frequency of intersection related crashes for base conditions (excluding vehicle pedestrian and vehicle-bicycle collisions)

N_{pedi} = predicted average crash frequency of vehicle-pedestrian collisions

N_{bikei} = predicted average crash frequency of vehicle-bicycle collisions

AMF_{1i} … $AMF6i$ = Accident Modification Factors for intersections

C_i = calibration factor for intersections developed for use for a particular geographical area

$N_{spf\ int}$ can be grouped into two components as follows:

$$N_{spf\ int} = N_{bimv} + N_{bisv} \tag{5}$$

Where,

N_{bimv} = predicted average number of multiple-vehicle collisions for base conditions

N_{bisv} = predicted average number of single-vehicle collisions for base conditions

Advantages of the predictive method include (HSM, 2009):

- Regression-to-the-mean bias is addressed, as the method concentrates on long-term expected average crash frequency rather than short-term observed crash frequency.
- Reliance on availability of limited crash data for any one site is reduced by incorporating predictive relationships based on data from many similar sites.
- The method accounts for the fundamentally nonlinear relationship between crash frequency and traffic volume.
- The SPFs in the HSM are based on the negative binomial distribution, which are better suited to modeling the high natural variability of crash data than traditional modeling.

As described, the Intersection Safety Performance function (SPF) determines the average number of crashes per year at a specified location based on exposure. Statistical multiple regression techniques are used with a few years of observed crash data at sites with similar characteristics and varying Annual Average Daily Traffic (AADT). The negative binomial regression model, which is an extension of Poisson distribution, is used because it is better-suited for modeling crash frequencies (HSM, 2009). Roadway and traffic attributes are important inputs in the calculation (FHWA, 2014; Srinivasan and Bauer, 2013). The equation of SPF is a function of AADT of the major and minor roadways as follows (FHWA, 2014):

$$\text{Predicted crashes} = \exp\left[a + b \times \ln(AADT_{maj}) + c \times \ln(AADT_{min})\right] \quad (6)$$

Where,

$AADT_{maj}$ = annual average daily traffic volume (vehicles/day) for major road (both directions of travel combined)

$AADT_{min}$ = annual average daily traffic volume (vehicles/day) for minor road (both directions of travel combined)

a, b, c = regression coefficients

CMFs or AMFs, on the other hand, represent the anticipated long-term effects of various treatments. CMFs estimate the anticipated number of crashes after application of specific countermeasures at particular locations. CMFs with values less than 1 are anticipated to reduce the number of crashes before treatments, whereas CMFs above 1 increase the number of crashes before treatments after application. For example, a statistically-significant CMF of 0.7 is anticipated to reduce the crash rate by 30% when other traffic factors and trends are held constant, whereas a CMF of 1.2 deteriorates the safety conditions at the location and can lead to a 20% increase in crashes (FHWA, 2015; Gross et al., 2010). The effects of CMFs vary by crash type, severity, area type, geometry, traffic control, traffic volume, functional classification, and/or jurisdiction; therefore, they should not be generalized to other conditions or applied to different location characteristics. Locally-calibrated CMFs are recommended, and it is important to note the source of the CMFs because methods used to measure speed and other factors may be different.

Similarly, Crash Modification Functions (CM-Functions) are equations used to estimate CMFs based on specific location attributes (e.g., traffic volumes). CM-Functions enable CMFs to vary based on intersection characteristics (Gross et al., 2010). Although perhaps difficult in practice because of the data collection requirement, this approach is preferred compared to estimating a single CMF value. CM-Functions are valuable because the features of an intersection inform its safety level.

Table 18. Matrix of Countermeasures to Mitigate Speeding-related Crashes

Matrix of Speeding-related Crash Countermeasures	Area Type			Location Type			Documented Effects	
Countermeasure Name	Urban	Subur-ban	Rural	Intersec-tion	Section/ corridor	Curve	Crash Reducing	[1]Speed Reducing
Design and Traffic Calming								
Speed Tables	X	X	X	X				X
Traffic Calming (varied)	X	X	X	X	X		X	X
Pavement Treatments, Markings, Signs, and Signals								
Enhanced Curve Delineation			X			X	X	
Optical Speed bars / Converging Chevrons	X	X	X	X	X	X		Possibly
Transverse (in lane) Rumble Strips for Speed Calming	Pos.[3]	Pos.	X	X		X	X	
Speed Management and Traffic Operations Measures								
Lower Speed Limits on Expressway	X	X	X		X		X	
Protected-only Left Turn Signal Phasing (high-speed intersections)				X			X	
Signal Coordination along a Corridor	X	X		X	X		X	
Variable Speed Limits on Expressway [4]	X				X		X	X
Enforcement and Publicity								
Automated Section Speed Enforcement								
Mobile Speed Camera Enforcement	X	X					X	X
Fixed Speed Camera Enforcement	X	X					X	X
Publicity of Automated Speed Enforcement Cameras	X	X					X	
Speed Display / Feedback Devices	X	X	X		X		X	X [5]

Source: (FHWA, 2015)

Following a CMF estimation, assessing its quality is crucial. One way to assess the dependability of CMFs is through standard error estimates, which "provide an indication of the variability and reliability of the estimate; small standard errors that are much less than the estimated CMF indicate more robust and reliable estimates" (FHWA, 2015). Standard deviations are also used to evaluate variation of estimates. Larger standard deviations mean greater variations in the estimates. Another way to ensure the quality and reliability of CMFs is to check the five-point-star-rating system (5 = highest quality) of expert reviewers from the CMF Clearinghouse. CMFs from the HSM show "HSM" under their ratings. A Crash Reduction Factor (CRF) is equal to (1-CMF), indicating the portion or percentage reduction in crashes due to implementations of countermeasures. Examples of CMFs for coordinated intersections are provided in Table 19.

Often, treatments could be combined to reach desired safety results. However, this should be done carefully because it may lead to overestimation of the effects, especially when the countermeasures are applied for the same crash type. Additionally, it is recommended to avoid using CMFs developed based on information from high-crash locations for sites with average crash history. The high crash location CMFs may overestimate safety effectiveness when applied to locations with average crash history (Gross et al., 2010).

Table 20 and Table 21 include adapted CMFs from the HSM that could be used to estimate crash effects for countermeasures that are based on changing average travel speed. The CMFs are grouped into injury and fatal crashes (FHWA, 2020).

Table 19. Crash Modification Factors for Coordinated Intersections

Category	Countermeasure	Crash Modification Factor (CMF)	Crash Reduction Factor (CRF) (%)	Quality (5 stars Max)	Crash Type	Crash Severity	Roadway Type	Area Type
Intersection Geometry	Presence of right-turn lane on arterial with signal coordination	0.06	93.6	3 stars	Rear-end	All	All	Urban and suburban
Intersection Geometry	Presence of right-turn lane on arterial with signal coordination	0.32	68.3	2 stars	Angle	All	All	Urban and suburban
Intersection Traffic Control	Change number of traffic signal cycles per hour on arterial with signal coordination from X to Y	$e^{-0.0444(Y-X)}$	$100 \times (1-e^{-0.0444(Y-X)})$	3 stars	Rear-end	All	All	Urban and suburban
Speed Management	Increase speed limit from X to Y mph	$100 \times (1-e^{0.158(Y-X)})$	$e^{0.158(Y-X)}$	3 stars	Rear-end	All	All	Urban and suburban
Speed Management	Increase speed limit from X to Y mph	$100 \times (1-e^{0.159(Y-X)})$	$e^{0.159(Y-X)}$	2 stars	Angle	All	All	Urban and suburban

Source: (Wei and Tarko, 2011) (retrieved from CMF Clearinghouse, 2020)

Table 20. Potential Injury Crash Modification Factors (CMFs) of Changes in Average Operating Speed for a Road Section

CMFs - Injury Crashes						
Change in avg. speed (mph)	*Baseline Average Speed 30 mph*	*Baseline Average Speed 40 mph*	*Baseline Average Speed 50 mph*	*Baseline Average Speed 60 mph*	*Baseline Average Speed 70 mph*	*Baseline Average Speed 80 mph*
-5	0.57	0.66	0.71	0.75	0.78	0.81
-4	0.64	0.72	0.77	0.8	0.83	0.85
-3	0.73	0.79	0.83	0.85	0.87	0.88
-2	0.81	0.86	0.88	0.9	0.91	0.92
-1	0.9	0.93	0.94	0.95	0.96	0.96
0	1	1	1	1	1	1
1	1.1	1.07	1.06	1.05	1.04	1.04
2	1.2	1.15	1.12	1.1	1.09	1.08
3	1.31	1.22	1.18	1.15	1.13	1.12
4	1.43	1.3	1.24	1.2	1.18	1.16
5	1.54	1.38	1.3	1.26	1.22	1.2

Based on Table 3E-2, Crash Modification Factors for Changes in Average Operating Speed, Highway Safety Manual, AASHTO, 2010, p. 3-57. Used by Permission.

Source: (FHWA, 2015)

Table 21. Potential Fatal Crash Modification Factors (CMFs) of Changes in Average Operating Speed for a Road Section

CMFs - Fatal Crashes						
Change in avg. speed (mph)	*Baseline Average Speed 30 mph*	*Baseline Average Speed 40 mph*	*Baseline Average Speed 50 mph*	*Baseline Average Speed 60 mph*	*Baseline Average Speed 70 mph*	*Baseline Average Speed 80 mph*
-5	0.22	0.36	0.48	0.58	0.67	0.75
-4	0.36	0.48	0.58	0.66	0.73	0.8
-3	0.51	0.61	0.68	0.74	0.8	0.85
-2	0.66	0.73	0.79	0.83	0.86	0.9
-1	0.83	0.86	0.89	0.91	0.93	0.95
0	1	1	1	1	1	1
1	1.18	1.14	1.11	1.09	1.07	1.05
2	1.38	1.28	1.22	1.18	1.14	1.1
3	1.59	1.43	1.34	1.27	1.21	1.16
4	1.81	1.59	1.46	1.36	1.28	1.21
5	2.04	1.75	1.58	1.46	1.36	1.27

Based on Table 3E-2, Crash Modification Factors for Changes in Average Operating Speed, *Highway Safety Manual*, AASHTO, 2010,
p. 3-57. Used by Permission.

Source: (FHWA, 2015)

2.4 Methodologies for CMF and SPF Development

Developing a CMF involves considering and choosing among many factors and study designs. The choice of an appropriate method depends on data availability, the goal of the study, and many other factors (e.g., sample size). This section elaborates on numerous study designs for developing a CMF and covers in greater detail the various approaches for evaluating the quality of a CMF.

Overall, two general types of studies are used to estimate CMFs—experimental and observational. Experimental designs involve planned studies, during which researchers randomly assign sites to a treatment or to a control group. Observational studies are not planned and are based on data collected retrospectively. For observational studies, safety is evaluated based on the effectiveness of existing treatments. Observational studies are commonly used due to the ethical concerns associated with experimental designs. Specific examples of observational studies used to estimate CMFs include before-after, cross-sectional, case-control, cohort, meta-analysis, expert panel, and surrogate (Table 22). This section focuses on before-after and cross-sectional studies. Table 23 gives an overview of data requirements for those two types of designs. Highlighted later are a few ideas related to SPF development.

Table 22. Summary of Study Designs for Developing CMFs

Study Design	General Applicability	Strengths	Weaknesses
Before-After with Comparison Group	Treatment is sufficiently similar among treatment sites. Before and after data are available for both treated and untreated sites. Untreated sites are used to account for non-treatment related crash trends.	Simple. Accounts for non-treatment related time trends and changes in traffic volume.	Difficult to account for regression-to-the-mean.
Before-After with Empirical Bayes	Treatment is sufficiently similar amongst treatment sites. Before and after data are available for both treated sites and an untreated reference group. A separate comparison group may be required where the treatment has an effect on the reference group.	Employs SPFs to account for: Regression-to-the-mean. Traffic volume changes over time. Non-treatment related time trends.	Relatively complex. Cannot include prior knowledge of treatment. Cannot consider spatial correlation. Cannot specify complex model forms.
Full Bayes	Useful for before-after or cross-section studies when: Complex model forms are required. There is a need to consider spatial correlation among sites. Previous model estimates or CMF estimates are to be introduced in the modeling.	Reliable results with small sample sizes. Can include prior knowledge, spatial correlation, and complex model forms in the evaluation process.	Implementation requires a high degree of training.
Cross-Sectional	Useful when limited before-after data are available. Requires sufficient sites that are similar except for the treatment of interest.	Possible to develop CMFunctions. Allows estimation of CMFs when conversions are rare. Useful for predicting crashes.	CMFs may be inaccurate for a number of reasons including: Inappropriate functional form. Omitted variable bias. Correlation among variables.
Case-Control	Assess whether exposure to a potential treatment is disproportionately distributed between sites with and without the target crash. Indicates the likelihood of an actual treatment through the odds ratio.	Useful for studying rare events because the number of cases and controls is predetermined. Can investigate multiple treatments per sample.	Can only investigate one outcome per sample. Does not differentiate between locations with one crash or multiple crashes. Cannot demonstrate causality.
Cohort	Used to estimate relative risk, which indicates the expected percent change in the probability of an outcome given a unit change in the treatment.	Useful for studying rare treatments because the sample is selected based on treatment status. Can demonstrate causality.	Only analyzes the time to the first crash. Large samples are often required.
Meta-Analysis	Combines knowledge on CMFs from multiple previous studies while considering the study quality in a systematic and quantitative way.	Can be used to develop CMFs when data are not available for recent installations and it is not feasible to install the strategy and collect data. Can combine knowledge from several jurisdictions and studies.	Requires the identification of previous studies for a particular strategy. Requires a formal statistical process. All studies included should be similar in terms of data used, outcome measure, and study methodology.

Table 22. Summary of Study Designs for Developing CMFs, Continued

Study Design	General Applicability	Strengths	Weaknesses
Expert Panel	Expert panels are assembled to critically evaluate the findings of published and unpublished research. A CMF recommendation is made based on agreement among panel members.	Can be used to develop CMFs when data are not available for recent installations and it is not feasible to install the strategy and collect data. Can combine knowledge from several jurisdictions and studies. Does not require a formal statistical process.	Traditional expert panels do not systematically derive precision estimates of a CMF. Possible complications may arise from interactions and group dynamics. Possible forecasting bias.
Surrogate Measures	Surrogate measures may be used to derive a CMF where crash data are not available or insufficient (e.g., there is limited after period data or the treatment is rarely implemented).	Can be used to develop CMFs in the absence of crash-based data.	Not a crash-based evaluation. The approach to establish relationships between surrogates and crashes is relatively undeveloped.

Source: (Gross et al., 2010)

Table 23. Overview of Data Needs and Inputs for Safety Effectiveness Evaluations

Data Needs and Inputs	Safety Evaluation Method			
	EB Before/After	Before/After with Comparison Group	Before/After Shift in Proportion	Cross-Sectional
10 to 20 treatment sites	✓	✓	✓	✓
10 to 20 comparable non-treatment sites		✓		✓
A minimum of 650 aggregate crashes in non-treatment sites		✓		
3 to 5 years of crash and volume "before" data	✓	✓	✓	
3 to 5 years of crash and volume "after" data	✓	✓	✓	✓
SPF for treatment site types	✓	✓		
SPF for non-treatment site types		✓		
Target crash type			✓	

Source: (HSM, 2009)

Each design has benefits and drawbacks that are important to consider before finalizing the selection. CMFs estimated using different designs may yield divergent results. For example, CMFs from cross-sectional designs are usually associated with smaller crash reductions than CMFs from before-after designs (Gross et al., 2010). Other factors that can affect the effectiveness of a study design include choosing inappropriate comparison groups, functional form, regression model specification, and incorrect interpretation of results. Figure 10 provides a flow chart to assist in selecting the appropriate design for a specific study. Several questions to assess the quality of a study for developing a CMF include (Elvik, 2002; Gross et al., 2010):

- How were units sampled for the study?
- Do the data collected in the study refer directly to the outcome of interest or to aggregated data?
- Was crash or injury severity specified?
- Were study results tested for statistical significance or their statistical uncertainty otherwise estimated?
- Did the study use appropriate techniques for statistical analysis?
- Can the causal direction between treatment and effect be determined?
- How well did the study control for confounding factors?
- Did the study have a clearly defined target group, and were effects found in the target group only?
- Are study results explicable in terms of well-established theory?

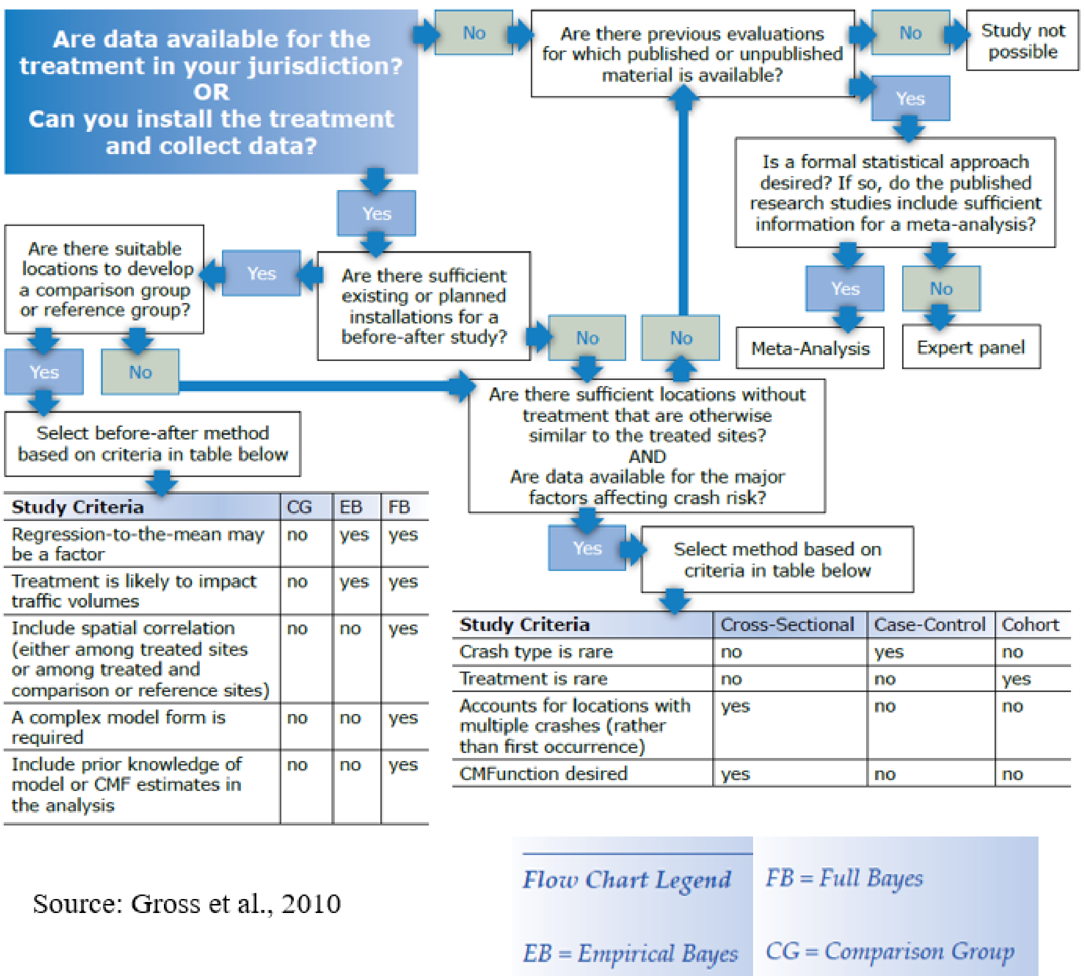


Figure 10. Flow chart for study design selection

2.4.1 Before-After Design

A before-after design is employed to evaluate safety conditions of a site before and after a treatment. For that particular design, safety data are gathered for the original conditions of the site (before treatment), then a treatment is applied, later followed by collection of another set of data for the site. Some drawbacks of before-after design include sample size and possible bias from factors not accounted for in the study. Larger sample sizes can lead to decrease standard error and, thus, more reliable estimates. With the before-after design, safety outcomes after treatments could also be due to factors such as traffic volume changes, changes in crash reporting practices, and regression-to-the-mean (HSM, 2009; Gross et al., 2010). Before-after studies that do not consider changes in other factors are called “naïve.” Many other before-after methods exist, including before-after with comparison group, Empirical Bayes, and Full Bayes. These methods are explained below.

2.4.2 Before-After with Comparison Group

For the before-after with comparison group method, untreated comparison sites are selected similar to the treatment sites with comparable geometric and operational characteristics. The ratio of crash frequency after a treatment period to crash frequency before a treatment period for the comparison group is multiplied by the crash frequency for the treatment group in the before period to get the expected crashes at the treatment sites before treatment. This number is later compared to observed crashes at the treatment sites after the treatment is applied to evaluate the benefits of the treatment:

$$N_{expected,T,A} = N_{observed,T,B} \left(N_{observed,C,A} / N_{observed,C,B}\right) \tag{7}$$

Where,

$N_{expected,T,A}$ = expected number of crashes for the treatment group that would have occurred in the after period without treatment

$N_{observed,T,B}$ = observed number of crashes in before period for treatment group

$N_{observed,C,A}$ = observed number of crashes in after period in comparison group

$N_{observed,C,B}$ = observed number of crashes in before period in comparison group

Selecting a comparison group can be tricky. An ideal comparison group is the one that yields the same before-after ratios of crash frequency as the treatment group before treatment is applied. A test of comparability could be used to identify a suitable comparison group, which can be done with a time series to compare annual trends in crash frequencies for a comparison group and a treatment group before treatment (Hauer, 1997; Gross et al., 2010). A suitable comparison group has similar trends in crash frequencies to the treatment group (Figure 11). Other things that should be considered while using the comparison group method (Gross et al., 2010; HSM, 2009) include the following:

- Before and after periods for treatment and comparison group should be the same.

- There should be reasons to believe that the change in factors other than the treatment under study (e.g., traffic volume changes) that influence safety are the same in the treatment and comparison groups.
- Crash counts must be sufficiently large (this point is discussed in more detail later).

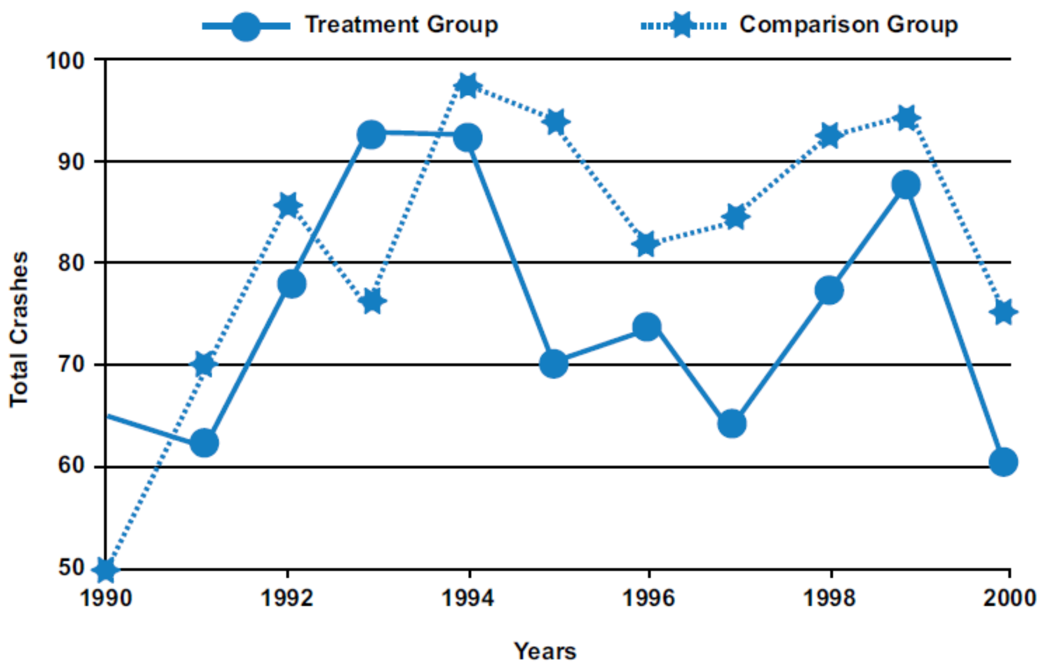


Figure 11. Example time series plot of crashes in treatment and comparison group

Source: (Gross et al., 2010)

A few formulas to calculate CMF for the before-after with comparison approach are as follows (Gross et al., 2010):

$$\mathrm{Var}\,(N_{expected,T,A}) = N_{expected,T,A}^{2}(1/\,N_{observed,T,B} + 1/\,N_{observed,C,B} + 1/\,N_{observed,C,A}) \tag{8}$$

$$\mathrm{CMF} = (N_{observed,T,A}\,/\,N_{expected,T,A})/(1+(\mathrm{Var}(N_{expected,T,A})/\,N_{expected,T,A}^{2})) \tag{9}$$

$$\text{Variance (CMF)} = \frac{\mathrm{CMF}^2[(1/N_{observed,T,A})+(\mathrm{Var}(N_{expected,T,A})/N_{expected,T,A}{}^2)]}{\left[1+var\,(N_{expected,T,A})\Big/N_{expected,T,A}{}^2\right]^2} \tag{10}$$

Where,

$N_{expected,T,A}$ = expected number of crashes for the treatment group that would have occurred in the after period without treatment

$N_{observed,T,B}$ = observed number of crashes in before period for treatment group

$N_{observed,T,A}$ = observed number of crashes in after period for treatment group

$N_{observed,C,B}$ = observed number of crashes in before period in comparison group

$N_{observed,C,A}$ = observed number of crashes in after period in comparison group

CMF = crash modification factor

Important elements that can convey if the sample size is appropriate are (Gross et al., 2010):

- Size of treatment group, in terms of number of crashes in before period (treatment sample could be increased by adding more sites or years of data)
- Relative duration of before and after periods
- Likely (postulated) CMF value
- Size of comparison group in terms of number of crashes in before and after periods

2.4.3 Empirical Bayes Before-After Studies

Similar to the before-after with comparison, the Empirical Bayes before-after method estimates crashes for individual treated sites and the expected crashes that would occur in the after-treatment period when no treatment is applied. It accounts for changes in crash frequencies due to regression-to-the-mean. The expected number of crashes in the before period is calculated as a function of observed number of crashes and predicted crashes for the same period. An SPF is employed to estimate predicted crashes in the before period as a function of traffic and physical characteristics of the sites. Example equations for SPFs for road segments and intersections are as follows (Gross et al., 2010):

For road segments:

$$\text{Crashes per year} = \alpha\ (\text{segment length})\ (\text{AADT})^{\beta} \tag{11}$$

For intersections:

$$\text{Crashes per year} = \alpha\ (\text{Major road entering AADT})^{\beta 1} \times (\text{Minor road entering AADT})^{\beta 2} \tag{12}$$

Where AADTs are traffic volumes and α, β, β_1, and β_2 are numbers estimated during SPF development.

