Existing Traffic Operations Technical Memorandum

Planning and Environment Linkages (PEL) Study

Connecticut Department of Transportation

March 2023

CONNECTICUT DEPARTMENT OF TRANSPORTATION

I-95 Stamford
Planning and Environment Linkages Study

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1 Introduction

The Connecticut Department of Transportation (CTDOT) is conducting a Planning and Environment Linkages (PEL) study for the reconstruction of I-95 from Exits 7 to 9, including Bridge 00032, in the City of Stamford. The PEL Study will examine alternatives to serve existing and future transportation needs, and identify ways to improve traffic operations, travel time, and safety.

The PEL process will be composed of:

- Existing conditions assessment to analyze and identify existing environmental conditions and issues along the corridor.
- Draft Preliminary Purpose and Need Statement that will serve as a vision statement for the Project and guide the evaluation of alternatives.
- Alternatives development to identify potential transportation improvements to the identified issues and minimize impacts to environmental resources.
- Comprehensive and inclusive public and stakeholder outreach program to seek the best possible transportation solutions.
- Identification of reasonable range of alternatives (to be further analyzed in NEPA environmental phase).

This technical memorandum represents the first phase of the PEL process to analyze and summarize the existing traffic conditions within this area of the I-95 corridor. This summary of existing conditions will be used to inform the development of the project's purpose and need, goals and objectives, and alternatives. Information from this technical memorandum will also be included in the project's *Analysis, Needs, and Deficiencies* Report.

2 Study Area Location and Description

2.1 Project Location and Study Area Description

The Project is located within the City of Stamford, CT and the study area includes a 3.2–mile section of I-95 from Exit 6 to 9 (Figure 1). Exit 6 was included in the study area due to its proximity to Exit 7 and its connectivity to other ongoing traffic studies in the corridor. I-95 is the main north-south Interstate Highway on the east coast running in an east-west direction for 111.57 miles in Connecticut, from the New York state line to the Rhode Island state line.

The study area's transportation network includes a variety of state routes, local roadways, bus and shuttle service, rail, and bicycle and pedestrian facilities. Many of the study area's roadways such as Route 1, Route 137, North and South State Street, Elm Street, Atlantic Street, and others are classified as principal and minor arterial roadways by CTDOT.[1] Arterials are intended to provide a high degree of mobility and carry a high proportion of travel for long distance trips. These facilities are designed to carry the major portion of trips entering and leaving an activity center, as well as the majority of through movements that either go directly through or bypass the area.[2] Other roadways throughout the study area are collector roadways. Collectors "collect" traffic from local roads and connect traffic to Arterial roadways. Collector routes are typically shorter than Arterial routes but longer than local roads. Collectors often provide traffic

[1] CTDOT Roadway Functional Classification Maps, https://portal.ct.gov/DOT/PP_SysInfo/Functional-Classification-Maps.

[2] Federal Highway Administration, Highway Functional Classification Concepts, Criteria and Procedures, https://dot.sd.gov/media/documents/HwyFunctionalClassification.pdf.

circulation within residential neighborhoods as well as commercial, industrial, or civic areas. Figure 2 displays the functional classification of roadways in and around the study area.

Figure 1: Study Area

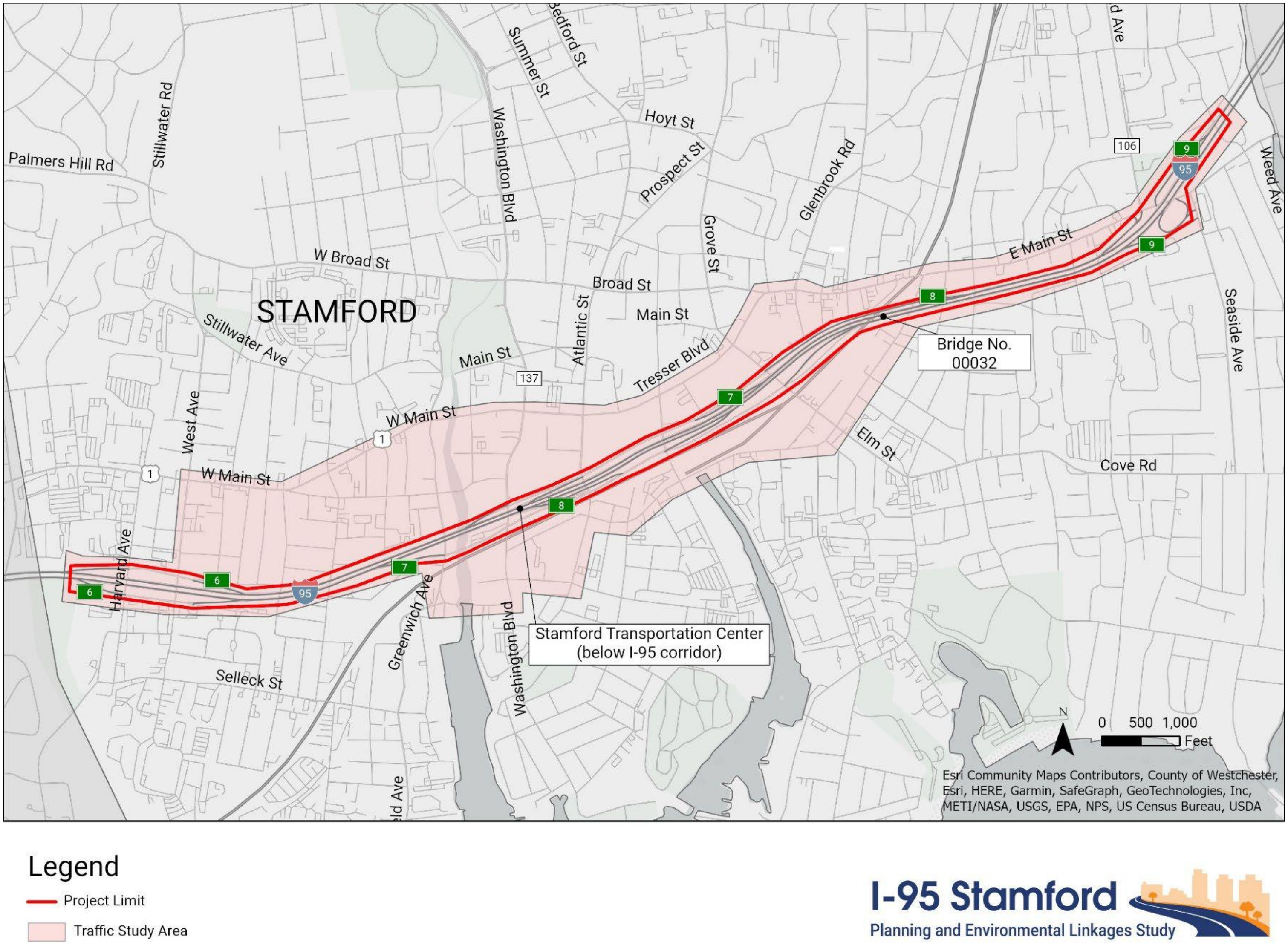

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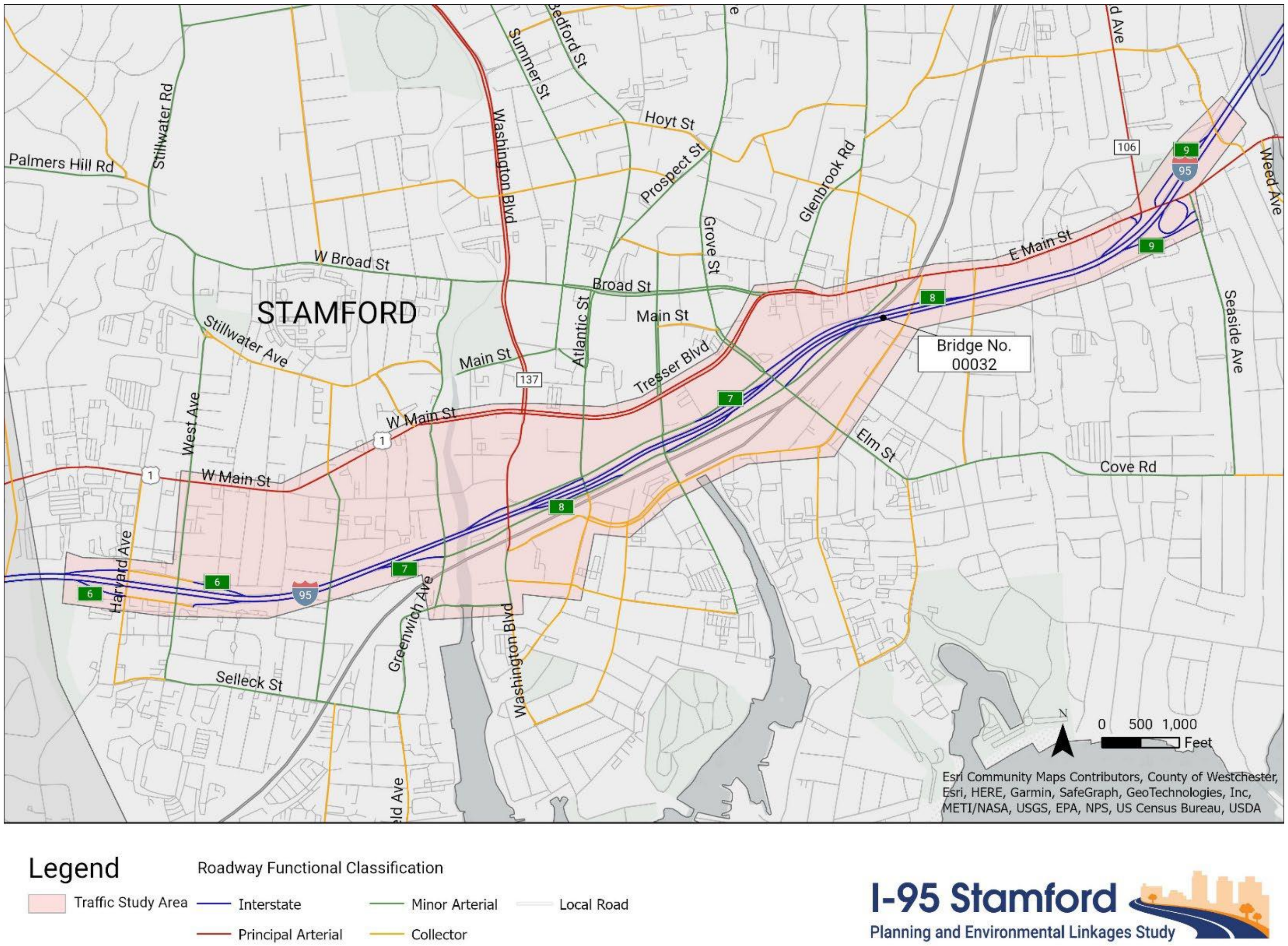
Figure 2: Study Area Roadway Functional Classifications



3 Review of Current and Prior Projects and Studies

This section provides an overview of current and prior studies within the study area.

DOT State Project No. 0135-0346 – Auxiliary Lanes, Resurfacing and Safety Improvements on I-95 (Ongoing)

There is an ongoing project located on the I-95 corridor in the city of Stamford from Bridge No. 00023 over West Avenue at Exit 6 to Bridge No. 00026 over Greenwich Avenue and the Rippowam River at Exit 7. The following are its goals and objectives:

- To provide operational benefits and alleviate congestion between Exit 6 and 7 in both directions.
- To rehabilitate the pavement, address roadside safety and perform bridge rehabilitation to extend the service life of the facility.

Stamford Transportation Center (STC) Master Plan (Ongoing)

The STC is the biggest passenger rail station in Connecticut, serving approximately 28,300 customers each weekday. The public and various service providers routinely struggle to gain access to a physically constrained STC site. The following are the goals and objectives of this project:

- Provide a comprehensive review and evaluation of the existing Stamford Transportation Center site and environs.
- Provide recommendations and conceptual improvements that will lead to increased public and private transit use as well as enhanced vehicular, bicycle and pedestrian access.

Stamford Parking Garage (Ongoing)

The objective of this project is to construct a new multi-level parking garage for the Stamford Transportation Center (STC) on South State Street in Stamford. The proposed garage is bounded on the south by Metro-North Railroad right-of-way, on the east by Washington Boulevard, on the west by Greenwich Avenue, and on the north by I-95. The garage will have a capacity of approximately 1,000 vehicles spread over eight floors.

Strategic Implementation Plan, I-95 West Corridor (2019)

The goal of this study was to reduce congestion on I-95 between New York and New Haven using targeted improvements to the corridor to remove or reduce major bottlenecks. The following were the objectives of the study:

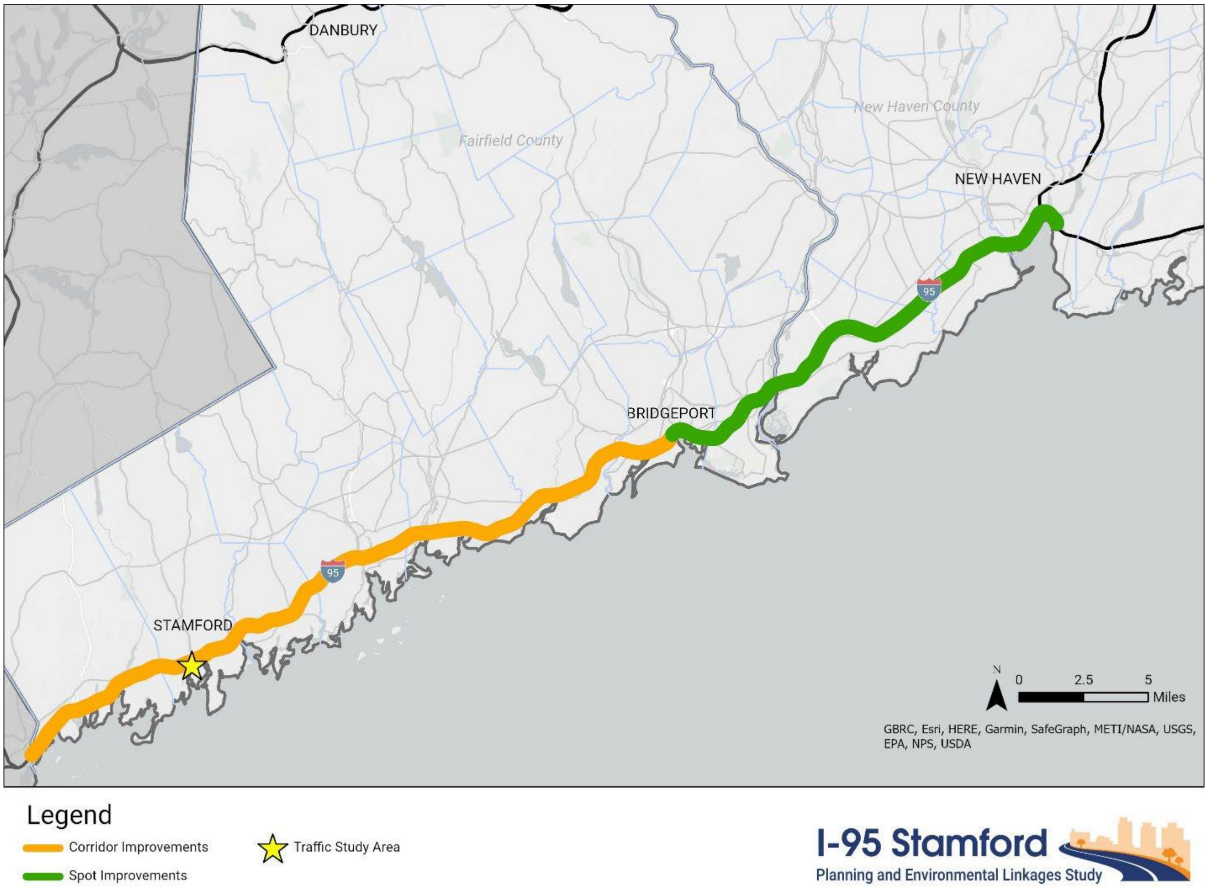
- Analyze existing studies and recent inspection reports, crash statistics, and traffic congestion data to determine the most serious safety issues and identify areas with the biggest bottlenecks.
- Conduct micro-simulation modelling of traffic to evaluate improvement scenarios.
- Identify localized projects that would provide the most benefit to users of the I-95 corridor from the city of New Haven to the New York state line by measuring the safety benefits, reduction of travel time, and vehicle hours of delay.
- Build on findings from prior studies that identified two distinct segments along the corridor – the New York state line to Bridgeport for corridor improvements and Bridgeport to New Haven for predominantly spot improvements.

The report divides its recommended improvements into three categories:

1. Short-range Improvements (1-5 years)
2. Mid-range Improvements (5-10 years)
3. Long-Range Improvements (20+ years)

The extents and level of improvements recommended are shown in Figure 3. The report states that these improvements will result in travel time savings and reduction in delays.

Figure 3: General Plan, New York State Line – New Haven



I-95 Improvements – Feasibility Study (Greenwich to New Haven) (2018)

The following were the goals and objectives of the 2018 I-95 Improvements – Feasibility Study:

- To evaluate the feasibility of adding one additional operational lane in each direction along I-95 between the Connecticut/New York state line in Greenwich and Bridgeport.
- To evaluate spot improvements that can be constructed between Bridgeport and New Haven, which provide safety and operational improvements to the corridor.

The report concludes that implementing a four (4) lane operation on I-95 from Greenwich (NY state line) to Bridgeport is feasible and practical.

Connecticut I-95 Corridor Congestion Relief Study (2016)

The Connecticut DOT performed a corridor congestion relief study of I-95 and the Merritt Parkway (Route 15) from New Haven to the New York state line. The primary objective of the study was to determine whether congestion pricing on I-95 and Route 15 using All Electronic Tolling could reduce congestion along the I-95 corridor. This was achieved through the following methods.

- The assembly and collection of traffic and travel time data

- A stated preference survey to estimate value of time in the study corridors
- Detailed traffic modelling and toll revenue evaluation for a variety of configuration and pricing alternatives

Based on two parameters, congestion reduction and net toll revenue, the study suggests that congestion pricing can significantly reduce congestion along I-95 between New Haven and New York. The revenue generated could then be used to support widening of the Interstate.

4 Existing Transportation Conditions

This section summarizes the traffic and infrastructure data collected to date for the PEL study.

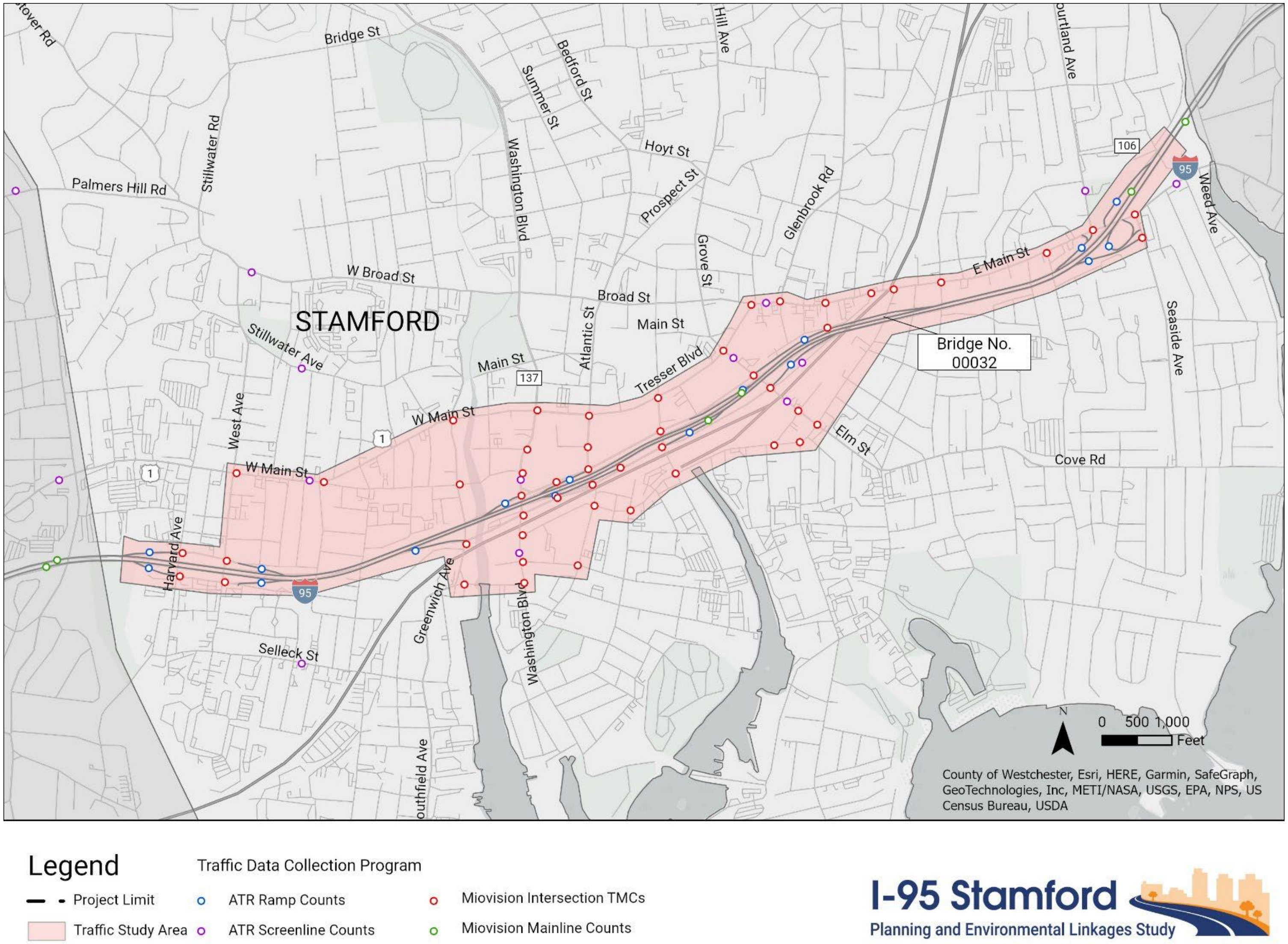
4.1 Vehicular Traffic Data

This section summarizes the existing vehicular traffic data collected for this study along I-95, Route 1, and other key State and local roadways within the study area. The traffic data was collected in April and May 2022. Despite impacts of the COVID-19 Pandemic on recent traffic volumes, CTDOT has determined that travel patterns have now normalized enough that newly collected data can serve as an accurate basis for use in the PEL study.

To evaluate traffic conditions, a comprehensive data collection program was conducted in April and May 2022. Generally, for weekdays, data was collected from 6 AM to 8 PM, with 24-hour counts collected on select days to reflect typical traffic conditions. The data collection program, as shown in Figure 4, consisted of the following:

- Miovision cameras to count volumes at six locations on I-95,
- Turning Movement Counts (TMC) using Miovision cameras at 50 intersections (city owned) within the study area, and
- Automatic Traffic Recorders (ATR) at I-95 ramps and along local roadways in Downtown Stamford.

Figure 4: Traffic Data Collection Program



4.1.1 Mainline (I-95) Traffic Counts

Mainline counts were taken in 15-minute intervals at six locations on I-95 from April 26-28, 2022 (Tuesday to Thursday) and from May 2-7, 2022 (Monday to Saturday).

Table 1: Miovision Mainline Counts

Location	Description
M1	NB I-95 South of Exit 6
M2	SB I-95 South of Exit 6
M3	NB I-95 South of Exit 7 On-Ramp
M4	SB I-95 South of Exit 7 Off-Ramp
M5	NB I-95 North of Exit 9
M6	SB I-95 South of Exit 9 Off-Ramp

Figure 5 shows variation in daily traffic volumes at I-95 northbound (M1) and I-95 southbound (M2). The total traffic at these two locations is higher on weekdays as compared to Saturday, except for Monday. On Saturdays, traffic volumes during midday are higher than during the same period on weekdays, which can be attributed to traffic flowing towards commercial establishments in the area. Total southbound traffic volumes are typically higher compared to northbound traffic volumes, regardless of the day of the week.

Figure 5: Variation in Daily Traffic Volumes (M1/M2)

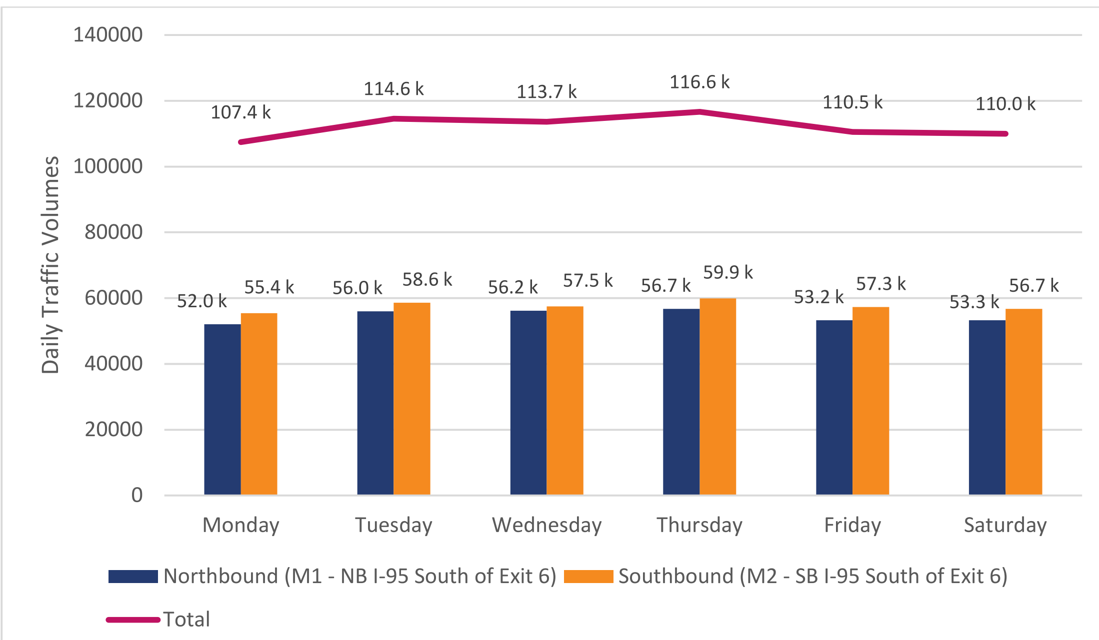


Figure 6 and Figure 7 show traffic volumes for average weekday (Tuesday – Thursday) on I-95 between Exits 6 and 9. On weekdays, traffic volumes are highest after 7 AM in the northbound direction, while volumes are highest at or before 7 AM in the southbound direction. Traffic volumes in the southbound direction decrease after 7 AM, although this is likely due to significant queuing and congestion as a result of increased demand. Traffic volumes are more uniform in the northbound direction after a slight dip during the post-AM peak period, while the southbound direction experiences troughs and crests through the day.

Figure 6: Northbound Mainline Weekday Hourly Traffic Volumes

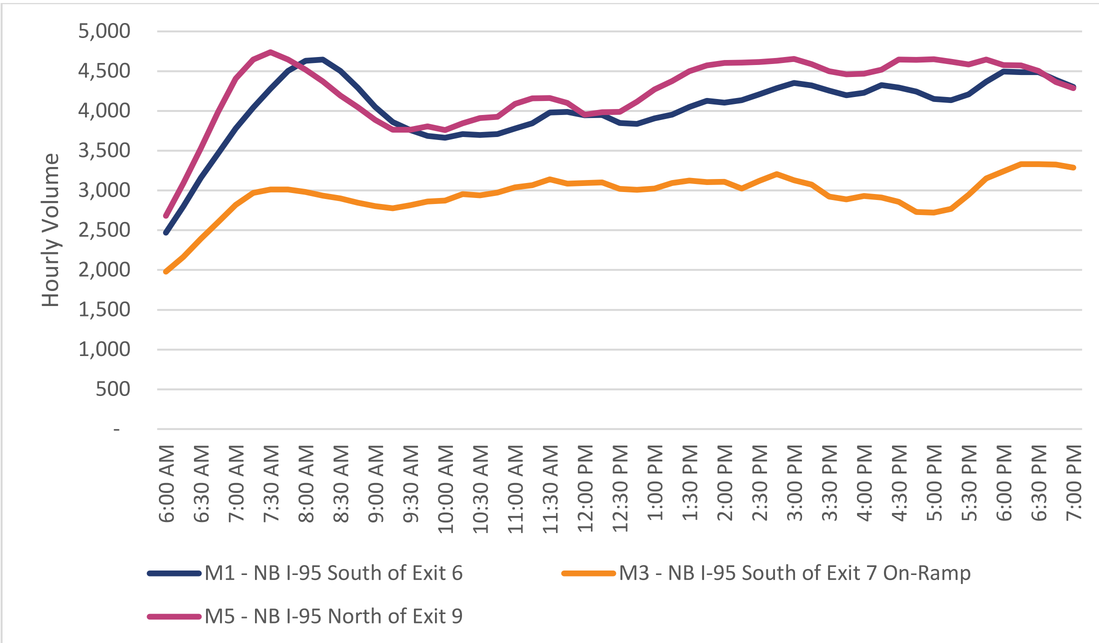
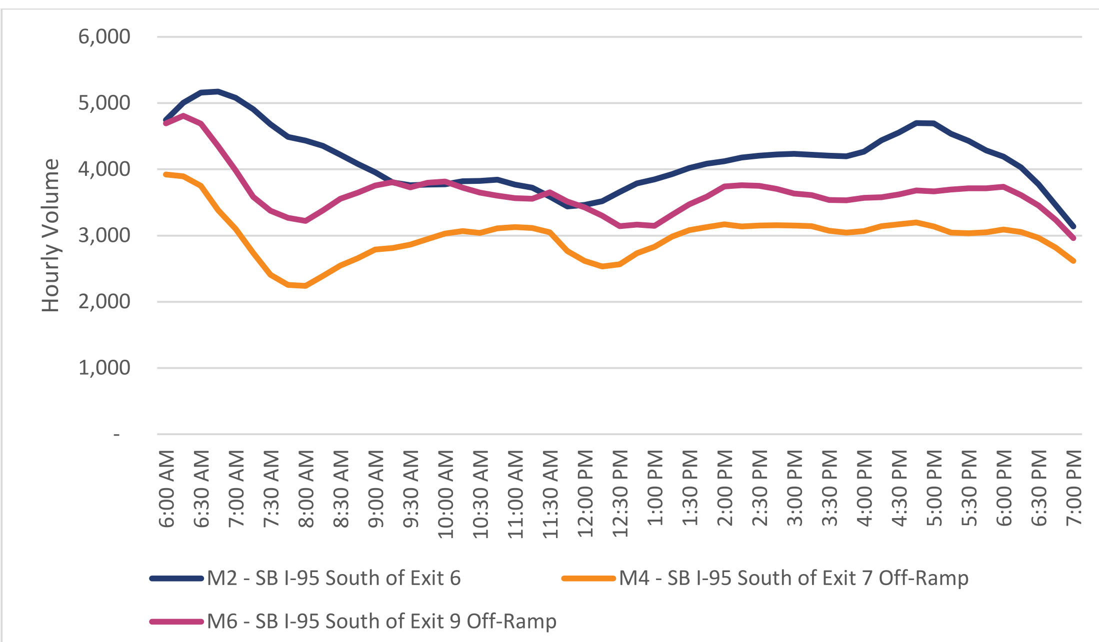


Figure 7: Southbound Mainline Weekday Hourly Traffic Volumes



4.1.2 Ramp and Local Street Traffic Counts

Automatic Traffic Recorder (ATR) counts were conducted in the study area to obtain volumes, speed data and vehicular classification for two different road types:

1) On- and off-ramps on I-95 and
2) Important local roadways.

