DESIGN YEAR CONDITIONS

TRAFFIC GROWTH

Traffic volumes on the study area roadways were projected out to a ten-year period for design purposes. To develop the ten-year forecast, two sources of traffic growth were considered.

First, an annual-average traffic-growth percentage was determined. Based on a review of historical traffic volume data from the automatic traffic recorders by NP&EDC on the major roadways within the Mid Island area (provided in the Appendix), an annual growth rate of approximately 2.0 percent has occurred over the past seven years (1997 to 2004). In addition, the historical Nantucket Steamship Authority (SSA) automobile, passenger and truck data and the Nantucket Memorial Airport passenger enplanement data were reviewed. The SSA data indicated that between 1993 and 2003, the automobiles, passengers and trucks carried have increased at a rate of approximately 2.1 percent per year. In comparison to the SSA, the Nantucket Memorial Airport has experienced a higher enplanement rate over the past ten years (1993 to 2003), with passenger enplanements increasing at a rate of approximately 3.5 percent per year. However, the enplanement forecast by the Nantucket Memorial Airport for the years 2003 to 2020 anticipates an annual growth rate of approximately three percent. Based on discussions with town officials, it was agreed that a 3.0 percent per year growth rate is representative of growth in this area. Therefore, a 3.0 percent compounded annual growth rate was used in this study.

Second, any planned or approved specific developments in the area that would generate a significant volume of traffic on study area roadways within the next ten years were included. Based on discussions with officials from the Town of Nantucket, four developments are currently planned and/or approved in the immediate area.

The first project involves the potential development of the *"Craig Property."* The Craig Family owns one of the very few large undeveloped parcels of land in the Mid Island Area. This parcel is located on the north side of Sanford Road between Sparks Avenue and Pleasant Street. The following are three potential development scenarios outlined in the *Mid-Island Plan* prepared by NP&EDC in March 2003 for this vacant parcel:

1. Expansion of the Boys and Girls Club

   This alternative includes the expansion of the existing facility to include a gymnasium on the north side of the building and playing fields on the south side of the building. This alternative would also potentially include the construction of the central stop for Nantucket Regional Transit Authority (NRTA) buses.

2. Playing Fields and Performing Arts/Community Center

   This alternative includes the construction of a Performing Arts Center, which could also serve as a Town meeting space in addition to providing staged performance. This alternative would also potentially include the NRTA central bus stop as described in number 1 above.

3. Commercial /Residential Development

   This alternative would be to develop the vacant land in accordance with the Mid Island Plan with buildings consisting of commercial space on the lower levels and housing on the upper levels, placed close to the street and parking in the rear. The alternative would also potentially include the NRTA central bus stop as described above.

The second development is the potential relocation of the existing Nantucket Fire Station. The fire station is presently located on a small parcel of land, adjacent to the Stop & Shop facility, and cannot meet the future needs of the island within the limits of this site. The Town is presently evaluating the needs of the Fire Department and has identified the existing Electric Company site (located at the intersection of Fairgrounds Road and Old South Road) as the primary location for a new Public Safety Building.

The third development includes the potential expansion of the existing Stop & Shop facility to include the enhancement of the warehousing capabilities and the produce area of their facility. Enhancements to its facility also include the expansion of its parking facilities to the east, onto the property currently occupied by the Nantucket Fire Department. This alternative could potentially include the construction of the central stop for Nantucket Regional Transit Authority (NRTA) buses as well as the inclusion of a “liner” building adjacent to the Pleasant Street sidewalk. This will create more of a downtown feeling with the buildings directly abutting, or “lining” the sidewalks. In addition, the *Mid Island Plan* recommends that as part of these enhancements to the existing facility, the Stop & Shop should consider providing Transportation Demand Management (TDM) opportunities, such as home shopping service, similar to the

"Peapod" service it established in the Boston area, and providing employee housing on the upper level of its expanded area.

The fourth development includes the potential relocation of the existing deficient post office facility, currently located on the northwest corner of Pleasant Street and Daves Street to the parcel of land currently serving as a boat yard on the west side of the Pacific Bank, near the intersection of Sparks Avenue and Pleasant Street.

In addition to the four aforementioned potential developments, based on discussions with town officials, there is the potential for development of 13 additional parcels of existing vacant land in the immediate area. There were five high priority and five medium priority parcels of land identified off of Old South Road between Forest Avenue and Bunker Road, one medium priority parcel located on Bartlett Road west of Mizzenmast Road, one high priority parcel located on Surfside Road south of Windy Way, and one high priority parcel located on Polpis Road, just north of Milestone Road. These developments have the potential for light commercial, retail or residential uses. Given the uncertainty for these 13 potential developments, along with the aforementioned four potential developments coupled with limits for growth of an island, it was agreed with Town officials that all the aforementioned potential developments would be included as part of the conservative 3.0 percent annual traffic growth rate.

Design year (2014) weekday AM, weekday PM, and Saturday midday peak hours were developed by applying a compounded 3.0 percent annual growth rate (or 34.4 percent compounded over ten years) to the existing volumes. The Design Year peak-hour traffic volumes are shown on Figures 13 to 15 for the weekday AM, weekday PM and Saturday midday peak hours, respectively.