The expected number of crashes without treatment for the Empirical Bayes approach is:

$$N_{expected,T,B} = \text{SPF weight}\ (N_{predicted,T,B}) + (1\text{-SPF weight})(N_{observed,T,B}) \tag{13}$$

Where,

$N_{expected,T,B}$ = unadjusted Empirical Bayes estimate

$N_{predicted,T,B}$ = predicted number of crashes estimated by SPF in before period

SPF weight = weight derived using the over-dispersion parameter from the SPF calibration process, but also depends on the number of years of crash data in the period before treatment

$N_{observed,T,B}$ = observed number of crashes in before period for treatment group

The SPF weight is the inverse of the over-dispersion parameter from the SPF calibration process and the number of years of crash data for the before period. SPFs can be calibrated to each year.

The result factors the relationship between crash frequency and traffic volume over time. For each year, the factor is estimated by taking the ratio of the sum of observed crashes to the sum of predicted crashes.

Figure 12 shows how SPF and observed crashes are used to get the expected number of crashes. The effect of the regression of the mean is obtained by taking the difference between observed and expected crashes before treatment.

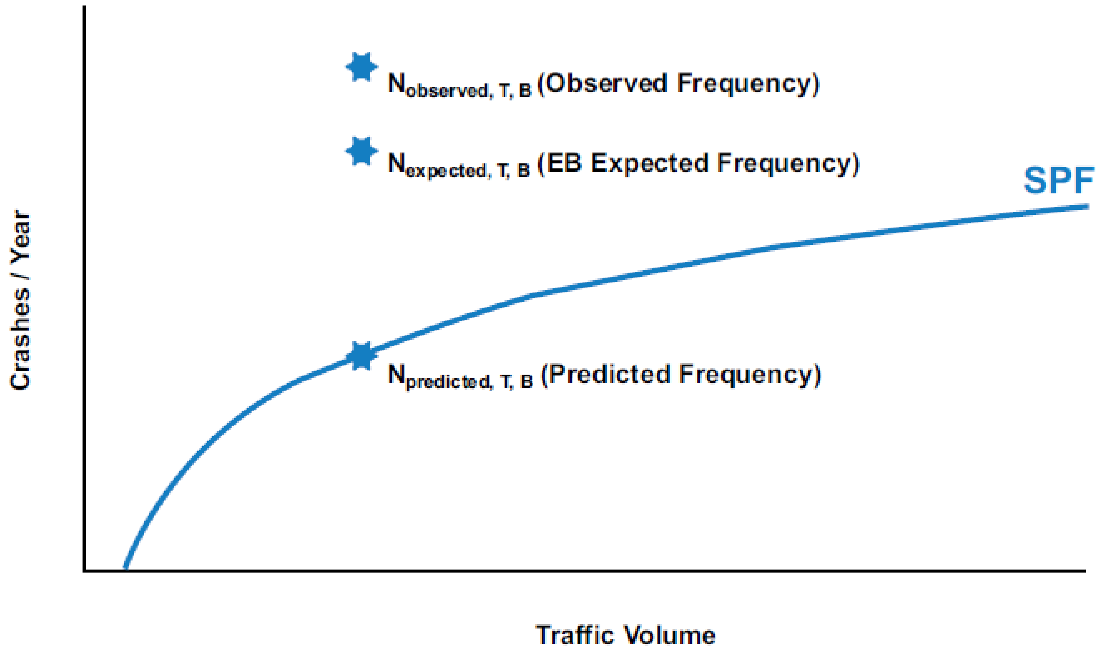


Figure 12. Illustration of regression-to-the-mean and Empirical Bayes estimate

Source: (Gross et al., 2010)

Two important equations for the expected number of crashes in the after-treatment period without treatment are (Gross et al., 2010):

$$N_{expected,T,A} = N_{expected,T,B} \left(N_{predicted,T,A} / N_{predicted,T,B}\right) \quad (14)$$

$$Var\left(N_{expected,T,A}\right) = N_{expected,T,A} \left(N_{predicted,T,A} / N_{predicted,T,B}\right)(1 - \text{SPF weight}) \quad (15)$$

Where,

$N_{expected,T,A}$ = expected number of crashes for the treatment group that would have occurred in the after period without treatment

$N_{expected,T,B}$ = unadjusted Empirical Bayes estimate

$N_{predicted,T,A}$ = predicted number of crashes estimated by SPF in after period

$N_{predicted,T,B}$ = predicted number of crashes estimated by SPF in before period

SPF weight = weight derived using the over-dispersion parameter from the SPF calibration process, but also depends on the number of years of crash data in the period before treatment

2.4.4 Full Bayes Studies

The Full Bayes before-after studies employ the reference population. Compared to the Empirical Bayes method, the Full Bayes study uses a distribution of possible values instead of a point estimate of the expected crash frequency and its variance (Gross et al., 2010). The Full Bayes approach enables complex modeling that considers multiplicative and additive factors. It is flexible and enables modeling smaller sample size data. More importantly, it can evaluate the spatial correlation between sites (the effects of proximity of site locations) and can include prior knowledge. Benefits of the Full Bayes approach include (Gross et al., 2010):

- Ability to specify complex model forms
- Potential for estimation of valid crash models with small sample sizes
- Ability to consider spatial correlation between sites in model formulation
- Ability to include prior knowledge on values of coefficients in modeling along with data collected

2.4.5 Cross-Sectional Design

Cross-sectional design involves comparing safety of sites with a particular treatment to different sites with no treatment at a single point in time (Gross et al., 2010). CMFs based on cross-sectional designs are estimated using the ratio of average crash frequencies for sites with and without countermeasures. It is preferable to use before-after designs. Cross-sectional designs are the next preferable options when the before-after studies do not have enough sites with the desired countermeasures (Gross et al., 2010). Multivariate regression models are often used to control for other factors that can affect safety and to estimate changes in crashes from a unit change of other variables. The issue with cross-sectional studies is that changes in the number of crashes may be due to unknown factors other than the controlled variables.

To obtain the required number of sites in cross-sectional studies, the following factors are important to consider (Gross et al., 2010):

- Average crash frequencies
- Number of variables desired in model
- Level of statistical significance desired in model
- Amount of variation in each variable of interest between locations

To assess the quality of CMFs obtained from cross-sectional studies, it is important to ask the following questions (Gross et al., 2010):

- Is the direction of effect (i.e., expected decrease or increase) in crashes in accord with expectations?
- Does the magnitude of the effect seem reasonable?
- Are the parameters of the model estimated with statistical significance?
- Do different cross-section studies come to similar conclusions?
- Do before-after studies come to similar conclusions?

2.4.6 Methods for Assessing Quality of CMFs

Several evaluation criteria are used to evaluate the quality of CMFs in the CMF Clearinghouse and HSM (Gross et al., 2010). As noted, a five-point rating is used to assess the quality of CMFs in the CMF Clearinghouse using study design, sample size, standard error, potential bias, and data source. Each factor is attributed specific points corresponding to an excellent, fair, or poor rating. The scores are weighted to generate a final point and a final rating. Factors with missing information do not contribute to the rating process. In addition to the CMF values, important factors to include in the CMF Clearinghouse are (Gross et al., 2010):

- *Study design* – Used to develop the CMF (i.e., comparison-group before-after, cross-sectional using regression models, etc.).
- *Sample size* – Number of sites and crashes in treatment group and comparison or reference group in all time periods analyzed.
- *Standard error* – Variability of outcome measure (i.e., the standard error, variance, or confidence interval for the CMF).
- *Potential bias* – Discussion of any potential biases to the data and how they were or were not accounted for; this may include potential spill-over or crash migration issues, traffic volume changes, regression-to-the-mean, and differences in crash reporting over time or between jurisdictions.
- *Data source* – Discussion of sources of all data and any steps and assumptions made in transforming raw data for analysis.

In the HSM, the following process is described for re-estimating reported CMFs and their standard errors for quality assessment (Gross et al., 2010; Bahar, 2010):

1. Determine estimate of safety effect of treatment as documented in respective evaluation study publication.
2. Adjust estimate of safety effect to account for potential bias from regression-to-the-mean and changes in traffic volume.
3. Determine ideal standard error of safety effect.
4. Apply method correction factor (MCF) to ideal standard error, based on evaluation study characteristics.
5. Adjust corrected standard error to account for bias from regression-to-the-mean and changes in traffic volume.
6. Combine CMFs when specific criteria are met.

Original information to be considered in the process include (Gross et al., 2010):

- Study design used to estimate CMF
- Reported CMF and its standard error

- Selection of treatment sites (i.e., if selected based on high crash counts)
- Summary of years of data used and number of observed crashes in all time periods
- Changes in traffic volume and how they were or were not accounted for

An accuracy test is used to determine if a CMF is robust enough to be included in the HSM. Accuracy tests evaluate how likely a CMF value will change if updated with information from future studies. It is also recommended that a CMF aligns with general acceptable knowledge or be reviewed by an expert panel before inclusion in the HSM (Gross et al., 2010). A sample annotated report outline for properly documenting the process of developing CMFs is presented in Figure 13.

Objective – this section should identify the treatment of interest, discuss the reason for conducting the study, and identify the target crash types and severities investigated (e.g., total crashes, injury crashes, angle crashes, etc.).

Background – this section should describe the treatment of interest, including details on its application. For example, a treatment may be applied and investigated on two-lane, undivided, rural roads. Items such as geometric characteristics are important to note so users of the CMF can determine the general applicability of the results.

Literature Review – this section should contain a summary of recent and salient literature related to the treatment of interest. This type of information is useful for comparing the consistency of results from the current study with the results of previous studies. A review of relevant literature is also useful for identifying potential variables to consider in the analysis. There are several sources for identifying CMFs from previous studies, including the CMF Clearinghouse (FHWA, 2010).

Methodology – this section should provide a discussion of the method used to develop the CMF. It is important to identify potential sources of bias in the analysis and how these biases are addressed (and those that cannot be addressed) using the selected method.

Data – this section should provide an overview of the data, including the data source(s), years of data, number of sites (and or miles of sites if applicable), average crashes per year, annual traffic volume, average traffic volume, minimum traffic volume, and maximum traffic volume. Similar to the background section, this information is useful for identifying the applicability of the CMFs developed from these data. It is also useful to provide this information for both the before and after periods when conducting a before-after study.

Results – this section should present the CMFs derived from the underlying study. It is important to include both the estimate of the CMF and the standard error. The standard error is used to calculate the confidence interval and, in general, used to judge the quality and significance of the results.

Figure 13. Sample annotated report outline

Source: (Gross et al., 2010)

2.4.7 SPF Development

Various methods exist for quantifying safety impacts at intersections. As noted, SPF is an important function in some of those estimations (Table 24). When estimating an SPF, it is important to consider the various statistical issues that can affect the results. Additionally, performing a series of trials before reaching the final estimates could be beneficial during the process. The general steps to consider when estimating SPFs include the following (Srinivasan and Bauer, 2013):

1. Determine use of SPF.
2. Identify facility type.
3. Compile necessary data.
4. Prepare and cleanup database.
5. Develop the SPF.
6. Develop the SPF for the base condition.
7. Develop CMFs for specific treatments.
8. Document the SPFs.

Table 24. Methods for Quantifying Safety Impacts

Methods for Quantifying Safety Impacts	Required Inputs			
	Applicable CMF	Applicable Crash History (Observed Crashes)	Applicable SPF (Predicted Crashes)	Engineering Judgment
Relative Comparison of CMFs	•			•
Observed Crash Frequency with CMF Adjustment	•	•		•
Predicted Crash Frequency			•	•
Predicted Crash Frequency with CMF Adjustment	•		•	•
Expected Crash Frequency		•	•	•
Expected Crash Frequency with CMF Adjustment	•	•	•	•

Source: (FHWA, n. d.)

Many advanced model forms in addition to the simple regression models are used to generate SPFs, such as generalized additive models, random-parameters models, and Bayesian Estimation Methods (Srinivasan and Bauer, 2013). In developing an SPF, the following elements need to be specified (Srinivasan and Bauer, 2013):

- Crash type(s)/severity(s) for which SPF estimated
- Total number of crashes (by type and severity) used in estimation
- Purpose of SPF (e.g., network screening, project level analysis, CMF development, etc.).
- State(s)/county(s)/city(s) used
- Facility type (e.g., rural 2-lane, 3-leg stop-controlled intersection, freeway-to-freeway exit ramp)
- Number of years used in estimation of SPF.
- Number of units (segments, intersections, ramps
- Minimum, maximum, and average length of segments
- Minimum, maximum, and average AADT

- Minimum, maximum, and average values for key explanatory variables
- Coefficient estimates of SPF
- Standard errors of coefficient estimates
- Goodness-of-fit statistics
- Discussion of potential biases or pitfalls

As noted, SFP can be calibrated to adjust for geographic or jurisdiction variations. Required data and variables in the calibration process for urban and suburban arterials are exemplified in Table 25. The calibration equation is of as follows (Gattis et al., 2017):

$$C_x = \sum \ all\ sites\ N_{observed} / \sum \ all\ sites\ N_{predicted} \tag{16}$$

Where C_x is the calibration factor, $N_{observed}$ is the observed crash frequency of each site, and $N_{predicted}$ is the unadjusted predicted crash frequency of each site.

Table 25. Data Elements Required or Desirable for Development of Calibration Factors for Urban and Suburban Arterials

For each site (i.e., intersection or roadway segment) historic data for a given time period (1-3 years)			
Facilities and HSM Prediction Models	*Crash Data*	*Traffic volume (veh/day)*	*Geometric and Traffic Management Data (numbering coincides with CMF1 in 2010 HSM; * **desirable**)*
Urban and suburban arterial two-lane undivided (2U) or three-lane including a TWLTL (3T) or four-lane undivided (4U) or four-lane divided (4D) or five-lane including a TWLTL (5T) - Roadway Segments	Observed crash frequency by severity for each year	Annual average daily traffic (AADT) for both traffic directions combined	Roadway segment length (mile) 1.1 On-street parking (presence or absence; if present, which proportion of curb length with street parking for both sides of roadway) 1.2 On-street parking type (angle or parallel, one or both sides or roadway by area type: residential/ industrial/commercial/institutional, other) 2.Roadside fixed object (presence or absence for 4 inches in diameter and not breakaway; if present, fixed object density on the right side of the roadway: fixed objects/mile; average offset from edge of traveled way (ft), and proportion of fixed object collisions of total crashes) * 3.Median width (ft – for divided roadway segments with traversable medians – not applicable to TWLTL) 4.Lighting (presence or absence; if present, proportion of total nighttime crashes by severity level for unlit roadway segments, and proportion of crashes that occur at night at unlit roadway segments) * 5.Automated speed enforcement (presence or absence) * Note: Data are also required for the use of SPFs: a)Driveway by type (major or minor commercial, major or minor industrial/institutional, major or minor residential, other) b)Posted speed limit

Table 25. Data Elements Required/Desirable for Development of Calibration Factors for Urban and Suburban Arterials, Continued

For each site (i.e., intersection or roadway segment) historic data for a given time period (1-3 years)(Cont')			
Facilities and HSM Prediction Models	*Crash Data*	*Traffic volume (veh/day)*	*Geometric and Traffic Management Data (numbering coincides with CMF₁ in 2010 HSM; * desirable)*
Urban and suburban arterial three-leg (3ST) <u>or</u> four-leg (4ST) STOP controlled on the minor-road approaches <u>or</u> three-leg (3SG) <u>or</u> four-leg (4SG) signalized intersections	Observed crash frequency by severity for each year	Annual average daily traffic ($AADT_{maj}$ and $AADT_{min}$) (i.e., the larger of the two AADTs for major road approaches and the larger of the two AADTs for minor road approaches; for 3ST and 3SG, $AADT_{min}$ of the single minor road leg	1. Intersection left-turn lanes (presence or absence on each approach for signalized intersections but only on major uncontrolled road approaches for STOP-controlled intersections) 2.Intersection left-turn phasing (permissive, protected, protected/permissive, or permissive/protected) 3.Intersection right-turn lanes (presence or absence on each approach for signalized intersections but only on major uncontrolled road approaches for STOP-controlled intersections) 4.Right-turn-on-red (number of signalized intersection approaches for which right-turn-on-red is prohibited) 5.Lighting (presence or absence; if present, the proportion of crashes that occur at night for unlit sites by intersection type based on intersection calibration samples or all jurisdiction's intersections, if available) 6.Red-light cameras (presence or absence; if present, proportion of multiple-vehicle right-angle collisions by severity level; proportion of multiple-vehicle rear-end collisions by severity level based on selected intersections for the RLC program or all jurisdiction's signalized intersections)

Table 25. Data Elements Required/Desirable for Development of Calibration Factors for Urban and Suburban Arterials, Continued

For each site (i.e., intersection or roadway segment) historic data for a given time period (3-5 years)(Cont')			
Facilities and HSM Prediction Models	*Crash Data*	*Traffic volume (veh/day)*	*Geometric, Traffic Management, and Adjacent Land Use Data (numbering coincides with CMF₁ in 2010 HSM; * **desirable**)*
Cont'			These attributes are required only for vehicle-pedestrian collisions at signalized intersections 1p)Bus stops (presence or absence within 1000 ft of center of intersection; if present, number of bus stops) * 2p)Schools(presence or absence within 1000 ft of center of intersection) * 3p)Alcohol sales establishment s ((presence or absence within 1000 ft of center of intersection; if present, number of establishments) * Note: Data are also required for the use of SPFs: a)maximum number of lanes to be crossed by a pedestrian in any crossing maneuver at signalized intersection * b)pedestrian daily total volume at signalized intersection *

Source: (Bahar and Hauer, 2014)

2.5 Potential Data Sources in Florida and Other States

Several data sources and applications exist in Florida and across the U.S. for the evaluation of safety and operation of roadway networks and provide crash and traffic information. This section provides data sources in Florida and across U.S. along with associated variables and links (Table 26). Additional details on the data sources are provided for Florida and the nation separately.

Table 26. Safety and Traffic Data Sources

Data Source	Coverage	Owner/ Authorization	Data Type	Variables	Public	Free	Link
Florida Crash Dashboard	FL	FLHSMV (Florida Department of Highway Safety and Motor Vehicles)	Safety	Crash details	Yes	Yes	https://www.flhsmv.gov/traffic-crash-reports/crash-dashboard/
Crash and Citation Reports & Statistics	FL	FLHSMV	Safety	Crash and Citation Report & Statistics	Yes	Yes	https://www.flhsmv.gov/resources/crash-citation-reports/
CAR (Crash Analysis Reporting) system	FL	FDOT	Safety	Crash details	No	Yes	https://fdotwp2.dot.state.fl.us/CrashAnalysisReporting/
SSOGis (State Safety Office Geographic Information System)	FL	FDOT	Safety	Year, date, time of crashes, crash type and severity, crash location details	Yes	Yes	https://fdotewp1.dot.state.fl.us/SSOGis/Home.aspx
Signal Four Analytics	FL	GeoPlan Center at UF (University of Florida)	Safety	Year, date, time of crashes, crash type and severity, crash location details, and other crash information	No	Yes	https://s4.geoplan.ufl.edu/
Florida Open Data Hub	FL	FDOT	Safety and traffic	Crash location, type, and other information, traffic volumes by types and signal locations	Yes	Yes	https://gis-fdot.opendata.arcgis.com/
Regional Integrated Transportation Information System (RITIS)	FL	CATT Lab	Traffic and roadway info.	Traffic volume, speed, class, and occupancy from sensors (loops, RTMS, Video detection, Sensys pucks, etc.), location of collisions, signal timing plans, etc.	No	Yes	https://www.ritis.org/login?r=Lw==

Table 26. Safety and Traffic Data Sources, Continued

Data Source	Coverage	Owner/ Authorization	Data Type	Variables	Public	Free	Link
FDOT Roadway Characteristics Inventory (RCI)	FL	FDOT	Traffic	Roadway location, classification, and characteristics, traffic volume, and traffic control device inventory		Yes	https://www.fdot.gov/docs/default-source/statistics/multimodaldata/multimodal/Roadway-Characteristic-Inventory-(RCI).pdf
NHTSA Fatality Analysis Reporting System (FARS)	US	NHTSA	Safety	Crash year, date, type, severity, other information	Yes	Yes	https://www.nhtsa.gov/data
Highway Safety Information System (HSIS)	US	FHWA	Safety and traffic	Fatal and injuries crashes	Yes	Yes	https://www.hsisinfo.org/
Waze	US	Waze	Traffic	Congestion and collision information			https://www.waze.com/ccp
INRIX	US	Analytics Company	Traffic	Historical and real-time speed and travel time	No	No	https://inrix.com/
ATR Data	US	Transportation Planning and Programming Division (TPP)	Traffic	24-hour traffic volumes	Yes	Yes	https://catalog.data.gov/dataset/automatic-traffic-recorder-atr-stations or http://onlinemanuals.txdot.gov/txdotmanuals/tda/automatic_traffic_recorder_volume_data.htm

2.5.1 Florida Data Sources

FDOT has a Traffic Safety portal that includes several safety data sources (FDOT, 2020a) such as information on specific crash types and pedestrian and bicycle crashes. Geographic Information System (GIS) safety data are also accessible on the portal. Although much information is available on the portal, the four most common crash data sources in Florida are as follows (FDOT, 2020a):

- FIRES (Florida's Integrated Report Exchange System) by FL DHSMV
- CAR (Crash Analysis Reporting) system by FDOT
- SSOGis (State Safety Office GIS) web-based map by FDOT
- Signal Four Analytics by the University of Florida GeoPlan Center with Florida TRCC (Traffic Records Coordinating Committee)

The Florida Open Data Hub and FDOT's Regional Integrated Transportation Information System (RITIS) also include traffic and roadway information in Florida (FDOT, 2020b; FDOT, 2020C). The FDOT Roadway Characteristics Inventory (RCI) is another data source that includes the following information (TRCC, n.d.; FDOT TDAO, 2018):

- Roadway location
- Classification (freeway, secondary road, arterial, etc.)
- Physical characteristics (two lane, multi-lane, shoulders, etc.)
- Traffic information (volume)
- Traffic control device inventory

FDOT's Transportation Data and Analytics Office has many shapefiles of traffic data that are accessible upon request, and signal timing and retiming information is available from various signalization projects. These data often are available by jurisdiction.

2.5.2 Other Data Sources

Several national safety and traffic data sources are available. For example, data files coded from police accident reports from 32 states are available through the National Highway Traffic Safety Administration (NHTSA, n.d.). The Highway Safety Information System (HSIS) is another database funded by FHWA that includes safety and traffic information from many states (HSIS, n.d.). FARS created by NHTSA, a nationwide source of crash data, comprises fatal injuries suffered in motor vehicle traffic crashes (NHTSA, n.d.). Private sources of traffic data also exist, including Waze (see Table 26).

3 Development of Experiment Design, Data Collection Plan, and Analysis Methodology

This chapter covers the evaluation plan for the development of CMFs for speed management and pedestrian safety using signal progression strategies. The experiment plan includes study design, data collection plan, and analysis methodology.

3.1 Study Design

The study design also elaborates on potential challenges and solutions.

3.1.1 Potential Challenges

Development of CMFs requires a data-driven approach, and data availability determines the study design. Due to the characteristics of pedestrian crashes and new traffic signal progression strategies, this study faced the following challenges:

- *Number of treated sites* – Traffic signal progression for speed management is an innovative strategy; most traffic signal progression projects aim to increase progression speed rather than speed management. Limited treated sites—corridors with traffic signal progression for speed management—can be identified. The small sample of treated sites (< 10) may not satisfy the data needs in the before-after study.
- *After observation period* – Identified traffic signal progression projects for speed management were implemented one or two years ago. No sufficient observation years (usually require ≥ 3 years) for the "after" stage (with the new strategy) are available on the identified treated sites.
- *Low-mean crash sample* – Pedestrian and bicyclist crashes are rare and random events, and most roadway segments experience very few or even zero observations of pedestrian and bicycle crashes. Modeling based on the low-mean sample, in either the before-after study or the cross-sectional study, may result in a biased and inefficient inference.
- *Selection bias* – Traffic agencies usually select sites with highly historical pedestrian and bicycle crashes for implementing speed management treatments. As the crash number naturally tends to be reduced after a year with high crash records (regression-to-mean issues), the non-random site selection may result in overestimating the effectiveness of the treatment.