Turning movement counts (TMCs) were conducted in the study area at 50 selected intersections using Miovision cameras and video processing. The TMCs collected volumes, vehicular classification, bike counts, and crossing pedestrian counts.

On- and Off-Ramps on I-95

Data was collected continuously on 16 ramps between May 4 – April 24, 2022.

Figure 8 shows hourly, weekday, on-ramp traffic volumes in the northbound direction. All ramps experience AM and PM peaking. Exit 6 experiences more traffic than any other exit for most of the day.

Figure 8: Northbound (On-Ramps) Hourly Weekday Traffic Volumes

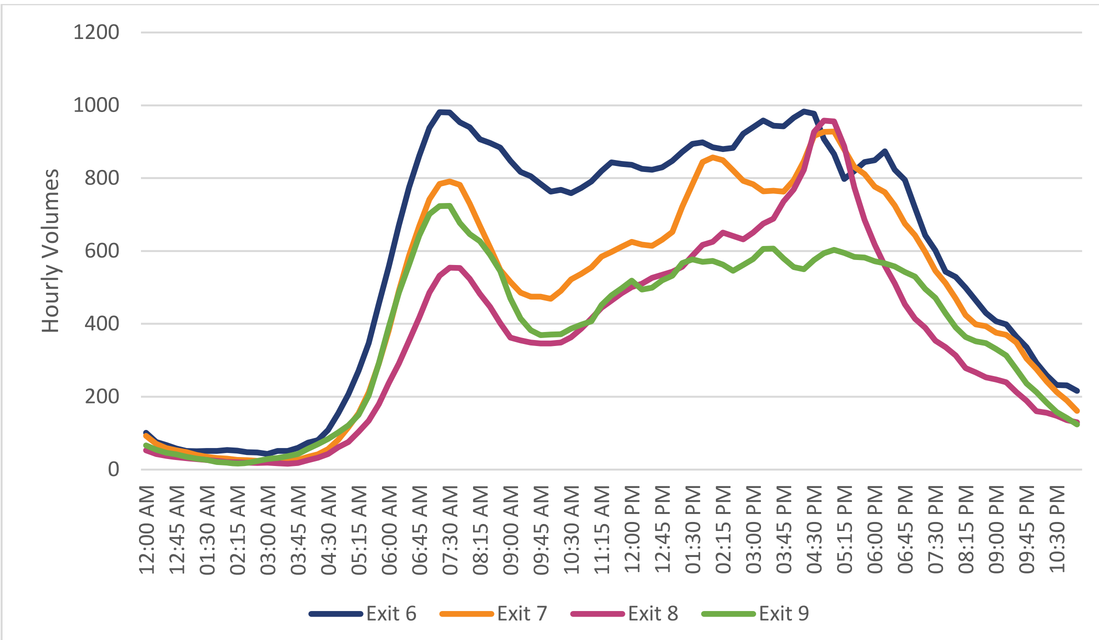


Figure 9 shows weekday hourly off-ramp traffic volumes in the northbound direction. All ramps experience AM peaking, while only Exit 8 has a pronounced PM peak. Exit 8 experiences more traffic than any other exit for most of the day.

Figure 9: Northbound (Off-Ramps) Hourly Weekday Traffic Volumes

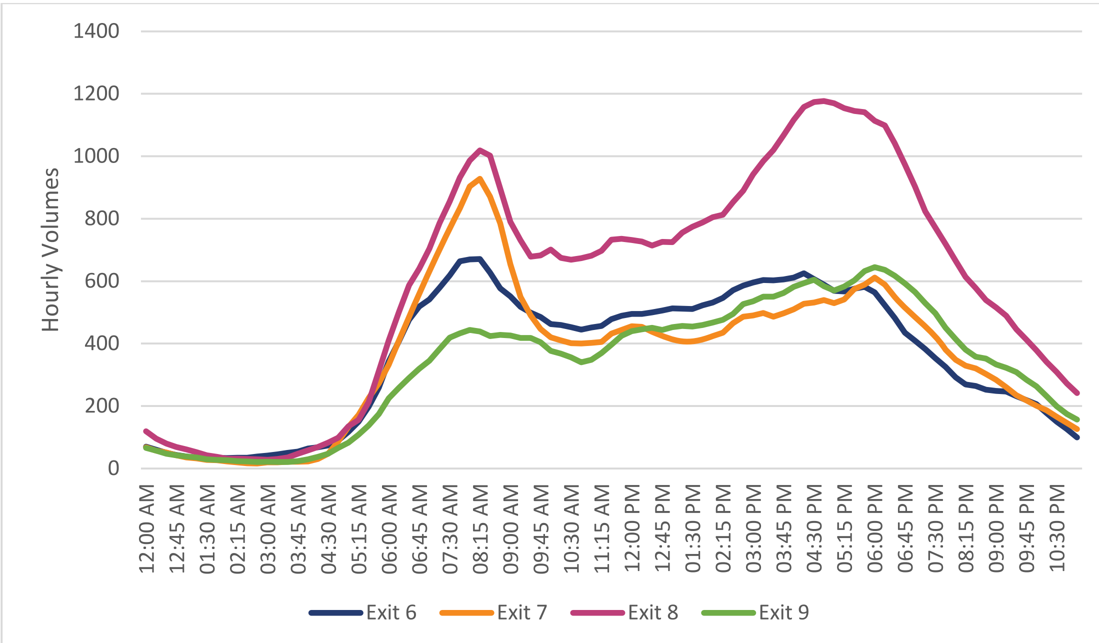


Figure 10 shows weekday hourly on-ramp traffic volumes in the southbound direction. All ramps experience distinct AM and PM peaking. Exit 7 experiences the most traffic during the AM peak. During the PM peak, both Exits 7 and 8 are simularly utilized.

Figure 10: Southbound (On-Ramps) Hourly Weekday Traffic Volumes

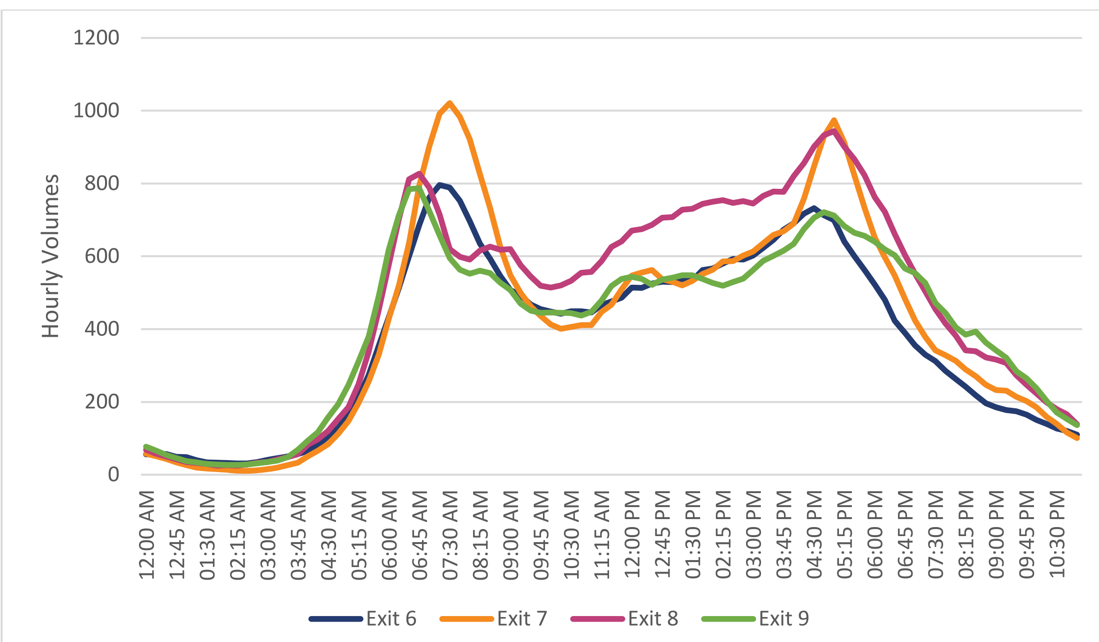
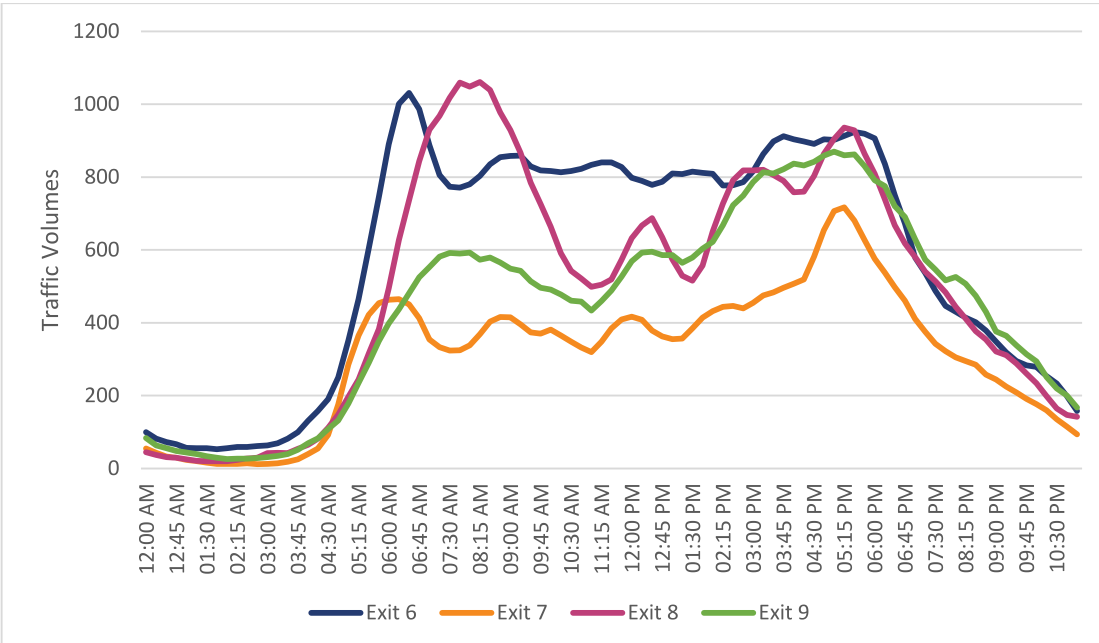


Figure 11 shows weekday hourly off-ramp traffic volumes in the southbound direction. Due to generally low volumes throughout the day, the AM and PM peaks are not as distinct as compared to other ramps. During the AM, Exits 6 and 8 carry the most volume and experience a peak.

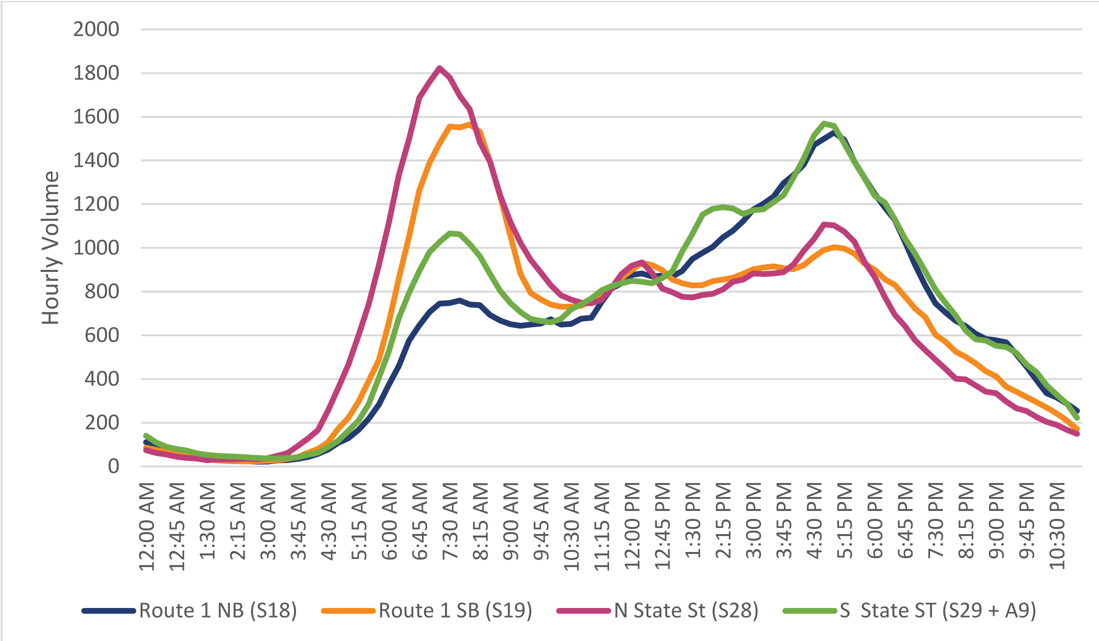
Figure 11: Southbound (Off-Ramps) Hourly Weekday Traffic Volumes



Screenline Counts

Two ATRs were placed on the service roads along I-95: on North State Street (S28) and South State Street (S29). Figure 12 compares the average weekday (Tuesday to Thursday) traffic volume on North and South State Street with that on Route 1 to compare the trends along these parallel roadways that provide local route alternatives to I-95. Traffic on Route 1 in the southbound direction and on North State Street have more pronounced AM peaks, while traffic on Route 1 in the northbound direction and on South State St. have a more pronounced PM peak. Overall, North and South State Streets carry about the same amount of traffic as Route 1.

Figure 12: Bi-Directional Hourly Weekday Volumes (Screenline ATR)



Intersection Counts

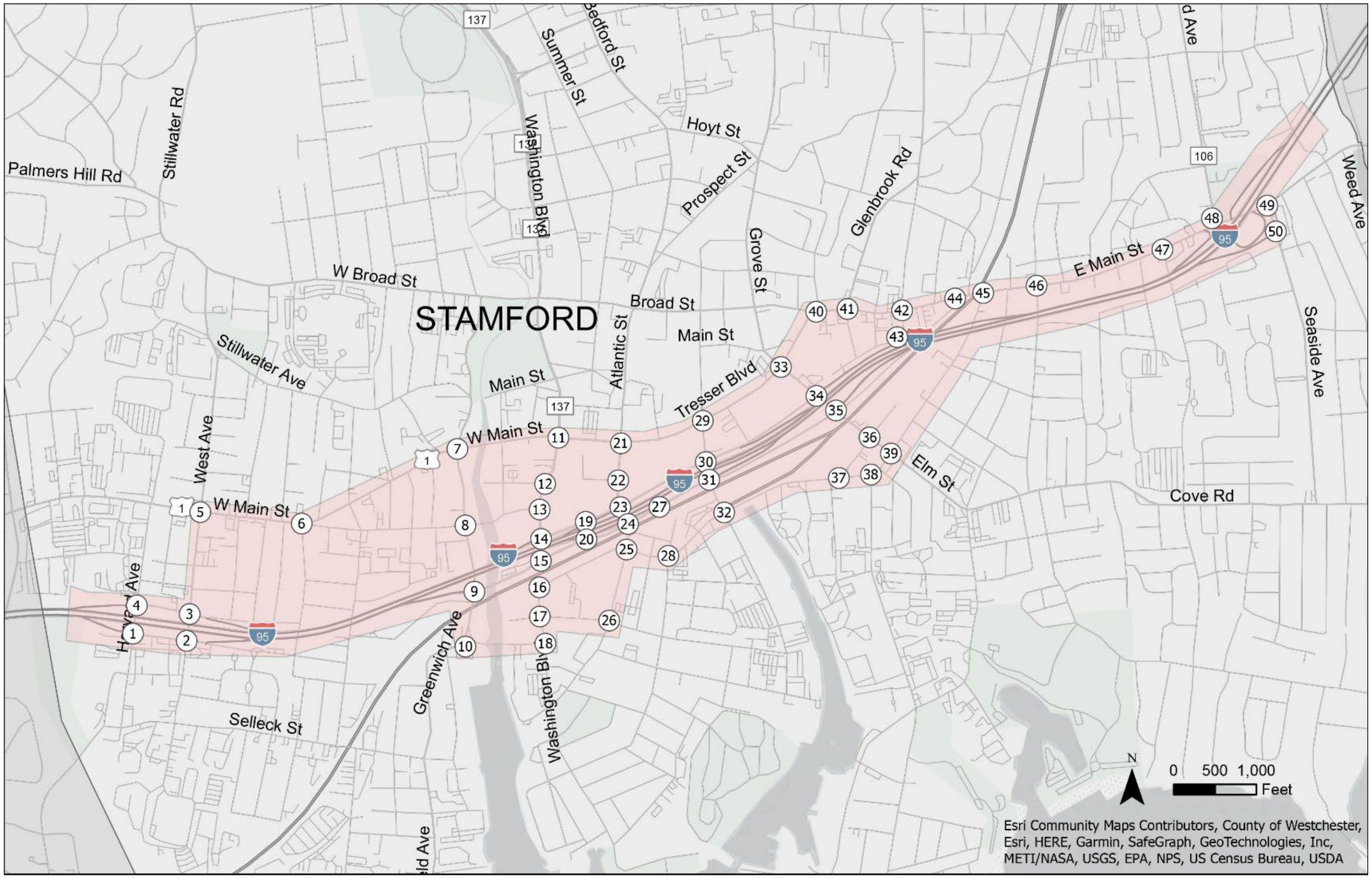
TMCs were collected at each of the 50 study area intersections listed in Table 2 for one weekday and one Saturday from 6AM to 8PM. Map locations can be seen in Figure 4 and numbered in Figure 13.

Table 2: Intersection TMC Count Locations

Study Location No.	City Int. No.	CTDOT Int. No.	Location
T1	255	135-284	Harvard Ave at Baxter Ave / NB I-95 Exit 6 Off-Ramp
T2	252	135-254	West Ave at Baxter Ave / NB I-95 Exit 6 On-Ramp
T3	253	135-253	West Ave at Grenhart Rd / SB I-95 Exit 6 Off-Ramp
T4	254	135-308	Harvard Ave at Grenhart Rd / SB I-95 Exit 6 On-Ramp
T5	267	135-218	U.S. Route 1 (West Main St) at West Ave
T6	263	135-219	U.S Route 1 (West Main St) at Richmond Hill Ave
T7	276	135-250	Greenwich Ave at U.S. Route 1 (West Main St / Tresser Blvd)
T8	248	Not Assigned	Greenwich Ave at Richmond Hill Ave
T9	285	135-278	Greenwich Ave at NB I-95 Exit 7 Off-Ramp / First Stamford Pl
T10	N/A	N/A	Greenwich Ave at Pulaski St
T11	274	135-256	SSR 493 (Washington Blvd) at U.S. Route 1 (Tresser Blvd)
T12	249	135-299	SSR 493 (Washington Blvd) at Division St
T13	246	135-300	SSR 493 (Washington Blvd) at Richmond Hill Ave
T14	245	135-282	SSR 493 (Washington Blvd) at North State St / SB I-95 Exit 7 On-Ramp
T15	284	135-297	SSR 493 (Washington Blvd) at State Route 790 (South State St)
T16	135	135-298	SSR 493 (Washington Blvd) at Station Place
T17	137	Not Assigned	Washington Blvd at Henry St
T18	139	Not Assigned	Washington Blvd at Pulaski St
T19	244	135-904	North State St at Guernsey Ave
T20	283	Not Assigned	State Route 790 (South State Street) at Guernsey Ave
T21	273	135-257	Atlantic St at U.S. Route 1 (Tresser Blvd)
T22	243	Not Assigned	Atlantic St at Federal St
T23	242/282	135-266	Atlantic St at North State St
T24	282	135-269	Atlantic St at State Route 790 (South State St)

Study Location No.	City Int. No.	CTDOT Int. No.	Location
T25	134	135-134	Atlantic St at Station Place & Dock St
T26	138	Not Assigned	Atlantic St at Henry St
T27	280	Not Assigned	State Route 790 (South State St) at NB I-95 Exit 8 Off-Ramp
T28	133	Not Assigned	Pacific St at Dock St
T29	271	135-258	Canal St at U.S. Route 1 (Tresser Blvd)
T30	241	Not Assigned	Canal St at North State St
T31	281	135-268	Canal St at State Route 790 (South State St)
T32	131	Not Assigned	Canal St at Dock St / Jefferson St
T33	233	135-259	Elm St at U.S. Route 1 (Tresser Blvd / East Main St)
T34	232	135-267	Elm St at North State St
T35	231	135-270	Elm St at State Route 790 (South State St)
T36	117 (118A)	Not Assigned	Elm St at Elm Ct / Cherry St
T37	121/130	Not Assigned	Cherry St at Jefferson St
T38	119	Not Assigned	Jefferson St at Magee Ave
T39	118	Not Assigned	Elm St at Jefferson St / Myrtle Ave
T40	219	135-260	U.S. Route 1 (East Main St) at Broad St
T41	218	135-261	U.S. Route 1 (East Main St) at Glenbrook Rd
T42	216	135-262	U.S. Route 1 (East Main St) at Lafayette St
T43	N/A	N/A	Lafayette St at North State St & South State St
T44	214	135-263	U.S. Route 1 (East Main St) at North State St
T45	212	135-264	U.S. Route 1 (East Main St) at Myrtle Ave
T46	210	135-265	U.S Route 1 (East Main St) at Lockwood Ave
T47	205	135-217	U.S Route 1 (East Main St) at Blachley Rd
T48	203	135-221	U.S. Route 1 (East Main St) at U.S. Route 106 (Courtland Ave) / SB I-95 Exit 9 Ramps
T49	202	135-222	U.S. Route 1 (East Main St) at Seaside Ave / NB I-95 Exit 9 On-Ramp
T50	201	135-283	Seaside Ave at NB/SB I-95 Exit 9 Ramps

Figure 13: Intersection Count Locations



Legend

Study Area

Study Intersections

4.1.2 *Commercial Vehicle Counts*

Data shows that heavy vehicle percentage of total traffic on I-95 varies between 12 and 16 percent on a weekday and between 5 and 7 percent on a Saturday. The percentage can fluctuate significantly during a typical weekday, with the percentage of heavy vehicles peaking around 6 AM in the northbound direction and around midday in the southbound direction. The total volume of heavy vehicles is generally consistent throughout an average weekday, with peaks in the AM and troughs after dark. The percentage of heavy vehicles is high (37% in the northbound direction) at 6AM due to fewer passenger cars on the roadway. Figure 14 shows the distribution of heavy vehicles as a percentage of all vehicles on an average weekday (Tuesday to Thursday) at two locations on the interstate. The other four mainline locations experience a similar trend based on the direction of flow.

Figure 14: Distribution of Heavy Vehicles on Weekdays

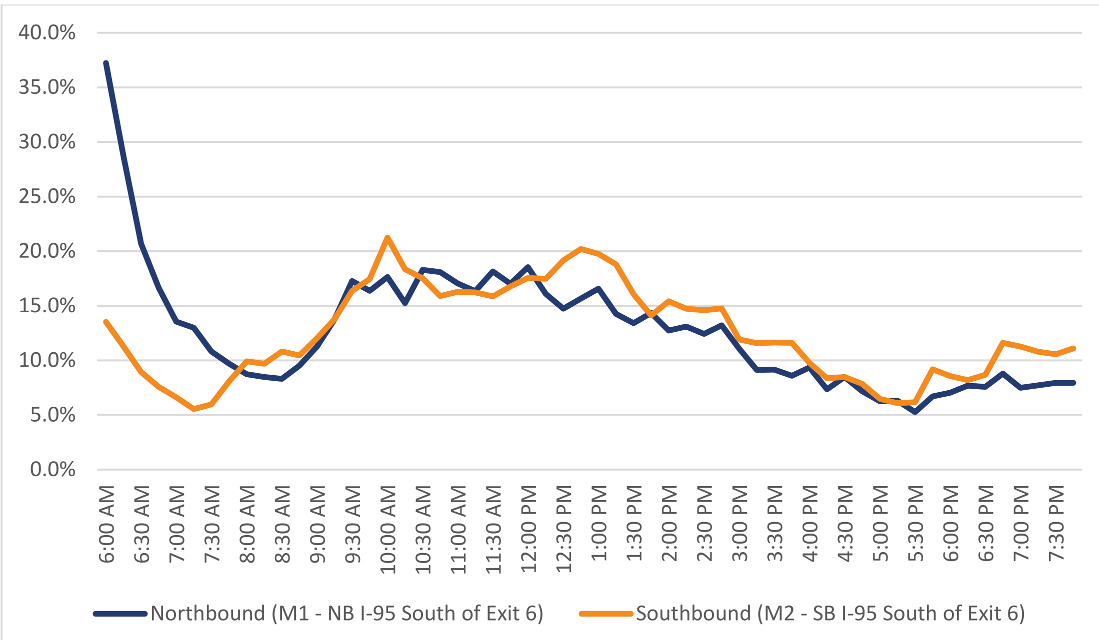


Table 3 shows the average daily volume and heavy vehicle percentage at each mainline count location during an average weekday (Tuesday to Thursday) and Saturday. The percentage is as high as 15.9% on an average weekday and 6.8% on a Saturday at M4. In general, the percentage of heavy vehicles is higher in the southbound direction.

Table 3: Average Daily Volume of Heavy Vehicles at Maline Count Locations

Intersection	Avg. Weekday		Saturday	
	Heavy Vehicle Volume	% Heavy Vehicle	Heavy Vehicle Volume	% Heavy Vehicle
NB I-95 South of Exit 6	6,756	12.1%	934	5.3%
SB I-95 South of Exit 6	6,988	12.1%	1,044	5.5%
NB I-95 South of Exit 7 On-Ramp	6,337	15.4%	862	6.1%
SB I-95 South of Exit 7 Off-Ramp	6,665	15.9%	1,017	6.8%
NB I-95 North of Exit 9	6,994	11.9%	954	4.9%
SB I-95 South of Exit 9 Off-Ramp	6,757	13.3%	1,008	5.9%

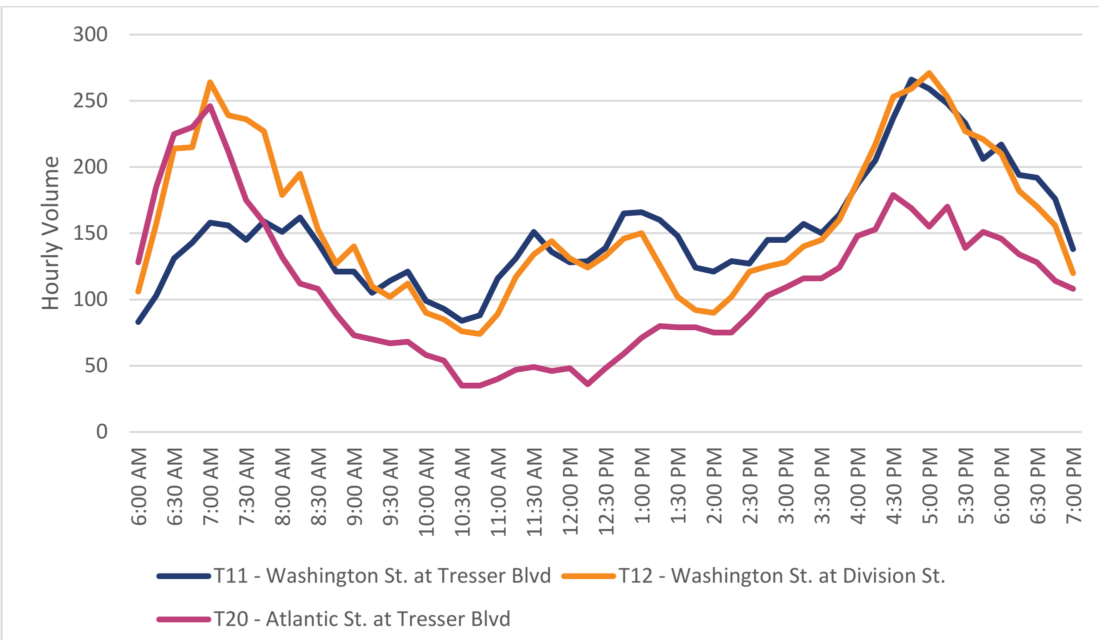
The two service roads along the interstate carry some of the heavy vehicle traffic leading into and out of the interstate. The heavy vehicle percentage on these roads varies from 3.5% to 7.0%, except for two intersections when it is as high as 12.3% and 16.3%. The two intersections are just to the south and north of the Stamford Transportation Center (STC) respectively, which carry significant bus traffic.

4.2 Pedestrian Traffic Data

Based on the Turning Movement Counts (TMC) gathered on weekdays between 6AM and 8PM, the intersection of Washington Blvd. with Tresser Blvd (T11) and Division St. (T12) carry the most pedestrian traffic with more than 2,000 pedestrian crossings. The intersections of Atlantic St. with Tresser Blvd (T21) and Federal St. (T22) experienced approximately 2,000 and 1,500 pedestrian crossings, respectively. The pedestrian crossing on South State St. (T20) between the Stamford Train Station and Stamford Transportation Center experienced more than 1,500 daily crossings.

Figure 15 shows the number of average weekday (Tuesday to Thursday) pedestrian crossings at three intersections. The number increases during the morning and evening peak periods, with a small increase during midday during lunch. T20, which is adjacent to the train station, does not experience any increase during midday.

Figure 15: Hourly Weekday Pedestrian Crossings



4.3 Bicycle Traffic Data

Based on the TMC data collected at 50 intersections in the study area, almost 50% of the intersections experience less than 20 bicycle crossings in the 14-hour period between 6AM and 8PM on an average weekday. However, there are two segments which experience a relatively high number of bicycle crossings:

1. East Main Street when it intersects North State St, Myrtle Ave and Lockwood Ave. These three locations experience 61, 53 and 59 bicycle crossings, respectively.
2. Elm Street when it intersects North State St, Cherry St and Myrtle Ave. These locations experience 50, 55 and 47 bicycle crossings, respectively.

4.4 Travel Time Reliability

The floating car method was used to collect travel time data on Tuesday, April 26, 2022 from 6AM-11AM and 3PM-8PM. The study area on I-95 was 3.6 miles long between Laddins Rock Rd (south of Exit 6) and Brookside Ave (north of Exit 9).

The results of the study, shown in Table 4, indicate that the average off-peak free flow travel time on the 3.6-mile segment of I-95 was approximately 3.2 minutes in both northbound and southbound directions. However, travel time varied significantly during the peak hours. In the non-peak directions (northbound in the AM and southbound in the PM), the travel time was consistent at approximately 4.2 minutes. However, in the peak directions (southbound in the AM and northbound in the PM), travel time ranged from approximately 15 minutes to over 20 minutes, an increase of approximately 400 percent to 600 percent compared to the off-peak free flow travel time. The travel speeds in the northbound direction ranged from 68 mph during free flow to 13 mph during peak congestion. The travel speeds in the southbound direction ranged from 69 mph during free flow and 9 mph during peak congestion.