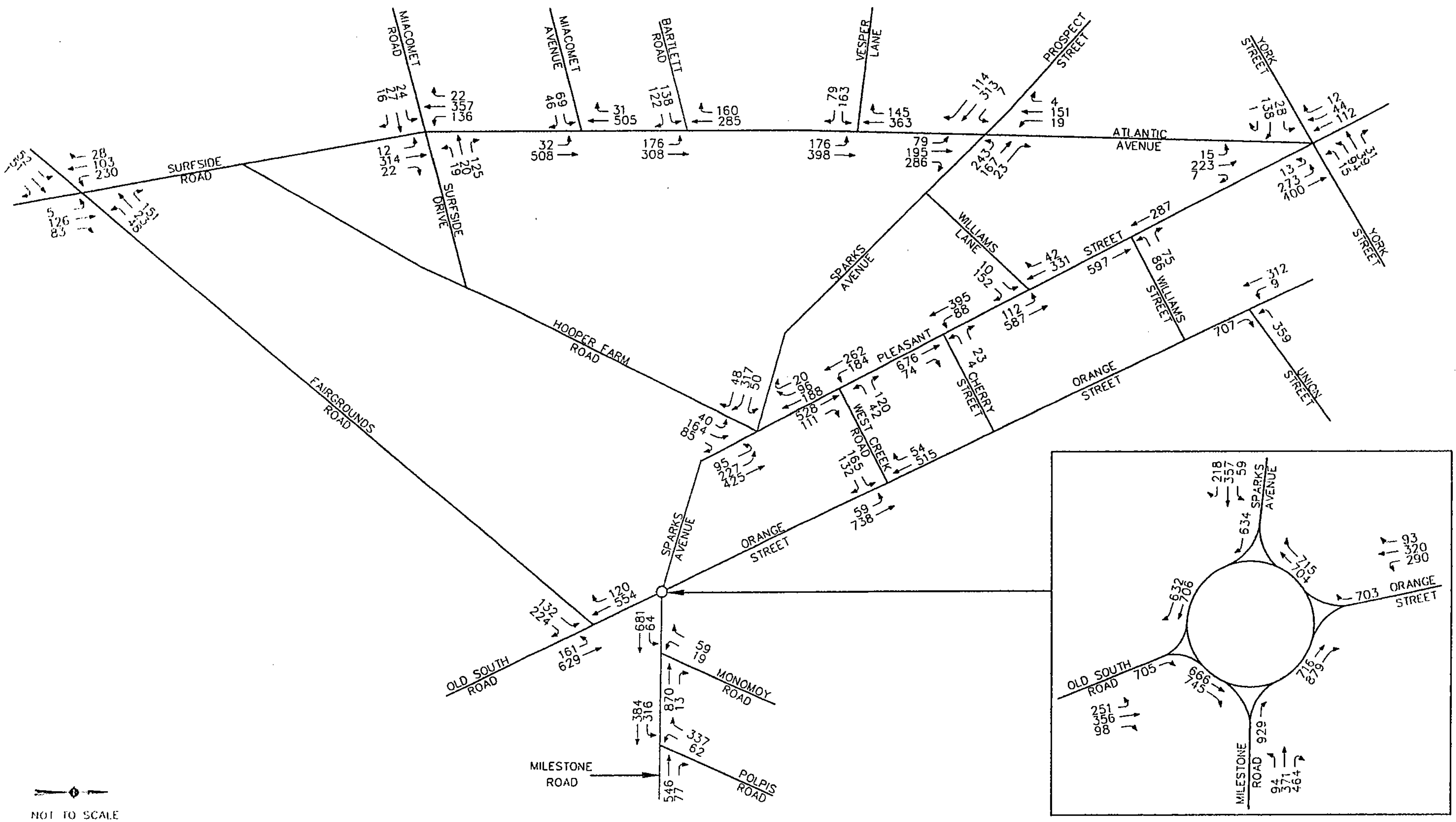

Figure 13
2014 Design Year
Weekday AM
Peak Hour Traffic Volumes

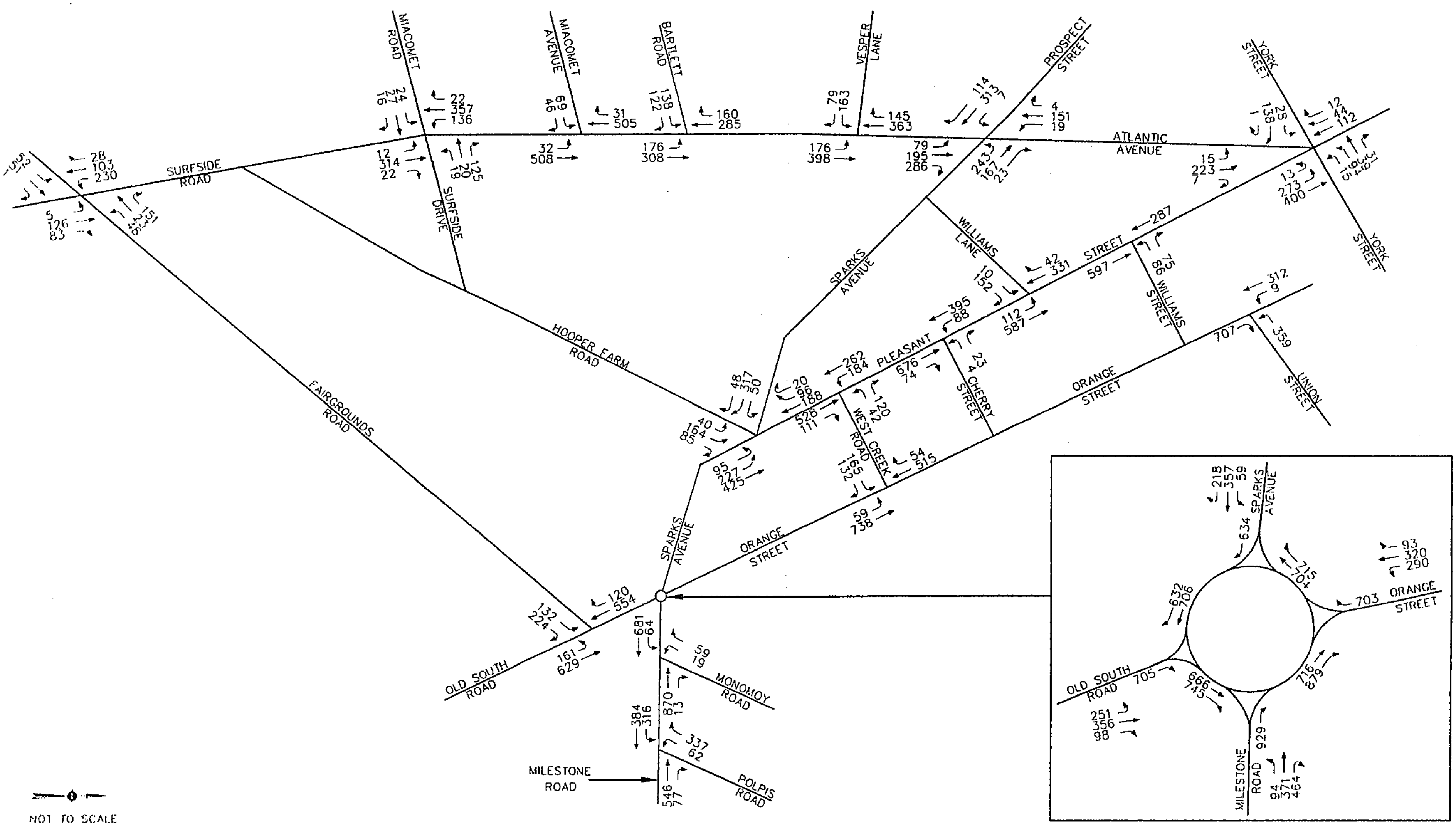

Figure 13

2014 Design Year
Weekday AM
Peak Hour Traffic Volumes

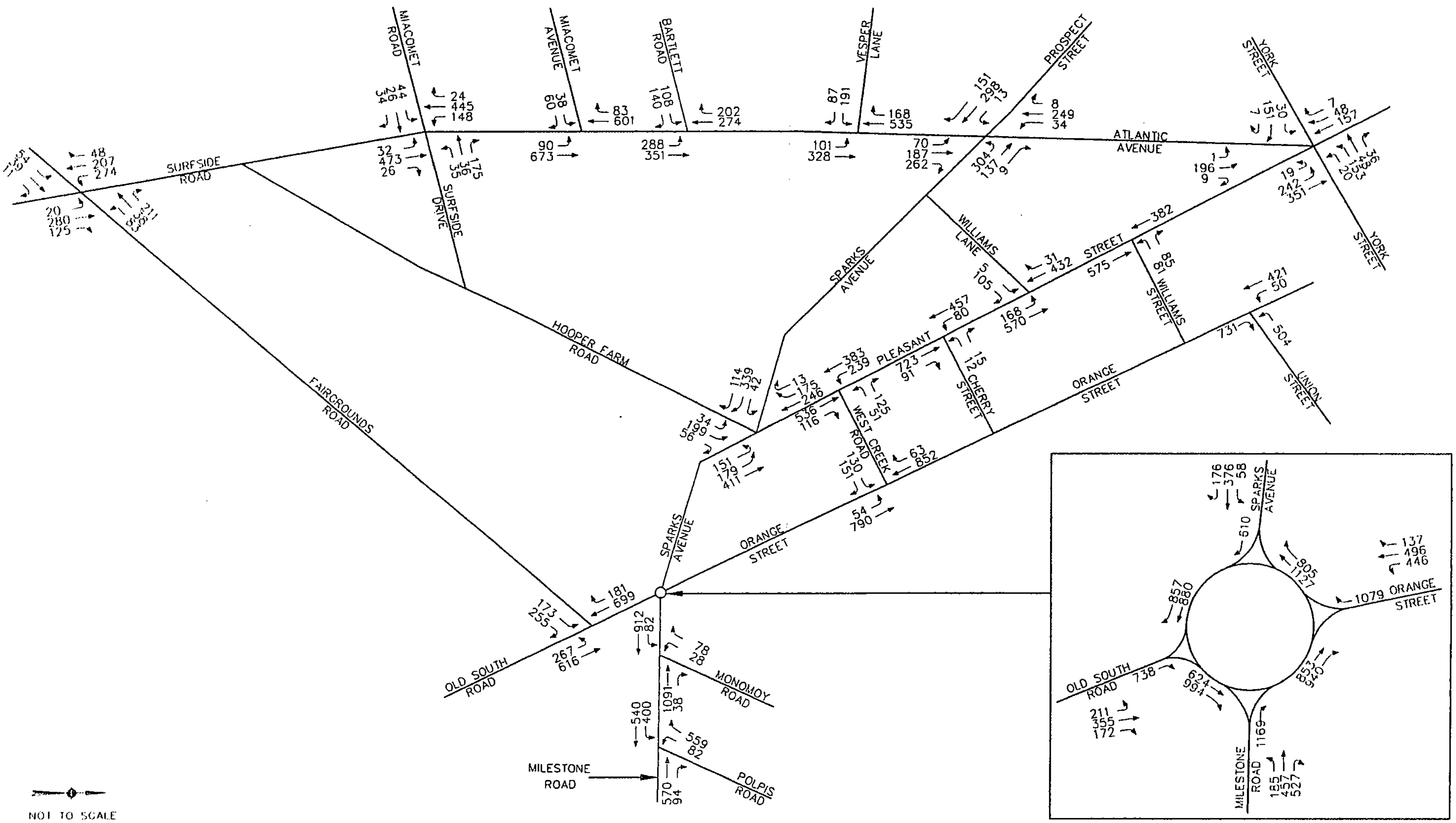

Figure 14

2014 Design Year
Weekday PM
Peak Hour Traffic Volumes

PLANNED ROADWAY IMPROVEMENTS

Based on discussions with officials from the Town of Nantucket, there is one roadway improvement project planned in the study area. The Sparks Avenue at Pleasant Street and Hooper Farm Road Intersection Improvement Project is currently under design. Improvements to this intersection include the reconstruction of the intersection to a modern single lane roundabout, consistent with national standards (FHWA Roundabout Design Guidelines). The proposed design will provide continuous traffic flow through the intersection and consists of reconstructing and realigning the four approaches to intersect at approximately 90 degrees at a modern roundabout. Eight-foot wide crosswalks will be provided across each intersection approach, with 5-foot wide sidewalks and wheelchair compliant ramps along both sides of each roadway, with the exception of the south side of Sparks Avenue where 8-foot wide multi-use paths will be provided. A 14-foot wide mountable apron will be provided at the outer edge of the 40-feet inner diameter central island (for a total central island diameter of 68 feet), providing additional paved area to allow for over-tracking of large trucks, but discouraging passenger vehicle travel. While it is hopeful that construction will begin in 2005, the project is currently under review by MassHighway at the 100 percent design stage and, therefore; the timing of construction is dependent on the scheduling of MassHighway. This intersection was analyzed both with and without the aforementioned improvements under both 2004 Existing and 2014 Design year conditions for comparison purposes, to demonstrate the benefits of the proposed roundabout and is included in the *Analysis* section of this report.