3.1.2 Solutions

The research team adopted the following solutions to address the challenges:

- *Observation years* – Projects were selected that were implemented several years ago so there were enough observation years for collecting "after" crashes.
- *Random site selection* – To avoid biased site selection, study sites were randomly selected, including treated sites and untreated sites. Sufficient study sites were used to

develop pedestrian crash prediction model (SPFs). Through the SPFs, the research team identified contributing factors and accounted for the influence of unnecessary factors in the study.

- *Model Selection* – Advanced statistical modeling methods were used to address several previously highlighted challenges.

3.1.3 Study Design

A before-after study and a cross-sectional study are the two major experiment designs in CMF development. A before-after study compares crash frequencies at the same sites before and after implementing a treatment. The Empirical Bayesian (EB) before-after is the preferred approach because this method can effectively address the regression-to-the-mean issue (HSM, 2009; Gross et al., 2010). The EB before-after method estimates the expected crashes for individual treated sites in the "after" period assuming no treatment is applied. An SPF, as a function of traffic and physical characteristics of the sites (Gross et al., 2010), is required for the prediction. Additionally, the method also requires sufficient crash observations in the after stage.

A cross-sectional study is applicable if enough treated sites and after-crash observations are unavailable. A cross-sectional study collects crash data from randomly selected study sites with various traffic signal progression treatments. A multivariate prediction model is developed based on cross-sectional data, and the CMF for traffic signal progression is derived based on the associated coefficients in the prediction model. The cross-sectional study needs enough study sites and sufficient pedestrian crashes on each individual site.

The project team also considered the case-control method with cross-sectional data to address the potential low sample mean issue (many sites with zero pedestrian crashes). The case-control design is based on cross-sectional data and randomly matches one case (sites with pedestrian crashes) with multiple controls (sites without pedestrian crashes). This matching strategy not only controls the number of cases but also eliminates the influence of confounders connecting both crashes and the factors of interest. However, the case control method, which determines the relative crash risk ratio (odds ratio) due to a factor change, cannot be used to predict crash frequency.

The flow chart for study design and model selection for this project is shown in Figure 14.

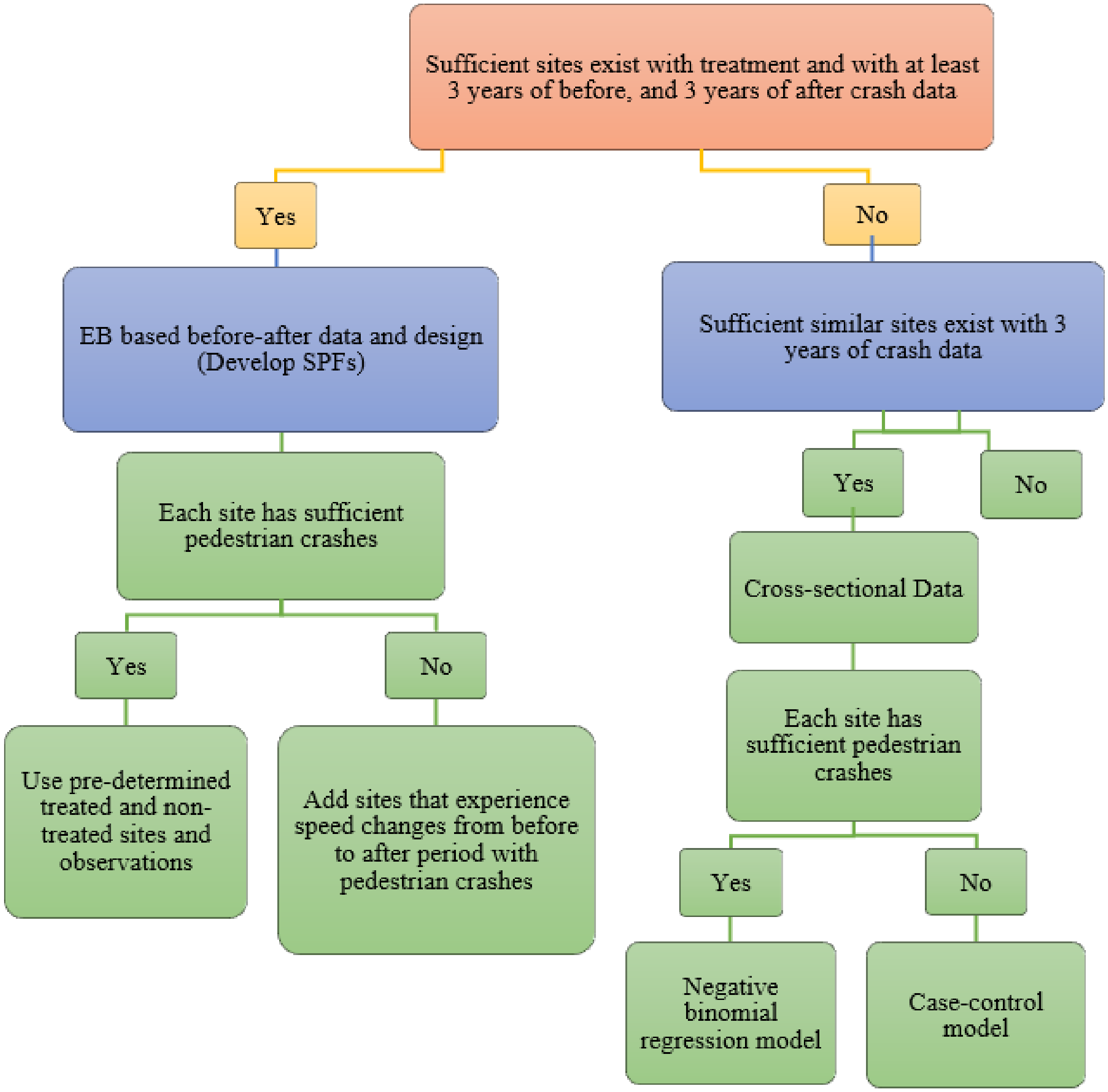


Figure 14. Flow chart for study design and model selection

3.2 Data Collection Plan

3.2.1 Required Data

This section describes the data collection plans and covers study sites, necessary data fields, sample size requirements, data sources, and data collection methods. Several data sources and applications exist in Florida. The required data, including category, variables, and sources in the study are provided in Table 27.

Table 27. Data for Collection and Potential Sources

Data Category	Variables	Potential Sources
Historical crash data	• Pedestrian and bicycle crashes for 5 years or more • Crash severity, date/time, location, …	• Signal Four Analytics • CARS [CAR (Crash Analysis Reporting) system]
Traffic signal timing	• Retiming date and time • Retiming sheets • Traffic signal coordination plan, • Time-space diagrams (progression speeds more than 15%; 10%–15%, 5%–10%, 0%–5% below posted speed limit; 0%–5%, 5%–10%, 10%–15%, or more than 15% above posted speed limit)	• Traffic signal retiming reports • Local transportation agencies • Consulting companies
Speed	• Progression speed	• Traffic signal retiming reports • RITIS Database • Simulation
Roadway	• Roadway functional classification • Lane configuration • Number of signals • Number of unsignalized intersections • Median and shoulder design • Traffic controls, such as speed limit • Other geometric data	• FDOT RCI database • FDOT GIS layers • Google Maps • Traffic signal retiming reports
Traffic	• AADT • Turning movements (if available) • K, D, T factors	• FDOT traffic information database • Traffic signal retiming reports

3.2.2 Data Collection Process and Methods

The process and method of intensive data collection from the research team is illustrated in Figure 15. Traffic signal retiming project reports were reviewed to identify treated sites, including speed management projects and projects without speed management. Many signal retiming projects without a specification for speed management showed speed management characteristics on some coordinated timing plans. The needed data, such as retiming date, spatial scope, speed analysis, turning movements, retiming sheets, time-space diagrams and revel time diagrams, etc., were retrieved from the reports for each site. Historical pedestrian/bicycle crash data, geometric data, and traffic data were collected for each site based on time and location information (Roadway ID + milepost or GIS coordinates). The collected data were imported into a project database for processing and analysis.

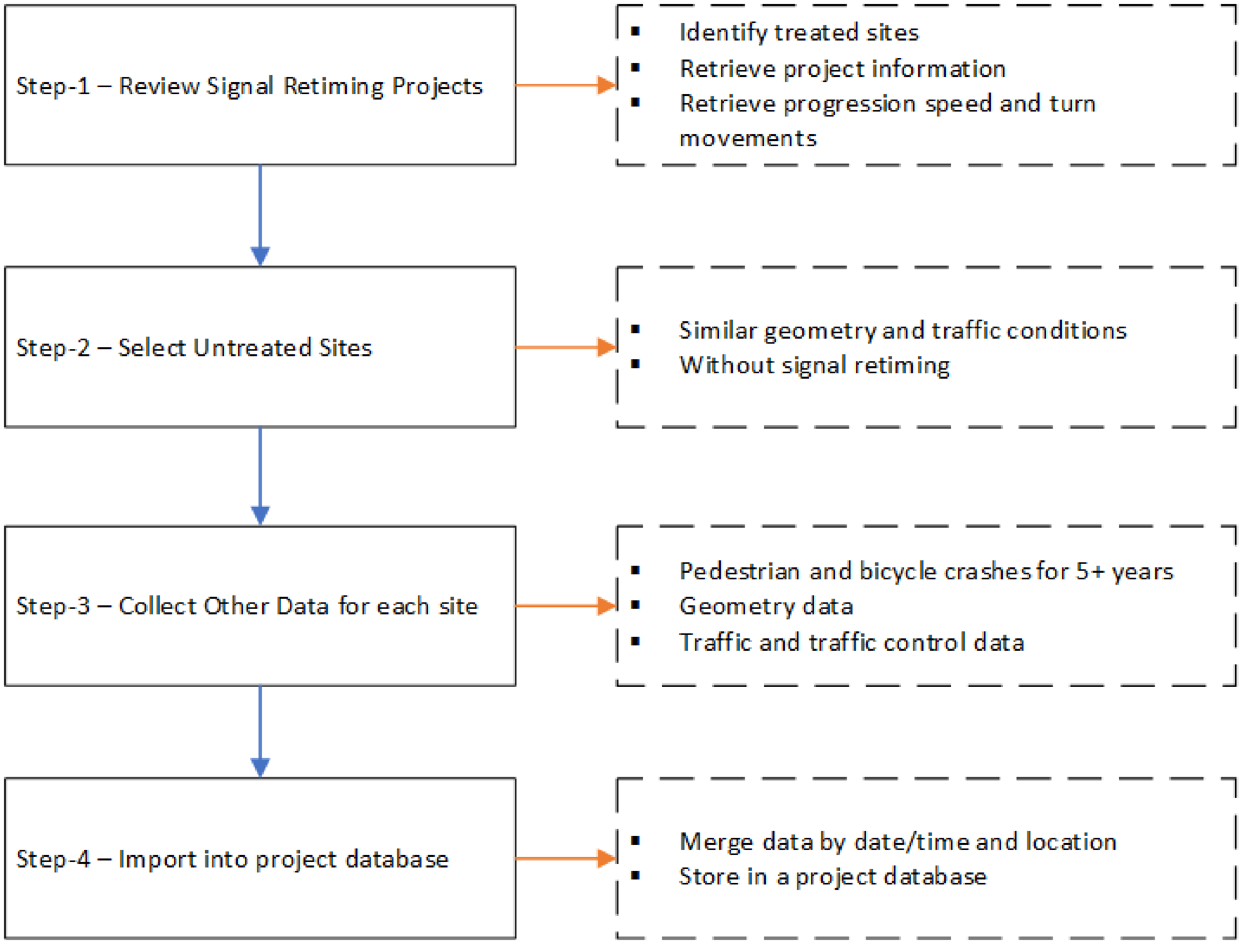


Figure 15. Data collection process and method

3.3 Analysis Methodology

The project team adopted proper statistical methods to develop CMFs based on the collected data. Qualitative and quantitative analyses were conducted to evaluate the effectiveness of traffic signal progression for managing speed and improving pedestrian safety on corridors. The analysis procedure comprised data assembly, descriptive analysis, modeling, assessment of results, and performance measures. A flow chart illustrating the data analysis procedure is provided in Figure 16.

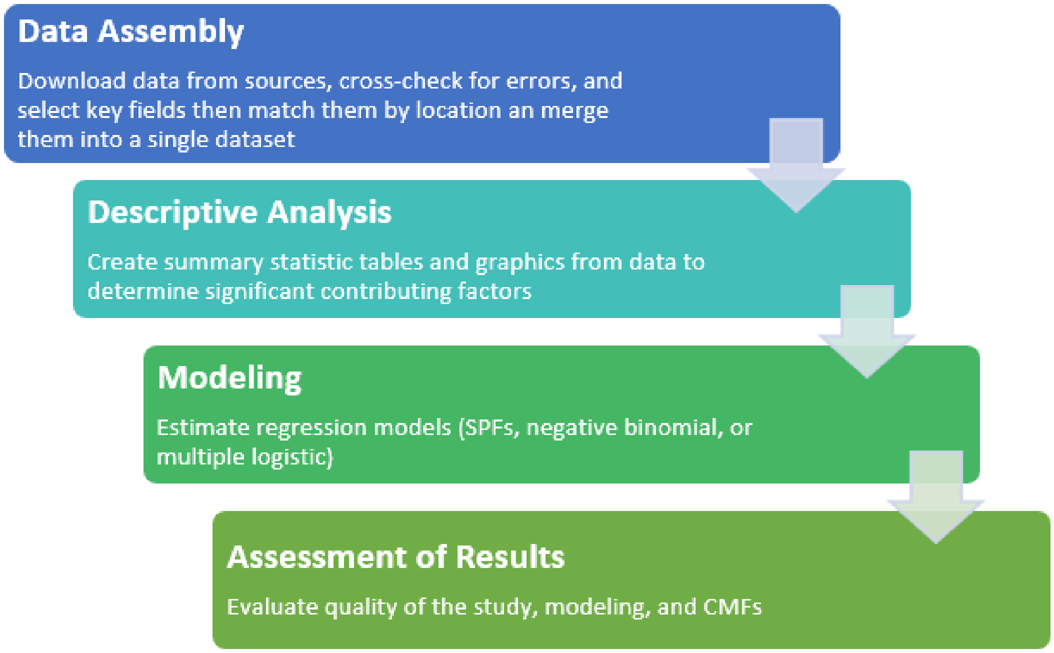


Figure 16. Flow chart of data analysis procedure

3.3.1 Data Assembly

Data were downloaded from the identified data sources and cross-checked for potential errors. In addition to current data, historical data were collected for some data types, such as crash and volume data. Key data fields from each source were retrieved for the desirable number of years. The collection of data fields was matched by locations and merged into a single dataset to prepare the stage for the descriptive analysis and the modeling stage.

3.3.2 Descriptive Analysis

The research team estimated descriptive statistics to describe the basic features of the data and to provide summaries about the sample and the measures. Tables and graphics were provided as the descriptive statistics of the collected data. The results assisted the research team understand the data characteristics and identify the potential contributing factors.

3.3.3 Modeling

The research team selected proper approaches, as described in Figure 2, to model the collected data. The modeling results included:

- SPFs for predicting pedestrian and bicycle crashes based on signal progression indicator, progression speed, geometry, and traffic information.
- CMFs to quantify the safety effects of signal progression strategies on pedestrians and bicyclists.
- Factors that significantly contribute to pedestrian and bicyclist crashes on coordinated urban corridors.

The three considered statistical modeling technologies are synthesized in Table 28.

Table 28. Potential Modeling Technologies

Design Type	Model Type
EB before-after	SPF models
Cross-sectional	Negative binomial regression model
Case-control	Conditional logistic regression model

EB Before-After Model (SPFs)

For the EB before-after model, the expected number of crashes in the before period was calculated as a function of observed number of crashes and predicted crashes for the same period. An SPF was employed to estimate predicted crashes in the before period as a function of traffic and physical characteristics of the sites (Gross et al., 2010). Equations (Eq. 17–21) for the EB before-after approach include (Gross et al., 2010):

$$Crashes\ per\ year = \alpha(Major\ road\ entering\ AADT)^{\beta_1}(Minor\ road\ entering\ AADT)^{\beta_2} \tag{17}$$

$$\mathrm{N_{expected,T,B}} = \mathrm{SPF_{weight}}\left(\mathrm{N_{predicted,T,B}}\right) + (1 - \mathrm{SPF_{weight}})(\mathrm{N_{observed,T,B}}) \tag{18}$$

$$\mathrm{N_{expected,T,A}} = \mathrm{N_{expected,T,B}}\left(\frac{\mathrm{N_{predicted,T,A}}}{\mathrm{N_{predicted,T,B}}}\right) \tag{19}$$

$$\mathrm{Var}\left(\mathrm{N_{expected,T,A}}\right) = \mathrm{N_{expected,T,A}}\left(\frac{\mathrm{N_{predicted,T,A}}}{\mathrm{N_{predicted,T,B}}}\right)(1 - \mathrm{SPF_{weight}}) \tag{20}$$

$$\mathrm{CMF} = \frac{\left(\frac{\mathrm{N_{observed,T,A}}}{\mathrm{N_{expected,T,A}}}\right)}{\left(1+\left(\frac{\mathrm{Var(N_{expected,T,A})}}{\mathrm{N_{expected,T,A}}^2}\right)\right)} \tag{21}$$

Where,

AADT = annual average daily traffic volume

α, β, β1, and β2 = numbers estimated during SPF development

$N_{observed,T,B}$ = observed number of crashes in before period for treatment group

$N_{observed,T,A}$ = observed number of crashes in after period for treatment group

$N_{predicted,T,B}$ = predicted number of crashes (i.e., sum of SPF estimates) in before period

$N_{predicted,T,A}$ = predicted number of crashes (i.e., sum of SPF estimates) in after period

$N_{expected,T,B}$ = unadjusted Empirical Bayes estimate

$N_{expected,T,A}$ = expected number of crashes for treatment group that would have occurred in after period without treatment

$Var(N_{expected,T,A})$ = variance of expected number of crashes for treatment group that would have occurred in after period without treatment

CMF = crash modification factor

SPF weight = weight derived using the over-dispersion parameter from the SPF calibration process, but also depends on the number of years of crash data in the period before treatment

Cross-Sectional Negative Binomial Model

For cross-sectional study, if each site has sufficient pedestrian crashes, a negative binomial regression model is estimated. Negative binomial regression is employed to model count variables (especially over-dispersed or under-dispersed count outcome variables). The model is of the following forms (see Eq. 22-23) (NCSS, n.d.):

$$P_r(Y = y_i|\mu_i, \alpha) = \frac{\Gamma(y_i+\sigma^{-1})}{\Gamma(\sigma^{-1})\Gamma(y_i+1)}\left(\frac{1}{1+\sigma\mu_i}\right)^{\sigma^{-1}}\left(\frac{\sigma\mu_i}{1+\sigma\mu_i}\right)^{y^i} \quad (22)$$

$$\mu_i = \exp\,(\ln(t_i) + \beta_1 x_{1i} + \beta_2 x_{2i} + \cdots + \beta_k x_{ki} \quad (23)$$

Where,

μ = mean incidence rate of y per unit of exposure (volume, crash)

t_i = exposure for a particular observation; when no exposure given, it is assumed to be 1

β1, β2, …, βk = regression coefficients or unknown parameters estimated from a set of data

Case-Control Conditional Logit Model

The project team considered a case-control model with cross-sectional data to address the low sample mean issue (many sites with zero pedestrian crashes). For the case-control study, a conditional logistic regression model was estimated. Conditional logistic regression is a particular kind of logistic regression used when case subjects with a particular condition or attribute are each matched with n control subjects without the condition. For S strata (matched sets) and p independent variables (x’s), the equation for the conditional logistic regression model is the following (see Eq. 24) (NCSS, n.d.):

$$logit(p) = \alpha_1 + \alpha_2 z_2 + \cdots + \alpha_s z_s + \beta_1 x_1 + \cdots + \beta_p x_p \quad (24)$$

Where,

z’s = binary indicator variables for each stratum (only S – 1 z variables needed)
α’s = regression coefficients associated with stratum indicator variables
x’s = covariates
β’s = population regression coefficients to be estimated

3.3.4 Assessment of Results

This step involved assessing the quality of CMFs obtained from the selected design and modeling technique. During that stage, the CUTR team checked the following questions:

- Is the direction of effect (i.e., expected decrease or increase) in crashes in accordance with expectations?
- Does the magnitude of the effect seem reasonable?
- Are the parameters of the model estimated with statistical significance?
- Do different studies come to similar conclusions?
- What is the variability of the outcome measure (i.e., standard error, variance, or confidence interval for CMF)?
- What are some potential biases? Discussion of potential biases to the data and how they were or were not accounted for; this may include potential spill-over or crash migration issues, traffic volume changes, regression-to-the-mean, and differences in crash reporting over time or between jurisdictions.

4 Collection and Processing of Data

4.1 Site Selection

The CUTR team used the retiming corridors provided by Iteris as study sites, which covered five main counties in Florida—Citrus, Hernando, Hillsborough, Pasco, and Pinellas. The retiming projects were completed from 2014 to 2020 The project roadways are shown in Figure 17.

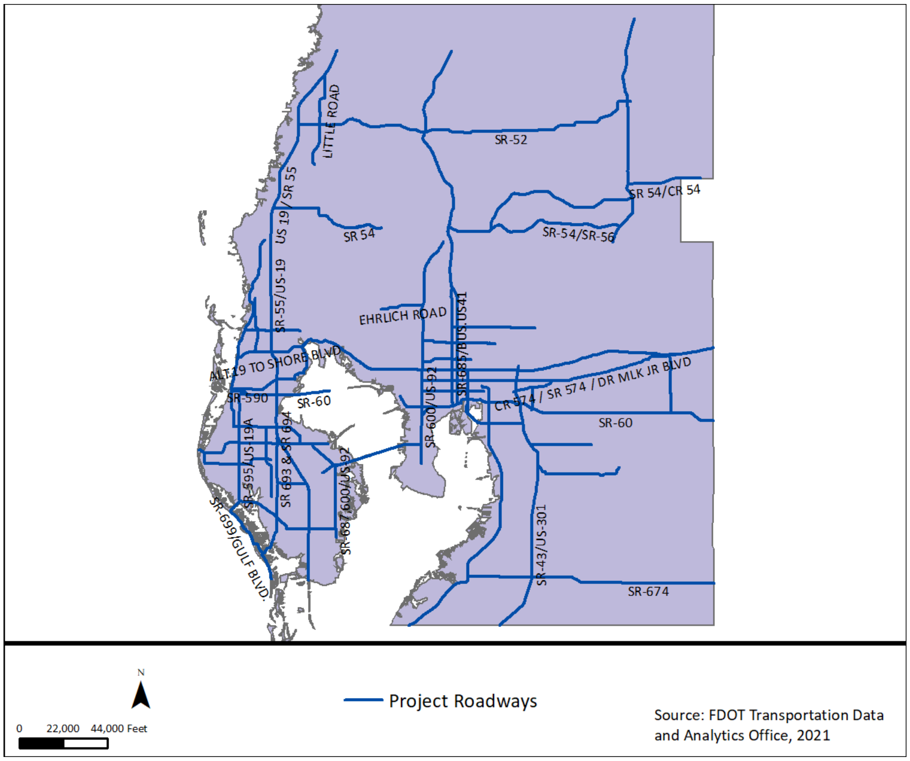


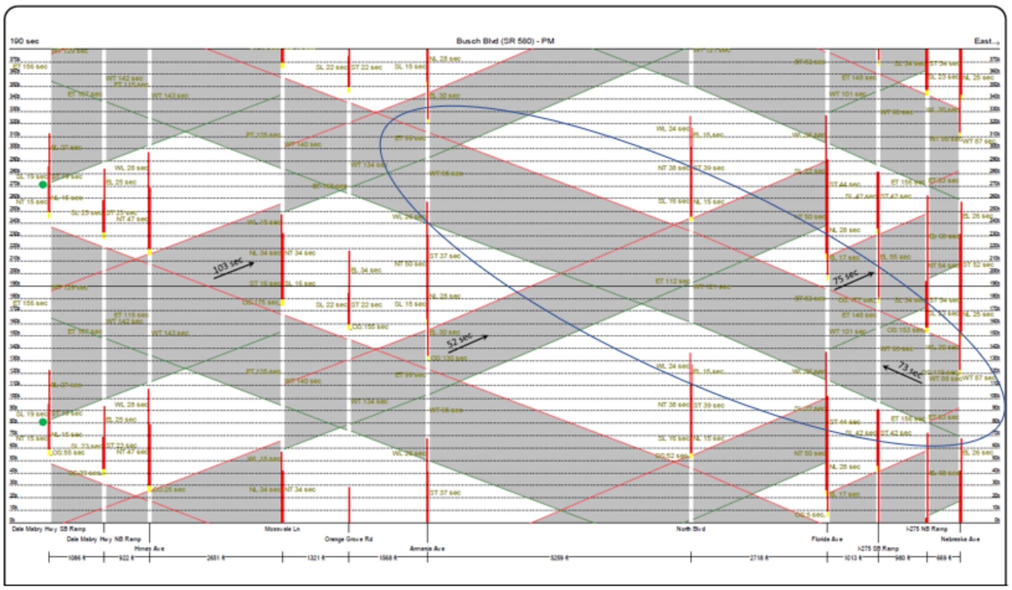
Figure 17. Project roadways

According to Iteris (2020), the three retiming projects with speed management as part of the goals were as follows:

- Busch Boulevard
- Florida Avenue and Tampa Street as part of Downtown Tampa retiming
- 56th Street between Netpark and Whiteway Drive

The three speed management projects were completed or anticipated to be complete in 2020 and beyond. The CUTR team also identified several additional and potential corridors with speed management among the retiming project list by reviewing progression speed, time-space diagrams, and other information from the final reports. It is important to note that speed management treatment can be applied during selected patterns (schedules) (AM Peak, MD Peak, PM Peak, Weekend AM Peak, Weekend MD Peak, Weekend PM Peak, etc.) or selected segments or directions of the same corridors. For example, Figure 18 illustrates how part of the same corridor can be with or without speed management by using a few Busch Boulevard time-space diagrams and how the speed management treatment can vary by patterns or schedules for the same corridor. As circled in blue in Figure 18, the research team considered a speed management treatment from time-space diagrams when red phases of traffic signals downstream in the bandwidth along the same direction encourage drivers in the front of a platoon to follow a designed progression speed and discourage them to speed up. It was deemed no speed management if the drivers see green phases downstream and are motivated to speed up in to pass through those intersections.

