4.1 TRANSPORTATION

4.1.1 Introduction

As discussed in Chapter 2, *Project Purpose and Need*, the South Coast Rail alternatives seek to improve public transit service between the South Coast region and Boston, Massachusetts. This improvement would contribute towards meeting the existing and future demand for public transportation between Fall River/New Bedford and Boston and enhance regional mobility. In addition, the South Coast Rail alternatives were developed by MassDOT to be supportive of MassDOT's objective to foster smart growth planning and development strategies in the affected communities.

The transportation chapter provides a regional overview of the transportation conditions in the South Coast region. In addition, this chapter discusses transportation conditions in the vicinity of the proposed alternative corridors and proposed station locations. Grade crossings along the rail corridors associated with the alternatives are analyzed, as well as stations within the alternatives' study corridors.

Section 4.1.2 of this chapter provides an overview of the methodology for analyzing transportation conditions. Section 4.1.3 describes the existing conditions and establishes a basis for projecting future conditions without and with the alternatives (No-Build and Build Alternatives). Direct, indirect, and cumulative effects of the proposed alternatives are analyzed in Section 4.1.4 with respect to ridership demand, quality of service, vehicle miles of travel, regional mobility, traffic operations, grade crossings, and intersection and roadway traffic operations, pedestrian and bicycle accommodations, parking, and public bus transportation at each planned station within the study corridors.

4.1.2 Methodology

Given the transportation focus of the project purpose, the transportation analyses in this chapter, in addition to assessing impacts, also inform the evaluation of the alternatives in meeting the project purpose: "*to more fully meet the existing the future demand for public transportation between Fall River/New Bedford and Boston, Massachusetts to enhance regional mobility*." In addition to analyzing the overall regional transportation conditions, safety and capacity analyses were performed for the regional roadway network, grade crossings for the potential rail corridors were analyzed, and station analyses were performed for each new proposed commuter station. The alternatives station analyses include capacity and safety analyses for the intersections near the proposed stations, traffic signal warrant analyses, and assessments for pedestrians and bicycles, parking, and public transportation. The methodology used for the transportation analyses conforms to the *Guidelines for EIS/EIR Traffic Impact Assessment*[1] and the *2000 Highway Capacity Manual.*[2]

4.1.2.1 Regional Transportation Analysis Methodology

The regional transportation network (both roadways and transit) was evaluated for both existing and future conditions with and without the South Coast Rail alternatives. Future regional transportation conditions were analyzed using four key criteria, which were applied to all alternatives, to assess their performance and impacts on the regional transportation system: ridership, quality of service, vehicle miles traveled (VMT), and regional mobility. This assessment was conducted in a manner compatible

[1] Executive Office of Energy & Environmental Affairs and Executive Office of Transportation and Construction, Guidelines for EIS/EIR Traffic Impact Assessment, July, 1989.

[2] 2000 Highway Capacity Manual, Transportation Research Board, National Research Council, Washington D.C., 2000.

with previous assessment methodologies used during the alternatives analysis process described in Chapter 3, *Alternatives*.

4.1.2.2 Capacity Analysis

The assessment of traffic operations evaluates the operational qualities of the key intersections and roadway sections using the procedures documented in the *2000 Highway Capacity Manual.*[3]

Level of service (LOS) is used to denote the different operating conditions that occur on a roadway segment or at an intersection under various traffic volume loads. It is a qualitative measure of the effect of a number of factors including roadway geometry, speed, travel delay, and freedom to maneuver. LOS provides an index to the operational qualities of a roadway segment or an intersection. LOS designations range from A to F, with LOS A representing the best operating conditions and LOS F representing the worst operating conditions.

LOS designations are reported differently for freeway sections, and signalized and unsignalized intersections. LOS for freeway sections is determined based on speed density and flow rates. For signalized intersections, the analysis considers the operation of each lane or lane group entering the intersection and the LOS designation is for overall conditions at the intersection. For unsignalized intersections, however, the analysis assumes that traffic on the mainline is not affected by traffic on the side streets. The LOS is only determined for left turns from the main street and all movements from the minor street. The overall LOS designation is for the most critical (i.e., worst) minor movement, which is many times the left–turn movement from the side street.

Freeways/Highways

The study methods outlined in Chapter 23 (Basic Freeway Segments) of the *Highway Capacity Manual*[4] (HCM) were used for the LOS analysis of the various freeway and highway segments within the South Coast Rail project study area.

LOS represents reasonable ranges in the three critical flow variables: speed, density of vehicles in the traffic stream, and the flow rate of the vehicles. Basically, as the density of vehicles increases, vehicle speed tends to decrease and the flow rate decreases correspondingly. A freeway can process approximately 2,400 passenger vehicles per lane per hour under optimal conditions (12-foot travel lanes, two-foot median lateral clearance, 6-foot right lane lateral clearance, level terrain, no heavy vehicles, and a driver population consisting of mostly regular users) in rural areas. The freeway capacity drops to about 2,300 passenger vehicles per lane per hour in urban areas. These volumes would result in LOS E operations, the point at which a highway is considered to be operating at capacity. Table 4.1-1 presents these criteria.

[3] 2000 Highway Capacity Manual, Transportation Research Board, National Research Council, Washington D.C., 2000.
[4] 2000 Highway Capacity Manual, Transportation Research Board, National Research Council, Washington D.C., 2000.

Table 4.1-1 Level of Service Criteria–Freeway Sections

Level of Service	Traffic Conditions	Description of Operations
LOS A (best LOS)	Free Flow	Vehicles almost completely unimpeded in their ability to maneuver within the traffic stream.
LOS B	Reasonable Free Flow	The ability to maneuver within the traffic stream is only slightly restricted.
LOS C	Stable Flow	Freedom to maneuver within the traffic stream is noticeably restricted.
LOS D	Approaching Unstable Flow	Freedom to maneuver within the traffic stream is more noticeably limited.
LOS E	Unstable Flow	Operations at capacity. No usable gaps in traffic stream.
LOS F (worst LOS)	Forced or Breakdown Flow	Queues form behind breakdown point and volume-to-capacity ratio exceeds 1.0.

Note: Description based on Association of American State Highway and Transportation Officials and HCM standards.

Once the capacity of a highway is determined, the density can be calculated and the LOS can be determined. The HCM does not recommend a specific LOS for design purposes, but does present a description of the conditions associated with each LOS. The manual describes LOS C as providing for flow with speeds at or near free flow speed; freedom to maneuver within the traffic stream is noticeably restricted; lane changes require additional care and vigilance; and queues may begin to form behind any substantial blockage.

As conditions deteriorate to LOS D, the HCM describes conditions as unstable flow; freedom to maneuver within the traffic stream is more noticeably limited; and a driver experience of reduced physical and psychological comfort levels. The HCM does indicate that the higher the design LOS, the more the highway facility can absorb additional atypical amounts of traffic and still function at a satisfactory level.

Signalized Intersections

Capacity at a signalized intersection is defined for lane groups rather than for approaches or the intersection as a whole. A lane group may be a single movement, a group of movements, or an entire approach, and is defined by the geometry of the intersection and the distribution of movements over the various lanes. Capacity of a lane group is calculated as the maximum rate of flow that may pass through the intersection under prevailing traffic, roadway, and signalization conditions. The rate of flow is generally measured or projected for a 15-minute period and capacity is stated in vehicles per hour. Capacity analysis of signalized intersections involves computing volume–to–capacity (v/c) ratios for each lane group, from which an overall intersection v/c ratio may be derived.

Generally, when two opposing flows are moving during the same signal phase, one of the lane groups will require more green time than the other to process all of its volume. This lane group is defined as the "critical" lane group for the subject signal phase. The concept of a critical v/c ratio is used to evaluate the intersection as a whole, considering only the critical lane groups or those with the greatest demand for green time. Thus, if the green time has not been appropriately allocated to the various approaches, it is possible to have an overall intersection v/c of less than 1.00 (under capacity) but still have individual movements saturated within the signal cycle.

The other major concept in signalized intersection analysis is LOS, which is an index used to grade intersection operations. LOS is defined in terms of delay and ranges from LOS A (free flow conditions) to

LOS F (long delays). Delay represents a measure of driver discomfort, frustration, fuel consumption, and lost time. Specifically, LOS delay criteria are stated in terms of control delay per vehicle during a peak 15–minute period. These criteria are listed in Table 4.1-2.

Table 4.1-2 Level of Service Criteria for Signalized Intersections

Level of Service	Control Delay per Vehicle (sec)[1]
A	<10.0
B	10.1 to 20.0
C	20.1 to 35.0
D	35.1 to 55.0
E	55.1 to 80.0
F	>80.0

Source: HCM, Special Report 209; Transportation Research Board, Washington, DC, 2000.

1 Average control delay per vehicle for a peak 15–minute period.

Delay is a complex measure that depends upon a number of variables such as quality of signal progression, cycle length, allocation of green time, and v/c ratio. Of all the factors cited, v/c ratios have the least effect on delay. Thus, for any given v/c ratio, a range of delay values (and, therefore, LOS) may result. Conversely, for a given LOS, the v/c ratio may lie anywhere within a broad range. The base saturation flow rate used in the signalized intersection analysis model is 1,900 passenger cars per hour of green time per lane. This value is adjusted for prevailing traffic conditions such as lane width, left turns, right turns, heavy vehicles, grades, parking, area type, bus blockage, and left–turn blockage.

Unsignalized Intersections

LOS for unsignalized intersections is based on the assumption that major street traffic is not affected by minor street movements (i.e.; minor street traffic must wait for a gap in major street traffic). The capacity of the intersection to accommodate minor street movements is based on the amount of traffic on the major street and the configuration of the intersection. LOS is based on the average control delay, which is the total elapsed time from the time a vehicle stops at the end of the queue to the time the vehicle departs from the stop line. The average control delay for any particular minor movement is a function of the service rate or capacity of the approach and the degree of saturation. The overall LOS designation is for the most critical (i.e., worst) minor movement, which is often the left–turn movement from the side street. Table 4.1-3 presents these criteria.

Table 4.1-3 Level of Service Criteria for Unsignalized Intersections

Level of service	Control Delay per Vehicle (sec)[1]
A	<10.0
B	10.1 to 15.0
C	15.1 to 25.0
D	25.1 to 35.0
E	35.1 to 50.0
F	>50.0

Source: HCM, Special Report 209; Transportation Research Board, Washington, DC, 2000.
1 Average control delay per vehicle for a peak 15–minute period.

4.1.2.3 Analysis Approach

The regional highway network is expected to be affected by the No-Build Alternative. The transportation capacity analyses (for the regional network and the proposed stations) are directly related to the projected ridership of the alternatives; therefore, to present the most conservative analysis, the following approach was used to determine the transportation benefits and impacts of the alternatives:

- To conservatively determine the *benefit* of the alternatives on the regional highway network, the Build Alternative with the lowest projected ridership analyzed in the DEIS (Rapid Bus Alternative[5]) was used since it would shift the fewest automobile users from a highway to a transit trip. While the Rapid Bus Alternative has been eliminated in the FEIS, it remains the most appropriate Build Alternative for analyzing the impacts on the regional highway network due to its low projected ridership, and maintains a conservative approach consistent with the DEIS. Although not specifically analyzed, the Stoughton and Whittenton Alternatives (both electric and diesel variants) would result in proportionally greater benefits to the regional highway system.

- Conversely, the Build Alternative with the highest projected ridership at each station was used to evaluate the impacts in the areas around proposed station locations. For station locations shared between alternatives, separate intersection analyses were not conducted for each alternative, because the lower projected ridership for these alternatives would result in equal or less impact than the analysis using the highest ridership.

To maintain a conservative approach consistent with the DEIS/DEIR, a specific Build Alternative was used for the analysis of each transportation study area. In some cases, Build Alternatives that have been eliminated from further consideration in the FEIS/FEIR were used as the basis for the transportation impact assessment. This approach is reasonable because the alternatives used in the analysis remain the most conservative in terms of estimating regional traffic benefits (alternative with lowest ridership) and station area traffic impacts (alternative with highest ridership). The following identifies the Build Alternative used for analysis of the various transportation study areas:

[5] As discussed in Chapter 3, the Rapid Bus Alternative evaluated in the DEIS/DEIR was eliminated from further consideration. However, the regional highway benefits assessment based on the Rapid Bus Alternative is retained in the FEIS/FEIR because it provides a conservative assessment of the regional highway benefits of the Stoughton and Whittenton Alternatives.

- The electric rail alternatives were analyzed because projected ridership is equal to or higher than projected ridership on the corresponding diesel alternative.
- Regional highway network (sections of Route 140, Route 24 and I-93)—Rapid Bus Alternative. The Rapid Bus Alternative has been eliminated from further consideration, but provides a conservative basis for evaluating the regional traffic benefits of the Stoughton and Whittenton Alternatives.
- King's Highway Station—Attleboro Electric. The Attleboro Electric Alternative has been eliminated from further consideration, but provides a conservative basis for evaluating station area traffic impacts because it had the highest ridership projection for this station of all the Build Alternatives.
- Whale's Tooth Station—Attleboro Electric. The Attleboro Electric Alternative has been eliminated from further consideration, but provides a conservative basis for evaluating station area traffic impacts because it had the highest ridership projection for this station of all the Build Alternatives.
- Freetown Station—Stoughton Electric
- Battleship Cove—Attleboro Electric. The Attleboro Electric Alternative has been eliminated from further consideration, but provides a conservative basis for evaluating station area traffic impacts because it had the highest ridership projection for this station of all the Build Alternatives.
- Fall River Depot—Attleboro Electric. The Attleboro Electric Alternative has been eliminated from further consideration, but provides a conservative basis for evaluating station area traffic impacts because it had the highest ridership projection for this station of all the Build Alternatives.
- Taunton Depot—Stoughton Electric
- Easton Village—Stoughton Electric
- North Easton—Stoughton Electric
- Taunton Station (Stoughton Alternatives Only)—Stoughton Electric
- Raynham Park Station—Whittenton Electric
- Stoughton Station (relocation)—Stoughton Electric
- Dana St. Station (Whittenton Alternatives only)—Whittenton Electric

Since there is only one set of transportation analyses for each station (worst case scenario), the results of the analyses are presented by community.

The methodology used in this chapter is standard transportation planning industry practice for the evaluation of transportation systems and infrastructure. Much of the evaluation was based on a 2030 traffic forecast with and without the Build Alternatives provided by the Central Transportation Planning Staff (CTPS) for the DEIS/DEIR. Certain key indicators such as ridership and VMT have been updated in the FEIS/FEIR for a 2035 traffic forecast. As discussed further below, the 2035 ridership analysis update results were also used to review and update the station-level traffic impact assessment where appropriate.

4.1.2.4 Traffic Growth Forecast

CTPS is the staff for the metropolitan planning organization (MPO) for the Boston region and works with the communities within the region to address issues such as transportation, land use, and economic development. The Boston Region Metropolitan Planning Organization (MPO) is responsible for conducting the federally required metropolitan transportation-planning process, and allocating federal and state transportation funds to programs and projects in the Boston metropolitan area. The MPO and CTPS function independently of MassDOT, and their activities are periodically reviewed by both the Federal Highway Administration and Federal Transit Administration.[6] MassDOT provided funding to CTPS to conduct the transportation modeling analyses for the South Coast Rail project.

The CTPS regional travel demand model was used to provide the traffic forecasts for the entire study area. This model is run using Emme software. CTPS's method of travel demand forecasting follows the traditional four steps of trip generation, trip distribution, modal split, and travel assignment. The model uses changes in population, number of households, employed residents, number of automobiles, and total employment to forecast changes in traffic over time.

Using the future No-Build model output, No-Build weekday morning and evening peak hour turning movement volume networks were created. For each municipality, a background growth rate was established based on model outputs. Table 4.1-4 shows the background traffic growth rate used in each community. These growth rates were applied to the existing traffic volumes to develop 2030 No-Build volumes. Traffic increases from specific development projects that were not included in the model were also added to the network to develop the final No-Build networks for local intersections.

Table 4.1-4 Background Traffic Growth Rate (by Community)

Community	Growth Rate[1]
New Bedford	4.1 %
Fall River	7.1 %
Freetown	18.4 %
Taunton	4.7 %
Norton	9.4 %
Raynham	8.1 %
Easton	6.9 %
Stoughton	5.0%

1 Total (aggregate) growth rate used to convert 2008 conditions to 2030 conditions
Source: CTPS Travel Demand Model.

A similar process was used to project 2030 No-Build traffic volumes on Route 24. A background growth rate was developed for each direction in each peak hour for each segment. As with local intersections, traffic from specific developments not included in the traffic model were added. Appendix 4.1–A

6 Boston Region Metropolitan Planning Organization. About the MPO. http://www.ctps.org/Drupal/mpo

provides specific information regarding the overall growth on each segment by direction and time of day.

No-Build Analysis

In order to evaluate access for the bus park-and-ride locations under future No-Build conditions with enhanced bus service, intersection capacity analyses were performed at park-and-ride driveway locations using 2030 projected traffic volumes. Traffic volumes for the 2030 design year were projected based on additional vehicle trips associated with the increased bus ridership projections provided by CTPS.

The resulting peak hour volumes were analyzed to evaluate how well the future infrastructure will accommodate the demands placed on it during the morning and evening peak hours. The analysis produces a LOS rating for each facility. The criteria for determining LOS at signalized and unsignalized intersections and on freeway sections is described above.

Station Area Analysis Methodology

Traffic Demand

Traffic demand estimated for the alternatives are based on the 2030 and 2035 ridership forecasts developed by the CTPS (see Appendix 3.2-H (2035) and Appendix 4.1-H (2030)). CTPS developed these forecasts based on a number of variables, such as observed commuter rail ridership in similar areas, magnitude of service to be provided, and future estimates of population and employment within the South Coast region and greater Boston area. All of these data were analyzed via a regional travel demand model, which ultimately provided a future ridership estimate for the proposed service. The basis for the model is documented in Appendix 3.2-G.

For the DEIS/DEIR, CTPS conducted 2030 Build model runs for each alternative by including the new bus or rail service as a travel option. The model was used to quantify the number of vehicle trips diverted from regional roadways to local roadways because of drivers and riders who change mode from passenger car to transit service. Trip generation for each station was based on projected park-and-ride (i.e., driving and parking at the station) and drop-off (i.e., being dropped off or picked up by another driver) ridership. The analyses of impacts on traffic operations are based on the peak hour park-and-ride and drop-off ridership projections for each station. The park-and-ride ridership was divided by a vehicle occupancy rate (VOR) of 1.05 to calculate the number of park-and-ride vehicles entering and exiting the stations. Two vehicle trips were assumed for each drop-off rider: one entering and one exiting the proposed station. The same basic methodology was used for the 2035 ridership forecasts (see the CTPS memorandum provided in Appendix 3.2-H).

Using the Build model outputs, peak hour turning movement volume networks were developed for each Build Alternative. The rail related trips were distributed as new traffic and assigned to the roadway network based on the distribution of trips from the travel demand model. To present a conservative analysis condition, no adjustments were made to the traffic volumes to account for diverted trips within the local street network. The peak hour volumes were then used to conduct LOS assessments for the Build Alternatives. When compared to the No-Build Alternative, the LOS assessment for the Build Alternatives will show the effect of the proposed action on transportation conditions.

Where impacts could not be avoided or minimized, mitigation was proposed and evaluated for effectiveness. Mitigation was proposed for intersections where LOS E/F conditions result because of the Build Alternatives and where LOS E/F conditions under the No-Build Alternative are notably worsened with the Build Alternatives (generally an increase in control delay of more than 10 seconds).

Safety Analysis

In order to identify crash trends, historical crash data were obtained from MassDOT Highway Division for each community for the most recent three-year period available at the time of the analysis. For each proposed station site, vehicle crashes were compiled by roadway and key intersection. Specific crash characteristics include year of crash, crash type, severity, weather, and time of day.

Crash rates are calculated based on the number of crashes at an intersection (i.e., crash frequency) and the volume of traffic traveling through the intersection (i.e., vehicle exposure) on an annual daily basis[7]. Rates that exceed the MassDOT Highway Division district or statewide average (i.e., arithmetic mean) could indicate safety or geometric issues at an intersection. The South Coast communities are location in District 5 of the MassDOT Highway Division. The District 5 average crash rate for unsignalized intersections is 0.59 crashes per million entering miles and the rate for signalized intersections is 0.84 crashes per million entering miles. The statewide crash rate is 0.66 for unsignalized intersections and 0.87 for signalized intersections.

Documentation of the crash data and crash rates is provided in Appendix 4.1- B.

Grade Crossings

An inventory of highway-railroad at-grade crossings was performed in November and December of 2008 to identify and document existing active (with freight activity) and inactive grade crossings along the following rail corridors.

- New Bedford Main Line—Cotley Junction to State Pier in New Bedford
- Fall River Secondary—Myricks Junction to Battleship Cove in Fall River
- Attleboro Secondary—portion in Taunton utilized by Whittenton Alternatives
- Stoughton Line—Canton Junction to Cotley Junction
- Whittenton Branch—Stoughton Line to Attleboro Secondary

The active rail crossings located along the Northeast Corridor were not included in the inventory. Those crossings are part of the current operating railroad and would not be altered under this project.

The existing conditions of each crossing were evaluated to determine the crossing geometry, sight distances, and roadway traffic patterns. Each rail and roadway approach was photographed and sketches were prepared to illustrate the warning systems in place and other physical features that will have to be considered during the layout and design of the proposed grade crossing.

[7] Statewide average crash rates reflect the average of crash rates contained in a database of signalized and unsignalized intersection crash rates compiled by MassDOT Highway Division, calculated for both signalized and unsignalized locations. MassDOT Crash Rate Information. 2012. http://www.mhd.state.ma.us/default.asp?pgid=content/traffic/crashRateInfo&sid=about.

Grade Crossing Incident Prediction Analysis Methodology—A highway/rail incident, as defined by the Federal Railroad Administration (FRA), is any impact between a rail and highway user at a crossing site, regardless of severity. This includes motor vehicles and other highway / roadway/ sidewalk users at both public and private crossings. From 2002 to 2011 for the 333 active at-grade crossings the MBTA operates, an average of three incidents occurred per year (0.99 incidents per million train miles). In comparison, the national average is 72 incidents per year or 1.57 incidents per million train miles.

In order to establish what may be the incident rate for future conditions, the FRA's Office of Safety Analysis has developed a Web Accident Prediction System (WBAPS), which is used to calculate the probability that an incident will occur in any given year. This system generates incident reports for public highway/rail intersections for a state, county, city, or railroad and ranks them by predicted collisions per year. A train incident is defined by the FRA as an event involving on-track rail equipment that results in monetary damage to the equipment and track above a certain threshold. Incident predictions are based on a current inventory of at-grade crossings and collisions from 2002 to 2011. Using the WBAPS, incident predictions were calculated for each town along the South Coast Rail project and compared to similar rates estimated for the entire MBTA system.

Gate Closure

The impact of the grade crossings on traffic operations requires the calculation of the amount of time the roadway would be blocked. In accordance with standard practice, it is assumed that the gate system would close 30 seconds prior to the train's arrival at the grade crossing and for 15 seconds after the train clears the crossing. This time is estimated by dividing the approximate length of the train by the approximate speed of the slowest train expected at that crossing. In most cases where the rail crossing is perpendicular to the roadway, the sum of these components yields the total time (60 seconds) that the roadway is blocked. A 70 second gate delay time was used for unusually wide or skewed crossings.

For crossings that are located within 500 feet of a station platform, the gates would operate differently depending on the direction of travel. The delay for a train passing through the crossing before stopping at the station would be 60 seconds, as defined above. However, as a safety measure, the gates must also be activated as a train pulls into a station prior to reaching the crossing. The train then stops at the station to drop off or pick up passengers and then continues through the crossing. The timing for this situation was determined based on:

- As the approaching train is detected, the gates would close.
- When the train stops at the platform the gates would open.
- The gates close again as the train leaves the station (it is estimated that approximately four cars would be able to clear the crossing while the train dwells in the station).
- After the train passes through the crossing the gates reopen a final time.

The total gate delay time is estimated to be 150 seconds. Since this time also includes station dwell time, the projected delays and queues were reduced to reflect the estimated four cars that would clear the crossing during the station dwell time.

Determination of Vehicle Volumes

Where available, existing traffic volume data at grade crossing locations were obtained from the MassDOT Highway Division. These data were supplemented by counts collected as part of the traffic analysis for the proposed project. The 2030 morning and evening peak hour traffic volumes were developed for each grade crossing by applying the annual growth rates obtained from the CTPS regional transportation demand model.

Traffic Queue and Delay Calculation

The peak direction traffic volumes were converted to an average arrival rate by dividing the hourly volume by the number of seconds in an hour (3,600). By applying the arrival rate to the total time that the roadway was blocked, an average queue estimate was developed. Assuming a random arrival of vehicles at the crossing, the average delay per stopped vehicle was estimated based on gate closure time plus the startup time for the vehicles in the queue. An average start up time of two seconds was used, representing a four second start up time for vehicles in the beginning of the queue and zero seconds toward the back of the queue. The average delay is therefore equal to one-half of the time that the roadway is blocked plus two seconds per vehicle for one-half of the average queue.

Determination of Impact

After the average queue was calculated, impacts of the queue on nearby intersections were determined. A value in the range of 20 to 25 feet per vehicle is generally used to estimate the length of queues. This length includes the length of the vehicle and the spacing between queued vehicles. For this analysis, the total number of vehicles was multiplied by 25 feet per vehicle to determine the total average length (in feet) of the queue.

Inactive or Abandoned Railroad Rights of Way

In locations where reactivation of inactive or abandoned railroad rights-of-way are proposed, the analysis provided includes more detail with respect to traffic flows and average delays. This is necessary to determine the projected impacts of gate closures due to the absence of physical gate closure data.

4.1.2.5 Stations

As shown in Figure 1.4-1, the commuter rail alternatives include potential commuter rail stations within New Bedford, Freetown, Fall River, Taunton, Easton, Stoughton, and Raynham. Intersections within the seven communities were selected for safety and traffic operation analyses based on the proposed locations of the new or relocated commuter rail stations.

Since boardings at existing commuter rail stations located near the end of the existing Stoughton Commuter Rail Line are not expected to increase as a consequence of the alternatives, no traffic analyses, beyond the identification of new grade crossing locations, were completed for existing stations or municipalities with existing stations.

Roadway and Intersection Inventory

A comprehensive field inventory of major roadways and key intersections was completed for each commuter rail station study area. Field reconnaissance included an inventory of roadway geometry, observed vehicle speeds, signalization (where applicable), other traffic control, and nearby land uses. Documentation of the intersection inventory field work is provided in Appendix 4.1-C. Detailed roadway and intersection descriptions are provided in Appendix 4.1-D.

Traffic Volume Data Collection

Traffic volume data were collected in September and October 2008 for roadways and critical intersections serving each of the proposed rail stations. This data included automatic traffic recorder (ATR) counts and manual turning movement counts (TMCs). ATRs were collected along major roadways to provide an understanding of daily and peak hour traffic flows in the vicinity of each potential commuter rail station site. Two–hour TMCs were conducted at key intersections during the weekday morning and evening commuter peak periods. Vehicles, bicycles, and pedestrians were counted. All TMCs were conducted midweek (Tuesday through Thursday) to capture traffic count data that depict typical weekday peak conditions. The TMCs were balanced, and rounded to form the traffic volume networks used to evaluate existing traffic operations. To determine whether or not it was necessary to seasonally adjust the recorded traffic volumes, historical traffic count data from the following MassDOT Highway Division permanent count stations were reviewed:

- Randolph, Route 24 south of I-93
- Raynham, I–495 north of Route 24
- Raynham, I–495 south of Route 24
- Freetown, Route 140 at the New Bedford city line
- Taunton, Route 24 north of Route 140
- Fall River, I-195 west of Route 24

Based on observed data from these locations, traffic volumes for September and October are generally 1 to 8 percent higher than the yearly average. Consequently, the actual traffic counts were not adjusted to reflect any seasonal difference in traffic volumes; and therefore represent a slightly higher than average condition.

Documentation of the traffic volume data collection is provided in Appendix 4.1-E.

Traffic Signal Warrant Analysis

The Manual of Uniform Traffic Control Devices (MUTCD) defines a traffic signal warrant analysis as an engineering study of traffic conditions, pedestrian characteristics, and physical characteristics of an intersection performed to determine whether installation of a traffic control signal is justified at a particular location. The study includes an analysis of factors related to the existing operation and safety at the intersection in question, the potential to improve these conditions, and standard criteria which could necessitate the installation of a traffic signal, known as "warrants." The satisfaction of a traffic signal warrant or warrants shall not in itself require the installation of a traffic control signal.[8]

Peak hour signal warrant analyses were conducted at study area intersections in conformance with the MUTCD[9] standards. For the purposes of this analysis, peak hour traffic signal warrants were evaluated for unsignalized intersections that exhibit poor traffic operations and would decline further as a result of the proposed project. If an unsignalized intersection does not meet the peak hour traffic signal warrant

[8] Chapter 4C. Traffic Control Signal Needs Studies. Manual of Uniform Traffic Control Devices (MUTCD) Federal Highway Administration Washington, DC 2003.

[9] Manual of Uniform Traffic Control Devices (MUTCD) Federal Highway Administration Washington, DC 2003.

based on projected 2030 traffic volumes, no additional analysis would be necessary. All site driveway locations were also evaluated for traffic signal installation.

Locations meeting traffic signal warrants under the peak condition would be evaluated for four and eight-hour traffic signal warrants as part of the preliminary design process. Meeting a traffic signal warrant indicates that a traffic signal could be placed at a particular location; however, satisfaction of a traffic signal warrant does not in itself require a traffic signal be installed. Locations where traffic signal installation is considered an appropriate mitigation measure are discussed later in this section. Documentation of the preliminary traffic signal warrant analysis is provided in Appendix 4.1-F.

Pedestrians and Bicycles

The travel demand model was also used to project total pedestrian and bicycle volume at each planned station for the Build Alternatives. For each transportation analysis zone (TAZ) within the regional model, CTPS provided the number of pedestrians and bicyclists using transit and the specific station they would access. The pathways of travel between zones and each station were mapped and pedestrians and bicyclists were assigned to routes accordingly. Bicycle accommodations were evaluated qualitatively for the Build Alternatives with respect to their ability to serve projected users and any projected impacts from project related traffic and planned or proposed roadway improvements. Pedestrian/bicycle volume networks for all alternatives can be found in Appendix 4.1-G.

Parking

The parking assessment for stations associated with the alternatives compares the planned number of parking spaces to the projected peak parking demand and identifies any existing parking supply that may be affected by the proposed project. Peak parking demand at each station was projected based on the daily passenger boardings determined by the CTPS travel demand model. For the purposes of this analysis the peak parking demand is equal to the number of passengers who would drive and park at the station prior to boarding the train. No reduction in parking demand was taken in order to account for carpooling. Locations where projected demand for parking exceeded the planned parking supply were identified. There were no parking demand analyses of the Battleship Cove and Easton Village stations because no parking is planned for either location.

The existing parking supply in the vicinity of each proposed station location was qualitatively evaluated in order to determine whether any existing parking is vulnerable to impacts due to the proposed project. Areas that have potential vulnerability have been identified and steps to mitigate impacts noted if applicable.

Public Bus Transportation

Existing bus services near the planned stations were reviewed to determine if route or service adjustments could be made to provide good connections between local transit services and commuter rail service. Using the CTPS travel demand model, potential bus route adjustments to provide direct service to planned stations were evaluated. Limited bus transit activity is anticipated at most stations. More substantial bus activity is projected at the Whale’s Tooth station due to proximity to regional bus transit hubs. Trip generation characteristics for this station are provided in Appendix 3.2-H.

4.1.3 Existing Conditions

This section presents the Affected Environment (Existing Transportation Conditions) for the South Coast Rail project. An overview of the South Coast region, including ridership demand, quality of service, vehicle miles of travel, and regional mobility is presented. In addition, existing traffic operations were analyzed for the highways and intersections within the South Coast region, existing grade crossings for the proposed rail corridors are identified, and proposed stations are analyzed. The existing station analyses include an inventory of roadways and intersections, existing traffic volumes, crash analysis summary, and traffic operations analysis.

4.1.3.1 Regional Overview

Quality of Service

The existing transportation system serving the South Coast region has inadequate capacity, leading to lack of regional mobility, between the South Coast region and Downtown Boston and within the South Coast region itself. This is due in part to the relative lack of public transit connections between New Bedford/Fall River and Boston and between South Coast cities (New Bedford, Fall River, Taunton and others).

In this regard the South Coast region is severely underserved relative to other regions. This is partially due to the absence of commuter rail, which in other regions provides intra (within) regional connectivity (mobility), partially as a byproduct of interregional connectivity with Boston.

The inadequacy of public transit service in the South Coast region is reflected in several aspects. The availability of public transit service in absolute terms and compared to other regions (especially those that have a large commuting segment to downtown Boston) is limited, and the quality of transit service as expressed in travel time and frequency of service is poor, especially during the peak hours. The geographic availability of transit service to people in the region is also relevant in terms of access to employment opportunities and services, including education and healthcare. In addition to transit services between the South Coast region and Boston, transit services within the South Coast region are also relevant. An indicator of quality of transit service is the MBTA's Service Delivery Policy.[10] This policy identifies minimum frequency of service levels that provides the guidelines by which the MBTA maintains accessibility to the transportation network within a reasonable waiting period. The minimum frequency of service standards is the minimum frequency that must be maintained in a service. For commuter rail and commuter bus minimum frequencies should provide three trips in a peak direction during the AM and PM peak periods.

Existing transportation in the South Coast region is predominantly auto-oriented and transit services within the South Coast region are limited to bus and demand-response services operated by regional transit authorities and private carriers. Most of the commuter trips from the South Coast region to the Boston market are in single occupant vehicles. Public transit accounts for a minor proportion of work trips in the service area. To a large extent, this can be attributed to the lack of public transit alternatives other than privately-operated bus service. As discussed below, many communities in the South Coast region lack public transit facilities other than private bus services and major population centers are as

[10] MBTA's Transit Service Policy is similar to other service delivery policies and standards from regional transit agencies, such as Los Angeles County MTA, Detroit DOT, Washington, D.C. MTA, Chicago Transit Authority, and others.

much as 25 miles from existing commuter rail stations. All commuter rail stations are located outside the South Coast region and are approaching capacity.

Bus Service

Local bus public transit within the South Coast region is provided in Taunton by Greater Attleboro Taunton Regional Transit Authority (GATRA) and in New Bedford and Fall River by Southeastern Regional Transportation Authority (SRTA). GATRA also operates intercity bus service between Taunton and Providence, Rhode Island.

Bus service to Boston from the South Coast region including the cities of New Bedford, Fall River, and Taunton is limited to private carriers (Figure 2.2-1). Private carriers also connect New Bedford, Fall River, and Taunton with each other and with Providence, Newport and points beyond. Bus service from the South Coast region to Boston uses the regional roadway system and is therefore subject to the same congestion and safety problems on the highway system as other vehicles, resulting in long and unpredictable travel times. The existing bus service between the South Coast region and Boston fails the MBTA's Service Delivery Policy. The bus service is also substantially more expensive than MBTA commuter rail services over similar distances, creating an additional constraint on usage of bus service, especially for lower income groups. Some bus service exists to commuter rail stations outside the South Coast region; however the transfer between two transit services increases overall travel cost and overall travel time, rendering it less attractive.

In addition, existing express bus services within the South Coast region are limited to a few stops in order to realize a total travel time competitive with automobiles. Serving additional communities with these bus services would substantially slow service to unacceptable levels, which would result in fewer riders. The second constraint that limits intraregional connections is bus capacity. In order to attract riders, existing bus services seek to minimize headway (maximize frequency) while operating at or near capacity almost from their initial point of departure, with very limited or no intermediate stops within the South Coast region. Existing bus services thus operate as exclusive routes with few in-between stops and thus do not provide substantial interregional connectivity.

While the current bus service plays an important role, especially as it is the only regular transit service between the South Coast region and Boston, its use is limited, reflecting constraints related to travel time, and service frequency.

Vanpools/Carpools

Vanpools in communities of the South Coast region are provided through MassRides, a program of MassDOT. Although relevant as a complementary service vanpool and carpool travel times are severely impacted by slow travel speeds on the expressway and secondary roads.

Park-and-Ride

Park-and-ride facilities and carpool/vanpool services are offered along the primary regional travel corridors in the South Coast region. Park-and-ride lots are associated with car-pooling, van-pooling, or private bus service to Boston. There are nine public park-and-ride lots located in the South Coast region, as illustrated in Figure 4.1-1, of which five are located along the primary roadways from the region to the Boston metropolitan area and four not in the immediate vicinity of the primary access routes to Boston. In addition, three private park-and-ride lots in the South Coast region are available exclusively for customers using the private bus services to Boston. Three public park-and-ride lots are outside the South Coast region, but still along the Route 24 access corridor to Boston. Park-and-ride facilities as feeders for bus and car-pooling and van-pooling services are limited in their effectiveness as a transportation connection with Boston, due to the inconvenience of transfers and travel times associated with the congested roadway system, both in terms of traveling to the park-and-ride facility and travel from the park-and-ride facility to Boston.

Commuter Rail

Many communities within the South Coast Rail study area do not currently have commuter rail service. The nearest commuter lines (MBTA's Providence Line and Middleborough Lines) terminate on the northwest and northeast edges of the South Coast region. Starting in May 2013, MBTA, in cooperation with the Cape Cod Regional Transit Authority, established a seasonal weekends-only service known as the Cape Flyer, extending the Middleborough line from its current terminus in Middleborough to Hyannis. However, this service is limited to three round-trips per week, all on weekends, and thus serves weekend tourists rather than daily commuters between Boston and the South Coast. In fact, the three major cities in the South Coast region; Taunton, Fall River, and New Bedford are the only cities within 50 miles of Boston that are not served by passenger rail. The closest commuter rail stations are Middleborough/Lakeville (MBTA Middleborough Line), and Attleboro Station and Providence Station (MBTA Providence Line). The Middleborough Line serves areas east of the South Coast region and southeast of Boston, with stations in Lakeville and Bridgewater, while the Attleboro/Providence and Stoughton Lines serve communities to the north and west of the South Coast region. The Attleboro and Mansfield Stations are the primary access points on the Attleboro/Providence Line. The Stoughton Station serves as the primary access point on the Stoughton Line. All of the communities in the heart of the South Coast region, are outside a 6-mile access radius of these stations, and some, including the major population centers such as New Bedford and Fall River (combined population approximately 182,000), are more than 20 miles and up to 25 miles from the nearest train station. Due to their distance to the nearest commuter rail station the existing commuter rail lines to Boston are difficult for residents to access, especially for those living in Taunton, Berkley, Freetown, Fall River, and New Bedford. Travel to these stations is also limited to local secondary roads, which further increases travel time.

For those commuters in the South Coast region who live closer to commuter rail stations outside the South Coast region, constraints to the usage of the existing stations are posed by station parking and system capacity issues, as exemplified by the seat utilization ratio on the Providence line in Table 4.1-5. Commuter rail services are currently approaching or over capacity and system capacity is limited by parking capacity at these stations. Commuter rail parking lots in Attleboro, Mansfield, and to a lesser degree in Stoughton are already heavily utilized, as shown in Table 4.1-6 and are not positioned either within the regional road network or within their local (developed) context to handle projected future growth. In addition, some peak hour trains already experience heavy passenger loads, which was especially evident before the recent economic downturn. Therefore, the existing commuter rail service, although within reach of some communities in the South Coast region, is not sufficient to handle the anticipated growth in ridership.

Table 4.1-5 Ridership on Providence and Stoughton Lines

Line	AM Peak Passengers	AM Peak Seating Capacity	AM Peak Seat Utilization*
Providence	11,017	8,532	129%
Stoughton	2,771	3,558	78%

Sources: MBCR Ride Check December 2006, MBTA South Side Equipment Schedule
* Assumes all passengers continue to South Station, Stoughton and Providence/Stoughton Lines.

Table 4.1-6 Parking Utilization at Providence and Stoughton Lines Stations

Station	Occupied Spaces	Total Spaces	Utilization
Providence Line+			
Providence	N/A	330	N/A
South Attleboro	918	992	93%
Attleboro	756	770	98%
Mansfield	812	805	101%
*Stoughton Line**			
Stoughton	350	441	79%

\+ MBTA, 2000
* OCPC 2004

In summary, commuter rail service currently does not extend into the South Coast region, making access to commuter rail difficult for area residents. The relatively small ridership share of South Coast commuters using commuter rail services terminating outside the South Coast region is low, which reflects the constraints associated with this service for South Coast region commutes to Boston.

Vehicle Miles Traveled

VMT measures the extent of motor vehicle operation or the total number of vehicle miles traveled within the study area on given day. It is an important gauge for air quality and greenhouse gas emissions, as emissions of air pollutants and greenhouse gases are related to the distance traveled by automobiles (and to a lesser degree congestion). Daily regional automobile VMT is expected to grow from 109,926,000 under existing conditions to 118,894,000 by 2035 under the No-Build Alternative (based on updated modeling conducted by CTPS in 2012, see Appendix 3.2-H).

Regional Mobility

In addition to the lack of one-seat transit rides from one municipality to another within the South Coast region and adjoining regions, the lack of regional mobility is reflected by poor connectivity between the South Coast region and Boston.

The current transportation system serving the South Coast region is primarily a highway system composed of major, limited access state routes, regional highways, and local roadways (Figure 1.2-1). There are five major highways in the South Coast Rail project study area providing the primary access within and to adjacent regions:

Route 24 is the main north-south highway between the South Coast region and the metropolitan Boston area. This limited access facility begins at the Rhode Island state line at Tiverton, connects with I-195 on the east side of Fall River, and terminates at I-93/Route 128. It passes through Fall River, Freetown, Berkley, Taunton, and Raynham within the projects' study area.

Route 140 is a limited access highway connecting New Bedford and Taunton. It passes through the South Coast region communities of New Bedford, Freetown, Lakeville, and Taunton. The limited access portion of Route 140 ends at Route 24 in Taunton, providing an important link between the South Coast cities and towns of New Bedford, Dartmouth, Mattapoisett, Acushnet, and Taunton. Route 140 continues north from Taunton, roughly paralleling I-495, but not as a limited access facility.

Route 79 is a limited access segment approximately 4 miles long, beginning at I-195 on the west side of downtown Fall River and ending at Route 24 in northern Fall River. Route 79 provides a link from downtown Fall River and the communities located along I-195 west of Fall River to Route 24.

Route 138 is primarily a two-lane facility that passes through the South Coast region communities of Fall River, Somerset, Dighton, and Taunton, and provides access north to Raynham, Easton, and Stoughton. It connects with I-195 and the limited access segment of Route 79 in Fall River, the non-access controlled section of Route 140 in Taunton, and I-495. Route 138 also provides access to the MBTA's existing Stoughton station and planned stations in Easton and Raynham.

I-495 is a circumferential highway around metropolitan Boston that runs primarily northwest/southeast in the South Coast region, linking Route 24 to the I-90 and I-95 corridors. This facility provides access for a portion of the region to MBTA commuter rail stations in Middleborough/Lakeville and Mansfield. I-495 passes through Wareham, Rochester, Middleborough, Raynham, Taunton, and Norton, connecting with I-95 near the Mansfield/Foxborough line and Route 24 in Raynham.

Traffic generated within the South Coast region must travel on I-93/Route 128 and I-93/Route 3 (Southeast Expressway) to reach downtown Boston. Route 128 is Boston's inner circumferential highway that provides access to much of the metropolitan Boston region. Following I-93 north/Route 128 south from Route 24 leads to I-93/Route 3 (Southeast Expressway) and downtown Boston, approximately 8 miles from the I-93/Route 128/Route 3 interchange in Braintree. Following I-93 south/Route 128 north from Route 24 leads to I-95 approximately 3 miles to the north, and to I-90 approximately 15 miles to the north. I-90 (Massachusetts Turnpike) provides the only limited-access highway to Boston from west of the city. Route 128 and the Southeast Expressway are heavily congested roadways, particularly during peak periods.