ANALYSIS

CAPACITY ANALYSIS METHODOLOGY

A primary result of capacity analysis is the assignment of levels of service to traffic facilities under various traffic flow conditions. The capacity analysis methodology is based on the concepts and procedures in the *Highway Capacity Manual* (HCM).[2] The concept of level of service (LOS) is defined as a qualitative measure describing operational conditions within a traffic stream and their perception by motorists and/or passengers. A level-of-service definition provides an index to quality of traffic flow in terms of such factors as speed, travel time, freedom to maneuver, traffic interruptions, comfort, convenience, and safety.

Six levels of service are defined for each type of facility. They are given letter designations from A to F, with LOS A representing the best operating conditions and LOS F the worst. Since the level of service of a traffic facility is a function of the traffic flows placed upon it, such a facility may operate at a wide range of levels of service, depending on the time of day, day of week, or period of year. A description of the operating condition under each level of service is provided below:

LOS A describes conditions with little to no delay to motorists.

LOS B represents a desirable level with relatively low delay to motorists.

LOS C describes conditions with average delays to motorists.

LOS D describes operations where the influence of congestion becomes more noticeable. Delays are still within an acceptable range.

[2]*Highway Capacity Manual 2000*, Transportation Research Board; Washington, D.C.; 2000.

LOS E represents operating conditions with high delay values. This level is considered by many agencies to be the limit of acceptable delay.

LOS F is considered to be unacceptable to most drivers with high delay values that often occur, when arrival flow rates exceed the capacity of the intersection.

Unsignalized Intersections

Levels of service for unsignalized intersections are calculated using the operational analysis methodology of the HCM. The procedure accounts for lane configuration on both the minor and major street approaches, conflicting traffic stream volumes, and the type of intersection control (STOP, YIELD, or all-way STOP control). The definition of level of service for unsignalized intersections is a function of average *control* delay. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. The level-of-service criteria for unsignalized intersections are shown in Table 3.

Roundabout Analysis

Roundabout (or rotary) capacity analysis is based on the concepts and procedures described in the *Signalized and Unsignalized Intersection Design and Research Aid (aaSIDRA).*[3] The main features of the aaSIDRA method for roundabout capacity estimation are the dependence of gap acceptance parameters on roundabout geometry, circulating flows and entry lane flows, and the designation of approach lanes as dominant and subdominant lanes that have different capacity characteristics. The aaSIDRA output produces level-of-service results based on the concepts described in the HCM. The level-of-service criteria for roundabouts are the same as for signalized intersections as shown in Table 3.

[3] *Signalized & Unsignalized Intersection Design & Research Aid, aaSIDRA 2.0 Version 2.0.3.217*; Akcelik & Associates Pty Ltd, Greythorn, Victoria, Australia; 2002.

Table 3
LEVEL-OF-SERVICE CRITERIA FOR INTERSECTIONS

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

Source: *Highway Capacity Manual 2000*, Transportation Research Board; Washington, D.C.; 2000. Pages 10-16 and 17-2.

QUEUE ANALYSIS METHODOLOGY

For unsignalized intersections, the 95th percentile queue represents the length of queue of the critical minor-street movement that is not expected to be exceeded 95 percent of the time during the analysis period (typically one hour). In this case, the queue length is a function of the capacity of the movement and the movement's degree of saturation.

ANALYSIS RESULTS

The following tables and discussion illustrate the results of the analysis performed on existing 2004 volumes and projected 2014 volumes under the existing geometric configurations. All locations were analyzed in accordance with the above methodology and are summarized in Tables 4 to 19. All analysis worksheets are provided in the Appendix.

Milestone Road at Polpis Road

Under both 2004 Existing and 2014 Design Year conditions, left-turns from Milestone Road onto Polpis Road operate at acceptable LOS B or better during the weekday AM, weekday PM and Saturday midday peak hours. In addition, there is sufficient roadway width on Milestone Road to allow through vehicles to bypass left turning vehicles. Left- and right-turning movements from Polpis Road onto Milestone Road are projected to operate with capacity constraints (LOS E/F) under future traffic-volume conditions during all three peak hours studied. In addition, the accident records indicate a safety issue at this intersection due to the existing geometric issues, the numerous merging and conflict points. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 4
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Milestone Road at Polpis Road

	2004 Existing				2014 Design Year			
Milestone Road at Polpis Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Milestone Rd EB left turns	0.25	9.6	A	25	0.39	11.5	B	48
Polpis Rd SB left turns	0.44	53.4	F	48	1.41	388.3	F	172
Polpis Rd SB right turns	0.52	17.4	C	75	0.86	43.4	E	224
Weekday PM:								
Milestone Rd EB left turns	0.31	10.1	B	34	0.49	13.0	B	68
Polpis Rd SB left turns	0.90	161.2	F	121	3.48	NC	F	305
Polpis Rd SB right turns	0.90	43.6	E	266	1.51	265.7	F	909
Saturday Midday:								
Milestone Rd EB left turns	0.28	9.4	A	29	0.43	11.3	B	54
Polpis Rd SB left turns	0.64	82.7	F	80	2.14	772.0	F	246
Polpis Rd SB right turns	0.55	17.2	C	84	0.89	43.8	E	252

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).
NC = No capacity available.