Speed Management (Westbound)



No Speed Management

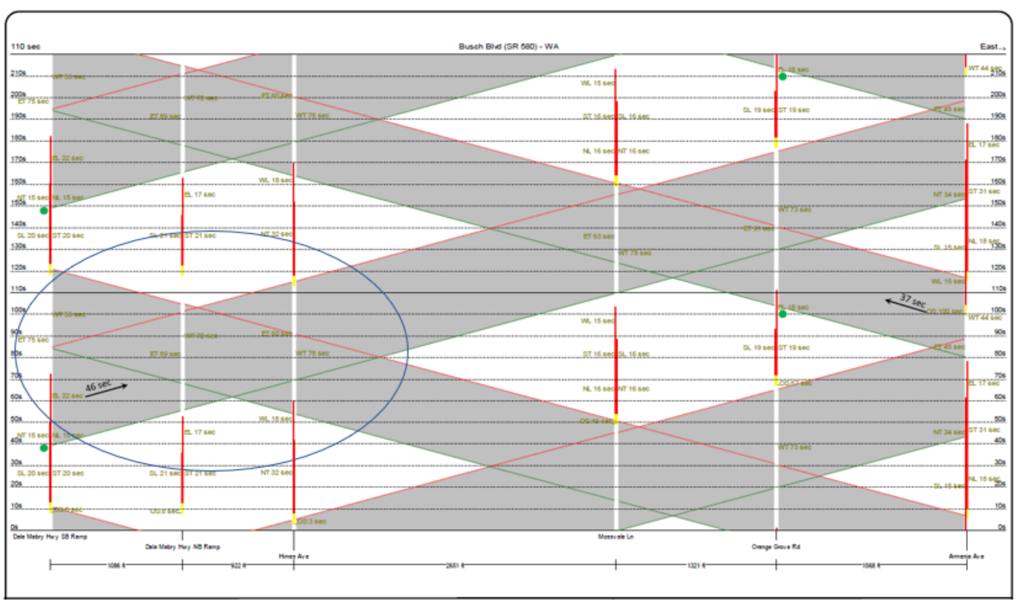


Figure 18. Example time-space diagrams with and without speed management treatments

Source: (Iteris, Inc., 2020)

In addition to the speed management data obtained from time-space diagrams, another important data related to speed management to be collected is the quality of a traffic signal progression. A good traffic signal progression quality is that drivers can smoothly travel from one signal to another while maintaining the same or similar speed. Their progression speeds are consistent throughout the coordinated corridor. On the other hand, it is defined as a poor progression quality if drivers in a platoon need to drive faster on some arterial segments and slower on the other segments of a coordinated corridor resulting in inconsistent progression speeds. A poor progression quality can potentially increase risks to pedestrians and bicyclists. The progression quality data can be collected from travel time study. Examples of good and poor traffic signal progression quality from travel time diagrams are provided in Figure 19.

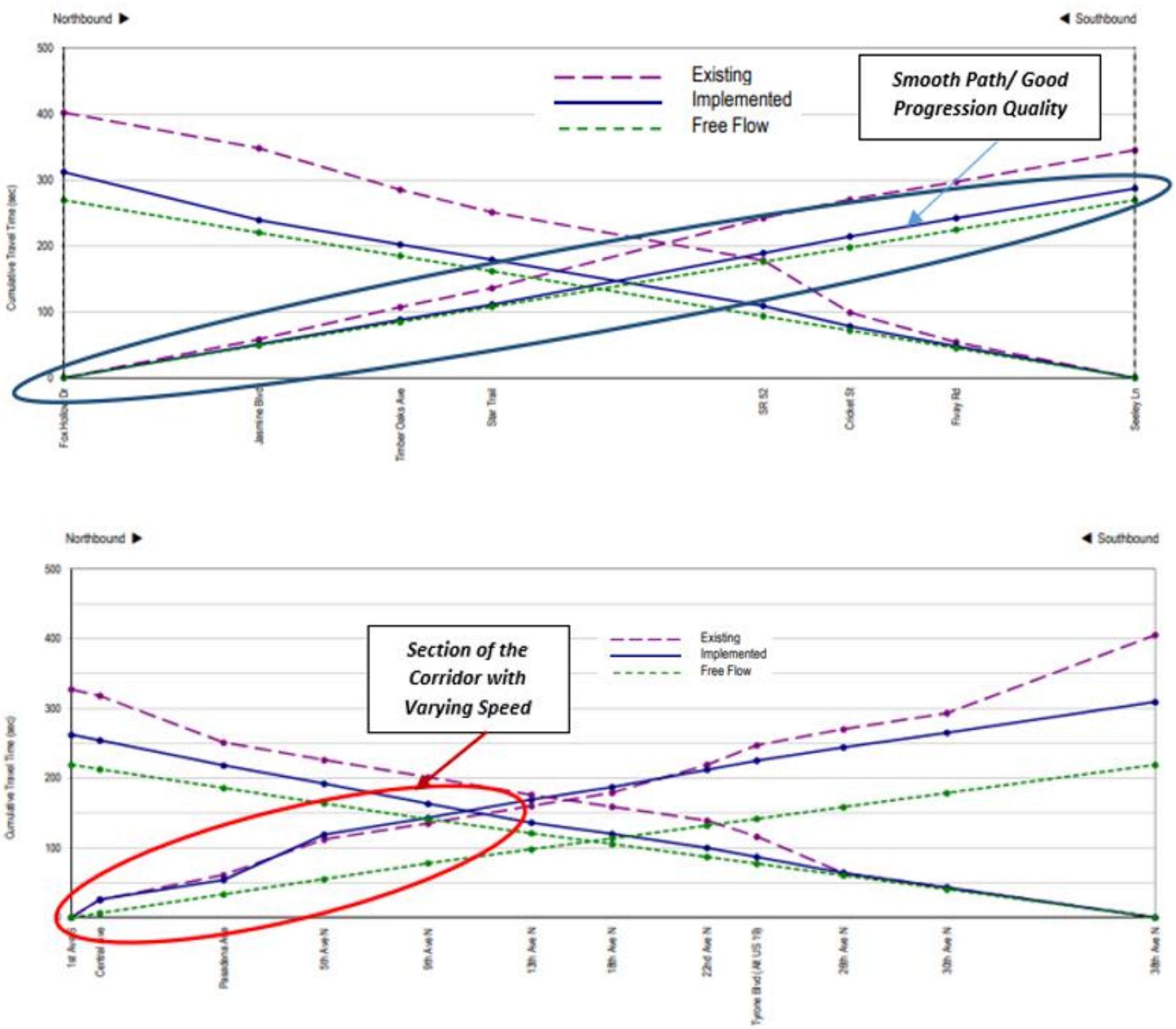


Figure 19. Examples of good and poor quality of a traffic signal progression observed from travel time diagrams

Source: (Iteris, Inc., 2020)

4.2 Data Collection

4.2.1 Site Data Collection

The CUTR team retrieved travel time runs data from retiming project final reports and converted the data into speed data. Table 29 shows an example of average travel time information used. Before and after travel times for different periods/patterns/schedules were included, such as AM Peak, MD Peak, PM Peak, PM Off-peak, Weekend AM Peak, Weekend MD Peak, Weekend PM Peak. Segment length and directions of the runs were also provided. Using segment lengths and travel times, the project team calculated the speed for each segment with travel time data. Travel time records sometimes were not available for entire corridors but only for segments of the corridors; these segments may have been the critical sections that were worth evaluating.

In addition to travel time information, roadway and traffic data were obtained from the FDOT Transportation Data and Analytics Office. As shown in Figure 20, AADT is an example of important data in the modeling downloaded from the FDOT website; traffic signal locations and information are also relevant for the upcoming analysis. Roadway functional classification, speed, and other data were also acquired from the site. A map of the roadway functional classification of the study corridors is shown in Figure 21.

Table 29. Travel Time Runs Summary

Average Total Travel Time & Delay

Busch Blvd (SR 580): 1 miles

		AM Peak		MD Peak		PM Peak		PM Off-peak		Weekend AM Peak		Weekend MD Peak		Weekend PM Peak	
		Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)	Travel Time (s)	Delay (s)
Eastbound	Existing	234	142	171	79	221	129	159	67	174	82	186	94	165	74
	Implemented	191	99	133	41	231	139	115	24	127	35	140	48	97	5
	Difference	-43		-38		10		-44		-47		-46		-68	
	% Difference	-18.4%	-30.3%	-22.2%	-48.1%	4.5%	7.8%	-27.7%	-65.7%	-27.0%	-57.3%	-24.7%	-48.9%	-41.2%	-91.9%
Westbound	Existing	263	171	142	50	206	114	153	62	147	56	275	183	211	119
	Implemented	234	142	116	24	177	85	120	28	108	16	124	32	103	12
	Difference	-29		-26		-29		-33		-39		-151		-108	
	% Difference	-11.0%	-17.0%	-18.3%	-52.0%	-14.1%	-25.4%	-21.6%	-53.2%	-26.5%	-69.6%	-54.9%	-82.5%	-51.2%	-90.8%

Eastbound : North Blvd to Nebraska Ave

Westbound : Nebraska Ave to North Blvd

Source: (Iteris, Inc., 2020)

Traffic Data - Shapefiles of GIS Traffic Data Layers:

The links below contain shapefiles of traffic count and other traffic-related data. The data is provided statewide in WinZip format and is updated weekly. The data projection is UTM 17, and the datum is NAD 83.

Shapefiles:

Annual Average Daily Traffic - 6.65 MB Zip File

Historical Annual Average Daily Traffic - 31.7 MB Zip File

Portable Traffic Monitoring Sites - 1.12 MB Zip File

Telemetered Traffic Monitoring Sites - 82.6 KB Zip File

Traffic Signal Locations - 610 KB Zip File

Metadata:

Annual Average Daily Traffic

Historical Annual Average Daily Traffic

Portable Traffic Monitoring Sites

Telemetered Traffic Monitoring Sites

Traffic Signal Locations

Figure 20. FDOT traffic GIS data

Source: (FDOT, 2019)

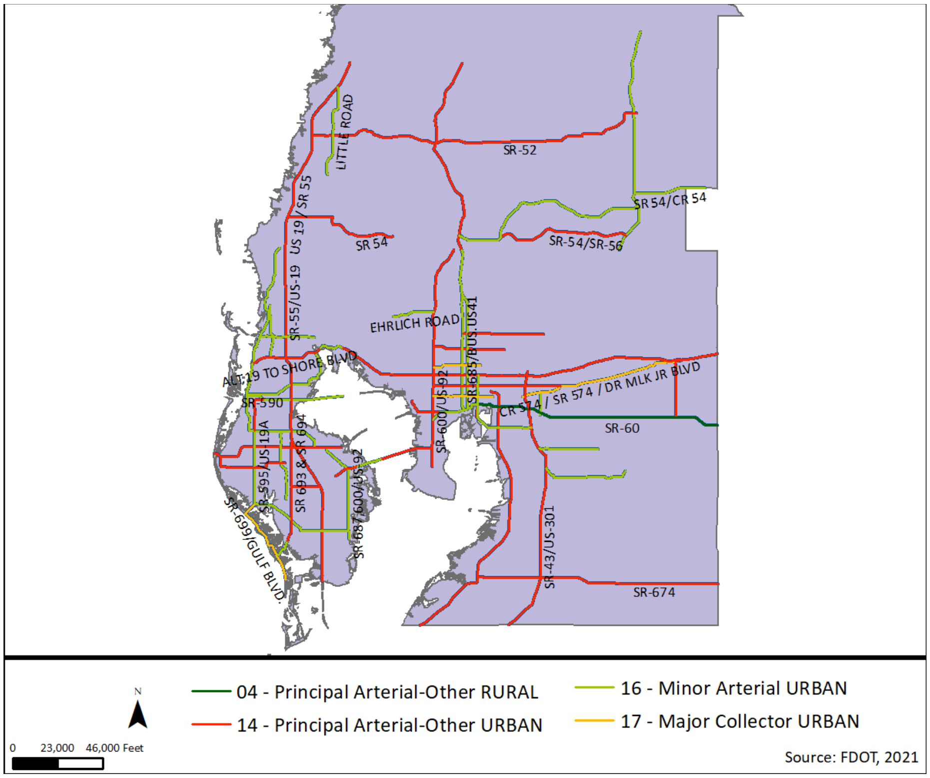


Figure 21. Study roadway functional classification

4.2.2 Crash Data Collection

Crash data, including historical crashes, were retrieved from Signal Four Analytics for crashes that occurred in 2011–2021. The search query to obtain the data is shown in Figure 22 and included the five counties noted previously and pedestrian and bicycle injuries and fatalities.

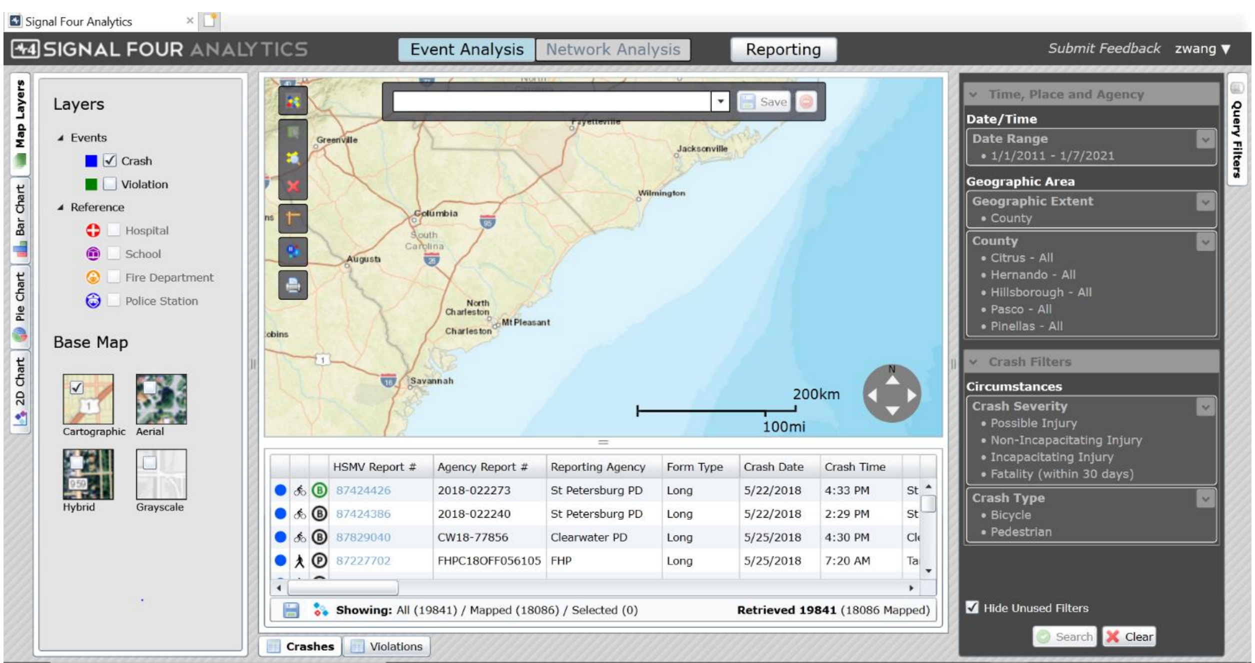


Figure 22. Signal Four Analytics search query

Pedestrian and bicycle fatalities and injuries were limited on the study roadways during the 10-year period (2011–2021), as shown in Figure 23 and Figure 24 (top maps). The CUTR team also explored the difference between crashes on the study roadways and crashes within a 0.5-mile buffer of the roadways. A spatial overview of the crashes within the buffer is provided at the bottom of Figure 23 and Figure 24.

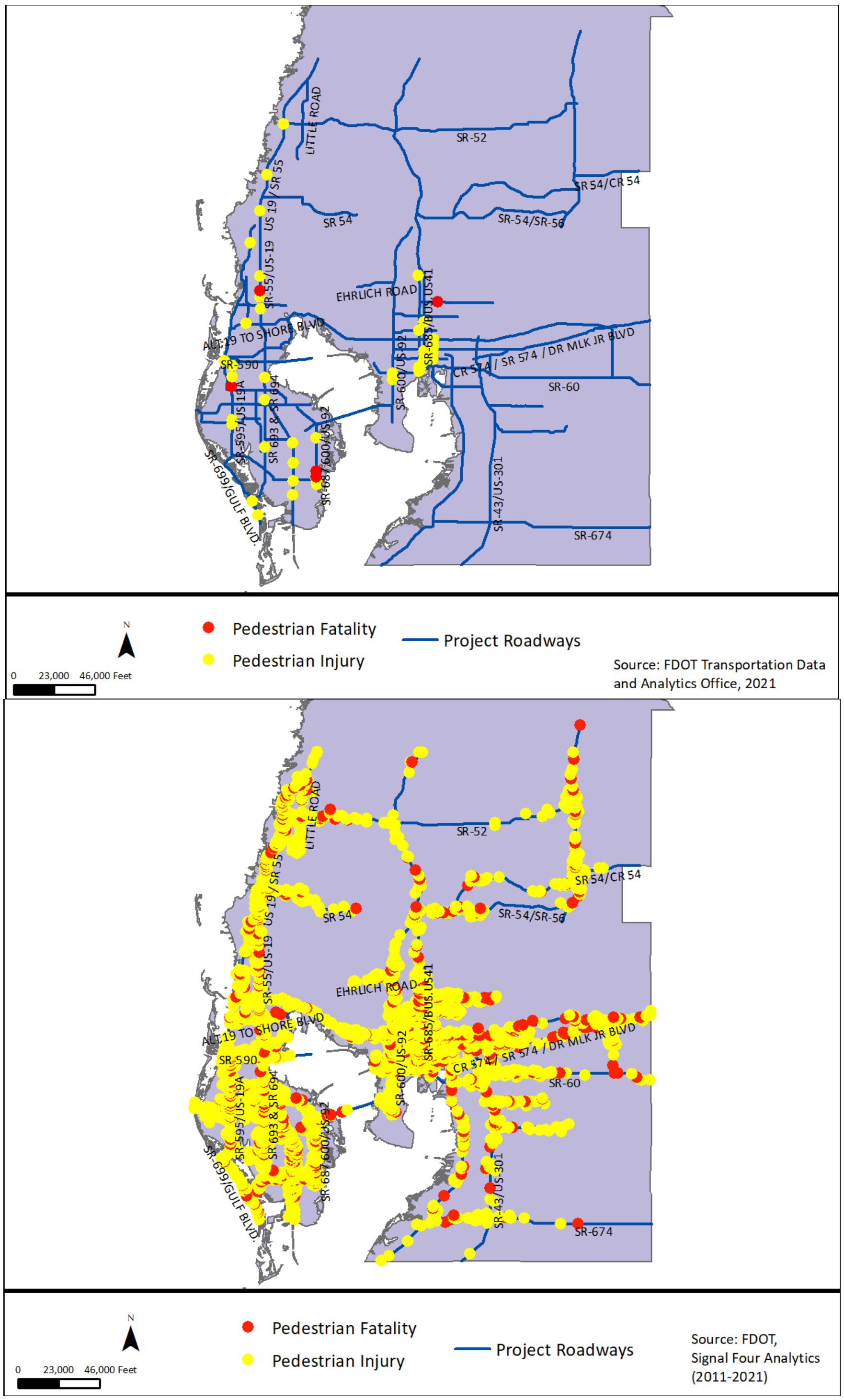


Figure 23. Study site pedestrian crashes

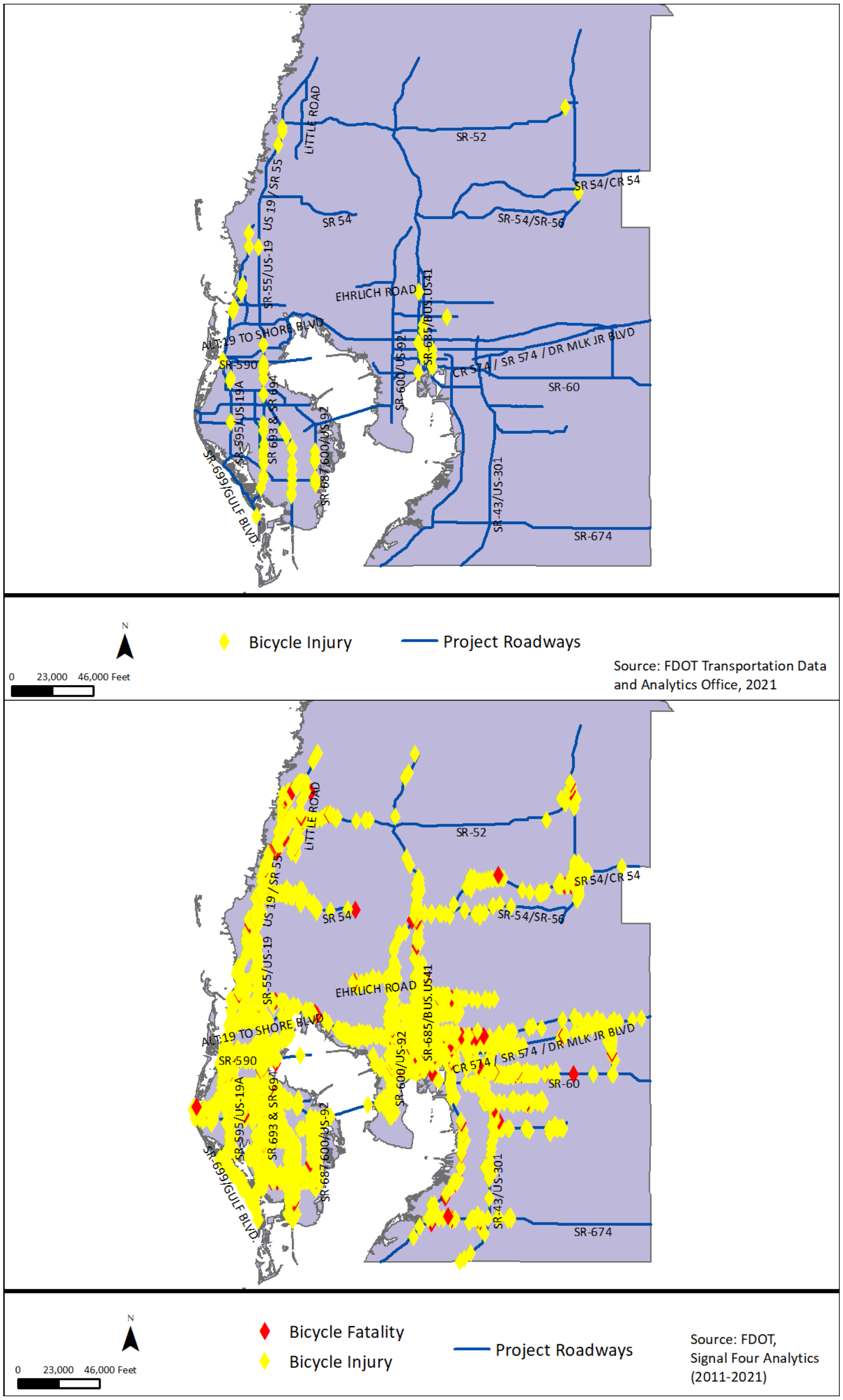


Figure 24. Study site bicycle crashes

4.3 Data Processing and Matching

The speed, roadway, and traffic datasets were selected and matched, as described in Figure 25. Data were matched spatially and temporally based on the retiming year of the selected study corridor. For example, if the retiming year was 2016, historical roadway, traffic, speed, and crash data prior to and after 2016 were collected, respectively, for the before-after study.