Table 4: I-95 Mainline Travel Times

Travel Direction	Off-Peak	AM Peak (6AM-11AM)		PM Peak (2PM-8PM)	
	Travel Time (minutes)	Travel Time (minutes)	Change from Off- Peak	Travel Time (minutes)	Change from Off- Peak
Northbound	3.2	4.2	32%	15.7	391%
Southbound	3.1	22.4	623%	4.2	36%

Additionally, Google API was used to obtain historical travel times along I-95 within the study area, with data available at 15-minute intervals. Figure 16 and Figure 17 compare the travel times obtained by the floating car method and Google API for northbound AM and southbound PM on April 26, 2022. As the figures show, the observed travel times (floating car) and historical travel times (Google API) are very similar, indicating that Google API data can be used as a reliable supplement to the floating car method.

Figure 16: Southbound I-95 Mainline Travel Times Between Exits 9 & 6 on April 26, 2022

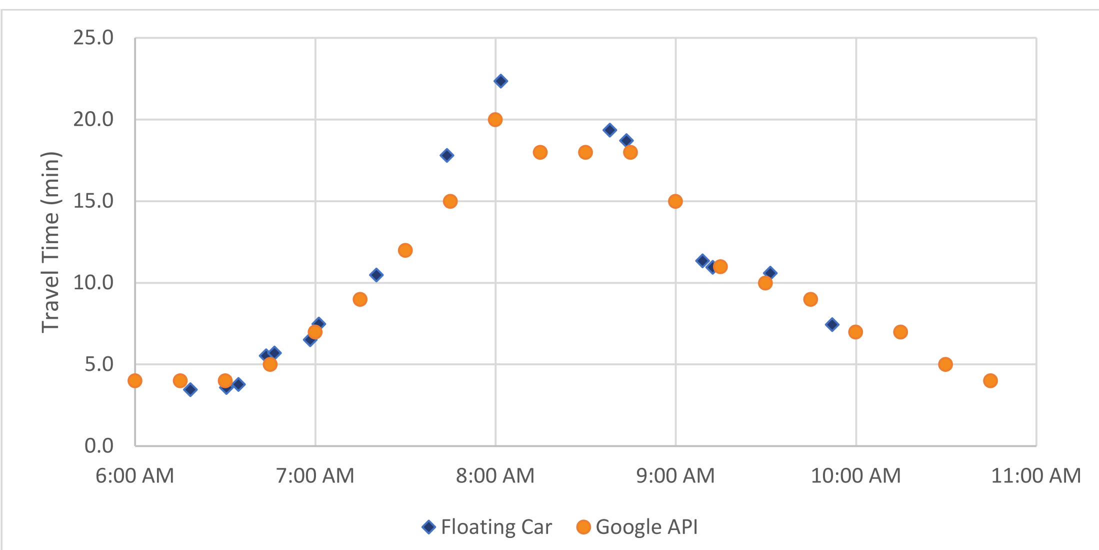
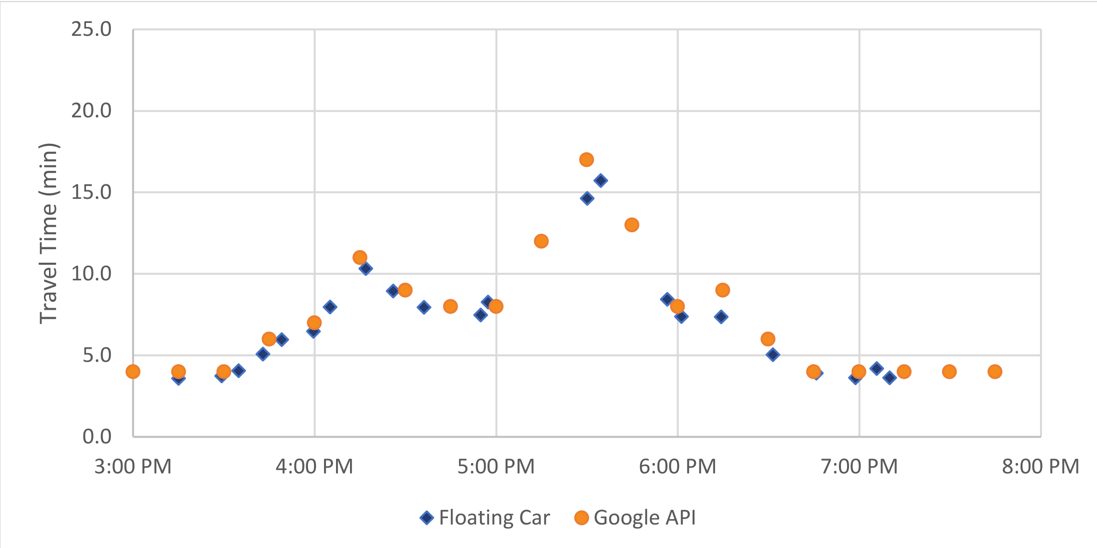
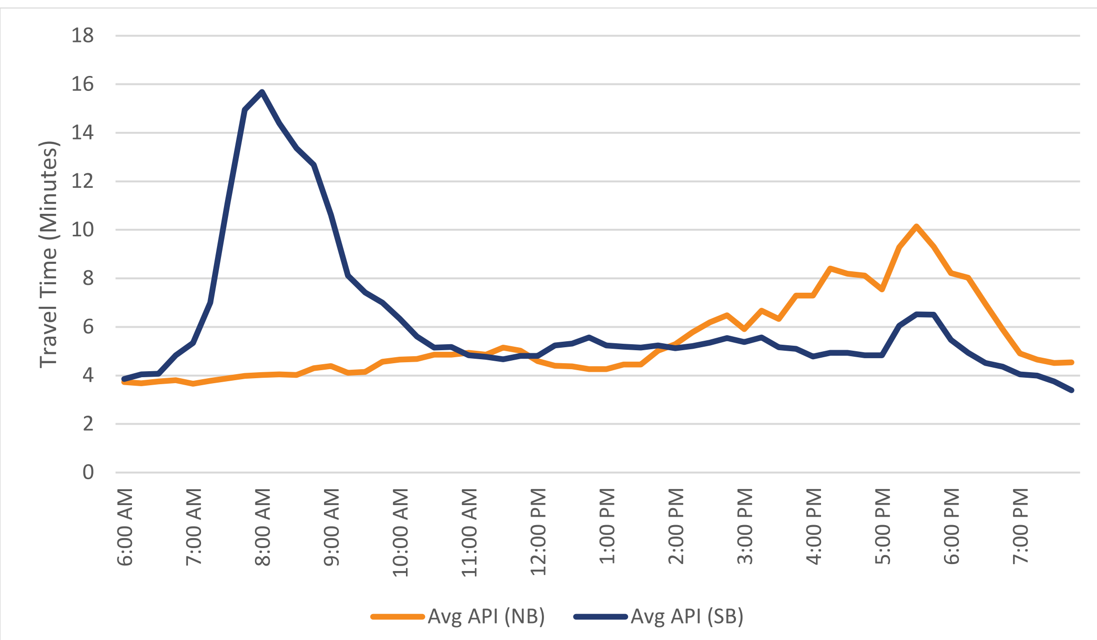


Figure 17: Northbound I-95 Mainline Travel Times Between Exits 6 & 9 on April 26, 2022



Average weekday (Tuesday to Thursday) travel times were calculated using Google API data from February to May 2022. Figure 18 shows the average travel time over the 3.6-mile segment on I-95. The free flow travel time for this section is close to four minutes in either direction. However, the travel time increases to 16 minutes in the southbound direction during the AM peak and 10 minutes in the northbound direction during the PM peak.

Figure 18: Average Google API Travel Times for Weekdays from February to May 2022



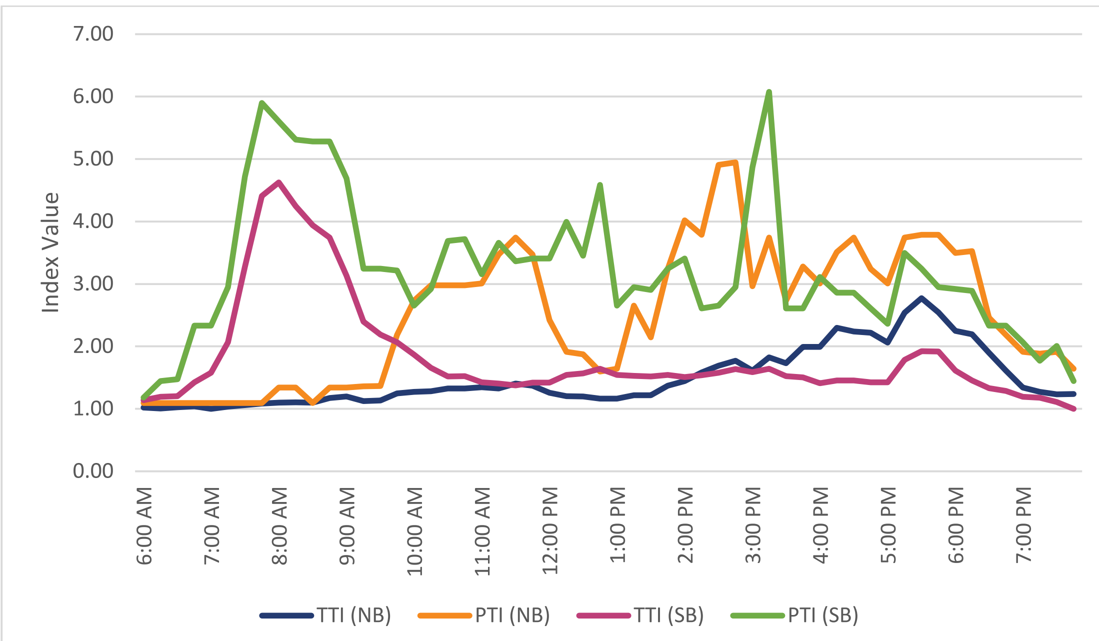
Travel time reliability can be used to calculate travel time delays. This section reviews two such tools to analyze travel time reliability on the 3.6-mile segment of I-95. Travel Time Index (TTI) establishes a relationship between free flow and peak hour travel time and is the ratio of travel time to free flow travel time. For example, if it takes 10 minutes to travel a section of roadway during free flow conditions and takes

17 minutes to travel the same section during a particular time of the day, the TTI in this scenario would be 1.7 (17 divided by 10).

The second tool is the Planning Time Index (PTI), which considers days with the highest delay and represents the time a traveler should allow to assure on-time arrival 95 percent of the time. For example, for a PTI of 1.5 for the previous example, the traveler should plan for a trip of 15 minutes.

Figure 19 shows the TTI and PTI for northbound and southbound traffic on I-95 on weekdays (Tuesday to Thursday). In the northbound direction, the TTI can be as high as 2.77, while the PTI can be as high as 3.55. This translates to a travel time as high as 10.1 minutes during peak traffic conditions, but the traveler should plan for at least 13 minutes to reach their destination on time 95 percent of the time. In the southbound direction, the TTI can be as high as 4.63, while the PTI can be as high as 5.9. This translates to a traveler planning for 20 minutes for a free flow trip of 3.4 minutes to reach their destination on time 95 percent of the time.

Figure 19: TTI and PTI for I-95 Mainline on Weekdays



4.5 Origin-Destination Data

StreetLight data was used to study origin-destination (OD) patterns in the study area. StreetLight uses data from mobile phones and other connected devices and algorithmically transforms it into normalized travel patterns. Data was obtained for March and April 2022, the most recently available data when this analysis was conducted. Analysis was done for an average weekday (Tuesday to Thursday).

4.5.1 I-95 OD Analysis

The following figures show the number of vehicles travelling in the northbound and southbound direction on I-95 during the AM (6AM-11 AM) and PM (2PM-8 PM) peak periods.

Figure 20 shows the destinations for AM peak traffic on I-95 north of Exit 9 travelling southbound. The largest proportion of traffic (43%) continues along the I-95 mainline south of Exit 6, while the remaining 57% has local destinations at Exits 9, 8, 7 or 6. The most prominent local destination is Exit 8 (Elm Street) with 21% of traffic.

Figure 20: Southbound Traffic North of Exit 9 Destinations During the AM Peak

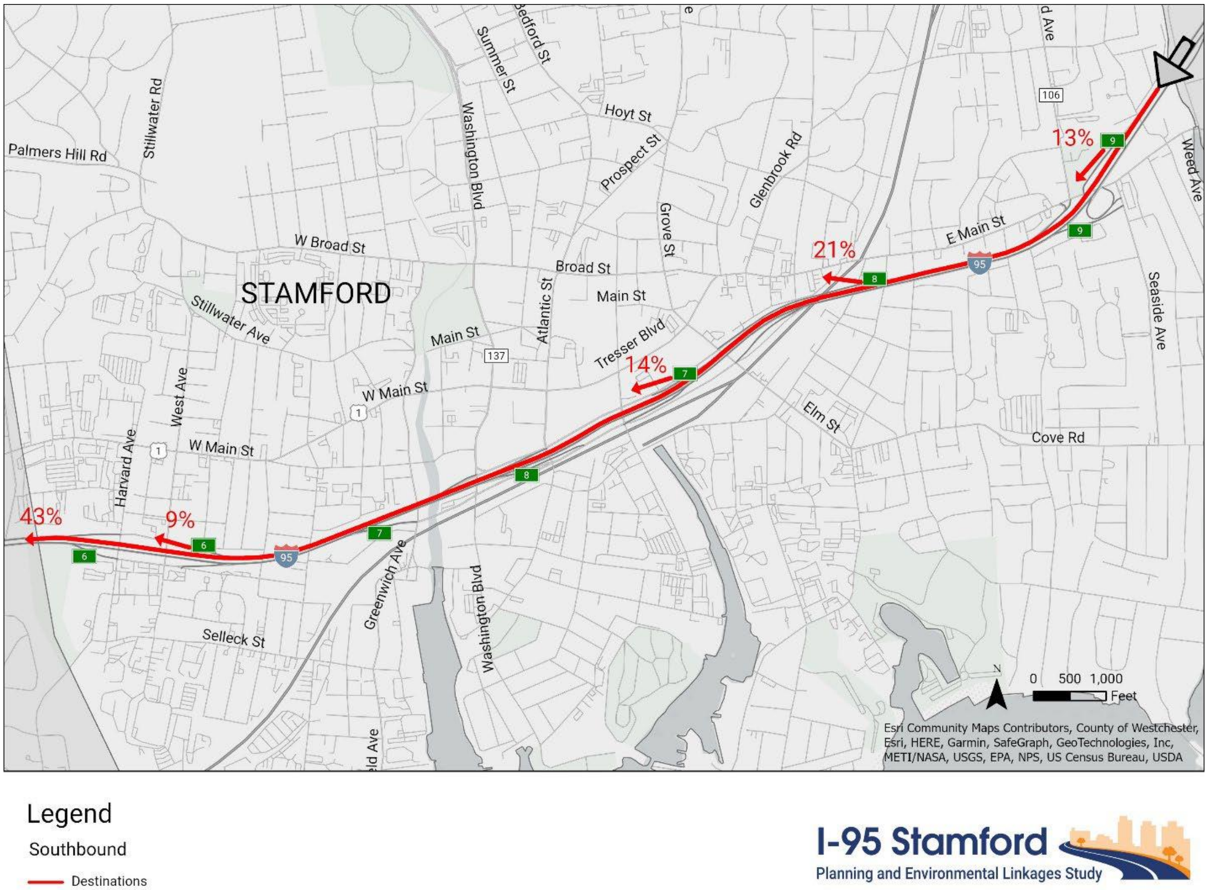


Figure 21 shows the origins for traffic during the AM Peak on the I-95 mainline south of Exit 6 travelling southbound. The largest proportion of traffic (48%) originates from the I-95 mainline north of Exit 9, while the remaining 52% has local origins at Exits 9, 8, 7 or 6. The most prominent local origin is Exit 8 (Atlantic Street) with 15% of traffic.

Figure 21: Southbound Traffic South of Exit 6 Origins During the AM Peak

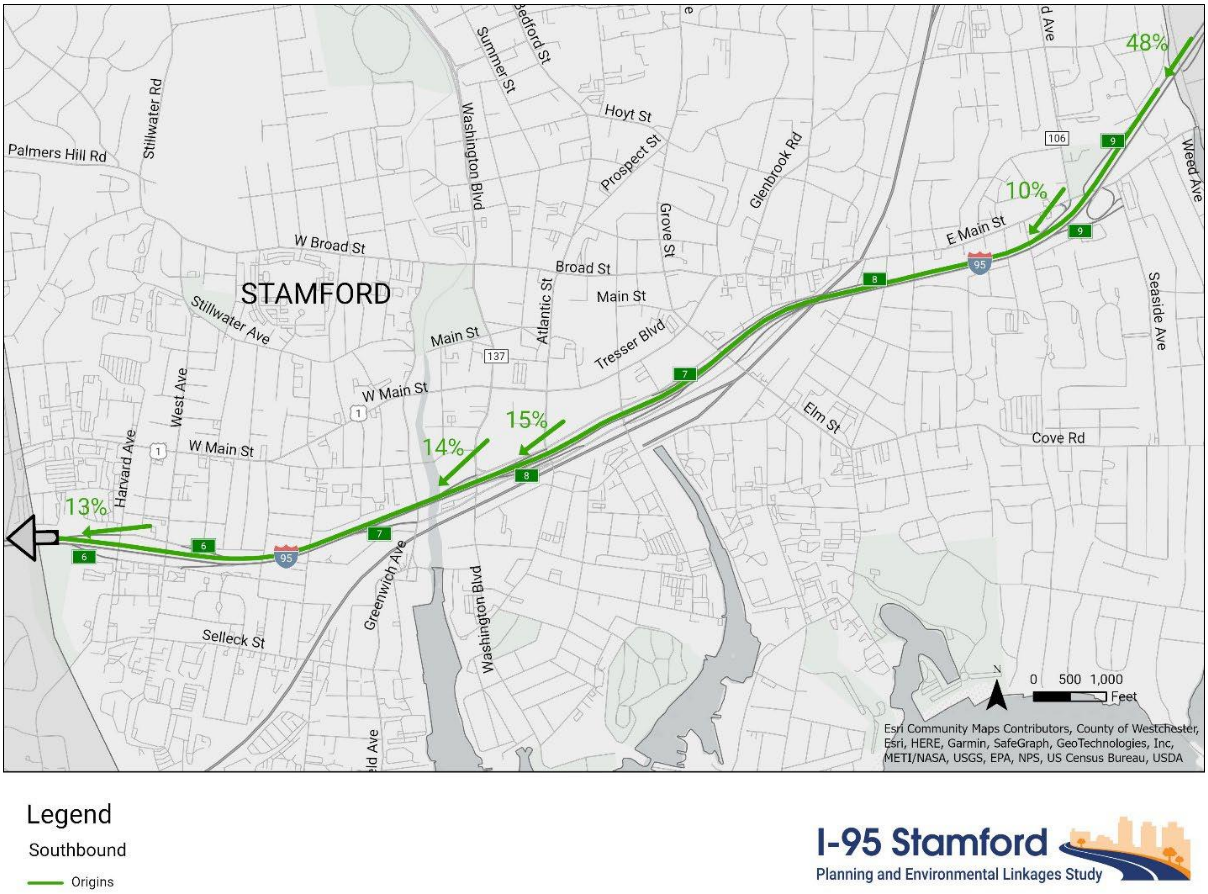


Figure 22 shows the destinations for traffic during the PM Peak on the I-95 mainline south of Exit 6 travelling northbound. The majority of traffic (51%) continues along the I-95 mainline north of Exit 9, while the remaining 49% has local destinations at Exits 6, 7, 8 or 9. The most prominent local destination is Exit 8 (Canal Street) with 18% of traffic.

Figure 22: Northbound Traffic South of Exit 6 Destinations During the PM Peak

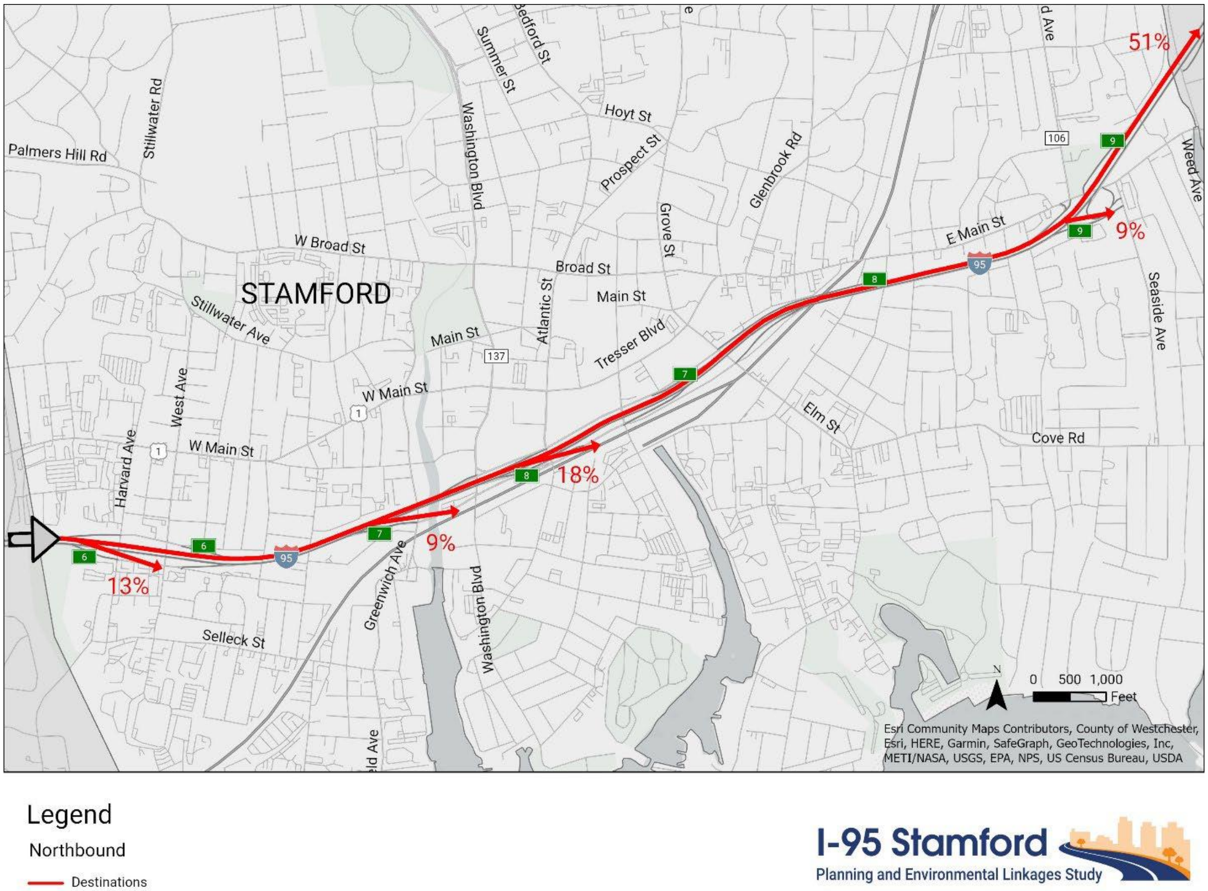
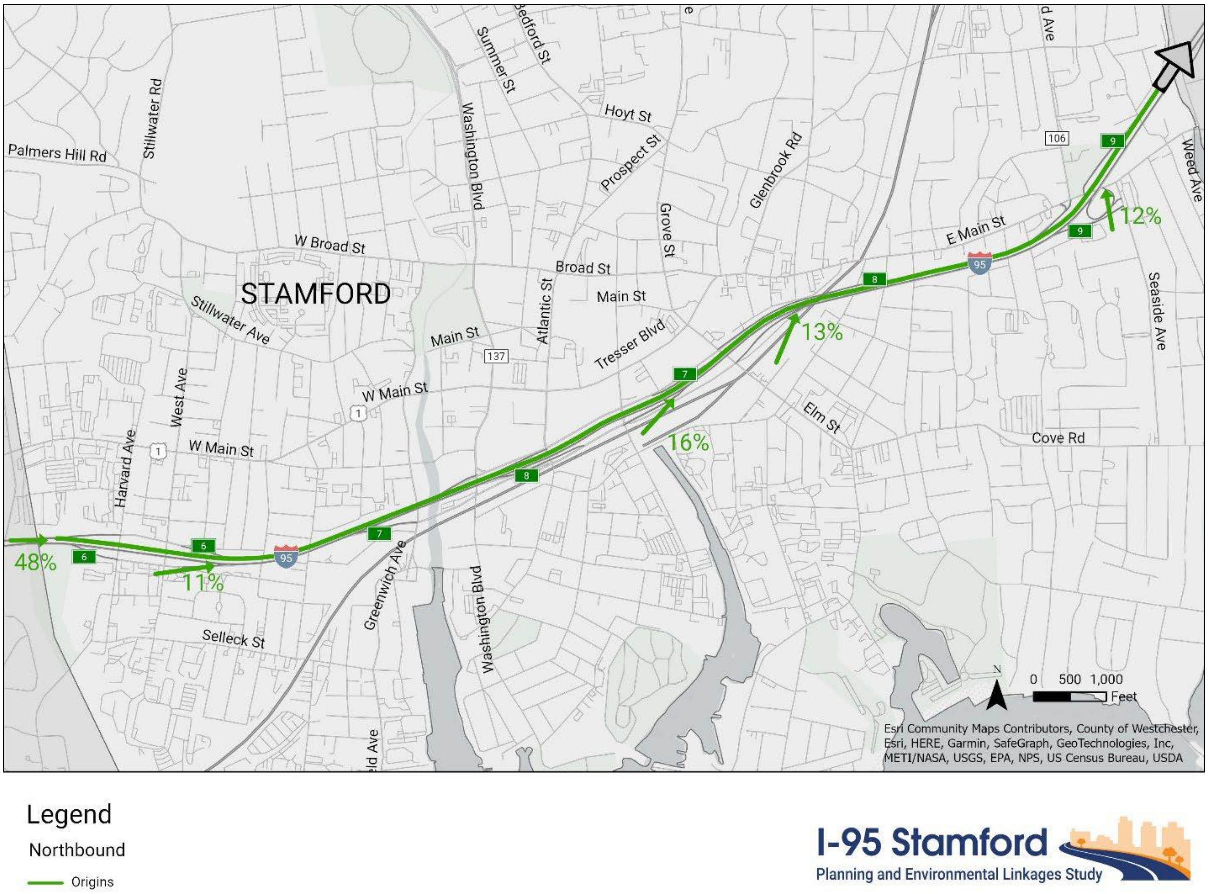


Figure 23 shows the origins for traffic during the PM Peak on the I-95 mainline north of Exit 9 travelling northbound. The largest proportion of traffic (48%) originates from the I-95 mainline south of Exit 6, while the remaining 52% has local origins at Exits 6, 7, 8 or 9. The most prominent local origin is Exit 7 (Canal Street) with 16% of traffic.

Figure 23: Northbound Traffic North of Exit 9 Origins During the PM Peak



During both peak periods and directions, the split between local and through traffic along I-95 in Stamford is around 50%. Additionally, the most prominent local origins/destinations have ramp terminals along a 0.6 mile stretch in downtown Stamford (Elm Street to Atlantic Street).

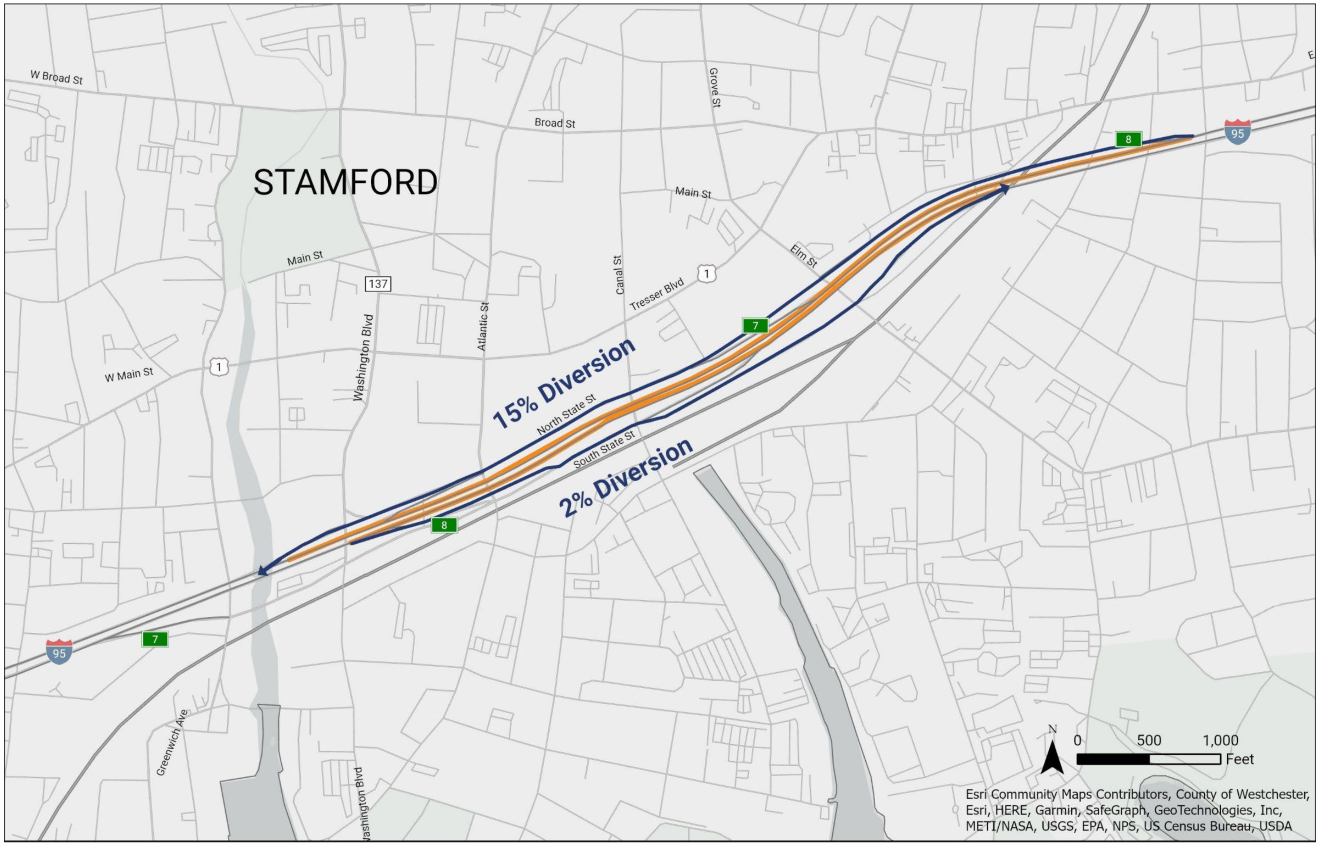
4.5.2 Queue Jumping

Additionally, StreetLight data was used to assess queue jumping. Queue jumping is an instance when a road user takes a detour to avoid traffic or congestion on their preferred route. In the case of I-95 passing through Stamford, road users might exit the interstate only to enter it again using another street if they feel that the alternative route has less congestion than their primary route. This makes traffic worse on local streets, reducing travel speeds and increasing congestion.