Traffic volumes on Route 128 are approximately 135,000 vehicles per day north of Route 24 (towards I-95) and 167,000 vehicles per day to the south (towards I-93/Route 3). I-93/Route 128 provides four general purpose travel lanes in each direction between Route 24 and I-93/Route 3. North of the I-93/Route 3 interchange in Braintree, four general-access lanes and one high occupancy vehicle (HOV) lane in the peak direction and three general access lanes in the non-peak direction are provided during peak periods. During off-peak periods, the roadway provides four lanes in each direction through Southampton Street Massachusetts Highway Department operates HOV lanes on I-93/Route 3 from just south of the Furnace Brook Parkway exit in Quincy to the Columbia Road exit in Dorchester. As of 2009, the HOV lanes are open to all two-person carpools. Traffic volumes on I-93/Route 3 are as high as approximately 191,000 vehicles per day.

Freight Operations

The existing freight service for the South Coast region is shared between Mass Coastal and CSX. CSX dispatches several lines from its Selkirk, New York control office. The MBTA transferred dispatching of the Middleborough Secondary to CSX in 2009. There are several secondary tracks referred to as the Framingham (portion of track from Framingham to Mansfield), Attleboro (area of track from Attleboro side track to Cotley Junction), and the Middleborough (from Cotley Junction to the Middleborough branch of the Old Colony Railroad), as well as the New Bedford and Fall River branches.

CSX transferred ownership of the Fall River Secondary and New Bedford Main Line to MassDOT in June 2010. CSX simultaneously transferred the freight operating rights along these corridors to Mass Coastal. Currently, the existing freight service for the Fall River Secondary and New Bedford Main Line is therefore owned by the Commonwealth of Massachusetts and operated by Mass Coastal, while the Attleboro Secondary is owned by the Commonwealth of Massachusetts and operated by CSX. Freight service operates at maximum authorized speed of 40 mph with multiple civil and operational speed restrictions. All operations on these secondaries are under Dark Territory Control (no vital wayside signaling system). Figure 1.2-1 shows the existing rail transportation system, and Figure 3.2-10 shows the ownership of the rail segments.

CSX Freight Operations

The existing long haul freight service in this region is provided by CSX. CSX runs a late night/early morning train from Framingham to Attleboro where the train makes a run-around (reversing) operation and heads North on the NEC to Canton Junction (if warranted by customer demands) or east towards Cotley Junction to exchange cars with Mass Coastal. The train then continues on to Middleborough, exchanging cars with CSX local trains at Middleborough Junction. During this operation the Middleborough Secondary is often impeded as CSX uses the secondary as a switching lead, a track used by the switch engine while sorting railcars that gives it room to pull back while switching.

Additionally, CSX runs a freight train north to exchange cars with the Fore River Railroad at Greenbush Junction as well as to service sidings along the Middleborough branch of the OCRR. This movement occurs once per weekday between Braintree and Middleborough.

CSX runs this train during daylight hours in response to community concerns. This constrained operation is very difficult to complete at times trying to fit switching operations in small windows so as not to conflict with the MBTA passenger service.

Mass Coastal Freight Operations

Locally in the South Coast Region, Mass Coastal services both the New Bedford Mainline and the Fall River Secondary from the Cotley siding track north of Cotley Junction, where it interchanges with CSX for the South Coast Region. New Bedford is serviced 2 days per week, except during “sludge season,” when it is serviced three times per week, typically Tuesdays and Thursdays. “sludge season” refers to annual dredging in the New Bedford Harbor, the duration of which varies from year to year.[11] Fall River is serviced three days per week, typically Mondays, Wednesdays, and Fridays. According to Mass Coastal, the dredging activities in New Bedford require few, if any, additional trains. The tracks from Cotley and

[11] The “sludge season,” refers to the USEPA’s dredging project in New Bedford Harbor. According to the Water Quality Monitoring Summary Reports prepared for the USACE, the dredging seasons in 2010 through 2012 went from June to September.

Myricks Junctions southward are in poor shape, and Mass Coastal trains are typically unable to safely operate at speeds exceeding 10 mph.

Freight activity on the New Bedford Mainline track includes the Watuppa Line, which runs east/west between New Bedford and Westport. Approximately half of the Watuppa Line is owned by Bay Colony Railroad and the other half by MassDOT (currently operated by Bay Colony Railroad for Mass Coastal). The interchange point for Bay Colony and Mass Coastal is at the Watuppa Wye between Nash Road and Deane Street in New Bedford.

The majority of the existing freight traffic on the Fall River line is from/to Wharf Yard at Battleship Cove.

In Taunton, Mass Coastal operates the Dean Street Industrial Track, which runs approximately 1.5 miles from Weir Junction in Taunton north to Longmeadow Road near the Taunton/Raynham line. MCRR picks up/drops off cars for the Dean St. line at the Cotley siding track, the interchange point with CSX. In addition to the "main" track on the Dean Street line there are two double ended storage tracks adjacent to the Gallo Construction property, which occupies land between the Dean St. line and the CSX mainline to Attleboro. All three tracks are heavily used on a daily basis for switching and storage purposes to manage the large number of rock salt cars inbound and outbound from Gallo. Daily moves between Cotley Junction and the Dean Street line are required to deliver carloads and to retrieve empties.

4.1.3.2 Traffic Operations Analysis

This section presents information regarding existing traffic volumes, safety, and operational conditions along the highways or limited access freeway facilities in the study area. This section also provides existing safety and traffic operations information for the critical intersections at two existing park-and-ride lots. These park-and-ride lots, located in West Bridgewater and New Bedford, are important nodes as part of the No-Build (Enhanced Bus) Alternative. Based on field observations of current intersection operations and driveway configurations, these two locations appeared to have possible safety or capacity issues: The Mt. Pleasant Street park-and-ride facility in New Bedford and the Route 106/Route 24 park-and-ride access roadway intersection in West Bridgewater. These two unsignalized locations were analyzed further as they contain substantial parking capacity, exhibit some peak hour delay, and are located on higher volume collector and arterial roadways. The other park-and-ride locations were not studied for operations, as they appear to have less delay and or safety concerns.

Existing Traffic Volumes

Traffic volume data for the regional highway study area were collected in September and October 2008 and included ATRs. The location of all the traffic counters is shown in Figure 4.1-2.

Table 4.1-7 presents a summary of the recorded ATR volumes on a daily basis and during peak periods. Interstate 93 in Quincy carries approximately 175,000 vehicles per day (vpd) on a typical weekday, with approximately 7,800 northbound vehicles during the weekday morning peak hour and 7,300 vehicles during the weekday evening peak hour. Daily traffic volumes along Route 24 gradually increase from Fall River to Randolph more than doubling from approximately 49,000 vpd to 115,000 vpd.

To evaluate the traffic associated with No-Build (Enhanced Bus) Alternative, the TMCs were conducted during the weekday morning (7:00 to 9:00 AM) and weekday evening (4:00 to 6:00 PM) peak periods at the two park-and-ride lot study area intersections where such bus services would be provided to commuters driving to these lots These traffic volumes were reviewed, balanced, and rounded to the nearest five to develop the traffic volume networks used to evaluate existing traffic operations in the vicinity of the park-and-ride lots associated with the future bus services. Peak hour traffic flow networks for the existing traffic to and from these Park-and-Ride bus stops during weekday morning and evening peak hours are shown in Figures 4.1-3 and 4.1-4 for the summer and fall, respectively.

Table 4.1-7 Existing Traffic Volumes—Regional Highways

Location (Figure 4.1-2 number)	ADT [1]	Direction	Weekday Morning Peak Hour	Weekday Evening Peak Hour
1. Route 24 at Fall River-Freetown Line	48,650	NB	2030	1890
		SB	1770	2590
2. Route 24, south of Route 140	41,070	NB	1110	1250
		SB	1355	2875
3. Route 24, north of Route 44	74,810	NB	3930	2475
		SB	2110	3860
4. Route 24, north of I-495	96,420	NB	5260	3435
		SB	2630	4755
5. Route 24, north of Route 123	101,820	NB	5405	3255
		SB	2350	5445
6. Route 24, south of Pond Street	109,840	NB	5355	3330
		SB	3070	6010
7. Route 24, south of I-93	115,440	NB	5100	2770
		SB	3400	6110
8. I-93, south of Furnace Brook Pkwy	175,230	NB	7840	5310
		SB	5085	7255
9. I-93, south of Route 3	166,670	NB	5955	4750
		SB	6980	7375
10. Route 138, south of Bay Street	20,660	-	1345	1565
11. Route 138, south of Route 106	17,640	-	1400	1555
12. Route 140, north of Hathaway Road	51,580	NB	2015	2085
		SB	2160	2225
13. Route 140, south of Route 24	32,580	EB	830	1740
		WB	1595	1060

1 average daily traffic volume expressed in vehicles per day

Regional Growth

As the population in the South Coast Region and employment in the Boston area have grown, the demands on the roadway system linking Southeastern Massachusetts to the rest of the region have increased. Traffic volumes on the limited-access state routes linking the South Coast Region to the employment centers of Boston have been growing over the past decade, as shown in Table 4.1-8. Overall, traffic volumes on the roadways in the South Coast Region have grown at an annual rate of two to three percent over the past decade. However, traffic volumes have grown even more rapidly in some areas.

The largest increases in traffic volumes have been on Route 24 in Raynham and Taunton, where the traffic volumes have had annual increases of 4.1 and 5.0 percent respectively. Traffic volumes on Route 140 in Taunton have been increasing at an annual rate of 2.2 percent. Route 128 and I-93 (the Southeast Expressway) exhibit relatively stable traffic volumes. They are already some of the most congested highways in the state and traffic volumes on these roadways are at or near capacity for long portions of the day, making further increases in average daily traffic volumes infeasible. The minor decrease in traffic on portions of I-93 may reflect changes in motorist route choices due to Central Artery/Tunnel project construction, and demand reductions from the Route 3 corridor due to the restoration of the Old Colony Commuter Rail service.

Table 4.1-8 Average Daily Traffic Volume Growth

	Average Daily Traffic (vehicles per day)			Growth Rate (percent)		
Count Location	**Historic**	**Recent**	**Change**	**Total**	**Period**	**Annual**
Route 24						
Randolph (south of Route 128)	96,601	115,440	18,839	20	1989-2008	0.9
Avon (south of Pond Street)	90,196	109,840	19,644	22	1989-2008	1.1
Raynham (north of Route 44)	42,168	74,810	32,642	77	1989-2008	3.1
Taunton (north of Route 140)	37,734	68,109	30,375	80	1989-2005	3.7
Freetown (at Fall River city line)	29,822	48,650	18,828	63	1989-2008	2.6
Fall River (south of Wilson Road)	19,000	26,700	7,700	41	1989-2003	2.5
Route 140						
Taunton (south of Route 24)	23,133	32,580	9,447	41	1989-2008	1.8
Freetown (north of New Bedford city line)	25,250	32,447	7,197	29	1989-2004	1.7
New Bedford (north of Phillips Road)	23,449	32,400	8,951	38	1989-2005	2.0
New Bedford (north of Hathaway Road)	35,631	51,580	15,949	45	1989-2008	2.3
Route 79						
Fall River (north of Hermon Street)	16,460	25,400	8,940	54	1989-2004	2.9
I-95						
Foxborough (north of I-495)	57,800	93,200	35,400	61	1997-2003	8.2
Canton (south of I-93 / Route 128 / Route 1)	80,800	98,700	17,900	22	1997-2004	2.9
I-495						
Mansfield (south of Route 140)	37,400	69,900	32,500	87	1996-2005	7.2
Taunton (south of Bay Street)	40,400	69,100	28,700	71	1996-2005	6.1
Raynham (north of Route 24)	48,277	67,098	18,821	39	1996-2005	3.7
Middleborough (between Route 44 and Route 18)	35,100	56,100	21,000	60	1996-2005	5.4
I-195						
Fall River (west of Route 24)	66,053	81,339	15,286	23	1996-2005	2.3
New Bedford (east of Route 140)	55,300	73,500	18,200	33	1996-2005	3.6
Route 3						
Braintree (north of Union Street)	130,000	133,600	3,600	3	1996-1997	3.0
Route 128 / I-93 / I-95						
Quincy (north of Route 28, east of Route 24)	168,955	166,670	-2,285	-1	1989-2008	-0.1
Canton (at Dedham town line, west of Route 24 / I-95	128,537	134,684	6,147	5	1989-2004	0.3
Route 3 / I-93 (S.E. Expressway)						
Boston (north of Granite Avenue)	174,612	190,993	16,381	9	1999-2004	1.7
Boston (north of Southampton Street)	176,322	174,284	-2,038	-1	1989-2006	-0.1

ADT Average Daily Traffic (vehicles per day)
Source: Massachusetts Highway Department

The increases in traffic volumes on the principal highways linking the South Coast region to downtown Boston have led to deteriorating LOS on these roadways, especially during peak periods. Delays on these roadways are now common and have become worse over the past decade. These delays are especially prevalent on Route 24 as it approaches Route 138/I-93 in Randolph. Increases to peak-hour volumes of up to 3,500 and 4,000 vehicles per hour on Route 24 and on I93/Route 138 in Braintree in Raynham, respectively, have led to deterioration of LOS down to F on these major roadways, which are intended to relieve the local roadways from regional traffic. Several mitigation measures have been implemented on I-93 to reduce congestion (high-occupancy vehicle lanes, improved MBTA Red Line service, and Old Colony Commuter Rail service). However, this highway continues to operate at poor levels of service, resulting in substantial congestion. There are no roadway alternatives to the use of Route 24 and I-93, and no mitigation measures are planned to reduce congestion.

The lack of adequate capacity of the roadway system and the resultant reduction in LOS is anticipated to become even more problematic with the increased demand for transportation resulting from the growth of the South Coast region, especially as commuters living near Boston are moving away to areas further from the metropolitan core. Southeastern Massachusetts has been one of the fastest growing areas in the Commonwealth. Between 1960 and 2000, this area experienced a growth rate of 31 percent. Between 1960 and 1990, this area had an annual growth of over 2,500 people per year from a base population of 343,353 to its 1990 population of 430,846. Growth slowed somewhat between 1990 and 2000, to an annual growth of approximately 1,950 people per year. These figures translate to a growth of 4.5 percent between 1990 and 2000, which is greater than the growth rate of the Commonwealth as a whole. Each 10,000 new residents coming into the area are expected to generate a need for 3,500 new residential units, and are predicted to generate 27,650 new vehicle trips per day, further degrading the level of service provided by the regional transportation system.

Furthermore, as described in greater detail in the next sections, the LOS of the roadway system connecting the South Coast region to Boston will deteriorate even further, resulting in a concurrent increase in congestion, accidents, travel time, and air pollution; not only on the highways themselves but potentially also on nearby local roadways that may absorb the traffic spillover from nearby congested highways.

Access from the South Coast region to Boston is primarily via Route 24 to Interstate 93. These principal, limited-access highways currently operate at or over capacity, with peak-hour volumes of up to 4,000 vehicles per hour and level-of-service F on Route 24 in Raynham, and 3,500 vehicles per hour and level-of-service F on I-93/Route 128 in Braintree. Notwithstanding the beneficial effects on reducing congestion of several transportation improvements such as high-occupancy vehicle lanes on I-93, improved MBTA Red Line service, and Old Colony Commuter Rail service, these measures have not been able to fully accommodate the growth in transportation demand between Boston and the South Coast region. Route 24 continues to operate at poor levels-of-service, resulting in substantial congestion and decreased safety. For travel between Boston and the South Coast region there are no other direct highway routes besides Route 24 and I-93. Measures to fundamentally reduce congestion on these highways have proven to be limited in their effectiveness. Roadway improvement measures are being proposed (as described in Chapter 2); however while these measures will improve intraregional traffic conditions they will not address the need for increased transportation capacity between the South Coast region and Boston.

Regional Transportation Conditions

The freeway/highway analysis portion of the study reviews highway capacity at critical locations along the I-93, Route 24, and Route 140 limited access freeways and Route 138. Highway capacity directly affects bus operations along each bus route to Boston. The highway corridors in Southeastern Massachusetts experience more congestion in the morning peak period as traffic increases in a northbound direction towards the urban core of Metropolitan Boston. Traffic volumes are substantially less as traffic travels southbound away from the Metropolitan Boston urban core with traffic peaking again to a lesser degree near the urban centers of New Bedford, Fall River, and Taunton.

Freeways/Highways

Thirteen freeway and highway locations were identified as important roadway segments that influence bus travel times to downtown Boston. These locations are segments located between major highway interchanges with substantial traffic merging and diverging at each highway interchange. The analyses include seven locations on Route 24, and two each on Route 140, Route 138, and I-93. The freeway/highway capacity analysis for these segments gives an understanding of the existing directional traffic operations on each segment for each weekday peak hour. However, it should be understood that highway operations are also impacted by merging/diverging traffic at interchanges.

Table 4.1-9 shows LOS for 13 freeway segments. All Route 24 locations, north of Route 44, operate at LOS D or E conditions in the peak direction in each peak hour. Route 24 south of Pond Street and Route 24 south of I-93 both have LOS E conditions during the weekday evening peak hour in a southbound direction. This coincides with the outbound evening commuter peak from Boston. I-93 south of Furnace Brook Parkway also has LOS E conditions during the weekday morning peak hour in a northbound direction. I-93 South of Route 3 does not exhibit worse than LOS D conditions because of lower volumes than on I-93 south of Furnace Brook Parkway. Although observed traffic conditions often times indicate heavy congestion (LOS E /F) on these segments, this is often associated with merging/diverging traffic between travel lanes and the HOV lane, construction activities, crashes, and other factors that are not considered in the freeway analysis methodology. The analysis results are only for the segments and do not reflect interchange circulation and event dynamics with resultant delay and queuing.

Table 4.1-10 shows the analysis results for Route 138 highway segments. The HCM analysis procedures for highways (which are not limited access or divided) differ from the freeway analysis procedures and are reported separately. The results indicate LOS D operations on these roadway segments in both peak hours.

Table 4.1-9 Freeway Capacity Analyses Summary

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location/Movement	**Volume[1]**	**Density[2]**	**LOS[3]**	**Volume**	**Density**	**LOS**
I-93, south of Furnace Brook Parkway						
Northbound Travel Lane	7845	38.7	E	5310	24.5	C
Southbound Travel Lane	5085	23.5	C	7255	33.4	D
I-93, south of Route 3						
Northbound Travel Lane	5955	24.8	C	4755	19.3	C
Southbound Travel Lane	6985	29.4	D	7375	31.7	D
Route 24, south of I-93/128						
Northbound Travel Lane	5100	34.3	D	2775	15.6	B
Southbound Travel Lane	3400	19.2	C	6110	36.2	E
Route 24, south of Pond Street						
Northbound Travel Lane	5355	29.6	D	3330	17.8	B
Southbound Travel Lane	3075	16.5	B	6010	35.0	E
Route 24, north of Route 123						
Northbound Travel Lane	5405	30.6	D	3260	17.5	B
Southbound Travel Lane	2350	12.4	B	5445	28.5	D
Route 24, north of I-495						
Northbound Travel Lane	5260	29.3	D	3435	18.6	C
Southbound Travel Lane	2630	15.1	B	4755	26.5	D
Route 24, north of Route 44						
Northbound Travel Lane	3930	34.4	D	2475	19.7	C
Southbound Travel Lane	2110	16.6	B	3860	33.2	D
Route 24, north of Route 140						
Northbound Travel Lane	3795	19.3	C	2060	10.9	A
Southbound Travel Lane	1860	9.7	A	3910	20.0	C
Route 24, south of Route 140						
Northbound Travel Lane	1110	8.7	A	1255	10.2	A
Southbound Travel Lane	1355	11.0	A	2875	22.4	C
Route 24, north of Exit 9						
Northbound Travel Lane	1835	10.1	A	1610	13.9	B
Southbound Travel Lane	1430	13.1	B	2390	21.6	C
Route 24, south of Exit 8 ½						
Northbound Travel Lane	2030	16.4	B	1890	15.8	B
Southbound Travel Lane	1770	15.9	B	2590	22.9	C
Route 140, south of Route 24						
Eastbound Travel Lane	830	7.2	A	1740	14.2	B
Westbound Travel Lane	1595	13.2	B	1060	8.9	A
Route 140, north of Hathaway Road						
Northbound Travel Lane	2015	16.3	B	2085	16.7	B
Southbound Travel Lane	2160	19.3	C	2225	19.3	C

1 Volume in vehicles per hour.
2 Expressed as passenger cars per lane per mile
3 Freeway level of service

Table 4.1-10 Highway Capacity Analyses Summary

Location/Movement	Weekday Morning Peak Hour volume[1]	v/c[2]	LOS[3]	Weekday Evening Peak Hour volume	v/c	LOS
Easton						
Route 138, south of Route 106						
North/Southbound Travel Lane	1405	0.47	D	1565	0.54	D
Taunton						
Route 138, south of Bay Street						
North/Southbound Travel Lane	1350	0.44	D	1575	0.51	D

1 Volume expressed in vehicles per hour
2 Volume to capacity ratio
3 Level of service for Class II roadway as defined by HCM CH. 12 pp. 12-12, 12-13

Intersections

Intersection LOS was completed at park-and-ride locations observed with potential operational or safety issues. The two park-and-ride lots that were analyzed include West Center Street (Route 106) at Pleasant Street in West Bridgewater and Mt. Pleasant Street a park-and-ride lot in New Bedford.

In order to compare operations during off peak and peak seasonal traffic conditions, turning movement counts were completed during the summer and fall 2008. The West Bridgewater Route 106 park-and-ride lot is located directly south of Route 106 off of Pleasant Street. The lot is inaccessible by bus, so the Taunton bus parks outside the lot at the corner of Pleasant Street/internal connector roads to pick-up and drop-off passengers. The New Bedford Mt. Pleasant Street park-and-ride lot is located north of the Route 140 southbound ramps, on the east side of the street approximately 500 feet north of the ramp system.

The Route 106 park-and-ride lot is accessed via Pleasant Street or connector roads south of Route 106. Route 106 at this location is an arterial road with minimal gaps in traffic available at peak hour during the signal change at Route 106/Manley Street. Existing traffic operations indicate high delay for the minor street approach during the fall 2008 peak condition. During this period LOS for the Pleasant Street intersection minor approach is LOS F.

The Mount Pleasant Street park-and-ride lot is accessed directly from Mount Pleasant Street as a “T” type intersection. Mount Pleasant Street is a collector type road with average gaps at peak hour. Traffic operations indicate average to better than average delay for exiting traffic from the park-and-ride lot. During both summer and fall periods, traffic operations are acceptable at LOS C or better for all movements.

The HCM methodologies used for the analyses of unsignalized intersections are based on conservative analysis variables including periods of high critical gaps in traffic. However, actual traffic operations indicate that drivers on minor streets and driveways accept smaller gaps in traffic than the default values used in the analysis procedures and therefore experience less delay than reported by the HCM.

Also, the HCM methodologies do not fully take into account the beneficial grouping or platoon effects caused by the nearby signalized intersections. The results of HCM analysis procedures are the over-estimation of calculated delays at unsignalized intersections in the study area. A detailed review of the results should be completed when interpreting the capacity analysis results at unsignalized intersections. A summary of the unsignalized capacity analyses during both the summer and fall 2008 is presented in Tables 4.1-11 and 4.1-12, respectively.

Table 4.1-11 Existing Conditions–Park-and-Ride Lots Intersection Level of Service Analysis (Summer 2008)

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location/Movement	**v/c[1]**	**Delay[2]**	**LOS[3]**	**v/c**	**Delay**	**LOS**
West Center St. (Route 106) at Pleasant St.						
West Center St. WB – LT[4]	0.07	2.2	A	0.16	1.8	A
Pleasant St. NB – LR[5]	0.21	23.4	C	0.53	44.6	E
Mt. Pleasant St. at Park-and-Ride						
Mt. Pleasant St. SB – LT[6]	0.00	0.0	A	0.00	0.1	A
Park-and-Ride WB – LR[7]	0.01	12.9	B	0.17	16.5	C

1 Volume expressed in vehicles per hour
2 Volume to capacity ratio
3 Level of service for Class II roadway as defined by HCM CH. 12 pp. 12-12, 12-13
4 Indicates westbound left-through lane movement
5 Indicates northbound left-right lane movement
6 Indicates southbound left-through lane movement
7 Indicated westbound left-right lane movement

Table 4.1-12 Existing Conditions–Park-and-Ride Lots Intersection Level of Service Analysis (Fall 2008)

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location/Movement	**v/c[1]**	**Delay[2]**	**LOS[3]**	**v/c**	**Delay**	**LOS**
West Center St. (Route 106) at Pleasant St.						
West Center St. WB – LT[4]	0.09	2.6	A	0.12	3.7	A
Pleasant St. NB – LR[5]	0.51	52.4	F	0.63	53.9	F
Mt. Pleasant St. at Park-and-Ride						
Mt. Pleasant St. SB – LT[6]	0.00	0.0	A	0.00	0.1	A
Park-and-Ride WB – LR[7]	0.01	12.3	B	0.20	17.5	C

1 Volume expressed in vehicles per hour
2 Volume to capacity ratio
3 Level of service for Class II roadway as defined by HCM CH. 12 pp. 12-12, 12-13
4 Indicates westbound left-through lane movement
5 Indicates northbound left-right lane movement
6 Indicates southbound left-through lane movement
7 Indicated westbound left-right lane movement

As indicated in Tables 4.1-11 and 4.1-12, the West Center Street (Route 106) at Pleasant Street intersection northbound approach is operating with substantial delay during the evening peak hour. This delay was evident in both the summer and fall 2008 analysis periods with delays being longer during the fall likely because of higher traffic volumes related to school and vacation schedules. The morning peak hour also operates at a deficient LOS F in the fall. Based on the traffic characteristics of this location, higher delay should be expected with the high volume on Route 106 and the multiple conflicts represented along Route 106 including traffic from the Route 24 southbound off ramp which deposits Route 106 westbound traffic immediately east of this intersection.

4.1.3.3 Safety Analysis

The three years of crash data (2004-2006) for the regional highways were obtained and reviewed. Summary tables for the safety analysis are provided in Appendix 4.1-B.

Freeway/Highway Safety

In order to identify crash trends, historical crash data were obtained from MassDOT Highway Division for the most recent three-year period available for the regional highways in the study area: I-93, Route 24, and Route 140. For each highway, vehicle crashes were compiled by specific community. Data analyzed for each crash include year of incident, crash type, severity, weather, and time of day.

A brief summary of the highway crash data by roadway is provided below for I-93, Route 24 and Route 140.

I- 93

- As might be expected of the section of I-93 with the highest traffic levels between Randolph through Boston, the Quincy section experienced 755 crashes (28 percent) during the three-year period.
- Fifty-seven percent of all the crashes were rear-end type collisions, typical of heavily congested corridors.
- Approximately 31 percent of the crashes involved fatalities or injuries, 59 percent involved property-damage only and 10 percent of the crashes were unknown.
- Seventy-two percent of the crashes occurred during dry conditions.

Route 24

- The Fall River section of Route 24 has experienced the most crashes (432 crashes or 15 percent) during the latest three year period. Approximately 36 percent of the crashes involved fatalities or injuries, 58 percent involved property-damage only, and 2 percent of the crashes were unknown.
- Sixty-nine percent of the crashes occurred during dry roadway conditions.
- Single vehicle and rear-end type collisions each represented 37 percent of total crashes.

Route 140

The limited access portion of Route 140 between Route 24 in Taunton and New Bedford experienced a total of 758 crashes in the most three year period for which data were available.

- Approximately 36 percent of the crashes involved fatalities or injuries, 59 percent involved property damage only.
- Seventy percent of the crashes occurred during dry pavement conditions.

Park-and-Ride Locations Intersection Safety

The following summarize the crash numbers and characteristics at the two proposed park-and-ride locations analyzed.

Route 106/Route 24 Park-and-Ride in West Bridgewater

The proposed park-and-ride facility would be located north of Route 106 (West Center Street) opposite Pleasant Street in the northwest quadrant of the Route 106/Route 24 interchange. Some of the issues prompting more detailed analysis of the Route 106 site include:

- The average daily traffic volume on Route 106 at this location is 23,500 vehicles per day based on the MassDOT Highway Division 2006 traffic volume database.
- The Pleasant Street/West Center Street (Route 106) intersection is unsignalized with STOP sign control on Pleasant Street.
- Traffic speeds on Route 106.
- The gaps in peak hour traffic are limited with high volumes in both directions.
- Observed minor street delay was substantial during peak hours.

Mount Pleasant Street Park-and-Ride Lot in New Bedford

The site of the proposed facility is in the northwest quadrant of the Route 140 and King's Highway interchange (Exit 4). Some of the issues prompting a more detailed analysis of the Mount Pleasant Street site include:

- The Mount Pleasant Street park-and-ride site driveway is located close to a horizontal curve that limits sight distance to the north.
- Based on recent utilization surveys completed in the summer and fall of 2008, this 201-space lot is heavily utilized at 80 percent of capacity.
- During peak hour this intersection experiences substantial turning and vehicle conflicts to enter and exit the parking lot.
- The average daily traffic volume on Mt. Pleasant Street at this location is 13,500 vehicles per day based on the MassDOT Highway Division 2004 traffic volume database.

In order to check the safety record of each location, crash records were obtained from MassDOT Highway Division. Each location has a crash rate that is substantially less than the MassDOT Highway Division District 5 average crash rate of 0.59 crashes per million entering vehicles for unsignalized intersections.

Traffic operations at these two park–and-ride facilities are discussed further in the intersection analysis section.

Summary of Existing Safety Conditions

The number of accidents on the primary travel routes within the South Coast region has generally been increasing over the past years. Projected future growth in traffic volume on the principal South Coast region roadways cannot be sustained by the current regional transportation system. Recurring traffic congestion is becoming a more significant problem for the region, as is the increasing frequency of traffic accidents, especially along congested roadway corridors. Traffic volume increases may thus contribute to increased risk of injury and property damage for the commuting public. Not only has the number of accidents increased, but also the number of injuries has increased on two area highways. The annual growth rate in injuries was 11.6 percent on Route 24 and 8.0 percent on Route 93. However, Route 140 experienced an annual decline rate in injuries, at -5.9 percent. Although increasing the capacity of the region's highways might improve safety temporarily, substantial highway capacity expansions are constrained by transportation policy and due to the constraints posed by available space within existing rights-of-way, the potential for physical expansion of the highway links is limited.

4.1.3.4 Grade Crossings

Conditions at existing grade crossings were identified, as the rail alternatives using these grade crossings would increase train frequency at these grade crossings and could thus affect traffic flows and roadway capacity on either side of each grade crossing. This section presents information regarding the existing grade crossings at each of the alternatives' alignment, including existing traffic volumes and the existing frequency of both commuter and freight train service at the existing grade crossings.

Southern Triangle Study Area (Common to All Build Alternatives)

There are 50 public and private existing grade crossings within the Southern Triangle. Existing train frequency at these crossings ranges from two to five roundtrip freight trains per week (four to ten trains in total). Specific data for each crossing are provided in Tables 4.1-13 and 4.1-14 for the New Bedford Main Line and the Fall River Secondary, respectively.

Whittenton Alternative—Attleboro Secondary Line

There are 10 public and private grade crossings within the Attleboro Secondary Line segment of the Whittenton Alternative alignment. Existing train frequency at these crossings ranges from two to five roundtrip freight trains per week (four to ten trains in total). Specific data for each crossing are provided in Table 4.1-15.

Stoughton/Whittenton Alternatives—Stoughton Line from Canton Junction to Weir Junction, including Whittenton Branch

There are 41 existing public and private grade crossings along these portions of the Stoughton and Whittenton alignments. Train frequency from Canton Junction station to Stoughton station, along the existing MBTA Stoughton Commuter Rail Line alignment, ranges from 16 roundtrip (32 total trains) passenger trains per day on weekdays to no passenger trains on weekends. There is also freight service several times a week between Canton Junction station and Central Street in Stoughton. There is no existing train frequency along the unused rail alignment from Stoughton station to Longmeadow Road in Taunton. Between Weir Junction and Longmeadow Road, train frequency is approximately two roundtrip freight trains (four total trips) per month. Train frequency near Ingell Street at Weir Junction varies weekly, approximately 10 roundtrip freight trains operated by CSX and three roundtrips operated by Mass Coastal weekly). The Whittenton Alternative has six public and private grade crossings along the currently inactive Whittenton branch. Specific data for each crossing are provided in Table 4.1-16.

Table 4.1-13 Existing Conditions–Southern Triangle (New Bedford Main Line) At-Grade Crossing Summary

Name	Town	Approx. Milepost[1]	Type	Existing Track Use[2]	Posted Speed (MPH)	Traffic Volumes (AADT)	AADT Year
Ingell Street	Taunton	35.46	PUBLIC	FRT-5days/wk2	40	6,500	2000
Hart Street	Taunton	35.98	PUBLIC	FRT-5days/wk2	30	11,050	2000
Silva Crossing	Taunton	36.48	PRIVATE	FRT-5days/wk2	0	-----	-----
W. Stevens Street	Taunton	37.81	PRIVATE	FRT-5days/wk	10	200	-----
Cotley Street	Berkley	38.34	PUBLIC	FRT-5days/wk	10	240	-----
Padelford Street	Berkley	39.85	PUBLIC	FRT-5days/wk	40	1,900	2000
Myricks Street	Berkley	40.52	PUBLIC	FRT-5days/wk	40	3,840	-----
Malbone Street	Lakeville	40.96	PUBLIC	FRT-3days/wk	30	1,300	2001
Obed Crossing	Lakeville	41.34	PRIVATE	FRT-3days/wk	0	-----	-----
Plank Crossing	Lakeville	42.69	PRIVATE	FRT-3days/wk	0	-----	-----
Gravel Bank	Lakeville	42.99	PRIVATE	FRT-3days/wk	0	-----	-----
Stonewall Crossing	Lakeville	43.56	PRIVATE	FRT-3days/wk	0	-----	-----
Jeep Crossing	Lakeville	43.98	PRIVATE	FRT-3days/wk	0	-----	-----
Jeep Crossing	Lakeville	44.17	PRIVATE	FRT-3days/wk	0	-----	-----
Townline Crossing	Freetown	44.36	PRIVATE	FRT-3days/wk	0	-----	-----
Pierce Gravel Pit	Freetown	45.09	PRIVATE	FRT-3days/wk	0	-----	-----
Gas Line	Freetown	45.51	PRIVATE	FRT-3days/wk	0	-----	-----
Chace Road	Freetown	45.62	PUBLIC	FRT-3days/wk	40	3,100	2003
Private Road	Freetown	46.06	PRIVATE	FRT-3days/wk	0	-----	-----
Lucas Crossing	Freetown	46.37	PRIVATE	FRT-3days/wk	0	-----	-----
Lawrence Crossing	Freetown	46.66	PRIVATE	FRT-3days/wk	0	-----	-----
Braley Road	Freetown	47.24	PUBLIC	FRT-3days/wk	40	1,800	2000
Occupation Crossing	Freetown	47.35	PRIVATE	FRT-3days/wk	0	-----	-----
Pittsley Crossing	Freetown	47.44	PRIVATE	FRT-3days/wk	0	-----	-----
East Chipaway Rd.	Freetown	47.84	PUBLIC	FRT-3days/wk	40	2,500	2000
Private Road	Freetown	48.21	PRIVATE	FRT-3days/wk	0	-----	-----
Samuel Barnet Rd.	New Bedford	49.03	PUBLIC	FRT-3days/wk	30	5,100	2001
Polaroid Crossing	New Bedford	49.10	PRIVATE	FRT-3days/wk	0	-----	-----
Pig Farm Road	New Bedford	51.17	PUBLIC	FRT-3days/wk	10	-----	-----
Tarkiln Hill Road	New Bedford	51.93	PUBLIC	FRT-3days/wk	30	29,050	2001
Nash Road	New Bedford	52.91	PUBLIC	FRT-3days/wk	30	12,700	2000

1 Mileposts for NB Mainline Measure from Canton Junction to New Bedford Station
2 Existing Track Use Referenced From, NBFR Document ID: 46, Track Condition Assessment Report, 09/1995, (Pg 11-12)
FRT Freight service

Table 4.1-14 Existing Conditions – Southern Triangle (Fall River Secondary) At-Grade Crossing Summary

Name	Town	Approx. Milepost[1]	Type	Existing Track Use[2]	Posted Speed (MPH)	Traffic Volumes (AADT)	AADT Year
Mill Street	Berkley	40.73	PUBLIC	FRT-2days/wk	-----	-----	-----
Adams Lane	Berkley	41.19	PRIVATE	FRT-2days/wk	0	-----	-----
Private Road	Lakeville	41.31	PRIVATE	FRT-2days/wk	-----	-----	-----
Private Road	Freetown	41.41	PRIVATE	FRT-2days/wk	-----	-----	-----
Beechwood Street	Assonet	41.83	PUBLIC	FRT-2days/wk	0	300	2002
Richmond Road – North	Freetown	41.88	PUBLIC	FRT-2days/wk	40	3,000	2001
Private Road	Freetown	42.53	PRIVATE	FRT-2days/wk		-----	-----
Private Road	Freetown	42.84	PRIVATE	FRT-2days/wk	-----	-----	-----
Forge Road -North	Freetown	42.93	PUBLIC	FRT-2days/wk	10	900	2001
Richmond Road – South	Freetown	42.98	PUBLIC	FRT-2days/wk	40	3,600	2001
Forge Road - South	Freetown	43.25	PUBLIC	FRT-2days/wk	30	2,700	2001
Elm Street	Freetown	43.57	PUBLIC	FRT-2days/wk	40	4,200	2001
High Street	Freetown	44.31	PUBLIC	FRT-2days/wk	30	920	2001
Private Road	Freetown	44.97	PRIVATE	FRT-2days/wk	-----	-----	-----
Copicut Road	Freetown	45.31	PUBLIC	FRT-2days/wk	30	450	2001
Brightman Lumber	Freetown	46.10	PRIVATE	FRT-2days/wk	-----	-----	-----
Golf Club Road	Fall River	48.17	PRIVATE	FRT-2days/wk	-----	-----	-----
Near Canedy Street-Culvert	Fall River	48.51	PRIVATE	FRT-2days/wk	-----	-----	-----
Private Road	Fall River	49.60	PRIVATE	FRT-2days/wk	-----	-----	-----

1 Mileposts for NB Mainline Measure from Canton Junction to New Bedford Station
2 Existing Track Use Referenced From, NBFR Document ID: 46, Track Condition Assessment Report, 09/1995, (Pg 11-12)
FRT Freight service

Table 4.1-15 Existing Conditions–Whittenton Alternative Study Area (Attleboro Secondary Portion) At-Grade Crossing Summary

Street Name	Town	Approx. Milepost[1]	Type	Existing Track Use[2]	Posted Speed (MPH)	Traffic Volume (AADT)	AADT Year
West Britannia	Taunton	33.00	PUBLIC	FRT	30	4,600	2000
Danforth Street	Taunton	33.64	PUBLIC	FRT	-----	3,800	2000
Tremont Street	Taunton	34.06	PUBLIC	FRT	-----	15,500	2000
Oak Street	Taunton	34.23	PUBLIC	FRT	-----	11,500	2000
Porter Street	Taunton	34.47	PUBLIC	FRT	-----	3,000	2000
Cohannet Street	Taunton	34.54	PUBLIC	FRT	-----	1,900	2000
Winthrop Street	Taunton	34.60	PUBLIC	FRT	35	16,300	2000
Harrison Avenue	Taunton	34.74	PUBLIC	FRT	-----	1,900	2000
Somerset Avenue	Taunton	34.92	PUBLIC	FRT	-----	8,100	2000
Weir Street	Taunton	35.00	PUBLIC	FRT	-----	13,000	2001

1 Mileposts for NB Mainline Measure From Canton Junction to New Bedford Station
2 Existing Track Use Referenced From, NBFR Document ID: 46, Track Condition Assessment Report, 09/1995, (Pg 11-12)
FRT Freight service

Table 4.1-16 Existing Conditions–Stoughton/Whittenton Alternatives Study Area At-Grade Crossing Summary

Name	Town	Approx Milepost[1]	Type	Existing Track Use[2]	Posted Speed (MPH)	Traffic Volume (AADT)	AADT Year
Washington Street	Canton	15.57	PUBLIC	CR	20	18,900	2002
Pine Street	Canton	16.64	PUBLIC	CR	25	4,000	2000
Will Drive	Canton	17.05	PUBLIC	CR	-----	2,000	2002
Central Street	Stoughton	17.86	PUBLIC	CR	-----	15,400	2000
Simpson Street	Stoughton	18.16	PUBLIC	CR	-----	2,000	2000
School Street	Stoughton	18.65	PUBLIC	CR	-----	6,500	2004
Porter Street (RTE 27)	Stoughton	18.80	PUBLIC	CR	40	10,800	2000
Wyman Street	Stoughton	18.88	PUBLIC	CR	-----	3,500	2000
Brock Street	Stoughton	19.14	PUBLIC	CR	-----	3,050	2,001
Plain Street	Stoughton	19.54	PUBLIC	NA	-----	6,700	1998
Morton Street	Stoughton	20.15	PUBLIC	NA	45	-----	-----
Pearson's Crossing	Stoughton	20.26	PRIVATE	NA	45	-----	-----
Stanley Prod. Co.	Stoughton	20.32	PRIVATE	NA		-----	-----
Fish and Game Club	Stoughton	20.41	PRIVATE	NA	45	-----	-----
Elm Street	Easton	22.55	PUBLIC	NA	-----	4,250	2006
Oliver Street	Easton	22.68	PUBLIC	NA	-----	-----	-----
Williams Street	Easton	23.19	PUBLIC	NA	-----	-----	-----
Easton DPW	Easton	23.56	PUBLIC	NA	-----	-----	-----
Gary Lane	Easton	24.08	PUBLIC	NA	-----	-----	-----
Short Street	Easton	24.48	PUBLIC	NA	-----	4,000	2001
Depot Street - Route 123	Easton	24.90	PUBLIC	NA	-----	16,900	2006
Purchase Street	Easton	25.10	PUBLIC	NA	-----	2,100	2004
Prospect Street	Easton	25.82	PUBLIC	NA	-----	1,850	2003

Name	Town	Approx Milepost[1]	Type	Existing Track Use[2]	Posted Speed (MPH)	Traffic Volume (AADT)	AADT Year
Country Club	Easton	26.32	PRIVATE	NA	-----	-----	-----
Foundry Street - Route 106	Easton	26.71	PUBLIC	NA	-----	10,900	2004
Power Line	Easton	27.34	PUBLIC	NA	-----	-----	-----
Race Track Crossing	Raynham	29.00	PRIVATE	NA	-----	-----	-----
Elm Street	Raynham	30.35	PUBLIC	NA	-----	-----	-----
Carver Street	Raynham	30.79	PUBLIC	NA	-----	-----	-----
Route 138	Raynham	31.31	PUBLIC	NA	-----	-----	-----
Britton Street	Raynham	31.44	PUBLIC	NA	-----	-----	-----
King Phillip Street	Raynham	32.02	PUBLIC	NA	-----	-----	-----
East Britannia Street	Raynham	33.04	PUBLIC	NA	-----	-----	-----
Longmeadow Road	Taunton	33.82	PUBLIC	NA	40	11,550	2006
Dean Street - Route 44	Taunton	34.36	PUBLIC	FRT	40	28,750	2002
Whittenton Branch (inactive)							
Private Road	Raynham	29.99	PRIVATE	NA	-----	-----	-----
Private Road	Raynham	30.47	PRIVATE	NA	-----	-----	-----
Private Road	Raynham	30.84	PRIVATE	NA	-----	-----	-----
Private Road	Taunton	31.25	PRIVATE	NA	-----	-----	-----
Whittenton Street	Taunton	32.01	PUBLIC	NA	-----	-----	-----
Warren Street	Taunton	32.28	PUBLIC	NA	-----	-----	-----

1 Mileposts for NB Mainline Measure From Canton Junction to New Bedford Station
2 Existing Track Use Referenced From, NBFR Document ID: 46, Track Condition Assessment Report, 09/1995, (Pg 11-12)
NA Not Active
FRT Freight service
CR Commuter Rail

4.1.3.5 Station Area Traffic Conditions

There are 12 potential new or relocated commuter rail stations proposed for the Stoughton and/or Whittenton Alternatives. These stations are located in the following communities:

- New Bedford—King's Highway and Whale's Tooth
- Freetown—Freetown
- Fall River—Fall River Depot and Battleship Cove
- Taunton—Taunton, Dana St. and Taunton Depot
- Stoughton—Stoughton (relocated)
- Easton—Easton Village and North Easton
- Raynham—Raynham Park

Traffic impact study areas were based on the proposed station locations. This section provides roadway and intersection inventories, traffic volume data, safety data, and traffic operations for each station study area.