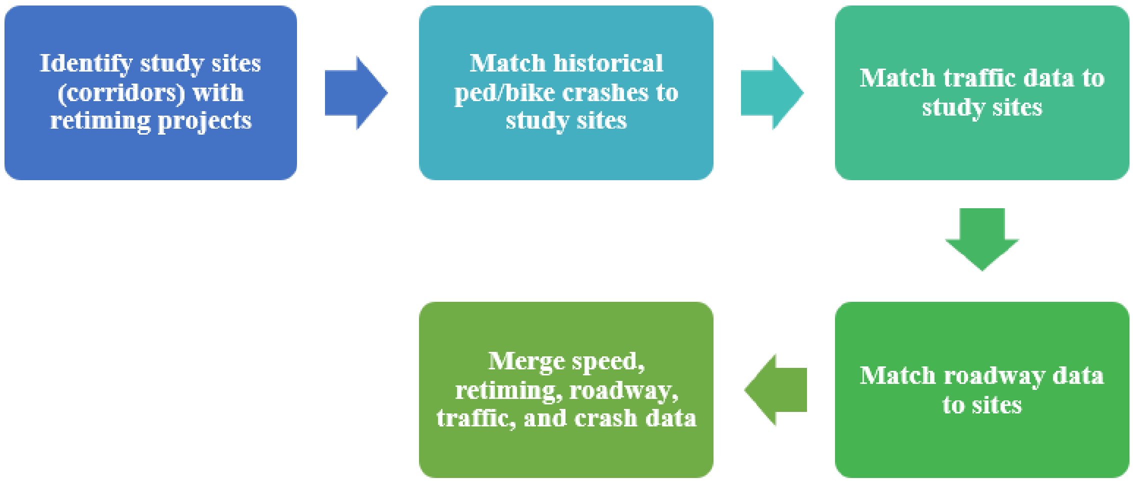


Figure 25. Data matching process

4.3.1 Historical Crash Data Matching

The historical crash data were matched to the identified study sites following the procedure below.

Crash Collection

Historical crash data were retrieved from Signal Four Analytics with the following the query conditions:

- Location: FDOT District 7
- Crash dates: 1/1/2011–12/31/2019
- Pedestrian or bicyclist involved

Not included were crash data for 2020 to eliminate the influence of the COVID-19 pandemic. A total of 16,189 pedestrian and bicyclist crashes were collected for 9 years (2011–2019).

Crash Mapping

As Signal Four data do not include information on roadway ID and milepost for crash events, the CUTR team imported collected historical pedestrian and bicycle crashes in ArcGIS based on their coordinates. The 85 identified study sites (retiming corridors) were mapped into ArcGIS based on the roadway ID and beginning/ending mileposts.

Crash Matching

A buffer of 100-ft on each side was created for each study site in ArcGIS. The pedestrian and bicycle crashes were spatially joined to a study site if they fell into the site buffer, as shown in Figure 26. In total, 4,750 pedestrian and bicycle crashes were matched to the 85 study sites.

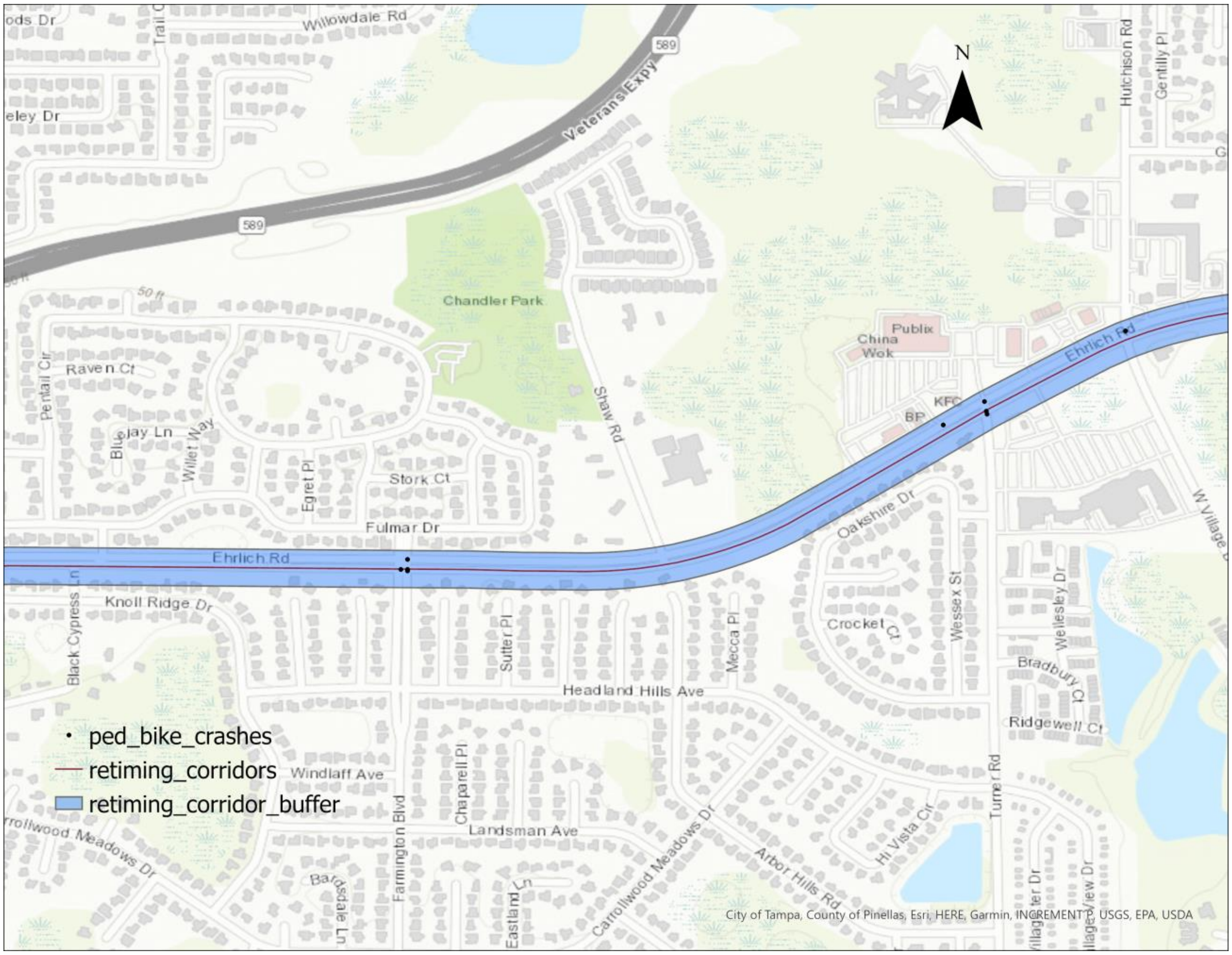


Figure 26. Example of crash data matching

Target Crash Identification

The spatially-joined crashes included those that occurred on both target corridors and side streets. The CUTR team compared the street name string between crash data and site data to identify the pedestrian and bicycle crashes occurring on the target corridors. As the street name string may not have been the exact same between different data sources, even for the same corridor, a natural language processing (NPL) technology (FuzzyWuzzy) was used to fuzzily match the street name strings. The technology gave a score that measures the similarity of two strings from 0 (totally unmatched) to 100 (perfectly matched). Crashes with high matching scores (>70) were identified as crashes on target corridors; for crashes with middle scores (40–

70), a human review was conducted to verify if they occurred on target corridors. In total, 2,306 crashes were identified to link to study sites, as shown in Figure 27.

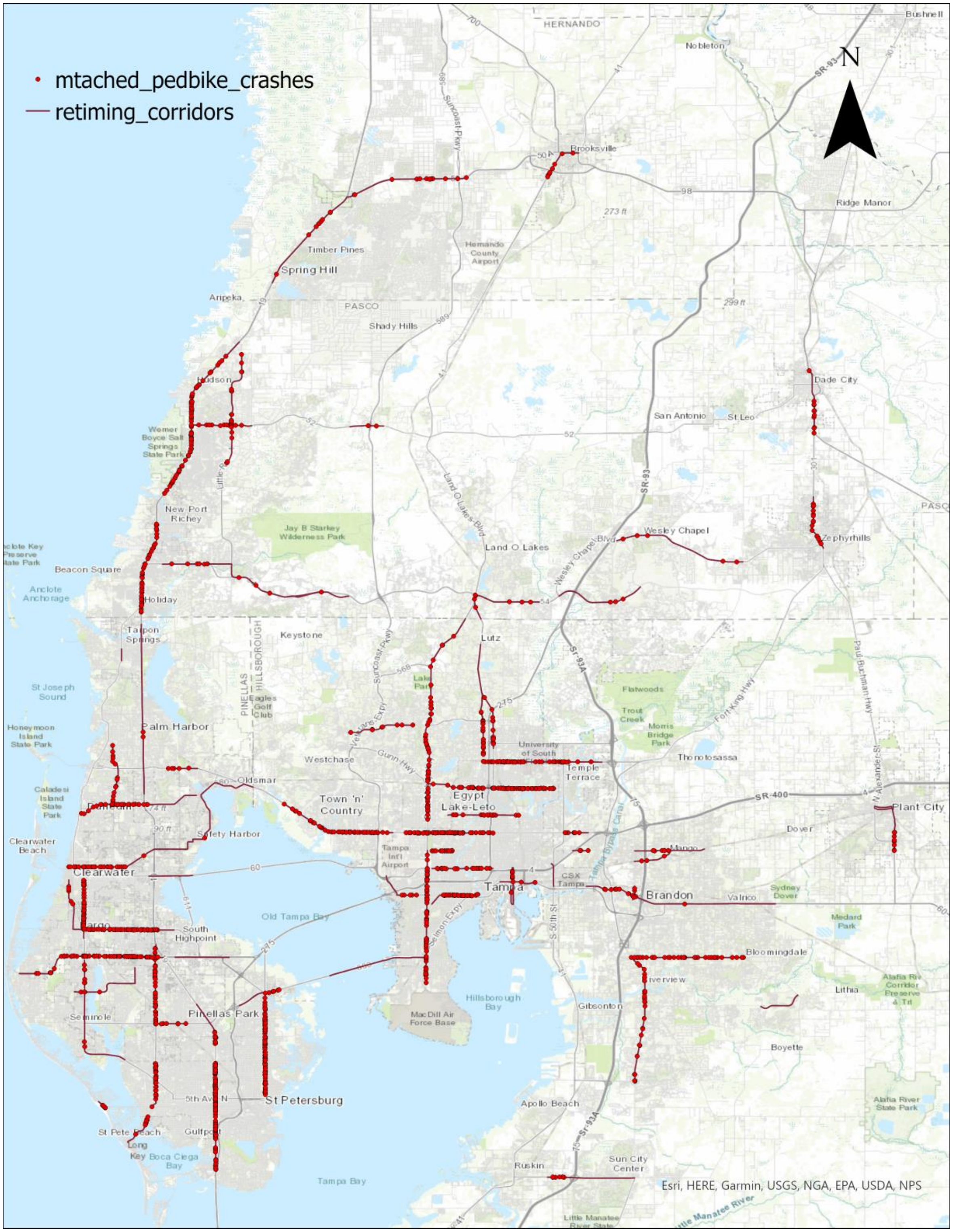


Figure 27. Identified pedestrian and bicycle crashes on study sites

4.3.2 Traffic and Roadway Data Matching

After matching historical crash data, the CUTR team matched traffic and roadway data to each study site. Historical traffic data (2012–2019) were retrieved from the Florida Traffic GIS layers, and the FDOT RCI was used as the source for roadway data. As both traffic and RCI data have roadway ID and milepost information, the traffic and roadway data were matched by these two location measures. The traffic data were collected by year and were aggregated for the before and after stages. The CUTR team verified that roadway data were constant for the past 10 years; thus, the roadway data were collected for the latest year (2019). The matched data are summarized in Table 30.

Table 30. Matched Traffic and Roadway Data

Category	Data Item	Year	Source
Traffic	AADT	2012–2019	Florida Traffic GIS Layer
	Truck Percentage		
	Directional Percentage		
Roadway	Roadway Type	Latest year (2019)	RCI Database
	Functional Classification		
	Median Type		
	Median Width		
	Speed Limit		
	Number of through lanes		
	Shoulder Type		
	Shoulder Width		
	Pavement Conditions		
	Number of Access Points		
	Number of Signals		
	Number of Railway Crossings		
	Number of Interchanges		

4.3.3 Data Assembly

The matched crash data and traffic/roadway data were merged at the site level, with each row representing one retiming corridor and containing multiple columns indicating associated characteristics of speed analysis, retiming parameters, crash, traffic, and roadway. The crash and traffic data will be aggregated into two groups—before and after retiming—based on analysis needs.

5 Analysis of Data and Development of CMFs

This chapter focuses on evaluating the effects of signal progression speed management on pedestrian and bicycle crash patterns on Florida urban roadways. A crash analysis was conducted based on historical crash data from Signal Four Analytics. Retiming project information from Iteris, Inc., and traffic data from the FDOT RCI were collected as described previously. Pedestrian and bicycle crash frequency and severity are important factors in the study:

- *Pedestrian and bicycle crash frequency* represents the likelihood or number of pedestrian and bicycle crashes occurring during a specific period (two years for this project) on a roadway. A large number of crashes on a roadway suggest the need for a safety investigation of that roadway.
- *Pedestrian and bicycle crash severity frequency* represents the number of severe pedestrian and bicycle crashes occurring during a specific period (two years for this project) on a roadway. In the Signal Four crash database, the outcome of a pedestrian or bicycle crash is classified as one of the following: fatality, incapacitating–injury, non-incapacitating injury, possible injury, and no injury. For this study, incapacitating injuries and fatalities are considered to be severe crashes.

As noted, a before-after or cross-sectional study is conducted to assess the safety effects of various safety countermeasures; it compares crash frequencies at the same sites before and after implementing a treatment, which requires sufficient crash observations in the "after" stage and all needed before-after data. A cross-sectional study is applicable if there are enough treated sites and after-crash observations are not fully available; it collects crash data from randomly-selected sites with various traffic signal progression treatments. A cross-sectional study needs enough study sites and sufficient pedestrian and bicycle crashes on each individual site. Due to the unavailability of all required data for a before-after study, cross-sectional designs were used and included the following three countermeasures to the extent possible: 1) progression speed – the actual corridor travel speed including delays at signals; 2) progression speed management – when two or more signals are coordinated to avoid speeding temptations, and 3) progression quality – when drivers can smoothly move from one signal to the other while maintaining the same or similar speed, as described in Table 31.

Table 31. Levels of Progression Speed Management and Progression Quality

Progression Speed Management	Value
No speed management (progression design greatly increases the chance of speeding)	1
Little speed management (progression design increases the chance of speeding)	2
Some speed management (progression design partly decreases the chance of speeding)	3
Good speed management (progression design decreases the chance of speeding)	4
Perfect speed management (progression design greatly decreases the chance of speeding)	5
Progression Quality	**Value**
Poor (unstable vehicle flow with varying speed and several stops or no progression)	1
Average (stable vehicle flow and speed with fewer stops)	2
Good (driving smoothly keeping the same speed and almost no stops)	3

Cross-sectional models were selected and developed to reflect the pedestrian and bicycle safety effects of the progression speed management after signal retiming for the following reasons:

- Although progression speed and progression quality data were available both before and after retiming, progression speed management information was not. Therefore, it was determined to use cross-sectional models to show pedestrian and bicycle safety effects for all three countermeasures.
- Data to estimate predicted and expected crashes for the before-after models were not available at the time of the study.

Estimates from the cross-sectional models for all crashes and severe crashes were used to estimate the CMFs. Data analysis involved several steps, as described in Figure 28.

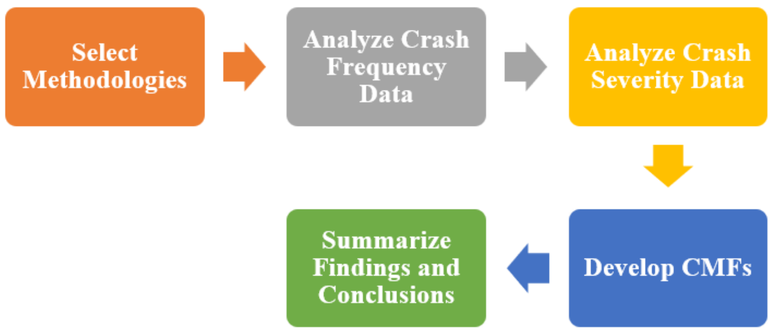


Figure 28. Data analysis process

5.1 Analysis Methodologies

This section describes the data analysis methods used and reasons for their selection to analyze the frequency and severity of the pedestrian and bicycle crashes before and after signal retiming projects. Methods for developing CMFs are included.

5.1.1 Crash Analysis and Modeling Considerations

Crash frequency and severity information is usually analyzed using statistical techniques, including descriptive statistics and regression models. These two methods were used to analyze crash data collected for this study, coupled with a qualitative evaluation of the safety effects. Regression models describe the relationships between the dependent variables—frequency of overall crashes, fatalities, and severe injuries and the various available explanatory variables (traffic data and retiming information).

Several aspects should be considered when selecting the type of regression models to fit crash frequency and severity data:

- *Unobserved heterogeneity* – Unobserved factors often correlated with observed variables can contribute to crashes. Unobserved factors not considered in modeling efforts can lead to unobserved heterogeneity and biased/erroneous predictions. For example, several factors can change a few months after retiming, including traffic demand and signal timing information. To reduce the effects of unknow factors, the appropriate modeling techniques were selected, as were only two years of data both before and after retiming.

- *Confounding variables* – Variables may exist that are correlated with dependent and independent variables. Controlling those confounding variables in the model is important to avoid biased results. Traffic volume is an example of such variables. These variables were incorporated to the extent possible.
- *Self-selectivity/endogeneity* – When a countermeasure was previously placed on roadways or applied to improve traffic and safety on roadways due to the high frequency or severity of crashes, incorporating that factor (countermeasure) in the model without noting that aspect can lead to self-selectivity issue and wrong conclusions. Thus, understanding this issue is important for correct interpretation of the results.
- *Low sample-mean and excess zero observations* – Pedestrian and bicycle crashes, fatalities, and injuries are not frequent on every roadway, as many have zero crashes, fatalities, or injuries. It is important to use the correct modeling technique to analyze these data with excess zero observations to avoid biased results.
- *Risk compensation* – Drivers may adjust their behavior depending on their perceived levels of risk by being more careful when they sense a greater risk and less careful if risk is visibly absent or reduced. The latter can lead to pedestrian and bicycle crashes, fatalities, and injuries. This aspect is useful to consider for avoiding underestimation or overestimation of the models. It is also important to note that in addition to the built environment, traffic, and roadway conditions, driver behavior is an important contributing factor to crashes. For example, drivers under the influence of alcohol or drugs, being distracted, and driving recklessly can contribute to crashes, injuries, and fatalities.

Appropriate data collection design, modeling technique, and interpretation of results were used to address these issues. Random parameter models were selected to address the issues of unobserved heterogeneity, which can also assist with the risk compensation and endogeneity problems.

5.1.2 Cross-Sectional Analysis of Pedestrian and Bicycle Crash Frequency and Severity Data: Random Parameter Negative Binomial Models

Poisson distribution is the standard method to model count data if the variance equals the mean. As overdispersion is frequently observed in crash frequency data, a negative binomial model is the preferred distribution for over-dispersed count data. Moreover, as multiple crash counts are observed for the same roadway at different times, they typically are correlated. A typical approach to modeling such correlated data is to introduce random effects into the linear predictors. Therefore, to model the collected crash data using the cross-sectional approach, random parameter negative binomial models were used. These models can address issues related to overdispersion and correlation in the panel data (crash frequency collected for AM=morning, MD=midday, PM=afternoon/evening, WA=weekend morning, WM=weekend midday, and WP=weekend afternoon/evening for the same roadway).

For example, assume crash frequency Y_{ij}, i= 1, …, r and j =1, …, n_i, where Y_{ij} denotes the jth measurement on the ith roadway. If $Y_i = (Y_{i1}, Y_{i2}, \ldots, Y_{ini})$, assume that x_{ij} and z_{ij} are known vectors of covariates associated with Y_{ij}. Note that x_{ij} and z_{ij} may or may not have common components. It is presumed that conditional on a (q-dimensional) vector of cluster specific random effects, u_i the elements of Y_i are independent negative binomial random variables.

$$Y_{ij}[u_i \sim \text{nb}\ (\alpha, \mu_{ij}) \text{ with } \mu_{ij} = E(y_{ij}[u_i) = \exp\ \{x'_{ij}\beta + z'_{i}j_{ui}\} \qquad (25)$$

Where β is an unknown p x 1 vector of regression coefficients.

The normal distribution is a convenient choice for the random parameters because it allows for a flexible specification of the correlation structure. Hence, assume that $u_1, \ldots, u_r$ are independent and identically distributed random variables (iid) N_q (0, Σ) and that Σ is known up to a vector of variance components, ϴ. The entire vector of unknown parameters is denoted as ψ ˆ (α, β, ϴ).

The random parameter negative binomial model in which conditional on iid random effects, ui ~ N (0, σ2), the expected crash frequency can satisfy the following equation:

$$\log \mu_{ij} = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3 + \ldots + \beta_m X_m + u_i \qquad (26)$$

The pedestrian safety effects (CMFs) of the various covariates can be calculated from the cross-sectional model estimates using Eq. 27:

$$\text{CMF}_{\text{ of }} X_{1,\,2,\,3,\ldots,\,m} = \exp\ (\beta_{1,\,2,3,\,\ldots,m}) \qquad (27)$$

5.2 Cross-Sectional Analysis of Pedestrian and Bicycle Crash Frequency Data

This section focuses on cross-sectional analysis of pedestrian and bicycle crash frequency data. It covers descriptive statistics of pedestrian and bicycle crash frequency, estimated random parameter negative binomial model, model interpretation, and CMF computation.

5.2.1 Descriptive Statistics of Pedestrian and Bicycle Crash Frequency Data After Retiming

Based on 2011–2019 crash data and the signal retiming year of each roadway, two years of data for pedestrian and bicycle crash frequencies after retiming were obtained for each pattern and traffic direction. A roadway could have crash frequency information for all seven traffic patterns (AM, MD, PM, PMO, WA, WD, and WP) and for both traffic directions. Data for 21 roadways were retained after data cleaning, preparation, and the final modeling efforts, producing a total of 267 records. The roadways were selected from Iteris signal retiming projects, and traffic data and roadway information were collected from both Iteris and the FDOT RCI database. In total, 267 frequency records of pedestrian and bicycle crashes after the two-year retiming period were temporally (by traffic pattern) and spatially matched to the roadways. Descriptive statistics of the variables used in the final model are provided in Table 32.