Milestone Road at Monomoy Road

Under both 2004 Existing and 2014 Design Year conditions, left-turns from Milestone Road onto Monomoy Road operate at acceptable LOS B or better during the weekday AM, weekday PM and Saturday midday peak hours. In addition, there is sufficient roadway width on Milestone Road to allow through vehicles to bypass left turning vehicles. Left-turning movements from Monomoy Road onto Milestone Road are projected to operate with capacity constraints (LOS E/F) during all three peak hours studied, along with right-turning movements during the weekday PM peak hour under future traffic-volume conditions. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 5
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Milestone Road at Monomoy Road

	2004 Existing				2014 Design Year			
Milestone Road at Monomoy Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Milestone Rd EB left turns	0.07	9.7	A	5	0.11	11.2	B	10
Monomoy Rd SB left turns	0.14	40.2	E	12	0.43	114.1	F	40
Monomoy Rd SB right turns	0.14	15.4	C	12	0.26	22.2	C	25
Weekday PM:								
Milestone Rd EB left turns	0.12	12.3	B	10	0.23	17.1	C	22
Monomoy Rd SB left turns	0.60	160.4	F	56	2.71	NC	F	139
Monomoy Rd SB right turns	0.37	31.3	D	40	0.88	118.7	F	136
Saturday Midday:								
Milestone Rd EB left turns	0.06	9.2	A	4	0.09	10.4	B	8
Monomoy Rd SB left turns	0.11	35.7	E	9	0.30	87.0	F	26
Monomoy Rd SB right turns	0.11	14.3	C	9	0.20	19.3	C	19

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).
NC = No capacity available.

Milestone Rotary

As shown in Table 6, under 2004 Existing conditions all approaches at the Milestone Rotary operate at acceptable LOS (LOS D or better) during the weekday AM, weekday PM and Saturday midday peak hours. However, under 2014 conditions, the Sparks Avenue and Old South Road approaches will operate with capacity constraints and long delays (LOS F) during the peak hours. While longer-term improvements of this location will require further study, short-term improvements are offered within the *Findings/Recommendations* section that better define right-of-way through the rotary.

Table 6
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Milestone Rotary

	2004 Existing				2014 Design Year			
Milestone Rotary	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Sparks Ave EB approach	0.67	11.3	B	188	1.23	136.6	F	1,500
Old South Rd NB approach	0.79	19.8	B	328	1.27	151.5	F	1,852
Milestone Rd WB approach	0.51	8.8	A	102	0.73	13.8	B	220
Orange St SB approach	0.36	6.7	A	48	0.55	9.1	A	92
Weekday PM:								
Sparks Ave EB approach	0.82	26.0	C	305	1.91	453.8	F	2,742
Old South Rd NB approach	0.89	31.8	C	495	1.23	134.5	F	1,800
Milestone Rd WB approach	0.58	9.3	A	128	0.80	15.4	B	288
Orange St SB approach	0.50	8.0	A	80	0.80	14.7	B	202
Saturday Midday:								
Sparks Ave EB approach	0.82	21.4	C	305	1.73	363.8	F	2,785
Old South Rd NB approach	0.92	36.1	D	542	1.23	136.7	F	1,822
Milestone Rd WB approach	0.49	8.4	A	92	0.68	12.3	B	185
Orange St SB approach	0.47	7.1	A	75	0.72	11.3	B	168

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Sparks Avenue at Pleasant Street and Hooper Farm Road

As shown in Table 7, under both 2004 Existing and 2014 Design Year conditions, without improvements, the Hooper Farm northbound, the Pleasant Street southbound and the Sparks Avenue eastbound approaches operate with long delays and queues (LOS F). Intersection improvements to provide a single lane roundabout at this location are proposed under a separate study, as described in the *Design Year Conditions* section of this report. The proposed improvements under both 2004 Existing and 2014 Design Year conditions will bring this intersection up to an acceptable LOS (LOS A/B) and provide improved traffic operations, with shorter vehicle queue lengths.

Table 7
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Sparks Avenue at Pleasant Street and Hooper Farm Road

	2004 Existing				2004 Existing with Planned Intersection Improvements				2014 Design Year				2014 Design Year with Planned Intersection Improvements			
Sparks Avenue at Pleasant Street and Hooper Farm Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue	V/C	Delay	LOS	Queue	V/C	Delay	LOS	Queue
Weekday AM:																
Pleasant St SB approach	0.90	67.3	F	275	0.28	11.4	B	55	1.75	729.6	F	2,700	0.44	14.3	B	88
Sparks Ave EB approach	1.24	276.7	F	1,432	0.36	9.5	A	68	2.57	NC	F	5,098	0.55	11.6	B	128
Hooper Farm Road NB approach	1.85	833.9	F	2,348	0.37	12.9	B	70	5.07	NC	F	5,470	0.57	16.7	B	140
Sparks Ave WB approach	0.39	0.0	A	0	0.52	9.4	A	110	0.52	0.0	A	0	0.75	12.4	B	255
Weekday PM:																
Pleasant St SB approach	1.02	116.3	F	585	0.32	12.0	B	65	2.00	900.1	F	3,752	0.49	13.3	B	105
Sparks Ave EB approach	1.34	362.1	F	1,950	0.45	10.4	B	88	2.75	NC	F	5,892	0.71	17.9	B	235
Hooper Farm Road NB approach	1.61	632.3	F	1,615	0.28	11.1	B	50	4.52	NC	F	4,298	0.47	14.5	B	95
Sparks Ave WB approach	0.33	0.0	A	0	0.42	9.5	A	82	0.44	0.0	A	0	0.62	10.4	B	145
Saturday Midday:																
Pleasant St SB approach	1.18	216.5	F	1,188	0.36	12.7	B	75	2.25	NC	F	4,760	0.55	14.8	B	135
Sparks Ave EB approach	1.44	444.4	F	2,162	0.45	10.6	B	85	3.02	NC	F	5,962	0.71	18.2	B	228
Hooper Farm Road NB approach	1.96	947.0	F	2,440	0.36	11.8	B	68	5.55	NC	F	5,470	0.61	19.3	B	160
Sparks Ave WB approach	0.35	0.0	A	0	0.44	9.2	A	90	0.47	0.0	A	0	0.66	10.6	B	172

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).
NC = No capacity available.