Southern Triangle

New Bedford Stations Study Area (King's Highway Station and Whale's Tooth Station)

The traffic impact study areas within the City of New Bedford were selected for the two proposed commuter rail station locations. Figure 4.1-5 shows the location of the New Bedford stations and selected study area intersections.

New Bedford has two station locations proposed for all rail alternatives. The following paragraphs summarize the locations and features of the King's Highway stations and Whale's Tooth station.

The King's Highway station, located in northern New Bedford along King's Highway east of Route 140, would serve all of the rail alternatives. The station would serve walk-in, bike-in, and drive-in customers.

The Whale's Tooth station, located at the Whale's Tooth parking lot would serve all of the rail alternatives. Located on the New Bedford waterfront, the City of New Bedford has constructed a parking lot on the site in anticipation of the commuter rail project. The station would include intermodal connections, potentially including ferry services. The site would serve walk-in, bike-in, and drive-in customers with primary access from Herman Melville Boulevard.

Existing Traffic Volumes

Traffic volume data for the Whale's Tooth station and King's Highway station were collected in September 2008 and included ATR counts and manual TMCs. TMCs were collected in June and July 2009 for three intersections in the King's Highway station study area and one intersection in the Whale's Tooth station study area.

Table 4.1-17 presents a summary of the daily and peak hour roadway volumes. King's Highway carries the highest volume in the vicinity of the King's Highway station with approximately 19,500 vehicles per day (vpd) on a typical weekday, approximately 1,300 vehicles during the weekday morning peak hour and 1,500 vehicles during the weekday evening peak hour. Coggeshall Street carries the highest volume in the vicinity of the Whale's Tooth station with approximately 11,500 vpd on a typical weekday, approximately 750 vehicles during the weekday morning peak hour and 850 vehicles during the weekday evening peak hour.

The TMCs were collected during the weekday morning (7:00 to 9:00 AM) and weekday evening (4:00 to 6:00 PM) peak periods at each of the study area intersections. These volumes were reviewed, balanced and rounded to the nearest five to develop the traffic volume networks used to evaluate existing traffic operations. The morning network peak hour occurred from 7:45 to 8:45 AM and the evening network peak hour occurred from 4:00 to 5:00 PM. Peak hour traffic flow networks for an existing weekday morning and evening peak hour for Whale's Tooth and King's Highway stations are shown in Figures 4.1-6 through 4.1-9.

Table 4.1-17 Existing Traffic Volumes–New Bedford

		Weekday Morning Peak Hour			Weekday Evening Hour		
Location	Daily Weekly Traffic[1]	Volume (vph)[2]	"K" Factor[3]	Peak Directional Flow[4]	Volume (vph)	"K" Factor	Peak Directional Flow
King's Highway, east of Route 140 NB Ramps	19,300	1,295	6.7%	WB 50%	1,455	7.6%	SB 53%
Church St., south of Park St.	11,500	790	6.9%	NB 53%	1,040	9.0%	SB 53%
Hillman St., west of Acushnet St/Route 18	4,900	360	7.3%	EB 60%	410	8.2%	NB 56%
McArthur Dr., north of Union St.	6,800	495	7.3%	NB 49%	600	8.8%	WB 55%
Union St., west of JFK Highway	8,500	630	7.4%	EB 59%	600	7.1%	EB 70%
Kempton St., east of Pleasant St.	6,630	920	13.8%	WB 74%	1,205	18.2%	WB 64%
Coggeshall St., west of North Front St.	11,500	750	6.5%	EB 61%	855	7.5%	EB 63%
Purchase St., south of Logan St.	10,100	630	6.2%	NB 53%	795	7.9%	NB 53%
Logan St., west of North Front St.	2,800	245	8.6%	EB 70%	220	2.1%	EB 62%
Acushnet Ave./Route 18, north of Hillman St.	2,000	145	7.4%	NB 59%	165	1.6%	SB 53%

Based on ATR counts conducted in September 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

Crash Analysis Summary—The New Bedford study area is made up of two separate subareas, the Whale's Tooth station and the King's Highway station. A total of 175 crashes occurred within the Whale's Tooth station study area, and 117 crashes occurred within the King's Highway station study area. There were seven intersections in the Whale's Tooth station study area and two intersections in the King's Highway station study area which exceeded the District 5 crash rates for signalized and unsignalized intersections.

The number of crashes and crash rates for the seven intersections that exceeded the District 5 crash rate in the Whale's Tooth station study area are:

- Four crashes occurred at the Acushnet Street at Hillman Street (0.65 vs. 0.59).
- Fourteen crashes occurred at the intersection of Purchase Street at Coggeshall Street (1.01 vs. 0.59).

- Sixteen crashes occurred at the intersection of Coggeshall Street at Acushnet Avenue/Route 18 Northbound (0.86 vs. 0.84).
- Fifty-one crashes occurred at the intersection of Coggeshall Street at Ashley Boulevard/Route 18 Southbound (2.92 vs. 0.84).
- Nine crashes occurred at the intersection of Logan Street at North Front Street (1.18 vs. 0.59).
- Fourteen crashes occurred at the intersection of Logan Street at Purchase Street (1.18 vs. 0.59).
- Six crashes occurred at the intersection of Wamsutta Street at Acushnet Avenue/Route 18 Northbound (3.30 vs. 0.59).
- Fifty percent of the crashes were angle type.
- Sixty-nine percent of the crashes occurred on dry pavement.

Two intersections (Church Street at Park Avenue (1.69 vs. 0.59) and King's Highway at Jones Street (0.64 vs. 0.59) exceed the District 5 crash rate in the King's Highway station study area.

- These intersections account for 38 percent of the incidents that occur in the King's Highway station study area.
- Sixty-one percent of the crashes at these two locations were angle type incidents. This may be a result of the high eastbound right turning volume.
- Forty-three percent of the crashes involved personal injuries and 48 percent involved property damage only.
- Sixty-eight percent of the crashes occurred during daylight hours on a dry road surface.

Traffic Operations Analysis—An analysis of the existing conditions near the Whale's Tooth and King's Highway stations was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-18.

The Whale's Tooth station study area consists of seven signalized and eleven unsignalized intersections. Under existing conditions, two signalized intersections operate at deficient levels of service. The Union Street at Route 18 Southbound intersection operates at LOS F in the evening peak hour and LOS E in the morning peak hour due to heavy southbound through movements on Route 18 that are unable to adequately pass through the intersection in the allocated green time. The six approaches and moderate traffic volumes are the primary reasons that the Kempton Street at Purchase Street intersection operates at LOS F and E in the morning and evening peak hours, respectively. Three of the unsignalized intersections operate at a LOS E in the evening peak hour and one intersection operates at LOS E and F in the morning and evening peak hours, respectively. These unsignalized intersections experience long delays in the evening peak hour for left-turning movements from the minor street to the major street. The delays are primarily due to the high through traffic volumes on the major street.

Table 4.1-18 New Bedford Intersection Capacity Analysis–2008 Existing Conditions

	Weekday Morning Peak Hour			**Weekday Evening Peak Hour**		
Signalized Intersections	**V/C**[1]	**Delay**[2]	**LOS**[3]	**V/C**	**Delay**	**LOS**
Whale's Tooth Station						
Hillman St at Purchase St.	0.35	12	B	0.49	14	B
Kempton St at Purchase St	0.76	>80	F	0.86	69	E
Union St. at Route 18	0.82	58	E	>1.00	>80	F
State Pier at McArthur Dr.	0.42	28	C	0.42	39	D
Route. 18 NB at Coggeshall St.	0.48	17	B	0.53	18	B
Route. 18 SB at Coggeshall St.	0.80	32	C	0.67	23	C
Coggeshall St. at Belleville Ave.	0.66	19	B	0.67	19	B
King's Highway Station						
King's Hwy. at Route. 140 NB Ramps	0.63	13	B	0.86	23	C
Route. 18 at Wood St	0.55	21	C	0.66	16	B
Church St. at Nash Rd	0.55	17	B	0.87	27	C
Church St. at Tarkiln Hill Rd	0.69	17	B	0.81	29	C
King's Hwy. at Stop & Shop driveway	0.46	8	A	0.66	12	B
King's Hwy. at Shaw's driveway	0.47	6	A	0.59	8	A
Unsignalized Intersections	**Critical**	**Delay**[1]	**LOS**[2]	**Critical**	**Delay**	**LOS**
Whale's Tooth Station						
Hillman St. at McArthur Dr.	EB L/R	11	B	EB L/R	12	B
McArthur Dr. at Herman Melville	WB L/R	14	B	WB L/R	17	C
Coggeshall St. at Purchase St.	SB All	17	C	NB All	39	E
Coggeshall St. at N. Front St.	NB All	50	E	NB All	>50	F
Purchase St. at Weld St.	WB L	23	C	WB L	43	E
Logan St. at Purchase St.	WB All	16	C	WB All	21	C
Logan St. at Acushnet Ave.	EB All	11	B	WB All	12	B
Logan St. at N. Front St.	EB All	21	C	EB All	20	C
Wamsutta St. at Herman Melville	EB All	11	B	EB All	12	B
Wamsutta St. at Acushnet Ave.	WB L/R	10	A	WB L/R	9	A
Purchase St. at Rt. 18 SB Exit Ramp	WB L/R	23	C	WB L/R	37	E
King's Highway Station						
Mt. Pleasant St. at Route. 140 SB	Rt. 140 WB	>50	F	Rt. 140 WB	>50	F
King's Hwy. at Mt. Pleasant St.	King's WB L	>50	F	King's WB L	>50	F
Church St. at Park Ave.	Park WB All	21	C	Park WB All	>50	F
Church St. at Irvington St.	Irvington WB	15	B	Irvington WB	20	C
King's Hwy. at Tarkiln Hill Rd.	Tarkiln EB L/R	25	D	Tarkiln EB L/R	>50	F

Source: Synchro 7.0 Software; Build 763
1 average control delay for critical movements, rounded to the nearest whole second, for unsignalized intersections.
2 level of service
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect the worst level of service intersections (LOS = F)

The King's Highway station study area consists of six signalized and five unsignalized intersections. All of the signalized intersections provide a good LOS in both the morning and evening peak hours. Two of the unsignalized intersections operate at LOS F in the morning and evening peak hours and two others operate at LOS F in the evening peak hour. These unsignalized intersections experience long delays for left-turning movements from the minor street to the major street. The delays are primarily due to the high through traffic volumes on the major street.

Freetown Station Study Area (Freetown Station)

The traffic impact study area within Freetown was selected based on the location of the proposed commuter rail station. Figure 4.1-10 shows the location of Freetown station and selected study area intersections.

The Freetown station, located on South Main Street south of the Route 24 and Route 79 interchange (Exit 9) would serve all of the rail alternatives. The station would serve drive-in customers and customers shuttled between the station and the nearby industrial parks, as well as pedestrians and bicyclists.

Existing Traffic Volumes—Traffic volume data for the Freetown station study area were collected in September 2008 and included ATRs and manual TMCs.

For the Freetown station study area, ATR data were collected on Route 79 between Route 24 northbound and southbound ramps (Exit 9) and on South Main Street, south of Route 24 Exit 9. Table 4.1-19 presents a summary of the daily and peak hour traffic volumes.

Table 4.1-19 Existing Traffic Volumes–Freetown

		Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location	Daily Weekly Traffic[1]	Vol. (vph)[2]	"K" Factor[3]	Peak Directional Flow[4]	Vol. (vph)	"K" Factor	Peak Directional Flow
S. Main St (Route 79), between Route 24 Ramps	10,100	825	8.1%	SB 70%	825	8.1%	SB 64%
S. Main St (Route 79), south of Route 24 Ramps	9,000	630	7.0%	SB 50%	705	7.8%	SB 50%

Based on ATR counts conducted in September 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

As presented in Table 4.1-19, Route 79 between the Route 24 Exit 9 ramps carries approximately 10,000 vehicles per day (vpd) on a typical weekday, with approximately 825 vehicles during the weekday morning peak hour and 825 vehicles during the weekday evening peak hour. South of the interchange, South Main Street carries approximately 9,000 vpd on a typical weekday, with approximately 650 vehicles during the weekday morning peak hour and 700 vehicles during the weekday evening peak hour.

The TMCs were collected during the weekday morning (7:00 to 9:00 AM) and weekday evening (4:00 to 6:00 PM) peak periods at each of the study area intersections. These volumes were reviewed, balanced and rounded to the nearest five to develop the traffic volume networks used to evaluate existing traffic operations. The morning network peak hour generally occurred from 7:00 to 8:00 AM and the evening network peak hour generally occurred from 4:15 to 5:15 PM. Peak hour traffic flow networks for an existing weekday morning and evening peak hour are shown in Figures 4.1-11 and 4.1-12, respectively.

Crash Analysis Summary—Crash rates at the intersections analyzed were less than the District 5, and Massachusetts statewide averages. Crashes occurred over the most recent three year period from 2004 to 2006. A brief summary of the crash data shows that:

- Most of the crashes that occurred in the study area are angle type (60 percent) collisions.
- The majority of the crashes occurred during the daylight hours (70 percent) on dry roadways (80 percent).

Traffic Operations Analysis—An analysis of the existing conditions in the vicinity of the Freetown station site was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-20.

Table 4.1-20 Freetown Intersection Capacity Analysis—Existing Conditions

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Unsignalized Intersections	Critical Movement	Delay[1]	LOS[2]	Critical Movement	Delay[1]	LOS[2]
Freetown Station						
S. Main St. at High St.	NW All	15	B	NW All	12	B
S. Main St. at Ridge Hill Rd.	NW All	46	E	NW All	41	E
S. Main St. at Route. 24 SB Ramps	SB L/R	16	C	SB L/R	36	E
S. Main St. at Route. 24 NB Ramps	NB L/R	41	E	NB L/R	49	E
S. Main St. at Narrows Rd.	EB L/R	16	C	EB L/R	18	C
S. Main St. at Copicut St.	WB L/R	11	B	WB L/R	11	B

Source: Synchro 7.0 Software; Build 763
1 average control delay by for the critical movement, rounded to the nearest whole second, for unsignalized intersections.
2 level of service
L= Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Under existing conditions, the Freetown station study area consists of six unsignalized intersections. Two of the unsignalized intersections currently operate at LOS E in the morning peak hour. Both those intersections as well as a third intersection operate at LOS E in the evening peak hour. These unsignalized intersections experience long delays in the evening peak hour for left-turning movements from the minor street to the major street. The delays are primarily due to the high through traffic volumes on South Main Street.

Fall River Stations Study Area (Battleship Cove Station and Fall River Depot Station)

The traffic impact study areas within the City of Fall River were selected based on the proposed commuter rail station locations. Figure 4.1-13 shows the location of the Fall River stations and selected study area intersections.

Fall River has two station locations proposed for the rail alternatives. The following paragraphs summarize the locations and features of the Battleship Cove and Fall River Depot stations.

Located within a block of Battleship Cove, the proposed Battleship Cove station is not anticipated to serve a substantial amount of regular commuter rail ridership. The station is intended, rather, to provide tourist access to the attractions at Battleship Cove with limited parking available. Traffic analysis for existing conditions was completed for this station study area, however future conditions analysis may only focus on pedestrian circulation and improving existing infrastructure deficiencies rather than full traffic impact analysis.

The Fall River Depot station, located 1 mile north of downtown Fall River at Route 79 and Davol Street, would serve all the rail alternatives. The site is envisioned to be a multi-modal transportation center with new mixed-use development and parking facilities. The site would serve walk-in, bike-in, and drive-in customers. Access will likely be provided from either Pierce Street or North Main Street in proximity to the Route 79 corridor.

Existing Traffic Volumes—Traffic volume data for the Battleship Cove station and Fall River Depot station were collected in September and October 2008 and included ATR counts and manual TMCs.

For the Battleship Cove station study area, ATR data were collected at the North Davol Street northbound U-turn, which merges with Davol Street Southbound near Cedar Street. Table 4.1-21 presents a summary of the daily and peak hour volumes.

As presented in Table 4.1-21, the North Davol Street U-turn (to Davol Street southbound) carries approximately 500 vehicles per day (vpd) on a typical weekday, with approximately 45 vehicles during the weekday morning peak hour and 40 vehicles during the weekday evening peak hour. The Davol Street U-turn to South Davol Street northbound carries twice as much on a daily basis. North Davol Street, south of President Avenue in the vicinity of the Fall River Depot station carries approximately 8,000 vpd northbound. Davol Street in the same area carries 10,000 vpd southbound on a typical weekday. In the morning peak hour approximately 500 vehicles travel northbound and 850 vehicles travel southbound, while in the evening peak hour 500 vehicles travel northbound and 650 vehicles travel southbound.

Table 4.1-21 Existing Traffic Volumes–Fall River

		Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location	**Daily Weekly Traffic [1]**	**Vol. (vph)[2]**	**"K" Factor[3]**	**Peak Directional Flow[4]**	**Vol. (vph)**	**"K" Factor**	**Peak Directional Flow**
S. Davol St. U-Turn, near Cedar St.	500	45	9.1%	NB 100%	40	8.1%	NB 100%
Route 79 NB Off-Ramp, north of President Ave.	6,400	435	6.8%	NB 100%	335	5.3%	NB 100%
Davol St. U-Turn, near Cedar St.	1,000	120	11.8%	SB 100%	65	6.4%	SB 100%
Route 79 NB Off-Ramp, south of N. Davol St. U-Turn	3,400	270	7.9%	NB 100%	200	5.9%	NB 100%
N. Davol St., south of President Ave.	8,100	525	6.4%	NB 100%	525	6.4%	NB 100%
Davol St., south of President Ave.	10,800	830	7.7%	SB 100%	635	5.9%	SB 100%

Based on ATR counts conducted in September and October 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

The TMCs were collected during the weekday morning (7:00 to 9:00 AM) and weekday evening (4:00 to 6:00 PM) peak periods at each of the study area intersections. These volumes were reviewed, balanced and rounded to the nearest five to develop the traffic volume networks used to evaluate existing traffic operations. The morning network peak hour generally occurred from 7:15 to 8:15 AM and the evening network peak hour generally occurred from 4:15 to 5:15 PM. Peak hour traffic flow networks for an existing weekday morning and evening peak hour are shown in Figures 4.1-14 and 4.1-15.

Crash Analysis Summary—Crash rates at the following Fall River intersections exceed the statewide average:

- North Davol Street at President Avenue (2.42 vs. 0.84 – 60 crashes)
- North Main Street at President Avenue (1.14 vs. 0.84 – 29 crashes)
- Water Street at Anawan Street (0.63 vs. 0.59 – 4 crashes)

A total of 117 crashes occurred over the three-year period from 2004 to 2006, with the majority (76 percent) occurring at the intersection of North Davol Street at President Avenue. A brief summary of the crash data shows that:

- Most of the crashes that occurred in the study are angle type (44 percent) and rear-end type (31 percent) collisions.
- There were no fatalities between the years 2004 and 2006.

Traffic Operations Analysis—An analysis of the existing conditions in the vicinity of the Fall River Depot station and Battleship Cove station was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-22.

The Battleship Cove station study area consists of four unsignalized intersections. The Central Street at Davol Street intersection operates at LOS E in the morning and LOS F in the evening peak hours. This intersection experiences long delays due to heavy westbound movements from the minor street (Central Street). Anawan Street at Davol Street is an all-way STOP-controlled intersection that currently operates at LOS F in the morning and evening peak hours. This intersection experiences long delays due to heavy southbound movement from Anawan Street onto Davol Street.

Under existing conditions, the Fall River Depot station study area consists of three signalized and four unsignalized intersections. All of the signalized and unsignalized intersections provide a good LOS (LOS C or better) in both the morning and evening peak hours.

Table 4.1-22 Fall River Intersection Capacity Analysis—Existing Conditions

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Signalized Intersections	V/C [1]	Delay [2]	LOS [3]	V/C	Delay	LOS
Fall River Depot Station						
N. Main St. at President Ave.	0.73	21	C	0.82	26	C
N. Davol St. at President Ave.	0.48	20	B	0.62	20	B
Davol St. at President Ave.	0.63	28	C	0.58	19	B
Unsignalized Intersections	Critical Movement	Delay[4]	LOS	Critical Movement	Delay	LOS
Fall River Depot Station						
N. Davol St at Pearce St	WB R	12	B	WB R	14	B
N. Davol St at Turner St	WB R	13	B	WB R	14	B
Davol St at northern U-turn near Cedar St (Davol SB to NB)	NE L	12	B	NE L	12	B
N. Davol St at southern U-turn near Cedar St (S. Davol NB to SB)	SW L	13	B	SW L	13	B
Battleship Cove Station						
Water St at Anawan St	EB All	15	C	WB All	15	C
Ferry St at Ponta Delgada St	EB L/R	14	B	EB L/R	12	B
Anawan St at Davol St	SB All	>50	F	SB All	>50	F
Central St at Davol St	WB L	45	E	WB L	>50	F

Source: Synchro 7.0 Software; Build 763
1 volume-to-capacity ratio
2 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
3 level of service
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect worst level of service intersections (LOS = F)

Taunton Stations Study Area (Taunton Station, Dana St. Station and Taunton Depot Station)

The traffic impact study areas within the City of Taunton were selected based on the proposed commuter rail station locations. Figure 4.1-16 shows the location of the various Taunton stations and selected study area intersections.

There are three proposed stations located in Taunton, the Taunton, Taunton Depot, and Dana Street Stations. Although only the Taunton Depot Station is located in the Southern Triangle, all three stations are addressed in this section.

The Taunton Depot station, located at the rear of Target Plaza, would serve the Stoughton and Whittenton Alternatives. This station site is approximately 14 acres and is located off of Route 140. The station would serve customers that drive to the station, as well as potential future walk-in or bike-in customers if redevelopment were to occur in the area.

The Taunton station, located along Arlington Street near Dean Street (Route 44), would serve the Stoughton Alternative. The location is within walking distance of downtown Taunton. The station would be a multimodal transportation center serving walk-in, bike-in, and drive-in customers.

Since the DEIS/DEIR, the Downtown Taunton Station site has been developed and is no longer available for the South Coast Rail project. The Dana Street Station would instead become the station that serves the Whittenton Alternative. The Dana Street Station is approximately 0.5 mile north of the previously-proposed Downtown Taunton Station and would be served by many of the same roadways that provided access to the Downtown Taunton Station.

Existing Traffic Volumes—Traffic volume data for the Taunton Depot and Taunton stations were collected in September and October 2008 and included ATR counts and manual TMCs. Table 4.1-23 presents a summary of the daily and peak hour traffic volumes. The highest daily two-way volume for an undivided roadway was almost 29,000 vehicles on Route 44 (Dean Street) west of Route 104. The peak hour volumes at that location were also the highest with 1,850 and 1,975 vehicles, respectively, in the morning and evening. The highest daily volume in one direction was about 22,700 vehicles on Route 140 westbound between the Route 24 ramps. The eastbound direction in that location carried about 12,400 vehicles for a daily two-way volume of 35,100.

Table 4.1-23 Existing Traffic Volumes—Taunton Stations Study Area

		Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location	Daily Weekly Traffic [1]	Vol. (vph)[2]	"K" Factor[3]	Peak Directional Flow[4]	Vol. (vph)	"K" Factor	Peak Directional Flow
Dean St/Route 44, west of Route 104	28,840	1,850	6.4%	EB 54%	1,975	6.8%	WB 56%
Winter St., south of King St	11,490	825	7.2%	NB 53%	1,070	9.4%	SB 52%
Dean St./Route 44, west of Prospect St.	19,560	1,255	6.4%	WB 51%	1,365	7.0%	WB 54%
County St./Route 140, east of Gordon Owen	21,390	1,390	6.5%	NB 60%	1,645	7.7%	SB 58%
Oak St., west of Maple St.	11,090	770	6.9%	EB 65%	840	7.6%	WB 55%
Tremont St., north of Washington St.	16,850	1,190	7.1%	SB 55%	1,355	8.1%	NB 52%
Washington St., east of Park St.	14,130	940	6.7%	EB 62%	1,070	7.6%	WB 59%
Frederick Martin Parkway, west of Cohannet St.	8,240	540	6.6%	SB 56%	715	8.7%	SB 67%
Route 140 EB between Route 24 Ramps	22,730	1,170	5.2%	EB 100%	2,315	10.2%	EB 100%
Route 140 WB between Route 24 Ramps	12,360	1,370	10.7%	WB 100%	1,005	8.1%	WB 100%
Route 140 EB east of Stevens St (Exit 11)	15,950	810	5.1%	EB 100%	1,740	10.9%	EB 100%
Route 140 WB east of Stevens St (Exit 11)	16,630	1,590	9.5%	WB 100%	1,050	6.3%	WB 100%

Based on ATR counts conducted in September and October 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

Crash Analysis Summary—A total of 345 crashes occurred in the Taunton study area over the three-year period from 2004 to 2006. Crash rates at the following eight intersections were all higher than the District 5 and Massachusetts statewide averages.

- Hart Street at County Street/Route 140
- Stevens Street /County St at Route 140 NB Ramps/Galleria Mall Ramp
- Kilmer Street at Lowell St at Oak Street
- Post Office Square at Taunton Green Street at Court Street
- Longmeadow Road/Hon Gordon Owen Riverway at Dean Street/Route 44
- School Street at Arlington Street /Purchase Street
- Spring Street at Summer Street (Route 140)
- Winter Street at School Street
- Purchase Street at Washington Street (This intersection had an extremely high calculated crash rate of 3.77 [vs. a District 5 average of 0.59]. There was a large occurrence of angle type collisions [92 percent] that may be due to the large northbound left turn movement.)

A brief summary of the crash data shows that:

- 62 percent of all the crashes involved property-damage only. Twenty-nine percent of the crashes involved a non-fatal injury
- 57 percent of the crashes were angle-type collisions

Traffic Operations Analysis—An analysis of the existing conditions in the vicinity of the Taunton Depot and Taunton stations was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-24.

Under existing conditions, the Taunton Depot station study area consists of seven signalized intersections. Only one location operates at a deficient LOS. Route 140 at the Route 24 southbound ramps operates at LOS F in the evening peak hour due to long delays for the left-turning eastbound traffic from the Route 24 ramp. It appears that the delay is primarily due to lack of adequate capacity to accommodate the high traffic volume on the Route 140 southbound approach.

Peak hour traffic flow networks for an existing weekday morning and evening peak hour are shown in Figures 4.1-17 and 4.1-18.

The Taunton station study area consists of seven signalized and three unsignalized intersections. All of the signalized intersections provide a good LOS in both the morning and evening peak hours except for Route 44 at Longmeadow Road, which operates at a LOS F and E during the morning and evening peak hours, respectively. One unsignalized intersection operates at LOS F in both the morning and evening peak hours and another operates at LOS F in the evening peak hour. These locations experience long delays for the minor street movements that are unable to find suitable gaps in the main street traffic. The delays are primarily due to the high through traffic volumes on the major street.

Table 4.1-24 Taunton Intersection Capacity Analysis—Existing Conditions

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Signalized Intersections	**V/C [1]**	**Delay [2]**	**LOS [3]**	**V/C**	**Delay**	**LOS**
Taunton Depot Station						
Route. 140 at Hart St.	0.81	38	D	0.92	41	D
Route. 140 at Route. 24 SB Ramps	0.80	28	C	>1.00	>80	F
Route. 140 at Route. 24 NB Ramps	0.84	5	B	0.65	3	A
Route. 140 at Taunton Depot Dr.	0.53	14	B	0.57	19	B
Route. 140 at Mozzone Boulevard	0.42	2	A	0.83	13	B
Route. 140 NB Ramps at Stevens St.	0.29	12	B	0.38	13	B
County St. at Silver City Galleria Mall Entrance/Exit	0.07	4	A	0.38	7	A
Taunton Station						
Route. 138 at Washington St	0.72	32	C	0.84	43	D
Route 44 at Dean St. /Route. 104	0.71	8	A	0.65	11	B
Route 44 at Longmeadow Rd	>1.00	>80	F	>1.00	65	E
Route 44 at Arlington St	0.93	34	C	0.95	41	D
Main St. at Union St.	0.87	29	C	0.84	27	C
Spring St at Summer St	0.67	24	C	0.75	25	C
Summer St at Hon. Gordon Owen Riverway	0.73	16	B	0.92	33	C
Unsignalized Intersections	**Critical Movement**	**Delay [1]**	**LOS [2]**	**Critical Movement**	**Delay**	**LOS**
Taunton Station						
Arlington St at School St	NB All	15	C	NB All	25	D
Washington St at Purchase St	SB All	23	C	NB All	>50	F
School St at Winter St	SB All	>50	F	SB All	>50	F

Source: Synchro 7.0 Software; Build 763
1 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
2 level of service
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect worst level of service intersections (LOS = F).

Stoughton Alternatives

Relocated Stoughton Station Study Area

Existing traffic conditions and impacts that could result from the South Coast Rail project in the vicinity of the existing Stoughton Station were not evaluated as part of the DEIS/DEIR because no changes to parking at this station were proposed at the time. As discussed in Chapter 3, subsequent to the DEIS/DEIR, MassDOT has proposed relocating Stoughton Station. The station would be shifted from its present location between Porter and Wyman streets to a new location south of the Wyman Street at-grade crossing. As a result, an inventory of existing transportation conditions and potential impacts was prepared to address the change in the location of the station platform, consolidation of parking and addition of new station driveways; and the increase in the number of parking spaces.

A field inventory of traffic conditions on study area roadways was conducted in April 2012. Nine intersections were included in the study area (Figure 4.1-19). Descriptions of intersection and roadway geometry, along with field inventory notes are provided in Appendix 4.1-K.

Existing Traffic Volumes—Traffic volume data for the intersections shown in Figure 4.1-19 were collected in April 2012. TMCs were conducted at the entrance and exit points to the existing MBTA parking lot driveways. Figure 4.1-19 shows the study area intersections as well as the entrance and exit locations of the existing MBTA parking lots.

Forty-eight hour ATR data were collected along Washington Street and Brock Street. Table 4.1-25 presents a summary of the recorded ATR volumes on a daily basis and during peak hours.

Traffic volumes include about 3,300 daily vehicles along Brock Street and 13,550 daily vehicles along Washington Street. Peak hour traffic represents 6 to 10 percent of the overall daily volume on the roadway network, meaning that there is a constant flow of traffic traveling along these roadways during the majority of the day.

Speed data were also collected along Washington Street and Brock Street in the locations described above. The average speeds along Washington Street and Brock Street were 33 mph and 26 mph, respectively.

Table 4.1-25 Stoughton Station Existing Traffic Volumes

Location	Daily Weekday[2]	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
		Vol[3]	K Factor[4]	Dir. Dist.[5]	Vol	K Factor	Dir. Dist.
Washington Street, north of Brock Street	13,550[1]	900	6.7	64% NB	1,170	8.6	67% SB
Brock Street, west of the railroad tracks	3,260[1]	350	10.7	60% WB	350	10.7	54% WB

Source: Daily and peak hour traffic counts
1 based on automatic traffic recorder counts conducted in April 2012.
2 average daily traffic volume expressed in vehicles per day
3 expressed in vehicles per hour
4 percent of weekday daily traffic that occurs during the peak hour
5 directional distribution of traffic

Crash Analysis Summary—Appendix 4.1-K provides the vehicle crash data for the study area intersections between 2007 and 2009. The crash data show that angle crashes were the leading type of crashes, followed by rear end crashes. The majority of crashes occurred on dry pavement, during off-peak times on a weekday. Approximately 74 percent of crashes resulted in property damage only.

During the 3-year period, the intersection of Pleasant Street at Park Street/ Washington Street had the highest number of crashes (35), which included a crash that involved a bicyclist. Wyman Street at Washington Street and Brock Street/ Kinsley Street at Washington Street were the intersections with the next highest number of accidents, with 23 accidents and 22 accidents, respectively.

Pleasant Street at Park Street/Washington Street exceeds both the state and district crash rate. For unsignalized intersections, Brock Street at Morton Street and Wyman Street at Brock Street are the only intersections with crash rates below the state and district crash rates. Crash rate calculations are provided in Appendix 4.1-K.

Traffic Operations Analysis—The existing traffic operations conditions were determined using the existing traffic volume networks. The morning and evening peak hour volume networks are depicted in Figures 4.1-20 and 4.1-21. The results of the signalized and unsignalized intersection capacity analyses for each of the study area intersections are summarized in Table 4.1-26 and 4.1-27, respectively. Complete traffic operations data for each location are provided in Appendix 4.1-K.

Table 4.1-26 Stoughton Station Existing Conditions Signalized Intersection Capacity Analysis

		Existing Conditions		
Location	**Period**	**v/c[1]**	**Delay[2]**	**LOS[3]**
Porter Street at Washington Street	Weekday Morning	0.69	21	C
	Weekday Evening	0.90	49	D
Pleasant Street at Park Street/	Weekday Morning	0.92	36	D
Washington Street	Weekday Evening	0.79	24	C

Source: Synchro 7 (Build 773, Rev 8) software
Notes:
1 volume-to-capacity ratio
2 average delay in seconds per vehicle
3 level of service

As shown in Table 4.1-26, both signalized intersections operate at acceptable levels of service under existing conditions. As indicated in Table 4.1-27, stop-controlled approaches to three unsignalized study area intersections operate at unacceptable LOS E or LOS F conditions.

Two intersections in the Stoughton Station study area experience excess queues during peak hours:

- The queue for eastbound Porter Street at the intersection with Washington Street exceeds the available storage length by approximately 60 feet during the evening peak hour. The northbound left-turn lane queue on Washington Street exceeds the available storage length by approximately 350 feet during the morning and evening peak hour. The queue on Southbound Washington Street exceeds the available storage length by 450 feet.

- For the intersection of Pleasant Street at Park Street/Washington Street, the northbound, southbound left turn, and through lanes on Park Street all experience queues that are longer than the available storage length during both the morning and evening peak hour.

Average queues for all lanes at the study area intersections are accommodated with two exceptions on Washington Street: the northbound left-turn lanes and southbound through lanes between the intersections with Freeman Street and Porter Street. The average queue is 120 feet in excess of available storage along the northbound direction and 60 feet in the southbound direction. See Appendix 4.1-K for details of the queue analysis.

Table 4.1-27 Stoughton Station Existing Conditions Unsignalized Intersection Capacity Analysis

	Critical	Morning Peak Hour				Evening Peak Hour Condition			
Location	Movement	Dem[1]	v/c[2]	Del[3]	LOS[4]	Dem	v/c	Del	LOS
Porter Street at Washington Street	WB RT	15	0.07	14	B	25	0.08	12	B
Freeman Street at Washington Street	WB RT	10	0.19	52.4	F	15	0.12	29	D
Wyman Street at Washington Street	EB RT	125	0.32	16	C	125	0.42	22	C
Morton Street/Trackside Plaza South Drive/MBTA Lot Driveway at Wyman Street	EB LT-TH-RT	290	0.09	3	A	130	0.02	1	A
	WB LT-TH-RT	65	0	1	A	140	0	1	A
	NB LT-TH-RT	Neg	0.01	14	B	5	0.04	14	B
	SB LT-TH-RT	10	0.04	11	B	30	0.07	10	B
Summer Street at Wyman Street	EB LT-RT	30	0.04	9	A	65	0.07	9	A
Brock Street at	EB LT-TH-RT	120	0.62	40	E	145	1.13	>120	F
Washington Street	WB LT-TH-RT	50	0.32	30	D	70	1.08	>120	F
	NB LT-TH-RT	410	0.14	4	A	465	0.09	3	A
	SB LT-TH-RT	345	0	0	A	775	0.01	1	A
Brock Street at Morton Street	EB LT-TH-RT	60	0.10	9	A	75	0.12	9	A
	WB LT-TH-RT	205	0.37	11	B	160	0.30	10	A
	NB LT-TH-RT	220	0.42	11	B	80	0.16	9	A
	SB LT-TH-RT	75	0.16	9	A	155	0.30	10	A
Brock Street at Wyman Street	WB LT-RT	95	0.13	9	A	115	0.15	10	A
Park Avenue/Sumner Street at	EB LT	205	>1.20	>120	F	120	1.05	>120	F
Park Street	EB TH-RT	15	0.05	16	C	25	0.10	18	C
	WB LT-TH-RT	20	0.09	21	C	50	0.26	23	C

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour for unsignalized intersections
2 volume-to-capacity ratio for the critical movement, values over 1.0 indicate demand in excess of capacity.
3 Control delay per vehicle, expressed in seconds, includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.
4 level of service of the critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn, Neg = negligible

Pedestrians and Bicycles—Stoughton Station is currently accessible via Porter Street, Wyman Street, Morton Street, Brock Street and Washington Street. Sidewalks are provided on the east side of Morton Street, north side of Brock Street and along both sides of Porter Street, Wyman Street and Washington Street.

Parking—Parking for Stoughton Station commuters is currently provided in a number of parking lots accessible from Porter Street, Wyman Street and Washington Street.

Public Transportation—The Providence/Stoughton Line is the only public transportation provided by the MBTA in this area. The existing Stoughton Station ridership is approximately 1,050 inbound boardings per day. The Brockton Area Transit Authority (BAT) provides a bus service between the Brockton Area Transit Center and Cobbs Corner via Washington Street with stops at the Westgate Mall, south of the study area, along Washington Street, and a terminal stop at Cobbs Corner, north of the study area. BAT provides services Monday through Saturday between 6:00 AM and 6:00 PM.

Easton Station Study Area

The traffic impact study areas within the Town of Easton were selected based on the locations of the proposed commuter rail stations. Figure 4.1-22 shows the location of the Easton stations and selected study area intersections.

Easton has two station locations proposed for the Stoughton Alternative. The following paragraphs summarize the locations and features of the Easton Village and North Easton stations.

The Easton Village station is proposed to be located next to the Old Colony Railroad Station which is part of the discontiguous H.H. Richardson National Historic Landmark. The site is currently limited to the railroad right-of-way and is within walking distance of downtown Easton. The site would be a village-style station serving walk-in and bike-in customers. No commuter parking would be provided, however approximately 12 kiss and ride spaces would be designated in an existing private lot. Traffic analysis for existing conditions was completed for this station study area, however, future conditions analysis may only focus on pedestrian circulation and improving existing infrastructure deficiencies rather than full traffic impact analysis.

The North Easton station is proposed on the Stoughton town line at the rear of the Roche Brothers plaza and accessible from an existing traffic signal on Route 138. The station would have a surface parking lot and would primarily serve drive-in customers, although the station may also attract some walk-in customers from the existing plaza development and from limited nearby residences.

Existing Traffic Volumes—Traffic volume data for the Easton Village and North Easton stations within the Easton study area were collected in September 2008 and included ATRs and manual TMCs. Table 4.1-28 presents a summary of the daily and peak hour volumes. Route 138 north of Elm Street carries the highest traffic volumes near the North Easton station. It carries approximately 19,500 vehicles per day (vpd) on a typical weekday, with approximately 1,700 vehicles during the morning peak hour and 1,650 vehicles during the evening peak hour.

The TMCs were collected during the weekday morning (7:00 to 9:00 AM) and weekday evening (4:00 to 6:00 PM) peak periods. The volumes were reviewed, balanced and rounded to the nearest five to develop the traffic volume networks used to evaluate existing traffic operations. The network morning peak hour occurred from 7:15 to 8:15 AM and the network evening peak hour occurred from 4:45 to

5:45 PM. Peak hour traffic flow networks for an existing weekday morning and evening peak hour are shown in Figures 4.1-23 and 4.1-24, respectively.

Table 4.1-28 Existing Traffic Volume Summary–Easton

		Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location	Daily Weekly Traffic [1]	Vol. (vph)[2]	"K" Factor[3]	Peak Directional Flow[4]	Vol. (vph)	"K" Factor	Peak Directional Flow
Route 138, south of Main St.	17,000	1,395	8.2%	NB 61%	1,415	8.3%	SB 53%
Route 138, north of Elm St.	19,400	1,690	8.7%	NB 75%	1,660	8.6%	SB 60%
Route 138, north of Roche Bros.	15,200	1,455	9.6%	NB 72%	1,355	8.9%	NB 62%
Main St, east of Center St.	13,600	1,160	8.5%	EB 76%	1,140	8.3%	WB 60%

Based on ATR counts conducted in September and October 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

It should be noted that the Central Street Bridge was closed during the initial data collection period. Subsequent traffic counts were conducted in 2009 and traffic volumes did not change within the Route 138 and Center Street corridors.

Crash Analysis Summary—A total of 79 crashes occurred in the Easton study area over three-year period from 2004 to 2006. Only the crash rate at the intersection Elm Street at North Main Street exceeded the MassDOT District 5 average crash rate. The following summarizes the crash data:

- The majority of the crashes in the area appear to be at the intersections of Route 138/Washington Street at Elm Street (17 crashes), and Route 138/Washington Street at Main Street (24 crashes);
- Fifty-nine percent of all the crashes in this area were angle-type collisions; and
- Sixty-five percent of the crashes involved property damage only. Twenty-seven percent of the crashes involved injury to one or more persons. None of the crashes were fatal.

Traffic Operations Analysis—An analysis of the existing conditions in the vicinity of East Village station and North Easton station was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-29.

Under existing conditions, the North Easton station study area consists of one signalized and two unsignalized intersections. The signalized intersection provides a good LOS in both the morning and evening peak hours. The two unsignalized intersections on Route 138 operate at a LOS F in the morning and evening peak hours. These intersections experience long delays for the minor street traffic that is unable to find suitable gaps in the main stream traffic. The delays are primarily due to high through traffic volumes on Route 138.

Table 4.1-29 Easton Intersection Capacity Analysis—Existing Conditions

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Signalized Intersections	V/C[1]	Delay[2]	LOS[3]	V/C	Delay	LOS
North Easton Station						
Rt. 138 at Roche Bros. Way	0.71	12	B	0.66	13	B
Easton Village Station						
Rt. 138 at Belmont St./Rt. 123	0.70	15	B	>1.00	43	D
Rt. 138 at Main St.	0.82	>80	F	0.89	39	D
Unsignalized Intersections	Critical Movement	Delay[4]	LOS	Critical Movement	Delay	LOS
North Easton Station						
Rt. 138 at Elm St.	Elm WB All	>50	F	Elm WB All	>50	F
Rt. 138 at Union St.	Union WB L/R	>50	F	Union WB L/R	>50	F
Easton Village Station						
Elm St. at North Main St.	Elm WB L/R	13	B	Elm WB L/R	14	B
Main St. at Center St. at Lincoln St.	Center NB All	>50	F	Center NB All	>50	F
Lincoln St. at Barrows St.	Barrows NB All	11	B	Barrows NB All	21	C
Rt. 138 at Roosevelt Circle	Roosevelt EB L	45	E	Roosevelt EB L	24	C

Source: Synchro 7.0 Software; Build 763
1 volume-to-capacity ratio
2 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
3 level of service
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections.
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect the worst level of service intersections (LOS = F).