Table 32. Descriptive Statistics after Retiming Pedestrian and Bicycle Crash Frequency, Traffic, and Roadway Variables

Variable Description	N	Mean	Std Dev	Min.	Max.
Dependent Variable					
Number of pedestrian and bicycle crashes after retiming (2 years)	267	0.715	1.366	0	8
Traffic Progression Variables					
Progression speed after retiming (mph)	261	34	7.443	16	51
Good-to-great progression speed management (1 = good-to-great progression speed management, 0 otherwise)	267	0.382	0.487	0	1
Average progression speed management (1 = average progression speed management, 0 otherwise)	267	0.356	0.480	0	1
Average progression quality after retiming (1 = average progression quality, 0 otherwise)	267	0.509	0.501	0	1
Good progression quality after retiming (1 = good progression quality, 0 otherwise)	267	0.363	0.482	0	1
Other Traffic and Roadway Characteristics					
Weekend morning traffic (1 = weekend morning traffic pattern, 0 otherwise)	267	0.075	0.264	0	1
Annual Average Daily Traffic after retiming (AADT/10,000)	267	4.159	1.386	1.597	6.367
Speed limit (mph)	257	44.008	6.365	35	55
Median width (ft)	267	23.996	13.667	8	50
Access density (access points/ segment length)	267	4.339	1.812	0.566	7.811

5.2.2 Estimated Random Parameter Negative Binomial Model

The software package NLOGIT 5 was used to estimate the random parameter negative binomial model of the frequency of all crashes after retiming periods. Several combinations of progression, traffic, and roadway variables were included in previous versions of the model. The best combination of variables that fitted the data was retained as the best model. The final retained model structure is shown in Eq. 28, and the model results are illustrated in Table 33.

$$\log(\text{Pedestrian and Bicycle Crash Frequency}) = 0.033X_1 - 0.743X_2 - 0.797X_3 - 0.429X_4 - 0.734X_5 - 2.519X_6 + 0.576X_7 - 0.091X_8 - 0.052X_9 + 0.390X_{10} \quad (28)$$

Where,

X_1 = progression speed after retiming (mph)

X_2 = average progression speed management (1 = average progression speed management, 0 otherwise))

X_3 = good-to-great progression speed management (1 = good-to-great progression speed management, 0 otherwise)

X_4 = average progression quality after retiming (1 = average progression quality, 0 otherwise)

X_5 = good progression quality after retiming (1 = good progression quality, 0 otherwise)

X_6 = weekend morning traffic (1 = weekend morning traffic pattern, 0 otherwise)
X_7 = Annual Average Daily Traffic after retiming (AADT/10,000)
X_8 = speed limit (mph)
X_9 = median width (ft)
X_{10} = access density (access points/ segment length)

Table 33. Estimated Random-Parameter Negative Binomial Model of Pedestrian and Bicycle Crash Frequency after Signal Retiming

<table>
<tr><th>Variable Description</th><th>Estimate (Marginal Effect)</th><th>Standard Error</th><th>Z</th><th>Prob |z|>Z*</th><th colspan="2">95% Confidence Interval</th></tr>
<tr><td colspan="7">Nonrandom Parameters</td></tr>
<tr><td colspan="7">Traffic Progression Variables</td></tr>
<tr><td>Progression speed after retiming (mph)</td><td>0.033** (0.009**)</td><td>0.014</td><td>2.33</td><td>0.02</td><td>0.005</td><td>0.061</td></tr>
<tr><td>Poor progression speed management</td><td colspan="6">Baseline</td></tr>
<tr><td>Average progression speed management</td><td>-0.743*** (-0.195***)</td><td>0.196</td><td>-3.79</td><td>0.00</td><td>-1.127</td><td>-0.359</td></tr>
<tr><td>Good-to-great progression speed management</td><td>-0.797*** (-0.209***)</td><td>0.211</td><td>-3.77</td><td>0.00</td><td>-1.211</td><td>-0.383</td></tr>
<tr><td>Poor progression quality after retiming</td><td colspan="6">Baseline</td></tr>
<tr><td>Average progression quality after retiming^{+}</td><td>-0.429* (-0.112**)</td><td>0.226</td><td>-1.90</td><td>0.06</td><td>-0.872</td><td>0.014</td></tr>
<tr><td>Good progression quality after retiming</td><td>-0.734** (-0.192***)</td><td>0.331</td><td>-2.21</td><td>0.03</td><td>-1.383</td><td>-0.084</td></tr>
<tr><td colspan="7">Other Traffic and Roadway Characteristics</td></tr>
<tr><td>Weekend morning traffic (1 = weekend morning traffic pattern, 0 otherwise)</td><td>-2.519*** (-0.660***)</td><td>0.968</td><td>-2.60</td><td>0.01</td><td>-4.416</td><td>-0.622</td></tr>
<tr><td>Annual Average Daily Traffic after retiming (AADT/10,000)</td><td>0.576*** (0.151***)</td><td>0.124</td><td>4.64</td><td>0.00</td><td>0.333</td><td>0.819</td></tr>
<tr><td>Speed limit (mph)</td><td>-0.091*** (-0.024***)</td><td>0.013</td><td>-6.82</td><td>0.00</td><td>-0.118</td><td>-0.065</td></tr>
<tr><td>Median width (ft)</td><td>-0.052*** (-0.013***)</td><td>0.015</td><td>-3.53</td><td>0.00</td><td>-0.080</td><td>-0.023</td></tr>
<tr><td>Access density (access points/ segment length)</td><td>0.390*** (0.102***)</td><td>0.065</td><td>5.96</td><td>0.00</td><td>0.262</td><td>0.518</td></tr>
<tr><td colspan="7">Scale Parameters for Distributions of Random Parameters</td></tr>
<tr><td>Average progression after retiming</td><td>0.388***</td><td>0.099</td><td>3.94</td><td>0.00</td><td>0.195</td><td>0.582</td></tr>
<tr><td colspan="7">Dispersion Parameter for Negative Binomial Distribution</td></tr>
<tr><td>Scale parameter</td><td>$0.103\text{x}10^{8}$</td><td>$0.886\text{x}10^{13}$</td><td>0.00</td><td>1.00</td><td>$-0.174\text{x}10^{14}$</td><td>$0.174\text{x}10^{14}$</td></tr>
<tr><td colspan="7">Model Statistics</td></tr>
<tr><td>Log-likelihood at convergence</td><td colspan="6">-209.419</td></tr>
<tr><td>McFadden pseudo R-squared (ρ2)</td><td colspan="6">0.345</td></tr>
<tr><td>AIC/N (N=267 observations)</td><td colspan="6">1.659</td></tr>
</table>

***Significance at 1% **Significance at 5% * => Significance at 10%. $^{+}$*Random parameter*

5.2.3 Model Interpretation

Progression Speed

The model shows the after-retiming progression speed to be a positive fixed parameter, which means that the average number of pedestrian and bicycle crashes increased with an increase in progression speed. The marginal effects, also provided in Table 33, reveal that an increase in progression speed by 1 mph resulted in an average increase of 0.009 pedestrian and bicycle crashes during the two-year after-retiming period. These results were expected but the increase of progression speed only slightly increase pedestrian and bicycle crash frequency.

Progression Speed Management

In addition to progression speed, progression speed management was an important countermeasure in this study. Measures of the progression speed management were acquired from the time-space diagrams in the retiming project final reports. Based on the modeling effort that considers poor progression speed management as the baseline, progression speed management was composed of non-random parameters. Poor progression speed management was considered as the baseline because most Florida arterials in urban and suburban areas are retimed for vehicle mobility and favor high progression speeds. Average speed management was a non-random parameter with marginal effect of -0.195. This indicates that average progression speed management reduced pedestrian and bicycle crashes, on average, by 0.195 during the two-year after-retiming period. Good-to-great progression speed management was also a non-random parameter with marginal effect of -0.209. This implies that making the progression speed management good or great decreases pedestrian and bicycle crash frequency on urban roadways by 0.209. A good progression speed management reduces the need for speeding and aggressive driving, thus leading to less frequent crashes on urban roadways. On the other hand, for no to little progression speed management (comparison baseline in model for average, good, and great progression speed management), drivers seeing a green signal at several successive intersections may be tempted to speed up, leading to more frequent pedestrian and bicycle crashes. It is important to note that roadways with no to little speed management may still have great progression, but they encourage speeding.

Progression Quality

Progression quality, which represents how smoothly traffic flows from one intersection to another and how consistent running speeds are, was another important countermeasure considered in this project. Information on progression quality was obtained from travel time runs diagrams in the retiming project final reports. After considering poor progression quality as the baseline, the model results suggest that the average progression quality was a normally distributed random parameter with a mean of -0.429 and a standard error of 0.226. This suggests that improving the progression speed management from poor to average almost always decreases pedestrian and bicycle crash frequency on urban roadways, but to varying degrees. The good progression quality (compared to poor progression quality) was a fixed parameter, with marginal effect -0.192. Therefore, improving progression quality to good can reduce pedestrian and bicycle crashes, on average, by 0.192. Poor progression quality that results in varying speeds, several stops, and increased delays along a corridor can be irritating and lead to aggressive driving. Improving those conditions with better progression quality can reduce pedestrian and bicycle crashes.

Traffic Characteristics

Frequencies of pedestrian and bicycle crashes based on traffic patterns were evaluated. Based on the model, weekend morning (WA) traffic was a fixed parameter with a marginal effect of -0.660. Results indicate that pedestrian and bicycle crashes decreased, on average, by 0.660 during weekend morning traffic. This finding is reasonable, as during the week people are in a hurry to get home from work, pick up kids from school, or conduct other necessary trips. Although people may be rushing during weekday mornings, there is less pedestrian and bicyclist exposure during that time. Compared to weekdays, weekends are usually more flexible, with reduced activities in the early morning rather than midday. As a result, pedestrian and bicycle crash frequency tends to decrease during weekend mornings.

AADT was a positive fixed parameter. As the volume of traffic increased by 10,000 vehicles per day, the number of pedestrian and bicycle crashes increased by 0.151 during the two years after retiming. An increase in AADT means an increase in exposure to vehicles by pedestrians and bicyclists.

Roadway Characteristics

Several roadway variables were included in the model, including speed limit. Modeling efforts suggest that an increase in speed limit by 1 mph decreased the frequency of pedestrian and bicycle crashes by 0.024, which may be due to lack of exposure of pedestrians and bicyclists on higher-speed roadways.

An increase in roadway median width was associated with a decrease in pedestrian and bicycle crash frequency. When median width increased by 1 ft, pedestrian and bicycle crashes decreased, on average, by 0.013. This finding was expected, as larger median width serves as a good refuge for pedestrians and bicyclists.

Access density (or access points per mile) resulted in a positive fixed parameter. An increase in access points per mile led to increased potential traffic conflicts. The marginal effect indicates that each additional access point increased pedestrian and bicycle crash frequency by 0.102 during the two years after retiming.

5.2.4 Crash Modification Factors (CMFs)

CMFs for progression speed management are measures that depict the effects of progression factors on pedestrian and bicycle crash frequency. Using Eq. 3, cross-sectional design CMFs for progression speed, progression speed management, and progression quality were estimated. In addition, CMFs were estimated for other significant traffic and roadway variables. CMFs and percentage changes for all crashes are shown in Table 34.

Table 34. CMFs and Percent Changes in All Crashes Based on Progression, Traffic, and Roadway Variables

Variable Description	CMF	CRF
Progression speed after retiming (mph)	1.034	-3%
Average progression speed management	0.476	52%
Good-to-great progression speed management	0.451	55%
Average progression quality after retiming	0.651	35%
Good progression quality after retiming	0.480	52%
Weekend morning traffic (1 = weekend morning traffic pattern, 0 otherwise)	0.081	92%
Annual Average Daily Traffic after retiming (AADT/10,000)	1.779	-78%
Speed limit (mph)	0.913	9%
Median width (ft)	0.949	5%
Access density (access points/ segment length)	1.477	-48%

The percentage changes in overall crashes based on various progression, traffic, and roadway characteristics are illustrated in Figure 29. Among the top four factors contributing to decreasing the percentage of overall crashes are weekend morning traffic (92%), good-to-great progression speed management (55%), good progression quality after retiming (52%), and average progression speed management (52%). Other factors contributing to decreasing the percentage of crashes, but to a lesser extent, include average progression quality (35%), speed limit (9%), and median width (5%). Three factors were identified that increased the percentage changes in crashes—AADT (78%), access density (48%), and progression speed (3%).

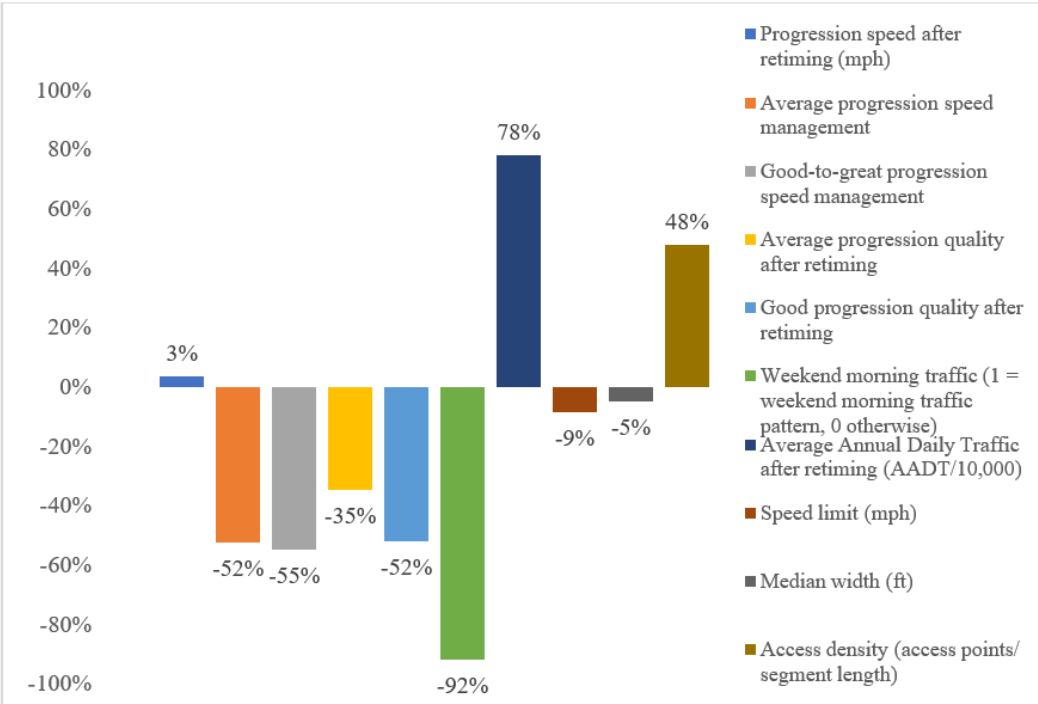


Figure 29. Percent changes in all crashes after retiming based on significant progression, traffic, and roadway factors

5.3 Cross-Sectional Analysis of Pedestrian and Bicycle Crash Severity Data

This section focuses on cross-sectional analysis of pedestrian and bicycle crash severity data. It also includes descriptive statistics of pedestrian and bicycle crash frequency, estimated random parameter negative binomial model, model interpretation, and CMF computation.

5.3.1 Descriptive Statistics of Pedestrian and Bicycle Crash Severity Data

Frequency data for serious injuries and fatalities for the two years after retiming of signals on study roadways were used for a severity analysis. These data were obtained to the extent possible for each of the seven traffic patterns and for both directions of traffic separately. For the 21 roadways considered, as in the all-crash frequency analysis, 267 records were temporally (by traffic pattern) and spatially matched to the roadways. Descriptive statistics of the variables included in the severity model are shown in Table 35.

Table 35. Descriptive Statistics of Crash Severity Variables

Variable Description	N	Mean	Std Dev	Min.	Max.
Dependent Variable					
Number of severe pedestrian and bicycle crashes (severe injuries and fatalities) after retiming (2 years)	267	0.210	0.589	0	4
Traffic Progression Variables					
Progression speed after retiming (mph)	261	34	7.443	16	51
Good-to-great progression speed management (1 = good-to-great progression speed management, 0 otherwise)	267	0.382	0.487	0	1
Average progression speed management (1 = average progression speed management, 0 otherwise)	267	0.356	0.480	0	1
Average progression quality after retiming (1 = average progression quality, 0 otherwise)	267	0.509	0.501	0	1
Good progression quality after retiming (1 = good progression quality, 0 otherwise)	267	0.363	0.482	0	1
Other Traffic and Roadway Characteristics					
Annual Average Daily Traffic after retiming (AADT/10,000)	267	4.159	1.386	1.597	6.367

5.3.2 Estimated Random Parameter Negative Binomial Model

The software package NLOGIT 5 was used to estimate the random parameter negative binomial model of the frequency of incapacitating injuries and fatalities after retiming of signals on the study roadways. Several versions of the model were estimated, including different covariates, before selecting the final model. Once the parameters were estimated, each version of the model was assessed for plausibility and to understand if it fitted the underlying data. If issues were identified during the model validation process, then the process is repeated for a better fitted model. The final estimated model structure and results are shown in Eq. 29 and Table 36.

$$\log(\text{Severe Pedestrian and Bicycle Crash Frequency}) = -0.012X_1 - 1.383X_2 - 1.081X_3 - 0.671X_4 - 1.259X_5 - 0.102X_6 \quad (29)$$

Where,

X_1= Progression speed after retiming (mph)

X_2= Good-to-great progression speed management (1 = good-to-great progression speed management, 0 otherwise)

X_3= Average progression speed management (1 = average progression speed management, 0 otherwise)

X_4 = average progression quality after retiming (1 = average progression quality, 0 otherwise)

X_5 = good progression quality after retiming (1 = good progression quality, 0 otherwise)

X_6= Annual Average Daily Traffic after retiming (AADT/10,000)

Table 36. Estimated Random-Parameter Negative Binomial Model of Severe Pedestrian and Bicycle Injury and Fatality Frequency after Signal Retiming

Variable Description	Estimate (Marginal Effect)	Standard Error	Z	Prob |z|>Z*	95% Confidence Interval	
Means for Random Parameters						
Traffic Progression Variables						
Progression speed after retiming (mph)	-0.012 (-0.001)	0.019	-0.60	0.55	-0.049	0.026
Poor progression speed management	Baseline					
Good-to-great progression speed management	-1.383*** (-0.111***)	0.497	-2.78	0.01	-2.356	-0.409
Average progression speed management	-1.081** (-0.087***)	0.425	-2.54	0.01	-1.915	-0.248
Poor progression quality after retiming	Baseline					
Average progression quality after retiming	-0.671 (-0.054)	0.409	-1.64	0.10	-1.472	0.131
Good progression quality after retiming ^{x}	-1.259** (-0.101**)	0.569	-2.21	0.03	-2.374	-0.144
Other Traffic and Roadway Characteristics						
Annual Average Daily Traffic after retiming (AADT/10,000)	-0.102 (-0.008)	0.122	-0.83	0.40	-0.341	0.137
Scale Parameters for Distributions of Random Parameters						
Progression speed after retiming (mph)	0.012***	0.004	2.88	0.00	0.004	0.020
Good-to-great progression speed management	1.406***	0.305	4.61	0.00	0.808	2.004
Average progression speed management	1.050***	0.268	3.92	0.00	0.525	1.575
Average progression quality after retiming	0.595***	0.189	3.15	0.00	0.225	0.965
Annual Average Daily Traffic after retiming (AADT/10,000)	0.128***	0.033	3.90	0.00	0.063	0.192
Dispersion Parameter for Negative Binomial Distribution						
Scale parameter	0.514×10^{7}	0.637×10^{13}	0.00	1.00	-0.125×10^{14}	0.125×10^{14}
Model Statistics						
Log-likelihood at convergence	-141.53					
McFadden pseudo R-squared (ρ2)	0.153					
AIC/N (N=267 observations)	1.150					

**** Significance at 1%   ** Significance at 5%   * Significance at 10% ^{x} Non-random parameter*

5.3.3 Model Interpretation

Progression Speed

Based on the severe crash frequency model, the after-retiming progression speed estimate was a random parameter with a non-significant mean of -0.012 and a significant standard error of 0.019. This may imply that as the progression speed increases, severe pedestrian and bicycle crashes may decrease at some levels, but not always. The fact that higher progression speed may decrease severe crashes may be partly due to a lack of exposure of pedestrians and bicyclists on higher-speed roadways. Also, the effects of speed on severe injuries and fatalities may be reflected in progression speed management.

Progression Speed Management

The model results show that both average progression speed management and good-to-great progression speed management affect the frequency of severe injuries and fatalities on roadways compared to poor speed management. The average progression speed management and good-to-great progression speed management were normally distributed random parameters with means of -1.081 and -1.383 and standard errors of 0.425 and 0.497, respectively. Improving the progression speed management from poor to average and good almost always decreases pedestrian and bicycle crash severity on urban roadways, but to varying degrees. The progression speed management designs that increase chances of speeding when several green signals follow each other (no to little speed management) can lead to pedestrian and bicyclist severe crashes. On the other hand, enhancing the progression speed management to good or great decreases pedestrian and bicyclist severe injury and fatality frequency on urban roadways to an even greater level than the average. Better progression speed management correlates to lower speeding and less aggressive driving, consequently decreasing the frequency of severe injuries and fatalities even to a greater degree than the frequency of all crashes.

Progression Quality

The average progression quality compared to poor progression quality after retiming was a random parameter with a mean of -0.671 (not significant) and a significant standard error of 0.409. This indicates that enhancing the progression quality from poor to average may decrease pedestrian and bicycle crash severity on urban roadways, but to different extents. Alternatively, the good progression quality compared to poor progression quality was a fixed parameter with marginal effect of -0.101. As the progression quality improves from poor to good, the frequency of severe crashes decreases by 0.101.

Traffic Characteristics

For the crash severity data, AADT was a normal distributed random parameter with means of -0.102 (not significant) and significant standard error of 0.122. AADT can increase severe crashes because of more conflicts between vehicles and pedestrians. However, when the relation between AADT and severe crashes is negative it may reflect the roadway geometry or roadway with less pedestrian and bicycle exposure.

5.3.4 CMFs

To measure the effects of progression speed management and other traffic variables on the frequency of severe pedestrian and bicycle crashes, cross-sectional CMFs were estimated from the model coefficients for variables with significant means, as shown in Table 37.

Table 37. CMFs and Percent Changes in Severe Crashes Based on Progression and Traffic Factors

Variable Description	CMF	CRF
Good-to-great progression speed management	0.251	75%
Average progression speed management	0.339	66%
Good progression quality after retiming	0.284	72%

Three progression variables contribute to the decline of severe injuries and fatalities—good-to-great speed management (75%), average progression speed management (66%), and good progression quality after retiming (72%).

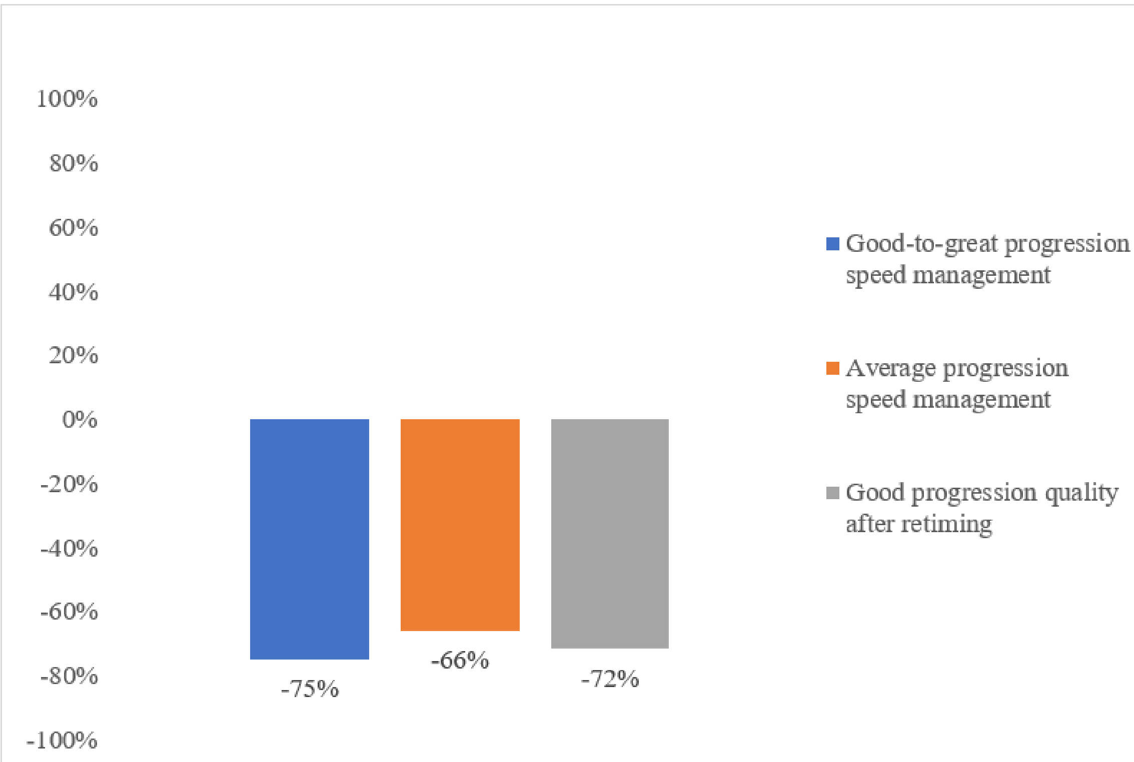


Figure 30. Percent changes in severe crashes after retiming based on signal progression and traffic variable

6 Case Studies and Guidelines & Recommendations

This chapter describes three case studies selected from the study roadways considered during the modeling process and reflects three combinations of progression speed management and progression quality. The intention is to provide examples with detailed illustrations and demonstrate the benefits of good progression speed management and good progression quality. Each case study includes site description, progression speed management, progression quality, and computation of CMFs.

6.1 Case Studies

The three combinations of progression speed management and progression quality in the case studies are shown as follows:

- Good progression speed management & good progression quality (Fowler Ave/SR-582)
- Good progression speed management & poor progression quality (SR-693/66th St)
- Poor progression speed management & good progression quality (Little Rd)

The case studies are limited to evaluating the effects of the progression variables of only a selected traffic pattern (AM, MD, PM, PMO, WA, WD, WP) of the roadway. Data values for the selected three cases are presented in Table 38.

Table 38. Data for Selected Three Case Studies

Variable	Fowler Ave	SR-693 (66th St)	Little Rd
Roadway number	10290000	15230000	14000087
Beginning mile post	0	0.535	1.746
Ending mile post	5.955	3.133	8.289
Direction of travel (1 = east or north, 2 = west or south)	1	1	1
Traffic pattern (1= weekday morning, 6= weekend midday)	6	6	1
Retiming year	2014	2017	2016
Pedestrian and bicycle crashes before retiming	4	2	0
Pedestrian and bicycle crashes after retiming	0	0	1
Severe pedestrian and bicycle crashes before retiming	2	0	0
Severe pedestrian and bicycle crashes after retiming	0	0	0
Progression speed management (4=good, 3=average, 2&1=poor)	4	4	1
Progression quality before retiming (3=good, 2=average, 1=poor)	3	1	2
Progression quality after retiming (3=good, 2=average, 1=poor)	3	1	3
Progression speed before retiming (mph)	35	23	35
Progression speed after retiming (mph)	48	30	43
Roadway speed limit (mph)	50	45	45
Annual Average Daily Traffic (AADT) before retiming (AADT/10,000)	6.267	4.000	4.317
AADT after retiming (AADT/10,000)	6.367	4.025	5.450
AADT before retiming (AADT/10,000) (opt 2&3)	6.175	4.000	4.425
AADT after retiming (AADT/10,000) (opt 2&3)	6.275	4.025	5.400
Truck percentage before retiming	2.633	2.700	4.800
Truck percentage after retiming	2.933	2.350	4.700
Truck percentage before retiming (opt 2&3)	2.700	2.650	4.500
Truck percentage after retiming (opt 2&3)	2.800	2.350	4.500
Access density (access points/ segment length)	5.955	2.598	6.543
Median width (ft)	32	16	44
Weekday afternoon traffic (1=weekday afternoon traffic pattern, 0 otherwise)	0	0	0
Weekend morning traffic (1=weekend morning traffic pattern, 0 otherwise)	0	0	0
Weekend midday traffic (1=weekend midday traffic pattern, 0 otherwise)	1	1	0

In addition to describing the site and illustrating the quality of progression speed management and the quality of progression, CMFs are estimated for each case based on the updated model equations in Chapter 5. As the cross-sectional design was used to model and fit the data, CMFs were estimated separately from the coefficient of each variable in the models. For the case studies, the CMFs of the variables associated with the treatment, which are progression variables, were considered. Given that the functional form of the estimated model was log-linear, the exponent of the coefficient of each signal progression variable was used, as illustrated in Eq. 30:

$$\text{CMF} = \exp\left(\beta_k \times (X_{kt} - X_{kb})\right) \tag{30}$$

Where, X_{kt} = linear predictor k of treated sites and X_{kb} = linear predictor k of untreated sites (baseline condition).