As shown in Figure 24, in the southbound direction, some users exit the interstate at Exit 7 (Elm Street), travel on North State St, and re-enter I-95 at Exit 8 after Atlantic St. During the AM peak hour, 10% of I-95 traffic queue jumps, while 14% queue jump in the PM. The average southbound total daily percentage of queue jumping is about 15% of vehicles.

There are significantly fewer queue jumpers in the northbound direction, with 1.3% and 2.2% queue jumping in the morning and evening, respectively. The average northbound total daily percentage of queue jumping is about 2% of vehicles.

Figure 24: I-95 Mainline Queue Jumping along North and South State Streets



Legend
- I-95 Mainline Path
- State Street Local Road Path

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4.6 Existing Intersection Traffic Operations

This section discusses the development of traffic simulation models to evaluate traffic operations within the study area as well as the result of the existing conditions analysis for mainline (I-95) segments, ramps, and intersections.

4.6.1 Intersection Operations

Traffic operations were evaluated for the study area local roadway intersections during the weekday morning, weekday midday, weekday evening, and Saturday midday peak hours. Peak hour volumes at each intersection were tabulated and the entire roadway network was balanced based on peak hour volumes at each intersection. These volumes can be found in Appendix A. This method provides a conservative result for each intersection. Capacity and queue analyses were conducted using Trafficware Synchro Studio 11 – Traffic Analysis Software based on the methodology provided in the *Highway Capacity Manual (HCM), 6th Edition*. The analyses quantify the operations of the intersections under the existing conditions to identify locations that operate well and under capacity, and those that are operating close to or over capacity.

4.6.2 Intersection Analysis Methodology

An intersection's operational condition is assessed by average control delay per vehicle and volume to capacity ratio (V/C). Average control delay is measured in seconds of delay that occurs at an intersection, per vehicle, due to the traffic control. The V/C ratio is a measurement of the volume of particular traffic movement or approach in comparison with the capacity of the movement/approach. V/C ratios closer to zero represent that the approach has significant capacity remaining while approaches with V/C values approaching or exceeding 1.0 indicates that the approach is near or at capacity and not able to accommodate the traffic flow.

The average control delay and V/C ratio are combined to assign a LOS to a particular intersection or intersection approach movement. LOS is defined by HCM, using average control delay and V/C, to assign letter grades A through F to indicate the efficiency of the traffic control at an intersection. The definitions of the letter grades in terms of average control delay and V/C are provided in Table 5 below.

Table 5: Highway Capacity Manual (HCM) Level of Service (LOS) Definitions

Level of Service	**Signalized Intersection Criteria** Average Control Delay (Seconds per Vehicle)	**Unsignalized Intersection Criteria** Average Control Delay (Seconds per Vehicle)	**V/C Ratio >1.00[a]**
A	≤10	≤10	F
B	>10 and ≤20	>10 and ≤15	F
C	>20 and ≤35	>15 and ≤25	F
D	>35 and ≤55	>25 and ≤35	F
E	>55 and ≤80	>35 and ≤50	F
F	>80	>50	F

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

Source: *Highway Capacity Manual, 6th Edition: A Guide for Multimodal Mobility Analysis.* Washington, D.C.: Transportation Research Board, 2016. Exhibit 19-8, Pg. 19-16.

In general, intersections that exhibit LOS A or B are considered to have excellent to good operating conditions with little congestion or delay. LOS C indicates an intersection with acceptable operations. LOS D indicates an intersection that has tolerable operations with average delays approaching one minute. Intersections with LOS E or F are operating with poor or failing conditions and typically warrant a more thorough review and possible improvement to mitigate the capacity issues.

In addition to LOS, the HCM methodology also allows for the calculation of queues. Queues are the expected length of vehicles waiting at an intersection due to the delay incurred by the traffic control. The 50^{th} percentile queues, or average queues, are the average number of vehicles expected on an approach at any given time. The 95^{th} percentile, or design queues, are the maximum expected queues on a given approach. For unsignalized intersections, queues are quantified for 95^{th} percentile (design) queues. For signalized intersections, queues are quantified by 95^{th} percentile (design) and 50^{th} percentile (average) queues.

4.6.3 Intersection Analysis Results

The LOS, volume-to-capacity ratio, and queue results for the intersections under 2022 Existing conditions are presented in Appendix A. The tables depict the results by lane group and overall intersection for the weekday AM, weekday midday, weekday evening, and Saturday midday peak hours. The results are also summarized geographically for weekday AM and PM in Figure 25 and Figure 26, respectively.

Figure 25: Intersection Level of Service – Weekday AM

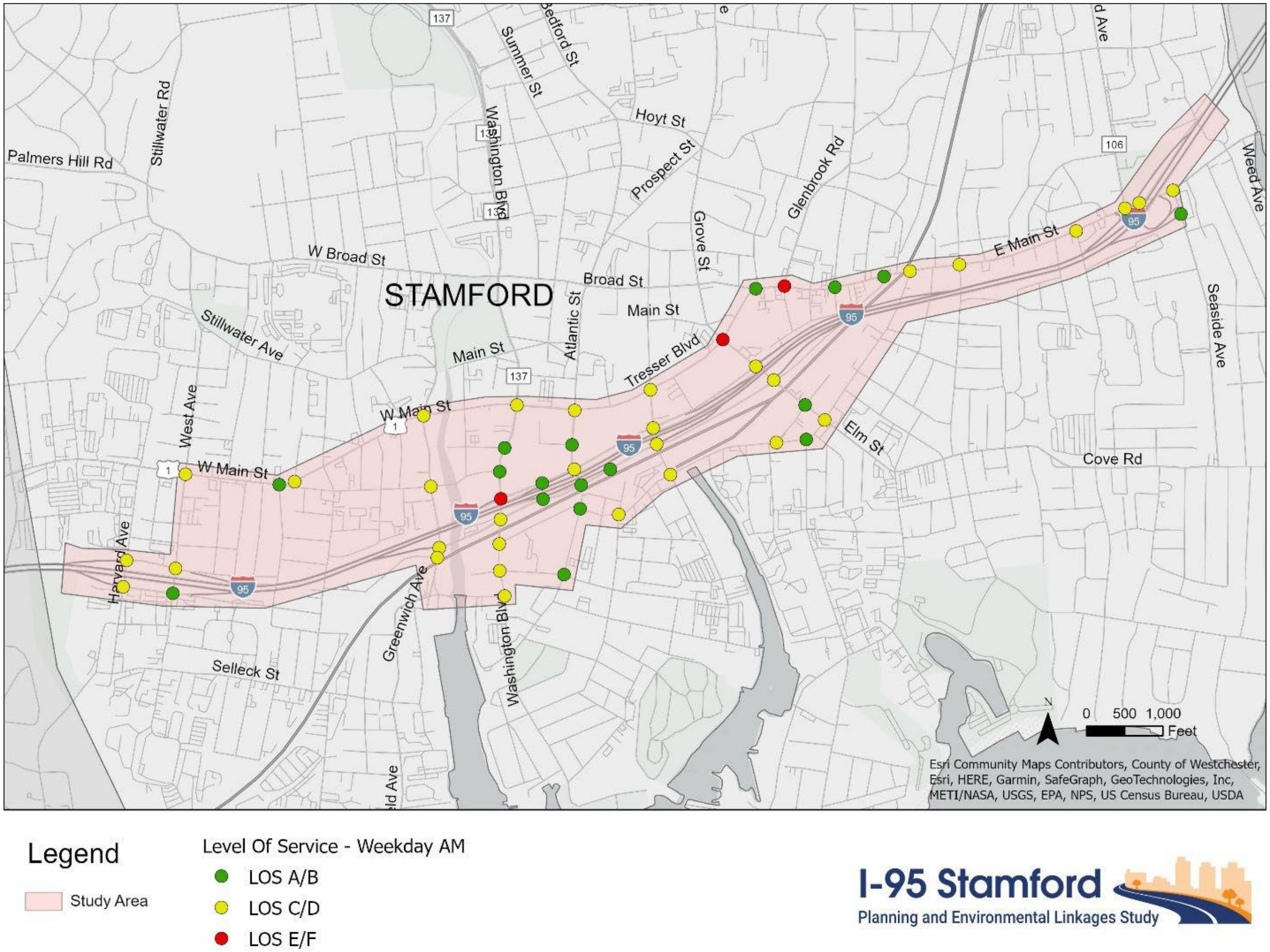

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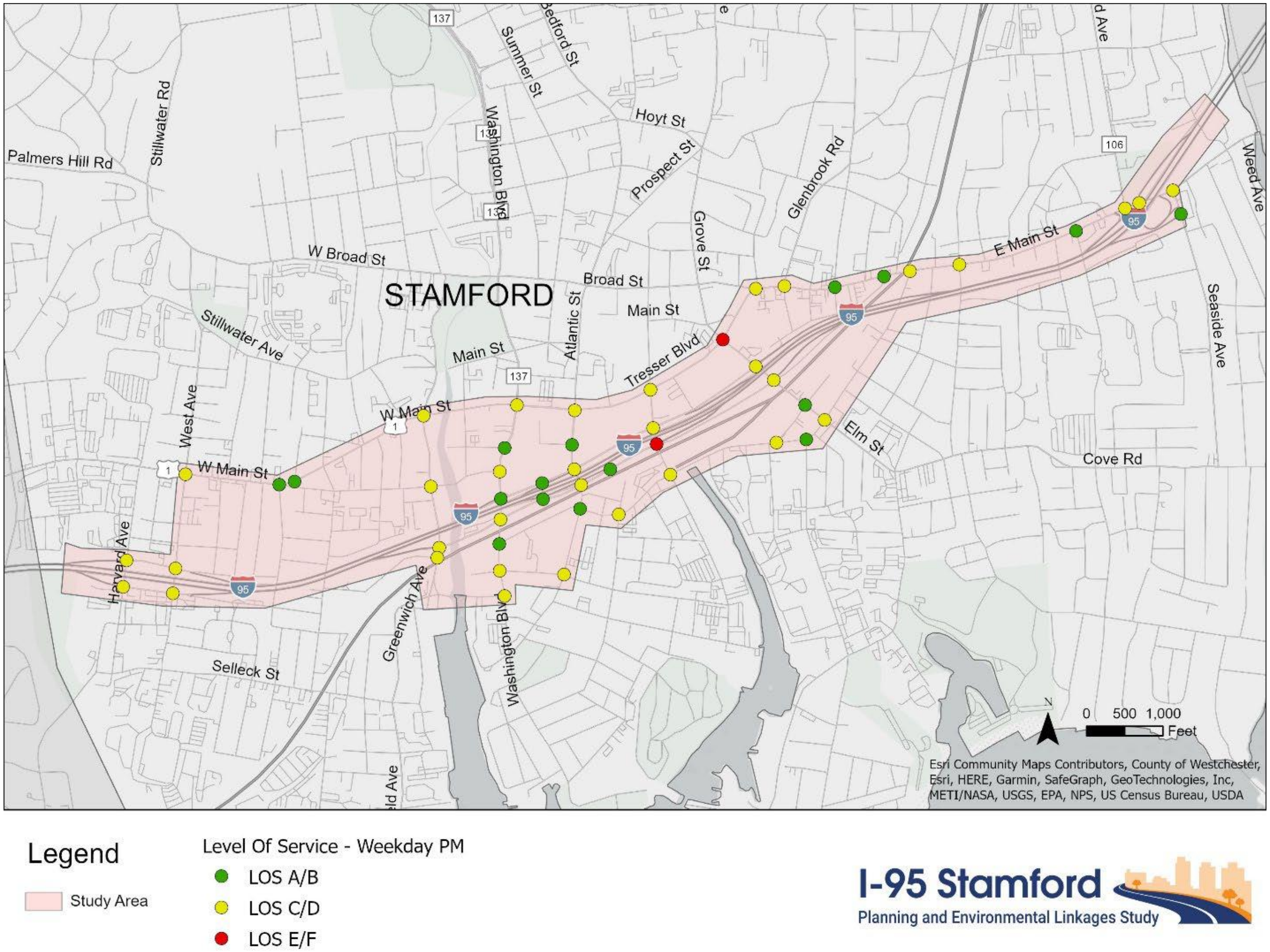
Figure 26: Intersection Level of Service – Weekday PM



Many of the study area intersections operate at an acceptable overall LOS D or better with the following exceptions, where either the V/C ratio exceeds 1.0 or delays surpass the LOS E or F thresholds:

- **Int. No. 14 – Special Service Road (SSR) 493 (Washington Boulevard) at North State Street & I-95 SB Exit 7 On-Ramp:** LOS E operations with overall intersection average delays of 60.1 seconds during the weekday AM peak hour.
- **Int. No. 31 - Canal Street at State Route 790 (South State Street) & I-95 NB Exit 7 On-Ramp:** LOS E operations with overall intersection average delays of 79.3 seconds during the weekday PM peak hour.
- **Int. No. 33 - Elm Street at U.S. Route 1 (Tresser Boulevard / East Main Street):** LOS E operations during the weekday AM and weekday PM peak hours with with overall intersection average delays of 66.7 and 60.5 seconds, respectively.
- **Int. No. 41 - U.S. Route 1 (East Main Street) at Glenbrook Road & Clarks Hill Avenue:** LOS E operations with overall intersection average delays of 61.1 seconds during the weekday AM peak hour.
- **Int. No. 10 - Greenwich Avenue at Pulaski Street & Davenport Street (All-Way Stop Control):** LOS F operations during the weekday AM and weekday PM peak hours with overall intersection average delays of 62.5 and 132.7 seconds, respectively.

A majority of the intersection queue lengths are accommodated within available storage. However, the list below summarizes locations where intersection through movements significantly exceed available storage and therefore queues extend into adjacent signalized intersections. In addition to these locations, there are queues that exceed available storage for dedicated left or right turn lanes extending into adjacent through lanes and/or signalized intersections at select intersections during select peak hours. Queues exceeding available storage, including the through movements listed below and the left and right turn movements, are highlighted in Table A-2 of Appendix A.

- **Int. No. 5 - U.S. Route 1 (West Main Street) at West Avenue:** eastbound through-right queues extend approximately 180 feet past available storage during the weekday PM peak hour.
- **Int. No. 7 - Greenwich Avenue at U.S. Route 1 (West Main Street / Tresser Boulevard):** eastbound through-right queues extend 110 to 125 feet past available storage during the weekday PM and Saturday midday peak hours, respectively.
- **Int. No. 12 – Special Service Road (SSR) 493 (Washington Boulevard) at Division Street & Driveway:** northbound through-right queues extend approximately 270 feet past available during the weekday PM peak hour.
- **Int. No. 15 - Special Service Road (SSR) 493 (Washington Boulevard) at State Route 790 (South State Street):** northbound queues extend approximately 180 and 100 feet past available storage during the weekday AM and weekday PM peak hours, respectively, and southbound through queues extend approximately 215 feet past available storage during the weekday AM peak hour.
- **Int. No. 31 - Canal Street at State Route 790 (South State Street) & I-95 NB Exit 7 On-Ramp:** eastbound through-left queues extend approximately 285, 555, and 240 feet past available storage during the weekday AM, weekday PM, and Saturday midday peak hours, respectively
- **Int. No. 33 - Elm Street at U.S. Route 1 (Tresser Boulevard / East Main Street):** southbound through-right queues approximately 200 and 230 feet past available storage during the weekday AM and PM peak hours, respectively.
- **Int. No. 35 - Elm Street at State Route 790 (South State Street) & I-95 NB Exit 8 On-Ramp:** northbound queues extend approximately 265 and 215 feet past available storage during the weekday AM and Saturday midday peak hours, respectively, and southbound through queues extend approximately 355 and 260 feet past available storage in the weekday PM, and Saturday midday peak hours, respectively.

- **Int. No. 39 - Elm Street at Jefferson Street / Myrtle Avenue:** eastbound through-right queues extend approximately 165 feet past available storage in the weekday PM peak hour and southbound through-right queues extend approximately 285 feet past available storage during the weekday PM peak hour.
- **Int. No. 40 - U.S. Route 1 (East Main Street) at Broad Street & Lindale Street:** westbound through-right queues extend approximately 255 and 130 feet past available storage during the weekday PM and Saturday midday peak hours, respectively.
- **Int. No. 44 - U.S. Route 1 (East Main Street) at North State Street & Plaza Driveway:** westbound queues extend approximately 115, 105, and 110 feet past available storage during the weekday AM, weekday PM, and Saturday midday peak hours, respectively.
- **Int. No. 45 - U.S. Route 1 (East Main Street) at Myrtle Avenue:** eastbound queues extend approximately 235, 485, and 325 feet past available storage during the weekday AM, weekday PM, and Saturday midday peak hours, respectively.
- **Int. Nos. 48A & 48B - U.S. Route 1 (East Main Street) at State Route 106 (Courtland Avenue) & I-95 Exit 9 On-Ramp:** westbound through queues extend approximately 265, 105, 175, and 150 feet past available storage during the weekday AM, weekday midday, weekday PM, and Saturday midday peak hours, respectively.

The capacity analysis measures of effectiveness support the focus of future analyses during the weekday AM and weekday PM peak hours only. The weekday midday and Saturday midday peak hours have less severe capacity and congestion issues compared to the weekday AM and PM peak hours.

4.7 Existing Mainline Traffic Operations

Existing traffic conditions for the year 2022 on the I-95 corridor, including North and South State Streets between Exit 6 and Exit 9 were modeled using microsimulation software PTV VISSIM 2021. VISSIM provides traffic operational outputs such as travel time, roadway capacity, delay, queuing, level of service, and other metrics. It uses information from vehicles to simulate and generate those metrics rather than formulas used by empirical software like Synchro or the Highway Capacity Software (HCS). Therefore, when calibrated, it can be an effective tool to analyze complex and congested conditions as existing on the study corridor and thus serving as an excellent tool to compare various alternatives in the year 2050.

4.7.1 Mainline Analysis Methodology

The micro-simulation model focuses on operations on I-95 between Exit 6 and Exit 9 including the service roads and intersections along these streets as seen in Figure 1. Following are the intersections which were modelled:

1. Harvard Avenue and Grenhart Road
2. Harvard Avenue and Baxter Avenue
3. West Avenue and Grenhart Road
4. West Avenue and Baxter Avenue
5. Greenwich Avenue and S State Street
6. Washington Boulevard and S State Street
7. Washington Boulevard and N State Street
8. Atlantic Street and S State Street
9. Atlantic Street and N State Street
10. Canal Street and S State Street
11. Canal Street and N State Street
12. Elm Street and S State Street
13. Elm Street and N State Street
14. Seaside Avenue and I-95 NB off-ramp
15. Seaside Avenue and E Main Street
16. I-95 SB off-ramp, E Main Street, Courtland Avenue, I-95 on-ramp from E Main Street

The model simulates traffic during the AM (7AM to 9AM) and PM (3PM to 6PM) peak periods. Based on the peaking patterns seen in Section 4.1 Vehicular Traffic Data, it was deemed appropriate to only model the AM and PM peak periods as the level of usage during the midday and Saturday periods are typically less than the AM or PM peaks.

The micro-simulation modeling process requires four general steps:

1. Coding the roadway network to be analyzed (lanes, ramps, intersections, local roadways),
2. Adding traffic control devices to the model (traffic signals, stop signs, etc.),
3. Developing and inputting traffic volumes, and
4. Iteratively calibrating the VISSIM model to match observed existing conditions, such that it is a reliable predictor of current and future traffic operations.

Roadway Network

Roads and interchanges in the VISSIM network were coded with the aid of satellite images obtained from Google and Bing Maps and verified during a site visit.

Traffic Signals

Traffic signal timings obtained from the City of Stamford were used to code the signal timings of the 16 intersections mentioned above. These signal timings were verified during the site visit.

Volume Input

The primary traffic input for the VISSIM micro-simulation model is an origin and destination (OD) matrix of volumes for the AM and PM peak hours. Balanced existing conditions traffic volumes for the AM and PM peak periods are used in conjunction with OD patterns (as discussed in Section 4.5) to develop OD matrices. These volumes result in each simulated vehicle being assigned a network origin and destination. OD volumes were developed for 7AM-9 AM and 3PM-6PM time periods. To accurately simulate traffic, it is necessary to preload the traffic network with vehicles prior to the start of the peak hour to be analyzed. Traffic OD matrices were developed for the 5:45AM-7:00 AM and 2:00PM-3:00 PM periods to preload the network with traffic and complete the peak hour simulation. OD volumes were input for every 15-minutes. OD demands reflect the demand for the OD pair to match demand on the mainline and ramps. Demands were adjusted in an iterative process to obtain accurate throughputs on mainline and ramps.

Model Convergence

The volume input OD matrices and roadway network coded for this model allows for multiple possible paths to satisfy the input parameters. Therefore, dynamic assignment using the stochastic model was used. This process was iteratively run until the chosen convergence criteria (80% or more of the paths varied by 20% or less compared to the previous run for two consecutive runs) was met. The resulting paths were then used as a static input before model calibration.

Model Calibration, General Procedure

The base year model was calibrated based on FHWA guidelines provided in Traffic Analysis Toolbox Volume III[3]. The VISSIM Model calibration followed the three steps outlined below, as listed in the guidelines for quantitative calibration, and peak direction mainline queueing was chosen as a qualitative calibration criterion.

Step 1: Identify representative day

[3] *Traffic Analysis Toolbox Volume III: Guidelines for Applying Traffic Microsimulation Modeling Software 2019 Update to the 2004 Version*

Travel time data along southbound and northbound I-95 from April 25 – May 13, 2022 was used to identify a representative day of April 26, 2022 for the AM peak period and April 27, 2022 for the PM peak period. Days with non-typical weather or crashes were removed from the analysis.

The FHWA guidelines[3] described below were used for this analysis.

- Travel time measurement along southbound and northbound I-95 between Exit 6 and Exit 9 were used to identify the representative day. Travel time is believed to represent typical peak travel conditions along the corridor.
- For each peak travel direction (southbound I-95 during AM and northbound I-95 during PM peak period), average travel time was calculated using Equation 5 of FHWA guidelines [1] for each 15-minutes time interval across all days.

$$\bar{m}_{t,j} = \frac{\sum_i m_{i,j}(t)}{N_{cluster}} \ \forall\, m, t, j \text{ (Equation 5)}$$

$\bar{m}_{t,j}$ Average travel time in time interval *t* along I-95 Mainline in peak direction *j*

$m_{i,j}(t)$ Value of the Travel Time measure on day *i* in time interval *t* along I-95 Mainline in peak direction *j*.

$N_{cluster}$ Number of days in cluster= 9 days

- The difference between the average value and the value observed on a particular day was calculated using equation 6 of FHWA guidelines [1] and expressed as a percentage of the mean value.

$$\dot{m}_{i,j}(t) = \frac{\sqrt{(\bar{m}_{t,j} - \bar{m}_{i,j(t)})^2}}{\bar{m}_{t,j}} \text{ (Equation 6)}$$

- A representative day was then selected from the days with both mainline and ramp counts (26 April to 28 April 2022). The day with the minimum difference from the average was considered as the representative day for the peak direction of travel on I-95. The representative day for the AM and PM peak period was not constrained to being the same day.

$$i^* = min_i[\sum_m \sum_i \sum_t \dot{m}_{i,j}(t)] \text{ (Equation 7)}$$

Step 2: Prepare Variation Envelope - Calibration Performance Measures

VISSIM Models were calibrated to travel time and throughput at bottleneck locations along northbound I-95 during the PM peak period and southbound I-95 during the AM peak period.

Figure 27 and Figure 28 are heatmaps of selected I-95 travel time runs using the floating car method as discussed in Section 4.4Travel Time Reliability. The speeds shown suggest that Canal Street and Washington Blvd are bottlenecks for southbound I-95 and Canal Street and West Avenue are bottlenecks for northbound I-95. Therefore, the throughput of the mainline directly downstream of these locations were used in calibration.

Figure 27: Southbound I-95 Mainline Selected AM Travel Time Run Speed Map

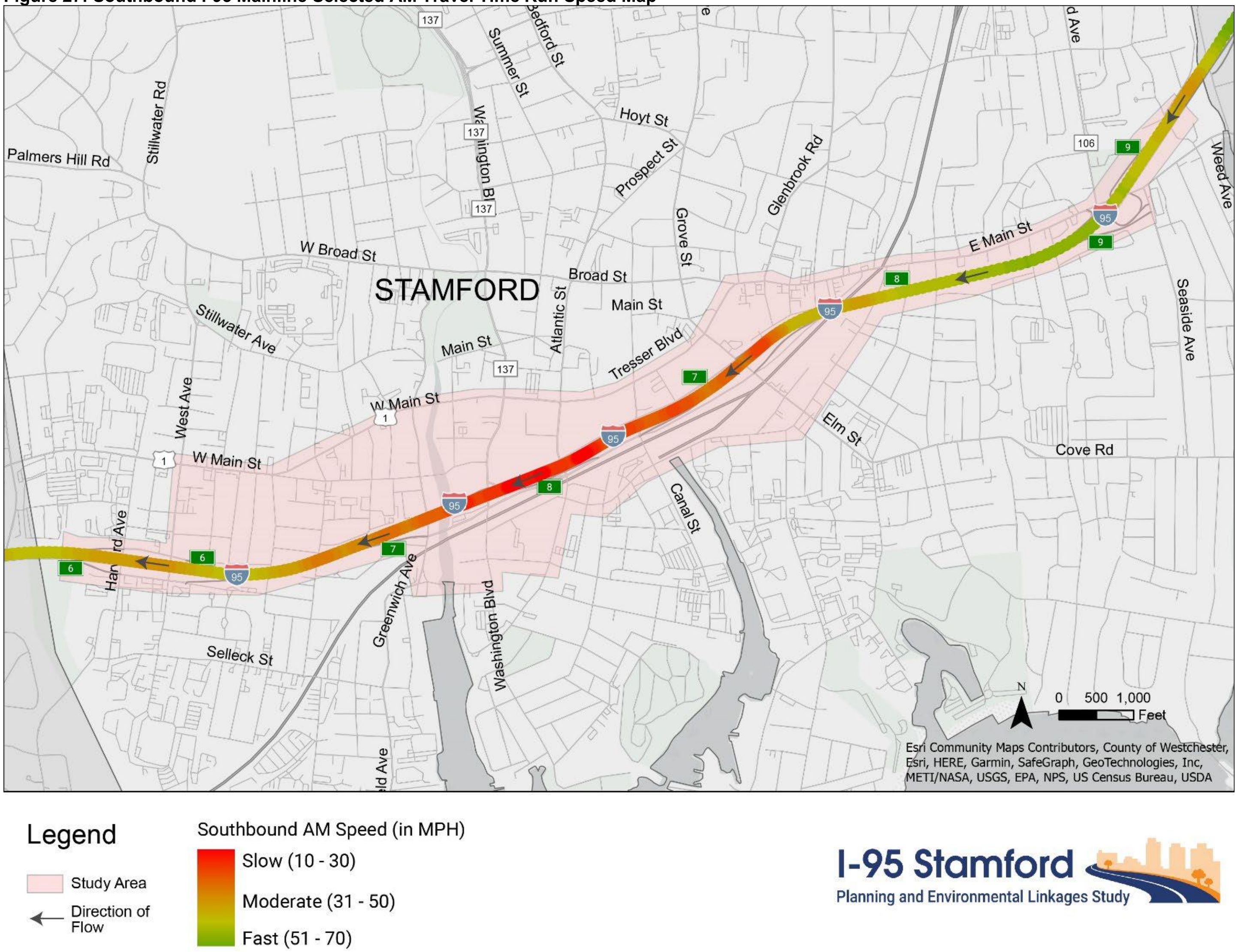
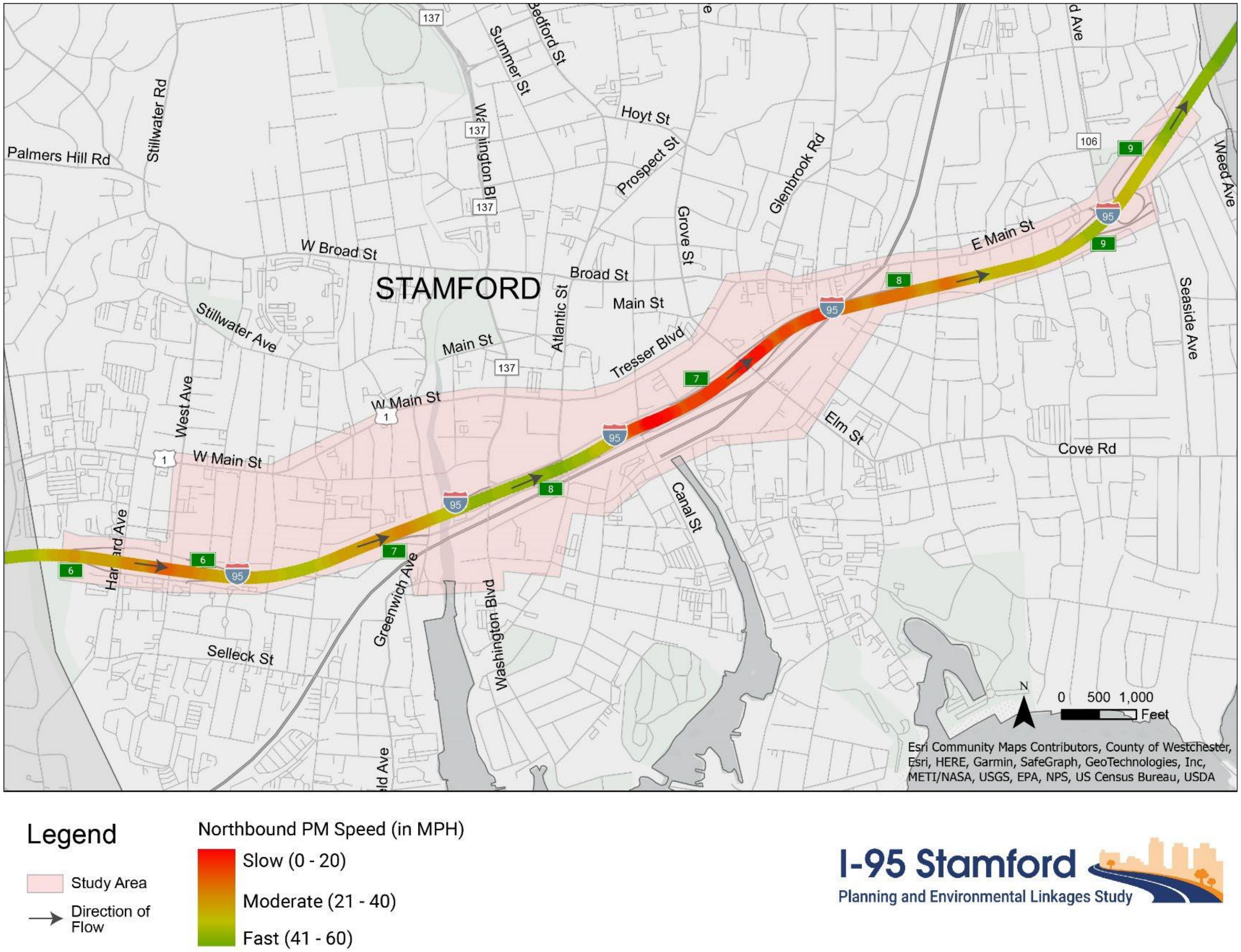


Figure 28: Northbound I-95 Mainline Selected PM Travel Time Run Speed Map



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Step 3: Create Variation Envelopes

Variation envelopes for the travel time and bottleneck throughput along northbound I-95 during the PM peak period and southbound I-95 during the AM peak period were prepared for the representative day for each 15-minute period, using travel time and traffic count data along southbound and northbound I-95 from April 25 – May 13, 2022. Days with non-typical weather or crash incidents were disregarded in the representative day identification described above.