The Easton Village station study area consists of two signalized and four unsignalized intersections. One of the signalized intersections (Route 138 at Main Street) operates at a LOS F in the morning peak hour due to heavy eastbound left-turning movements from Main Street that cannot be processed through the intersection in the allocated green time. The unsignalized intersection of Main Street at Center Street and Lincoln Street operates at LOS F in both the morning and evening peak hours. Roosevelt Circle at Route 138 operates at LOS E in the morning peak hour. These intersections experience long delays for the minor street traffic that is unable to find suitable gaps in the main road traffic. The delays are primarily due to the high traffic volume on Route 138.

Raynham

The traffic impact study areas within the Town of Raynham were selected based on the locations of the proposed commuter rail station. Figure 4.1-25 shows the location of the Raynham Park station and selected study area intersections.

The Raynham Park station, located at the former Raynham-Taunton Greyhound Park in Raynham, would serve the Stoughton Alternative. The site is now occupied by a simulcast center, and has a large surface parking lot along Route 138 near the Raynham/Easton town line. The site would serve mostly drive-in customers with additional walk-in customers being drawn from planned redevelopment on the site.

Existing Traffic Volumes—Traffic volume data for the Raynham Park station were collected in September 2008 and included ATR counts and manual TMCs. ATR data were collected at Route 138 north of the Dog Track. Table 4.1-30 presents a summary of daily and peak hour volumes. Route 138 carries 17,000 vehicles daily and 1,460 and 1,560 vehicles, respectively in the morning and evening peak hours.

Table 4.1-30 Existing Traffic Volume Summary–Raynham

		Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Location	Weekly Daily Traffic[1]	Vol. (vph)[2]	"K" Factor[3]	Peak Directional Flow[4]	Vol. (vph)	"K" Factor	Peak Directional Flow
Route 138, north of Dog Track	17,060	1,460	8.6%	NB 76%	1,560	9.2%	SB 67%

Based on ATR counts conducted in September 2008.
1 average daily traffic (ADT) volume expressed in vehicles per day
2 peak period traffic volumes expressed in vehicles per hour
3 percent of daily traffic that occurs during the peak period
4 directional distribution of peak period traffic
Note: peak hours do not necessarily coincide with the peak hours of the individual intersection turning movement counts

Crash Analysis Summary—A total of 34 crashes occurred during the three-year period from 2004 to 2006 in the Raynham study area. Crash rates at all intersections were less than the District 5 and Massachusetts statewide averages. The following summarize some of the crash data:

- Forty-six percent of all crashes in this area are angle-type collisions.
- Forty-nine percent of the crashes in this area contained damage to property only.
- Thirty-two percent of the crashes involved a non-fatal injury. No fatal crashes occurred in this area.

Traffic Operations Analysis—An analysis of the existing traffic operating conditions in the vicinity of the Raynham Park station was performed to assess the ability of intersections to process traffic. The results of the analyses for these intersections for 2008 Existing Conditions are presented in Table 4.1-31.

The Raynham Park station study area consists of three signalized and eight unsignalized intersections. Under existing conditions, all the signalized intersections provide a good LOS in both the morning and evening peak hours.

Five of the unsignalized intersections operate at LOS E or F in both the morning and evening peak hours. These intersections experience long delays for the minor street traffic that is unable to find suitable gaps in the high volume of Route 138 through traffic. Peak hour traffic flow networks for an existing weekday morning and evening peak hours are shown in Figures 4.1-26 and 4.1-27, respectively.

Table 4.1-31 Raynham Intersection Capacity Analysis–Existing Conditions

	Weekday Morning Peak Hour			Weekday Evening Peak Hour		
Signalized Intersections	V/C[1]	Delay[2]	LOS[3]	V/C	Delay	LOS
Raynham Park Station						
Route 138 at Route 106 (Foundry St)	0.81	19	B	0.93	28	C
Route 138 at Elm St.	0.70	21	C	0.68	19	B
Route 138 at Carver St.	0.79	14	B	0.85	18	B
Unsignalized Intersections	**Critical Movement**	**Delay[4]**	**LOS**	**Critical Movement**	**Delay**	**LOS**
Raynham Park Station						
Route 138 at Wilbur St.	Wilbur WB L/R	33	D	Wilbur WB L/R	30	D
Route 138 at I-495 NB On/Off-Ramp	I-495 Ramp WB All	>50	F	I-495 Ramp WB All	>50	F
Route 138 at I-495 SB On/Off-Ramp	I-495 Ramp EB All	>50	F	I-495 Ramp EB All	>50	F
Route 138 at Center St.	Center WB L	>50	F	Center WB L	>50	F
Route 138 at Britton St. (East)	Britton WB L/R	>50	F	Britton WB L/R	>50	F
Route 138 at Britton St. (West)	Britton EB L/R	38	E	Britton EB L/R	>50	F
Route 138 at Robinson St.	Robinson WB L/R	26	D	Robinson WB L/R	13	B
Route 138 at Dog Track driveway	Driveway EB All	36	C	Driveway EB All	34	D

Source: Synchro 7.0 Software; Build 763
1 volume-to-capacity ratio
2 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
3 level of service
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect the worst level of service intersections (LOS = F)

4.1.4 Analysis of Impacts by Alternative

4.1.4.1 No-Build (Enhanced Bus) Alternative

The No-Build Alternative includes enhanced bus service. The impact analysis of the No-Build Alternative is focused on the roadways serving each station in the study area and analyzes its impact on traffic operations. No analyses of pedestrian and bicycle conditions, parking, or public bus transit service were conducted for the No-Build Alternative because they are not expected to change near proposed station locations. The purpose of the No-Build analysis is to provide a base against which the results of the analysis of the Build Alternatives can be compared to determine the impacts of each Build Alternative.

The No-Build (Enhanced Bus) Alternative consists of potential transportation improvements for the Boston commute to and from South Coast communities that could be implemented at minimal cost and limited impact to the environment. Currently, South Coast commuters to Boston must drive (alone, or in a carpool), commute to the nearest bus station, commute to a park-and-ride facility, or commute to a MBTA commuter rail station. The closest existing MBTA commuter rail stations with linkage to Boston are located outside the South Coast region in Attleboro, South Attleboro, Mansfield, and Lakeville. Refer to Chapter 3 for a detailed description of the No-Build Alternative.

Background Development/Infrastructure Improvements

While the CTPS travel demand model accounts for the majority of future development areas within its demographic forecasts, a number of large development projects were not specifically included in the model's future land use assumptions. Identification of these projects was coordinated with the Massachusetts Environmental Policy Act office, MassDOT Highway Division, SRPEDD, and OCPC. The No-Build Alternative transportation analysis includes travel demands from these specific planned developments in the study area, roadway improvements planned or programmed to be completed by or before 2030, and bus service improvements. These development projects and transportation improvements, including bus enhancements, are described in detail in Appendix 4.1-L. The existing traffic volume networks were projected into future conditions using annual traffic growth factors combined with project-specific traffic volumes to the traffic volumes to create the 2030 No-Build condition traffic volume networks, which are depicted in Figures 4.1-28 through 4.1-43.

Traffic Operations Analysis

The following section describes how the No-Build (Enhanced Bus) Alternative was analyzed for traffic operations. Traffic operations on Route 24, Route 140, I-93, Route 138, and at the driveways to new and expanded park-and-ride facilities were analyzed to assess the impact of the enhanced bus service under 2030 No-Build conditions. Intersections around each proposed rail station location were analyzed to establish a base condition for projecting traffic impacts from the rail alternatives.

Traffic operations were analyzed for the No-Build Alternative using the methodology previously described. The results of these analyses are presented in tables that include existing LOS and highlight locations that would operate at unacceptable levels of service during at least one peak hour. Intersections that would degrade to unacceptable levels of service under 2030 No-Build conditions are denoted in **bold**. LOS analyses for all highways and intersections are provided in Appendix 4.1-I

CTPS provided ridership projections by transportation mode for the South Coast Rail project in the 2030 horizon year as well as projections for future traffic growth along the major corridors between Boston and the South Coast. The No-Build Enhanced Bus Alternative ridership projections and projected freeway volumes were used to analyze expected future traffic conditions. Traffic volume estimates from the specific No-Build development projects were added to the CTPS traffic volume projections to create the 2030 No-Build condition traffic volume networks, which are shown in Figures 4.1-28 through 4.1-43.

The freeway analysis includes 11 locations on Route 24, Route 140, and I-93. The highway analysis was conducted for two locations on Route 138 in Taunton and Easton. Intersection analyses were conducted for the park-and-ride facility driveways on West Center Street in West Bridgewater and on Mt. Pleasant Street and Acushnet Street in New Bedford. The analysis results for intersections near each proposed rail stations are presented by municipality below.

Freeways/Highways

LOS was reviewed on two freeway segments on I-93, nine segments on Route 24, and two segments on Route 140. Table 4.1-32 provides the results of the freeway operations analysis. The results of the analysis indicate that freeway levels of service are expected to decline in the peak direction (northbound in the morning peak hour and southbound in the evening) on a number of segments. Typically, there is a one-letter grade reduction at each location. On eleven segments, the decline results in a deficient LOS, especially in the northbound direction during the morning peak hour. The most dramatic changes would occur near the Fall River-Freetown line in the vicinity of the new Exit 8A interchange because of proposed new development in that area.

Table 4.1-32 2030 No-Build Freeway Capacity Analyses Summary

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	2030 No-Build			Existing	2030 No-Build		
Location/Direction	LOS[1]	Volume[2]	Density[3]	LOS	LOS	Volume	Density	LOS
I-93, south of Furnace Brook Pkwy.								
Northbound Travel Lane	E	8760	>45.0	F	C	5450	25.2	C
Southbound Travel Lane	C	5220	24.1	C	D	8100	39.7	E
I-93, south of Route 3								
Northbound Travel Lane	C	6645	28.1	D	C	4880	19.8	C
Southbound Travel Lane	D	7160	30.4	D	D	8235	38.1	E
Route 24, south of I-93/Route 128								
Northbound Travel Lane	D	5700	43.4	E	B	2875	16.2	B
Southbound Travel Lane	C	3520	19.8	C	E	6830	>45.0	F
Route 24, south of Pond Street								
Northbound Travel Lane	D	5985	35.2	E	B	3445	17.5	B
Southbound Travel Lane	B	3180	17.0	B	E	6715	44.7	E
Route 24, north of Route 123								
Northbound Travel Lane	D	6050	37.1	E	B	3435	18.4	C
Southbound Travel Lane	B	2510	13.2	B	D	6100	34.2	D
Route 24, north of I-495								
Northbound Travel Lane	D	5910	35.0	E	C	3720	20.1	C
Southbound Travel Lane	B	2900	16.6	B	D	5355	30.4	D
Route 24, north of Route 44								
Northbound Travel Lane	D	5105	>45.0	F	C	3705	33.1	D
Southbound Travel Lane	B	3320	27.1	D	D	5070	>45.0	F
Route 24, north or Route 140								
Northbound Travel Lane	C	5020	>45.0	F	A	3740	33.6	D
Southbound Travel Lane	A	3520	29.2	D	C	5240	>45.0	F
Route 24, south of Route 140								
Northbound Travel Lane	A	3535	30.6	D	A	3560	30.0	D
Southbound Travel Lane	A	3285	27.8	D	C	3705	31.1	D
Route 24, north of Exit 9								
Northbound Travel Lane	A	2475	13.6	B	B	3670	37.9	E
Southbound Travel Lane	B	3460	32.9	D	C	3185	29.1	D
Route 24, south of Exit 8 ½								
Northbound Travel Lane	B	4840	>45.0	F	B	2585	21.6	C
Southbound Travel Lane	B	2740	24.7	C	C	5490	>45.0	F
Route 140, south of Route 24								
Eastbound Travel Lane	A	1320	11.5	B	B	2150	17.6	B
Westbound Travel Lane	B	1985	16.5	B	A	1590	13.3	B
Route 140, north of Hathaway Road								
Northbound Travel Lane	B	2300	18.4	C	B	2390	19.1	C
Southbound Travel Lane	C	2465	21.9	C	C	2545	22.0	C

1 Level of service
2 Vehicles per hour
3 Passenger cars/per mile/per lane

Table 4.1-33 depicts the highway operations for 2030 under the No-Build Alternative. Based on CTPS and historical growth projections, traffic volumes were projected to 2030 No-Build conditions. The results of the 2030 highway capacity analysis indicate that the two segments of the highway analyzed are expected to continue to operate at LOS D during each peak hour.

Table 4.1-33 2030 No-Build Highway Capacity Analyses Summary–Route 138

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	2030 No-Build			Existing	2030 No-Build		
Location/Movement	LOS[1]	Volume[2]	V/C[3]	LOS	LOS	Volume	V/C	LOS
Easton, south of Route 106								
North/Southbound Travel Lane	D	1570	0.53	D	D	1750	0.60	D
Taunton, south of Bay Street								
North/Southbound Travel Lane	D	1510	0.49	D	D	1760	0.57	D

1 Level of service for Class II roadway as defined by HCM CH. 12 pp. 12-12, 12-13
2 Vehicles per hour
3 Volume to capacity ratio

Intersections (Enhanced Bus Park-and-Ride Locations)

In order to evaluate the proposed access for the bus park-and-ride locations under future conditions, intersection capacity analyses were performed at driveway locations using 2030 projected traffic volumes. For the two existing park-and-ride locations, traffic volumes for the 2030 design year were projected based on the estimated annual growth rate and by adding additional vehicle trips associated with the increased ridership projections provided by CTPS. Volumes were projected using the existing fall 2008 volumes as a base, which represent a more conservative analysis than the summer 2008 volumes.

Results for the capacity analyses of the two signalized intersections providing access to the new expanded Galleria Mall park-and-ride lot in Taunton are summarized in Table 4.1-34, which also depicts the 2030 No-Build analysis at the park-and-ride lot proposed in West Bridgewater on Route 106 and at the two lots proposed in New Bedford on Mt. Pleasant Street and Acushnet Avenue (Whale's Tooth). All three locations are unsignalized.

Galleria Mall Park-and-Ride, Taunton—In order to assess the impacts of additional ridership predicted at the new expanded Galleria Mall park-and-ride in Taunton, capacity at the nearby signalized intersections that provide access to the Mall were reviewed for the 2030 No-Build Alternative. The two intersections reviewed include Stevens Street at the Route 140 Northbound Ramps and County Street at the Galleria Mall Drive/Route 140 Southbound on-ramp. The analyses assumed no geometric or traffic control changes are proposed under future 2030 conditions. The added traffic volume from the expected increase in ridership was distributed to the two intersections based on existing travel patterns. The results of the analyses indicate that the weekday morning and weekday evening peak hour LOS at the Stevens Street at Route 140 Ramps is expected to remain at LOS B under future volume conditions. Weekday morning and weekday evening peak hour LOS is expected to remain at LOS A at the intersection of County Street at Galleria Mall Drive/Route 140 southbound ramp. The results conclude that the added traffic from the expanded park-and-ride will not impact the capacity of the intersections that provide access to the site.

Table4.1-34 2030 No-Build Intersection Capacity Analyses Summary (Park-and-Ride Locations)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	2030 No-Build			Existing	2030 No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS[1]	V/C	Delay	LOS
Galleria Park-and-Ride								
Stevens Street at Galleria Mall Drive at Route 140 Northbound Ramp	B	0.56	15	B	B	0.70	20	B
County Street at Galleria Mall Drive at Route 140 Southbound Ramp	A	0.06	3	A	A	0.54	7	A
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
W. Bridgewater Park-and-Ride								
West Center Street (Route 106) at Pleasant Street	F	Pleasant NB L/R	>50	F	F	Pleasant NB L/R	>50	F
Mt. Pleasant Street Park-and-Ride								
Mt. Pleasant Street at Park-and-Ride Drive	B	Site Dr. WB L/T	13	B	C	Site Dr. WB L/T	20	C
Whale's Tooth Park-and-Ride								
Acushnet Avenue at Whale's Tooth Park-and-Ride	N/A	Site Dr. WB L/T	11	B	N/A	Site Dr. WB L/T	11	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Route 106 Park-and-Ride, West Bridgewater—The proposed park-and-ride facility in West Bridgewater would use an existing driveway at the intersection of Pleasant Street and West Center Street (Route 106). No geometric or traffic control changes are proposed under future 2030 conditions. Weekday morning and evening peak hour levels of service on the West Center Street (Route 106) approach are expected to remain at LOS A and levels of service on the Pleasant Street approach are expected to remain at LOS F for future No-Build conditions. The analysis results are based on the assumption of random arrivals on the main street. The intersection should still function effectively because of the nearby signal at Manley Street, which will create gaps in traffic on West Center Street, allowing vehicles to exit Pleasant Street and the park-and-ride lot driveway.

Mt. Pleasant Street, New Bedford—At the intersection of Mt. Pleasant Street and the park-and-ride driveway in New Bedford, no geometric or traffic control changes are proposed under future 2030 conditions. Weekday morning and evening peak hour levels of service are expected to remain the same under future volume conditions, operating at LOS C or better. The intersection will function effectively with brief spikes in traffic exiting the park-and-ride lot when buses arrive.

Whale's Tooth, New Bedford—Access to the proposed Whale's Tooth park-and-ride facility in New Bedford will be provided via a new driveway on Acushnet Avenue. Traffic volumes on Acushnet Avenue were projected to 2030 by applying an annual growth rate. Traffic volumes entering and exiting the main park-and-ride entrance in the peak hour periods were estimated based on the lot being at full capacity and 25 percent of daily users arriving during the morning peak hour and departing during the evening peak hour. The results of the capacity analysis indicate that LOS for vehicles entering and exiting the park-and-ride will be LOS B or better during each of the peak hours.

Intersections (Rail Station Areas)

New Bedford—No-Build conditions in New Bedford were analyzed for the two station locations proposed in New Bedford. These stations would serve both the Whittenton and Stoughton Alternatives:

- Whale’s Tooth
- King’s Highway

Intersections near these station locations were analyzed for the No-Build condition. The access to an expanded park-and-ride facility on Mount Pleasant Street was also analyzed for the No-Build Alternative.

The Whale’s Tooth Station would be located east of Route 18 and north of Route 6 near the downtown and the waterfront. The 2030 No-Build traffic volume projections for the Whale’s Tooth Station area are shown in Figures 4.1-28 and 4.1-29. Table 4.1-35 provides a comparison of traffic operations between No-Build and Existing Conditions. Under No-Build conditions, there are minor or no changes in LOS projected at the signalized intersections and most of the unsignalized intersections analyzed for the Whale’s Tooth station location. One unsignalized location, Purchase Street at Route 18 SB ramp, currently operates at a LOS E during the evening peak hour and is expected to operate at the same LOS E under No-Build conditions. Three unsignalized locations are expected to decline from LOS E to LOS F during one peak hour; these include Coggeshall Street at North Front Street during the morning peak hour and Coggeshall Street at Purchase Street and Purchase Street at Weld Street during the evening peak hour.

Table 4.1-35 New Bedford Intersection Capacity Analysis–2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Whale’s Tooth Station								
Hillman St at Purchase St.	B	0.37	13	B	B	0.50	14	B
Mill St at Pleasant St.	F	0.79	>80	F	E	0.89	73	E
Union St. at Rt. 18	E	0.85	66	E	F	>1.00	>80	F
Union St at McArthur Dr.	C	0.43	29	C	D	0.44	41	D
Rt. 18 NB at Coggeshall St.	B	0.50	17	B	B	0.55	18	B
Rt. 18 SB at Coggeshall St.	C	0.86	42	D	C	0.71	27	C
Coggeshall St. at Belleville Ave.	B	0.70	20	B	B	0.71	20	B
King’s Highway Station								
King’s Hwy. at Rt. 140 NB Ramps	B	0.65	14	B	C	0.90	27	C
Rt. 18 at Wood St	C	0.57	21	C	B	0.68	17	B
Church St. at Nash Rd	B	0.58	18	B	C	0.92	31	C

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Church St. at Tarkiln Hill Rd	B	0.72	18	B	C	0.88	36	D
King's Highway at Stop & Shop driveway	A	0.48	8	A	B	0.69	13	B
King's Highway at Shaw's driveway	A	0.49	6	A	A	0.61	9	A
King's Highway at Mt. Pleasant St.	N/A	0.52	16	B	N/A	>1.00	58	E
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
Whale's Tooth Station								
Hillman St. at McArthur Dr.	B	Hillman EB L/R	11	B	B	Hillman EB L/R	13	B
McArthur Dr. at Herman Melville Blvd.	B	Melville WB L/R	15	B	C	Melville WB L/R	18	C
Coggeshall St. at Purchase St.	C	Purchase SB All	18	C	E	Purchase NB All	>50	F
Coggeshall St. at N. Front St.	E	N. Front NB All	>50	F	F	N. Front NB All	>50	F
Purchase St. at Weld St.	C	Weld WB L	24	C	E	Weld WB L	>50	F
Logan St. at Purchase St.	C	Logan WB L/R	17	C	C	Logan WB L/R	22	C
Logan St. at McArthur Dr.	B	Logan EB All	11	B	B	Logan WB All	12	B
Logan St. at N. Front St.	C	Logan EB All	23	C	C	Logan EB All	21	C
Wamsutta St. at N. Front St.	B	Wamsutta EB L/R	11	B	B	Wamsutta EB All	12	B
	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Wamsutta St. at McArthur Dr.	A	Wamsutta WB L/R	10	A	A	Wamsutta WB L/R	9	A
Purchase St. at Rt. 18 SB Exit Ramp	C	Rt. 18 WB All	26	D	E	Rt. 18 WB All	47	E
King's Highway Station								
Mt. Pleasant St. at Rt. 140 SB Ramps	F	Off-Ramp WB L	>50	F	F	Off-ramp WB L	>50	F
King's Highway at Mt. Pleasant St.	F	N/A	N/A	N/A	F	N/A	N/A	N/A
Church St. at Park Ave.	C	Park WB All	22	C	F	Park WB All	>50	F
Church St. at Irvington St	B	Irvington WB All	15	C	C	Irvington EB All	22	C
King's Highway at Tarkiln Hill Rd.	D	Tarkiln EB L/R	28	D	F	Tarkiln EB L/R	>50	F

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound
Shaded rows reflect over capacity intersections (LOS = F)

The King's Highway Station would be located off King's Highway, east of the Route 140 interchange. The 2030 No-Build traffic volume projections for the King's Highway Station area are shown in Figures 4.1-30 and 4.1-31. With one exception, there are no changes in LOS projected at any of the locations analyzed for the King's Highway station location. The unsignalized intersection of King's Highway at Mount Pleasant Street operates at LOS E and LOS F, during the morning and evening peak hours respectively, under Existing Conditions. Under No-Build conditions, the intersection is expected to be signalized and to operate at LOS B and LOS E, respectively, during the morning peak and evening peak hours.

Freetown—No-Build conditions in Freetown were analyzed for one station location proposed in Freetown. This station would serve the Whittenton, and Stoughton Alternatives. The station would be located on the east side of South Main Street south of Route 24 Exit 9 between the Stop & Shop Distribution Center and the planned entrance to the Riverfront Business Park. The Riverfront Business Park is a proposed 1.7-million square foot commercial development on the west side of South Main Street south of the Stop & Shop Distribution Center.

Under Existing Conditions, the Freetown station study area consists of six unsignalized intersections. Under No-Build conditions, the two unsignalized locations at the Route 24 Exit 9 northbound and southbound ramps are expected to be signalized as mitigation for the Payne's Crossing project. A seventh location at Payne's Crossing driveway, which would also be signalized, has been added to the No-Build analyses. On the west side of South Main Street just south of Route 24 Exit 9, the Payne's Crossing development is proposed to include:

- A 167,000 square foot home-improvement warehouse store
- A 195,000 square foot discount superstore
- 15,000 square feet of other retail space
- 1,530 parking spaces

Proposed traffic mitigation for the Payne's Crossing project includes proposed improvements at Route 24 Exit 9:

- Widening a portion of South Main Street between the Payne's Crossing driveway and the northbound ramps intersection at Exit 9.
- Installing traffic signals at the South Main Street intersections with the Route 24 northbound and southbound ramps.

The 2030 No-Build traffic volume projections for the Freetown Station are shown in Figures 4.1-32 and 4.1-33. Table 4.1-36 provides a comparison of Existing and No-Build traffic operations.

One of the signalized intersections is projected to operate at a deficient LOS under No-Build conditions. South Main Street at the Route 24 northbound ramps is projected to operate at LOS E during the evening peak hour. Under the No-Build Alternative, two of the unsignalized intersections are expected to decline to LOS F during both the morning and evening peak hours. One additional unsignalized intersection is expected to decline to LOS F during the evening peak hour because of the expected increased volume of traffic on South Main Street resulting from already planned projects in the station vicinity.

Table 4.1-36 Freetown Intersection Capacity Analysis– 2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	2030-No-Build			Existing	2030-No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS[1]	V/C	Delay	LOS
Freetown Station								
S. Main St. at Rte. 24 SB Ramps	N/A	0.59	7	A	N/A	0.62	10	B
S. Main St. at Rte. 24 NB Ramps	N/A	0.96	33	C	N/A	1.04	60	E
S. Main St. at Payne's Crossing Site Driveway	N/A	0.29	2	A	N/A	0.48	13	B
Executive Park Dr. at S. Main St.	N/A	0.81	19	B	N/A	0.83	41	D
Executive Park Dr. at Rt. 24 SB Off-Ramps	N/A	0.86	30	C	N/A	0.90	25	C
Executive Park Dr. at Rt. 24 NB Off-Ramps	N/A	0.83	15	B	N/A	0.52	8	A
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
Freetown Station								
S. Main St. at High St.	B	High NB All	>50	F	B	High NB All	>50	F
S. Main St. at Ridge Hill Rd.	E	Ridge Hill WB All	>50	F	E	Ridge Hill WB All	>50	F
S. Main St. at Rte. 24 SB Ramps	C	N/A	N/A	N/A	E	N/A	N/A	N/A
S. Main St. at Rte. 24 NB Ramps	E	N/A	N/A	N/A	E	N/A	N/A	N/A
S. Main St. at Narrows Rd.	C	Narrows EB L/R	26	D	C	Narrows EB L/R	>50	F
S. Main St. at Copicut St.	B	Copicut WB L/R	15	B	B	Copicut WB L/R	15	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All moves
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Fall River—No-Build conditions in Fall River were analyzed for two proposed station locations. These stations would serve the Whittenton and Stoughton Alternatives:

- Fall River Depot
- Battleship Cove

The existing Fall River traffic volume networks were projected to create the 2030 No-Build condition traffic volume networks, which are depicted in Figures 4.1-34 and 4.1-35. A comparison of Existing and No-Build capacity analysis results for the Fall River station study areas are shown in Table 4.1-37. The Fall River Depot station site is located 1 mile north of downtown Fall River on North Davol Street at Pearce Street. Three signalized and four unsignalized intersections were analyzed for Fall River Depot Station. All are projected to experience no change in LOS from Existing Conditions to No-Build conditions.

Table 4.1-37 Fall River Intersection Capacity Analysis– 2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	2030 No-Build			Existing	2030 No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS[1]	V/C	Delay	LOS
Fall River Depot Station								
S. Davol St. at President Ave.	C	0.67	28	C	B	0.62	20	C
N. Davol St. at President Ave.	B	0.51	20	B	B	0.66	20	C
N. Main St. at President Ave.	C	0.79	28	C	C	0.90	38	D
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
Battleship Cove Station								
Ponta Delgada Blvd. at Anawan St.	C	Anawan EB All	15	C	C	Anawan WB All	16	C
Ferry St. at Ponta Delgada	B	Ferry EB L/R	14	B	B	Ferry EB L/R	12	B
Anawan St. at Davol St.	F	Davol SB All	>50	F	F	Davol SB All	>50	F
Central St. at Davol St.	E	Central WB L	>50	F	F	Central WB L	>50	F
Fall River Depot Station								
Turner St. at N. Davol St.	B	Turner R	13	B	B	Turner R	14	B
Pearce St. at N. Davol St.	B	Pearce R	12	B	B	Pearce R	14	B
Davol St. SB to NB U-turn near Cedar St.	B	U-turn SW L	13	B	B	U-turn SW L	12	B
Davol NB to SB U-turn near Cedar St	B	U-turn NE L	14	B	B	U-turn NE L	14	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

The proposed Battleship Cove station site would be on Ponta Delgada Boulevard west of Route 138 and south of I-195 and the Fall River Heritage State Park (Battleship Cove). Four unsignalized locations were analyzed for the Battleship Cove Station and all but one are expected to experience no change in LOS. The exception is Central Street at Davol Street where the westbound Central Street approach is projected to decline from LOS E to LOS F.

Taunton—No-Build traffic conditions in Taunton were analyzed for two station locations in the City of Taunton: Taunton Depot and Taunton.

A detailed No-Build traffic assessment was not prepared for the Dana Street Station, but potential impacts were addressed qualitatively through a screening analysis using traffic data for the nearby Downtown Taunton Station analyzed in the DEIS/DEIR and the 2035 Whittenton Electric boarding estimates provided by CTPS. See Section 4.1.4.2 for further information on the methodology and results of the screening analysis for the Dana St. Station.

The Taunton Depot station location is common to both the Stoughton and Whittenton Alternatives. It is accessible from Route 140 west of the Route 24 interchange. The Taunton 2030 No-Build traffic volume projections are shown in Figures 4.1-36 and 4.1-37 (the figures were developed for the DEIS/DEIR and also show the Downtown Taunton Station that has been replaced by the Dana Street Station under the Whittenton Alternatives). Table 4.1-38 presents the traffic operations comparison between Existing and No-Build conditions.

Table 4.1-38 Taunton Intersection Capacity Analysis—2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Taunton Depot Station								
Rt. 140 at Hart St.	D	>1.00	70	E	D	>1.00	79	E
Rt. 140 at Rt. 24 SB Ramps	C	0.78	17	B	F	>1.00	61	E
Rt. 140 at Rt. 24 NB Ramps	B	0.90	7	A	A	0.70	3	A
Rt. 140 at Taunton Depot Dr.	B	0.55	14	B	B	0.61	20	B
Rt. 140 at Mozzone Blvd.	A	0.40	2	A	B	0.95	21	C
County St at Silver City Galleria Mall driveway/Rt. 140 Ramps	A	0.09	4	A	A	0.41	8	A
Stevens St. at Rt. 140 NB Ramps	B	0.46	15	B	B	0.58	18	B
Downtown Taunton Station								
Weir St/Broadway at Cohannet St	B	0.61	16	B	B	0.58	16	B
Washington St at Court St	C	0.79	27	C	D	0.88	53	D
Washington St at Tremont St	D	0.79	39	D	D	0.87	48	D
Taunton Station								
Broadway St at Washington St	C	0.75	34	C	D	0.86	47	D
Rt. 44 at Dean St./Rt. 104	A	0.76	9	A	B	0.68	11	B
Rt. 44 at Longmeadow Rd	F	1.00	>80	F	E	>1.00	78	E
Rt. 44 at Arlington St	C	0.97	43	D	D	0.99	53	D
Main St. at Union St.	C	0.92	33	C	C	0.88	30	C
Spring St at Summer St (Rt. 140)	C	0.70	26	C	C	0.80	27	C
Rt. 140 at Hon. Gordon Owen Riverway	B	0.75	16	B	C	0.95	41	D
Taunton Station								
Arlington St at School St	C	School NB All	20	C	D	School NB All	30	D
Washington St at Purchase St	C	Washington SB All	25	C	F	Washington NB All	>50	F
School St at Winter St	F	School SB All	>50	F	F	School SB All	>50	F

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Five of the seven signalized intersections analyzed for this station location are expected to operate at acceptable levels of service during both peak hours under No-Build conditions. One location is expected to operate at acceptable levels of service during the morning peak hour but at LOS E during the evening peak hour, and one location is expected to operate at LOS E during both morning and evening peak hours. Route 140 at the Route 24 southbound ramps is expected to improve from LOS F to LOS E during the evening peak hour because of the planned improvements at that location described earlier. Route 140 at Hart Street will experience an increase in delay that would cause operations to decline slightly and operate at LOS E during both peak hour periods. No unsignalized intersections were analyzed for the Taunton Depot station location.

Under the Stoughton Alternative, the Taunton Station would be located on Arlington Street just north of Route 44 (Dean Street). Six of the seven signalized intersections analyzed for the Taunton station location are expected to operate at acceptable levels of service during both peak hours. The intersection of Route 44 at Longmeadow Road is projected to remain at LOS F during the morning peak hour and LOS E during the evening peak hour under No-Build conditions. LOS at the three unsignalized intersections analyzed is not expected to change from Existing Conditions to No-Build conditions.

Stoughton—The 2030 No-Build condition traffic volumes for the Stoughton Station study area were developed by applying a background growth rate of 5 percent to the existing traffic volumes. Vehicle trips associated with the projected No-Build condition growth in ridership at the station were then added to the base, and the traffic volume networks were developed. The No-Build condition morning and evening peak hour volume networks are depicted in Figure 4.1-38 and Figure 4.1-39.

To assess the change in traffic operations, roadway capacity analyses were conducted for the No-Build condition and compared to the existing conditions. The results of the signalized and unsignalized intersection capacity analyses for each of the study area intersections are summarized in Table 4.1-39 and Table 4.1-40. Complete traffic operations analysis results are provided in Appendix 4.1-K.

Table 4.1-39 Stoughton Station Signalized Intersection Capacity Analysis– No-Build Condition vs. Existing Conditions

		Existing Conditions			No-Build Condition		
Location	Period	v/c[1]	Delay[2]	LOS[3]	v/c	Delay	LOS
Porter Street at Washington Street	Weekday Morning	0.69	21	C	0.73	22	C
	Weekday Evening	0.90	49	D	0.94	60	E
Pleasant Street at Park Street/ Washington Street	Weekday Morning	0.92	36	D	0.96	45	D
	Weekday Evening	0.79	24	C	0.83	27	C

Source: Synchro 7 (Build 773, Rev 8) software
1 volume-to-capacity ratio
2 average delay in seconds per vehicle
3 level of service

As shown in Table 4.1-39, there would be no change in level of service for the signalized intersection of Pleasant Street at Park Street/Washington Street under the No-Build condition. The intersection of Porter Street at Washington Street would continue to operate at an acceptable level of service during the morning peak hour but the level of service would decline from LOS D to LOS E during the evening peak hour.

As presented in Table 4.1-40 and Table 4.1-41, all locations operating at poor levels of service under existing conditions will continue to operate poorly in the future. Although a few of the unsignalized intersections experienced a slight increase in delay under the No-Build condition, none are projected to degrade the level of service.

Table 4.1-40 Stoughton Station Unsignalized Intersection Capacity Analysis (Morning Peak Hour)– No-Build Condition vs. Existing Conditions

Location	Critical Movement	Existing Conditions Dem[1]	v/c[2]	Del[3]	LOS[4]	No-Build Condition Dem	v/c	Del	LOS
Porter Street at Washington Street	WB RT	15	0.07	14	B	15	0.07	15	B
Freeman Street at Washington Street	WB RT	10	0.19	52	F	10	0.22	63	F
Wyman Street at Washington Street	EB RT	125	0.32	16	C	130	0.35	17	C
Morton Street/Trackside Plaza South Drive/MBTA Lot Driveway at Wyman Street	EB LT-TH-RT	290	0.09	3	A	317	0.1	3	A
	WB LT-TH-RT	65	0	1	A	69	0	1	A
	NB LT-TH-RT	Neg	0.01	14	B	Neg	0.01	14	B
	SB LT-TH-RT	10	0.04	11	B	14	0.04	11	B
Summer Street at Wyman Street	EB LT-RT	30	0.04	9	A	33	0.04	9	A
Brock Street at Washington Street	EB LT-TH-RT	120	0.62	40	E	125	0.70	50	E
	WB LT-TH-RT	50	0.32	30	D	50	0.36	34	D
	NB LT-TH-RT	410	0.14	4	A	435	0.15	4	A
	SB LT-TH-RT	345	0	0	A	365	0	0	A
Brock Street at Morton Street	EB LT-TH-RT	60	0.10	9	A	65	0.12	9	A
	WB LT-TH-RT	205	0.37	11	B	215	0.40	11	B
	NB LT-TH-RT	220	0.42	11	B	237	0.46	12	B
	SB LT-TH-RT	75	0.16	9	A	82	0.17	10	A
Brock Street at Wyman Street	WB LT-RT	95	0.13	9	A	100	0.14	10	A
Park Avenue/Sumner Street at Park Street	EB LT	205	>1.20	>120	F	215	>1.20	>120	F
	EB TH-RT	15	0.05	16	C	15	0.06	17	C
	WB LT-TH-RT	20	0.09	21	C	20	0.10	22	C

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour for unsignalized intersections
2 volume-to-capacity ratio for the critical movement, values over 1.0 indicate demand in excess of capacity.
3 Control delay per vehicle, expressed in seconds, includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.
4 level of service of the critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn
Neg = negligible

Table 4.1-41 Stoughton Station Unsignalized Intersection Capacity Analysis (Evening Peak Hour)– No-Build Condition vs. Existing Conditions

Location	Critical Movement	Existing Conditions Dem[1]	v/c[2]	Del[3]	LOS[4]	No-Build Condition Dem	v/c	Del	LOS
Porter Street at Washington Street	WB RT	25	0.08	12	B	25	0.08	13	B
Freeman Street at Washington Street	WB RT	15	0.12	29	D	15	0.14	32	D
Wyman Street at Washington Street	EB RT	125	0.42	22	C	140	0.50	26	D
Morton Street/Trackside Plaza South Drive/MBTA Lot Driveway at Wyman Street	EB LT-TH-RT	130	0.02	1	A	143	0.02	1	A
	WB LT-TH-RT	140	0	1	A	150	0	1	A
	NB LT-TH-RT	5	0.04	14	B	5	0.04	14	B
	SB LT-TH-RT	30	0.07	10	B	33	0.08	11	B
Summer Street at Wyman Street	EB LT-RT	65	0.07	9	A	70	0.08	9	A
Brock Street at Washington Street	EB LT-TH-RT	145	1.13	>120	F	155	>1.20	>120	F
	WB LT-TH-RT	70	1.08	>120	F	70	>1.20	>120	F
	NB LT-TH-RT	465	0.09	3	A	490	0.10	3	A
	SB LT-TH-RT	775	0.01	1	A	820	0.01	0	A
Brock Street at Morton Street	EB LT-TH-RT	75	0.12	9	A	80	0.13	9	A
	WB LT-TH-RT	165	0.30	10	A	170	0.31	10	B
	NB LT-TH-RT	90	0.18	9	A	97	0.19	9	A
	SB LT-TH-RT	155	0.30	10	B	165	0.32	10	B
Brock Street at Wyman Street	WB LT-RT	115	0.15	9	A	120	0.16	9	A
Park Avenue/Sumner Street at Park Street	EB LT	120	1.05	>120	F	125	>1.20	>120	F
	EB TH-RT	25	0.10	18	C	25	0.11	19	C
	WB LT-TH-RT	50	0.26	23	C	50	0.28	25	D

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour for unsignalized intersections
2 volume-to-capacity ratio for the critical movement, values over 1.0 indicate demand in excess of capacity.
3 Control delay per vehicle, expressed in seconds, includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.
4 level of service of the critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn
Neg = negligible

Easton—Traffic operations analyses under No-Build conditions were conducted at intersections near two proposed station locations in Easton:

- North Easton
- Easton Village

The North Easton station would be located west of Route 138 on the Easton-Stoughton town line. The Easton 2030 No-Build traffic volume projections are shown in Figures 4.1-40 and 4.1-41. As shown in Table 4.1-42, two signalized and two unsignalized intersections were analyzed and only one location is expected to change in LOS, from acceptable LOS D to LOS F under No-Build conditions. The two unsignalized locations are projected to continue operating at LOS F during both peak hours and the signalized intersection of Route 138 is projected to continue to operate at LOS F during the morning peak hour.

Table 4.1-42 Easton Intersection Capacity Analysis - 2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
North Easton Station								
Rt. 138 at Roche Bros. Way	B	0.75	13	B	B	0.62	15	B
Rt. 138 at Main St.	F	0.96	>80	F	D	>1.00	57	E
Easton Village Station								
Rt. 138 at Belmont St. (Rt. 123)	B	0.86	53	D	D	94	>80	F
Rt. 138 at Roosevelt Circle	N/A	0.61	6	A	N/A	0.79	18	B
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
North Easton Station								
Rte. 138 at Elm St.	F	Elm EB All	>50	F	F	Elm WB All	>50	F
Rte. 138 at Union St.	F	Union WB L/R	>50	F	F	Union WB L/R	>50	F
Easton Village Station								
Elm St. at Main St	B	Elm WB L/R	13	B	B	Elm WB L/R	15	B
Center St. at Main St. at Lincoln St.	F	Center NB All	>50	F	F	Center NB All	>50	F
Lincoln St. at Barrows St.	B	Barrows NB All	11	B	C	Barrows NB All	26	D
Rt. 138 at Roosevelt Circle	E	N/A	N/A	N/A	C	N/A	N/A	N/A

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Raynham—The Raynham Park station site would be located on the west side of Route 138 just south of the former Raynham-Taunton Greyhound Park, now the Raynham Park Simulcast Center. Traffic operations at three signalized and six unsignalized intersections were analyzed for Existing Conditions. The Raynham 2030 No-Build traffic volume projections are shown in Figures 4.1-42 and 4.1-43. As shown in Table 4.1-43 and described earlier, under No-Build conditions three of the unsignalized intersections along Route 138 are expected to be signalized, including the northbound and southbound I-495 ramps, and Center Street. All three of these intersections operate at LOS F as unsignalized intersections but are expected to operate at LOS C or better under signalization. The original three signalized intersections are expected to continue operating at acceptable levels of service. Both unsignalized Britton Street intersections with Route 138 are projected to operate at LOS F and the Wilbur Street intersection with Route 138 is expected to decline from LOS D to LOS E during both peak hours under No-Build conditions. The Raynham Park driveway is projected to decline from LOS E to LOS F during both peak hours.

Table 4.1-43 Raynham Intersection Capacity Analysis– 2030 No-Build Conditions vs. Existing Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Existing	No-Build			Existing	No-Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Raynham Park Station								
Rt. 138 at Rt. 106 (Foundry St., Easton)	B	0.88	23	C	C	>1.00	43	D
Rt. 138 at Elm St.	C	0.74	16	B	B	0.71	16	B
Rt. 138 at I-495 NB Ramps	N/A	0.68	16	B	N/A	0.82	18	B
Rt. 138 at I-495 SB Ramps	N/A	0.93	25	C	N/A	0.69	14	B
Rt. 138 at Carver St.	B	0.86	21	C	B	>1.00	42	D
Rt. 138 at Center St.	N/A	0.57	7	A	N/A	0.94	22	C
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
Raynham Park Station								
Rt. 138 at Wilbur St.	D	Wilbur L/R	39	E	D	Wilbur L/R	36	E
Rt. 138 at I-495 NB Ramps	F	N/A	N/A	N/A	F	N/A	N/A	N/A
Rt. 138 at I-495 SB Ramps	F	N/A	N/A	N/A	F	N/A	N/A	N/A
Rt. 138 at Center St.	F	N/A	N/A	N/A	F	N/A	N/A	N/A
Rt. 138 at Britton St. (East)	F	Britton WB L/R	>50	F	F	Britton WB L/R	>50	F
Rt. 138 at Britton St. (West)	E	Britton EB L/R	>50	F	F	Britton EB L/R	>50	F
Rt. 138 at Robinson St.	D	Robinson WB L/R	31	D	B	Robinson WB L/R	14	B
Rt. 138 at Dog Track Driveway	C	Driveway EB All	30	D	D	Driveway EB All	45	E

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

4.1.4.2 Build Alternatives

Regional Transportation Impacts

This section evaluates the impact on regional transportation with regard to the four key criteria identified in Section 4.1.2 and as utilized in preceding alternatives analyses with regard to achieving the project purpose. They include positive and negative impacts on the ability of the transportation system to meet projected ridership demand, the impact of an alternative on the quality of service of the transportation system as expressed in transit travel time, adherence to MBTA service delivery policy and reliability of the transportation system, impact on VMT and the impact of an alternative on regional mobility (i.e. the connectivity among transportation services). Ridership projections were developed by CTPS based on established methodologies for transportation projects. Documentation of the ridership modeling methodology is provided in Appendix 3.2-G and 3.2-H.