For the case studies, the safety effects of a combination of signal progression countermeasures are of interest. As there is no universal approach to combining CMFs, a guide provided by the Massachusetts Department of Transportation (MassDOT) was employed to combine the CMFs of progression variables included in the model. Based on Figure 31 and the updated estimated CMFs in Chapter 5, there could be some overlap between the two countermeasures: 1) progression speed management and 2) progression quality. Therefore, the research team estimated the CMFs for both the dominant effect and the dominant common residuals as suggested by MassDOT (2020). As the CMFs for the dominant effect provide the greatest reduction, that approach was used using the formula in Eq. 31:

$$CMF_{combined} = \begin{cases} CMF_{countermesure\ 1}, & (1 - CMF_{countermeasure\ 1}) > (1 - CMF_{countermeasure\ 2}) \\ CMF_{countermeasure\ 2}, & (1 - CMF_{countermeasure\ 2}) > (1 - CMF_{countermeasure\ 1}) \end{cases} \tag{31}$$

Where, $CMF_{countermeasure\ 1}$ = CMF for progression speed management and $CMF_{countermeasure\ 2}$ = CMF for progression quality.

From the updated model results in the Task 4 report, the coefficients for good speed management and good progression quality are -0.797 and -0.734, respectively, in the **all pedestrian and bicycle crashes model**. By plugging these coefficients in to Eq. 30, when the progression speed management is good, X_{kt} is 1 and X_{kb} is 0 (baseline condition = poor progression speed management), the new equations are:

$$CMF_{good\ progression\ speed\ management} = \exp(-0.797 \times (1 - 0)) = 0.451 \tag{32}$$

$$CMF_{good\ progression\ quality} = \exp(-0.734 \times (1 - 0)) = 0.480 \tag{33}$$

For the **severe pedestrian and bicycle crashes model**, the coefficient for good speed management and good progression speed management is -1.383 and -1.259, then Eq.32 and Eq. 33 become:

$$CMF_{good\ progression\ speed\ management} = \exp(-1.383 \times (1 - 0)) = 0.251 \tag{34}$$

$$CMF_{good\ progression\ quality} = \exp(-1.259 \times (1 - 0)) = 0.284 \tag{35}$$

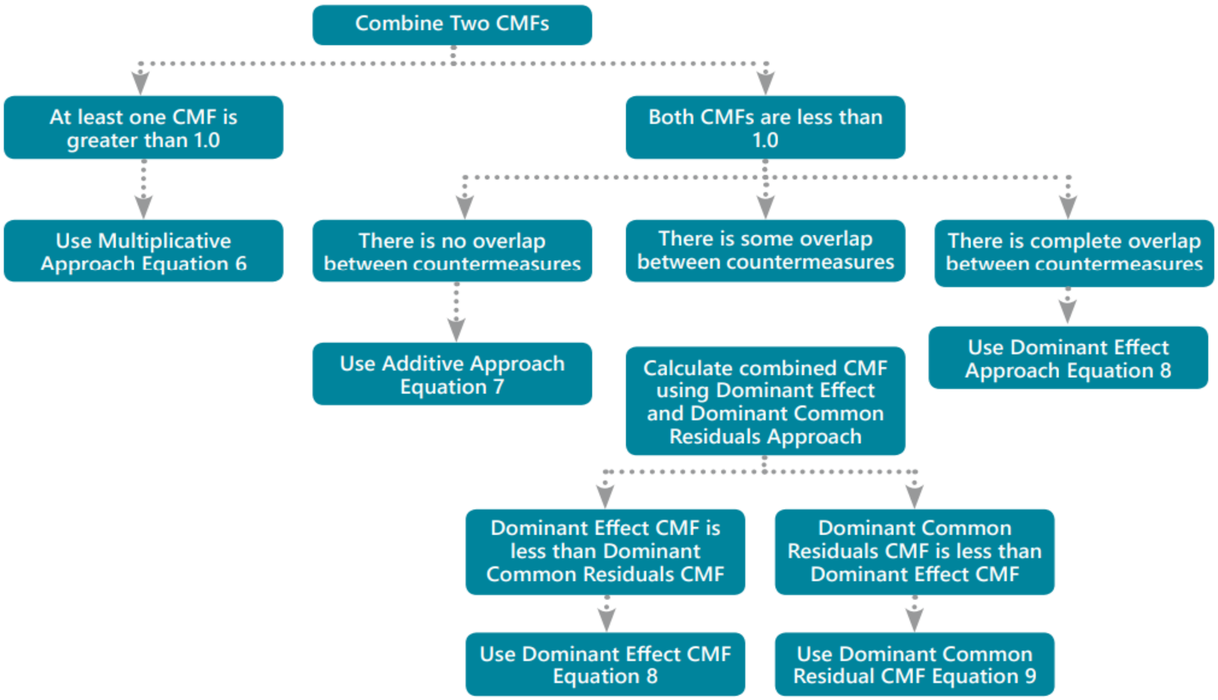


Figure 31. Method for selecting best approach to combine two CMFs

Source: (MassDOT, 2020)

The estimates and CMFs are summarized in Table 39. The combined CMFs will be estimated for each case in the subsequent sections.

Table 39. CMFs for Two Countermeasures

Model	Factor	Coefficient/ Estimate	CMF	CRF
All pedestrian and bicycle crash frequency	Good progression speed management	-0.797	0.451 (45%)	0.549 (55%)
	Good progression quality	-0.734	0.480 (48%)	0.520 (52%)
Severe pedestrian and bicycle crash frequency	Good progression speed management	-1.383	0.251 (25%)	0.749 (75%)
	Good progression quality	-1.259	0.284 (28%)	0.716 (72%)

6.1.1 Fowler Ave (SR-582) (EB Weekend Midday Pattern) Good Progression Speed Management and Good Progression Quality

Site Description

The Fowler Ave (SR-582) weekend midday pattern was used for the first case study. The corridor, located in Hillsborough County, Florida, has 17 intersections and mainly constitutes an eight-lane divided arterial roadway from Florida Ave to Bruce B. Downs Blvd and a six-lane divided arterial between Bruce B. Downs Blvd and Morris Bridge Rd. Figure 32 shows the case study corridor and intersections.

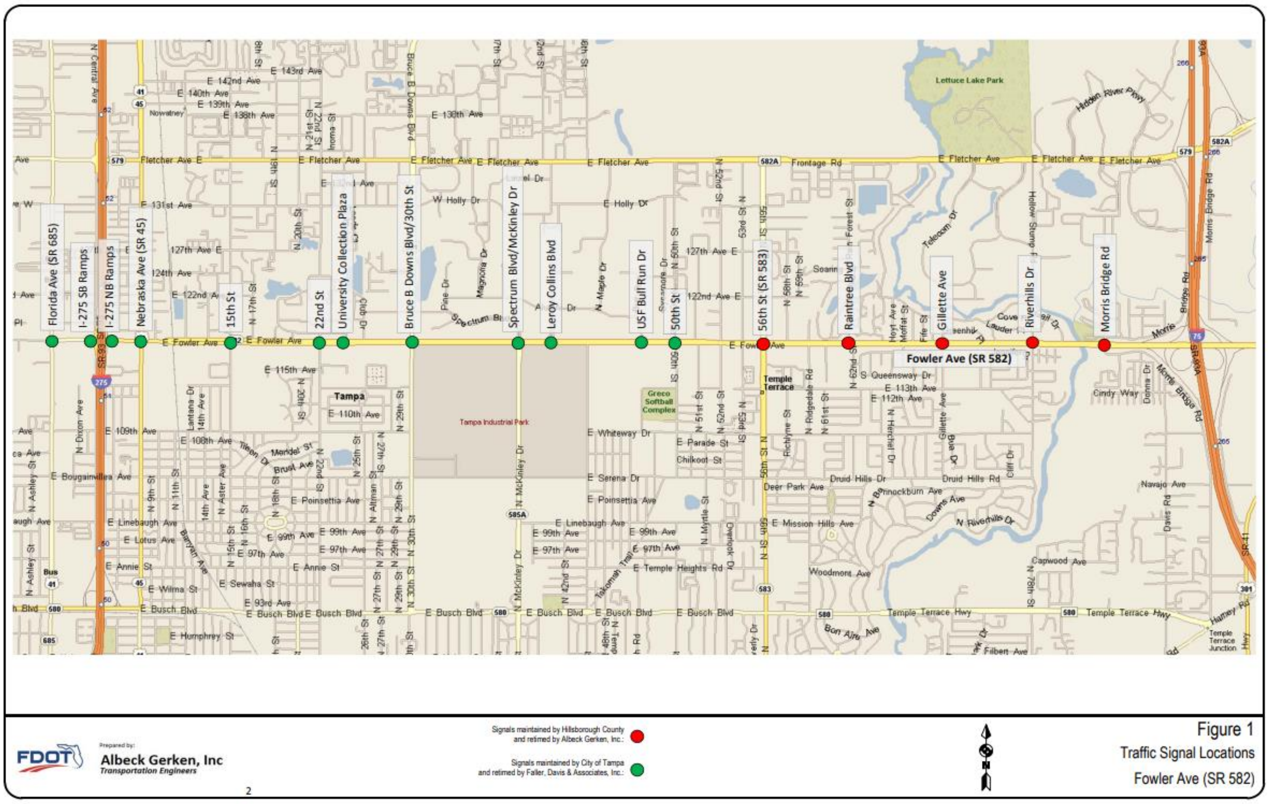


Figure 32. Fowler Ave (SR-582) corridor and intersections

Source: (Iteris, Inc., 2020)

Five intersections on the corridor are maintained by Hillsborough County Public Works and were retimed by Albeck Gerken, Inc.; the other 12 are maintained by the City of Tampa, Transportation Division and were retimed by Faller, Davis & Associates, Inc. The objectives of the retiming project included the following (Iteris, Inc., 2020):

- Collect existing geometric, volume, and traffic signal timing data.
- Conduct field visits to develop understanding of intersection and corridor issues.
- Develop existing traffic operations modeling to benchmark existing capacity analysis.
- Update basic timing parameters and clearance values.
- Modify day plan schedules.
- Implement new signal timing plans.
- Perform post-implementation observation, fine-tune timing, and conduct travel time runs.
- Develop implemented operations models to compare and measure improvements.
- Evaluate capacity and operational improvements and provide recommendations, as necessary.

Progression Speed Management

Eastbound (EB) Fowler Ave (SR-582) weekend midday pattern progression speed management is considered good by the project team (Figure 33). When drivers see alternating red and green traffic lights following each other, they know that speeding may not get them through all of them and that they will probably be stopped by one of the lights at an intersection, which discourages them from speeding. The time-space diagram of the midday pattern of EB Fowler Ave shows such a pattern, as demonstrated in Figure 33. Previous studies highlighted that this kind of design, coupled with signage indicating that signals are timed at specific speeds to avoid stopping, can lead to successful speed management. When speeding is minimized, the frequency of crashes, especially pedestrian and bicycle crashes, may be reduced. For example, it can be seen in Table 38 that the number of crashes before retiming was 4 for this pattern but was zero after retiming. Similarly, the frequency of severe crashes before retiming was 2, which also became zero after retiming.

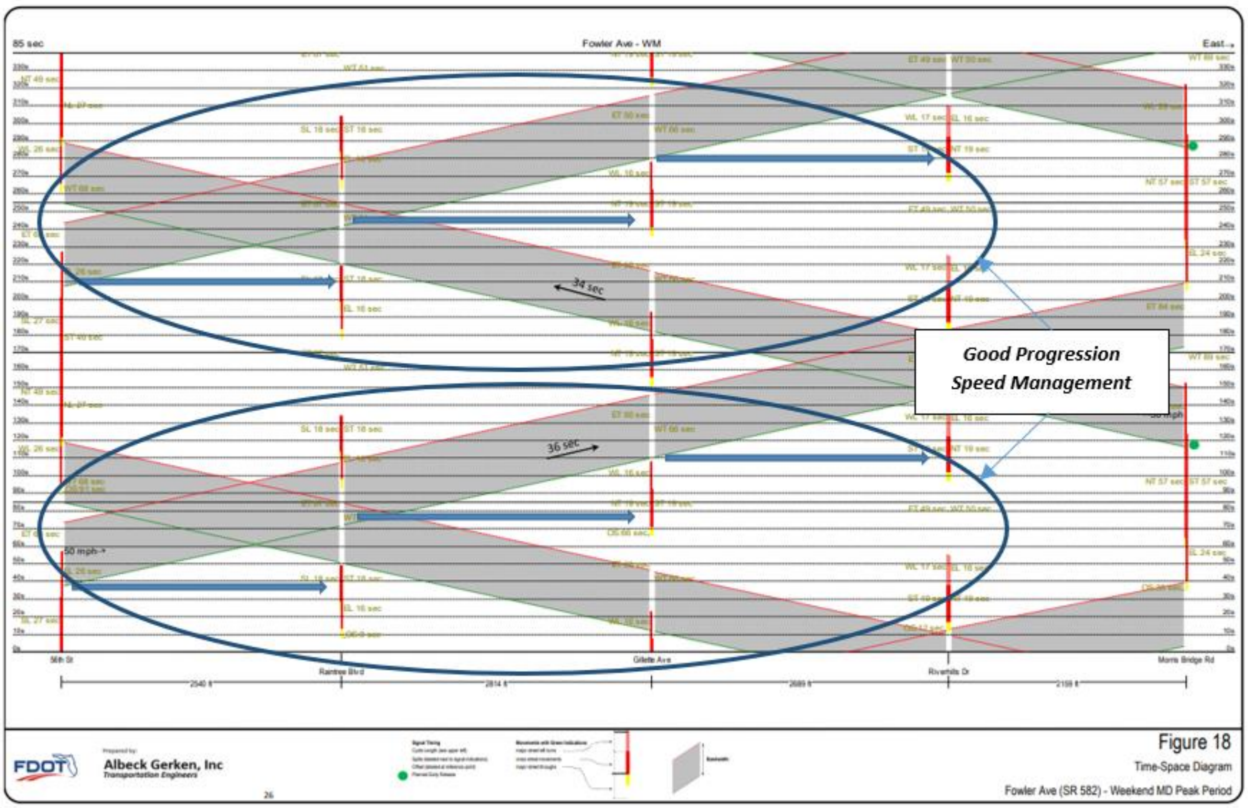


Figure 33. Fowler Ave weekend midday time-space diagram showing good speed management

Source: (Iteris, Inc., 2020)

Progression Quality

The weekend midday pattern of Fowler Ave (SR-582) was classified as good progression quality. Figure 34 shows the average cumulative travel time with existing and implemented signal timings, indicating how smoothly drivers can travel through the coordinated section of the corridor during the weekend midday pattern for the EB implemented pattern. Based on Figure 34, during this pattern, there are seldom any variations in speed. Non-varying speed can lead to non-irritated drivers and non-aggressive driving and a reduction in crashes. The combination of good progression speed management and good progression quality can increase the safety of roadways, as assumed for the case of the EB Fowler Ave weekend midday pattern. It is important to note that other unknown factors could also contribute to the reduction of pedestrian and bicycle crashes on EB Fowler Ave during the weekend midday pattern.

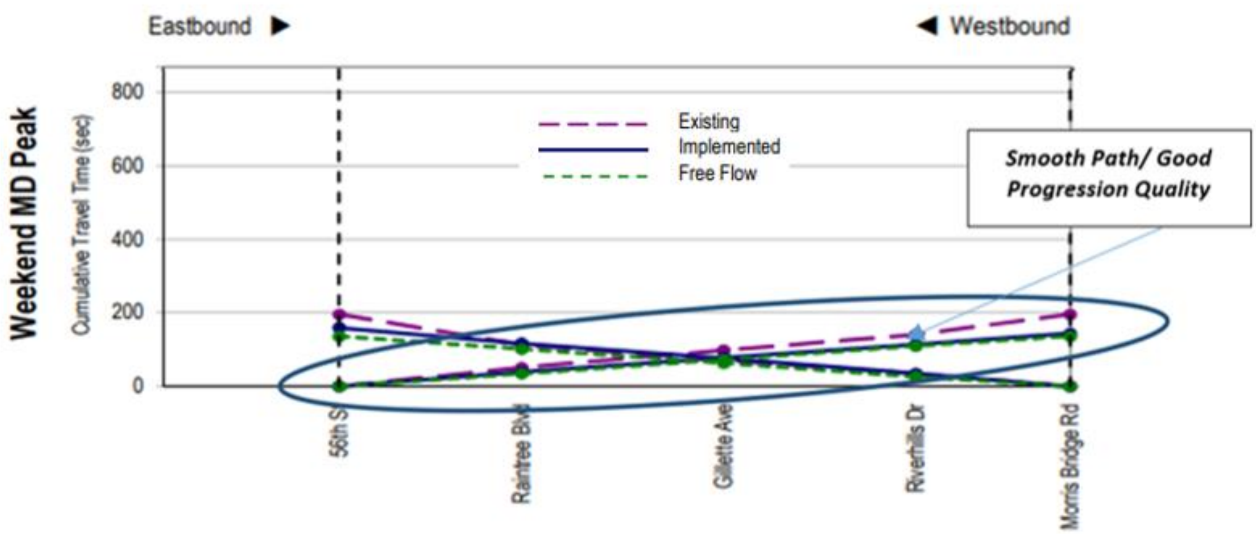


Figure 34. Fowler Ave (SR-582) segment weekend midday travel time diagram showing good progression quality

Source: (Iteris, Inc., 2020)

Crash Modification Factors (CMFs)

Based on the updated and estimated models in Chapter 5 and considering the baselines to be poor progression speed management and poor progression quality, the combined CMFs for all pedestrian and bicycle crashes and for the severe pedestrian and bicycle crashes are estimated as follows (see Table 39):

$$CMF_{combined\ for\ all\ crashes\ model} = 0.451, \text{ since } 0.549 > 0.520 \quad (36)$$

$$CMF_{combined\ for\ severe\ crashes\ model} = 0.251, \text{ since } 0.749 > 0.716 \quad (37)$$

The combination of CMFs for good progression speed management and good progression quality leads to the same percent reduction in crashes as the reduction of crashes associated with the dominant countermeasure (in this case, the progression speed management factor).

6.1.2 SR-693 (66th St) (NB Weekend Midday Pattern): Good Progression Speed Management and Poor Progression Quality

Site Description

66th St/Pasadena Ave (SR-693) in St. Petersburg, Florida, was selected for the second case study, specifically the NB weekend midday pattern. The section of the roadway that was retimed was about three miles long and was mainly a six-lane divided roadway. A visual of the roadway and intersections is provided in Figure 35.

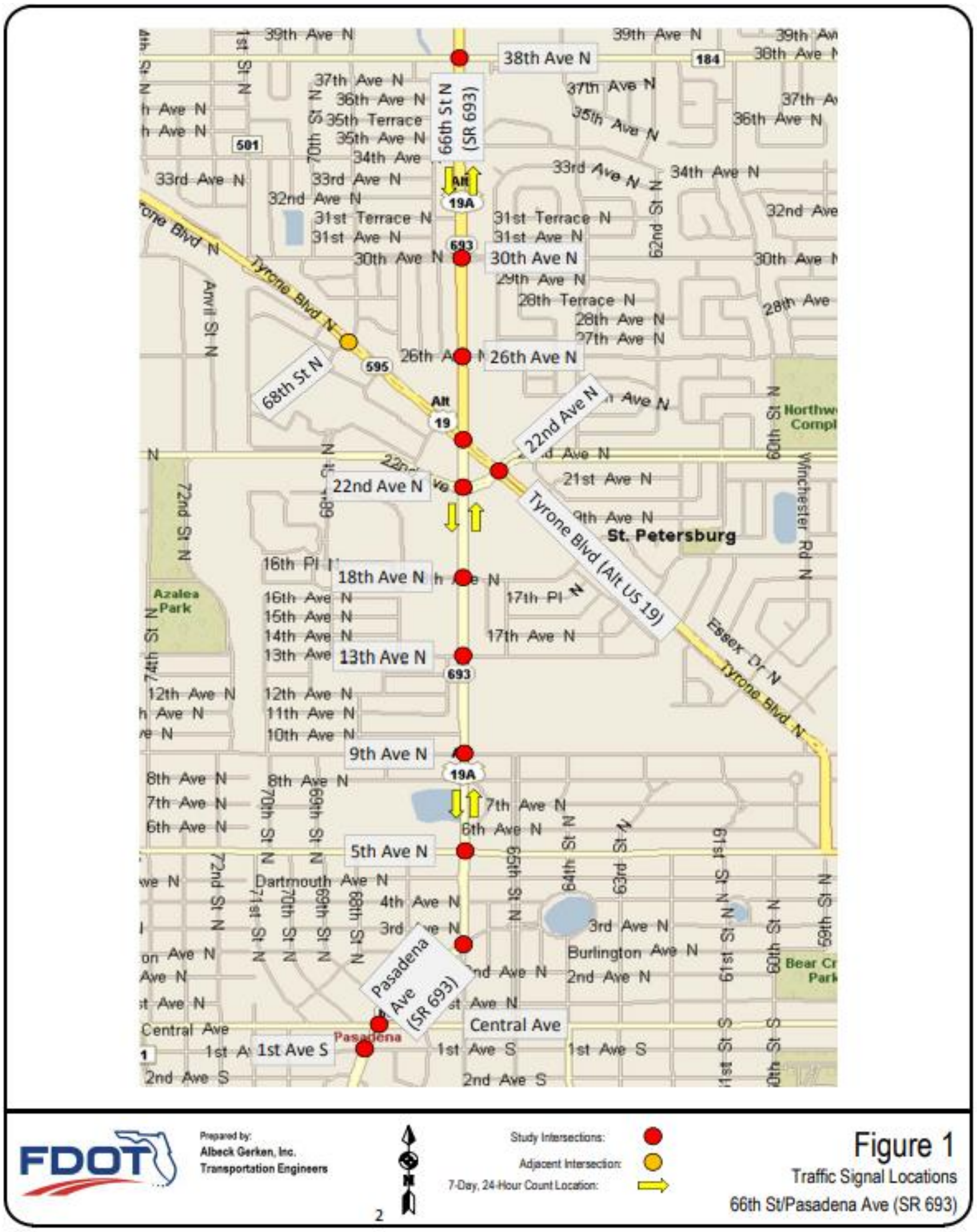


Figure 35. SR-693 (66th St) corridor and intersections

Source: (Iteris, Inc., 2020)

Albeck Gerken, Inc., was responsible for retiming the 13 intersections on the case study corridor. The purposes of the retiming included the following (Iteris, Inc., 2020):

- Collect existing geometric, volume, and traffic signal timing data.
- Conduct field visits to develop understanding of intersection and corridor issues.
- Develop existing traffic operations modeling to benchmark existing capacity analysis.
- Perform before travel time runs and observations of existing conditions.
- Update basic timing parameters.
- Develop appropriate timing patterns to address weekday and weekend traffic flow.
- Modify day plan schedule.
- Implement new signal timing plans.
- Perform post-implementation observation, fine-tune timings, and conduct travel time runs.
- Develop implemented operations models to compare and measure improvements.
- Evaluate capacity and operational improvements and provide recommendations as needed.
- Update timing sheets.

Progression Speed Management

The progression speed management for the NB weekend midday pattern of SR-693 (66th St) is classified as good. The signal timing design illustrated on the time-space diagram in Figure 36 was used during the classification process. There are several occasions shown on the graph (blue arrows) where drivers see that the next green light is followed by a red light or two red lights. That signal timing design (good progression speed management) discourages speeding and likely reduces crashes. Although other factors can contribute to the outcome, the data showed that the frequency of crashes that occurred before the retiming of the NB weekend midday pattern of SR-693 (before pedestrian and bicycle crash frequency = 2) was reduced to zero after retiming. Progression speed management may be a contributing factor to improving safety for that pattern and corridor.