Variation envelopes for the travel time were developed using the FHWA guidelines[3] as described in the steps below.

- First, a variation envelope was created for 95% confidence intervals using Z-Statistics (a Z-value of 1.96 will be used in this case).

$$\sim 2\ Sigma\ band\ Maximum\ Value\ \hat{I}_{\sim 2} = C_r(t) + Z_{95\%}\left(\sigma(t)\right) \text{ (Equation 8)}$$

$$\sim 2\ Sigma\ band\ Minimum\ Value\ \hat{I}_{\sim 2} = C_r(t) - Z_{95\%}\left(\sigma(t)\right) \text{ (Equation 9)}$$

$C_r(t)$ Observed travel times from the representative day

$\sigma(t)$ Observed travel times from the representative day

- Second, a narrower variation envelope was created to describe roughly 2/3 of the observed variation based on a single standard deviation.

$$1\ Sigma\ band\ Maximum\ Value\ \hat{I}_1(t) = C_r(t) + \sigma(t) \text{ (Equation 10)}$$

$$1\ Sigma\ band\ Minimum\ Value\ \hat{I}_1(t) = C_r(t) - \sigma(t) \text{ (Equation 11)}$$

Variation envelopes were created for bottleneck throughputs in peak direction of travel on I-95 using the same methodology as the Travel Time envelopes.

Calibrate model variants within acceptability criteria

Model results were evaluated against the calibration criteria outlined in FHWA guidelines[3].

- ***Criterion 1, Control for Time-Variant Outliers***: For more than 20 time intervals, 95% of simulated outputs fall within the ~2 Sigma Band, $C_r(t) \pm 1.96\left(\sigma(t)\right)$. If AM and PM peak periods have less than 20 time intervals, this criterion will be reduced to require one or fewer simulated output falls outside the ~2 Sigma Band.
- ***Criterion 2, Control for Time-Variant Outliers***: 2/3rd of simulated outputs, including two critical time intervals, are to fall within the 1 Sigma Band, $C_r(t) \pm \sigma(t)$. For travel time, the first critical time interval is the time interval with the highest observed travel time. The second critical time interval is the time interval with the second highest observed travel time in a non-adjacent time interval (non-adjacent means that the second-time interval should be more than one time interval earlier or later than the first critical time interval). For bottleneck throughput, the critical time intervals will be the same time intervals as identified by the travel time.
- ***Criterion 3, Bounded Dynamic Absolute Error (BDAE)***: This criterion ensures that, on average, simulated results are close to the observed representative day. BDAE was calculated using equation 12 in FHWA guidelines [1]

$$BDAE\ Threshold = \frac{\sum_{i \neq r} \sum_t \frac{|c_r(t) - c_i(t)|}{N_T}}{N_{cluster} - 1}$$ (Equation 12 [1])

$c_r(t)$ Observed value of representative day during time interval *t*

$c_i(t)$ Observed value of non-representative day during time interval *t*

N_T Number of time intervals

$N_{cluster}$ Number of days of data collection = 9 days

The average of absolute differences between simulation outputs and the representative day over all time periods were calculated using equation 13 in FHWA guidelines [1] below to ensure they are less than or equal to the BDAE threshold

$$\frac{\sum_t |c_r(t) - \tilde{c}_i(t)|}{N_T} \leq BDAE\ Threshold$$ (Equation 13 [1])

$\tilde{c}_i(t)$ Simulation output during time interval *t*

- ***Criterion 4, Bounded Dynamic Systematic Error***: This criterion ensures simulated data are not excessive over- or under-estimators. The absolute value of the average of differences between simulation outputs and representative day values as calculated using Equation 14 in FHWA guidelines [1] are less than or equal to one-third of the BDAE Threshold.

$$\left| \frac{\sum_t c_r(t) - \tilde{c}_i(t)}{N_T} \right| \leq \frac{1}{3} \times BDAE\ Threshold$$ (Equation 14 [1])

The VISSIM model was calibrated for both AM and PM peak durations along peak direction of travel only (southbound I-95 during AM peak period and northbound I-95 during PM peak period) using the methodologies and criteria discussed previously. Calibration was done for one random seed only.

Model Calibration, Application

The modeling team adjusted several of the available parameters in VISSIM to achieve model calibration. Parameters adjusted included driver behavior model, using unique driving behavior types on specific links; convergence criteria, adding reduced speed areas, coordinating signal timings; and the OD matrices. This included using field calibrated reduced speed areas to simulate downstream congestion outside the study area that impacted behavior within the modeled study area.

Qualitative Calibration

For the qualitative calibration of queuing, the VISSIM model was observed during model runs and compared to travel time vehicle video footage, traffic count video footage, and field visit notes. VISSIM model segment speed graphics are shown below to illustrate the queuing observed during model runs, since they are a static and reportable representation of queueing observed in the model.

Figure 29 shows existing year AM VISSIM model segment speeds in the southbound direction. The first queue develops before the analysis period at the Exit 7 on-ramp from Washington Blvd merge area. It extends back to Exit 8 until 7:45AM where it then gradually extends back to Exit 9. A second queue also develops at 7:30 AM at the Exit 9 on-ramp merge area but is not particularly distinct because of the extension of the first queue through Exit 9 starting at 7:45AM. Additionally, there is downstream (to the south) queue development outside of our study area which extends back to Exit 6 at 7:45AM and quickly continues to extend back to the Exit 7 on-ramp where its extent is no longer distinct because of the existing queue originating at that point. These patterns are consistent with queueing on I-95 observed during field visits.

Figure 29: Southbound AM Model Segment Speeds

	← Southbound Speed (mph) from Exit 9 to Exit 6														
	Exit 6 Harvard Avenue		West Avenue		Exit 7 Greenwhich Ave Washington Blvd	Exit 8	Atlantic Street Canal Street	Elm Street				Exit 9	East Main Street		
Segment Length (mi)	0.2	0.3	0.2	0.2	0.4	0.2	0.3	0.3	0.1	0.4	0.2	0.2	0.2	0.2	0.2
Measurement Start Time 7:00 AM	66	66	61	47	16	10	11	18	35	67	65	71	71	70	71
7:15 AM	66	67	62	48	17	9	8	12	25	66	57	70	71	70	70
7:30 AM	67	67	62	48	13	7	8	8	9	58	37	68	71	70	70
7:45 AM	15	19	34	39	18	7	8	11	12	14	42	71	72	71	71
8:00 AM	14	14	15	22	13	6	8	8	12	17	23	16	70	70	70
8:15 AM	13	12	17	17	13	7	7	6	9	11	13	9	21	68	70
8:30 AM	13	13	18	22	12	5	7	9	9	11	18	8	9	71	72
8:45 AM	14	15	23	24	15	8	9	7	10	46	18	6	38	72	71

Figure 30 shows existing year PM VISSIM model segment speeds in the northbound direction. The first queue develops before the analysis period at the Exit 8 on-ramp from Elm Street merge area. It extends back to the Atlantic Street overpass for the duration of the study period. There is a small second queue that develops and dissipates around Exit 6 throughout the analysis period. Throughout the PM study period there is downstream (to the north) queue development outside of our study area which extends into our study area at Exit 9 around 3:15PM and 5:00PM. These patterns are consistent with queueing on I-95 observed during field visits.

Figure 30: Northbound PM Model Segment Speeds

	Northbound Speed (mph) from Exit 6 to Exit 9 →														
	Exit 6	Harvard Avenue	West Avenue		Exit 7	Greenwhich Ave	Washington Blvd	Exit 8 Atlantic Street Canal Street		Elm St			Exit 9		East Main Street
Segment Length (mi)	0.2	0.2	0.2	0.2	0.1	0.2	0.1	0.4	0.2	0.1	0.2	0.4	0.2	0.1	0.4
Measurement Start Time 3:00 PM	69	70	72	68	59	65	67	26	10	12	12	16	47	19	18
3:15 PM	68	69	71	66	55	60	67	70	19	14	13	17	27	19	17
3:30 PM	69	70	71	64	55	61	66	70	12	9	11	16	42	21	18
3:45 PM	69	69	71	38	51	57	66	69	11	11	12	15	57	42	19
4:00 PM	69	70	71	52	53	58	63	28	11	11	12	15	59	66	49
4:15 PM	68	70	71	30	51	58	63	62	11	13	13	15	56	64	60
4:30 PM	68	69	71	67	55	60	66	70	11	13	12	17	58	65	55
4:45 PM	67	69	69	38	52	58	65	70	28	15	16	16	58	52	18
5:00 PM	68	69	69	45	51	57	63	70	33	8	11	15	57	49	23
5:15 PM	68	68	66	20	49	56	63	67	9	10	11	11	58	66	60
5:30 PM	68	68	70	59	53	60	67	49	10	9	10	15	55	63	63
5:45 PM	69	69	70	64	56	60	61	8	9	10	11	15	58	63	48

Quantitative Calibration

Figure 31 shows the bottleneck throughput for the AM calibrated model in the southbound direction in comparison to the calculated Criteria 1 and 2 variance envelopes. The throughput after the bottleneck falls within the one sigma band for all measured time periods which satisfies both Criteria 1 and 2.

Figure 31: Southbound AM Model Bottleneck Throughput Criteria 1 & 2

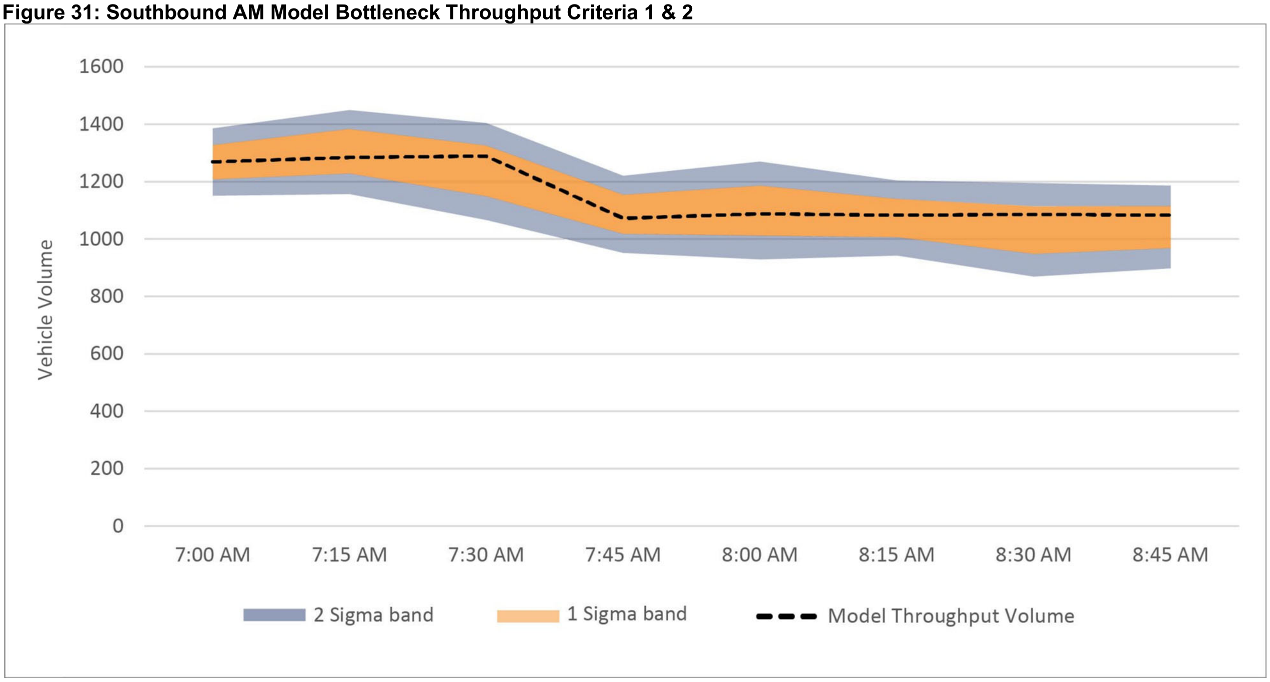


Figure 32 shows the southbound mainline travel time from north of Exit 9 to south of Exit 6 in the AM calibrated model in comparison to the calculated Criteria 1 and 2 variance envelopes. The southbound travel time falls within the two-sigma band for all except one time period (8:15AM-8:30AM) which has an observed travel time of 22.2 minutes. This satisfies Criterion 1. The southbound travel time falls within the one sigma band for five out of the seven time periods, including the critical times of 8:00AM and 8:30AM, which satisfies Criterion 2.

Figure 32: Southbound AM Model Mainline Travel Time Criteria 1 & 2

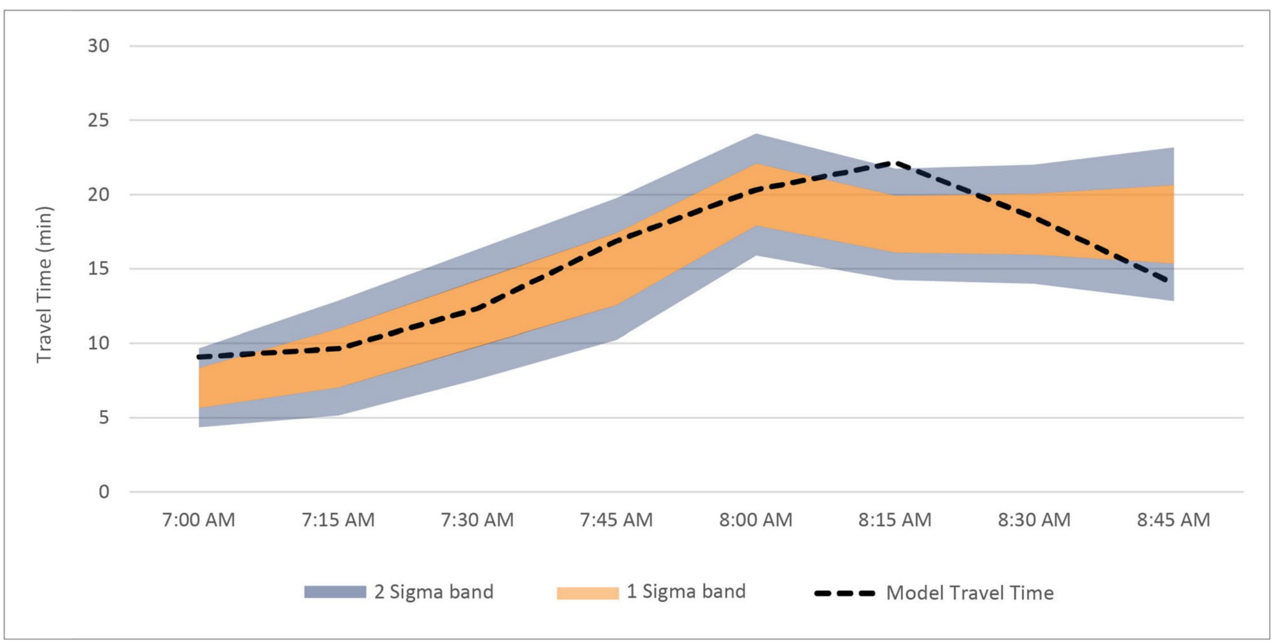


Figure 33 shows the bottleneck throughput for the PM calibrated model in the northbound direction in comparison to the calculated Criteria 1 and 2 variance envelopes. The throughput after the bottleneck falls within the two-sigma band for all measured time periods which satisfies Criterion 1. The northbound bottleneck throughput falls within the one sigma band for all but two time periods, and one of the two critical times (5:45PM-6PM) which partially satisfies Criterion 2. During the other critical time, 5:15PM, the model throughput is 1,162 vehicles, which is five vehicles below the lower bound of the one sigma band (1,167), which partially satisfies Criterion 2.

Figure 33: Northbound PM Model Bottleneck Throughput Criteria 1 & 2

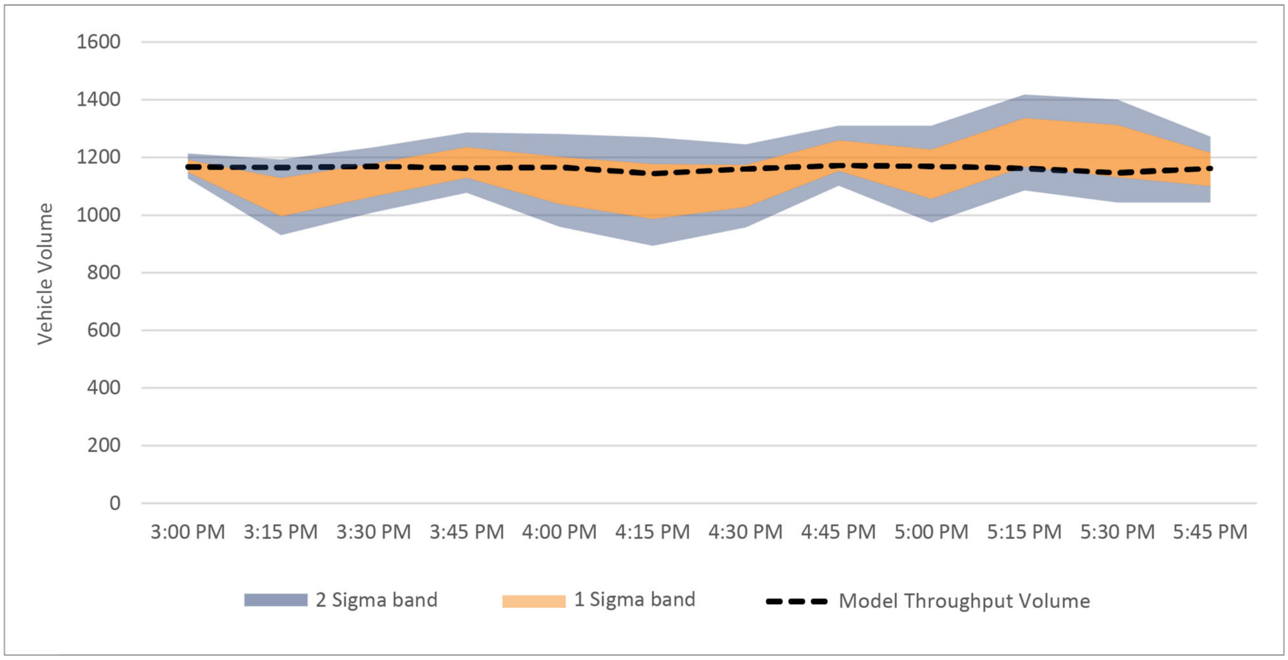


Figure 34 shows the northbound mainline travel time from south of Exit 6 to north of Exit 9 in the PM calibrated model in comparison to the calculated Criteria 1 and 2 variance envelopes. The PM northbound travel time falls within the two-sigma band for all time periods which satisfies Criterion 1. The northbound travel time falls within the one sigma band for all but one time period, with both critical times also falling within the one sigma band, which satisfies Criterion 2.

Figure 34: Northbound PM Model Mainline Travel Time Criteria 1 & 2

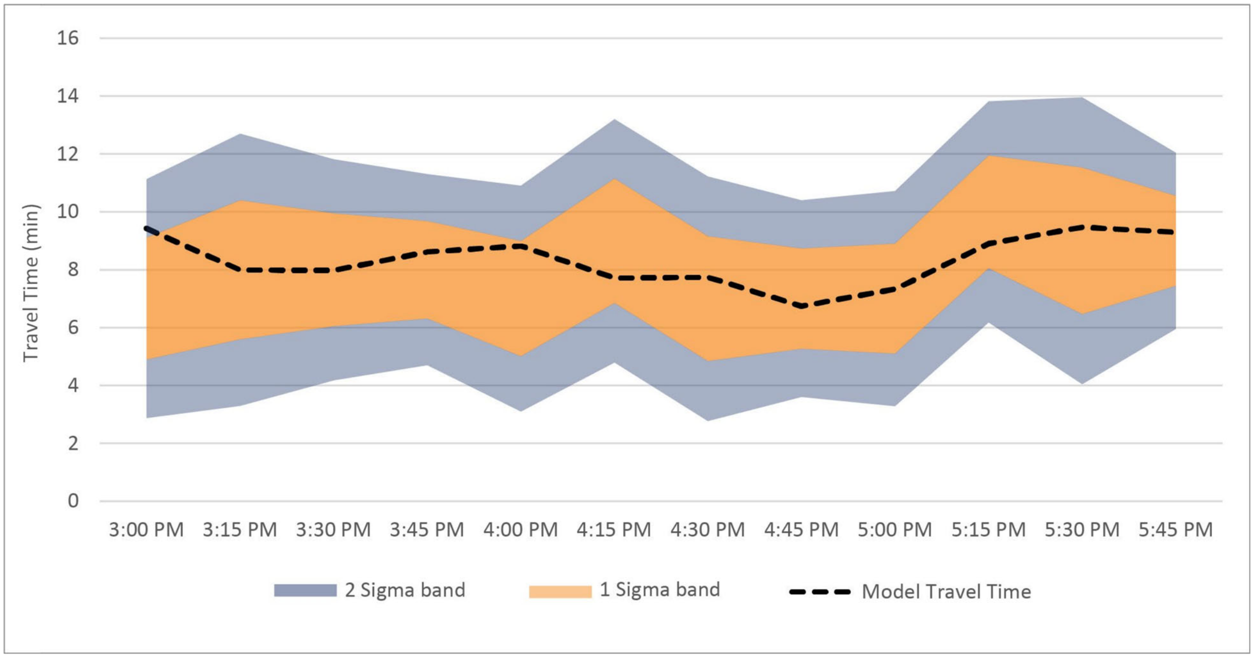


Table 6 shows the calculated BDAE values in comparison to Criteria 3 and 4. Both criteria were met by both the AM and PM calibrated models for both the bottleneck throughput and travel time.

Table 6: AM and PM BDAE Values for Criteria 3 & 4

	BDAE	average of absolute differences	Criteria 3 met?	absolute value of the average of differences	Criteria 4 met?
AM Travel Time	2.51	1.7	Yes	0.7	Yes
AM Bottleneck Throughput	70.02	25.6	Yes	14.6	Yes
PM Travel Time	1.96	0.8	Yes	0.3	Yes
PM Bottleneck Throughput	53.88	47.4	Yes	10.6	Yes

We believe that the above methodologies align with FHWA guidelines[3] and effectively calibrated the base year model for the purposes of this project, i.e., assessing the effectiveness of alternatives in easing congestion along the I-95 corridor during peak time in Stamford, CT.

4.7.2 Mainline Analysis Results

This section discusses the existing traffic operation metrics for AM and PM peak hours.

The base year model in VISSIM was developed for the following key performance measures, or measures of effectiveness (MOEs):

- Travel Time on I-95 in peak direction of travel and
- Traffic throughputs at bottleneck(s).

These key performance measures are the primary MOEs to evaluate alternative scenarios in future years. In addition to these, we will compare the following metrics:

- Vehicle Throughput. Measures the number of vehicles that are able to be processed through a segment of roadway. An increase in vehicle throughput is generally thought of as an improvement.

- Vehicle Hours Traceled (VHT) – is the combined travel time of all vehicles on a link or in a network. This measure is distinct from average travel time for any one trip because it places accounts for the number of vehicles making the trips. Reducing VHT is generally interpreted as an improvement.

- Vehicle Hours Delay (VHD) – is equal to the measured VHT minus the VHT if all trips were traveling at free-flow speeds. For example, if the VHT is measured at 30 minutes, but the free-flow time is 20 minutes, the VHD is 10 minutes. This delay statistic is tallied for every vehicle in a segment or the network to arrive at VHD. Reducing VHD is generally interpreted as an improvement.

- Vehicle Miles Travel (VMT) – is the aggregate distance traveled by all vehicles in a segment or network. An increase in VMT is generally thought of as an improvement provided that VHD is the same or reduced. It is also important to consider VMT when comparing a no-build and build condition because a change in VHT, VHD, or throughput may simply be attributable to an increase in overall demand. Increases in demand can be expressed as an increase in VMT.

Table 7 contains the list of proposed MOEs to study future year scenarios to assist the assessment of alternative scenarios. The I-95 mainline and ramps will be assessed as primary MOEs. Service roads will be assessed as secondary MOEs.

Table 7: Measures of Effectiveness (MOEs)

Measure of Effectiveness	Corridor	Time Period
Travel Time and Vehicle Travel Time (VHT)	• NB and SB I-95 Mainline (between Exit 6 and Exit 9) • NB S State St (between Exit 7 and Exit 8) • SB N State St (between Exit 7 and Exit 8) • Networkwide (I-95 and surrounding roadways)	7-9 AM 3-6 PM
Vehicle Throughputs (Capacity Analysis)	• NB and SB I-95 Mainline bottleneck(s) • NB and SB I-95 Mainline between the exits • NB and SB I-95 Ramps • Baxter Avenue, Grenhart Rd, N State St, S State St, Courtland Ave, and Seaside Ave including intersections between Exit 6 and Exit 9 • Networkwide (I-95 and surrounding roadways)	7-9 AM 3-6 PM
Vehicle Hour Delay (VHD)	• NB and SB I-95 Mainline (between Exit 6 and Exit 9) • NB S State St (between Exit 7 and Exit 8) • SB N State St (between Exit 7 and Exit 8) • Networkwide (I-95 and surrounding roadways)	7-9 AM 3-6 PM
Vehicle Miles Travel (VMT)	• NB and SB I-95 Mainline (between Exit 6 and Exit 9) • NB S State St (between Exit 7 and Exit 8) • SB N State St (between Exit 7 and Exit 8) • Networkwide (I-95 and surrounding roadways)	7-9 AM 3-6 PM

The above MOEs will provide sufficient information to effectively examine various future year scenarios.

4.7.3 Mainline Level of Service

Capacity analyses were performed at all freeway mainline segments, ramp merge sections, and ramp diverge sections along I-95 within the study area for the peak direction of travel during the AM and PM model analysis periods. The analyses were performed using the link results from the VISSIM models developed as previously described. Links from the model were assigned a facility type (basic freeway, merge, diverge, or weave) based on the influence area definition of segments in the HCM 7th edition chapter 12, section 2. This was used as a guide, rather than a precise definition, since the model link lengths did not always precisely coincide with influence area lengths. However, these definitions were determined to provide satisfactory results for the purpose and level of detail of this analysis.

The LOS within each analysis segment is determined by the density (measured in passenger cars per mile per lane, or pc/mi/ln) for all cases in this analysis. Typically, LOS F exists when corridor demand exceeds the capacity of the corridor segment or where the off-ramp demand exceeds the off-ramp capacity, but for this analysis for all segment types densities greater than 45pc/ln/mi were considered LOS F. Table 8 presents the LOS criteria for freeway sections and Table 9 presents the LOS criteria for merge and diverge sections as presented in the HCM.

Table 8: LOS Criteria – Basic Freeway Segments

LOS	Density (pc/mi/ln)
A	≤ 11
B	> 11 – 18
C	> 18 – 26
D	> 26 – 35
E	> 35 – 45
F	Demand exceeds capacity or >45

Source: 7th Edition Highway Capacity Manual

Table 9: LOS Criteria – Merge and Diverge Segments

LOS	Density (pc/mi/ln)
A	≤ 10
B	> 10 – 20
C	> 20 – 28
D	> 28 – 35
E	> 35
F	Demand exceeds capacity

Source: 7th Edition Highway Capacity Manual

An AM peak hour of 7:45AM to 8:45AM was determined based on highest hourly density in the AM VISSIM model results. Table 10 shows the resulting existing 2022 LOS results for the AM peak hour along the peak direction of travel only (southbound).

Table 10: Southbound AM Interstate Segment Simulated LOS Results

Segment	Segment Type	Simulated Density (pc/mi/ln)	Simulated LOS
I95 SB ML North of Exit 9	Mainline	18.3	C
I95 SB Exit 9 Off-Ramp	Diverge	15.5	B
I95 SB ML @ E Main Street	Mainline	89.7	F
I95 SB Exit 9 On-Ramp	Merge	98.2	F
I95 SB Exit 8 Off-Ramp	Diverge	98.0	F
I95 SB ML North of Elm St	Mainline	151.0	F
I95 SB Exit 7 Off-Ramp	Diverge	98.4	F
I95 SB ML @ Canal Street	Mainline	164.2	F
I95 SB Exit 7 On-Ramp (Atlantic St)	Merge	119.0	F
I95 SB Exit 7 On-Ramp (Washington Blvd)	Merge	95.7	F
I95 SB Exit 6 Off-Ramp	Diverge	85.6	F
I95 SB ML @ West Ave	Mainline	112.8	F
I95 SB Exit 6 On-Ramp	Merge	69.0	F
I95 SB ML South of Exit 6	Mainline	71.4	F

(*) Red text denotes mainline locations with unacceptable LOS.

A PM peak hour of 5:00PM to 6:00PM was determined based on highest hourly density in the PM VISSIM model results. Table 11 shows the resulting existing 2022 LOS results for the PM peak hour along the peak direction of travel only (northbound).

Table 11: Northbound PM Interstate Segment Simulated LOS Results

Segment	Segment Type	Simulated Density (pc/mi/ln)	Simulated LOS
I95 NB ML South of Exit 6	Mainline	24.6	C
I95 NB Exit 6 Off-Ramp	Diverge	18.8	B
I95 NB ML @ West Ave	Mainline	28.4	D
I95 NB Exit 6 On-Ramp	Merge	72.7	F
I95 NB ML @ Fairfield Ave	Merge/Diverge	42.4	E
I95 NB Exit 7 Off-Ramp	Diverge	24.8	C
I95 NB Exit 8 Off-Ramp	Diverge	24.6	C
I95 NB ML @ Canal Street	Mainline	119.8	F
I95 NB Exit 8 On-Ramp (Canal St)	Merge	104.5	F
I95 NB Exit 8 On-Ramp (Elm St)	Merge	74.7	F
I95 NB ML South of Maher Rd	Merge/Diverge	34.8	D
I95 NB Exit 9 Off-Ramp	Diverge	22.2	C
I95 NB ML @ E Main St	Mainline	47.9	F
I95 NB Exit 9 On-Ramp	Merge	59.9	F
I95 NB ML North of Exit 9	Mainline	65.1	F

(*) Red text denotes mainline locations with unacceptable LOS.

The results are also summarized geographically for Weekday AM and PM peak hour in Figure 35 and Figure 36.