Ridership Demand

To conservatively determine the effects of the Build Alternatives on the regional highway network, the transit ridership projections for the No-Build (Enhanced Bus) Alternative and the Build Alternatives were modeled and compared. The No-Build ridership projections reflect the Enhanced Bus boardings and alightings. The Build Alternative ridership projections reflected both boardings and alightings for the existing regional bus and proposed commuter rail services. To determine the benefit (Build vs. No-Build), the No-Build Enhanced Bus Alternative ridership was subtracted from the Build Alternative ridership to determine the amount of additional transit ridership that the Build Alternatives are projected to attract. All boarding and alighting projections were calculated for three-hour morning and evening peak periods.

Ridership demand was evaluated to determine how well an alternative would be able to meet existing and future demand for public transportation between Fall River/New Bedford and Boston. In order to estimate overall transit demand for the region, an optimal transit system with no constraints such as construction costs or environmental impacts would have to be simulated. While this optimal transit demand has not been quantified, demand was measured in terms of the number of daily work-related trips between South Coast communities and Boston. For this screening analysis, transit demand was based on 2000 Journey-to-Work (JTW) data.

Total service to the South Coast region was considered the total station boardings as projected for each alternative in addition to boardings at existing commuter bus services, which is anticipated to continue to operate with the South Coast Rail project in place. According to the JTW data, the number of daily work trips from the South Coast region to Boston is approximately 8,000. The ability of the alternative to meet possible future ridership potential was calculated as the percent of met ridership demand.

As shown in Table 4.1-44, the rail alternatives would result in 3,930 to 4,570 daily boardings at the new stations. Private bus service boardings would decline substantially to 1,100 to 1,350 (compared to 6,000 in the 2035 No-Build condition) as a result of the diversion of passengers to the new rail option. When the rail ridership and remaining bus ridership are considered together, the alternatives meet 65.5 to 71.0 percent of the demand for approximately 8,000 work trips from the South Coast region to Boston.

Due to a faster travel time to Boston, the Stoughton Alternatives achieve greater ridership in the Southern Triangle than the Whittenton Alternatives. For example, the Stoughton Electric would have 840 daily boardings at Fall River Depot compared to 750 under the Whittenton Electric Alternative. The Whittenton Alternatives ridership is also less than the Stoughton Alternatives because the Whittenton alignment does not include the Taunton Station, which has 670 daily boardings under the Stoughton Electric Alternative. The Whittenton Alternative station closest to downtown Taunton (Dana Street) has substantially lower ridership (320 daily boardings under the electric alternative). The Whittenton Electric Alternative boardings at Raynham Park (520) would be higher than under the Stoughton Electric (430), because in the absence of Taunton Station, some riders would proceed to Raynham Park rather than Dana Street.

Table4.1-44 Daily Ridership Demand by Alternative (2035)

Name	New Station Boardings*	Boardings at Existing Commuter Bus Services	Total Service to South Coast Region	Percentage of Met Ridership Demand[1]
Stoughton Electric Alternative	4,570	1,100	5,670	70.9%
Stoughton Diesel Alternative	4,430	1,250	5,680	71.0%
Whittenton Electric Alternative	4,040	1,200	5,240	65.5%
Whittenton Diesel Alternative	3,930	1,350	5,280	66.0%

1 Total Service to South Coast region divided by the number of daily work trips from the South Coast region to Boston (approximately 8,000)
* Relocated Stoughton Station not considered “new”

The difference in ridership between the electric and diesel versions of the alternatives is small, with the diesel alternative rail ridership at new stations being approximately three percent lower than the corresponding electric alternative due to slightly longer travel times. Despite having lower rail ridership, the Stoughton Diesel Alternative has the highest total service to the South Coast Region when considered together bus service (although the difference from the electric version is negligible—10 boardings).

Quality of Service

The following two sections evaluate how well each alternative provides a transit service. It focuses on two factors: travel time and reliability. Travel time measures how quickly an alternative would be able to get a passenger from the South Coast region into Boston and reliability measures how often that service would be on time and, therefore, how dependable the service would be to the passengers who ride it. An alternative that does not improve the quality of transit services over the existing services provided in the region provides no functional benefit to the communities. Quality of service is assessed based on commuting time, reliability, comfort, convenience and safety. For the purposes using quantifiable criteria, only run time and reliability are used as subcriteria.

Travel Time—Since New Bedford/Fall River commuters currently rely on cars and private bus services, an improved quality of service would have to provide a comparable or competitive *travel time* and improved *reliability* with respect to existing commuter options during peak commuting periods. The average commuting time by car during rush hour in 2009 was 90 minutes and travel time by car is projected by CTPS to deteriorate further to 100-120 minutes under the No-Build scenario. There would be no measurable change in travel time by car under the Build Alternatives because due to the saturated nature of the corridor, any trips that shift to rail with the Build Alternatives would be replaced and would result in no change to travel time by car. Travel time for the rail alternatives was based on rail operations analysis,[12] which identified the segments of the rail corridors that would operate at top speed as well as segments where speed is constrained due to speed restrictions, geometry, vehicles, power mode, dwell times and number of stations and civil restrictions. Each commuter rail alternative has two overall run times: one for electric locomotives and one diesel locomotives. The primary factor

[12] Capacity Utilization Analyses Technical Memorandum, Systra USA, November 17, 2008.

differentiating the travel time performance of the electric vs. diesel option is the greater acceleration time for diesel trains.

Table 4.1-45 summarizes travel time provided by each alternative and shows the reduction in travel time compared to the 2035 No-Build travel time by automobile in the peak period.

Table 4.1-45 Average Travel Times by Alternative (New Bedford to South Station Peak Period)

Name	Rail Travel Time (min)	Change from 2035 Auto Travel Time (100 minutes)
Stoughton Electric Alternative	77	-23
Stoughton Diesel Alternative	82	-18
Whittenton Electric Alternative	84	-16
Whittenton Diesel Alternative	89	-11

The Stoughton Electric Alternative achieves the fastest travel times (77 minutes between New Bedford and Boston during the peak period). The Stoughton Diesel Alternative takes approximately 5 minutes longer than the electric alternative to travel the same route because of the additional time diesel locomotives need to accelerate from the stations and the lower maximum speed of the diesel trains.

The longer route, and the lower speed needed to maintain safety on the sharp curves in Taunton under the Whittenton Electric Alternative, results in a total travel time approximately seven minutes longer than the Stoughton Electric Alternative (84 minutes compared to 77 minutes). The Whittenton Diesel Alternative takes 5 minutes longer to travel from New Bedford to Boston than the Whittenton Electric Alternative and has the longest travel time of the rail alternatives.

Service Delivery Policy

While an alternative might offer many benefits for the transit system in the South Coast region, it may be an unattractive service for the communities it is designed to serve because it offers too few trips. In order to maintain acceptable service, the MBTA has established a Service Delivery Policy[13] to ensure it provides quality transit services that meet the needs of the riding public. The minimum frequency of service levels provides the guidelines by which the MBTA maintains accessibility to the transportation network within a reasonable waiting period. The minimum frequency of service standards is the minimum frequency that must be maintained in a service. Commuter Rail minimum frequencies should provide 3 trips in a peak direction during the AM and PM peak periods.[14]

The Stoughton and Whittenton Alternatives (electric and diesel variants) would all meet the minimum service delivery policy standard.

Vehicle Miles Traveled

VMT is an important gauge for an alternative's transportation system benefits. VMT measures the extent of motor vehicle operation or the total number of vehicle miles traveled within the study area on given day. This particular measure quantifies how many miles of travel would be removed from the regional roadway network by commuters who elect to travel by train or bus rather than drive. This

[13] Massachusetts Bay Transportation Authority, *Service Delivery Policy*, MBTA Board of Directors approved January 14, 2009.
[14] Between LIRR, MNRR, MBTA, and METRA, the average service provided is 2.9 peak period trains.

reduction in driving has several environmental benefits, notably, cleaner air and a reduction in greenhouse gas emissions. Fewer cars on the road also ease congestion along highway corridors. The alternative with the greatest VMT change (reduction) receives the highest score under this criterion.

Table 4.1-46 summarizes the daily reduction in VMT provided by each alternative based on updated CTPS projections for 2035 and how the alternatives score against each other with regard to meeting the project purpose to reduce VMT.

Table 4.1-46 Regional VMT Reductions by Alternative (2035, Auto and Bus Transit)

Name	VMT Reduction (daily miles)
Stoughton Electric Alternative	(-255,932)
Stoughton Diesel Alternative	(-240,348)
Whittenton Electric Alternative	(-201,232)
Whittenton Diesel Alternative	(-186,306)

The Stoughton Electric Alternative achieves the greatest reduction in daily VMT of all the alternatives, approximately 54,700 VMT per day greater than the Whittenton Electric Alternative. The Stoughton Diesel Alternative has the second greatest VMT reduction, approximately 6.1 percent less than the Stoughton Electric Alternative. With the longest travel time and lowest ridership, the Whittenton Diesel Alternative is also the least effective of the rail alternatives in reducing regional VMT, although it still provides substantial benefits (reduction of 186,306 VMT per day).

Regional Mobility

The following sections discuss the number of interregional links provided by each alternative as an indication of how well each alternative meets the project purpose to improve regional mobility. As all the alternatives provide a connection from Fall River and New Bedford to Boston, an alternative will be considered more favorable if it also enhances mobility between points within the region. An interregional link is a link that provides a one-seat ride from one municipality to another. Connections within a municipality were not counted. For instance, New Bedford, which would accommodate two stations, would provide a one-seat ride from Whale’s Tooth to King’s Highway. However, this connection was not considered an improvement to regional mobility as it is contained within New Bedford.

The Stoughton and Whittenton Alternatives are equivalent in terms of meeting the regional mobility project purpose—both alternatives provide 41 interregional links.

Table 4.1-47 highlights the interregional links provided by the Stoughton and Whittenton Alternatives.

Table 4.1-47 Interregional Links–Stoughton and Whittenton Alternatives[1]

	Boston	Westwood	Canton	Stoughton	Easton	Raynham	Taunton	Freetown	Fall River	New Bedford
Boston		X	X	X	X	X	X	X	X	X
Westwood	X		X	X	X	X	X	X	X	X
Canton	X	X		X	X	X	X	X	X	X
Stoughton	X	X	X		X	X	X	X	X	X
Easton	X	X	X	X		X	X	X	X	X
Raynham	X	X	X	X	X		X	X	X	X
Taunton	X	X	X	X	X	X		X	X	X
Freetown	X	X	X	X	X	X	X		X	
Fall River	X	X	X	X	X	X	X	X		
New Bedford	X	X	X	X	X	X	X			

1 Inter-municipal connections not included.

Impacts to Freight Operations

An improved infrastructure would improve the future of freight operations in the South Coast region. The current lines operate at a class 3 or higher, only allowing for very slow speed operations. With the infrastructure improvements that will come as part of the South Coast Rail passenger service the growth of freight operations could certainly occur if properly planned.

By far the most difficult part of future freight operations will occur in and around Weir Junction. The Massachusetts Coastal Railroad (Mass Coastal) handles this labor intensive switching in this area and has noted that this is practically a full time (5 days per week) operation, which could grow even more successful with more infrastructure improvements. In addition the track geometry here only allows for slow speed operations. The current and even future proposed freight operation splits the proposed main line (under the Stoughton Alternative). This is because two of the current three Mass Coastal customers are on the east side of the proposed main line, while the third and largest of their shippers is on the west side. This sets up conflicts between operating passenger trains and freight trains during the same period of time.

The need to somehow segregate freight and passenger operations will be critical to the success of both. Under the original design work completed in 2001 new infrastructure was proposed for this area. It consisted of new infrastructure in the Taunton area that would support freight interchanges and "run-arounds"[15] on dedicated freight tracks between Cotley and Weir Junctions. This includes an interchange track at Weir Junction, a diamond crossing to access the New England Refrigerated along with freight set-off/run-around tracks located between Hart Street and Cotley Junction. The main line freight track (currently known as the Attleboro Secondary) will exist adjacent to the MBTA main line between Weir Junction and Hart Street, continuing on to Cotley Junction and Middleborough secondary or continue down to New Bedford/Fall River at Myricks Junction.

[15] A run-round loop (or run-around loop) is a track arrangement that enables a locomotive to attach to the opposite end of the train. This process is known as "running round a train". It is commonly performed to haul wagons onto a siding, or at a terminal station to prepare for a return journey

The MBTA's proposed passenger train operation will use two tracks from a point just south of Hart Street, through and including Cotley Junction. Freight trains will operate on a dedicated freight track to the west side of the passenger tracks. Freight trains wishing to gain access onto the Middleborough Secondary will wait just north of Cotley Junction for clear operating windows to cross the proposed passenger tracks. Freight trains wishing to head south to either New Bedford or Fall River will also wait here until any passenger trains in these sections have cleared. It should be noted that while the third track's primary purpose is to store/hold freight trains, it will be designed and constructed to support passenger trains so as to maintain operational flexibility.

Cotley Junction is configured to support the direct movement of trains between Middleborough and Attleboro. A freight train coming from either Fall River or New Bedford will need to access the freight track at Cotley Junction before moving on to either Attleboro or Middleborough. This should not present a problem for the freight operations as shuttle type service makes sense from both of these points. However, it must be noted that interchanges between New Bedford and Fall River with the Middleborough line must occur via a reverse direction movement at Taunton. These maneuvers will predominantly depend upon the Cotley Freight Runaround track. It should be noted that the Cotley Freight Runaround should not be used to set off (store) freight cars or freight operations will be severely impacted.

Possible Benefits of the Future Infrastructure

Future local freight switching operations from or via Taunton must support service to three potential territories:

- the Stoughton line;
- Taunton area customers; and
- the New Bedford and Fall River branches.

Freight service to New Bedford and Fall River may operate one of three different ways:

- from Framingham or Readville via Attleboro and Cotley Junction proceeding directly to New Bedford or Fall River;
- from Middleborough, making a run-around move via the proposed Cotley Freight Runaround and then proceeding to New Bedford or Fall River; or
- via Canton Junction, proceeding directly via Taunton to New Bedford or Fall River.

Presently, the only access to the remaining active freight rail customers on the existing Stoughton Branch is via the Northeast Corridor through Canton Junction. The MBTA may or may not grant a freight carrier access to the Canton area through the proposed reconstructed line between Longmeadow Road, Taunton, and the present location of end-of-track in Stoughton. Then railcars consigned to or released by customers located on the line between Taunton and Stoughton could be set-off/switched on the proposed Interchange Track at Weir Junction. Then they would be forwarded via either Middleborough or Attleboro, thence to Beacon Park Yard or Framingham.

Daytime rail freight service on the line segment between Longmeadow Road, Taunton and Stoughton is possible, but not practical. The density of proposed passenger rail service indicates that adequate "windows" for daytime freight operations exist. However, more opportunities exist due to the future track infrastructure. These include the short sidings at Longmeadow Road, Raynham, and the longer siding at North Easton.

Nighttime will provide the freight carriers the best opportunity to complete their rail operations daily. The existing MBTA operations begin at 5:38 AM when the first westbound equipment move reaches Canton Junction from Boston and end at midnight. Proposed layover terminals at Freetown and New Bedford will eliminate these early morning and late night MBTA train movements thereby further increasing the window of opportunity for freight service.

Taunton area customers, including the Rand McNally plant and Mass Coastal's existing customers located on the portion of the New Bedford Line known as the Dean Street Industrial Track, could be served by a switcher based at Taunton. This switcher would also perform the interchange with a line-haul train at Taunton.

Freight service on the New Bedford and the Fall River branches south of Myricks Junction could be a daytime operation. Mid-day MBTA service frequency to each branch is on a 120-minute interval as proposed. This is enough time for a freight train to operate between Fall River and Freetown or Myricks on the Fall River branch. If required the train could pull into one of the proposed Controlled Passing Sidings to clear the main track for an MBTA passenger train. Likewise, this is true for a freight train to operate between Myricks and the Watuppa Branch junction point located just north of New Bedford. On the New Bedford route, since the freight operation is uniquely separated from the freight service operations between Myricks and New Bedford can occur at any time.

The potential of the rail alternatives for impacts to freight operations was investigated by exploring various operating scenarios, as described below.

- A line-haul train originates at Framingham, Massachusetts on the existing CSX Boston Line (and MBTA's Worcester Line route). The train would operate to Attleboro via Mansfield, reverse at Attleboro and proceed to Middleborough via Cotley Junction, stopping in Taunton as necessary to pick up and set-out cars for the Mass Coastal Railroad at Weir Junction. The train would deliver the rest of its cars to Middleborough. Since this train needs access from the Northeast Corridor the train must operate at night between Mansfield and Attleboro.

- A switcher and crew would be called at Middleborough every weekday morning as demand dictated and would operate to Taunton, serving any local customers en route. The train would include cars for either the New Bedford line or the Fall River branch. The train would reverse at Taunton using the Cotley Freight Runaround. When MBTA traffic permits, it would proceed to the New Bedford line or to the Fall River Branch via Myricks Junction.

- All cars collected by the trains operating as per (2) above and cars being collected by the Mass Coastal Railroad along the Middleborough main line would be brought back to an expanded Middleborough yard to be re-assembled into a nighttime line-haul train. This train could then proceed through to Framingham (or perhaps to Beacon Park Yard via the Middleborough/Braintree and the South Station Wye.

- A nighttime Mass Coastal Railroad switch engine and crew would locally deliver the cars left on the Weir Junction Interchange Track by the line-haul train as described at Step (1) above, and return all outgoing cars to the interchange track for pickup. This switch engine might, or might not, have rights to operate as far north at Stoughton. This would depend upon whether this access is negotiated with, and granted by, the MBTA and CSX.

Assuming adequate capacity of the Weir Junction Interchange Track, none of the operational changes would require storage of freight cars on the proposed Cotley Freight Runaround. Should additional capacity be needed beyond that provided by the Weir Junction Interchange Track, the excess cars could be placed on the Runaround Track for collection by the line-haul train the same night. Any daytime switching operations in Taunton would be limited to run-around moves at the Cotley Runaround Track and potentially switching the Rand McNally plant located adjacent to Route 140 near Cotley Junction. Freight customers requiring service at Taunton but lacking a private industrial siding would take deliveries at one of the existing lngell Street spurs.

As described above, feasible scenarios could be developed that would enable co-existence of freight operations and the rail alternatives without impacting freight operations. While during the construction process of the proposed rail alternatives, freight operations would be temporarily impacted, the operation of the rail alternatives would not interfere with freight operations. The permanent long-term infrastructure improvements to the rail network associated with the rail alternatives would also benefit freight operations.

Traffic Operations Analysis

Regional Freeway Benefits

As discussed in Section 4.1.2.3, regional freeway benefits were conservatively assessed based on ridership for the Rapid Bus Alternative. The regional freeway benefits of the Stoughton and Whittenton Alternatives would be greater than the results discussed below.

As shown in **Table** 4.1-48, the four freeway segments analyzed on Route 24 between I-495 and I-93/Route 128 would see an improvement in LOS in the Build condition. During the morning peak hour all four segments would see LOS in the peak northbound direction improves from LOS E to LOS D. The two segments of Route 24 south of I-93 and south of Pond Street would experience similar improvement in the southbound direction in the evening peak hour. Because of these changes, all Route 24 freeway segments from I-495 to I-93 in the Build condition will operate at LOS D or better. There would also be improvements on I-93. I-93 south of Furnace Brook Parkway would also improve in the northbound direction in the morning peak hour from LOS F to LOS E. The two segments of I-93 south of Furnace Brook Parkway and south of Route 3 would improve from LOS E to LOS D. Under the Build condition, the two segments of Route 140 that were analyzed would continue to operate at LOS C or better.

Table 4.1-48 Freeway Capacity Analyses Summary, 2030

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	Build (Rapid Bus)			No-Build	Build (Rapid Bus)		
Location/Movement	LOS[1]	Volume[2]	Density[3]	LOS	LOS	Volume	Density	LOS
I-93, south of Furnace Brook Pkwy.								
Northbound Travel Lane	F	7816	38.5	E	C	5361	24.8	C
Southbound Travel Lane	C	5156	23.8	C	E	7207	33.1	D
I-93, south of Route 3								
Northbound Travel Lane	D	5701	23.7	C	C	4791	19.5	C
Southbound Travel Lane	D	7096	30.0	D	E	7342	31.5	D
Route 24, south of I-93/128								
Northbound Travel Lane	E	4756	31.1	D	B	2786	23.5	C
Southbound Travel Lane	C	3456	30.4	D	F	5937	34.4	D
Route 24, south of Pond Street								
Northbound Travel Lane	E	5041	27.3	D	B	3356	26.3	D
Southbound Travel Lane	B	3116	25.2	C	E	5822	33.1	D
Route 24, north of Route 123								
Northbound Travel Lane	E	5106	28.3	D	C	3346	27.5	D
Southbound Travel Lane	B	2446	19.3	C	D	5207	26.9	D
Route 24, north of I-495								
Northbound Travel Lane	E	4988	27.2	D	C	3635	19.7	C
Southbound Travel Lane	B	2847	16.3	B	D	4484	24.9	C
Route 24, north of Route 44								
Northbound Travel Lane	D	4183	21.5	C	C	3620	19.2	C
Southbound Travel Lane	B	3267	17.2	B	D	4199	21.8	C
Route 24, north of Route 140								
Northbound Travel Lane	D	4387	22.5	C	C	3692	19.6	C
Southbound Travel Lane	C	3485	18.2	C	D	4639	24.1	C
Route 24, south of Route 140								
Northbound Travel Lane	D	3509	31.9	D	E	3535	35.1	E
Southbound Travel Lane	D	3267	30.4	D	E	3660	35.3	E
Route 24, north of Exit 9								
Northbound Travel Lane	B	2449	13.4	B	E	3645	37.4	E
Southbound Travel Lane	D	3442	32.6	D	D	3140	28.6	D
Route 24, south of Exit 8 ½								
Northbound Travel Lane	F	4846	>45	F	C	2573	21.5	C
Southbound Travel Lane	C	2728	24.5	C	F	5496	>45	F
Route 140, south of Route 24								
Eastbound Travel Lane	B	1289	11.2	B	B	1616	13.2	B
Westbound Travel Lane	B	1400	11.6	B	B	1540	12.9	B
Route 140, north of Hathaway Road								
Northbound Travel Lane	C	1715	13.8	B	C	2340	18.7	C
Southbound Travel Lane	C	2434	21.7	C	C	2011	17.4	B

1 Level of service
2 Vehicles per hour
3 Passenger cars/per hour/per lane

Traffic Impacts Associated with Grade Crossings

This section provides an evaluation of the transportation impacts associated with the public grade crossings that would be in service along the South Coast Rail project alternatives. Figures 4.1-44 through 4.1-53 present all of the crossing locations for each rail corridor with each crossing's recommended treatment (grade separation, closure, or at-grade crossing). The figures also show the grade crossings in relation to primary emergency vehicle routes, emergency response service providers, and schools. A preliminary assessment of the rail corridors identified 52 existing active public grade crossings. Along the Fall River Secondary (common to all alternatives), four public crossings would be recommended for closure. The Stoughton Alternative would result in 43 active public grade crossings, and the Whittenton Alternative would result in 50 active public grade crossings. Transportation impacts at the proposed public grade crossings were assessed. Based on the traffic and safety analysis conducted, it is recommended that each location would be suitable for public use equipped with a combination of new, state of the art, Automatic Highway Crossing Warning (AHCW) systems and minor geometric modifications such as driveway reconfiguration, driveway closures, vegetation clearing and utility pole relocations. The delay and queue technical analysis for all locations can be found in Appendix 4.1-J.

Southern Triangle Grade Crossings Impacts (Common To All Rail Alternatives)

The majority of grade crossings in the Southern Triangle are projected to be closed only three to four times an hour, or approximately five to seven percent of the peak hour as a result of the introduction of commuter rail service. The Taunton grade crossings would be closed six times an hour, or ten percent of the peak hour. A description is provided below of the effects on traffic conditions at grade crossings in the Southern Triangle resulting from all rail alternatives.

New Bedford Grade Crossings (3) (all Rail Alternatives)—Three grade crossings in New Bedford currently carry active freight traffic and would be upgraded to accommodate the proposed commuter rail service.

- **Samuel Barnet Boulevard**. Samuel Barnet Boulevard serves mainly industrial park-related traffic and the minor queuing anticipated would not affect the traffic operations of these driveways.

Table 4.1-49 shows the traffic volumes and average delay expected along Tarkiln Road and Nash Road where more substantial queuing impacts may occur. An overview of the conditions at both roads is provided below.

Table 4.1-49 New Bedford Grade Crossings–Traffic Volumes[1] and Average Delay (All Rail Alternatives)

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Tarkiln Hill Road	34,000	815	1285	550	51
Nash Road	14,900	510	745	325	42

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Tarkiln Hill Road**. On Tarkiln Hill Road, a calculated queue length of 550 feet and average delays of 51 seconds are projected during peak periods. Existing vehicle queues on the Tarkiln Hill Road eastbound approach to Church Street extend over the grade crossing and beyond the intersection of King's Highway at Stop & Shop. The existing vehicle queues currently impact traffic at two unsignalized intersections (Tarkiln Hill Road at King's Highway and Tarkiln Hill Road at Worcester Street/Park Avenue) as well. Grade separation was considered at this location but cannot be achieved due to both horizontal and vertical curvature constraints and the crossing's proximity to the proposed King's Highway Station platform. Tarkiln Hill Road is proposed to be closed north of its intersection with King's Highway. Traffic along Tarkiln Hill Road would be rerouted through the Stop & Shop driveway intersection. As part of the proposed project, traffic signal preemption is recommended at the intersections of King's Highway and Stop & Shop driveway and Tarkiln Hill Road at Church Street to clear vehicle queuing that extends over the tracks when a train is approaching. Since queues from the adjacent intersections are projected to extend to or over the track location, the need for pre-signals at this grade-crossing, to prevent vehicles from queuing back to the grade crossing during the pre-emption period, will be evaluated as part of the preliminary design phase of the project.

- **Nash Road**. On Nash Road, a calculated queue length of 325 feet and average delays of 42 seconds are projected during peak periods. Existing vehicle queues on the Nash Road westbound approach to Church Street back up over the grade crossing. The vehicle queues could affect traffic at the unsignalized intersection of Nash Road and King Street and at driveways within 325 feet of the crossing. As part of the proposed project, traffic signal preemption is recommended at the intersection of Nash Road and Church Street to clear vehicle queues that extend over the tracks when a train is approaching. Since projected queues from the adjacent intersections are projected to extend to or over the track location, the need for pre-signals at the Nash Road eastbound approach to the grade-crossing will be evaluated as part of the preliminary design phase of the project.

Fall River Grade Crossings (None)—There are no at-grade crossings in Fall River. All major grade crossings within Fall River are grade-separated and all remaining private roadways crossings are expected to be closed.

Freetown Grade Crossings (11) (All Rail Alternatives)—Eleven existing public grade crossings in Freetown currently carry active freight traffic and would be upgraded to accommodate the proposed commuter rail trains. Seven of these crossings are expected to cause minor delays and have little impact on the surrounding roadways.

- **Chace Road**. On Chace Road, the maximum queue lengths and average delays are expected to be minimal. The sand and gravel operation driveway and the residential driveway could be affected by the vehicle queues at the crossing; however, delays are expected to be minimal. The existing driveway on the west side of the crossing may need to be reconfigured or closed.

- **Braley Road**. The maximum queue lengths and average delays at Braley Road are expected to be minimal. The driveway located about 75 feet west of the tracks on the north side of the road is expected to be affected by vehicles queued at the crossing; however, delays would be minimal. On East Chipaway Road, the maximum queue lengths and average delays are expected to be moderate. The residential driveway located approximately 20 feet east of the tracks may be affected by vehicles stopped at the crossing; however, delays would be minimal.

- **Elm Street**. The maximum queue lengths and average delays along Elm Street are expected to be minimal. The driveways located 50 feet west and 120 feet east of the tracks would be impacted by the vehicles queued at the grade crossing; however, delays would be minimal.

- **High Street**. The maximum queue lengths and average delays at High Street are expected to be minimal. The residential driveway located on the east side of the tracks and Alexandra Drive on the west side of the tracks may be impacted due to the anticipated queued vehicles at the grade crossing; however, delays would be minimal.

- **Copicut Road**. On Copicut Road, the maximum queue lengths and average delays are expected to be minimal. The dirt driveway immediately east of the tracks may be impacted by vehicle queues; however the driveway serves very few vehicles and motorists would not likely be affected.

- **Beachwood Road**. The crossing along Beachwood Road is located approximately 150 feet east of the intersection of Route 79 at Beachwood Road. The safety implications of this proximate crossing require the Beachwood Road crossing to be closed and a cul-de-sac would be constructed on the east side of the tracks. Residential traffic destined to Route 79 would divert to Malbone Road. Since there is only one home on Beachwood Road, impacts of additional traffic on Malbone Road should be minimal.

- **Richmond Road/Route 79 (North)**. Richmond Road/Route 79 (North) is expected to have minimal queue lengths and average delays. The residential driveways located on both sides of the tracks would be slightly affected by the vehicles queued at this crossing.

Table 4.1-50 shows the traffic volumes and average delay at the remaining three grade crossings, which are expected to experience the most substantive delay.

Table 4.1-50 Freetown Grade Crossings—Traffic Volumes[1] and Average Delay (All Rail Alternatives)

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Forge Road (North)	1,200	80	80	50	31
Richmond Road (South)	4,900	215	200	100	34
Forge Road (South)	3,400	205	175	100	33

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Forge Road (North)**. The Forge Road (North) crossing occurs immediately north of the intersection of Richmond Road and Forge Road. The safety implications of this proximate crossing require the Forge Road (North) crossing to be closed and a cul-de-sac would be constructed on the west side of the tracks just west of the existing stream. Residential traffic currently using Forge Road to access Richmond Road would be diverted to Locust Street. Since Forge Road is a small residential street serving about 25 homes, traffic impacts on Locust Street due to this diversion are expected to be minimal. The southern leg of the

Richmond Road/Forge Road intersection would remain open to traffic. Queuing impacts are not expected along this section of Forge Road.

- **Richmond Road/Route 79 (South)**. On Richmond Road/Route 79 (South), a calculated queue length of 100 feet and average delays of 34 seconds are projected during peak periods. The residential driveway west of the tracks would be affected by any vehicles queued at the crossing and may need to be reconfigured to ensure vehicles exiting the driveway will be adequately protected by the proposed crossing signalized gate.

- **Forge Road (South)**. On Forge Road (South) a calculated queue length of 100 feet and average delays of 33 seconds are projected during peak periods. There may be impacts to driveways on both sides of the crossing due to the anticipated queued vehicles at the grade crossing.

Lakeville Grade Crossing (1) (All Rail Alternatives)—

- **Malbone Street**. The only public grade crossing in Lakeville, Malbone Street, currently carries active freight traffic and would be upgraded to accommodate the proposed commuter rail trains. The maximum queue lengths and average delays at this location are expected to be minimal.

Berkley Grade Crossings (5) (All Rail Alternatives)—All five grade crossings in Berkley (Cotley Street, Padelford Street, Myricks Street (Route 79), Mill Street, and Adams Lane) currently carry active freight traffic. Crossings at these locations would be upgraded to accommodate the proposed commuter rail trains.

- **Cotley Street and Padelford Street**. On Cotley Street and Padelford Street, the maximum queue lengths and average delays are expected to be minimal and there would be no impacts to driveways or intersections due to this grade crossing.

- **Myricks Street (Route 79)**. On Myricks Street (Route 79), maximum queue lengths and average delays are also expected to be minimal. Left turns from Grove Street could be affected by vehicles queued at the crossing. Grove Street could be delineated to accommodate separate left and right turn lanes to mitigate any delays. Vehicle queues at this crossing would also impact driveways on the west side of the crossing. Gates and locks are proposed to access the utility road on the northwest corner of the crossing.

- **Mill Street and Adams Lane (private)**. The Mill Street and Adams Lane private crossings are proposed to be closed.

Taunton Grade Crossings (2) (All Rail Alternatives)—Two public grade crossings on the New Bedford Main Line corridor are located in Taunton. Both the Ingell Street and Hart Street crossings currently are active crossings with freight train activity. These crossings would be upgraded to accommodate the proposed commuter rail trains. Table 4.1-51 shows the traffic volumes and average delay at these grade crossings.

Table 4.1-51 Taunton Grade Crossings—Traffic Volumes[1] and Average Delay

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Ingell Street	7,500	435	460	200	38
Hart Street	13,000	460	430	200	38

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Ingell Street**. Calculated queue lengths of 200 feet and average delays of 38 seconds are projected at Ingell Street during peak periods. Vehicle queues at this crossing will affect driveways on both sides of the crossing. The driveway immediately to the west of the crossing is proposed to be closed. There are no anticipated impacts to any intersections due to queued vehicles at the grade crossing.

- **Hart Street**. On Hart Street, a calculated queue length of 200 feet and average delays of 38 seconds are projected during peak periods. Alegi Avenue and driveways located within 250 feet of the tracks would be impacted by minor delays associated with the anticipated queues at the grade crossing.

Stoughton Alternatives Grade Crossing Impacts

The Stoughton Alternatives will require gates at grade crossings within Taunton, Raynham, Easton, Stoughton and Canton to be closed approximately six times an hour, or approximately 10 percent of the peak hour.

Taunton Grade Crossings (2) – Stoughton Alternative—Two public grade crossings associated with the Stoughton Alternative are located in Taunton. One grade crossing would be reactivated as part of the Stoughton Alternative (Longmeadow Road). The other grade crossing, Dean Street (Route 44) is currently active for freight rail only with frequencies of a few times a week. As described in Section 4.1.3.4, between Weir Junction and Winter Street in Taunton, existing train frequency is approximately two roundtrip freight trains (four total trips) per month. Train frequency near Ingell Street at Weir Junction ranges from three to five roundtrip freight trains (six to ten total trains) per week. There is no existing train frequency along the unused rail alignment from Stoughton station to Winter Street in Taunton. The Dean Street crossing would be upgraded to accommodate the proposed commuter rail trains. Table 4.1-52 shows the traffic volumes and average delay at both grade crossings. The Thrasher Street crossing is currently grade separated and is therefore not discussed in this section.

Table 4.1-52 Taunton Grade Crossings—Traffic Volumes[1] and Average Delay Stoughton Alternatives

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Winter Street/Longmeadow Road	13,300	510	635	275	41
Dean Street	33,500	910	880	875	109

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Longmeadow Road**. On Longmeadow Road, a calculated queue length of 275 feet and average delays of 41 seconds are projected during peak periods, and may affect the commercial driveways on both sides of the crossing. Existing driveways and parking areas immediately adjacent to the crossing would be reconfigured and/or closed.

- **Dean Street**. On Dean Street, a calculated queue length of 875 feet and average delays of 109 seconds are projected during peak periods, which may affect the driveways on both sides of the crossing and traffic operations at the adjacent Arlington Street intersection. This active grade crossing currently experiences similar (albeit infrequent) delays when freight trains service the various industrial uses in Taunton. As part of the proposed project, new traffic signal equipment and preemption phasing is recommended at the intersection of Dean Street and Arlington Street. The new signal layout will be coordinated with the AHCW system and preemption installed to adequately clear the vehicles queuing onto the tracks when a train is approaching. The intersection may also need to be reconfigured to safely direct pedestrians to the appropriate route.

Raynham Grade Crossings (6)—Six public grade crossings are located in Raynham. Five of the crossings are currently inactive and would be reactivated as part of the Stoughton Alternatives. The sixth grade crossing, across Broadway (Route 138), is projected to have relatively high traffic volumes (27,400 vehicles per day and 1,415 and 1,425 vehicles during the AM and PM peak), which would result in relatively long queues (700 feet) and delays (63 seconds). These queues and delays could affect Center Street and Britton Street traffic as well as numerous driveways in the proximity of the crossing. This public grade crossing would therefore be converted to a grade-separated crossing to minimize traffic impacts along this section of Route 138. Table 4.1-53 shows the traffic volumes and average delay at the five remaining grade crossings.

Table 4.1-53 Raynham Grade Crossings—Traffic Volumes[1] and Average Delay Stoughton Alternative

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Elm Street	1,900	100	65	50	32
Carver Street	6,800	335	385	175	37
Britton Street	1,300	65	65	50	31
King Phillip Street	4,100	295	350	150	36
East Britannia Street	4,700	335	415	175	37

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Elm Street**. On Elm Street, a calculated queue length of 50 feet and average delays of 32 seconds are projected during peak periods and could affect a residential driveway located 35 feet to the west of the crossing.

- **Carver Street**. A calculated queue length of 175 feet and average delays of 37 seconds are projected on Carver Street during peak periods and could affect a residential driveway located 100 feet west of the crossing. There is a culvert that may need to be reconstructed in proximity to this crossing.

- **Britton Street**. On Britton Street, a calculated queue length of 50 feet and average delays of 31 seconds are projected during peak periods, and may affect the residential driveways on both sides of the crossing.

- **King Phillip Street**. A calculated queue length of 150 feet and average delays of 36 seconds are projected at King Phillip Street during peak periods, and may affect residential driveways on both sides of the crossing. The driveway located adjacent to the tracks is currently within the railroad right-of-way for approximately 300 feet connecting with a property set back from King Phillip Street, and would need to be relocated outside of the railroad right-of-way to accommodate the proposed alignment.

- **East Britannia Street**. On East Britannia Street, calculated queue lengths of 175 feet and average delays of 37 seconds are projected during peak periods. Driveways and intersections along East Britannia Street are not expected to realize impacts due to the crossing.

Easton Grade Crossings (7) - Stoughton Alternatives—Seven public grade crossings are located in Easton. All of the crossings in Easton would be reactivated as part of the Stoughton Alternatives. Table 4.1-54 shows the traffic volumes and average delay at these grade crossings. The Main Street crossing is currently grade separated and is therefore not discussed in this section.

Table 4.1-54 Easton Grade Crossings—Traffic Volumes[1] and Average Delay Stoughton Alternatives

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Elm Street	5,000	175	295	125	35
Oliver Street	1,100	80	100	25	75
Short Street	4,800	220	240	100	34
Depot Street (Route 123)	19,700	1,085	885	475	48
Purchase Street	2,500	105	140	75	32
Prospect Street	2,200	90	120	75	32
Foundry Street (Route 106)	12,800	570	635	275	41

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Elm Street**. On Elm Street, a calculated queue length of 125 feet and average delay of 35 seconds are projected during the peak periods and could affect traffic operations at driveways near the crossing. Of particular concern is the driveway to the office/industrial building on the east side of the crossing. This driveway would be reconfigured.

- **Oliver Street**. On Oliver Street, a calculated queue length of 25 feet and average delay of 75 seconds during peak periods may affect driveways near the crossing. Of particular concern is the driveway to the office/industrial building on the northwest side of the crossing, which is adjacent to a play area. This driveway is within the railroad right-of-way; the driveway would be reconfigured and the play area (which is part of day care operation) would be relocated to safe location. The sidewalk would be extended through the crossing.

- **Short Street**. A calculated queue length of 100 feet and average delay of 34 seconds during peak periods on Short Street may affect the driveways immediately on either side of the crossing.

- **Depot Street (Route 123)**. On Depot Street (Route 123), a calculated queue length of 475 feet and average delays of 48 seconds during peak periods may affect the commercial and residential driveways immediately on either side of the crossing. The driveway immediately to the west of the crossing may need to be reconfigured.

- **Purchase Street**. A calculated queue length of 75 feet and average delay of 32 seconds at Purchase Street during the peak periods are considered to be minimal. However, the queue during peak periods may affect driveways and Granite Lane immediately adjacent to the crossing.

- **Prospect Street**. A calculated queue length of 75 feet and average delay of 32 seconds at Prospect Street during the peak periods are considered to be minimal. However, the queue may affect driveways immediately adjacent to the crossing.

- **Foundry Street**. On Foundry Street, the projected queue length of 275 feet and average delays of 41 seconds during peak periods may affect a residential driveway located 100 feet to the east.

- **Easton DPW driveway (private) and Gary Lane (private)**. On the Easton DPW driveway and Gary Lane (both private ways), the maximum queue lengths and average delays at the location are expected to be minimal. Gates and locks are being proposed for these locations. This location is not a public crossing.

Stoughton Grade Crossings (8) - Stoughton Alternatives—Eight public grade crossings are located in Stoughton and would be affected by the Stoughton and Whittenton Alternatives. Five of these grade crossings (Central Street, Simpson Street, School Street, Porter Street, and Wyman Street) are currently active rail crossings carrying commuter rail that would be modified to allow double-track operations. The addition of a second track and additional trains would result in negligible changes in traffic conditions or queue lengths at these crossings. A sixth crossing, at Brock Street crossing is considered active and has working signals but is rarely used today; therefore, for the purposes of this analysis, Brock Street is considered a reactivated crossing. Table 4.1-55 shows the traffic volumes and average delay at grade crossings in Stoughton that would be reactivated as part of the proposed project.

Table 4.1-55 Stoughton Grade Crossings—Traffic Volumes[1] and Average Delay Stoughton Alternatives

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Brock Street	3,260	440	810	750	105
Plain Street	8,000	370	510	225	39
Morton Street	1,700	125	180	100	33

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Brock Street**. On southbound Brock Street, a calculated queue length of 750 feet and average delay of 105 seconds are projected during the evening peak hour and would affect traffic operations at the intersection of Washington Street and Brock Street. Table 4.1-55 shows the traffic volume and average delay expected at the Brock Street grade crossing under the Build Condition.

- **Plain Street**. On Plain Street, the calculated queue length of 225 feet and average delay of 39 seconds may impact traffic operations at the intersection of Washington Street and Plain Street. These impacts are similar to those that may be realized at Brock Street, including impacts to driveways. Further study of the benefits of signalizing this intersection is also required and should be incorporated into a study with the Brock Street intersection.

- **Morton Street**. On Morton Street, a calculated queue length of 100 feet and an average delay of 33 sections would impact operations at the intersection of Washington Street and Morton Street. The grade crossing would be located approximately 25 feet west of this unsignalized intersection. When the crossing gates are down there would be insufficient storage distance for vehicles turning onto Morton Street from Washington Street. Private driveways immediately south of Morton Street would also experience the same difficulties. Additionally, the steep grade of Morton Street may pose a safety hazard in wet or snowy weather. To mitigate these concerns, Morton Street and the private driveways to the south would be closed and a bypass roadway constructed to the private grade separated crossing on Totham Farm Road. This concept would be further studied to evaluate the traffic impacts of these closures and the potential of rerouting traffic to Plain Street.

Canton Grade Crossings (3) - Stoughton Alternatives—Three crossings studied in Canton (Washington Street, Pine Street, and Will Drive) are located along the active commuter rail line. The construction of a second track along this section of the alignment and increased train activity would not result in substantial changes in traffic conditions or queue lengths at these crossings. As part of the proposed project, traffic signal preemption is recommended at the intersection of Washington Street and Revere Street to address queuing that may extend over the tracks during the peak hours.

Whittenton Alternatives Grade Crossing Impacts

Taunton Grade Crossings (12) – Whittenton Alternatives (12)—Twelve public grade crossings associated with the Whittenton Alternative are located in Taunton. This includes ten existing grade crossings along the existing, active Attleboro Secondary. The remaining two grade crossings consist of the reactivation of two inactive grade crossings at Whittenton Street and Warren Street. Table 4.1-56 shows the traffic volumes and average delay at these grade crossings. The Bay Street crossing is currently grade separated and is therefore not discussed in this section.