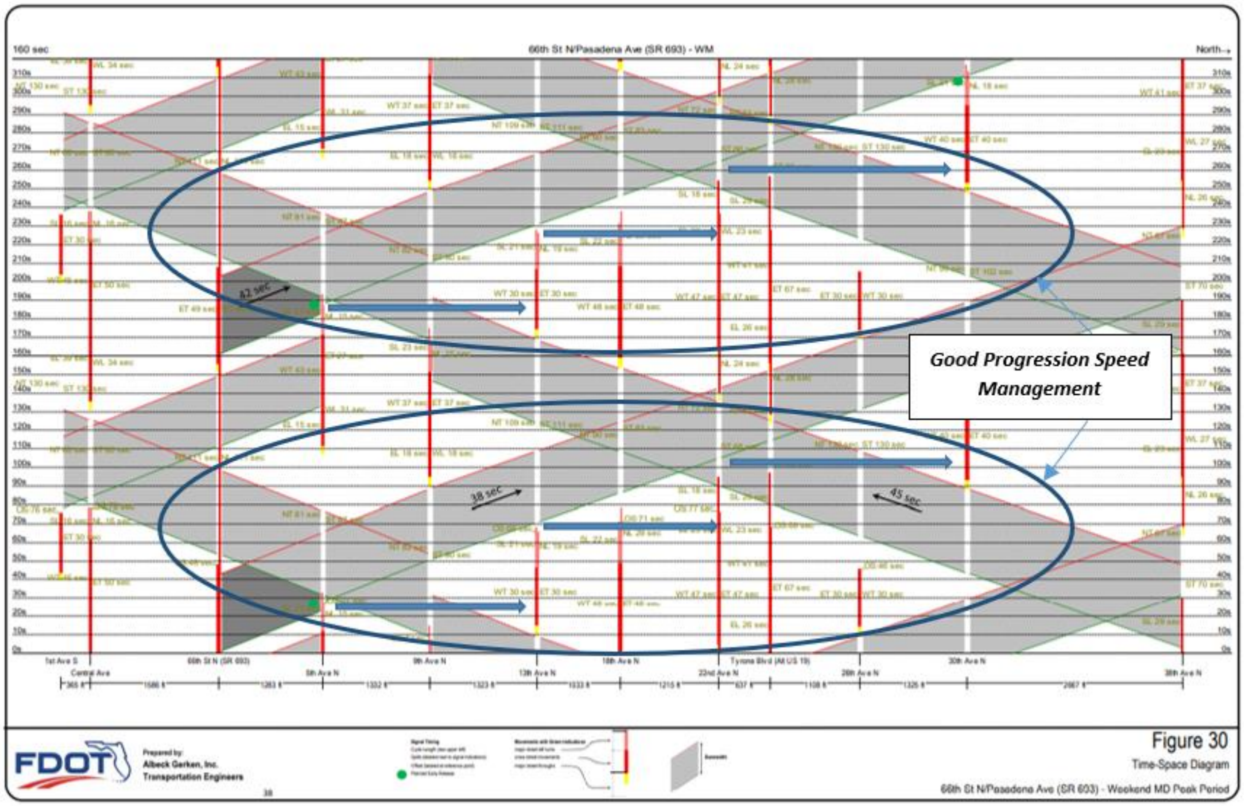


Figure 36. SR-693 (66th St) weekend midday time-space diagram showing good speed management

Source: (Iteris, Inc., 2020)

Progression Quality

The weekend midday pattern of SR-693 (66th St) was categorized as having poor progression quality. Figure 37 illustrates the average cumulative travel time with existing and implemented signal timings and the varying speed experienced by drivers when traveling NB during the weekend midday for a section of the roadway (highlighted in red). Varying speed can be frustrating and cause some drivers to be aggressive, which can increase the chance of crashes on roadways. Although the effects of the poor progression quality could not be demonstrated from this case study data, they may be evident when poor progression quality is coupled with poor progression speed management.

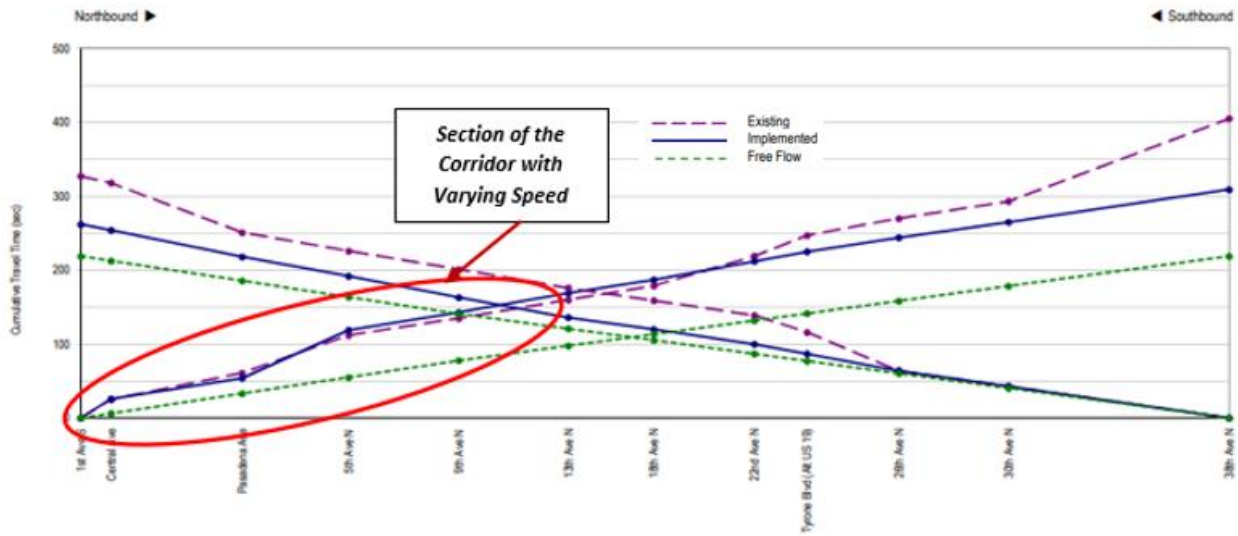


Figure 37. SR-693 (66th St) weekend midday travel time diagram showing poor progression quality

Source: (Iteris, Inc., 2020)

Crash Modification Factors (CMFs)

Considering the baselines to be poor progression speed management and poor progression quality, the combined CMFs for all pedestrian and bicycle crashes and for the severe pedestrian and bicycle crashes were calculated as follows (see Table 39):

$$CMF_{combined\ for\ all\ crashes\ model} = 0.451, \text{ since } 0.549 > 0 \quad (38)$$

$$CMF_{combined\ for\ severe\ crashes\ model} = 0.251, \text{ since } 0.749 > 0 \quad (39)$$

The combination of CMFs for good progression speed management and poor progression quality is the same as the CMFs for good progression speed management.

6.1.3 Little Rd (NB Morning Pattern): Poor Progression Speed Management and Good Progression Quality

Site Description

The third case study was the NB morning pattern of Little Rd in Pasco County, Florida. The case study corridor is about 6.6 miles long and comprises a four-lane divided roadway from Denton Ave to Fivay Rd and a six-lane divided roadway from Fivay Rd to Fox Hollow Dr (Figure 38).

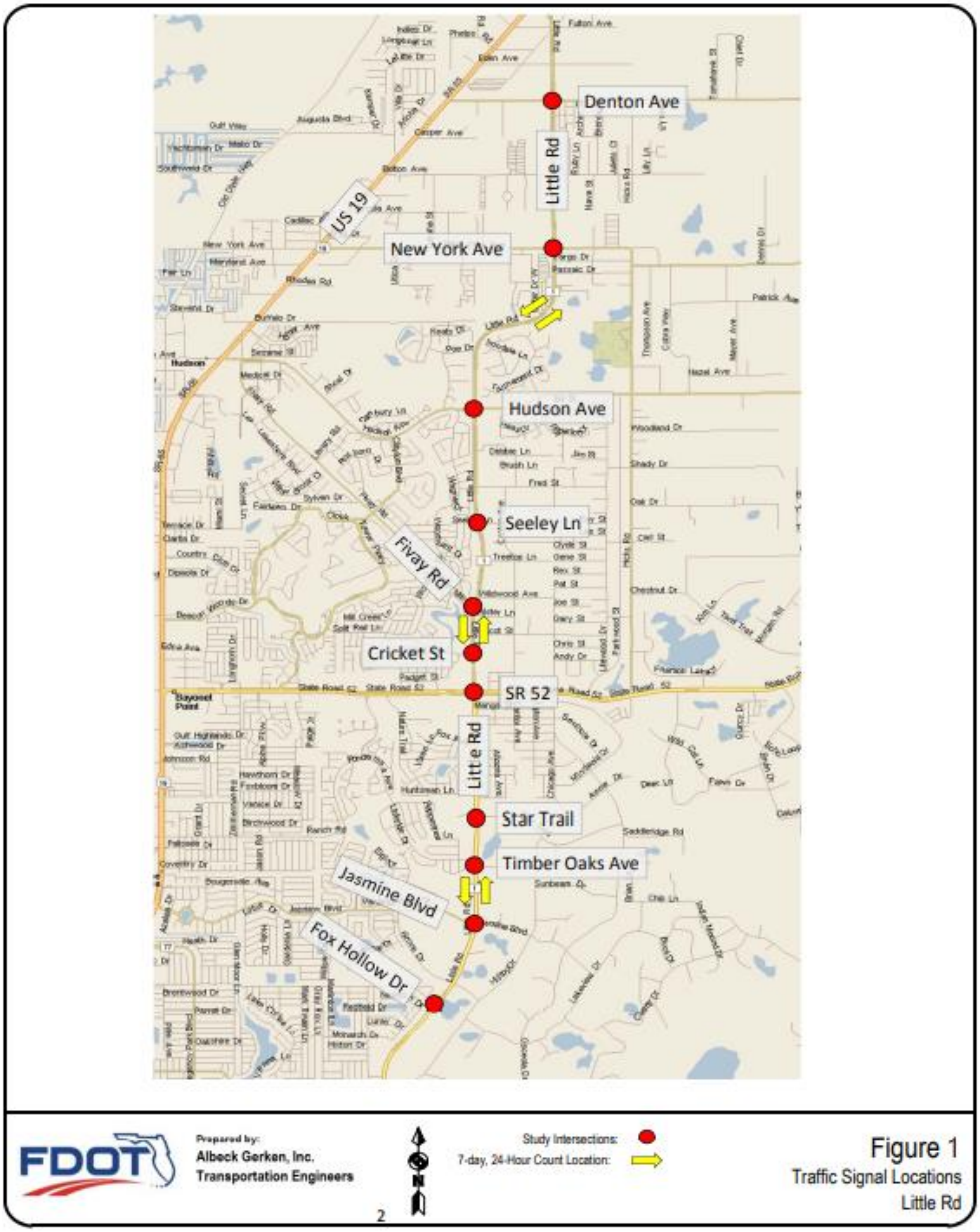


Figure 38. Little Rd corridor and intersections

Source: (Iteris, Inc., 2020)

Albeck Gerken, Inc., oversaw the retiming of the signals at the 11 intersections along Little Rd. The goals of the retiming project encompassed the following (Iteris, Inc., 2020):

- Collect existing geometric, volume, and traffic signal timing data.
- Conduct field visits to develop understanding of intersection and corridor issues.
- Develop existing traffic operations modeling to benchmark existing capacity analysis.
- Perform before travel time runs and observations of existing conditions.
- Update basic timing parameters.
- Develop appropriate timing patterns to address weekday and weekend traffic flow.
- Modify day plan schedule.
- Implement new signal timing plans.
- Perform post-implementation observation, fine-tune timings, and conduct travel time runs.
- Develop implemented operations models to compare and measure improvements.
- Evaluate capacity and operational improvements and provide recommendations as needed.
- Update timing sheets.

Progression Speed Management

The NB weekday morning pattern of Little Rd was categorized as poor or no progression speed management. This is a special case of poor or no progression speed management because the speed management is poor for only some sections of the corridor, as illustrated in Figure 39. The overall progression speed management could be classified as average or good, but the speed management where the crashes occurred on the corridor segment with the first four signals for northbound during the morning pattern was poor. Drivers can see at least two successive green lights head, which can motivate them to speed up. Although the distance is long between the first two signals with low probability of making it through the next couple of intersections, motorists can still attempt to pass through the green signals. This case points to an important aspect of progression speed management—it is not enough to have overall good progression speed management; the quality of the progression speed management should be good for all intersections for a better safety outcome.

Progression Quality

The progression quality for Little Rd is categorized as good. Based on Figure 40, drivers can travel smoothly on Little Rd during the AM peak hours; the graph shows no sign of varying speed or stop-and-go trends for the implemented signal timing. Good progression quality can improve safety, and fewer stops can reduce issues related to aggressive driving; for the Little Rd morning peak case, this contributed to reducing severe crashes from 1 to 0. The case of good progression quality coupled with good progression speed management is preferred for an adequate balance of safety and mobility.

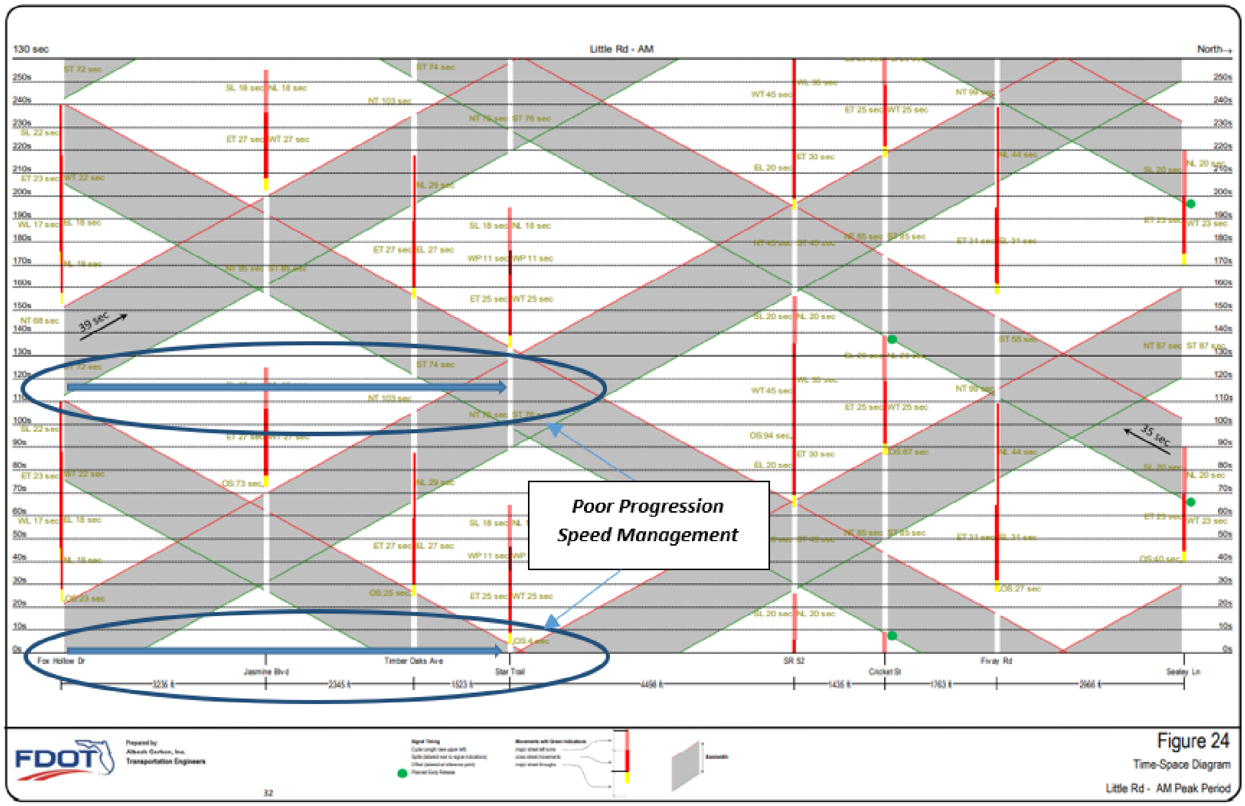


Figure 39. Little Rd morning time-space diagram showing poor speed management

Source: (Iteris, Inc., 2020)

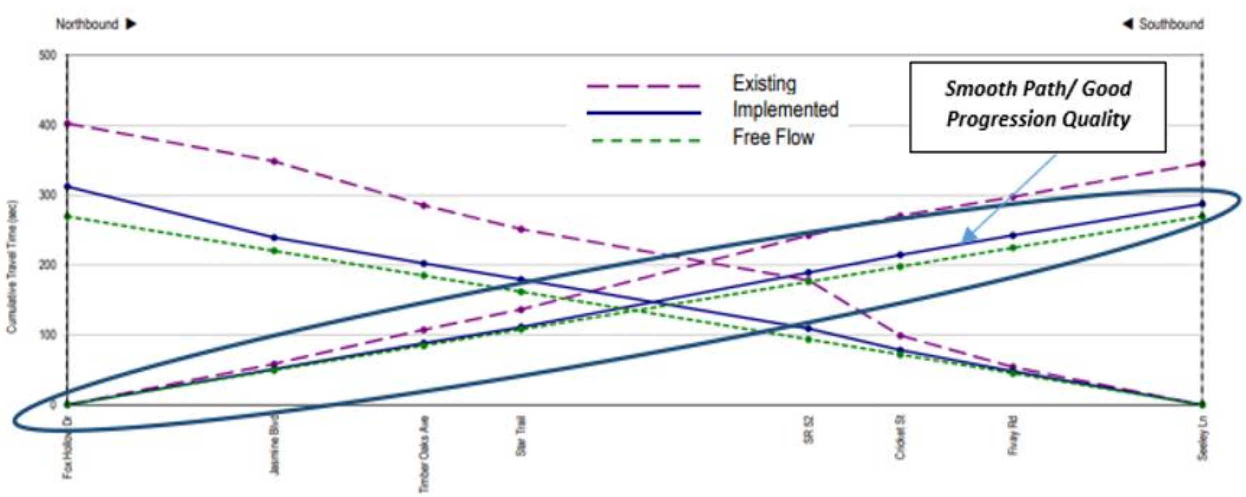


Figure 40. Little Rd morning travel time diagram showing good progression quality

Source: (Iteris, Inc., 2020)

Crash Modification Factors (CMFs)

Considering the baselines to be poor progression speed management and poor progression quality, the combined CMFs for all pedestrian and bicycle crashes and for severe pedestrian and bicycle crashes on Little Rd can be estimated as (see Table 39):

$$CMF_{combined\ for\ all\ crashes\ model} = 0.480\text{, since } 0.520 > 0 \tag{40}$$

$$CMF_{combined\ for\ severe\ crashes\ model} = 0.284 \text{ since } 0.716 > 0 \tag{41}$$

The combination of CMFs for poor progression speed management and good progression quality is the same as the CMFs for good progression quality for this example.

6.2 Guidelines and Recommendations

Progression speed management and progression quality are two different factors that could help improve the safety of pedestrians and bicyclists on urban arterials. The findings from this project reveal that good progression speed management and good progression quality not only reduce overall pedestrian and bicycle crashes but decrease severe crashes involving pedestrians and bicyclists. Hence, it is important to intentionally aim for good progression speed management and good progression quality during the retiming process.

The objective of the following guidelines and recommendations is to assist traffic engineers in designing good progression speed management and good progression quality for safer urban arterials for pedestrians and bicyclists.

6.2.1 Progression Speed Management

Progression speed management can be determined from time-space diagrams. When drivers see green as the next traffic signal indication, they are encouraged to drive faster to pass through the traffic signal. On the other hand, when they see the next traffic signal indication is red, they tend to drive at a constant speed or slower to avoid delay at the next traffic signal. Therefore, alternating red and green traffic light design generally can discourage speeding and lead to good progression speed management. With this type of design, drivers understand that speeding will not get them through the traffic lights smoothly—the probability of being stopped at one or more intersections becomes high with non-successive green lights, which discourages speeding. The following steps are recommended for ensuring the design of good progression speed management:

1. During signal retiming design, engineers should design progression speed at or lower than the posted speed limit to improve progression speed management. By following posted speed limit or designed progression speed, drivers can smoothly drive through most or all signalized intersections on the corridor.
2. After completing a coordinated signal timing plan using signal timing design software, engineers should examine the time-space diagram produced by the software to determine

if the design discourages speeding. If speeding will increase the chance of drivers passing through a couple of traffic signals immediately downstream, engineers should properly adjust the offsets of these downstream traffic signals to reduce and discourage speeding.

3. Engineers should repeat the process for each direction of each signal coordination timing plan.
4. During the fine-tuning process, engineers should confirm that the progression speed management is good-to-great by making sure that drivers at the onset of green cannot see consecutive green signal indications immediately downstream except for a short distance between any two nearby signals such as in downtown CBD.

6.2.2 Progression Quality

For medium (4–7 traffic signals) and long (8 signals or more) corridors, the corridor can be divided into several subsections or groups based on the distance between signals and other factors. A good progression quality is defined by how smoothly the traffic flows from one subsection to another. This enables drivers to experience consistent speed thorough the corridor. With good progression quality, the progression speed of subsections of the corridor is the same or similar to the overall progression speed. Varying speed among subsections can frustrate drivers and lead to aggressive driving, which increases the chance of crashes on roadways. Engineers should follow the following steps to ensure good progression quality (consistent progression speed in subsections) through the corridor:

1. After completion of initial signal timing design, engineers should divide a medium or a long corridor into several subsections to check progression speeds via a time-space diagram.
2. Engineers should check the progress speed (slope of progression band) of each subsection and compare progression speeds of subsections and also should compare them with the progression speeds of the entire corridor. If they are not consistent, engineers should properly adjust signal offsets to ensure consistency of progressing speeds throughout the entire corridor.
3. Engineers should repeat the process for each direction of each signal coordination timing plan.
4. During the fine-tuning process, engineers should confirm that the progression quality is good with consistent progression speed through subsections of the corridors.

If traffic engineers or traffic signal timing engineers are familiar with the concepts and steps to include progression speed management and progression quality into coordinated signal timing design and fine-tuning, they can integrate these two processes into one to save time. Engineers should check both progression speed management and progression quality and adjust signal offsets as needed during the signal timing design and fine tuning.

Improving mobility and safety is challenging but is possible and achievable. Design and implementation of coordinated signal timing plans are commonly used to improve mobility via reduction of vehicle stops and delays and increase of progress speeds on arterials. With proper adjustments to signal timing design and fine-tuning, coordination timing plans can improve mobility and enhance safety, especially for pedestrians and bicyclists via better speed management and smoother signal progression. In addition, when evaluating improvements from traffic signal retiming, CMFs can be developed to analyze the effects of good progression speed management and good progression on crash reduction and severity.

7 Conclusions

This project evaluated the effects of progression speed, progression speed management, and progression quality on pedestrian and bicycle safety. Cross-sectional study designs were used to estimate random parameter models of the safety effects of these three countermeasures. Based on the results, significant contributing factors to pedestrian and bicycle crashes, serious injuries, and fatalities after signal retiming, coupled with their effects on the percent changes of all and severe crashes, are presented in Table 40.

Table 40. Factors Contributing to Pedestrian and Bicycle Crashes after Retiming

Contributing Factors* (Focusing on Study Factors and Keeping Others Constant)	Effects and Percent Change in Overall Pedestrian and Bicycle Crash Frequency	Effects and Percent Change in Severe Pedestrian and Bicycle Crash Frequency
Increase in progression speed after retiming by 1 mph	↑ (3%)	---
Improving poor progression speed management to average progression speed management	↓ (52%)	↓ (66%)
Improving poor progression speed management to good-to-great progression speed management	↓ (55%)	↓ (75%)
Improving poor progression quality to average progression quality after retiming	↓ (35%)	---
Improving poor progression quality to good progression quality after retiming	↓ (52%)	↓ (72%)
Weekend morning traffic conditions compared to all other traffic conditions	↓ (92%)	---
Increase in Annual Average Daily Traffic (AADT) after retiming by 10,000 vehicles	↑ (78%)	---
Increase in speed limit by 1 mph (higher speed limit roadways correlate with less pedestrian exposure)	↓ (9%)	
Increase in median width by 1 ft	↓ (5%)	---
Increase in access density by 1 access point per mile	↑ (48%)	---
**Results based on Tampa Bay Area data and cross-sectional designs*		

Based on detailed quantitative analysis, researchers developed random parameter negative binomial models and CMFs to identify the benefits of signal progression speed management and signal progression quality on improving pedestrian and bicycle safety.

Study findings show the pronounced impacts of well-designed progression speed management on reducing the frequency of pedestrian and bicycle crashes and the frequency of severe injuries and fatalities. Improving poor progression speed management to average progression speed management can reduce pedestrian and bicycle crash frequency by 52% and severe pedestrian and bicycle crash frequency by 66%. Furthermore, improving poor progression speed management to good-to-great progression speed management can reduce pedestrian and bicycle crash frequency by 55% and severe pedestrian and bicycle crash frequency by 75%.

Similarly, the findings prove that good progression quality (smooth traffic flow) can enhance the safety of pedestrians and bicyclists on urban roadways. Improving poor progression quality to average progression quality from traffic signal retiming can reduce pedestrian and bicycle crash frequency by 35%. Improving poor progression quality to good progression quality from signal retiming can reduce pedestrian and bicycle crash frequency by 52% and severe pedestrian and bicycle crash frequency by 72%.

Research results indicate that an increase in progression speed (improvement of mobility) after signal retiming will slightly increase overall pedestrian and bicycle crash frequency. Findings show that good speed management and good progression quality can significantly reduce pedestrian and bicycle crash frequency and severity (improvement of safety). When a traffic signal retiming project can increase progression speed and also improve speed management and progression quality, both driver mobility and pedestrian and bicycle safety can be achieved simultaneously. This is an important finding and conclusion based on intensive data analysis and modeling.

Lower progression speeds and lower posted speed limits are not necessarily the key to pedestrian and bicycle safety. If there are large speed differentials among vehicles on urban arterials, a slower average vehicle speed does not ensure safety for pedestrians and bicyclists. A higher progression speed is not always unsafe for pedestrians and bicyclists if appropriate designs are used with good speed management and good progression quality.

Data analysis results show that an increase in AADT is highly correlated with an increase in pedestrian and bicycle crash frequency. Therefore, for similar urban arterials, an arterial with a higher AADT likely has a larger number of pedestrian and bicycle crashes. Good progression speed management combined with progression quality becomes essential for arterials with moderate to high traffic volumes.

Research results based on cross-sectional data analysis show that an increase in posted speed limits is correlated with a decrease in pedestrian and bicycle crash frequency. The result is likely due to less pedestrian and bicycle exposure on roadways with higher posted speed limits; therefore, there are fewer pedestrian and bicycle crashes.

Findings from this research project also confirm that arterials with wider medians likely reduce pedestrian and bicycle crashes. An increase in access density can lead to a decrease in pedestrian and bicycle safety, indicating that access management on urban arterials is important.

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