Orange Street at West Creek Road

As illustrated in Table 8, under both 2004 Existing and 2014 Design Year conditions, left-turns from Orange Street onto West Creek Road operate at acceptable LOS B or better during the weekday AM, weekday PM and Saturday midday peak hours. Left- and right-turning movements from West Creek Road onto Orange Street operate with long delays and queues (LOS F) under both 2004 Existing and 2014 Design Year conditions during all three peak hours studied.

Table 8
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Orange Street at West Creek Road

	2004 Existing				2014 Design Year			
Orange Street at West Creek Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Orange St NB left turns	0.05	8.7	A	4	0.08	9.4	A	6
West Creek Rd EB approach	1.00	97.0	F	244	2.33	666.9	F	730
Weekday PM:								
Orange St NB left turns	0.05	9.3	A	4	0.08	10.4	B	6
West Creek Rd EB approach	1.06	121.2	F	259	2.61	804.2	F	729
Saturday Midday:								
Orange St NB left turns	0.06	9.5	A	5	0.11	10.8	B	9
West Creek Rd EB approach	1.08	134.3	F	258	2.82	907.9	F	709

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Pleasant Street at West Creek Road

As illustrated in Table 9,under both 2004 Existing and 2014 Design Year conditions, left-turns from Pleasant Street onto West Creek Road operate at acceptable LOS B or better during all three peak hours studied. Left- and right-turning movements from West Creek Road onto Pleasant Street operate at LOS C under 2004 Existing conditions during all three peak hours studied. Under 2014 Design Year conditions, the West Creek Road turning movements deteriorate to LOS E during the weekday AM peak hour and LOS F during both the weekday PM and Saturday midday peak hours. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 9
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Pleasant Street at West Creek Road

	2004 Existing				2014 Design Year			
Pleasant Street at West Creek Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Pleasant St SB left turns	0.15	9.1	A	14	0.24	10.3	B	23
West Creek Rd WB approach	0.35	18.7	C	39	0.73	47.9	E	128
Weekday PM:								
Pleasant St SB left turns	0.18	9.3	A	17	0.29	10.8	B	30
West Creek Rd WB approach	0.45	23.7	C	56	1.02	117.1	F	227
Saturday Midday:								
Pleasant St SB left turns	0.18	9.3	A	16	0.28	10.8	B	28
West Creek Rd WB approach	0.45	24.4	C	56	1.02	117.4	F	222

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Pleasant Street at Cherry Street, Williams Lane and Williams Street

As shown in Table 10, acceptable operating conditions (LOS D or better) are experience for all turning movements at the unsignalized intersections of Pleasant Street at Cherry, Williams Lane and Williams Street under both 2004 Existing and 2014 Design Year conditions during the weekday AM, weekday PM and Saturday midday peak hours. However, the accident records indicate a safety issue at these intersections due to the existing geometric deficiencies. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 10
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Pleasant Street at Cherry Street, Williams Lane and Williams Street

	2004 Existing				2014 Design Year			
Intersection/Peak Hour/Movement	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Pleasant Street at Cherry Street								
Weekday AM:								
Pleasant St SB left turns	0.09	9.0	A	7	0.14	10.1	B	12
Cherry St Rd WB approach	0.05	13.9	B	4	0.10	18.5	C	9
Weekday PM:								
Pleasant St SB left turns	0.08	9.0	A	6	0.12	10.1	B	10
Cherry St Rd WB approach	0.07	17.7	C	6	0.17	28.5	D	15
Saturday Midday:								
Pleasant St SB left turns	0.06	9.3	A	5	0.10	10.5	B	8
Cherry St Rd WB approach	0.13	17.2	C	11	0.27	27.4	D	26
Pleasant Street at Williams Lane								
Weekday AM:								
Pleasant St NB left turns	0.07	8.1	A	6	0.11	8.6	A	9
Williams Ln EB approach	0.27	12.4	B	28	0.46	17.1	C	60
Weekday PM:								
Pleasant St NB left turns	0.11	8.5	A	10	0.17	9.3	A	16
Williams Ln EB approach	0.20	12.6	B	18	0.34	16.8	C	38
Saturday Midday:								
Pleasant St NB left turns	0.14	8.5	A	12	0.21	9.2	A	19
Williams Ln EB approach	0.23	13.2	B	23	0.43	19.7	C	52
Pleasant Street at Williams Street								
Weekday AM:								
Williams St WB approach	0.34	17.1	C	38	0.63	32.6	D	100
Weekday PM:								
Williams St WB approach	0.33	17.1	C	36	0.62	32.7	D	97
Saturday Midday:								
Williams St WB approach	0.32	17.6	C	34	0.61	34.1	D	93

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95^{th} percentile queue length in feet per lane (assuming 25 feet per vehicle).

Orange Street at Union Street

As illustrated in Table 11, under 2004 Existing conditions, left-turns from Orange Street onto Union Street operate at acceptable LOS D or better during the weekday AM, weekday PM and Saturday midday peak hours. Under 2014 Design Year conditions, this movement deteriorates to a LOS D during the weekday AM and Saturday Midday peak hours and LOS F during the weekday PM peak hour. Left-turning movements from Union Street onto Orange Street are projected to operate with capacity constraints (LOS F) under future traffic-volume conditions during the weekday PM and Saturday midday peak hours. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Due to discrepancies between the data collected as part of this study and past historical data, it is recommended that this location be further studied based on 2005 summer traffic levels.