- **Whittenton Street**. A calculated queue length of 100 feet and average delays of 34 seconds are projected at Whittenton Street during peak periods, and may affect the commercial driveways on both sides of the crossing.

- **Warren Street**. Although traffic volume data was unavailable, Warren Street traffic volumes are anticipated to be low as a minor residential roadway. The maximum queue lengths and average delays are expected to be minimal at the Warren Street grade crossing location.

- **Danforth Street**. On Danforth Street, the maximum queue lengths and average delays are expected to be minimal and the projected queue of 125 feet would not impact any driveways or the Grosvenor Street or Perry Avenue intersections.

Table 4.1-56 Taunton Grade Crossings—Traffic Volumes[1] and Average Delay Whittenton Alternatives

Crossing	Traffic Volume (vpd)	AM Peak Volume	PM Peak Volume	Queue Length (feet)	Average Delay (seconds)
Whittenton Street	3,300	120	225	100	34
Warren Street	N/A	N/A	N/A	-	-
West Britannia St.	4,900	288	309	150	35
Danforth St.	4,045	213	272	125	35
Tremont St.	16,505	666	798	350	43
Oak St.	12,245	763	548	800	107
Porter St.	3,195	149	197	100	39
Cohannet St.	2,025	138	224	100	34
Winthrop St.	17,360	800	812	350	44
Harrison Ave.	2,025	163	124	75	33
Somerset Ave.	8,625	434	483	225	38
Weir St.	13,815	613	666	350	48

Source: MassDOT Highway Division supplemented by counts.
1 2030 Build Condition

- **Tremont Street**. The railroad corridor intersects Tremont Street at a skewed angle in a congested urban area with a number of business and residential driveways. This active grade crossing experiences similar (albeit infrequent) delays when freight trains service the various industrial uses in Taunton. The calculated queue length of 350 feet and average delays of 43 seconds are projected during peak periods, which may affect the driveways on both sides of the crossing and traffic operations at the adjacent Granite Street intersection. One driveway on the southbound approach would be reconfigured to access Tremont Street from the adjacent driveway curb cut.

- **Oak Street**. Located adjacent to the proposed Downtown Taunton Station and platform, the Oak Street crossing would have longer queues and delay due to the extended gate closure interval. A calculated 800 foot queue and 107 seconds of delay are projected during peak periods. The nearby traffic signal at the Oak Street and Tremont Street intersection has existing pre-emption for the tracks with an advance traffic signal mast arm located just west of the tracks to prevent queuing across the tracks. The South Coast Rail project would optimize the pre-emption settings for the Oak Street and Tremont Street intersection.

- **Porter Street**. With 39 seconds of delay and queue lengths of 100 feet or less, impacts are projected to be minimal at the Porter Street crossing. The projected queues may affect one or two residential driveways on either side of each crossing. Proposed grade crossing signal equipment locations will require the modification of one driveway. Guardrail is proposed along the railroad right-of-way to limit vehicular access from the abutting business.

- **Cohannet Street**. On Cohannet Street, the maximum queue lengths and average delays are expected to be minimal. However, the proposed grade crossing signal equipment locations would require reconfiguration of two driveways immediately on either side of the tracks and the closure of the driveway in the northwest quadrant.

- **Winthrop Street**. The Winthrop Street crossing is located between a small shopping center to the east and a residential area to the west. A calculated queue length of 350 feet and average delays of 44 seconds are projected during peak periods. Walnut Street, Harrison Street and driveways located within 350 feet of the tracks would be impacted by minor delays associated with the anticipated queues at the grade crossing. Supplemental advance railroad crossing signs are suggested for both Winthrop Street approaches due to sight distance restrictions to the east (horizontal alignment) and the west (vertical alignment).

- **Harrison Avenue**. On Harrison Avenue, the maximum queue lengths and average delays are expected to be minimal and the projected queue of 75 feet would only have minor impacts to a residential driveway and Walnut Street.

- **Somerset Avenue**. On Somerset Avenue, a calculated queue length of 225 feet and average delays of 38 seconds are projected during peak periods. East Walnut Street, Barnum Street and driveways located within 225 feet of the tracks may be impacted by minor delays associated with the anticipated queues at the grade crossing. The signalized intersection of Weir Street and Somerset Avenue is located approximately 430 feet to the north of the grade crossing. If the Whittenton Alternative is determined to be the LEDPA, intersection operations and queues should be evaluated to determine if signal pre-emption is required.

- **Weir Street**. On Weir Street, a calculated queue length of 350 feet and average delay of 48 seconds are projected during the peak periods and could affect traffic operations at driveways near the crossing and the intersections at White Street, Sumner Street and McSoley Avenue. Of particular concern is the proximity of the McSoley Street to the Weir Street crossing. McSoley Street intersects Weir Street within the active grade crossing area and therefore is proposed to be closed and traffic diverted to a new outlet to Weir Street. In addition, the driveway to the residence at the corner of Weir Street and White Street would be relocated from Weir Street to White Street. The driveway serving the property in the southeast quadrant would also be reconfigured.

Grade Crossing Incident Analysis

Table 4.1-57 summarizes the probability of an incident (regardless of the severity) occurring over the span of a year at each of the proposed at-grade crossings along the Stoughton Electric Alternative alignment as well as the probability of an incident occurring at each of the at-grade crossings that currently contain rail operations.

Table 4.1-57 Stoughton Electric Alternative Incident Predictor

Town/City	Street	Existing Probability. of an Incident/Year	Proposed Probability of an Incident/Year
Canton	Washington Street	7.9%	9.2%
	Pine Street	2.6%	2.9%
	Will Drive	2.2%	2.6%
Stoughton	Central Street	3.4%	4.1%
	Simpson Street	2.2%	2.6%
	School Street	2.7%	3.4%
	Porter Street	3.0%	3.5%
	Wyman Street	2.4%	2.9%
	Brock Street	2.4%	2.9%
	Plain Street	N/A	3.4%
Easton	Elm Street	N/A	4.0%
	Oliver Street	N/A	2.9%
	Gary Lane	N/A	3.6%
	Short Street	N/A	4.1%
	Depot Street	N/A	6.5%
	Purchase Street	N/A	3.6%
	Prospect Street	N/A	3.6%
	Foundry Street	N/A	6.0%
Raynham	Greyhound Park	N/A	0.4%
	Elm Street	N/A	4.0%
	Carver Street	N/A	5.7%
	Britton Street	N/A	3.3%
	King Phillip Street	N/A	4.0%
	East Britannia Street	N/A	4.4%
Taunton	Longmeadow Road	N/A	5.7%
	Dean Street – Route 44	1.3%	7.4%
	Ingell Street	8.9%	4.5%
	Pratt Street	0.8%	3.8%
Berkley	Cotley Street	0.3%	1.7%
	Padelford Street	0.5%	2.6%
	Myricks Street (Route 79)	0.6%	3.7%
Lakeville	Malbone Street	0.4%	2.4%
Freetown	Chace Road	0.4%	0.0%
	Braley Road	0.4%	4.0%
	East Chipaway Road	0.4%	3.8%
	Richmond Road - North	0.4%	4.0%
	Richmond Road - South	0.4%	4.0%
	Forge Road - South	0.4%	2.6%
	Elm Street	0.4%	2.8%
	High Street	0.3%	2.0%

Town/City	Street	Existing Probability. of an Incident/Year	Proposed Probability of an Incident/Year
	Copicut Road	0.2%	2.4%
	Brightman Lumber	0.1%	0.5%
New Bedford	Samuel Barnet Road	0.5%	2.9%
	Pig Farm Road	0.1%	4.0%
	Tarkiln Hill Road	0.5%	4.1%
	Nash Road	0.7%	4.0%

NA – Not Active

- *Canton* – Washington Street has the highest probability at 9.2 percent. This would be approximately one incident every 11 years.
- *Stoughton* – Central Street has the highest probability at 4.1 percent. This would be approximately one incident every 24 years.
- *Easton* – Depot Street has the highest probability at 6.5 percent. This would be approximately one incident every 15 years.
- *Raynham* – Carver Street has the highest probability at 5.7 percent. This would be approximately one incident every 18 years.
- *Taunton* – Dean Street (Route 44) has the highest probability at 7.4 percent. This would be approximately one incident every 14 years.
- *Berkley* – Myricks Street (Route 79) has the highest probability at 3.7 percent. This would be approximately one incident every 27 years.
- *Lakeville* – Malbone Street has the highest probability at 2.4 percent. This would be approximately one incident every 42 years.
- *Freetown* –Braley Road and Richmond Road have the highest probabilities at 4.0 percent. This would be approximately one incident every 25 years.
- *New* Bedford –Tarkiln Hill Road has the highest probability at 4.1 percent. This would be approximately one incident every 24 years.
- *Taunton* – West Britannia Street has the highest probability of future incidents at 4.1 percent. This would be approximately one incident every 25 years.

Table 4.1-58 summarizes the probability of an incident occurring over the span of a year at each of the proposed at-grade crossings along the Attleboro Secondary and Whittenton Branch portion of the Whittenton Electric Alternative alignment. This is the only portion of the Whittenton Alternatives alignment that differs from the Stoughton Alternatives. Incident probabilities along the shared portions of the alignment would be the same under the Whittenton Alternatives as listed in Table 4.1-57 for the Stoughton Alternatives.

Table 4.1-58 Whittenton Electric Incident Predictor, Attleboro Secondary and Whittenton Branch

Town/City	Street	Existing Probability of an Incident/Year	Proposed Probability of an Incident/Year
Taunton	Whittenton Street	0.0%	0.4%
	Warren Street	0.0%	0.4%
	West Britannia Street	0.7%	4.1%
	Danforth Street	0.7%	2.6%
	Tremont Street	1.0%	3.5%
	Oak Street	1.0%	3.5%
	Porter Street	0.7%	2.6%
	Cohannet Street	0.6%	2.6%
	Winthrop Street	1.0%	3.7%
	Harrison Avenue	0.6%	2.6%
	Somerset Avenue	0.8%	3.5%
	Weir Street	0.8%	3.5%

NA – Not Active

Along the Attleboro Secondary and Whittenton Branch portion of the Whittenton Alternatives, West Brittania Street would have the highest future incident probability at 4.1 percent. This would be equivalent to approximately one incident every 25 years. Danforth Street, Porter Street, Cohannet Street, and Harrison Avenue have the lowest future probability at 2.6 percent. This would be equivalent to approximately one incident every 39 years. The average probability that an incident would occur at any of the Whittenton Alternative at-grade crossings is 4.677 percent per year. By comparison, the Stoughton Alternative's Dean Street (Route 44) grade crossing along the portion of the Stoughton Line bypassed by the Whittenton Alternatives has the highest future incident probability at 7.4 percent, which would be equivalent to approximately one incident every 14 years. The average probability that an incident would occur at any of the Stoughton Alternative at-grade crossings is 4.944 percent per year.

Although both the Stoughton and Whittenton Alternatives have similar probabilities of an incident occurring at any one crossing, the probability of an incident along the Whittenton Alternative alignment in Taunton is double that of the Stoughton Alternative alignment because there are roughly double the number of grade crossings on the Whittenton alignment in Taunton.

According to MBTA data, the predicted frequency of an incident occurring throughout the MBTA's system and its 333 active at-grade crossings is 0.0199 in one year. The historical data from the past 10 years of an incident at any of the 333 active at-grade crossings in the MBTA's system has an observed probability of 0.009 in one year. Although the predicted frequency of an incident under the Stoughton Alternatives is 0.03618 in one year, the measures and precautions taken by the MBTA have made the probability less likely and provide a historical probability of 0.0009 in one year. With the MBTA continuing to take safety measures and precautions at all of their crossings on the South Coast Rail project, the predicted incident rate of 0.03618 is likely to be less.

Stations

Transportation analyses for the alternatives were conducted for all the planned station locations associated with the rail alternatives. The analysis of transportation impacts is based on projected ridership at each station. Since some stations are included in more than one alternative, each station was analyzed only once using the highest ridership projection for the station from among the alternatives. This approach results in a worst case scenario analysis. As with the No-Build analysis, the Build analysis results are presented by community and station. For each of the stations analyzed (except for Taunton Station and Dana St. Station as explained below), vehicle trip generation was estimated based on these 2030 ridership forecasts.

To determine the potential impact the revised 2035 ridership results could have on the DEIS/DEIR traffic analyses and findings, 2035 ridership data were compared to the 2030 ridership data. Details of the comparison for the Stoughton and Whittenton Alternatives are shown in Appendix 3.2-H. In general, 2035 boardings are lower than the 2030 boardings, with a few exceptions. The Stoughton Electric 2035 ridership projects slightly higher inbound boardings during the morning peak period at three stations: Taunton, Fall River Depot, and Kings Highway. Breaking these increases down further to peak hour analysis of various travel modes, less than 26 additional vehicles are projected to drive and park at Fall River Depot and Kings Highway stations. Approximately 10 additional kiss and ride trips are projected for these two stations. Increases of peak hour trips at the Taunton Station are more significant, with Taunton Station projected to add 78 park and ride trips and 43 kiss and ride peak hour trips over the trip generation estimated in the DEIS/DEIR. While Fall River Depot and Kings Highway reflect minimal change in ridership, updated 2035 traffic analysis is provided for the Taunton Station.

In addition to Taunton Station, this section also presents traffic analysis for Dana Street Station, which was not included in the DEIS/DEIR station-level traffic impact analysis. It analyzes the transportation impacts of relocating the proposed Downtown Taunton Station, previously proposed as part of the Whittenton Alternative.

The 2030 DEIS/DEIR station boarding estimates were used to prepare traffic impact analyses for the relocated Stoughton Station.

The results of the Build analyses are presented for signalized and unsignalized intersections by community. The results include No-Build conditions LOS and highlight locations that operate at unacceptable levels of service during at least one peak hour. Intersections that degrade to unacceptable levels of service from No-Build conditions are denoted in **bold**. LOS analyses for all intersections are provided in Appendix 4.1-I.

New Bedford Transportation Impacts (All Rail Alternatives)

The two station locations proposed in New Bedford include:

- Whale’s Tooth, which would be located east of Route 18 and north of Route 6
- King’s Highway, which would be located south of King’s Highway, east of the Route 140 interchange

The Whale’s Tooth Station would be located between the intersections of Acushnet Avenue at Hillman Street and the intersection of Acushnet Avenue at Pearl Street. Access to the proposed station would be via an unsignalized driveway on Acushnet Avenue. An existing bus stop is located immediately adjacent to the proposed station. Logan Street and Hillman Street provide pedestrian and bicycle connections to the station from the neighborhood west of Route 18.

The King's Highway Station is located behind the existing retail mall in the Shaw's Shopping Center. Access to the proposed station would be provided via the signalized Shaw's Shopping Center driveway. Pedestrian access would be provided via a pedestrian walkway across from Tarkiln Hill Road. Bicycle access would be provided via King's Highway and the proposed station driveway.

Traffic Operations—Design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within New Bedford. These site generated volumes were added to the No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are depicted in Figures 4.1-54 through 4.1-57.

The projected number of vehicle trips in and out of the Whale's Tooth and King's Highway stations during the morning and evening peak hours are shown in Table 4.1-59. The trip generation for the New Bedford stations is based on ridership projections for the Attleboro Alternative.

Table 4.1-59 Park-and-Ride and Vehicular Drop-Off Vehicle Trips:[1] New Bedford Stations

		Morning Peak Hour		Evening Peak Hour	
Station	**Type of Trip**	**In**	**Out**	**In**	**Out**
Whale's Tooth	Park-and-Ride	146	16	10	120
	Drop-off	44	44	35	35
	Total Vehicles	190	60	45	155
King's Highway	Park-and-Ride	143	16	8	114
	Drop-off	28	28	21	21
	Total Vehicles	171	44	29	135

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of vehicular drop-off vehicle trips assumes one rider per vehicle.

The directional distribution of station-generated traffic is a function of population distribution, vehicle-owning households, existing travel patterns on area roadways, and traffic conditions. The trip distribution for the park-and-ride trips associated with New Bedford stations is based on ridership data provided by CTPS, which take into account these factors. Table 4.1-60 provides the geographic distribution of these trips.

Table 4.1-60 New Bedford Trip Distribution

To/From	King's Highway Station	Whale's Tooth Station
North	8%	21%
South	27%	17%
East	23%	30%
West	43%	32%

Source: CTPS Travel Demand Model.

The park-and-ride traffic was distributed to the study area roadways based on these percentages. Drop-off traffic was added separately and is based on existing travel patterns on area roadways near the proposed station locations.

The intersection levels of service based on the addition of rail related traffic are shown in Table 4.1-61. At most of the signalized or unsignalized intersections analyzed, no traffic operating deficiencies would be created by the Whale's Tooth Station. Four unsignalized locations would continue to operate at a deficient LOS E and LOS F during one or both peak hours. These include Coggeshall Street at North Front Street during both peak hours and Coggeshall Street at Purchase Street, Purchase Street at Weld Street and Purchase Street at Route 18 SB ramp during the evening peak hour. The station driveway would operate at LOS B during both peak hours.

There would be no changes from acceptable LOS at the intersections analyzed for the King's Highway station. The unsignalized intersections of Mount Pleasant Street and Route 140 SB Ramps would continue to operate at LOS F during the morning peak hour as it does under No-Build conditions. The intersection of King's Highway at Mount Pleasant Street and the unsignalized intersections of Church Street at Park Avenue, Mount Pleasant Street and Route 140 SB Ramps and King's Highway at Tarkiln Hill Road would continue to operate at LOS E or F during the evening peak hour as they do under No-Build conditions.

Traffic Signal Warrants—Six intersections were evaluated against the traffic signal warrant for the peak hour period:

- Coggeshall Street at North Front Street
- Coggeshall Street at Purchase Street
- Purchase Street at Weld Street and Route 18 southbound ramp
- Purchase Street at Route 18 southbound ramp
- Mount Pleasant Street at Route 140 southbound ramps
- Acushnet Avenue at Station Driveway

The intersection of Coggeshall Street at North Front Street meets the requirements set forth by the MUTCD for traffic signal installation based on future peak hour traffic volumes.

The Coggeshall Street at Purchase Street intersection potentially meets the crash experience warrant by having more than five correctable crashes in a recent one-year period. A full eight-hour warrant analysis will be required to confirm this warrant. This analysis would be completed during the preliminary engineering phase of the project. The Mount Pleasant Street at Route 140 southbound ramps intersection is projected to meet peak hour traffic signal warrants with or without the South Coast Rail project. Project traffic through this intersection constitutes only a minor 2 percent increase in traffic from No-Build conditions.

The Purchase Street at Weld Street and Route 18 southbound ramp, Purchase Street at Route 18 southbound ramp and the Acushnet Avenue at Station Driveway intersections do not meet peak hour traffic signal warrants based on the projected future traffic volumes.

Table 4.1-61 New Bedford Intersection Capacity Analysis–2030 Build Conditions vs. 2030 No-Build Conditions All Alternatives

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	Build			No-Build	Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Whale's Tooth Station								
Hillman St at Purchase St.	B	0.42	13	B	B	0.60	15	B
Mill St at Pleasant St	F	0.82	>80	F	E	0.94	79	E
Union St. at Rt. 18 SB	E	0.92	78	E	F	>1.00	>80	F
Union St at McArthur Dr.	C	0.50	33	C	D	0.47	43	D
Rt. 18 NB at Coggeshall St.	B	0.51	18	B	B	0.58	19	B
Rt. 18 SB at Coggeshall St.	D	0.87	44	D	C	0.74	31	C
Coggeshall St. at Belleville Ave.	B	0.72	20	C	B	0.72	20	C
King's Highway Station								
King's Hwy. at Rt. 140 NB Ramps	B	0.60	22	C	C	0.93	29	C
Rt. 18 at Wood St	C	0.58	21	C	B	0.68	17	B
Church St. at Nash Rd	B	0.58	18	B	C	0.92	31	C
Church St. at Tarkiln Hill Rd	B	0.71	28	C	D	0.89	37	D
King's Highway at Stop & Shop driveway	A	0.50	9	A	B	0.73	15	B
King's Highway at Shaw's driveway (Station driveway)	A	0.41	7	A	A	0.62	9	A
King's Highway at Mt. Pleasant St.	B	0.54	26	C	E	>1.00	62	E
Unsignalized Intersections	No-Build	Critical Movement	Delay[4]	LOS	No-Build	Critical Movement	Delay	LOS
Whale's Tooth Station								
Hillman St. at McArthur Dr.	B	Hillman EB L/R	17	C	B	Hillman EB L/R	16	C
McArthur Dr. at Herman Melville Blvd.	B	Melville WB L/R	16	C	C	Melville WB L/R	19	C
Coggeshall St. at Purchase St.	C	Purchase SB All	20	C	F	Purchase NB All	>50	F
Coggeshall St. at N. Front St.	F	N. Front St. NB All	>50	F	F	N. Front St. NB All	>50	F
Purchase St. at Weld St.	C	Weld WB L	27	D	F	Weld WB L	>50	F
Logan St. at Purchase St.	C	Logan WB L/R	17	C	C	Logan WB L/R	24	C
Logan St. at McArthur Dr.	B	Logan WB All	12	B	B	Logan WB All	13	B
Logan St. at N. Front St.	C	Logan EB All	28	D	C	Logan EB All	27	D
Wamsutta St. at N. Front St.	B	Wamsutta EB All	11	B	B	Wamsutta EB All	13	B
Wamsutta St. at McArthur Dr.	A	Wamsutta WB L/R	10	B	A	Wamsutta WB L/R	10	B
Whale's Tooth Station driveway at McArthur Dr.	N/A	Driveway WB L/R	11	B	N/A	Driveway WB L/R	12	B
Purchase St. at Rt. 18 SB Exit Ramp	D	Rt. 18 WB All	29	D	E	Rt. 18 WB All	49	E
King's Highway Station								

Unsignalized Intersections (continued)	No-Build	Critical Movement	Delay[4]	LOS	No-Build	Critical Movement	Delay	LOS
Mt. Pleasant St. at Rt. 140 SB Ramps	F	Off-Ramp WB L/R	>50	F	F	Off-Ramp WB L/R	>50	F
Church St. at Park Ave.	C	Park WB All	23	C	F	Park WB All	>50	F
Church St. at Irvington St.	B	Irvington WB All	16	C	C	Irvington EB All	23	C
King's Highway at Tarkiln Hill Rd.	D	Tarkiln EB L/R	28	D	F	Tarkiln EB L/R	>50	F

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 150 pedestrian/bicycle trips would access the Whale's Tooth Station on a daily basis; which would increase pedestrian and bicycle activity in the vicinity of Acushnet Avenue. At King's Highway Station, approximately 120 pedestrian/bicycle trips could be expected. The majority of the infrastructure needed to support non-motorized transportation at both proposed station exists currently and would not be adversely impacted by the change in number of pedestrians or bicyclists on study area roadways.

Traffic signal timing and phasing changes would be required at the intersection of Mill Street at Pleasant Street to accommodate pedestrian demands. These changes are discussed further in Section 4.1.5, Mitigation Measures. Pedestrian demands associated with the proposed Whale's Tooth Station would also require a new sidewalk on Acushnet Avenue between Hillman Street and the proposed station driveway and a crosswalk across Acushnet Avenue at Hillman Street.

To accommodate increased pedestrian demand at King's Highway Station, changes to the pedestrian signal phases at the intersections of Church Street/Tarkiln Hill Road and Jones Street/Mount Pleasant Street would be required. These changes are discussed further in Section 4.1.5, Mitigation Measures.

Neither of the proposed New Bedford Station locations would physically alter designated bicycle facilities nor disrupt future plans for either on-road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—The Whale's Tooth station is proposed to have 694 parking spaces (15 of these handicapped accessible) to serve as a shared use parking facility with existing ferry service. The proposed project would not physically alter the existing public parking supply or impact parking availability within New Bedford. Based on the projected daily park-and-ride ridership, the parking supply would be sufficient to meet the peak parking demand for 310 spaces. The surplus of 384 spaces would remain available for ferry passenger use.

The King's Highway station is proposed to have 360 spaces (12 of these handicapped accessible) to serve as a shared use parking facility with the existing cinema. Since peak parking demand for the cinema would occur during the evening, after most commuters have returned home, the available parking supply should be adequate to meet the commuter rail peak demand for 300 spaces.

Freetown Transportation Impacts

The Freetown station site would be located east of South Main Street south of Route 24 Exit 9 between the Stop & Shop Distribution Center and the planned entrance to the Riverfront Business Park (on the opposite side of the roadway). Access to the proposed station would be via an unsignalized driveway and adjacent sidewalk, thus providing access for all users.

Traffic Operations—As discussed above, design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within Freetown. These site generated volumes were added to the No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are depicted in Figures 4.1-58 and 4.1-59.

The projected number of vehicle trips in and out of the Freetown station during the morning and evening peak hours are shown in Table 4.1-62. The trip generation for this station is based on the projected ridership with the Stoughton Alternative.

Table 4.1-62 Park-and-Ride and Vehicular Drop-Off Vehicle Trips:[1] Freetown Station

Station	Type of Trip	Morning Peak Hour In	Morning Peak Hour Out	Evening Peak Hour In	Evening Peak Hour Out
Freetown	Park-and-Ride	81	9	5	45
	Drop-off	17	17	9	9
	Total Vehicles	98	26	14	54

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of vehicular drop-off vehicle trips assumes one rider per vehicle.

The trip distribution for the park-and-ride trips associated with the Freetown Station is based on ridership data provided by CTPS, which take into account factors such as population, existing travel patterns, and traffic congestion, as noted above. Table 4.1-63 provides the geographic distribution of these trips.

Table 4.1-63 Freetown Trip Distribution

To/From	Distribution
North	54%
South	41%
East	5%
West	0%

Source: CTPS Travel Demand Model.

The park-and-ride traffic was distributed to the study area roadways based on these percentages. Drop-off traffic was added separately and is based on existing travel patterns on area roadways near the proposed station locations.

The intersection levels of service based on the addition of rail related traffic are shown in Table 4.1-64. Seven signalized intersections were analyzed under No-Build and Build conditions. All but one location would operate at acceptable levels of service under both conditions. The intersection of South Main Street at the Route 24 northbound ramps would continue to operate at LOS E during the evening peak hour. No additional unsignalized intersections would become deficient during either the morning or evening peak hour.

Table 4.1-64 Freetown Intersection Capacity Analysis—2030 Build Conditions vs. No-Build Conditions All Alternatives

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	Build			No-Build	Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS[1]	V/C	Delay	LOS
Freetown Station								
S. Main St. at Rte. 24 SB Ramps	A	0.59	7	A	B	0.64	10	B
S. Main St. at Rte. 24 NB Ramps	C	0.99	37	D	E	>1.00	74	E
S. Main St. at Payne's Crossing Driveway	A	0.33	2	A	B	0.49	13	B
Executive Park Dr. at S. Main St.	B	0.83	21	C	D	0.84	44	D
Executive Park Dr. at Rt. 24 SB Off-Ramps	C	0.86	30	C	C	0.90	24	C
Executive Park Dr. at Rt. 24 NB Off-Ramps	B	0.84	15	B	A	0.52	8	A
Unsignalized Intersections	**LOS**	**Critical Movement**	**Delay[4]**	**LOS**	**LOS**	**Critical Move-ment**	**Delay**	**LOS**
Freetown Station								
S. Main St. at High St.	F	NW All	>50	F	F	NW All	>50	F
S. Main St. at Ridge Hill Rd.	F	NW All	>50	F	F	NW All	>50	F
S. Main St. at Narrows Rd.	D	Narrows L/R	34	D	F	Narrows L/R	>50	F
S. Main St. at Copicut St.	B	Copicut L/R	16	C	B	Copicut L/R	15	C
Freetown Station Driveway at S. Main St.	N/A	Driveway L/R	14	B	N/A	Driveway L/R	16	C

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All=all movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound Traffic Signal Warrants

Traffic Signal Warrants—Four intersections were evaluated against the traffic signal warrant for the peak hour period:

- South Main Street at High Street
- South Main Street at Ridge Hill Road
- South Main Street at Narrows Road
- South Main Street at Freetown Station Driveway

The South Main Street and Ridge Hill Road intersection is projected to meet peak hour traffic signal warrant during the evening peak hour with or without the South Coast Rail project. Project traffic through this intersection would constitute only a minor 1.5 percent increase in traffic from No-Build conditions.

The other unsignalized intersections along South Main Street do not meet peak hour traffic signal warrants based on the projected future traffic volumes.

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 40 pedestrian/bicycle trips would access Freetown Station on a daily basis which would increase pedestrian and bicycle activity along South Main Street. The majority of the infrastructure needed to support pedestrian and bicycle traffic to the proposed station exists currently and would not be adversely impacted by the change in number of pedestrians or bicycles on study area roadways.

To accommodate pedestrian demands, the existing sidewalk along the east side of South Main Street would be extended south (about 1,600 feet) to the station driveway.

The proposed station location would not physically alter designated bicycle facilities or disrupt future plans for either on road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—Freetown Station is proposed to have 174 parking spaces (of which seven would be handicapped accessible). An additional eight parking spaces would be reserved for drop-off activity. The proposed project would not physically alter the existing public parking supply or impact parking availability within Freetown. Based on the projected daily park-and-ride ridership, the parking supply would be sufficient to meet the peak parking demand for 170 spaces.

Fall River Transportation Impacts (All Rail Alternatives)

Fall River has two proposed station locations that would serve both the Stoughton and Whittenton Alternatives:

- Fall River Depot, which would be located 1 mile north of downtown Fall River on North Davol Street between Pearce Street and Turner Street.
- Battleship Cove, which would be located on Ponta Delgada Boulevard west of Route 138 and south of I-195 and the Fall River Heritage State Park.

Access to the proposed Fall River Depot Station would be via an unsignalized driveway located on North Davol Street. A separate entrance and exit driveway are provided for drop-off traffic and connecting local bus service. Pearce Street and Turner Street provide pedestrian and bicycle connections to the station from the neighborhood east of the railroad tracks.

At Battleship Cove, access to the proposed station would be provided via a drop-off loop on Ponta Delgada Boulevard. No parking is proposed for this station. Pedestrian and bicycle access would also be provided via Water Street and Ponta Delgada Boulevard.

Traffic Operations—As discussed above, design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within Fall River. These site generated volumes were added to the

No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are depicted in Figures 4.1-60 and 4.1-61.

The projected number of vehicle trips in and out of the Fall River Depot and Battleship Cove stations in the morning and evening peak hours are shown in Table 4.1-65. The trip generation of the Fall River stations is based on projected ridership for the Attleboro Alternative.

Table 4.1-65 Park-and-Ride and Vehicular Drop-Off Vehicle Trips:[1] Fall River Stations All Alternatives

		Morning Peak Hour		Evening Peak Hour	
Station	Type of Trip	In	Out	In	Out
Fall River Depot	Park-and-Ride	184	25	14	166
	Drop-off	26	26	22	22
	Total Vehicles	210	51	36	188
Battleship Cove	Park-and-Ride	0	0	0	0
	Drop-off	34	34	25	25
	Total Vehicles	34	34	25	25

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of drop-off vehicle trips assumes one rider per vehicle.

The directional distribution of station-generated traffic is a function of population distribution, vehicle-owning households, existing travel patterns on area roadways, and traffic conditions. The trip distribution for the park-and-ride trips associated with Fall River Depot Station is based on ridership data provided by CTPS, which take into account these factors. Table 4.1-66 provides the geographic distribution of these trips.

Table 4.1-66 Fall River Trip Distribution

To/From	Distribution
North	20%
South	58%
East	22%
West	0%

Source: CTPS Travel Demand Model.

The park-and-ride traffic was distributed to the study area roadways based on these percentages. Drop-off traffic was added separately and is based on existing travel patterns on area roadways near the proposed station locations. Only drop-off traffic was generated by Battleship Cove Station, as no long-term parking is planned.

The intersection levels of service based on the addition of rail related traffic are shown in Table 4.1-67. Three signalized and five unsignalized intersections, including the station driveway, were analyzed for the Fall River Depot station under Build conditions. All intersections would operate at acceptable levels of service.

Table 4.1-67 Fall River Intersection Capacity Analysis—2030 Build Conditions vs. No-Build Conditions, All Rail Alternatives

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	**Build**			**No-Build**	**Build**		
Signalized Intersections	**LOS[1]**	**V/C[2]**	**Delay[3]**	**LOS**	**LOS[1]**	**V/C**	**Delay**	**LOS**
Fall River Depot Station								
S. Davol St. at President Ave.	C	0.70	25	C	C	0.66	24	C
N. Davol St. at President Ave.	B	0.53	19	B	C	0.72	22	C
N. Main St. at President Ave.	C	0.86	37	D	D	0.93	38	D
Unsignalized Intersections	**LOS**	**Critical Movement**	**Delay[4]**	**LOS**	**LOS**	**Critical Movement**	**Delay**	**LOS**
Fall River Depot Station								
Turner St. at N. Davol St.	B	Turner WB R	16	C	B	Turner WB R	15	C
Pearce St. at N. Davol St.	B	Pearce WB R	13	B	B	Pearce WB R	17	C
Davol St. SB to NB U-turn near Cedar St.	B	U-turn SW L	13	B	B	U-turn SW L	13	B
Davol NB to SB U-turn near Cedar St	B	U-turn NE L	19	C	B	U-turn NE L	14	B
Fall River Depot Station Driveway at N. Davol St.	N/A	Driveway WB R	13	B	N/A	Driveway WB R	17	C
Battleship Cove Station								
Ponta Delgada at Anawan St.	C	Anawan EB All	16	C	C	Anawan WB All	17	C
Ferry St. at Ponta Delgada	B	Ferry EB L/R	16	C	B	Ferry EB L/R	13	B
Anawan St. at Davol St.	F	Davol SB All	>50	F	F	Davol SB All	>50	F
Central St. at Davol St.	F	Central WB L	>50	F	F	Central WB L	>50	F
Battleship Cove Station driveway at Ponta Delgada	N/A	Driveway WB L/R	12	B	N/A	Driveway WB L/R	12	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

The Battleship Cove station is not anticipated to serve a substantial amount of regular commuter rail ridership but is intended to provide tourist access to the attractions at Battleship Cove. There would be limited space available to accommodate drop-off and pick-up activity. No substantial change in LOS would occur at the four unsignalized intersections that were analyzed. The proposed station driveway would operate at LOS B during both peak hours.

Traffic Signal Warrants—Three intersections were evaluated against the traffic signal warrant for the available peak hour periods:

- Anawan Street at Davol Street
- Central Street at Davol Street
- North Davol Street at Station Driveway

The Anawan Street at Davol Street intersection is projected to meet peak hour traffic signal warrants with or without the South Coast Rail project. Project traffic through this intersection constitutes only a minor 3 percent increase in traffic from No-Build conditions.

The Central Street at Davol Street and the North Davol Street at Station Driveway intersections do not meet peak hour traffic signal warrants based on the projected future traffic volumes.

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 280 non-motorized person trips (pedestrians and bicycles) would access Fall River Depot Station on a daily basis which would increase pedestrian and bicycle activity in the vicinity of President Avenue, Davol Street, and North Main Street. At Battleship Cove Station, approximately 180 pedestrian/bicycle trips would be expected. The majority of the pedestrian and bicycle infrastructure needed to support both proposed stations exists currently and would not be adversely impacted by the change in number of pedestrians or bicyclists on study area roadways.

Traffic signal timing and phasing changes would be required at the intersections of Davol Street Northbound/President Avenue and North Main Street/President Avenue to accommodate pedestrian demands at Fall River Depot Station. These changes are discussed further Section 4.1.5, Mitigation Measures.

To accommodate increased pedestrian demand at Battleship Cove Station, crosswalks across Broadway and Central Street would be restriped. Sidewalks and crosswalks elsewhere in the vicinity of Battleship Cove are adequate to handle the expected demand.

Neither of the proposed station locations would physically alter designated bicycle facilities nor disrupt future plans for either on-road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—The Fall River Depot station is proposed to have 513 parking spaces (of which 11 would be handicapped accessible). An additional 10 parking spaces would be reserved for drop-off activity. The proposed project would not physically alter the existing public parking supply or impact parking availability within Fall River. Based on the projected daily park-and-ride ridership, the parking supply would be sufficient to meet the peak parking demand for 430 spaces. No short or long-term parking would be provided at Battleship Cove.

Taunton Transportation Impacts

Traffic operations were analyzed for three station locations in the City of Taunton:

- Taunton Depot (all alternatives), which would be accessible from Route 140 west of the Route 24 interchange
- Dana Street (Whittenton Alternatives)
- Taunton (Stoughton Alternative), which would be located on Arlington Street just north of Dean Street (Route 44)

The Taunton Depot Station (associated with both rail alternatives) would be located behind the existing retail mall at Taunton Depot Drive. Access to the proposed station would be provided via the signalized intersection of Route 140 and Taunton Depot Drive. Pedestrian access would be provided via a pedestrian walkway along Route 140 and pedestrian crossing controls at Taunton Depot Drive. Bicycle access would be provided via Route 140 and Taunton Depot Drive.

Access to the proposed Dana Street Station (associated with the Whittenton Alternative) would be via unsignalized intersections Dana Street. Pedestrian walkways would be provided that lead to the platform. Additional sidewalks would be constructed along Dana Street and Danforth Street.

At Taunton Station (Dean Street) (associated with the Stoughton Alternative), access to the proposed station would be provided via an unsignalized intersection on Arlington Street. Major access to the station would be provided from the signalized intersection of Arlington Street with Dean Street. Pedestrian access would be provided via pedestrian sidewalks along Dean Street and Arlington Street. Bicycle access would be provided via Arlington Street and Dean Street.

Traffic Operations- Taunton Depot Station—Design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within Taunton. These site generated volumes were added to the No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are depicted in Figures 4.1-62 and 4.1-63.

The projected number of vehicle trips in and out of the Taunton Depot station in the morning and evening peak hours are shown in Table 4.1-68. The trip generation is based on the DEIS/DEIR projected ridership for the Stoughton Alternative.

Table 4.1-68 Park-and-Ride and Vehicular Drop-Off Vehicle Trips:[1] Taunton Depot Station

		Morning Peak Hour		Evening Peak Hour	
Station	Type of Trip	In	Out	In	Out
Taunton Depot (all alternatives)	Park-and-Ride	160	20	12	128
	Drop-Off	18	18	14	14
	Total Vehicles	178	38	36	144

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of vehicular drop-off vehicle trips assumes one rider per vehicle.

The intersection levels of service based on the addition of rail-related traffic are shown in Table 4.1-69. There would be no change in LOS under Build conditions at six of the seven signalized intersections analyzed for the Taunton Depot station location. The intersection of Route 140 at Hart Street during the morning and evening peak hours would continue operating at a deficient LOS, declining from LOS E to LOS F. No unsignalized intersections were analyzed for the Taunton Depot Station.

Table 4.1-69 Taunton Depot Station Intersection Capacity Analysis–2030 Build Conditions vs. 2030 No-Build Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	Build			No-Build	Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Taunton Depot Station (all alts.)								
Rt. 140 at Hart St.	E	>1.00	>80	F	E	>1.00	>80	F
Rt. 140 at Rt. 24 SB Ramps	B	0.78	17	B	E	>1.00	70	E
Rt. 140 at Rt. 24 NB Ramps	A	0.90	8	A	A	0.72	3	A
Rt. 140 at Taunton Depot Dr.	B	0.56	15	B	B	0.61	22	C
Rt. 140 at Mozzone Blvd.	A	0.44	3	A	C	0.97	26	C
County St at Silver City Galleria Mall driveway/ Rt. 140 Ramps	A	0.09	4	A	A	0.41	8	A
Stevens St. at Rt. 140 NB Ramps	B	0.46	15	B	B	0.58	18	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Traffic Operations- Dana Street Station—Dana Street Station is approximately 0.5 mile north of the previously-proposed Downtown Taunton Station and would be served by many of the same roadways that provided access to the Downtown Taunton Station. In addition to the station site relocation, revised ridership projections have been developed, which further change traffic operations. The ridership results show a decrease in proposed auto demand to the station.

Future 2030 ridership projections were developed by the Central Transportation Planning Staff (CTPS) for the previously proposed Downtown Taunton Station. These projections have since been revised to represent a 2035 condition at the proposed Dana Street Station. Table 4.1-70 summarizes the previous and current ridership projections for the two conditions under the Whittenton Alternative. As shown, ridership to the Dana Street station is projected to be between 48 and 63 percent less than was projected for the Downtown Taunton Station.

Table 4.1-70 Downtown Taunton/Dana Street Station Ridership Projection Comparison

Boardings	2030 Downtown Taunton Condition	2035 Dana Street Condition	Difference	Percent Difference
Daily	850	310	-540	-64%
AM Peak	460	240	-220	-48%

Source: CTPS

The reduction in ridership results in reduced vehicle trips to Dana Street station when compared to the Downtown Taunton Station. The reduction in vehicle trips is shown in Table 4.1-71. The DEIS/DEIR presented a full analysis of the Downtown Taunton Station for both the morning and evening peak hours using ridership boarding and alighting information provided by CTPS. Only morning boarding information was provided as part of the current ridership estimates, therefore for the purposes of this analysis it is assumed peak hour trips are the same magnitude (reversed direction) during the morning and evening peak hours.

Table 4.1-71 Downtown Taunton/Dana Street Station Vehicle Trip Comparison

Trips (vph)	2030 Downtown Taunton Station	2035 Dana Street Station	Difference
AM Peak Hour			
Enter	270	130	-140
Exit	44	25	-19
Total	314	155	-159
PM Peak Hour[1]			
Enter	44	25	-19
Exit	270	130	-140
Total	314	155	-159

Source: CTPS
vph vehicles per hour
1 PM data not provided by CTPS; assumed to be reverse impact of AM peak hour.

The vehicle trips related to the proposed Dana Street Station are less than half of the previous estimates; directly attributable to an overall reduction in ridership projected by CTPS. This removes a substantial amount of project-related vehicular traffic from the downtown Taunton area and reduces project impacts related to the station. Although it is projected by CTPS that a higher percentage of riders would drive to a station on Dana Street (69 percent of riders) when compared to Downtown Taunton (44 percent of riders drive), the overall vehicle trips are still substantially lower.

To assess the effects of these changes, a level of service analysis was revised for the intersection of Route 140/Taunton Street at Oak Street, which is the highest-volume intersection in the study area previously defined for the Downtown Taunton Station. As traffic accessing the new Dana Street Station would also likely use this critical intersection, a revised analysis was prepared to assess new impact. Table 4.1-72 presents a comparison of the traffic operations using 2030 Whittenton ridership estimates for Downtown Taunton and 2035 Whittenton ridership estimates for Dana Street.

Table 4.1-72 Downtown Taunton/Dana Street Station Route 140/Taunton Street at Oak Street, Signalized Intersection Traffic Operations

		2030 Downtown Taunton Condition						2035 Dana Street Condition					
		AM Peak			PM Peak			AM Peak			PM Peak		
Location	Lane Group	v/c a	Del b	LOS c	v/c	Del	LOS	v/c	Del	LOS	v/c	Del	LOS
Route 140/	EB LT	0.80	50	D	0.94	78	E	0.71	38	D	0.96	78	E
Tremont Street	EB LT-TH-RT	0.83	54	D	0.97	88	F	0.74	40	D	1.02	95	F
Oak Street/	WB LT-TH-RT	0.84	67	E	1.00	118	F	0.74	49	D	0.92	88	F
Parking Lot	NB LT-TH-RT	0.78	35	C	n/a	n/a	n/a	0.81	39	D	n/a	n/a	n/a
	NB LT[1]	n/a	n/a	n/a	0.51	29	C	n/a	n/a	n/a	0.61	33	C
	NB TH-RT[1]	n/a	n/a	n/a	0.81	40	D	n/a	n/a	n/a	0.59	32	C
	SB LT-TH-RT	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	SB LT-TH	0.81	39	D	>1.2	>120	F	0.64	33	C	0.92	66	E
	SB RT	0.25	9	A	0.34	15	B	0.29	10	B	0.35	16	B
	Overall	0.82	39	D	1.11	75	E	0.77	33	C	0.96	57	E

1 Defacto left-turn during weekday evening peak hour

Given the substantial reduction in ridership between the Downtown Taunton Station and the currently proposed Dana Street Station, traffic operations at the intersection of Route 140/Tremont Street at Oak Street are projected to be improved when compared to the previous analysis. Several intersection movements are still projected to operate at a poor LOS E or LOS F during the 2035 evening peak hour. Although traffic impacts are lower, the mitigation committed to in the DEIS/DEIR would still be recommended because of the proximity of the intersection to the adjacent grade crossing. These measures are described in the mitigation section below.