Figure 35: Interstate Segment Simulated Level of Service – Southbound Weekday AM

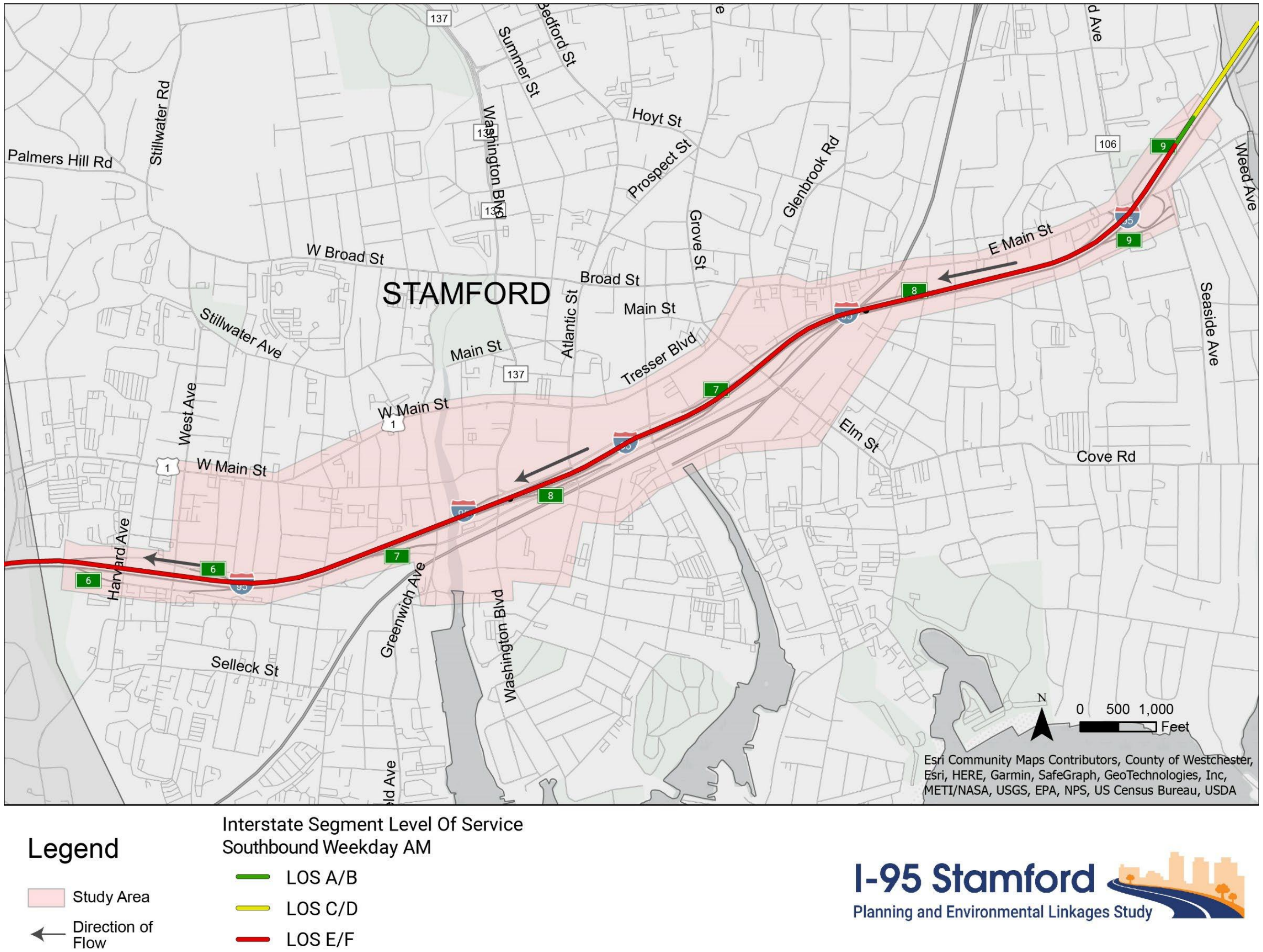
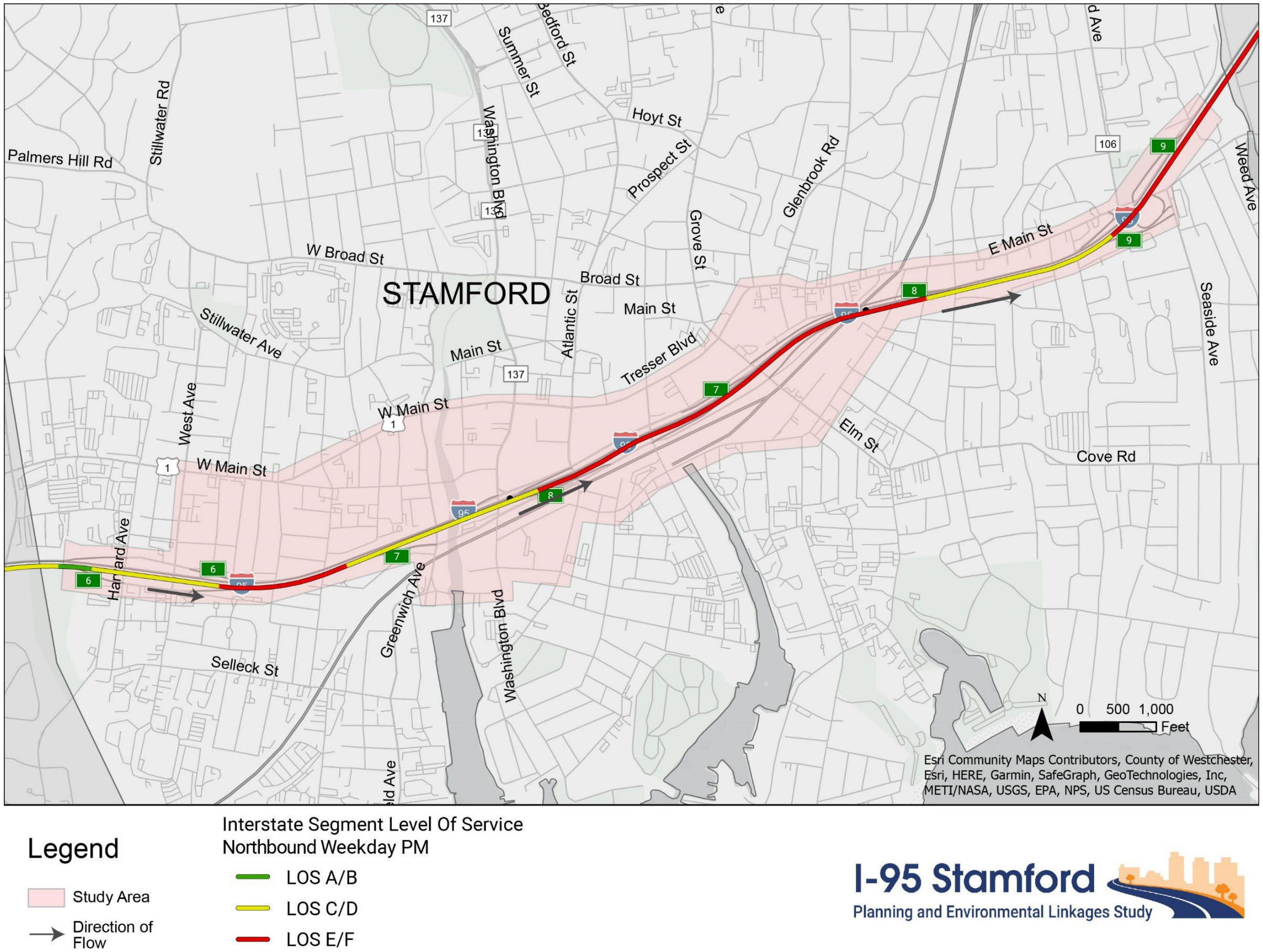


Figure 36: Interstate Segment Simulated Level of Service – Northbound Weekday PM



5 Review of Crash Data

This section summarizes the evaluation of existing crash data within the study area.

5.1 Scope

As part of this project, opportunities to improve traffic operations and safety will be identified through analysis of traffic and crash data within the study area.

Phase 1 of the PEL process includes the collection and analysis of crash data to document the existing safety issues along I-95 and the adjacent intersections within the study area. The overall crash trends and high crash locations were documented at each of the 50 intersections and eight segments along and adjacent to I-95 within the study area.

5.2 Introduction

The crash analysis study area encompasses I-95 within the City of Stamford limits (Milepost 6.29 to Milepost 9.63). This ±3.5-mile section of I-95, which includes the Exit 6 through 9 interchanges, was divided into eight segments for ease of analysis. The start and end mileposts for the segments were obtained from the Stamford Town Road (TRU) maps published by CTDOT. Each of the segments was further divided into sections between ramps (where applicable). The crash data for intersections within the study area was also examined and summarized. The study area includes eight mainline segments and 50 intersections as shown in Figure 37 and Figure 38.

Figure 37: Crash Analysis Study Segments

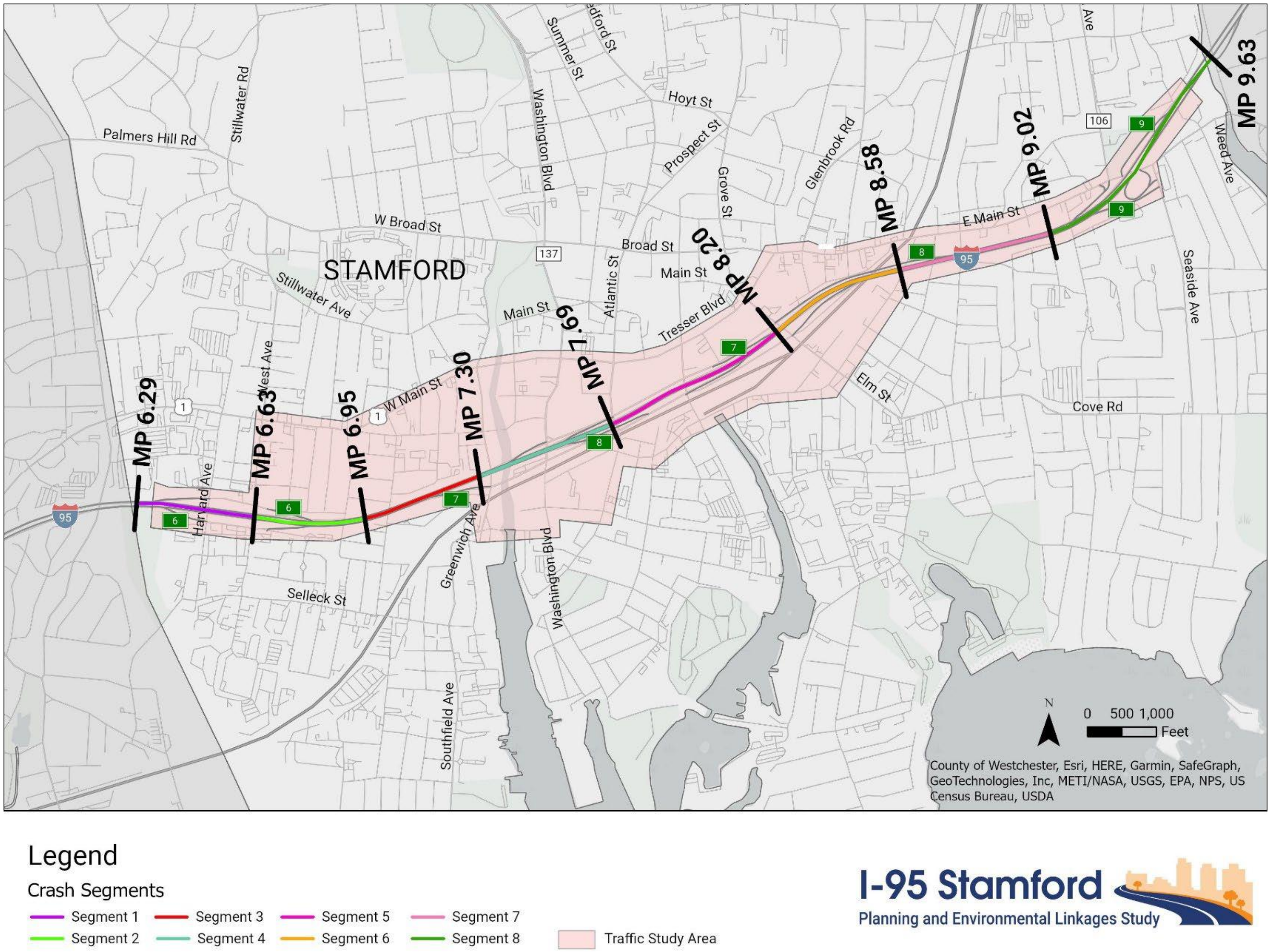
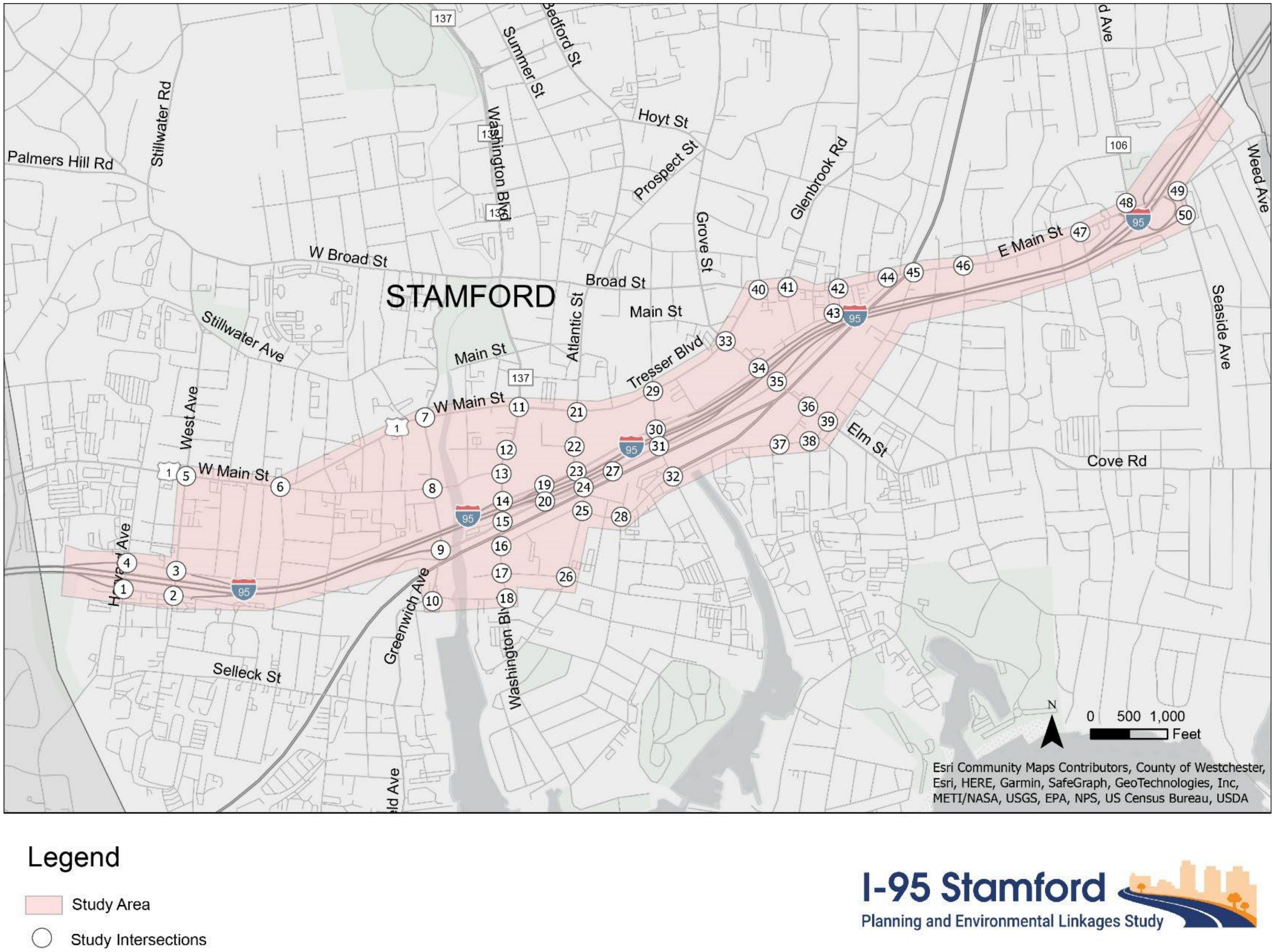


Figure 38: Crash Analysis Study Intersections



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5.3 Data & Methodology

The historical crash data was evaluated within the study area. The latest available data at the time of analysis was obtained from the UCONN Connecticut Crash Data Repository for the period between January 1, 2016, and October 31, 2021. These records identify the date and time, general location, severity, manner of collision, weather condition, lighting, and surface conditions for each crash. The data repository also provides a crash diagram for each crash showing the relative vehicle location before, during, and after the crash occurred. There were instances where the location mentioned in the data and location shown in individual diagrams did not match. For those cases, additional information such as vehicle position data, contributing factors, etc. were checked to identify the correct location.

Collision diagrams were prepared for each of the study locations to visually identify the problem areas and verify how the crashes were distributed along the different sections of a segment or at an intersection. While plotting these diagrams, each crash within the study area was examined to verify if the location and documented manner of collision matched the supporting information. The crash repository identifies fixed object crashes, pedestrian crashes, and bicycle crashes as “Other”. The manner of collision for these crashes was adjusted to correctly identify them in the analysis.

The crash data was then analyzed by location, severity, weather conditions, lighting condition and manner of collision. Crashes involving pedestrians and cyclists were analyzed separately. After summarizing the crash data, a network screening analysis was performed to rank the sections within the I-95 mainline segments and the intersections in the study area to identify locations with high crash rates.

Crash rates are calculated by dividing the total number of crashes at a given roadway section or intersection over a specified time period by a measure of exposure. Traffic volume is a commonly used measure of exposure. The locations are then ranked from high to low by crash rate. For each of the segments and intersections presented in this report, the crash rate was calculated using the following formulas:

$$R_{seg} = \frac{C * 10^8}{365 * N * V * L}$$

$$R_{int} = \frac{C * 10^6}{365 * N * V}$$

Where,

R_{seg} = Crash rate for the road segment expressed as crashes per 100 million vehicle-miles of travel (VMT).

R_{int} = Crash rate for the intersection expressed as crashes per million entering vehicles (MEV).

C = Total number of crashes in the study period.

N = Number of years of data.

V = Number of vehicles per day

L = Length of the roadway segment in miles.

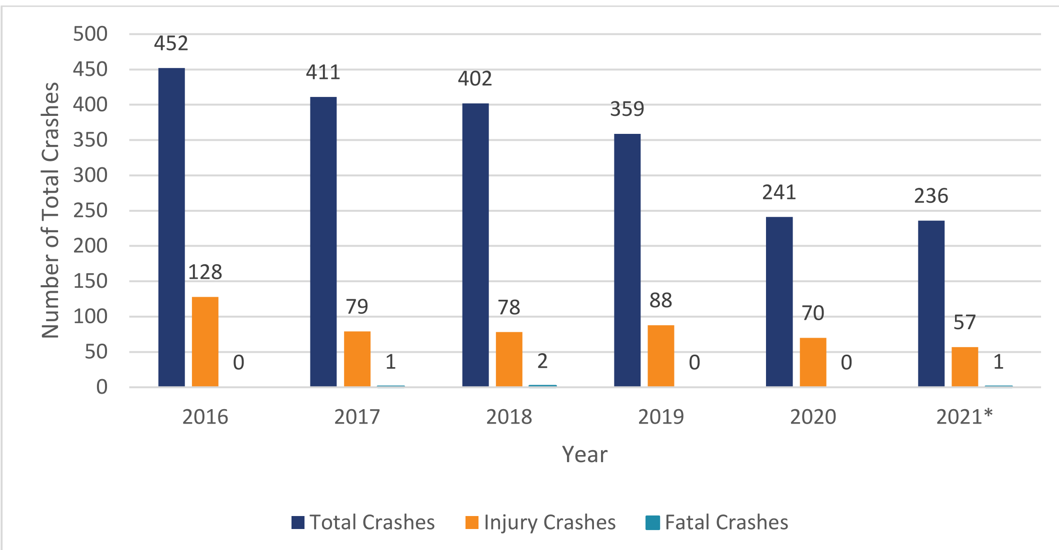
Traffic volume data was collected for all mainline segments, ramps, and intersections as part of this project. For each of the segments and ramps, the Average Daily Traffic (ADT) was calculated using average weekday hourly volume based on data collected in 2022. For the study intersections, the traffic data collection was limited to 6:00 AM to 8:00 PM. To account for the traffic volumes outside of the data collection window, traffic volume data was obtained from the CTDOT Traffic Monitoring Station Viewer and used to calculate the hourly volumes for 2022 conditions using historical directional split and hourly factor information. Where count station data was not available for an approach to an intersection, nearby stations from similar streets were used to compute the hourly factors and directional splits. Once the hourly factors and splits were calculated, the 2022 traffic volumes for the hours between 8:00 PM and 6:00 AM were estimated.

After the ADT was calculated for each location, they were converted to 2022 Average Annual Daily Traffic (AADT) by using the monthly and daily expansion factors provided by CTDOT. The AADTs for the segments and intersections were used to calculate the crash rates.

5.4 I-95 Crash Analysis

During the study period, there were a total of 2,101 crashes on I-95 within the limits of the City of Stamford. Among these, 504 crashes resulted in injuries. These crashes occurred both along the mainline and at the on- and off-ramps. Figure 39 shows the crash trend over the last five years. The total crashes generally decreased while the number of injury crashes remained relatively consistent. It is evident from the crash trends that the pandemic had a significant impact on travel patterns and traffic volumes; resulting in a lower number of crashes in 2020 and 2021 compared to previous years.

Figure 39: Number of Crashes by Year



***From 01.01.2021 to 10.31.2021**

The three most common crash types along I-95 were rear-end, sideswipes and fixed object crashes. Together, these three types accounted for 97.9% of all crashes as shown in Figure 40. A majority of the crashes happened during clear weather conditions or when natural or artificial lighting was present as shown in Figure 41. Adverse weather condition and no lighting contributed to 13.95% and 1.29% of total crashes, respectively as shown in Figure 42.

Figure 40: Crash Types

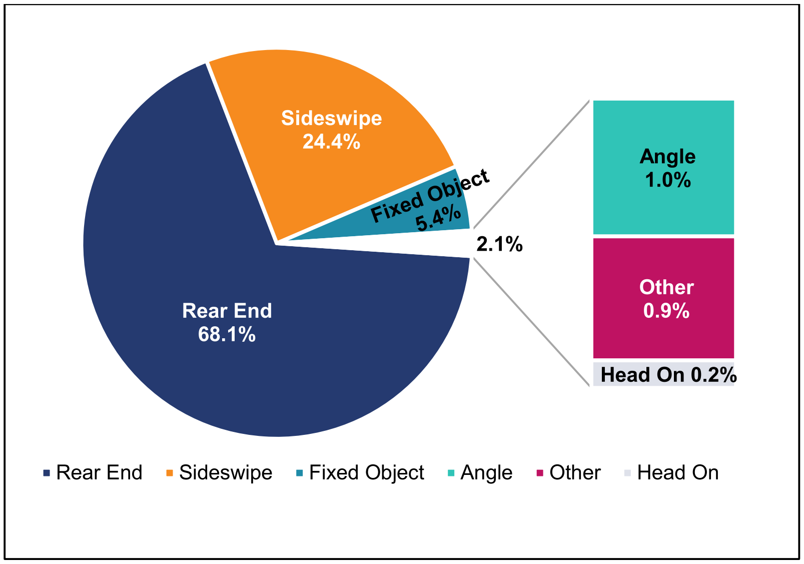


Figure 41: Weather Conditions During Crash Incidents

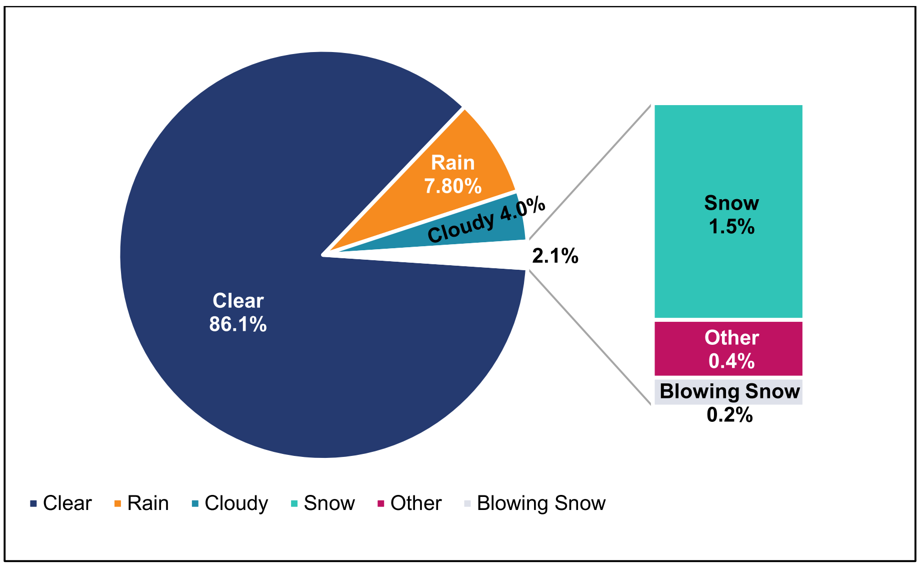
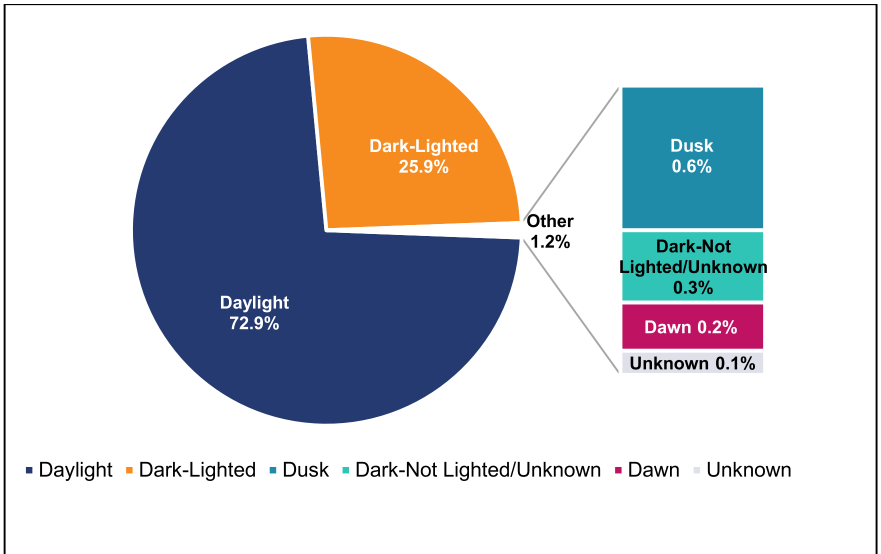


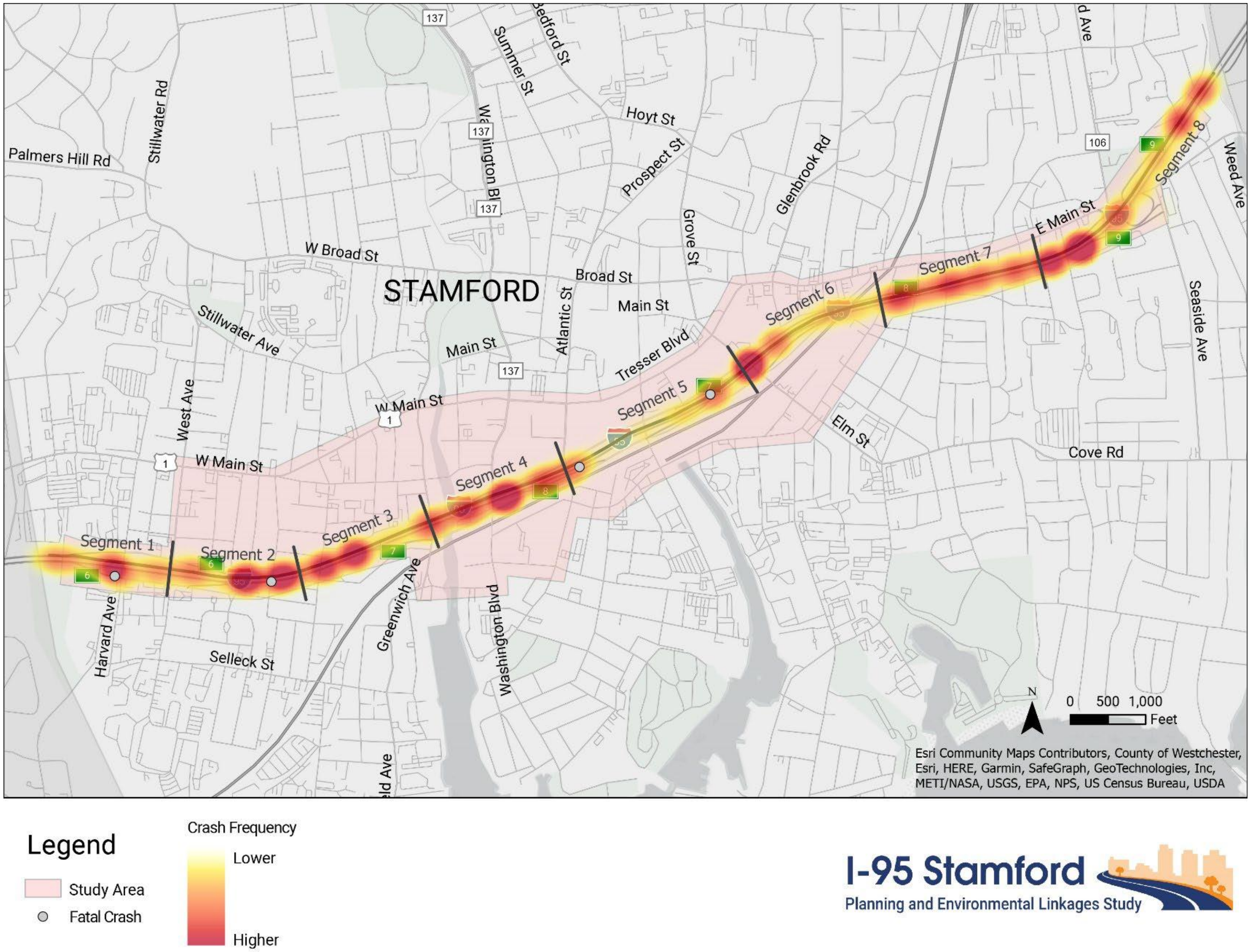
Figure 42: Light Conditions During Crash Incidents



5.4.1 Crash Heatmap for I-95 Segments

Figure 43 shows the distribution of crashes along I-95. Red areas identify the locations with the highest crash frequencies. It is evident from the map that there are certain locations or "hotspots" within the study area where a higher concentration of crashes occurred. In general, the locations adjacent to on- or off-ramps experienced a greater number of crashes. In ramp areas, drivers are more likely to weave, merge or diverge, making the interaction between vehicles more complex. Appendix B includes the crash diagrams for each of the segments along I-95 and provides a crash summary for each segment.

Figure 43: Distribution of Crashes along I-95



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5.4.3 High Crash Sections on I-95 Mainline

Although the statistics for the entire I-95 study area provide an overall safety perspective, a closer look is needed for the individual segments by direction to determine how the crashes are distributed within each segment. Each segment was further divided into multiple sections by change in AADT. The total crashes and injury crashes for each section were counted from the collision diagrams. A summary of the crashes for all segments along the I-95 mainline is presented in Table 12.