Table 11
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Orange Street at Union Street

	2004 Existing				2014 Design Year			
Orange Street at Union Street	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Orange St SB left turns	0.16	13.4	B	118	0.24	28.5	D	250
Union St WB lefts	0.73	22.8	D	225	1.35	193.9	F	1,135
Weekday PM:								
Orange St SB left turns	0.33	19.8	C	245	0.58	62.8	F	508
Union St WB lefts	1.26	152.1	F	1,020	2.54	727.1	F	2,958
Saturday Midday:								
Orange St SB left turns	0.25	12.0	B	158	0.37	28.0	D	365
Union St WB lefts	1.21	130.6	F	1,000	2.36	638.5	F	2,998

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Five Corners

As shown in Table 12, under 2004 Existing conditions, the Five Corners all-way STOP controlled intersection already operates with capacity constraints, with the Pleasant Street northbound approach operating at LOS E/F during all peak hours studied. These long delays and queues will be exacerbated with the addition of future growth. However, due to the historical nature of this location major geometric or operational modifications are not feasible. In addition, the accident records indicate a safety issue at this intersection due to the existing geometric issues, the numerous approaches and vast amount of pavement. However, minor improvements are described in the *Findings/Recommendations* section of this report to provide better definition of right-of-way and improve pedestrian access through the intersection.

Table 12
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Pleasant Street at York Street and Atlantic Avenue

Pleasant Street at York Street and Atlantic Avenue	2004 Existing				2014 Design Year			
	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
York St/Atlantic Ave EB approach	0.76	27.8	D	169	1.00	101.5	F	354
York St WB approach	0.41	14.6	B	48	0.55	20.9	C	81
Pleasant St NB approach	1.00	77.5	F	370	1.00	326.9	F	428
Pleasant St SB approach	0.39	13.8	B	45	0.51	19.4	C	70
Weekday PM:								
York St/Atlantic Ave EB approach	0.72	22.7	C	147	1.00	92.5	F	342
York St WB approach	0.45	15.0	C	57	0.66	26.3	D	116
Pleasant St NB approach	0.91	42.2	E	271	1.00	254.7	F	395
Pleasant St SB approach	0.41	13.8	B	49	0.58	22.6	C	90
Saturday Midday:								
York St/Atlantic Ave EB approach	0.64	20.8	C	112	0.96	59.4	F	289
York St WB approach	0.48	16.4	C	63	0.73	30.0	D	146
Pleasant St NB approach	1.00	76.1	F	370	1.00	367.9	F	429
Pleasant St SB approach	0.40	14.0	B	48	0.59	22.4	C	91

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Four Corners

As shown in Table 13, under 2004 Existing conditions, the Four Corners all-way STOP controlled intersection already operates with capacity constraints, with the Surfside Road approach operating at LOS E/F during all peak hours studied and the Sparks Avenue approach operating at LOS E during the weekday PM peak hour. These long delays and queues will be exacerbated with the addition of future growth. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 13
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Sparks Avenue at Prospect Street, Surfside Road and Atlantic Avenue

Sparks Avenue at Prospect Street, Surfside Road and Atlantic Avenue	2004 Existing				2014 Design Year			
	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Prospect St EB approach	0.84	33.5	D	207	1.00	159.7	F	351
Sparks Ave WB approach	0.81	31.5	D	188	1.00	136.2	F	337
Surfside Rd NB approach	0.91	43.6	E	264	1.00	228.3	F	378
Atlantic Ave SB approach	0.44	16.5	C	53	0.59	26.1	D	90
Weekday PM:								
Prospect St EB approach	0.83	32.9	D	200	1.00	157.4	F	338
Sparks Ave WB approach	0.87	39.4	E	224	1.00	197.2	F	345
Surfside Rd NB approach	0.90	42.6	E	251	1.00	228.3	F	364
Atlantic Ave SB approach	0.61	20.9	C	97	0.83	46.6	E	187
Saturday Midday:								
Prospect St EB approach	0.81	31.7	D	189	1.00	146.9	F	340
Sparks Ave WB approach	0.79	31.0	D	175	1.00	131.2	F	328
Surfside Rd NB approach	0.95	52.2	F	297	1.00	281.0	F	387
Atlantic Ave SB approach	0.52	18.4	C	72	0.72	34.1	D	133

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Surfside Road at Vesper Lane

As illustrated in Table 14, under both 2004 Existing and 2014 Design Year conditions, left-turns from Surfside Road onto Vesper Lane operate at acceptable LOS B or better during the weekday AM, weekday PM and Saturday midday peak hours. Turning movements from Vesper Lane onto Surfside Road operate with capacity constraints (LOS E/F) under both existing and future traffic-volume conditions during all three peak hours studied. Improvements at the adjacent Four Corners intersection are recommended and are anticipated to improve operations at this intersection. In addition, traffic calming and safety improvements are recommended for the Surfside Road intersections and are described in the *Findings/Recommendations* section of this report.

Table 14
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Surfside Road at Vesper Lane

	2004 Existing				2014 Design Year			
Surfside Road at Vesper Lane	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Surfside Rd NB left turns	0.14	8.8	A	12	0.21	9.7	A	20
Vesper Ln EB approach	0.67	40.7	E	110	1.53	316.9	F	411
Weekday PM:								
Surfside Rd NB left turns	0.09	9.1	A	8	0.15	10.2	B	13
Vesper Ln EB approach	0.84	57.6	F	178	1.84	442.2	F	604
Saturday Midday:								
Surfside Rd NB left turns	0.06	8.8	A	5	0.10	9.6	A	8
Vesper Ln EB approach	0.75	44.6	E	144	1.59	330.4	F	514

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Surfside Road at Bartlett Road

As illustrated in Table 15, under both 2004 Existing and 2014 Design Year conditions, left-turns from Surfside Road onto Bartlett Road operate at acceptable LOS B or better during the weekday AM, weekday PM and Saturday midday peak hours. Turning movements from Bartlett Road onto Surfside Road operate with capacity constraints (LOS E/F) under future traffic-volume conditions during all three peak hours studies. Traffic calming and safety improvements are recommended for the Surfside Road intersections and are described in the *Findings/Recommendations* section of this report.