Traffic Operations-Taunton Station—Table 4.1-73 summarizes the previous (2030) and current (2035) ridership projections for Taunton Station under the Stoughton Electric Alternative. As shown, 2035 ridership estimates at Taunton Station are 72 and 118 percent higher, for total daily and AM peak ridership, respectively, than previous 2030 estimates.

Table 4.1-73 Taunton Station Ridership Projection Comparison

Boardings	2030 Taunton Station	2035 Taunton Station	Difference	Percent Difference
Daily	360	620	260	72%
AM Peak	220	480	260	118%

Source: CTPS

The increase in ridership translates to a corresponding increase in vehicle trips to Taunton Station when compared to the DEIS/DEIR analysis. The revised vehicle trip projections are provided in Table 4.1-74. The DEIS/DEIR presented a full analysis for both the morning and evening peak hours using ridership boarding and alighting information provided by CTPS. Only morning boarding information was provided as part of the December 2012 ridership estimates, therefore for the purposes of this analysis it is assumed peak hour trips are the same magnitude (reversed direction) during the morning and evening peak hours.

Table 4.1-74 Taunton Station Vehicle Trip Comparison

Trips (vph)	2030 Taunton Station	2035 Taunton Station	Difference
AM Peak Hour			
Enter	61	119	58
Exit	37	58	21
Total	98	177	79
PM Peak Hour1			
Enter	23	58	35
Exit	36	119	83
Total	59	177	118

Source: CTPS
vph vehicles per hour
1 PM data not provided by CTPS; assumed to be reverse impact of AM peak hour.

The vehicle trips related to the projected changes in ridership are higher than previous 2030 estimates. This is attributed to an overall increase in ridership projected by CPTS and a projected increase in the percentage of riders who would drive to a station (56 percent of riders) when compared to the DEIS/DEIR analysis (38 percent of riders drive).

To assess the effects of these changes, the DEIS/DEIR level of service analysis was revised for all intersections in the Taunton Station study area. Table 4.1-75 presents a comparison of the 2030 No-Build and 2035 Build traffic operations under the Stoughton Electric Alternative.

Table 4.1-75 Taunton Station Signalized Intersection Traffic Operations–No-Build (2030) versus Build (2035)

Location	2030 No-Build Condition						2035 Taunton Station Build Condition					
	AM Peak			PM Peak			AM Peak			PM Peak		
	v/c [1]	Del [2]	LOS [3]	v/c	Del	LOS	v/c	Del	LOS	v/c	Del	LOS
Signalized Intersections												
Broadway St at	0.75	34	C	0.86	47	D	0.77	37	D	0.92	57	E
Rt. 44 at Dean St./Rte.	0.76	9	A	0.68	11	B	0.78	10	B	0.72	11	B
Rt. 44 at Longmeadow	1.00	>80	F	>1.00	78	E	>1.00	>80	F	>1.00	85	F
Rt. 44 at Arlington St	0.97	43	D	0.99	53	D	0.99	66	E	>1.00	70	E
Main St. at Union St.	0.92	33	C	0.88	30	C	0.96	40	D	0.91	36	D
Spring St at Summer St (Rt. 140)	0.70	26	C	0.80	27	C	0.73	26	C	0.80	27	C
Rt. 140 at Hon. Gordon Owen Riverway	0.75	16	B	0.95	41	D	0.77	17	B	0.97	47	D
Unsignalized Intersections	Critical Movement	Del [4]	LOS	Critical Movement	Del	LOS	Critical Movement	Del	LOS	Critical Movement	Del	LOS
Arlington St at School St	School NB	20	C	School NB	30	D	School NB	22	C	School NB	39	E
Washington St at Purchase St	Washington SB	25	C	Washington NB	>50	F	Washington SB	34	D	Washington NB	>50	F
School St at Winter St	School SB	>50	F	School SB	>50	F	School SB	>50	F	School SB	>5	F
Arlington St at Taunton Station Driveway	NA	NA	NA	NA	NA	NA	Driveway WB Left	15	C	Driveway WB Left	21	C

Source: Synchro 7.0 Software; Build 763
1 volume-to-capacity ratio
2 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
3 level of service
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; NA = Not Applicable

The overall results of the level of service analysis are generally the same as presented in the DEIS/DEIR. Mitigation measures are required to offset project related impacts and are described in the mitigation section below. One location, Arlington Street at School Street (where mitigation was not previously recommended), shows project-related impacts that affect level of service such that mitigation is now required.

When compared to the DEIS/DEIR delay and level of service results, the intersection of Dean Street at Longmeadow Street realizes a small increase in delay (10 additional seconds during the morning peak hour and five additional seconds during the evening peak hour).

Traffic Signal Warrants—Two intersections were evaluated against the traffic signal warrant for the peak hour period:

- Washington Street at Frederick Martin Parkway
- Arlington Street at Taunton Station Driveway

The intersection of Washington Street at Frederick Martin Parkway meets the requirements set forth by the MUTCD for traffic signal installation based on future peak hour traffic volumes.

The Arlington Street at Taunton Station Driveway intersection does not meet peak hour traffic signal warrants based on the projected future traffic volumes.

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 80 trips would access Taunton Depot Station (all alternatives) on foot or by bicycle on a daily basis, which would increase pedestrian and bicycle activity in the vicinity of Route 140 and Hart Street. At Dana Street Station (Whittenton Alternatives), approximately 50 pedestrian/bicycle trips would be expected and at Taunton Station about 230 pedestrian/bike trips would be expected. Increased pedestrian and bicycle demands at either of these stations would be realized in the vicinity of Downtown Taunton, particularly along Route 44, Route 138, Oak Street and/or Arlington Street. The majority of the infrastructure needed to support pedestrian and bicycle access to the proposed stations exists currently and would not be adversely impacted by the change in number of pedestrians on study area roadways.

To accommodate pedestrian demand related to Taunton Depot Station, a sidewalk would be required within the Target shopping center. The sidewalk is necessary to delineate the pedestrian right-of-way from Route 140 to the station platform. To accommodate pedestrian demand related to Taunton Station (Stoughton Alternative) traffic signal timing and phasing changes would be required at the intersection of Dean Street and Longmeadow Street. A high visibility crosswalk with a passive flashing pedestrian crossing sign would also be needed. Finally, to support Downtown Taunton pedestrian demands, a number of traffic signal timing adjustments would be needed. These adjustments would occur at the intersections of:

- Weir Street at Broadway
- Washington Street at Court Street
- Washington Street at Fredrick Martin Boulevard
- Washington Street at Tremont Street

These mitigation measures are discussed further below in Section 4.1.5, *Mitigation Measures*.

Neither of the proposed station locations would physically alter designated bicycle facilities nor disrupt future plans for either on-road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—The Taunton Depot Station (both rail alternatives) is proposed to have 442 parking spaces (eight of these handicapped accessible). An additional 14 parking spaces would be reserved for drop-off activity.

Dana Street Station (Whittenton Alternatives) would have 477 spaces (9 of which are handicapped accessible).

Two hundred and nine (209) spaces are proposed at Taunton Station (Stoughton Alternative), including seven that are handicapped accessible. The Build Alternatives would not physically alter the existing public parking supply or impact parking availability within Taunton. Based on the projected daily park-and-ride ridership, the parking supply at each station would be sufficient to meet the peak parking demand for 320, 590, 120 spaces, respectively.

Relocated Stoughton Station Transportation Impacts

Under the Stoughton and Whittenton Alternatives, the existing railroad tracks for the Stoughton Station will be realigned and the station platform will be relocated south to the site bounded to the north by Wyman Street, west by Morton Street, and south by Brock Street. The relocated station will have two driveways: a north driveway off of Morton Street, and a south driveway off of Brock Street. As part of the station relocation, parking will be consolidated to one parking lot and increased up to 701 parking spaces, which includes 6 kiss-and-ride spaces and 17 handicap spaces.

The following sections present the transportation analysis associated with the relocation of Stoughton Station and the increase in available parking. In general, traffic conditions would improve as a result of relocating the Stoughton Station.

Station Trip Generation and Redistribution—All station-related vehicle trips were redistributed to the new driveways and throughout the roadway network for the Build Condition analysis. New vehicle trips, generated by either the expanded service or increase in available parking, were then added to the redistributed traffic volume network to create the Build Condition traffic volume networks depicted in Figures 4.1-64 and 4.1-65. Table 4.1-76 presents the projected number of new vehicle trips expected under the Build Condition.

Table 4.1-76 Relocated Stoughton Station Projected New Vehicle Trips

	Morning Peak Hour		Evening Peak Hour	
Type of Trip	In	Out	In	Out
Park-and-Ride	46	11	12	52
Kiss-and-Ride	-10	1	0	-5

As shown in Table 4.1-76, the number of kiss-and-ride trips would decrease relative to the No-Build condition. This can be attributed to a shift in the mode of access by riders. With the expansion of service, some riders currently boarding in Stoughton would board farther south, eliminating the need to be dropped off at the station. Other riders who are currently dropped off would shift to park-and-ride, as the available parking will increase under the Build Condition.

Traffic Operations Analysis—The Build Condition traffic operation analyses are shown in Table 4.1-77 through Table 4.1-79.

Table 4.1-77 Relocated Stoughton Station Signalized Intersection Capacity Analysis

Location	Period	No-Build Condition v/c [1]	Delay [2]	LOS [3]	Build Condition v/c [1]	Delay [2]	LOS [3]
Porter Street at Washington Street(Route 138)	Weekday Morning	0.73	22	C	0.68	20	C
	Weekday Evening	0.94	60	E	0.88	53	D
Pleasant Street at Park Street (Route 27)	Weekday Morning	0.96	45	D	0.92	36	D
Washington Street (Route 138)	Weekday Evening	0.83	27	C	0.78	24	C

Source: Synchro 7 (Build 773, Rev 8) software
1 volume-to-capacity ratio
2 average delay in seconds per vehicle
3 level of service

As discussed above, relocating Stoughton Station parking would redistribute station related traffic through study area intersections. A portion of traffic would access the parking lot driveway at Brock Street and no longer travel through Stoughton Center. As a result, the delay for the signalized intersections would improve slightly. The level of service at the intersection of Porter Street at Washington Street would improve from LOS E to LOS D. Complete traffic operations analysis results are provided in Appendix 4.1-K.

Relocating Stoughton Station would also eliminate the existing MBTA Lot Driveway on Wyman Street and substantially reduce or eliminate traffic at the Trackside Plaza South Driveway, eliminating most vehicle conflicts at this location. Level of service results for this intersection are not provided in Tables 4.1-78 and 4.1-79 since no delay would occur. Field observations indicate that traffic is currently using the Trackside Plaza South Driveway to access the station, while patrons of Trackside Plaza businesses use other driveways on Summer Street and Canton Street.

At the intersection of Brock Street at Washington Street, the demand for the eastbound Brock Street and westbound Kinsley Street approaches would increase substantially. The eastbound Brock Street approach and westbound Kinsley Street approach would deteriorate from LOS E to LOS F and LOS D to LOS F, respectively, during the morning peak hour. During the evening peak hour, the eastbound and westbound approach would continue to operate deficiently at LOS F.

Table 4.1-78 Relocated Stoughton Station Unsignalized Intersection Capacity Analysis (Morning Peak Hour)

Location	Critical Movement	No-Build Condition Demand[1]	v/c [2]	Delay[3]	LOS[4]	Build Condition Demand	v/c	Delay	LOS
Porter Street at Washington Street	WB RT	15	0.07	15	B	15	0.07	15	B
Freeman Street at Washington Street	WB RT	10	0.22	63	F	10	0.19	52	F
Wyman Street at Washington Street	EB RT	130	0.35	17	C	115	0.29	15	C
Summer Street at Wyman Street	EB LT-RT	33	0.04	9	A	50	0.05	9	A
Brock Street at Washington Street	EB LT-TH-RT	125	0.70	50	E	285	>1.20	>120	F
	WB LT-TH-RT	50	0.36	34	D	100	>1.20	>120	F
	NB LT-TH-RT	435	0.15	4	A	440	0.17	5	A
	SB LT-TH-RT	365	0	0	A	355	0.0	1	A
Brock Street at Morton Street	EB LT-TH-RT	65	0.12	9	A	70	0.12	9	A
	WB LT-TH-RT	215	0.40	11	B	190	0.36	11	B
	NB LT-TH-RT	237	0.46	12	B	260	0.50	13	B
	SB LT-TH-RT	82	0.17	10	A	75	0.16	9	A
Brock Street at Wyman Street	WB LT-RT	100	0.14	10	A	100	0.14	10	A
Park Avenue/Sumner Street at Park Street	EB LT	215	>1.20	>120	F	215	>1.20	>120	F
	EB TH-RT	15	0.06	17	C	5	0.06	17	C
	WB LT-TH-RT	20	0.10	22	C	20	0.10	23	C
MBTA North Driveway at Morton Street	WB LT-RT		Does not exist			85	0.13	11	B
MBTA South Driveway at Brock Street	SB LT-RT		Does not exist			160	0.30	14	B

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour for unsignalized intersections
2 volume-to-capacity ratio for the critical movement, values over 1.0 indicate demand in excess of capacity.
3 Control delay per vehicle, expressed in seconds, includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.
4 level of service of the critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn; Neg = negligible; N/A = not applicable

Table 4.1-79 Relocated Stoughton Station Unsignalized Intersection Capacity Analysis (Evening Peak Hour)

		No-Build Condition				Build Condition			
Location	Critical Movement	Demand[1]	v/c[2]	Delay[3]	LOS[4]	Demand	v/c	Delay	LOS
Porter Street at Washington Street	WB RT	25	0.08	13	B	25	0.08	13	B
Freeman Street at Washington Street	WB RT	15	0.14	32	D	15	0.12	27	D
Wyman Street at Washington Street	EB RT	140	0.50	26	D	90	0.32	20	C
Summer Street at Wyman Street	EB LT-RT	70	0.08	9	A	85	0.10	10	A
Brock Street at Washington Street	EB LT-TH-RT	155	>1.20	>120	F	295	>1.20	>120	F
	WB LT-TH-RT	70	>1.20	>120	F	115	>1.20	>120	F
	NB LT-TH-RT	490	0.10	3	A	490	0.13	4	A
	SB LT-TH-RT	820	0.01	0	A	810	0.01	1	A
Brock Street at Morton Street	EB LT-TH-RT	80	0.13	9	A	80	0.14	9	A
	WB LT-TH-RT	170	0.31	10	B	170	0.32	11	B
	NB LT-TH-RT	97	0.19	9	A	105	0.21	9	A
	SB LT-TH-RT	165	0.32	10	B	180	0.35	11	B
Brock Street at Wyman Street	WB LT-RT	120	0.16	9	A	125	0.16	9	A
Park Avenue/Sumner Street at Park Street	EB LT	125	>1.20	>120	F	125	>1.20	>120	F
	EB TH-RT	25	0.11	19	C	25	0.11	19	C
	WB LT-TH-RT	50	0.28	25	D	50	0.29	26	D
MBTA North Driveway at Morton Street	WB LT-RT	Does not exist				155	0.23	11	B
MBTA South Driveway at Brock Street	SB LT-RT	Does not exist				150	0.26	13	B

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour for unsignalized intersections
2 volume-to-capacity ratio for the critical movement, values over 1.0 indicate demand in excess of capacity.
3 Control delay per vehicle, expressed in seconds, includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay.
4 level of service of the critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn; Neg = negligible; N/A = not applicable

Queue Analysis- A queue analysis was conducted to compare the queues at signalized study area intersections under the No-Build Condition and the Build Condition. Table 4.1-80 presents the results of the analysis; complete results are provided in Appendix 4.1-K.

Table 4.1-80 Relocated Stoughton Station Vehicle Queue Analysis

			95th Percentile Queue[1]			
		Available Storage Length	No-Build Condition		Build Condition	
Location	Lane Group	(feet)	Morning Peak Hour	Evening Peak Hour	Morning Peak Hour	Evening Peak Hour
Porter Street at	EB RT	250	199	373	84	310
Washington Street	NB LT-LT	125	508	513	422	420
	NB TH	135	243	123	243	123
	SB TH	365	#316	859	#338	#886
	SEB RT-RT	650	#271	#277	#247	#277
Pleasant Street at Park Street/	NB TH-TH-RT	215	#447	#355	#418	#323
Washington Street	SB LT	110	#308	#228	#266	#202
	SB TH	130	#630	#715	#500	#637
	SB RT	165	63	151	61	149
	NE LT-TH	845	#456	287	#462	287
	SW RT	340	0	0	0	0

Source: Synchro 7 (Build 773, Rev 8) software
Note:
1 95th percentile queue length in feet
# 95th percentile volume exceeds capacity; queue may be longer.
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; RT = right-turn

When compared to the No-Build Condition, queue lengths for the Build Condition would be noticeably shorter at the intersection of Porter Street at Washington Street: for the eastbound Porter Street right-turn lane during the evening peak hour and for the northbound Washington Street left-turn lane during both morning and evening peak hours.

At the intersection of Pleasant Street at Park Street/Washington Street, northbound Park Street queue lengths would be noticeably shorter during the morning and evening peak hours. The reduction in queue lengths is attributed to the redistribution in traffic on study area roadways that would result from relocating the Stoughton Station.

Pedestrians and Bicycles-The travel demand and ridership estimates completed by CTPS indicate that approximately 220 additional pedestrians/bicycle trips would be expected daily under the Build Condition. With the relocation of Stoughton Station, pedestrians will likely access the station via Morton Street, Brock Street and Washington Street. Currently, sidewalks are provided on the east side of Morton Street, north side of Brock Street and along both sides of Washington Street. The majority of the infrastructure needed to support pedestrian and bicycle access to the proposed station exists currently and would not be adversely impacted by the change in the number of pedestrians within the study area.

Signal Warrant Analysis- A signal warrant analysis was conducted to determine whether a traffic signal should be installed at the intersection of Washington Street at Brock Street. This intersection is expected to see a substantial increase in traffic volume due to relocating the Stoughton Station. The analysis showed that a signal is warranted at this intersection due to traffic volume.

Easton Transportation Impacts (Stoughton and Whittenton Alternatives)

There are two stations planned in Easton:

- North Easton, which would be located on the Stoughton town line off Roche Bros Way
- Easton Village, which would be located off Sullivan Street just south of Oliver Street

Access to the proposed North Easton Station would be via the existing signalized Roche Brothers Shopping Center driveway located on Route 138 just south of the Stoughton town line. This driveway would serve vehicular and bicycle users. A sidewalk would be constructed along the access road to provide access for pedestrians.

At Easton Village, access to the proposed station would be provided via a drop-off loop on Sullivan Street. No parking is proposed for this station. Pedestrian and bicycle access would be provided via Sullivan Street and Oliver Street.

Traffic Operations—Design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within Easton. These site generated volumes were added to the No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are depicted in Figures 4.1-66 and 4.1-67.

The projected number of vehicle trips in and out of the North Easton and Easton Village stations during the morning and evening peak hours are shown in Table 4.1-81. No park-and-ride trips are projected at Easton Village because no commuter parking is planned for that station, however 12 spots will be dedicated for kiss & ride accommodations within an existing private lot. The trip generation for the North Easton station is based on projected ridership on the Stoughton Alternative.

Table 4.1-81 Park-and-Ride and Vehicular Drop-Off Vehicle Trips:[1] Easton Stations (Stoughton and Whittenton Alternatives)

		Morning Peak Hour		Evening Peak Hour	
Station	**Type of Trip**	**In**	**Out**	**In**	**Out**
North Easton	Park-and-Ride	239	31	27	234
	Drop-off	27	27	26	26
	Total Vehicles	266	58	53	260
Easton Village	Park-and-Ride	0	0	0	0
	Drop-off	44	44	32	32
	Total Vehicles	44	44	32	32

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of drop-off vehicle trips assumes one rider per vehicle.

The directional distribution of station-generated traffic is a function of population distribution, vehicle-owning households, existing travel patterns on area roadways, and traffic conditions. The trip distribution for the park-and-ride trips associated with North Easton Station is based on ridership data provided by CTPS, which take into account these factors. Table 4.1-82 provides the geographic distribution of these trips.

Table 4.1-82 Easton Trip Distribution (Stoughton and Whittenton Alternatives)

To/From	Distribution
North	25%
South	18%
East	25%
West	32%

Source: CTPS Travel Demand Model.

The park-and-ride traffic was distributed to the study area roadways based on these percentages. Drop-off traffic was added separately and is based on existing travel patterns on area roadways near the proposed station locations. Only drop-off traffic was generated by Easton Village Station.

The intersection levels of service based on the addition of rail related traffic are shown in Table 4.1-83.

Table 4.1-83 Easton Intersection Capacity Analysis –2030 Build Conditions vs. 2030 No-Build Conditions (Stoughton and Whittenton Alternatives)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	Build			No-Build	Build		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS[1]	V/C	Delay	LOS
North Easton Station								
Rt. 138 at Roche Bros. Way	B	0.98	38	D	B	0.76	21	C
Rt. 138 at Main St.	F	>1.00	>80	F	E	>1.00	74	E
Easton Village Station								
Rt. 138 at Belmont St. (Rt. 123)	D	0.90	67	E	F	>1.00	>80	F
Rt. 138 at Roosevelt Circle	A	0.66	7	A	B	0.84	20	B
Unsignalized Intersections	LOS	Critical Movement	Delay[4]	LOS	LOS	Critical Movement	Delay	LOS
North Easton Station								
Rt. 138 at Elm St.	F	Elm WB All	>50	F	F	Elm WB All	>50	F
Rt. 138 at Union St.	F	Union WB L/R	>50	F	F	Union WB L/R	>50	F
Easton Village Station								
Elm St. at Main St	B	Elm WB L/R	14	B	B	Elm WB L/R	18	C
Center St. at Main St. at Lincoln St.	F	Center NB All	>50	F	F	Center NB All	>50	F
Lincoln St. at Barrows St.	B	Barrows NB All	12	B	D	Barrows NB All	>50	F

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Two signalized and two unsignalized locations were analyzed for the North Easton station under Build conditions. The signalized intersection of Roche Bros Way and Route 138, which provides access to the train station, would operate at acceptable levels of service during both the morning and evening peak hours. The other three locations would operate at LOS E or F. Two signalized and three unsignalized locations were analyzed for the Easton Village station. The signalized intersection of Route 138 at Belmont Street would decline to a deficient LOS, from LOS D to LOS E, during the morning peak hour and remain at LOS F during the evening peak hour. Only one change in LOS is expected at the unsignalized locations; Lincoln Street at Barrows Street is expected to become deficient, declining from LOS D under No-Build to LOS F for Build conditions.

Traffic Signal Warrants—Three intersections were evaluated against the traffic signal warrant for the peak hour period:

- Route 138 at Elm Street
- Route 138 at Union Street
- Main Street at Center Street

The intersections of Route 138 at Elm Street and Route 138 at Union Street meet the requirements set forth by the MUTCD for traffic signal installation based on future peak hour traffic volumes.

The Main Street at Center Street intersection is projected to meet peak hour traffic signal warrants with or without the South Coast Rail project. With the adjacent historic Rockery, a Civil War memorial, a traffic signal system with the required lane configurations cannot be installed, as impacts to the historic property could not be avoided.

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 180 pedestrian/bicycle trips would access North Easton Station (Stoughton and Whittenton Alternatives) on a daily basis, which would increase pedestrian and bicycle activity in the vicinity of Route 138. At Easton Village Station (Stoughton and Whittenton Alternatives), approximately 240 pedestrian/bicycle trips would be expected. The majority of the infrastructure needed to support pedestrian and bicycle access to both proposed stations exists currently and would not be adversely impacted by the change in number of pedestrians on study area roadways.

The intersections of Route 138 at Elm Street and Route 138 at Union Street meet the requirements set forth by the MUTCD for traffic signal installation based on future peak hour traffic volumes.

The Main Street at Center Street intersection is projected to meet peak hour traffic signal warrants with or without the South Coast Rail project. With the adjacent historic Rockery, a Civil War memorial, a traffic signal system with the required lane configurations cannot be installed, as impacts to the historic property could not be avoided.

Traffic signal timing and phasing changes would be required at the North Easton Station driveway intersection with Route 138 to accommodate pedestrian demands. Pedestrian phases would also be included at the newly signalized intersections of Route 138 at Elm Street and Route 138 at Union Street. These changes are discussed further in Section 4.1.5, Mitigation Measures.

To accommodate increased pedestrian demand at Easton Village Station, crosswalks would be restriped at the intersections of Main Street at Center Street, Lincoln Street at Barrows Street, and Main Street at Barrows Street. At the Main Street at Center Street intersection, a high visibility crosswalk with a passive flashing pedestrian crossing sign would also be installed at the Main Street crosswalk. Sidewalks and crosswalks elsewhere in the vicinity of Easton Village Station are adequate to handle the expected demand.

Neither of the proposed station locations would physically alter designated bicycle facilities nor disrupt future plans for either on-road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—The North Easton Station (Stoughton and Whittenton Alternatives) is proposed to have 509 parking spaces (12 of these handicapped accessible). The proposed project would not physically alter the existing public parking supply or impact parking availability within Easton in the vicinity of the North Easton Station. Based on the projected daily park-and-ride trips, the peak parking demand for North Easton Station is 520 spaces.

Ten vehicular drop-off parking spaces are proposed at Easton Village Station (Stoughton and Whittenton Alternatives). These parking spaces would be shared with the Easton Historical Society. The existing on-street parking supply in the vicinity of Easton Village is vulnerable to unauthorized use by commuters. Parking limit signage and increased enforcement may be needed to ensure parking is being properly utilized.

Raynham Transportation Impacts (Stoughton and Whittenton Alternatives)

The Raynham Park Station site (Stoughton and Whittenton Alternatives) is west of Route 138 just south of the Raynham-Taunton Greyhound Park. Access for all users would be provided via a newly signalized intersection with Robinson Road. Robinson Road would be realigned slightly to the north to create a four-way intersection with the station driveway.

Traffic Operations—Design year (2030) Build condition traffic volumes for the study area roadways were determined by estimating site-generated traffic volumes and distributing these volumes over study area roadways within Raynham. These site-generated volumes were added to the No-Build traffic volumes to create the 2030 Build condition traffic volume networks, which are shown in Figures 4.1-68 and 4.1-69.

The projected number of vehicle trips in and out of the Raynham Park Station (Stoughton and Whittenton Alternatives) during the morning and evening peak hours is shown in Table 4.1-84. The trip generation for this station is based on ridership projections for the Whittenton Alternative which generates the highest ridership projections for the Raynham Park Station.

The directional distribution of station-generated traffic is a function of population distribution, vehicle-owning households, existing travel patterns on area roadways, and traffic conditions. The trip distribution for the park-and-ride trips associated with the Raynham Park Station is based on ridership data provided by CTPS, which take into account these factors. Table 4.1-85 provides the geographic distribution of these trips.

Table 4.1-84 Park-and-Ride and Drop-off Vehicle Trips:[1] Raynham Park Station (Stoughton and Whittenton Alternatives)

		Morning Peak Hour		Evening Peak Hour	
Station	**Type of Trip**	**In**	**Out**	**In**	**Out**
Raynham	Park-and-Ride	183	21	17	166
	Drop-off	32	32	25	25
	Total Vehicles	215	53	42	191

1 The number of park-and-ride vehicle trips is calculated by dividing the number of park-and-ride riders by a 1.05 vehicle occupancy rate (VOR). The number of drop-off vehicle trips assumes one rider per vehicle.

Table 4.1-85 Raynham Park Station Trip Distribution (Stoughton and Whittenton Alternatives)

To/From	Distribution
North	5%
South	31%
East	15%
West	49%

Source: CTPS Travel Demand Model.

The park-and-ride traffic was distributed to the study area roadways based on these percentages. Drop-off traffic was added separately and is based on existing travel patterns on area roadways near the proposed station locations.

The intersection levels of service based on the addition of rail related traffic are shown in Table 4.1-86. All six signalized intersections would continue to operate at acceptable levels of service under Build conditions. There would be no change in levels of service at two of the three unsignalized intersections. During the morning peak hour, the intersection of Route 138 at Wilbur Street would decline from LOS E under No-Build to LOS F. Operations during the evening would remain unchanged. The unsignalized intersection of the existing driveway with Route 138, which would also serve as the station driveway, would continue to operate at LOS F. The operational discussion of the proposed traffic signal at Route 138 and Robinson Street/Station Driveway is discussed in Section 4.1.5, Mitigation Measures.

Table 4.1-86 Raynham Park Station Intersection Capacity Analysis–2030 Build Conditions vs. 2030 No-Build Conditions (Stoughton and Whittenton Alternatives)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	No-Build	**Build**			**No-Build**	**Build**		
Signalized Intersections	**LOS**[1]	**V/C**[2]	**Delay**[3]	**LOS**	**LOS**	**V/C**	**Delay**	**LOS**
Raynham Park Station (Stoughton and Whittenton Alternatives)								
Rt. 138 at Rt. 106 (Foundry St., Easton)	C	0.92	27	C	D	>1.00	48	D
Rt. 138 at Elm St.	B	0.80	20	C	B	0.82	18	B
Rt. 138 at I-495 NB Ramps	B	0.70	16	B	B	0.86	19	B
Rt. 138 at I-495 SB Ramps	C	0.98	37	D	B	0.72	16	B
Rt. 138 at Carver St.	C	0.90	23	C	D	>1.00	50	D
Rt. 138 at Center St.	A	0.61	9	A	C	0.96	24	C
Unsignalized Intersections	**LOS**	**Critical Movement**	**Delay**[4]	**LOS**	**LOS**	**Critical Movement**	**Delay**	**LOS**
Raynham Park Station (Stoughton and Whittenton Alternatives)								
Rt. 138 at Wilbur St.	E	Wilbur WB L/R	>50	F	E	Wilbur WB L/R	47	E
Rt. 138 at Britton St. (East)	F	Britton WB L/R	>50	F	F	Britton WB L/R	>50	F
Rt. 138 at Britton St. (West)	F	Britton EB L/R	>50	F	F	Britton EB L/R	>50	F
Rt. 138 at Robinson St.	D	Robinson WB L/R	40	E	B	Robinson WB L/R	13	B
Rt. 138 at Dog Track Driveway	D	Driveway EB L/R	>50	F	E	Driveway EB L/R	>50	F

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 average control delay for the critical movement, rounded to the nearest whole second, for unsignalized intersections
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Traffic Signal Warrants—Two intersections for the Raynham Park Station (Stoughton and Whittenton Alternatives) were evaluated against the traffic signal warrant for the peak hour period:

- Route 138 at Station Driveway
- Route 138 at Wilbur Street

The intersection of Route 138 at the proposed Station Driveway meets the requirements set forth by the MUTCD for traffic signal installation based on future peak hour traffic volumes. The Route 138 at Wilbur Street intersection does not meet peak hour traffic signal warrants based on the projected future traffic volumes.

Pedestrians and Bicycles—The travel demand and ridership estimates completed by CTPS indicate that about 140 pedestrian/bicycle trips would access Raynham Park Station (Stoughton and Whittenton Alternatives) on a daily basis, which would increase pedestrian activity along Route 138 and within the neighborhood to the east of Route 138. To accommodate pedestrian demands, a pedestrian phase would be incorporated into the signalized station driveway entrance to the site. Installation of this signal also requires the realignment of Robinson Street slightly to the north. It is expected that the crossing and roadway realignment would encourage the use of Robinson Street, a low volume roadway, as a pedestrian route rather than the more congested Route 138.

The proposed station location would not physically alter designated bicycle facilities or disrupt future plans for either on-road or off-road facilities in the study area. To accommodate demand, bicycle parking and storage locations would be maximized using available space.

Parking—Raynham Park Station (Stoughton and Whittenton Alternatives) is proposed to have 448 parking spaces (of which eight would be handicapped accessible). An additional seven parking spaces would be reserved for drop-off activity. The proposed project would not physically alter the existing public parking supply or impact parking availability within Raynham. Based on the projected daily park-and-ride ridership, the parking supply would be sufficient to meet the peak parking demand for 400 spaces.

Layover Facilities

The proposed overnight layover facilities would only generate traffic associated with MBTA personnel. Due to the low number of trips anticipated, any impacts on traffic would be negligible and do not warrant detailed analysis.

Temporary Construction Impacts

The Build Alternatives have the potential to cause temporary disruptions in local access and mobility during the construction period as a result of temporary street closures and detours. Temporary street closures could be required to make improvements to the grade-crossings, such as new crossing gates, modifications to intersections and construction of stations. Construction activities would also generate additional traffic related to construction employee commutes, and the transport of materials and equipment by truck. As part of that phase, MassDOT will develop transportation management plans to detour traffic around construction areas. These transportation management plans will be closely coordinated with the cities and towns affected by each construction element, including emergency response representatives. A robust outreach program would be developed, notifying the public of construction activities through telephone calls, email blasts, website notices, and flyer distributions. Public information meetings would be conducted, identifying bridge construction and roadway closure locations, intersection construction activities, construction schedules, and temporary traffic, safety, and pedestrian detours through construction areas. For additional information on the construction staging plans, refer to Appendix 3.2-F.

4.1.5 Mitigation Measures

This section discusses safety and mitigation measures associated with grade crossing impacts. In addition, the LOS results completed as part of the Build impact analysis identify locations where the proposed stations are likely to cause traffic operations on the local roadway network to degrade. Specific mitigation measures that could be undertaken by MassDOT, as discussed below, were developed to offset these impacts and ensure adequate access to the proposed stations. In the case where structural changes to the roadway and traffic control devices are proposed, the mitigation aims to improve traffic flow with minimal impacts to adjacent land uses and at reasonable cost. The benefit of these changes is noted in the discussions below. The traffic mitigation measures are presented by municipality and station.

4.1.5.1 Grade Crossings

The following components and characteristics are being considered to optimize safety at the proposed South Coast Rail at-grade crossings:

- **Vehicle Type and Condition**. At-grade crossings would be designed to anticipate different vehicle types (passenger cars, trucks, buses). All rail vehicles would be required to undergo frequent inspection programs to ensure each vehicle in active service is maintained to meet current safety standards in an effort to remove the possibility of equipment or materials falling off the vehicles at grade crossings, or the vehicles from breaking down in an at-grade crossing.

- **Geometry**. At-grade crossings would be designed with minimum curvature or profile changes to allow for optimal sight lines, allowing drivers more time to safely stop before the crossing. Some existing at-grade crossings would be closed in some locations to optimize safety, as noted in Section 4.1.4, Proposed At-Grade Crossings.

- **Signage and Markings**. All traffic control devices (such as highway signage, markings and devices, etc.) would be designed in compliance with the MUTCD[16]. Signs and markings would be placed a sufficient distance from the crossings to allow adequate warning to motorists and pedestrians.

- **Crossing Surface**. The condition of the roadway in the vicinity of the at-grade crossing and the condition of the track would be maintained at existing standards by maintaining the road surface and rail seal.

- **Site Conditions.** Physical obstructions in the vicinity of each crossing, such as trees and vegetation, buildings, signal cases and bungalows, signs, hills, fences, walls and parked vehicles, would be minimized or eliminated to provide drivers with optimal sight lines.

- **Illumination.** Visibility of the train and the general visibility of an at-grade crossing are important elements that would be considered. Methods for illumination would include lights and reflectorization of the train, and/or lighting at the at-grade crossings (i.e. street lights).

- **Traffic Signal Preemption.** Where a signalized intersection is located within 200 feet of an at-grade crossing, traffic signal preemption would be used to ensure that vehicle queues are cleared in advance of the train.

- **Signals and Operations.** A traditional at-grade crossing is made up of several types of warning devices. A bell serves as an audible warning that the gates would begin their downward track. At the same time the bell is initiated, the flashers both on the flasher pole and the gate arm are activated. This is a visual warning for the motorist that the gates would begin their descent. The MUTCD requires a minimum of 20 seconds of warning time at at-grade crossings. Both of these would be used to ensure proper visual and audible warnings for motorists.

[16] US Department of Transportation. Federal Highway Transportation. *Manual on Uniform Traffic Control Devices for Street and Highways*. May 2012. Web. Apr.-May 2012. <http://mutcd.fhwa.dot.gov/pdfs/2009r1r2/mutcd2009r1r2edition.pdf>

- **Gated Warning Devices.** Commonly used throughout the country. The gates are made out of a fiberglass resin, which is designed to break away should emergency vehicles or other vehicles need to drive through the gates. Gated crossings are typically outfitted with flasher units and bells for visual and audible warning devices.

- **Gate Timing.** Traditionally, railroad and transit agencies allow for 30 seconds of warning time, an additional 10 seconds over the MUTCD's requirements. This is generally due to varying conditions at an at-grade crossing, including gate lengths, wind conditions, weather condition and varying maintainer adjustments. This allowance would be used at at-grade crossings for the South Coast Rail project.

- **Vital Logic.** Vital railroad signal logic, equipment that identifies the train speed and location through circuitry in the rails and onboard computers in the locomotive, would be used at at-grade crossings to identify the direction of an approaching train, identify any hazards in the crossing, and create a failsafe that would close the gates automatically in the event of an emergency.

- **AHCW Systems.** Each proposed public and private at-grade crossing would be suitable for public use if equipped with a combination of new, state-of-the-art, Automatic Highway Crossing Warning (AHCW) systems and designed with minor geometric modifications (such as driveway reconfiguration, driveway closures, vegetation clearing and utility pole relocations). The advanced warning system would communicate with the MBTA Operational Control Center (OCC) and would allow MBTA train dispatchers to communicate with and receive indications directly from each at-grade crossing.

- **General Safety Enhancements.** Recommended at all South Coast Rail at-grade crossings that are proposed to remain active. These measures include:

  - Remove gates and signals at existing crossings and replace them with new gates, signals, and signal cases;

  - Remove vegetation at all at-grade crossings to improve sight distance;

  - Evaluate the need for guardrails at each location during final design; and

  - Evaluate the need to remove or relocate utility poles, walls, boulders and fences during final design.

In addition to the general improvements listed above, additional site specific improvements are recommended. These improvements range from minor (installing traffic signal pre-emption at existing intersections) to major construction (potential at-grade separation). These recommended improvements are summarized in Table 4.1-87 and briefly described subsequently.

Table 4.1-87 Stoughton Alternatives Proposed At-Grade Crossing Improvements

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
STOUGHTON LINE		
Canton	Washington Street	Install a traffic signal pre-emption system at two intersections in proximity of the crossing
	Pine Street	Relocate existing driveway to the north
	Will Drive	General improvements
Stoughton	Central Street	Relocate existing driveway to the west
		Coordinate crossing operation with fire station located 400 feet west
		Extend sidewalk through the crossing
		Install crosswalk across the Central Street eastbound approach to the crossing
	Simpson Street	General improvements
	School Street	Modify alignment at Cushing Street
	Porter Street (Route 27)	General improvements
	Wyman Street	Reconfigure parking lot and driveway
	Brock Street	Investigate installation of a traffic signal with pre-emption system at nearby intersection
		Reconfigure driveway to the east and relocate driveway to the west
	Plain Street	Investigate installation of a traffic signal with pre-emption system at nearby intersection
		Relocate driveways to the east
	Morton Street	Close Morton Street
		Construct frontage road to Totham Farm Road
Easton	Elm Street	Relocate driveway to the east
	Oliver Street	Relocate driveways to the northwest
		Relocate children's play area
		Extend sidewalk through crossing
	Gary Lane	Install gates and locks
	Short Street	General improvements
	Depot Street (Route 123)	Reconfigure driveway to the west
	Purchase Street	General improvements
	Prospect Street	General improvements
	Foundry Street (Route 106)	General improvements
Raynham	Race Track Crossing	General improvements
	Elm Street	General improvements
	Carver Street	Reconstruct culvert
	Broadway (Route 138)	At-grade separation
	Britton Street	General improvements
	King Philip Street	Relocate driveways
	East Britannia Street	General improvements
Taunton	Longmeadow Road	Reconfigure or close driveways
	Dean Street (Route 44)	Reconstruct Dean Street/Arlington Street traffic signal system
		Install traffic pre-emption phasing at Dean Street/Arlington Street
NEW BEDFORD MAIN LINE		
	Ingell Street	Close driveway to the west

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
	Hart Street	General improvements
Berkley	Cotley Street	General improvements
	Padelford Street	General improvements
	Myricks Street (Route 79)	General improvements
Lakeville	Malbone Street	General improvements
Freetown	Chace Road	Reconfigure or close driveway to the west
	Braley Road	General improvements
	East Chipaway Road	General improvements
New Bedford	Samuel Barnet Road	General improvements
	Pig Farm Road	General improvements
	Tarkiln Hill Road	Close Tarkiln Hill Road and reroute traffic through Stop & Shop driveway
		Signal pre-emption at King's Highway / Stop & Shop driveway
		Signal pre-emption at Tarkiln Hill Road / Church Street
		At-grade crossing pre-signals
	Nash Road	Signal pre-emption at Church Street / Nash Road
		At-grade crossing pre-signals
FALL RIVER SECONDARY		
Berkley	Mill Street	Close crossing
	Adams Lane	Close crossing
Freetown	Beachwood Road	Close crossing
	Richmond Road/Route 79 (North)	General Improvements
	Richmond Road/Route 79 (South)	Reconfigure driveway to the west
	Forge Road (North)	Close Forge Road
	Forge Road (South)	General improvements
	Elm Street	General improvements
	High Street	General improvements
	Copicut Road	General improvements
	Brightman Lumber	General improvements

The specific improvements within each municipality under the Stoughton Alternatives are described below. Except for the Longmeadow Rd. and Dean St. (Route 44) crossings in Taunton, these crossings are also part of the Whittenton Alternatives.

- **Canton**. Three at-grade crossings (Washington Street, Pine Street, and Will Drive) are located in Canton along the active commuter rail line. The construction of a second track along this section of the alignment and increased train activity would not result in substantial changes in traffic conditions or queue lengths at these crossings. As part of the proposed South Coast Rail project, traffic signal preemption is recommended at the intersection of Washington Street and Revere Street to address queuing that may extend over the tracks during the peak hours.

- **Stoughton**. Eight public at-grade crossings in Stoughton would be affected. Five of these at-grade crossings (Central Street, Simpson Street, School Street, Porter Street, and Wyman

Street) are active commuter rail at-grade crossings that would be modified to allow double-track operations. The addition of a second track and additional trains would result in negligible changes in traffic conditions or queue lengths at these crossings. A sixth crossing, at Brock Street, is considered active and has working signals but is rarely used today. For the purposes of this analysis, Brock Street is considered a reactivated crossing. A seventh crossing is proposed at Plain Street. An existing at-grade crossing at Morton Street would be closed and traffic would be rerouted to a proposed street that would run parallel to the proposed track and cross to the south underneath the track at a bridge.

- **Easton**. Eight currently inactive public at-grade crossings are located in Easton. All of the crossings in Easton would be reactivated as part of the South Coast Rail project. The Main Street crossing is currently grade separated and a new bridge that passes over the rail right-of-way will be constructed. A previous bridge at this location has been filled in; therefore, the new bridge would either be constructed on new abutments or the existing abutments that remain, and the embankment excavated to track grade below.