In general, sections near off-ramps experienced higher crash rates than other sections. Lane changing maneuvers increase near ramps and vehicles also slow down near off-ramps. This creates potential conflicts and is a possible contributing factor for the high number of crashes observed near ramps.

Table 12: I-95 Mainline Crashes

Segment		Description	Start MP	End MP	Length (miles)	Total Crashes	Fatal & Injury Crashes	AADT	Total Crash Rate (Crashes/ 100 MVM)	Injury Crash Rate (Crashes/ 100 MVM)	Ranking (Injury Crash Rate)	Ranking (Total Crash Rate)
Northbound												
1	Section 1	Stamford Town Line to Exit 6 Off-Ramp	6.29	6.35	0.06	29	8	69530	327	90	7	8
	Section 2	Exit 6 Off-Ramp to West Avenue	6.35	6.63	0.28	119	30	61275	326	82	8	9
2	Section 1	West Avenue to Exit 6 On-Ramp	6.63	6.8	0.17	37	11	61275	167	50	18	21
	Section 2	Exit 6 On-Ramp to 310 ft south of Fairfield Ave	6.8	6.95	0.15	49	15*	74694	206	63	12	17
3	Section 1	310 ft south of Fairfield Ave to Exit 7 Off-Ramp	6.95	7.06	0.11	70	18	74694	400	103	4	3
	Section 2	Exit 7 Off-Ramp to Greenwich Avenue	7.06	7.3	0.24	77	15	66245	228	44	20	15
4	Section 1	Greenwich Avenue to Exit 8 Off-Ramp	7.3	7.58	0.28	155	40	66245	393	101	5	4
	Section 2	Exit 8 Off-Ramp to Atlantic Street	7.58	7.69	0.11	83	14	53247	666	112	3	2
5	Section 1	Atlantic Street to Exit 7 On-Ramp	7.69	8.1	0.41	41	7*	53247	88	15	30	28
	Section 2	Exit 7 On-Ramp to Elm Street	8.1	8.2	0.1	42	8	64152	308	59	14	13
6	Section 1	Elm Street to Exit 8 On-Ramp	8.2	8.44	0.24	26	8	64152	79	24	25	30
	Section 2	Exit 8 On-Ramp to Myrtle Avenue	8.44	8.58	0.14	24	6	73137	110	28	24	25
7	Section 1	Myrtle Avenue to 180 ft south of Blachley Road	8.58	9.02	0.44	159	38	73137	232	55	16	14
8	Section 1	180 ft south of Blachley Road to Exit 9 Off-Ramp	9.02	9.13	0.11	120	25	73137	701	146	2	1
	Section 2	Exit 9 Off-Ramp to Exit 9 On-Ramp	9.13	9.28	0.15	65	14	65005	313	67	11	11
	Section 3	Exit 9 On-Ramp to Stamford Town Line	9.28	9.63	0.35	49	19	72703	90	35	22	27
Southbound												
1	Section 2	Exit 6 On-Ramp to Stamford Town Line	6.33	6.29	0.04	8	1	71127	132	17	28	23
	Section 1	West Avenue to Exit 6 On-Ramp	6.63	6.33	0.3	39	3	61758	99	8	31	26
2	Section 2	Exit 6 Off-Ramp to West Avenue	6.8	6.63	0.17	69	13	61758	309	58	15	12
	Section 1	310 ft south of Fairfield Ave to Exit 6 Off-Ramp	6.95	6.8	0.15	94	18	75547	390	75	10	5
3	Section 1	Greenwich Avenue to 310 ft south of Fairfield Ave	7.3	6.95	0.35	92	22	75547	164	39	21	22
4	Section 3	Exit 7 On-Ramp to Greenwich Avenue	7.38	7.3	0.08	4	2	75547	31	16	29	31
	Section 2	Exit 8 On-Ramp to Exit 7 On-Ramp	7.58	7.38	0.2	24	6	65219	86	22	26	29
	Section 1	Atlantic Street to Exit 8 On-Ramp	7.69	7.58	0.11	23	6	53677	183	48	19	20
5	Section 2	Exit 7 Off-Ramp to Atlantic Street	8.1	7.69	0.41	86	13	53677	184	28	23	19
	Section 1	Elm Street to Exit 7 Off-Ramp	8.2	8.1	0.1	47	28	62446	354	211	1	7
6	Section 1	Myrtle Avenue to Elm Street	8.58	8.2	0.38	103	31	62446	204	61	13	18
7	Section 2	Exit 8 Off-Ramp to Myrtle Avenue	8.66	8.58	0.08	34	8	62446	320	75	9	10
	Section 1	180 ft south of Blachley Road to Exit 8 Off-Ramp	9.02	8.66	0.36	122	30	74152	215	53	17	16
8	Section 3	Exit 9 On-Ramp to 180 ft south of Blachley Road	9.06	9.02	0.04	0	0	74152	0	0	32	32
	Section 2	Exit 9 Off-Ramp to Exit 9 On-Ramp	9.49	9.06	0.43	66	11	63850	113	19	27	24
	Section 1	Stamford Town Line to Exit 9 Off-Ramp	9.63	9.49	0.14	79	21	74664	355	94	6	6

** Fatality reported at this location.*

The top 10 segments along the I-95 mainline that had the highest crash rates are described below.

Rank 1: Segment 8-Section 1- Northbound I-95 - 180 feet south of Blachley Road to Exit 9 Off-Ramp

In the northbound direction, section 1 of segment 8 ranked 1st based on total crash rates among all segments. Segment 8 starts south of Blachley Road and ends at the Stamford town line. The first section is from 180 ft south of Blachley Road to the Exit 9 off-ramp. There was a total of 120 crashes in this section and 25 of them included injuries. This section also ranked 2nd based on injury crash rates. The majority (56%) of crashes in this section were rear-ends.

Rank 2: Segment 4-Section 2- Northbound I-95 - Exit 8 Off-Ramp to Atlantic Street

Section 2 of segment 4 ranked 2nd based on total crash rates. Segment 4 starts at Greenwich Avenue and ends at Atlantic Street. The second section is from the Exit 8 off-ramp to Atlantic Street. There was a total of 83 crashes in this section and 14 of them included injuries. This section also ranked 3rd based on injury crash rates. The majority (75%) of crashes in this section were rear-ends.

Rank 3: Segment 3-Section 1- Northbound I-95 - 310 feet south of Fairfield Avenue to Exit 7 Off-Ramp

Section 1 of segment 3 ranked 3rd based on total crash rates. Segment 3 starts south of Fairfield Avenue and ends at Greenwich Avenue. The first section is from 310 feet south of Fairfield Avenue to the Exit 7 off-ramp. There was a total of 70 crashes in this section and 18 of them included injuries. This section ranked 4th based on injury crash rates. The majority (67%) of crashes in this section were rear-ends.

Rank 4: Segment 4-Section 1- Northbound I-95 - Greenwich Avenue to Exit 8 Off-Ramp

Section 1 of segment 4 ranked 3rd based on total crash rates. Segment 4 starts at Greenwich Avenue and ends at Atlantic Street. The first section is from Greenwich Avenue to the Exit 8 off-ramp. There was a total of 155 crashes in this section and 40 of them included injuries. This section also ranked 5th based on injury crash rates. The majority (74%) of crashes in this section were rear-ends.

Rank 5: Segment 2-Section 1- Southbound I-95 - 310 feet south of Fairfield Avenue to Exit 6 Off-Ramp

Section 1 of Segment 2 ranked 1st based on total crash rates among all segments. Segment 2 starts south of Fairfield Ave and ends at West Avenue. The first section is from 310 ft south of Fairfield Ave to the Exit 6 Off-Ramp. There was a total of 94 crashes in this section and 18 of them included injuries. This section ranked 10th based on injury crash rates. The majority (82%) of crashes in this section were rear-ends.

Rank 6: Segment 8-Section 1- Southbound I-95 - Stamford Town Line to Exit 9 Off-Ramp

Section 1 of Segment 8 ranked 2nd based on total crash rates. Segment 8 starts at Stamford Town Line and ends south of Blachley Road. The first section is from the Stamford town line to the Exit 9 off-ramp. There was a total of 79 crashes in this section and 21 of them included injuries. This section ranked 6th based on injury crash rates. The majority (71%) of crashes in this section were rear-ends.

Rank 7: Segment 5-Section 1- Southbound I-95 - Elm Street to Exit 7 Off-Ramp

Section 1 of Segment 5 ranked 3rd based on total crash rates. Segment 5 starts at Elm Street and ends at Atlantic Street. The first section is from Elm Street to the Exit 7 Off-ramp. There was a total of 47 crashes in this section and 28 of them included injuries. This section ranked 1st based on injury crash rates. The majority (55%) of crashes in this section were rear-ends.

Rank 8: Segment 1-Section 1- Northbound I-95 - Stamford Town Line to Exit 6 Off-Ramp

Section 1 of Segment 1 ranked 5th based on total crash rates. Segment 1 starts at the Stamford town line and ends at West Avenue. The first section is from Stamford town line to the Exit 6 off-ramp. There was a total of 29 crashes in this section and 8 of them included injuries. This section also ranked 7th based on injury crash rates. The majority (79%) of crashes in this section were rear-ends.

Rank 9: Segment 1-Section 2- Northbound I-95 - Exit 6 Off-Ramp to West Avenue

Section 2 of Segment 1 ranked 9th based on total crash rates. Segment 1 starts at the Stamford town line and ends at West Avenue. The second section is from the Exit 6 off-ramp to West Avenue. There was a total of 119 crashes in this section and 30 of them included injuries. This section ranked 8th based on injury crash rates.

Rank 10: Segment 7-Section 2- Southbound I-95 - Exit 8 Off-Ramp to Myrtle Avenue

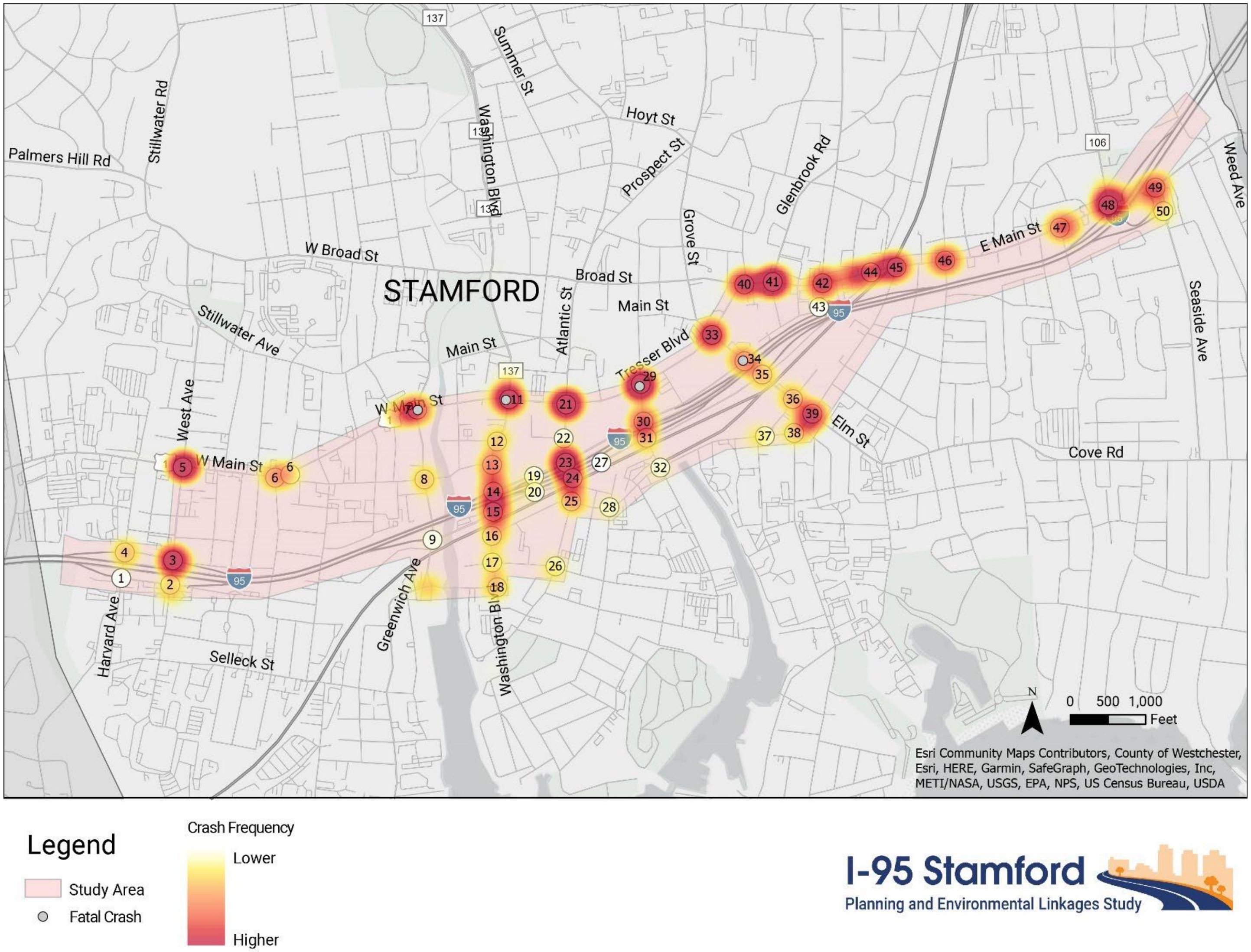
Section 2 of Segment 7 ranked 4th based on total crash rates. Segment 7 starts south of Blachley Road and ends at Myrtle Avenue. The second section is from the Exit 8 off-ramp to Myrtle Avenue. There was a total of 34 crashes in this section and 8 of them included injuries. This section ranked 9th based on injury crash rates. The majority (55%) of crashes in this section were rear-ends.

5.5 Intersection Crash Analysis

5.5.1 Crash Heatmap for Study Intersections

Figure 44 shows the distribution of crashes amongst the study intersections. This crash heat map identifies certain corridors, such as Atlantic Street, Elm Street, and parts of Route 1 where high crash frequencies were observed. Intersection geometry, vertical curvature, high density of access points and high traffic volumes are some of the contributing factors for high crash occurrences within these corridors.

Figure 44: Crash Heatmap for Study Intersections



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5.5.2 Fatal Crashes at Study Intersections

There were four fatalities reported during the study period. One fatality occurred at the intersection of Route 1 (Tresser Boulevard) and Greenwich Avenue, where two pedestrians were hit by a vehicle while crossing Tresser Boulevard along the south leg of the intersection. One of the pedestrians was fatally injured and the other sustained a serious injury.

Another fatality occurred at the intersection of Elm Street and North State Street. The pedestrian was crossing North State Street and was hit by a vehicle. According to the crash report, the driver was not paying attention.

The intersection of Route 1 (Tresser Boulevard) and Washington Boulevard also experienced a pedestrian fatality. The pedestrian fatality occurred when a vehicle failed to yield to the pedestrian crossing Route 1.

At the intersection of Canal Street with Route 1 (Tresser Boulevard), one fatal crash was reported. It happened when a vehicle turning left failed to yield to a motorcycle travelling straight. Both vehicles were travelling along Tresser Blvd. The driver of the motorcycle was not wearing a helmet and sustained a fatal injury.

5.5.3 High Crash Intersections

Within the study area, only two intersections are unsignalized and the remaining 48 intersections are signalized. Among all the study intersections, five intersections (10%) experienced more than 100 crashes, 18 intersections (36%) experienced between 50 and 100 crashes, 16 intersections (32%) experienced between 20 and 50 crashes and 11 intersections (22%) experienced less than 20 total crashes. Table 13 summarizes the crashes by intersections and ranks them based on crash rates. Appendix B provides crash diagrams and a crash summary for each study intersection.

Table 13: Crash Summary by Intersections

No.	Intersection Name	Primary Crash Type	Pedestrian & Bicycle Crashes	Fatal & Injury Crashes **	Total Crashes **	Total Entering Vehicle	Number of Years	Crash Rate (Crashes/MEV)	Ranking by Crash Rate
1	Harvard Ave at Baxter Ave / NB I-95 Exit 6 Off-Ramp	Angle, Sideswipe	0	1	2	21173	5.83	0.04	50
2	West Ave at Baxter Ave / NB I-95 Exit 6 On-Ramp	Rear End	0	3	25	30268	5.83	0.39	43
3	West Ave at Grenhart Rd / SB I-95 Exit 6 Off-Ramp	Sideswipe	0	13	89	28427	5.83	1.47	11
4	Harvard Ave at Grenhart Rd / SB I-95 Exit 6 On-Ramp	Angle	1	7	28	23334	5.83	0.56	39
5	U.S. Route 1 (West Main St) at West Ave	Sideswipe, Rear-End	3	24	97	26539	5.83	1.72	4
6	U.S Route 1 (West Main St) at Richmond Hill Ave	Angle, Sideswipe	2	11	50	15779	5.83	1.49	10
7	Greenwich Ave at U.S. Route 1 (Tresser Blvd)	Rear End	5	15*	79	23878	5.83	1.55	9
8	Greenwich Ave at Richmond Hill Ave	Angle	1	9	25	16170	5.83	0.73	32
9	Greenwich Ave at NB I-95 Exit 7 Off-Ramp	Angle, Rear End	0	2	5	14509	5.83	0.16	48
10	Greenwich Ave at Pulaski St	Angle	2	6	25	16155	5.83	0.73	31
11	Washington Blvd at U.S. Route 1 (Tresser Blvd)	Rear End	3	22*	113	36843	5.83	1.44	13
12	Washington Blvd at Division St	Rear End, Sideswipe	0	2	23	18531	5.83	0.58	37
13	Washington Blvd at Richmond Hill Ave	Sideswipe	4	13	39	23111	5.83	0.79	29
14	Washington Blvd at N State St / SB I-95 Exit 7 On-Ramp	Rear End	4	9	50	24874	5.83	0.94	24
15	Washington Blvd at S State St	Sideswipe	2	8	56	27545	5.83	0.96	23
16	Washington Blvd at Station Place	Sideswipe	3	5	39	17371	5.83	1.06	21
17	Washington Blvd at Henry St	Rear End	1	4	17	16326	5.83	0.49	41
18	Washington Blvd at Pulaski St	Rear End	1	4	26	18280	5.83	0.67	33
19	North State St at Guernsey Ave	Sideswipe	0	1	11	6981	5.83	0.74	30
20	South State Street) at Guernsey Ave	Rear End, Sideswipe	0	0	11	3981	5.83	1.30	16
21	Atlantic St at U.S. Route 1 (Tresser Blvd)	Angle	4	23	112	27908	5.83	1.89	2
22	Atlantic St at Federal St	Angle	0	2	6	11444	5.83	0.25	47
23	Atlantic St at North State St	Angle	4	21	90	27128	5.83	1.56	8
24	Atlantic St at State Route 790 (South State St)	Angle	3	15	53	14905	5.83	1.67	6
25	Atlantic St at Station Place & Dock St	Rear End	1	5	32	17248	5.83	0.87	27

No.	Intersection Name	Primary Crash Type	Pedestrian & Bicycle Crashes	Fatal & Injury Crashes **	Total Crashes **	Total Entering Vehicle	Number of Years	Crash Rate (Crashes/MEV)	Ranking by Crash Rate
26	Atlantic St at Henry St	Angle	0	4	19	10567	5.83	0.84	28
27	South State St at NB I-95 Exit 8 Off-Ramp	Angle	0	2	6	18653	5.83	0.15	49
28	Pacific St at Dock St	Angle, Sideswipe	0	3	10	16964	5.83	0.28	46
29	Canal St at U.S. Route 1 (Tresser Blvd)	Rear End	2	23*	115	37052	5.83	1.46	12
30	Canal St at North State St	Angle	2	11	48	35152	5.83	0.64	34
31	Canal St at State Route 790 (South State St)	Sideswipe	1	11	25	36341	5.83	0.32	45
32	Canal St at Dock St / Jefferson St	Angle	2	21	80	29155	5.83	1.29	17
33	Elm St at U.S. Route 1 (Tresser Blvd / East Main St)	Rear End	3	21	83	44383	5.83	0.88	26
34	Elm St at North State St	Angle, Sideswipe	5	7*	42	31659	5.83	0.62	35
35	Elm St at State Route 790 (South State St)	Sideswipe	0	1	27	33071	5.83	0.38	44
36	Elm St at Elm Ct / Cherry St	Rear End	3	7	29	22975	5.83	0.59	36
37	Cherry St at Jefferson St	Rear End	1	6	17	19295	5.83	0.41	42
38	Jefferson St at Magee Ave	Angle, Rear End	2	6	26	20951	5.83	0.58	38
39	Elm St at Jefferson St / Myrtle Ave	Rear End	0	7	64	33812	5.83	0.89	25
40	U.S. Route 1 (East Main St) at Broad St	Sideswipe	3	15	73	25841	5.83	1.33	15
41	U.S. Route 1 (East Main St) at Glenbrook Rd	Sideswipe	5	17	123	28280	5.83	2.04	1
42	U.S. Route 1 (East Main St) at Lafayette St	Angle	6	18	74	20452	5.83	1.70	5
43	Lafayette St at North State St & South State St	Angle	0	1	3	1203	5.83	1.17	18
44	U.S. Route 1 (East Main St) at North State St	Angle	2	23	80	20932	5.83	1.80	3
45	U.S. Route 1 (East Main St) at Myrtle Ave	Angle	3	22	90	26889	5.83	1.57	7
46	U.S Route 1 (East Main St) at Lockwood Ave	Rear End	3	16	63	26042	5.83	1.14	19
47	U.S Route 1 (East Main St) at Blachley Rd	Rear End	5	16	59	28022	5.83	0.99	22
48	U.S. Route 1 at U.S. Route 106 / SB I-95 Exit 9 Ramps	Rear End	2	33	134	44615	5.83	1.41	14
49	U.S. Route 1 at Seaside Ave / NB I-95 Exit 9 On-Ramp	Rear End, Sideswipe	3	12	64	28342	5.83	1.06	20
50	Seaside Ave at NB/SB I-95 Exit 9 Ramps	Angle	0	1	18	15722	5.83	0.54	40

*** Fatality reported at this location.**
**** Includes Pedestrian and Bicyclist crashes.**

The top 10 intersections with the highest crash rates are discussed below:

Rank 1: Intersection #41: Route 1 (East Main Street) at Glenbrook Road

This location ranked 1st among all the study intersections with 2.04 crashes per million entering vehicles. There were 123 total crashes and 17 injury crashes at this location. The largest proportion of crashes were sideswipe (35%). There are multiple driveway access points along Route 1 and Clarks Hill Avenue near this intersection. These are potential conflict points that contribute to both angle and sideswipe crashes. Street parking exists along westbound Route 1 at this location, which can be another source of conflict. Five pedestrian crashes were reported at this location during the study period. All of the pedestrian crashes occurred when turning vehicles failed to yield the right of way to crossing pedestrians.

Rank 2: Intersection #21: Atlantic Street at Route 1 (Tresser Boulevard)

This location ranked 2nd among all the study intersections with 1.89 crashes per million entering vehicles. There were 112 total crashes and 23 injury crashes at this location. The largest proportion of crashes were angle (49%). This is a large intersection with dedicated left-turn lanes on each approach. The traffic volumes are also high for all approaches. Four pedestrian crashes were reported at this location during the study period.

Rank 3: Intersection #44: Route 1 (East Main Street) at North State Street

This location ranked 3rd among all the study intersections with 1.80 crashes per million entering vehicles. There were 80 total crashes and 23 injury crashes at this location. The largest proportion of crashes were angle (44%). There are several restaurants and other businesses on Route 1 at this location. Vehicles accessing these businesses often create conflicts with vehicles travelling along Route 1. Two pedestrian crashes were reported at this location during the study period.

Rank 4: Intersection #5: Route 1 (West Main Street) at West Avenue

This intersection ranked 4th among all the study intersections with 1.72 crashes per million entering vehicles. There were 97 total crashes and 24 injury crashes at this location. The largest proportion of crashes were sideswipe (34%). There is street parking along West Avenue and several businesses exist along Route 1 and West Avenue with access points within the functional area of the intersection. Three pedestrian crashes were reported at this location during the study period.

Rank 5: Intersection #42: Route 1 (East Main Street) at Lafayette Street

This intersection ranked 5th among all the study intersections with 1.70 crashes per million entering vehicles. There were 74 total crashes and 18 injury crashes at this location. The largest proportion of crashes were angle (36%). Like other intersections on Route 1, this location also has a high density of access driveways, which results in conflicting movements in proximity to the intersection. Six pedestrian crashes were reported at this location during the study period. This location also experienced one bicycle crash.

Rank 6: Intersection #24: Atlantic Street at South State Street (State Route 790)

This intersection ranked 6th among all the study intersections with 1.67 crashes per million entering vehicles. There were 53 total crashes and 15 injury crashes at this location. The majority of crashes were angle (60%). This intersection is close to the train station and I-95 which results in a high volume of traffic moving through the intersection. Two pedestrian crashes and one bicycle crash were reported at this location during the study period.

Rank 7: Intersection #45: Route 1 (East Main Street) at Myrtle Ave

This intersection ranked 7th among all the study intersections with 1.57 crashes per million entering vehicles. There were 90 total crashes and 22 injury crashes at this location. The largest proportion of crashes were angle (32%). The overhead railroad at this intersection creates a vertical obstruction. Several fixed object crashes were reported at this location where vehicles hit the bottom of the bridge. There is also a stopcontrolled intersection within 250 feet of this intersection, which is another contributing factor to the number of angle crashes. Three bicycle crashes were reported at this location during the study period.

Rank 8: Intersection #23: Atlantic Street at North State Street

This intersection ranked 8th among all the study intersections with 1.56 crashes per million entering vehicles. There were 90 total crashes and 21 injury crashes at this location. The majority of crashes were angle (53%). Most of these angle crashes occurred because of vehicles disregarding the traffic signal (running red light) or conflicts between left-turning vehicles with vehicles traveling straight from the opposite direction. Four pedestrian crashes were reported at this location during the study period.

Rank 9: Intersection #7: Greenwich Ave at Route 1 (Tresser Boulevard)

This intersection ranked 9th among all the study intersections with 1.55 crashes per million entering vehicles. There were 79 total crashes and 14 injury crashes and one fatal crash at this location. The largest proportion of crashes were rear-end (34%). This is a wide intersection with high traffic volumes. The Stillwater Avenue and Tresser Boulevard intersection is within 250 feet of the intersection and also contributes to crashes at this location. Three pedestrian crashes and two bicycle crashes were reported at this location during the study period.

Rank 10: Intersection #6: Route 1 (West Main St.) at Richmond Hill Avenue

This intersection ranked 10th among all the study intersections with 1.49 crashes per million entering vehicles. There were 50 total crashes and 11 injury crashes at this location. The largest proportion of crashes were angle (34%) and sideswipe (34%). This is a skewed intersection with another signalized intersection within 250 feet of the intersection. Access points to businesses also create potential conflicting movements with vehicles traveling on Route 1. Two pedestrian crashes and one bicycle crash were reported at this location during the study period.

High Pedestrian and Bicycle Crash Intersections

Among the 50 study intersections, 34 experienced a bicycle or pedestrian crash within the study period. There were 97 pedestrian and bicycle crashes within the study area during the analysis period. There was a total of three fatalities and 84 injury crashes related to pedestrian and bicyclists. Most of these crashes were concentrated along Atlantic Street, Washington Boulevard, Elm Street and Route 1. Pedestrian activity is high on these corridors, due to the adjacent densely populated neighborhoods and the wide variety of businesses in the area. Shared lane use signs and pavement markings are present along these corridors to remind drivers to share the road with bicyclists. These corridors also include densely spaced access driveways, which create potential conflicts with vehicles and bicycles.,

Table 14 summarizes the pedestrian and bicycle crashes by intersections and ranks them based on the total number of pedestrian and bicycle crashes.

Table 14: Pedestrian and Bicycle Crash Summary by Intersection

No.	Intersection Name	Fatal Crashes	Injury Crashes	Total Crashes	Ranking by Crash Frequency
1	Harvard Ave at Baxter Ave / NB I-95 Exit 6 Off-Ramp	0	0	0	35
2	West Ave at Baxter Ave / NB I-95 Exit 6 On-Ramp	0	0	0	35
3	West Ave at Grenhart Rd / SB I-95 Exit 6 Off-Ramp	0	0	0	35
4	Harvard Ave at Grenhart Rd / SB I-95 Exit 6 On-Ramp	0	0	1	29
5	U.S. Route 1 (West Main St) at West Ave	0	3	3	10
6	U.S Route 1 (West Main St) at Richmond Hill Ave	0	2	3	10
7	Greenwich Ave at U.S. Route 1 (W Main St / Tresser Blvd)	1	3	5	2
8	Greenwich Ave at Richmond Hill Ave	0	0	0	35
9	Greenwich Ave at NB I-95 Exit 7 Off-Ramp	0	0	0	35

No.	Intersection Name	Fatal Crashes	Injury Crashes	Total Crashes	Ranking by Crash Frequency
10	Greenwich Ave at Pulaski St	0	2	2	21
11	Washington Blvd at U.S. Route 1 (Tresser Blvd)	1	2	3	10
12	Washington Blvd at Division St	0	0	0	35
13	Washington Blvd at Richmond Hill Ave	0	4	4	6
14	Washington Blvd at N State St / SB I-95 Exit 7 On-Ramp	0	3	4	6
15	Washington Blvd at S State St	0	2	2	21
16	Washington Blvd at Station Place	0	3	3	10
17	Washington Blvd at Henry St	0	1	1	29
18	Washington Blvd at Pulaski St	0	1	1	29
19	North State St at Guernsey Ave	0	0	0	35
20	South State Street) at Guernsey Ave	0	0	0	35
21	Atlantic St at U.S. Route 1 (Tresser Blvd)	0	4	4	6
22	Atlantic St at Federal St	0	0	0	35
23	Atlantic St at North State St	0	3	4	6
24	Atlantic St at State Route 790 (South State St)	0	3	3	10
25	Atlantic St at Station Place & Dock St	0	1	1	29
26	Atlantic St at Henry St	0	0	0	35
27	South State St at NB I-95 Exit 8 Off-Ramp	0	0	0	35
28	Pacific St at Dock St	0	0	0	35
29	Canal St at U.S. Route 1 (Tresser Blvd)	0	1	2	21
30	Canal St at North State St	0	2	2	21
31	Canal St at State Route 790 (South State St)	0	1	1	29
32	Canal St at Dock St / Jefferson St	0	2	2	21
33	Elm St at U.S. Route 1 (Tresser Blvd / East Main St)	0	3	3	10
34	Elm St at North State St	1	4	5	2
35	Elm St at State Route 790 (South State St)	0	0	0	35
36	Elm St at Elm Ct / Cherry St	0	3	3	10
37	Cherry St at Jefferson St	0	1	1	29
38	Jefferson St at Magee Ave	0	2	2	21
39	Elm St at Jefferson St / Myrtle Ave	0	0	0	35
40	U.S. Route 1 (East Main St) at Broad St	0	3	3	10
41	U.S. Route 1 (East Main St) at Glenbrook Rd	0	4	5	2
42	U.S. Route 1 (East Main St) at Lafayette St	0	4	6	1
43	Lafayette St at North State St & South State St	0	0	0	35
44	U.S. Route 1 (East Main St) at North State St	0	2	2	21
45	U.S. Route 1 (East Main St) at Myrtle Ave	0	2	3	10
46	U.S Route 1 (East Main St) at Lockwood Ave	0	3	3	10
47	U.S Route 1 (East Main St) at Blachley Rd	0	5	5	2
48	U.S. Route 1 at U.S. Route 106 / SB I-95 Exit 9 Ramps	0	2	2	21
49	U.S. Route 1 at Seaside Ave / NB I-95 Exit 9 On-Ramp	0	3	3	10
50	Seaside Ave at NB/SB I-95 Exit 9 Ramps	0	0	0	35

The intersections with pedestrian/bicycle fatalities are discussed below:

Elm Street at North State Street

This intersection experienced five (5) total pedestrian/bicycle crashes, including one fatality and four injury crashes within the study period. It is ranked 2nd highest in number of pedestrian/bicycle crashes by frequency.