- **Raynham**. Six public at-grade crossings and one private crossing, all inactive, are located in Raynham. Five public at-grade crossings would be reactivated as part of the South Coast Rail project. The private crossing at the Race Track would also be reactivated as part of the South Coast Rail project. A sixth public at-grade crossing, across Broadway (Route 138), is projected to have relatively high traffic volumes and is recommended for at-grade separation to minimize traffic impacts along this section of Route 138.

- **Taunton.** Four public at-grade crossings are located in Taunton. Both the Ingell Street and Hart Street crossings are currently active crossings with freight train activity. These crossings would be upgraded to accommodate the proposed commuter rail trains. The at-grade crossing at Longmeadow Road would be reactivated as part of the South Coast Rail project. The Dean Street (Route 44) at-grade crossing is active with freight rail activity a few times a week. Similarly to Main Street, the Thrasher Street crossing is currently grade separated and a new bridge that passes over the rail right-of-way will be constructed. A previous bridge at this location has been filled in; therefore, the new bridge would either be constructed on new abutments or the existing abutments that remain, and the embankment excavated to track grade below.

- **Berkley.** Four existing public at-grade crossings and one private at-grade crossings are located in Berkley. Cotley Street, Padelford Street, Myricks Street (Route 79), and Mill Street currently carry active freight traffic. Mill Street is proposed to be closed. The three other crossings would be upgraded to accommodate the proposed commuter rail trains. Adams Lane, a private at-grade crossing, is also proposed to be closed

- **Lakeville.** One public at-grade crossing is located in Lakeville. The crossing at Malbone Street currently carries active freight traffic. This crossing would be upgraded to accommodate the proposed commuter rail trains.

- **Freetown.** Ten public at-grade crossings, two of which have a northern and southern section, and one private at-grade crossing in Freetown currently carry active freight traffic. The northern part of Forge Road would be closed and the remaining ten crossings would be upgraded to accommodate the proposed commuter rail trains. Seven of these crossings are expected to cause minor delays and have little impact on the surrounding roadways.

- **New Bedford.** Three public at-grade crossings (Samuel Barnet Road, Tarkiln Hill Road, and Nash Road) and one private at-grade crossing (Pig Farm Road) currently carry active freight traffic and would be upgraded to accommodate the proposed commuter rail service.

- **Fall River.** There are no at-grade crossings in Fall River. All major street crossings within Fall River are grade-separated and all remaining private roadways crossings are expected to be closed.

Additional mitigation commitments specific to the Attleboro Secondary portion of the Whittenton Alternatives are summarized in Table 4.1-88. The Bay Street crossing is currently grade separated. The Bay Street Bridge has been filled in and would need to be reconstructed to provide adequate track clearance for the rail service. A new superstructure would be constructed on new abutments and the embankment fill excavated below to the proposed track grade.

Table 4.1-88 Attleboro Secondary Recommended Grade Crossing Mitigation Improvements (Whittenton Alternatives)

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
Taunton	Tremont Street	Reconfigure driveway to the north
	Oak Street	Optimize existing pre-emption at Oak Street / Tremont Street
	Porter Street	Reconfigure driveway to the east
	Cohannet Street	Reconfigure or close driveways adjacent to the tracks
	Winthrop Street	Additional advance RR warning signs
	Somerset Avenue	Investigate installation of a traffic signal with pre-emption system at nearby intersection
	Weir Street	Close McSoley Street
		Close and reconstruct driveway to the west
		Close and reconstruct driveway to the east

MBTA Grade Crossing Safety Policies and Programs

MBTA Safety Department officials are regularly in the field inspecting stations, buses, subways, commuter rail and boats to ensure a safe environment. All stations and vehicles have direct communication lines to the MBTA's Operations Control Center and stations are being upgraded with modernized public address systems and closed-circuit television camera systems. MBTA personnel are trained in emergency response and their safety program (coordinated with local, state, federal law enforcement agencies, as well as the MBTA Police) includes a schedule of simulated emergency response exercises geared toward preparing MBTA personnel to be equipped with state-of-the-art emergency response techniques.

The MBTA Safety Department tracks all accidents and incidents throughout the MBTA system and is responsible for reporting all required safety and security data to the National Transit Database (NTD)17 and the Department of Public Utilities (DPU). The NTD is maintained by the Secretary of Transportation, per Title 49 U.S.C. 5335(a) SECTION 5335 National transit database. This data is used by the MBTA to measure safety on the MBTA and by the Federal Transit Administration (FTA) to track incident trends in the industry. The MBTA posts a monthly incident report on their public website. NTD reportable incidents are also posted on the NTD website at: http://www.ntdprogram.gov/ntdprogram/.

In order to minimize incidents within the system, the MBTA Safety Department has undertaken and/or maintains the following measures:

- Performs routine safety audits of all transit stations to note and correct safety hazards.
- Increased the number of track and platform audits.
- Performs audits of tunnel lighting.
- Established a zero tolerance policy pertaining to use of cell phones and all other electronic devices while operating an MBTA vehicle.
- Established the Safety and Operations Rules Compliance Program, which has performed over 2000 safety observations.

Commuter Rail Safety Education

Similar to the MBTA Greenbush Line project, the South Coast Rail project will require a comprehensive grade-crossing safety awareness program.

The MBTA will educate the public using the “Operation Lifesaver” program at least one year prior to the scheduled revenue operation date. “Operation Lifesaver” is a national non-profit organization whose program is available to any transit agency who is seeking to improve safety and education for communities that contain rail traffic. The program’s railroad safety information and specially trained personnel can be used to train others to educate communities. The primary focus of the program is to communicate the importance of railroad public awareness, the potential hazards at highway/rail at-grade crossings, and the dangers of trespassing on railroad right-of-way.

During the design and early construction phases of the South Coast Rail project, the MBTA will:

- Train various groups and individuals, including students and community organizations, police officers, fire fighters, school officials, and agency staff.
- Conduct direct public contact through marketing, presentations, mass mailing, press releases, and conducting special safety fairs in every affected city and town.
- Training fire fighters and emergency response personnel in Emergency Evacuation Procedures.

[17] Title 49 U.S.C. 5335(a): SECTION 5335 National transit database. http://www.ntdprogram.gov/ntdprogram/ntd.htm.

4.1.5.2 Stations

New Bedford (Both Rail Alternatives)

The following intersection improvements are required to mitigate existing deficiencies at critical locations or adverse impacts caused by the alternatives. Table 4.1-89 presents a comparison of Build Alternatives without and with mitigation operations to illustrate the benefit of the proposed mitigation. The Mitigation associated with the Whale's Tooth and King's Highway stations are proposed as part of the Whittenton and Stoughton Alternatives.

Whale's Tooth Station Area Traffic Mitigation (Both Rail Alternatives)

Acushnet Avenue at Hillman Street—A pedestrian crosswalk is proposed at this location to accommodate the projected pedestrians. The crosswalk would be installed across the southern Acushnet Avenue approach to the intersection and provide a connection to the station from the residential area located to the west of Route 18.

Acushnet Avenue Sidewalk—Installation of a 6-foot wide sidewalk along the eastern side of Acushnet Avenue is proposed to complete the pedestrian connection from Hillman Street. The 300-foot long sidewalk would be between Hillman Street and the proposed station driveway.

Table 4.1-89 New Bedford Intersection Capacity Analysis –2030 Build with Mitigation Conditions vs. 2030 Build Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Whale's Tooth Station								
Mill Street at Pleasant Street	F	0.76	47	D	E	0.93	64	E
Coggeshall Street at N. Front Street[4]	F	0.66	12	B	F	0.71	14	B
Coggeshall Street at Purchase Street[4]	F	0.53	13	B	F	0.62	14	B
King's Highway Station								
King's Highway at Route 140 NB Ramps	C	0.60	20	C	C	0.89	29	C
Church Street at Tarkiln Hill Road	C	0.74	24	C	D	0.79	29	C
King's Highway at Stop & Shop Driveway	A	0.52	9	A	B	0.82	29	C
King's Highway at King's Highway Station (Shaw's) Driveway	A	0.41	6	A	A	0.55	9	A
King's Highway at Mt. Pleasant Street	C	0.54	24	C	E	0.93	42	D

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 Unsignalized in the Build condition
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Mill Street at Pleasant Street and Kempton Street—Signal timing adjustments are proposed to accommodate project related pedestrians and traffic at this location. Signal timing changes would be made to provide a longer crossing interval for the exclusive pedestrian phase. The proposed mitigation at this location would improve operations to LOS D during the morning peak hour.

Coggeshall Street at North Front Street—This unsignalized intersection processes a high amount of traffic and operates at LOS F during the morning and evening peak hours with or without the project. To offset project related traffic at this intersection, a traffic signal would be installed. The proposed signal would be designed to operate with two phases, the first phase servicing Coggeshall Street and the second phase for North Front Street.

Pedestrian crossings would occur concurrently with these phases. With the proposed improvement, the intersection of Coggeshall Street at North Front Street would operate at LOS B during both the morning and evening peak hour.

As Coggeshall Street is under the jurisdiction of the City of New Bedford, any improvements to this intersection will require review and authorization by the City of New Bedford. Should these improvements be desired, MassDOT could contribute to the construction or implementation of these intersection improvements based on their fair share of the impacts to the intersection.

Coggeshall Street at Purchase Street—This unsignalized, all-way STOP controlled intersection processes a high amount of traffic and operates at LOS F during the evening peak hour with or without the project. To improve the identified safety issues at this location as well as offset project related traffic impacts, a traffic signal would be installed at this location. The proposed signal would be designed to operate with three phases, the first phase servicing Purchase Street and the second phase exclusively for pedestrian crossings and the third phase for Coggeshall Street. With the proposed improvement, the intersection of Coggeshall Street at Purchase Street would operate at LOS B during both the morning and evening peak hour.

As Coggeshall Street is under the jurisdiction of the City of New Bedford, MassDOT has coordinated the design of this intersection with the city as part of the *Freight Railroad Bridge Improvement Project, Rehabilitation of Bridges over Deane Street, Sawyer Street, and Coggeshall Street*. This bridge rehabilitation project, which is functionally independent of the South Coast Rail project, received $20 million in Transportation Infrastructure Generating Economic Recovery (TIGER) federal funding, part of which will help improve the signal at Deane Street at Purchase Street and install new signals at the intersections of Purchase Street at Sawyer Street and Purchase Street at Coggeshall Street.

King's Highway Station Traffic Mitigation (Both Rail Alternatives)

King's Highway Corridor—To accommodate project traffic, interconnection and coordination of the traffic signals along King's Highway is proposed. Signal controller upgrades, interconnection infrastructure (conduit/cable), signal timing and phasing improvements would be required at the following locations:

- Mount Pleasant Street at Jones Road/King's Highway
- King's Highway at Shaw's Driveway
- King's Highway at Route 140 Northbound Ramps

- King's Highway at Stop & Shop Driveway
- Tarkiln Hill Road at Church Street

Mount Pleasant Street at Jones Road/King's Highway—To improve traffic operations and pedestrian crossing times at this location, traffic signal phasing would be revised to provide a permissive eastbound/westbound phase. Traffic signal timings would be modified to support the new phasing. Signal timing and phasing changes will allow this intersection to return to acceptable traffic operations during the evening peak hour.

King's Highway at Shaw's Driveway—To facilitate pedestrian movements at this intersection a crosswalk would be provided across the Shaw's Driveway entrance. Concurrent pedestrian phasing would be provided to facilitate the pedestrian crossing. During the evening peak hour, traffic operations degrade from LOS A to LOS B in order to accommodate pedestrians. However, the intersection would still operate at an acceptable LOS during both peak hours.

King's Highway at Stop & Shop Driveway—Several changes are recommended for this location due to its proximity to the King's Highway grade crossing. The traffic signal would be modified to allow for traffic signal pre-emption when the train approaches the station. Should the vehicle queue along King's Highway extend over the railroad tracks, the signal would operate such that the queue would clear prior to the train's arrival. Pre-signals would be required at the grade crossing to support this movement and prevent additional traffic from driving over the railroad tracks.

The intersection of Tarkiln Hill Road and King's Highway would be closed for safety purposes due to its proximity to the grade crossing. As shown on Figure 4.1-70, traffic currently turning into or out of Tarkiln Hill Road at this location would be diverted to Stop & Shop and enter Tarkiln Hill Road at the back of the property. Approximately 24 parking spaces associated with the Stop & Shop Plaza and Wendy's Restaurant would be impacted by this diversion of Tarkiln Hill Road. To maintain the fastest possible emergency response times, mountable curbing would be used to close the exiting intersection. In the event of an emergency, this curbing could be driven over by emergency responders.

Tarkiln Hill Road at Church Street—A concurrent pedestrian crossing phase is proposed for the intersection of Tarkiln Hill Road at Church Street. Signal timing changes would be required to accommodate pedestrian movements, but LOS would not be affected during either peak hour.

Similar to King's Highway at Stop & Shop Driveway, the traffic signal would also be modified to allow for traffic signal pre-emption when the train approaches the station. Pre-signals at the grade crossing would support this movement and prevent additional traffic from driving over the railroad tracks.

Freetown Station Area Traffic Mitigation (Both Rail Alternatives)

The following three pedestrian-related improvements are suggested to improve connectivity between residential areas within walking distance to the proposed Freetown Station. Freetown Station is proposed as part of the Whittenton and Stoughton Alternatives.

South Main Street

To facilitate pedestrian travel from the north, construction of a 6-foot sidewalk is proposed on the east side of South Main Street from the existing sidewalk's terminus at Stop & Shop to the station driveway (approximately 1,600 feet).

South Main Street at Narrows Road

The existing crosswalk across South Main Street at Narrows Road is proposed to be restriped. As part of this improvement, ADA/AAB compliant wheelchair ramps would be constructed at this location.

South Main Street at Copicut Street

A pedestrian crosswalk is proposed at this location. The crosswalk would be installed across Copicut Street on the east leg of the intersection. Compliant ADA/AAB wheelchair ramps are also proposed.

Fall River Station Area Traffic Mitigation (Both Rail Alternatives)

The following three intersection improvements are suggested to mitigate existing deficiencies at critical locations or adverse impacts caused by the alternatives. Table 4.1-90 presents a comparison of Build to Build with mitigation operations to illustrate the benefit of the proposed changes. Both Fall River Stations are proposed as part of the Whittenton and Stoughton Alternatives.

Table 4.1-90 Fall River Intersection Capacity Analysis–2030 Build with Mitigation Conditions vs. Build Conditions (both alternatives)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Fall River Depot Station								
President Avenue at N. Davol Street	B	0.62	26	C	C	0.84	32	C
N. Main Street at President Avenue	D	0.81	24	C	D	0.88	35	D

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections

Fall River Depot Station Area Traffic Mitigation (Both Rail Alternatives)

North Main Street and President Avenue—Intersection geometry, signal timing and phasing improvements are proposed for this location to accommodate project related pedestrians and traffic and mitigate existing safety problems. Crash data indicate a high number of angle crashes occur at this intersection. Slight widening of the North Main Street approaches is proposed to provide exclusive left-turn lanes and through-right-turn lanes. In addition, signal phasing would be revised to provide protected/permissive left-turn phasing for the westbound approach. Signal timing changes would be made to accommodate the proposed phasing change and provide a longer interval for the exclusive pedestrian phase. The proposed mitigation at this location would improve the morning peak hour from LOS D to LOS C. The evening peak hour would remain at LOS D.

President Avenue at North Davol Street—Pedestrian crossing times would increase to accommodate project related pedestrians at this location, which would cause an adverse impact to overall vehicular traffic operations (i.e. increased delay) under every alternative during at least one peak hour. However, the intersection is projected to operate at acceptable levels of service with these pedestrian timing improvements.

Battleship Cove Station Area Traffic Mitigation (all alternatives)

Broadway at Central Street—No changes are proposed to traffic operations at this location. Existing crosswalks across Broadway and Central Street (under the viaduct) would be restriped to facilitate the pedestrian pathway between the neighborhood and the proposed Battleship Cove Station. As part of this measure, existing wheelchair ramps would be evaluated to determine whether they comply with the current standards as prescribed by the Americans with Disabilities Act (ADA) and the Architectural Access Board (AAB). Non-compliant wheelchair ramps would be redesigned based on the prevailing ADA/AAB guidance in affect at that time.

Taunton Station Area Traffic Mitigation

The following intersection improvements are required to mitigate existing deficiencies at critical locations or adverse impacts caused by the alternatives. Table 4.1-91 presents a comparison of Build to Build with mitigation operations to illustrate the benefit of the proposed changes. The Taunton Depot station is proposed as part of the Whittenton and Stoughton Alternatives.

Table 4.1-91 Taunton Depot Intersection Capacity Analysis–2030 Build with Mitigations Conditions vs. 2030 Build Conditions

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Taunton Depot Station (all alternatives)								
Route 140 at Hart Street	E	>1.00	66	E	F	>1.00	73	E

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Taunton Depot Station Area Traffic Mitigation (Both Rail Alternatives)

Route 140 at Hart Street (Both Rail Alternatives)—The signal timing at the Route 140 and Hart Street intersection would be adjusted to reduce delays on the Hart Street approaches. This results in an improvement during the evening peak hour from LOS F to LOS E.

Sidewalk Improvements (Both Rail Alternatives)

To facilitate pedestrian travel from Route 140 to the station, construction of a 6-foot wide sidewalk is proposed on the north side of the Target Plaza parking lot from the terminus of Taunton Depot Drive's sidewalk to the station.

Dana Street Station Area Traffic Mitigation (Whittenton Alternatives)

Mitigation previously proposed in the DEIS/DEIR at the intersection of Route 140/Tremont Street at Oak Street was developed to support optimizing the grade crossing pre-emption timing at the Oak Street grade crossing. Although project impacts would be lower with the station on Dana Street, the mitigation measures are still being proposed to compliment adjacent grade-crossing improvements. Based on the projected traffic volumes, the Washington Street southbound approach would be reconfigured to provide an exclusive right-turn lane and a combined left turn/through lane. Traffic signal phasing would be revised to provide an overlap southbound right-turn phase during the Tremont Street eastbound phase. A longer crossing interval for the exclusive pedestrian phase would also be provided.

Due to the relocation of the station to Dana Street, additional mitigation measures are required. The existing crosswalks at the intersection of Route 140/Tremont Street and Granite Street should be restriped. Specialty (high visibility) materials should be considered for the crosswalk as it would provide a gateway to the station would likely get substantially more use than it does today. It does not appear that the proposed Dana Street Station would generate enough traffic such that a traffic signal would be warranted at the station driveway or at the intersection of Route 140/Tremont Street at Granite Street.

The Dana Street Station is proposed in a more residential area of Taunton than the previously proposed station. Traffic volumes along Danforth Street, Dana Street, Granite Street, Columbia Avenue, Hodges Avenue, and Morton Street would need to be monitored for cut-through traffic and speeds in order to alleviate the new flow in a residential area. Traffic calming mitigation plans may be needed to address these issues if and when the station opens to vehicular traffic.

Previously proposed traffic signal timing changes at the intersection of Washington Street and Court Street and a proposed traffic signal installation at the intersection of Washington Street at Frederick Martin Parkway are no longer being considered as part of the South Coast Rail project as they are no longer needed due to the lower numbers of, and a shift in, ridership.

Taunton Station Area Traffic Mitigation (Stoughton Alternative)

All mitigation measures related to Taunton Station that were proposed in the DEIS/DEIR are still recommended. Minor additional signal timing changes are needed at the intersection of Route 44 and Longmeadow Road. In addition to what was recommended in the DEIS/DEIR, based on new ridership estimates, mitigation measures at the intersection of Arlington Street and School Street were considered. Based on peak hour volume data, the intersection does not meet the peak hour traffic signal warrant. Consideration should be given to conversion of this two-way stop controlled intersection to an all-way stop controlled intersection to improve operations and safety.

Table 4.1-92 Signalized Intersection Traffic Operations–Build vs. Build with Mitigation

	2035 Weekday Morning Peak Hour				2035 Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
Broadway St at Washington St	D	0.70	28	C	E	0.90	51	D
Route 44 at Longmeadow Road	F	>1.00	74	E	F	>1.00	>80	F
Route 44 at Arlington Street	F	0.89	35	C	E	0.90	32	C

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Broadway at Washington Street (Stoughton Alternative)—The signal timing at the Broadway and Washington Street intersection would be adjusted to reduce delays on the Washington Street approaches during the evening peak hour. This timing adjustment results in an improvement during the evening peak hour from LOS F to LOS E.

Dean Street at Longmeadow Road (Stoughton Alternative)—Based on the projected traffic volumes, the Longmeadow Road southbound approach would be reconfigured to provide two general purpose lanes. Traffic signal timings would be modified to support revised signal timings and provide a longer crossing interval for the exclusive pedestrian phase. The increased pedestrian crossing times would cause an adverse impact to overall vehicular traffic operations (i.e. delay) during both peak hours. There is no opportunity at this location to increase capacity by adding lanes or changing lane allocation. However, once the project is in service, traffic and pedestrian signal timings would be further adjusted to balance the needs of pedestrians and motorists.

Dean Street at Prospect Street (Stoughton Alternative)—Proposed improvements at this intersection involve construction of ADA/AAB-compliant pedestrian ramps, new crosswalk and pavement markings across Dean Street. A passively-activated flashing pedestrian crossing sign would be installed at the Dean Street crosswalk. This sign, activated when a pedestrian entered a detection zone at the pedestrian ramps of the crossing, would highlight the location as an active pedestrian crossing to approaching motorists.

Dean Street at Arlington Street (Stoughton Alternative)—Improvements at this intersection would involve widening of the Arlington Street southbound approach to provide exclusive turning lanes and reconstruction of the existing traffic signal system in order to coordinate with the proposed gate and railroad signal improvements at the adjacent grade crossing. Signal timing and phasing changes will allow this intersection to remain at acceptable traffic operations during both peak hours.

As Dean Street (Route 44) is under the jurisdiction of the Massachusetts Department of Transportation Highway Division (MassDOT Highway Division) and the City of Taunton, MassDOT would coordinate construction and implementation of these intersection improvements with MassDOT Highway Division and the city at the appropriate time.

Stoughton Station Area Traffic Mitigation (Stoughton and Whittenton Alternatives)

Brock Street at Washington Street

A traffic signal would be warranted at the intersection of Brock Street at Washington Street under the Build Condition, and is recommended since the intersection would serve the primary station entrance. A capacity analysis for the signalized intersection was performed and the results were compared to the Build Condition (as an unsignalized intersection). The morning and evening peak hour under the signalized and unsignalized conditions are shown in Table 4.1-93. Signalizing the intersection upon relocation of the station would improve vehicle operations and mobility through the intersection.

Table 4.1-93 Brock Street/Kinsley Street at Washington Street–Build Condition

		Morning Peak Hour				Evening Peak Hour			
Condition	**Movement**	**Dem[1]**	**v/c[2]**	**Del[3]**	**LOS[4]**	**Dem**	**v/c**	**Del**	**LOS**
Unsignalized	EB LT-TH-RT	285	>1.20	>120	F	295	>1.20	>120	F
	WB LT-TH-RT	100	>1.20	>120	F	115	>1.20	>120	F
	NB LT-TH-RT	440	0.17	5	A	490	0.13	4	A
	SB LT-TH-RT	355	0.0	1	A	810	0.01	1	A
Signalized	**Approach**	**Dem**	**v/c**	**Del**	**LOS**	**Dem**	**v/c**	**Del**	**LOS**
	EB	285	0.66	21	C	295	0.65	27	C
	WB	100	0.22	13	B	115	0.29	19	B
	NB	440	0.76	17	B	490	0.60	11	B
	SB	355	0.42	9	A	810	0.87	19	B
	Overall	-	0.72	16	B	-	0.80	18	B

Source: Synchro 7 (Build 773, Rev 8) software
Note: Shaded cells denote LOS E/F conditions.
1 demand in vehicles per hour
2 volume-to-capacity ratio, values over 1.0 indicate demand in excess of capacity.
3 average delay in seconds per vehicle
4 level of service for critical movement
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound; LT = left-turn; TH = through; RT = right-turn

Under traffic signal control, the calculated 95th percentile queue along Brock Street is 119 feet during the morning peak hour and 166 feet during the evening peak hour. This does not include additional queuing due to the active grade crossing. The available queue storage between the intersection of Brock Street at Washington Street and the tracks is approximately 130 feet. As traffic signal design progresses, queue detection and separate traffic signal heads at the grade crossing should be incorporated.

Wyman Street at Summer Street/Morton Street

The intersection of Summer Street/Wyman Street/Morton Street has atypical geometry and only the Summer Street approach is currently under traffic control. The intersection also includes two driveways that serve existing MBTA parking lots. Relocating Stoughton Station provides an opportunity to reconstruct this intersection. The following mitigation measures are recommended:

- Eliminate the Morton Square MBTA driveway and parking area;
- Close the Trackside Plaza South driveway; and
- Realign Morton Street and install a stop sign.

Two measures to mitigate impacts at the Brock St. grade crossing are recommended:

- The proposed traffic signal design plans should consider the effects of incorporating gate operations and restricting movements from Washington Street to Brock Street while the crossing gates are down. This would require changes in geometry along Washington Street to provide a separate northbound left-turn lane and southbound right-turn lane. The existing shoulders on Washington Street may be sufficiently wide to make these changes without the need for land acquisition.

- The traffic signal design plans should modify the existing driveways immediately east of the crossing to discourage motorists from using the parking lot as a way to avoid the traffic signal.

Easton Station Area Traffic Mitigation (Stoughton and Whittenton Alternatives)

Preliminary mitigation measures have been developed for locations that are projected to accommodate a substantial amount of project-related traffic and operate at or over capacity. The proposed mitigation for the Easton stations include signalization of the Union Street and Elm Street intersections with Route 138, pedestrian-related improvements in Easton Village area, and signal timing adjustments at the intersections of Route 138 and Roche Brothers Drive and Route 138 and Belmont Street. Table 4.1-94 presents a comparison of Build to Build with mitigation operations to illustrate the benefit of the proposed changes. Both Easton Stations are proposed as part of the Whittenton and Stoughton Alternatives.

Easton Village Traffic Mitigation (Stoughton and Whittenton Alternatives)

Due to the historic nature of the Easton Village area, specifically the Rockery monument, structural improvements to provide additional capacity are infeasible. Pedestrian level improvements are proposed for the area near this village-style station.

Main Street at Center Street and Lincoln Street (Stoughton and Whittenton Alternatives)—Proposed improvements at this intersection involve construction of ADA/AAB-compliant pedestrian ramps, new crosswalk and pavement markings. A passively-activated flashing pedestrian crossing sign would be installed at the Main Street crosswalk. This sign, activated when a pedestrian entered a detection zone at the pedestrian ramps of the crossing, would highlight the location as an active pedestrian crossing to approaching motorists.

Table 4.1-94 Easton Intersection Capacity Analysis–2030 Build with Mitigation Conditions vs. 2030 Build Conditions (Stoughton and Whittenton Alternatives)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C	Delay	LOS
North Easton Station								
Route 138 at Roche Bros. Way	D	0.98	39	D	C	0.69	21	C
Route 138 at Main Street	F	1.00	39	D	E	>1.00	43	D
Route 138 at Elm Street[4]	F	0.84	27	C	F	0.84	36	D
Route 138 at Union Street[4]	F	0.70	10	A	F	1.00	46	D
Easton Village Station								
Route 138 at Belmont Street (Rt. 123)	E	0.87	53	D	F	0.93	58	E

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 Unsignalized in the Build condition
L = Left-turn; T = Through; R = Right-turn; All = All movements
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Lincoln Street at Barrows Street (Stoughton and Whittenton Alternatives)—New crosswalk and stop line pavement markings would be installed at the Lincoln Street and Barrows Street intersection to improve visibility and safety. Wheelchair ramps would be assessed for ADA/AAB compliance and reconstructed if necessary.

Route 138 at Belmont Street (Route 123) (Stoughton and Whittenton Alternatives)—Measures have been proposed to mitigate traffic impacts associated with the full build-out of Queset Commons, a proposed mixed-use development in Easton Village. These measures include the reconfiguration of the site's driveway approach to an exclusive left turn lane with a combined through-right turn lane to allow overlapping left-turn phasing with the Belmont Street approach. It is recommended that this lane and phasing adjustment not be installed and that the approach remain in the initial mitigation configuration of a left turn-through lane and an exclusive right-turn lane with split phasing. With that configuration, the intersection is projected to operate at LOS E or better during both peak periods.

North Easton Station Area Traffic Mitigation (Stoughton and Whittenton Alternatives)

Route 138 at Roche Brothers Driveway (Stoughton and Whittenton Alternatives)—Minor traffic signal timing adjustments are proposed for this location. These adjustments are recommended to increase the crossing time for pedestrians crossing Route 138 and to facilitate exiting station traffic during the evening peak period. While these improvements are recommended for mobility reasons, they are not required to mitigate adverse project impacts. Levels of service during the morning and evening peak hours remain unchanged and at acceptable levels.

Route 138 at Union Street (Stoughton and Whittenton Alternatives)—This unsignalized intersection processes a high amount of traffic and operates at LOS F during the morning and evening peak hour with or without the project. To offset project related traffic at this intersection, a traffic signal would be installed at this location. The proposed signal would be designed to operate with three phases; the first phase serving as a lead phase for Route 138 southbound and the second phase for both northbound and southbound Route 138. The third phase processes Union Street traffic. The new intersection would include concurrent pedestrian phases, wheelchair ramps and crosswalks. Pedestrian crossings would occur concurrently with these second and third phases. With the proposed improvement, the intersection of Route 138 and Union Street would operate at LOS A and LOS D during the morning and evening peak hour, respectively.

Signalization may be warranted at this intersection. Should these improvements be desired, MassDOT could contribute to the construction/implementation of these intersection improvements based on their fair share of the impacts to the intersection.

Route 138 at Elm Street (Stoughton and Whittenton Alternatives)—This unsignalized intersection processes a high amount of traffic and operates at LOS F during both the morning and evening peak hours with or without the proposed project. To offset project related traffic impacts at this intersection, a traffic signal would be installed. The proposed signal would be designed to operate with three phases; the first phase serving Route 138 northbound and southbound, the second phase serving Elm Street eastbound and the final phase serving Elm Street westbound. The new traffic signal would include concurrent pedestrian phases, wheelchair ramps and crosswalks. With the proposed improvement the intersection of Route 138 and Elm Street would operate at LOS C and LOS D during the morning and evening peak hour, respectively.

Signalization may be warranted at this intersection. Should these improvements be desired, MassDOT could contribute to the construction/implementation of these intersection improvements based on their fair share of the impacts to the intersection.

Route 138 at Main Street (Stoughton and Whittenton Alternatives)—This signalized intersection processes a high amount of traffic and would operate at LOS E during the morning peak hour and LOS F during evening peak hour without the project in place. With the proposed project, the intersection operates at LOS F during both peak hours. Traffic signal timing and phasing adjustments would be completed at this location to offset impacts from the proposed project. Specifically, a Main Street eastbound overlap right-turn phase would be added to the northbound/southbound Route 138 left-turn lead phase. The Main Street left-turn lead phase would be eliminated. Signal timing adjustments would be made to support the proposed changes. These proposed changes would allow the intersection of Route 138 at Main Street to operate at an acceptable LOS D during both the morning and evening peak hours.

Raynham Station Area Traffic Mitigation (Stoughton and Whittenton Alternatives)

The proposed mitigation for the Raynham Park Station includes signalization of the Raynham Park driveway, which would also be used as the station driveway, and signal timing adjustments at the intersection of Route 138 and Elm Street. Table 4.1-95 presents a comparison of Build to Build with mitigation operations to illustrate the benefit of the proposed changes. Raynham Park Station is proposed as part of the Whittenton and Stoughton Alternatives.

Table 4.1-95 Raynham Intersection Capacity Analysis–2030 Build with Mitigation vs. 2030 Build Conditions (Stoughton and Whittenton Alternatives)

	Weekday Morning Peak Hour				Weekday Evening Peak Hour			
	Build	Build with Mitigation			Build	Build with Mitigation		
Signalized Intersections	LOS[1]	V/C[2]	Delay[3]	LOS	LOS	V/C[2]	Delay[3]	LOS
Raynham Park Station								
Route 138 at Elm Street	B	0.79	21	C	B	0.83	22	C
Route 138 at Raynham Park Station Driveway[4]	F	0.56	12	B	F	0.63	14	B

Source: Synchro 7.0 Software; Build 763
1 level of service
2 volume-to-capacity ratio
3 average control delay for all vehicles, rounded to the nearest whole second, for signalized intersections
4 Unsignalized in the Build condition
L = Left-turn; T = Through; R = Right-turn
NB = Northbound; SB = Southbound; EB = Eastbound; WB = Westbound

Route 138 at Raynham Park Station (Stoughton and Whittenton Alternatives)

As part of the Raynham Park driveway signalization, the Robinson Street intersection on Route 138 would be shifted slightly north to align with the Raynham Park driveway, creating a four-way intersection. Route 138 would be widened at the intersection to accommodate an exclusive left-turn lane and two through lanes on the northbound approach and two general purpose lanes on the southbound approach. The new intersection would include pedestrian phases, wheelchair ramps and crosswalks. As shown in Table 4.1-95, the four-way signalized intersection would operate at LOE B in the morning and evening peak hours. These represent improved operations over the projected LOS F under Build conditions without mitigation.

As Route 138 is under the jurisdiction of MassDOT Highway Division, MassDOT would coordinate construction/implementation of these intersection improvements with MassDOT Highway Division at the appropriate time.

Route 138 at Elm Street (Stoughton and Whittenton Alternatives)

The signal timing at the Route 138 and Elm Street intersection would be adjusted to reduce delays on the Elm Street approaches and to provide adequate time for pedestrian crossings. The result is that all approaches would operate at acceptable levels of service but overall intersection operations would decline slightly from LOS B to LOS C.

4.1.6 Summary

The traffic analysis evaluated the traffic impacts of each of the commuter rail stations proposed as part of the Build Alternatives. Additionally, regional highway operations were evaluated to determine projected benefits of the regional transit enhancement associated with each of the alternatives. Traffic conditions in the vicinity of each station and along the regional highway network were analyzed for existing conditions and future 2030 conditions with and without the project. Mitigation would be implemented for roadways and intersections that would be most impacted by traffic associated with commuter rail stations associated with rail alternatives. In cases where Build Alternatives-related traffic would result in a degradation of operating conditions when compared to the No-Build Alternative, mitigation measures were evaluated and would be implemented to address these impacts. Table 4.1-96 presents the recommended traffic mitigation for the project summarized by alternatives and stations.

Table 4.1-96 Recommended Traffic Mitigation Summary

Station	Intersection/Roadway	Mitigation
Stoughton and Whittenton Alternatives		
Fall River Depot Station	North Main Street at President Avenue	Widen North Main Street to provide an exclusive northbound and southbound left-turn lane Modify traffic signal phasing to provide a westbound lead phase and exclusive pedestrian phase
	President Avenue at N. Davol Street	Pedestrian timing improvements
Battleship Cove Station	Broadway at Central Street	Crosswalk and pedestrian ramp improvements
	Broadway at Anawan Street	Crosswalk and pedestrian ramp improvements
Freetown Station	South Main Street	Construction of approx. 1,600 feet of sidewalk along the eastern side of South Main Street
	South Main Street at Narrows Road	Crosswalk and pedestrian ramp improvements
	South Main Street at Copicut Street	Crosswalk and pedestrian ramp improvements
Whale's Tooth Station	Acushnet Avenue at Hillman Street	Crosswalk and pedestrian ramp improvements
	Acushnet Avenue	Construction of approx. 300 feet of sidewalk along eastern side of Acushnet Avenue
	Mill Street at Pleasant Street and Kempton Street	Revised signal timing, including longer pedestrian timings
	Coggeshall Street at North Front Street	Install traffic signal
	Coggeshall Street at Purchase Street	Install traffic signal
King's Highway Station	King's Highway	Install signal interconnect infrastructure between Mount Pleasant Street and Church Street
	Mount Pleasant Street at Jones Road/King's Highway	Revised signal phasing and timings
	King's Highway at Shaw's Drive	Signal equipment, phasing and timing improvements to provide concurrent pedestrian crossing
	King's Highway at Stop & Shop Drive	Grade crossing signal pre-emption Reconfigure Stop & Shop Drive to accommodate diverted Tarkiln Hill Road traffic
King's Highway Station	Tarkiln Hill Road at Church Street	Grade crossing signal pre-emption Revised signal timing , including longer pedestrian timings

Station	Intersection/Roadway	Mitigation
Taunton Depot Station	Route 140 at Hart Street	Revised signal timing
	Taunton Depot Drive	Construction of sidewalk along the northern side of the Target Plaza parking lot to station area
Whittenton Alternative		
Dana Street Station	Tremont Street at Granite Street	Restripe existing crosswalks using high visibility materials
	Washington Street at Tremont Street	Review existing grade crossing pre-emption timing Restripe Washington Street for an exclusive right-turn and combined left/thru lanes Revised signal timing , including longer pedestrian timings
	General	Prepare traffic calming mitigation plan.
Stoughton Alternative		
Taunton Station	Broadway and Washington Street	Revised signal timing
	Dean Street at Longmeadow Street	Restripe Longmeadow Street to provide two southbound lanes Revised signal timing, including longer pedestrian timings
	Dean Street at Prospect Street	Install pavement marking and signage improvements
	Dean Street at Arlington Street	Reconstruct traffic signal system based on new adjacent grade crossing equipment Widen Arlington Street to provide two southbound lanes
	Arlington Street at School Street	Convert to all-way stop
Stoughton and Whittenton Alternatives		
Raynham Park Station	Route 138 at Elm St.	Revised signal timing, including longer pedestrian timings
	Route 138 at Dog Track/Station Driveway	Re-align Robinson Street to create 4-way intersection Widening of Route 138 to provide two lanes northbound and southbound Install traffic signal
Easton Village Station	Route 138 at Belmont Street	Revised signal phasing and timings
	Main Street at Center Street/Lincoln Street	Install pavement marking and signage improvements
	Lincoln Street at Barrows Street	Install pavement marking and signage improvements
North Easton Station	Route 138 at Roche Bros. Way	Revised signal timings
	Route 138 at Main St.	Revised signal timing, including longer pedestrian timings
	Route 138 at Elm St.	Widening of Route 138 to provide two lanes northbound and southbound

Station	Intersection/Roadway	Mitigation
		Install traffic signal
	Route 138 at Union St.	Widening of Route 138 to provide two lanes northbound and southbound Install traffic signal
Stoughton Station	Brock Street at Washington Street	Install traffic signal
	Wyman Street at Summer Street/Morton Street	Reconstruct intersection (eliminating driveways, realign Morton St. and install stop sign).

The impact analysis examined the traffic and safety impacts associated with the public grade crossings that would be in service along each of the Build Alternatives, with each crossing's recommended treatment (grade separation, closure, or at-grade crossing). Traffic conditions at existing grade crossings were evaluated, as increased train frequency at these grade crossings could affect traffic flows and roadway capacity on either side of each grade crossing. The grade crossing incident analysis summarized the probability of an incident occurring over the span of a year at each of the proposed at-grade crossings along each of the Build Alternatives as well as the probability of an incident occurring at each of the intersections that currently contain rail operations.

Based on the traffic and safety analysis conducted, general recommendations for traffic and safety improvements were made for all Build Alternatives. These general improvements include measures to optimize safety at the proposed at-grade crossings, including design features, signage, site conditions, signals and operations, vital logic and automatic highway crossing warning systems. Additionally, site-specific mitigation measures that could be undertaken by MassDOT to offset these impacts were presented by municipality and street. These specific improvements range from minor to major construction. Where structural changes to the roadway and traffic control devices are proposed, mitigation measures aim to improve traffic flow with minimal impacts to adjacent land uses and at reasonable cost. Table 4.1-97 presents the recommended at-grade crossing safety improvements for the Stoughton and Whittenton Alternatives, respectively, summarized by municipality and street.

Table 4.1-97 Recommended Grade Crossings Mitigation Summary

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
STOUGHTON LINE		
Canton	Washington Street	Install a traffic signal pre-emption system at two intersections in proximity of the crossing
	Pine Street	Relocate existing driveway to the north
	Will Drive	General improvements
Stoughton	Central Street	Relocate existing driveway to the west Coordinate crossing operation with fire station located 400 feet west Extend sidewalk through the crossing Install crosswalk across the Central Street eastbound approach to the crossing
	Simpson Street	General improvements
	School Street	Modify alignment at Cushing Street
	Porter Street (Route 27)	General improvements
	Wyman Street	Reconfigure parking lot and driveway

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
	Brock Street	Investigate installation of a traffic signal with pre-emption system at nearby intersection Reconfigure driveway to the east and relocate driveway to the west
	Plain Street	Investigate installation of a traffic signal with pre-emption system at nearby intersection Relocate driveways to the east
	Morton Street	Close Morton Street Construct frontage road to Totham Farm Road
Easton	Elm Street	Relocate driveway to the east
	Oliver Street	Relocate driveways to the northwest Relocate children's play area Extend sidewalk through crossing
	Gary Lane	Install gates and locks
	Short Street	General improvements
	Depot Street (Route 123)	Reconfigure driveway to the west
	Purchase Street	General improvements
	Prospect Street	General improvements
	Foundry Street (Route 106)	General improvements
Raynham	Race Track Crossing	General improvements
	Elm Street	General improvements
	Carver Street	Reconstruct culvert
	Broadway (Route 138)	At-grade separation
	Britton Street	General improvements
	King Philip Street	Relocate driveways
	East Britannia Street	General improvements
Taunton	Longmeadow Road	Reconfigure or close driveways
	Dean Street (Route 44)	Reconstruct Dean Street/Arlington Street traffic signal system Install traffic pre-emption phasing at Dean Street/Arlington Street
NEW BEDFORD MAIN LINE		
	Ingell Street	Close driveway to the west
	Hart Street	General improvements
Berkley	Cotley Street	General improvements
	Padelford Street	General improvements
	Myricks Street (Route 79)	General improvements
Lakeville	Malbone Street	General improvements
Freetown	Chace Road	Reconfigure or close driveway to the west
	Braley Road	General improvements
	East Chipaway Road	General improvements
New Bedford	Samuel Barnet Road	General improvements
	Pig Farm Road	General improvements

Town/City	Street	Recommended At-Grade Crossing Safety Improvements
	Tarkiln Hill Road	Close Tarkiln Hill Road and reroute traffic through Stop & Shop driveway Signal pre-emption at King's Highway / Stop & Shop driveway Signal pre-emption at Tarkiln Hill Road / Church Street At-grade crossing pre-signals
	Nash Road	Signal pre-emption at Church Street / Nash Road At-grade crossing pre-signals
FALL RIVER SECONDARY		
Berkley	Mill Street	Close crossing
	Adams Lane	Close crossing
Freetown	Beachwood Road	Close crossing
	Richmond Road/Route 79 (North)	General Improvements
	Richmond Road/Route 79 (South)	Reconfigure driveway to the west
	Forge Road (North)	Close Forge Road
	Forge Road (South)	General improvements
	Elm Street	General improvements
	High Street	General improvements
	Copicut Road	General improvements
	Brightman Lumber	General improvements
ATTLEBORO SECONDARY (Whittenton Alternatives Only)		
Taunton	Tremont Street	Reconfigure driveway to the north
	Oak Street	Optimize existing pre-emption at Oak Street / Tremont Street
	Porter Street	Reconfigure driveway to the east
	Cohannet Street	Reconfigure or close driveways adjacent to the tracks
	Winthrop Street	Additional advance RR warning signs
	Somerset Avenue	Investigate installation of a traffic signal with pre-emption system at nearby intersection
	Weir Street	Close McSoley Street Close and reconstruct driveway to the west Close and reconstruct driveway to the east

The MBTA Safety Department also seeks to minimize incidents within the system through grade crossing safety policies and programs, such as routine safety audits and the Safety and Operations Rules and Compliance Program.