Greenwich Avenue at Route 1 (West Main Street/Tresser Boulevard)

This intersection experienced five (5) total pedestrian/bicycle crashes, including one fatality and three injury crashes within the study period. It is also ranked 2nd highest in number of pedestrian/bicycle crashes by frequency.

Greenwich Avenue at Route 1 (West Main Street/Tresser Boulevard)

This intersection experienced three (3) total pedestrian/bicycle crashes, including one fatality and two injury crashes within the study period. It is ranked 10th highest in number of pedestrian/bicycle crashes by frequency.

6 Review of Infrastructure Data

This section describes the evaluation of existing roadway geometry and structures within the project study area (see Figure 1). Locations which do not currently meet highway design guidelines and other pertinent criteria are identified, as well as their potential impacts to traffic operations and safety.

I-95 and its associated interchange connections were constructed in 1958. As such, these facilities were designed to what was considered to be the appropriate design standards and design year (1975) traffic volumes at that time. Since these interchanges have been designed, traffic volumes along the I-95 corridor have continued to increase, resulting in further increases in congestion and deteriorations in safety.

6.1 Review of Infrastructure Data Sources

Roadway geometries evaluated within this section of the memo have been compared to the latest available standards, which are further identified below:

- CTDOT, Highway Design Manual (2003 Edition including Revisions to February 2013) (CTHDM); and
- American Association of State Highway and Transportation Officials (AASHTO), A Policy on Geometric Design of Highways and Streets (6th Edition, 2011).

Bridge condition information are obtained from the most recent inspection reports available on within the CTDOT COMPASS "Asset - Bridges" folder.

6.2 Roadway Geometry Review

6.2.1 Methodology

Using information gathered from several field visits, and ongoing reviews of the latest record plans, photogrammetry and survey data, inspection reports, and the latest available design criteria, geometric conditions were evaluated for Interstate 95 and local roads.

This review includes an evaluation of the following design criteria:

- Posted Speed Limit;
- Lane Width;
- Shoulder Width;
- Horizontal Curvature;
- Superelevation;
- Cross Slope;
- Stopping Sight Distance;
- Maximum / Minimum Grades; and
- Vertical Clearance.

In addition to controlling design, the following operational characteristics were also reviewed:

- Interchange Spacing; and
- Highway Ramp Weaving.

Minimum design values for each controlling design criteria listed are predicated on roadway classifications and selected corresponding speed. Based on the latest functional classification maps, CTDOT classifies roads within the project limits as shown in Table 15.

Table 15: Functional Classifications

Roadway	Facility Carried
Interstate 95	Urban Principal Arterial – Interstate
Route 1	Urban Principal Arterial – Other
Washington Blvd	Urban Principal Arterial – Other
Harvard Ave	Urban Collector
West Ave	Urban Minor Arterial
Fairfield Ave	Urban Minor Arterial
Greenwich Ave	Urban Minor Arterial
Atlantic Ave	Urban Minor Arterial
Canal St	Urban Collector
Elm St	Urban Minor Arterial
Jefferson St	Urban Collector
Myrtle Ave	Urban Collector
Maple Ave	Urban Local
Lockwood Ave	Urban Collector
Maher Rd	Urban Local
Blachley Rd	Urban Local
Seaside Ave	Urban Minor Arterial
North State St	Urban Minor Arterial
South State St	Urban Minor Arterial
Cove Rd	Urban Minor Arterial

For this project, the following geometric conditions were reviewed based on recommended standards from the CTHDM (Figure 5A) for Urban Freeways. Table 16 below summarizes typical design criteria for this class of highway for comparison purposes. Further information on each design element follows.

Table 16: Interstate 95 Design Criteria

Design Element	Unit	Standard
Design Speed	mph	55
Travel Lane width	ft	12
Shoulder Width Left (Approach road)	ft	8' (4' Paved + 4' Graded)
Shoulder Width Right (Approach road)	ft	10
Shoulder Width Left (Bridge)	ft	10
Shoulder Width Right (Bridge)	ft	10
Cross Slope (Travel Lane)	%	1.5-2.0
Cross Slope (Shoulder)	%	4-6
Bridge Width/Cross Slope	ft, %	Meet approach roadway width and cross slope
Roadside Clear Zones	ft	20 to 26, depending on shelf

Design Element	Unit	Standard
Stopping Sight Distance (Vertical)	ft	495
Stopping Sight Distance (Horizontal)	ft	495
Minimum Radius	ft	1,065
Superelevation Rate	%	6 Max
Maximum Grade	%	5
Vertical Curvature (K value) Crest		114
Vertical Curvature (K value) Sag		N/A
Min Length of Horizontal curve	ft	1,650
Median Width (Includes Left Shoulder)	ft	30" Median + 6'-12' Shoulder on each side

Posted Speed Limit

The posted speed limit creates a driver expectation of safe operating speed on a highway or interchange ramp. The posted speed is evaluated for each facility based on the following factors:

- 85th percentile speed
- Roadway geometrics
- Functional classification and type of area
- Type and density of roadside development
- Crash experience
- Pedestrian activity

I-95 is the mainline roadway within the project area and has posted speed limit of 55 mph. There are eleven ramps connecting I-95 to local roadways interchanges. Several off ramps have posted advisory speeds of 25 mph. Most ramps do not have posted advisory speed limits.

Lane Width

Based on the CTHDM and AASHTO, lane widths will influence the level of comfort for motorists and may vary between 9 feet and 12 feet depending on the functional classification and traffic volumes. On I-95, travel lanes in both directions are 12 feet wide. In general, the on- and off-ramps to I-95 have a 12-foot travel lane.

Shoulder Width

Based on AASHTO Section 4.4.1, the general description of a shoulder is the portion of the roadway contiguous with the travel way that accommodates stopped vehicles, emergency use and lateral support of subbase, base, and surface course. The “useable” width of a shoulder is the actual width that can be used when a driver makes an emergency or parking stop.

Several of the traffic-related advantages of a well-designed and maintained shoulder are as follows:

- Space is provided away from the travel lanes for vehicles to stop (emergencies, flat ties, mechanical difficulties, consult maps);
- Space is provided for evasive maneuvers to avoid potential crashes or reduce their severity;
- Space is provided for temporary storage of debris and snow until it can be completely removed;
- Sight distance is improved in cut sections, thereby potentially improving safety;
- Shoulder serves as a buffer between travel way and roadside as part of the clear zone;
- Highway capacity is improved because uniform speed is encouraged; and

- Space is provided for maintenance operations such as snow removal and storage.

For the project corridor, the CTHDM (Figure 5A, including footnote 1) requires that both the left and right shoulders should be 12 feet for 6+ lanes. Warrant for 16 foot shoulders for high-volume/incident management sites have not been assessed by CTDOT at this time. For a typical one lane or two lane ramp sections, the CTHDM (Section 12-4.02) requires a standard 4-foot left shoulder and 10-foot right shoulder.

Horizontal Curvature

For I-95 mainline, Figure 5A in the CTHDM requires a minimum radius of 1,065 feet for a 55 mph design speed. Figure 8-2A in the CTHDM list minimum radius for ramps (excluding loop ramps). Figure 8-2A indicated a minimum radius for 30 mph is 275 feet with a superelevation rate of 6.0 percent.

Loop ramps may be designed for 25 mph where mainline design speeds are greater than 50 mph. Figure 11-4D "Minimum Radii for Turning Roadways" in the CTHDM may be used for loops ramp minimum radii where a 25 mph design speed requires a 145 foot minimum radius with a superelevation of 6.0 percent. Ramp radii for on- and off-ramps within the project limits meet the minimum criteria for horizontal curvature.

Superelevation

Superelevation is the amount of cross slope or "bank" provided on a horizontal curve to help counterbalance, in combination with side friction, the centrifugal force of a vehicle traveling on a curve. There is a specific superelevation for each combination of horizontal curve and speed of travel. Based on Figure 5A "Urban Freeways" in the CTHDM for the I-95 criteria, the maximum superelevation (e max) is 6.0 percent for 55 mph. The maximum superelevation rate for ramps is also 6.0 percent per CTHDM Section 12-4. Curved sections of I-95 throughout the project limits require superelevation.

Cross Slope

Surface cross slopes are required for proper drainage of travel lanes on tangent sections. As per the CTHDM, cross slopes for freeways that do not require superelevation should be between 1.5 percent and 2.0 percent sloping away from the roadway centerline. Cross slopes on ramps not requiring superelevation should be 1.5 percent.

Stopping Sight Distance

Stopping Sight Distance (SSD) is the sum of the distance traveled during a driver's perception/reaction or braking reaction time and the distance traveled while braking to a stop. SSD is evaluated based on both the vertical and horizontal roadway geometry. Fixed objects may restrict the line of sight of motorists on the inside of horizontal curves. Per Figure 5A in the CTHDM a SSD of 495 feet is required for a 55 mph design speed. A design speed of 30 mph on ramps requires a 200-foot SSD.

Maximum / Minimum Grade

Longitudinal grades will significantly impact vehicular operations and safety. Maximum grades are based on functional classification, urban/rural location and design speeds. The maximum length of an upgrade should also be considered for truck traffic. The minimum longitudinal grade for all types of roadways is 0.5 percent to provide for surface drainage. For I-95, the CTHDM requires a maximum grade of 5 percent for a design speed of 55 mph. Figure 12-4C in the CTHDM gives maximum ramp grades ranging between 6-8 percent for 25 mph, between 5-7 percent for 30 mph, and between 4-6 percent 40 mph. Existing longitudinal grades on roadways within the project area fall within the required maximum grade criteria.

Vertical Clearance

Vertical clearance is the distance above a roadway that is free of obstructions. The minimum vertical clearance depends on its functional classification. Table 17 below is based on Section 9-4 of the CTHDM which summarizes the minimum vertical clearances for new bridges for various highway classifications and conditions. These minimum clearances apply to the entire roadway width. Table 18 summarizes existing vertical clearances for the twelve bridges within study area limits.

Table 17: Bridge Vertical Clearance Requirements

Roadway Type	Clearance
Freeway/Expressway/Arterial Under	16’-3”
Collector/Local Under	14’-6”
Railroad Under Highway (electrified)	22’-6”
Railroad Under Highway (non-electrified)	20’-6”
Railroad Under Freeway	23’-0”
Highway Under Sign Truss or Pedestrian Bridge	17’-3”

Table 18: Existing Bridge Vertical Clearances

Bridge	Facility Carried	Feature Crossed	Vertical Clearance (Item 54)
00026	Interstate 95	Rippowam River & Greenwich Ave	14'-6"
00027	Interstate 95	Washington Blvd (SSR 493)	13'-9"
00028	Interstate 95	Atlantic St	15'-0"
00029	Interstate 95	Canal St	16'-4"
00031	Interstate 95	Elm St	18'-2"
00032	Interstate 95	Metro-North Railroad and Local Roads	29'-0"
00033	Interstate 95	Maple Ave	16'-10"
00034	Interstate 95	Lockwood Ave	20'-6"
00035	Maher Rd	Interstate 95	14'-9"
00036	Blachley Rd	Interstate 95	24'-2"
02567	Interstate 95	Brook	n/a
06584	Interstate 95 Ramp 350	Metro-North Railroad and Local Roads	31'-5"

6.2.2 Review of Mainline Geometrics

This section presents the review of geometric conditions along I-95. It should be noted that, in the following table, values denoted in red indicate values that are not to standard.

Interstate 95

The I-95 corridor is comprised of several tangent sections with five horizontal curves through the project area between the Rippowam River and Exit 10. Three of the curves are slight back-to-back reverse curves between Washington Blvd and Elm St. The controlling minimum radius within the project limits is 2,292 feet.

The vertical alignment has a local high point as I-95 crosses over Metro-North Railroad, with longitudinal slopes approaching 3% maximum.

A summary of mainline geometrics for the I-95 corridor is presented in Table 19.

Table 19: I-95 Mainline Design Elements

Controlling Criteria	Design Element	Unit	Standard	Existing
*	Design Speed	mph	55	55 (Posted)
*	Travel Lane width	ft	12	12-13
*	Shoulder Width Left (Approach road)	ft	8' (4' Paved + 4' Graded)	4
*	Shoulder Width Right (Approach road)	ft	12	Varies 4-10
*	Shoulder Width Left (Bridge)	ft	12	Varies 2-3
*	Shoulder Width Right (Bridge)	ft	12	Varies 2-3
*	Cross Slope (Travel Lane)	%	1.5-2.0	1.04
*	Cross Slope (Shoulder)	%	4-6	4.17
*	Stopping Sight Distance (Vertical)	ft		
*	SB Curve 1		495	722
*	SB Curve 2		495	528
*	NB Curve 1		495	600
*	Stopping Sight Distance (Horizontal)	ft		
*	NB		495	396
*	SB		495	424
	Minimum Radius	ft	1,065	2,292
*	Superelevation Rate	%	6% Max	~5% Max
*	Maximum Grade	%	5	3
	Vertical Curvature (K value) Crest			
	SB Curve 1		114	167
	SB Curve 2		114	129
	NB Curve 1		114	167
	Vertical Curvature (K value) Sag		N/A	N/A
	Min Length of Horizontal curve	ft	1,650	1,073
	Median Width (Includes Left Shoulder)	ft	30" Median + 6'-12' Shoulder on each side	36" Median + 2'-6" Shoulder on each side

Note: Red text denotes design element does not meet standard.

6.2.3 Review of Interchange Geometrics

This section presents the review of geometric conditions at study area interchanges. It should be noted that, in the following tables, values denoted in red indicate values that are not to standard.

Interstate 95 Corridor

Within the project corridor between Greenwich Ave (NB) / Washington Boulevard (SB) and Elm Street, North and South State Streets serve as frontage roads for several closely spaced on- and off-ramps that comprise Exits 7 through 9. The frontage roads are further intersected by local roads Atlantic Street and Canal Street. AASHTO defines a “ramp” as all types, arrangements, and sizes of turning roadways that connect two or more legs at an interchange. Ramp types are defined by their geometry consisting of direct connections, semi-direct connections, diagonals, loops and outer connections are generally designed with one-way traffic. Ramp design speeds are based on their horizontal curvature, vertical alignment and stopping sight distance. Design speeds for ramps listed in Figure 12-4A of the CTHDM, and as shown below in Table 20, refer to the ramp proper, not the highway / ramp junction. Table 20 provides acceptable ranges of ramp design speed based on the design speed of the mainline. The design speed for direct connections should not be less than 40 mph while design speeds for semi-direct ramps should be between the mid and high ranges but not less than 30 mph. For mainline speeds greater than 50 mph, loop ramp design speeds should not be less than 25 mph.

Table 20: Ramp Design Speed Requirements

Mainline Design Speed	45 MPH	50 MPH	55 MPH	60 MPH	65 MPH	70 MPH
High-Range (85%) Ramp Design Speed	40	45	50	50	55	60
Mid-Range (70%) Ramp Design Speed	35	35	40	45	45	50
Low-Range (50%) Ramp Design Speed	25	25	30	30	30	35

Minimum radius for all ramps, except loop ramps, are indicated in Figure 8-2A of the CTHDM. Minimum radius for 30 mph is 275', 35 mph is 385' and 40 mph is 510. Loop ramp follow the criteria in Figure 11-4D of the CTHDM for "Minimum Radii for Turning Roadways" where a design speed of 25 mph requires a 145' radius with a 6.0 percent superelevation.

Deceleration and acceleration ramps should provide safety to comfortably allow vehicles to exit or enter the highway. Minimum lengths of deceleration lanes are given in Figure 12-3D of the CTHDM and are based on the mainline design speed and the ramp design speed. These deceleration lengths should also be adjusted for upgrades and downgrades greater than 3 percent.

The acceleration and deceleration lane lengths within the study area between Exit 6 and Exit 9 are shown in the tables below. All required lengths were based on a design speed of 55 mph for the highway and 30 mph for the ramps with the exception of the southbound exit to Elm Street where there is a speed advisory sign for 40 mph. Ramp acceleration / deceleration lane lengths that were shorter than the required lengths are as follows:

- West Avenue deceleration lane from Southbound I-95
- Route 1 acceleration land to southbound I-95
- Harvard Ave deceleration lane from northbound I-95
- Route 1 deceleration lane from northbound I-95
- West Avenue acceleration lane to southbound I-95

Ramp Deceleration Lanes from Southbound I-95

Exit To:	Mainline Design Speed	Ramp Design Speed	Required Deceleration Length	Actual Deceleration Ramp Length
Elm Street	55 mph	40 mph	285'	1,540 (4th lane drops off)
Canal Street	55 mph	30 mph	380'	470'
West Ave	55 mph	30 mph	380'	370'

Ramp Acceleration Lanes to Southbound I-95

Entrance From:	Mainline Design Speed	Ramp Design Speed	Required Acceleration Length	Actual Acceleration Ramp Length
Route 1	55 mph	30 mph	670'	485'
Atlantic Street	55 mph	30 mph	670'	1,140'
Washington Blvd	55 mph	30 mph	670'	1,010'
Harvard Ave	55 mph	30 mph	670'	1.024'

Ramp Deceleration Lanes from Northbound I-95

Exit To:	Mainline Design Speed	Ramp Design Speed	Required Deceleration Length	Actual Deceleration Ramp Length
Harvard Ave	55 mph	30 mph	380’	183’
Greenwich Ave	55 mph	30 mph	380’	453
Canal Street	55 mph	30 mph	380’	487’
Route 1	55 mph	30 mph	380’	205’

Ramp Acceleration Lanes to Northbound I-95

Entrance From:	Mainline Design Speed	Ramp Design Speed	Required Acceleration Length	Actual Acceleration Ramp Length
West Ave	55 mph	30 mph	670’	629’
Canal Street	55 mph	30 mph	670’	1088’
Elm Street	55 mph	30 mph	670’	752’

6.2.4 Review of Local Roadway (Service Road) Geometrics

Local roadway lane widths and link speeds are shown in Table 21 below.

Table 21: Local Roadway Speeds and Lane Widths

Road Name	Link Speed (mph)	Typ. Lane Width (ft)
East Main Street WB	30	10
East Main Street EB	30	10
Courtland Ave SB	30	11
Blachley Rd NB	30	11
Lockwood Ave NB	25	16
Lincoln Ave SB	25	16
Myrtle Ave NB	25	11
Lafayette St NB	25	12
Lafayette St SB	25	11
Elm St NB	25	10
Elm St SB	25	10
North State Street WB	25	12
South State St EB	25	11
Canal St SB	25	11
Canal St NB	25	10
Atlantic St SB	25	12
Atlantic St NB	25	12
Guernsey Ave Driveway SB	25	10
Guernsey Ave Underpass NB	25	12
Washington Blvd NB	25	11
Washington Blvd SB	25	11
Greenwich Ave SB	25	11
Greenwich Ave NB	25	12
Grenhart Rd WB	25	12
West Ave NB	25	10
West Ave SB	25	11
Harvard Ave NB	25	12
Harvard Ave SB	25	10

6.3 Roadway Structures Review

6.3.1 Methodology

The existing condition of structures within study area limits were assessed and documented utilizing the latest bridge inspection report data available from CTDOT.

6.3.2 General Description of Bridges

There are a total of twelve bridges within the study area limits that were assessed in this report. These bridges carry or cross I-95 mainline, ramps, local City Streets, the Rippowam River, and Metro-North Railroad.

General information about each bridge is presented in Table 22. For information on vertical clearances, see Table 18.

Table 22: I-95 Bridge Information

Bridge	Facility Carried	Feature Crossed	Structure Type	Bridge Length	Bridge Width	Year Built	Year Reconstr.	Date of Last Inspection
00026	Interstate 95	Rippowam River & Greenwich Ave	Steel Multi-Girder	435’	109.75’	1958	1990	2/8/2021
00027	Interstate 95	Washington Blvd (SSR 493)	Steel Multi-Girder	632’	127.75’	1958	1990	12/21/2020
00028	Interstate 95	Atlantic St	Steel Multi-Girder	89’	108.33’	1958	2000	5/3/2021
00029	Interstate 95	Canal St	Steel Multi-Girder	72’	117.58’	1958	2004	4/20/2021
00031	Interstate 95	Elm St	Steel Multi-Girder	77’	127.67’	1958	2005	5/4/2021
00032	Interstate 95	Metro-North Railroad and Local Roads	Steel Multi-Girder	1,065’	101.00’	1958	1993	10/19/2020
00033	Interstate 95	Maple Ave	Steel Multi-Girder	74’	131.08’	1958	1999	6/26/2020
00034	Interstate 95	Lockwood Ave	Steel Multi-Girder	79’	119.83’	1958	1991	5/4/2021
00035	Maher Rd	Interstate 95	Steel Multi-Girder	157’	52.67’	1958	1999	12/13/2021
00036	Blachley Rd	Interstate 95	Steel Multi-Girder	161’	52.67’	1958	2001	12/13/2021
02567	Interstate 95	Brook	Concrete Culvert	13’	0.00’	1958	-	8/10/2020
06584	Interstate 95 Ramp 350	Metro-North Railroad and Local Roads	Steel Multi-Girder	1,148’	29.50’	1999	-	11/6/2020

6.3.3 Existing Structural Conditions

Condition ratings established by the National Bridge Inspection Standards (NBIS) are used to describe an existing bridge compared with its conditions if in fact the bridge was new. As bridges are inspected, they are assigned conditions ratings. Ratings range from a high of 9 (excellent condition) to a low of 0 (failed condition). The rating scale is summarized in Table 23.

Table 23: I-95 NBIS Rating Scale

NBIS Code	Condition Description
9	**Excellent Condition**
8	**Very Good Condition** – no problems noted
7	**Good Condition** – some minor problems
6	**Satisfactory Condition** – structural elements show minor deterioration
5	**Fair Condition** – all primary structural elements are sound but may have minor corrosion, cracking or chipping. May include minor erosion on bridge piers
4	**Poor Condition** – advanced corrosion, deterioration, cracking or chipping. Also, significant erosion of concrete bridge piers
3	**Serious Condition** – corrosion, deterioration, cracking and chipping, or erosion of concrete bridge piers have seriously affected deck, superstructure, or substructure. Local failures possible.
2	**Critical Condition** – advanced deterioration of deck, superstructure, or substructure. May have cracks in steel or concrete, or erosion may have removed substructure support. It may be necessary to close the bridge until corrective action is taken.
1	**“Imminent” Failure Condition** – major deterioration or corrosion in deck superstructure, or substructure, or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service
0	**Failed Condition** – out of service and beyond corrective action.
N	**Not Applicable**

The NBIS rating scale is used in evaluating bridge components such as the deck, superstructure, substructure, and culvert/channel as well as in evaluating the bridge overall. Condition ratings for each bridge element along with the overall structural evaluation for each bridge within study area limits are presented in Table 24.

Table 24: I-95 Bridge Condition Ratings

Bridge	Deck	Superstructure	Substructure	Channel	Deck Geometry	Structural Evaluation
00026	6	6	6	N	6	6
00027	5	5	6	N	7	5
00028	6	7	6	N	9	6
00029	7	7	6	N	9	6
00031	7	6	6	N	9	6
00032*	4	4	4	N	4	4
00033	6	6	6	N	9	6
00034	6	6	6	N	9	6
00035	6	7	6	N	6	5
00036	6	7	6	N	4	6
02567	N	N	N	7	N	6
06584	6	7	6	N	8	6

**Note: Condition ratings for Bridge 00032 are compiled from the most recent available bridge inspection report (2020), and do not reflect rehabilitation work completed under Project No. 0135-0334 which was undertaken subsequent to the 2020 inspection.*

7 Preliminary Summary of Deficiencies

This section provides a preliminary summary of operational, safety, and infrastructure deficiencies in the study area, as previously identified in this memo.

7.1 Operational Deficiencies

7.1.1 I-95 Mainline

Table 25 and Table 26 show a summary of I-95 mainline segments that operate below an acceptable LOS (where delays surpass the LOS E or F thresholds) during either the southbound AM or northbound PM peak hour, respectively.

Table 25: Southbound AM Interstate Segment Simulated Below Acceptable LOS Results

Segment	Segment Type
I95 SB ML @ E Main Street	Mainline
I95 SB Exit 9 On-Ramp	Merge
I95 SB Exit 8 Off-Ramp	Mainline
I95 SB ML North of Elm St	Diverge
I95 SB Exit 7 Off-Ramp	Mainline
I95 SB ML @ Canal Street	Diverge
I95 SB Exit 7 On-Ramp (Atlantic St)	Mainline
I95 SB Exit 7 On-Ramp (Washington Blvd)	Merge
I95 SB ML @ Fairfield Ave	Mainline
I95 SB Exit 6 Off-Ramp	Merge
I95 SB ML @ West Ave	Mainline
I95 SB Exit 6 On-Ramp	Diverge
I95 SB ML South of Exit 6	Mainline

Table 26: Northbound PM Interstate Segment Simulated Below Acceptable LOS Results

Segment	Segment Type
I95 NB Exit 6 On-Ramp	Merge
I95 NB ML @ Fairfield Ave	Mainline
I95 NB ML @ Canal Street	Mainline
I95 NB Exit 8 On-Ramp (Canal St)	Merge
I95 NB ML @ Elm Street	Mainline
I95 NB Exit 8 On-Ramp (Elm St)	Merge
I95 NB ML @ E Main St	Mainline
I95 NB Exit 9 On-Ramp	Merge

7.1.2 Local Roadway Intersections

The following study area intersections operate below an acceptable LOS (where delays surpass the LOS E or F thresholds) during either the AM or PM peak hour:

1. Special Service Road (SSR) 493 (Washington Boulevard) at North State Street & I-95 SB Exit 7 On-Ramp (AM)
2. Canal Street at State Route 790 (South State Street) & I-95 Exit 7 On-Ramp (PM)
3. Elm Street at U.S. Route 1 (Tresser Boulevard / East Main Street) (AM and PM)
4. U.S. Route 1 (East Main Street) at Glenbrook Road & Clarks Hill Avenue (AM)
5. Greenwich Avenue at Pulaski Street & Davenport Street (All-Way Stop Control) (AM and PM)

7.2 Safety Deficiencies

7.2.1 I-95 Mainline

Table 27 shows the top 10 locations ranked by crash rate along the I-95 mainline during the study period.

Table 27: Top 10 Mainline Segments Ranked by Crash Rate

Direction	Description	Total Crashes	Fatal & Injury Crashes	Crash Rate (Crashes/100 MVM)	Ranking by Crash Rate
NB	180 ft south of Blachley Road to Exit 9 Off-Ramp	120	25	701	1
NB	Exit 8 Off-Ramp to Atlantic Street	83	14	666	2
NB	310 ft south of Fairfield Ave to Exit 7 Off-Ramp	70	18	400	3
NB	Greenwich Avenue to Exit 8 Off-Ramp	155	40	393	4
SB	310 ft south of Fairfield Ave to Exit 6 Off-Ramp	94	18	390	5
SB	Stamford Town Line to Exit 9 Off-Ramp	79	21	355	6
SB	Elm Street to Exit 7 Off-Ramp	47	28	354	7
NB	Stamford Town Line to Exit 6 Off-Ramp	29	8	327	8
NB	Exit 6 Off-Ramp to West Avenue	119	30	326	9
SB	Exit 8 Off-Ramp to Myrtle Avenue	34	8	320	10

7.2.2 Local Roadway Intersections

Table 28 shows the top 10 locations ranked by crash rate for local roadway intersections during the study period.

Table 28: Top 10 Intersections Ranked by Crash Rate

No.	Intersection Name	Primary Crash Type	Pedestrian & Bicycle Crashes	Injury Crashes	Total Crashes	Total Entering Vehicle	Crash Rate (Crashes/ MEV)	Ranking by Crash Rate
41	U.S. Route 1 (East Main St) at Glenbrook Rd	Sideswipe	5	17	123	28280	2.04	1
21	Atlantic St at U.S. Route 1 (Tresser Blvd)	Angle	4	23	112	27908	1.89	2
44	U.S. Route 1 (East Main St) at North State St	Angle	2	23	80	20932	1.80	3
5	U.S. Route 1 (West Main St) at West Ave	Sideswipe, Rear End	3	24	97	26539	1.72	4
42	U.S. Route 1 (East Main St) at Lafayette St	Angle	6	18	74	20452	1.70	5
24	Atlantic St at State Route 790 (South State St)	Angle	3	15	53	14905	1.67	6
45	U.S. Route 1 (East Main St) at Myrtle Ave	Angle	3	22	90	26889	1.57	7
23	Atlantic St at North State St	Angle	4	21	90	27128	1.56	8
7	Greenwich Ave at U.S. Route 1 (W Main St / Tresser Blvd)	Rear End	5	15*	79	23878	1.55	9
6	U.S Route 1 (West Main St) at Richmond Hill Ave	Angle, Sideswipe	2	11	50	15779	1.49	10

7.3 Infrastructure Deficiencies

Table 29 shows an overview of design elements on the I-95 mainline corridor that do not meet standard.

Table 29: I-95 Mainline Design Elements Not Meeting Standard

Controlling Criteria	Design Element	Unit	Standard	Existing
*	Shoulder Width Right (Approach road)	ft	10	Varies 4-10
*	Shoulder Width Left (Bridge)	ft	10	Varies 2-3
*	Shoulder Width Right (Bridge)	ft	10	Varies 2-3
*	Cross Slope (Travel Lane)	%	1.5-2.0	1.04
*	Stopping Sight Distance (Horizontal)	ft		
*	NB		495	396
*	SB		495	424
*	Ramp			349
	Min Length of Horizontal curve	ft	1,650	1,073
	Median Width (Includes Left Shoulder)	ft	30" Median + 6'-12' Shoulder on each side	36" Median + 2'-6" Shoulder on each side

Note: Red text denotes design element does not meet standard.

Table 30 summarizes bridges that have elements with one or more conditions rated 4 – Poor Condition or less on the NBIS rating scale.

Table 30: I-95 Bridge Condition Ratings Below Acceptable

Bridge	Deck	Superstructure	Substructure	Channel	Deck Geometry	Structural Evaluation
00032	4	4	4	N	4	4
00036	6	7	6	N	4	6

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