Table 15
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Surfside Road at Bartlett Road

	2004 Existing				2014 Design Year			
Surfside Road at Bartlett Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Surfside Rd NB left turns	0.12	8.4	A	10	0.18	9.1	A	16
Bartlett Rd EB approach	0.61	26.7	D	99	1.22	164.2	F	390
Weekday PM:								
Surfside Rd NB left turns	0.20	8.8	A	18	0.29	9.8	A	31
Bartlett Rd EB approach	0.66	33.4	D	111	1.49	285.4	F	466
Saturday Midday:								
Surfside Rd NB left turns	0.24	9.1	A	23	0.36	10.5	B	42
Bartlett Rd EB approach	1.00	84.5	F	287	2.40	683.7	F	966

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Surfside Road at Miacomet Avenue

As illustrated in Table 16, under both 2004 Existing and 2014 Design Year conditions, left-turns from Surfside Road onto Miacomet Avenue operate at acceptable LOS B or better during all three peak hours studied. Left- and right-turning movements from Miacoment Avenue onto Surfside Road operate at LOS C under 2004 Existing conditions during all three peak hours studied. Under 2014 Design Year conditions, the Miacomet Avenue turning movements deteriorate to LOS E during the weekday AM peak hour and LOS F during both the weekday PM and Saturday midday peak hours. In addition, the accident records indicate a safety issue at this intersection due to the existing geometric issues, the presence of adjacent parking and driveways, large pavement areas and poorly defined right-of-way. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 16
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Surfside Road at Miacomet Avenue

	2004 Existing				2014 Design Year			
Surfside Road at Miacomet Avenue	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Surfside Rd NB left turns	0.02	8.3	A	2	0.04	8.8	A	3
Miacomet Ave EB approach	0.28	18.5	C	28	0.56	36.0	E	77
Weekday PM:								
Surfside Rd NB left turns	0.08	9.0	A	6	0.12	10.0	B	10
Miacomet Ave EB approach	0.37	24.5	C	41	0.85	88.0	F	148
Saturday Midday:								
Surfside Rd NB left turns	0.04	8.8	A	3	0.06	9.5	A	5
Miacomet Ave EB approach	0.38	23.8	C	42	0.82	75.6	F	144

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Surfside Road at Miacomet Road and Surfside Drive

As illustrated in Table 17, under both 2004 Existing and 2014 Design Year conditions, northbound and southbound left-turns from Surfside Road onto Miacomet Road and Surfside Drive, respectively, operate at acceptable LOS A during all three peak hours studied. Turning movements from the Surfside Drive westbound approach operate at acceptable LOS D or better during all three peak hours studied under 2004 Existing conditions and deteriorate to LOS E during the weekday AM peak hour and LOS F during both the weekday PM and Saturday midday peak hours. Under 2004 Existing conditions, turning movements from the Miacomet Road eastbound approach operate at LOS C, F and D during the weekday AM, weekday PM and Saturday midday peak hours, respectively. Under 2014 Design Year conditions, the Miacomet Road turning movements are expected to deteriorate to LOS E and F during the weekday AM peak hour and Saturday midday peak hours, and continue to operate at LOS F during the weekday PM peak hour. Traffic calming and safety improvements are recommended for the Surfside Road intersections and are described in the *Findings/Recommendations* section of this report.

Table 17
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Surfside Road at Miacomet Road and Surfside Drive

Surfside Road at Miacomet Road and Surfside Drive	2004 Existing				2014 Design Year			
	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Surfside Rd NB approach	0.01	8.0	A	1	0.01	8.3	A	1
Surfside Rd SB approach	0.09	8.1	A	7	0.12	8.5	A	11
Surfside Dr WB approach	0.29	14.7	B	29	0.54	25.3	D	76
Miacomet Rd EB approach	0.23	21.4	C	21	0.53	49.4	E	66
Weekday PM:								
Surfside Rd NB approach	0.02	8.2	A	2	0.03	8.7	A	3
Surfside Rd SB approach	0.11	8.6	A	9	0.17	9.4	A	15
Surfside Dr WB approach	0.58	29.0	D	86	1.42	264.5	F	399
Miacomet Rd EB approach	0.59	53.9	F	79	2.76	680.9	F	319
Saturday Midday:								
Surfside Rd NB approach	0.02	8.2	A	2	0.04	8.7	A	3
Surfside Rd SB approach	0.07	8.3	A	6	0.10	8.8	A	8
Surfside Dr WB approach	0.52	24.5	C	72	1.17	158.3	F	315
Miacomet Rd EB approach	0.32	28.3	D	34	0.99	164.5	F	152

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Surfside Road at Fairgrounds Road

The major (Surfside Road) movements at this unsignalized intersection operate at desirable LOS A under both existing and future traffic volume conditions during the weekday AM, weekday PM and Saturday midday peak hours. However, as illustrated in Table 18, the minor (Fairgrounds Road) movements are expected to operate with capacity constraints (LOS E/F) under 2014 Design Year conditions during all three peak hours studies. In addition, the accident records indicate a safety issue at this intersection due to the existing sight distance deficiencies. Traffic calming and safety improvements are recommended for the Surfside Road intersections and are described in the *Findings/Recommendations* section of this report.

Table 18
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Surfside Road at Fairgrounds Road

	2004 Existing				2014 Design Year			
Surfside Road at Fairgrounds Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Surfside Rd NB approach	0.00	7.4	A	0	0.00	7.5	A	0
Surfside Rd SB approach	0.16	8.2	A	14	0.23	8.6	A	22
Fairgrounds Rd WB approach	0.37	16.6	C	42	0.77	46.0	E	150
Fairgrounds Rd EB approach	0.46	31.7	D	56	1.06	155.7	F	203
Weekday PM:								
Surfside Rd NB approach	0.01	7.8	A	1	0.02	8.0	A	1
Surfside Rd SB approach	0.19	8.6	A	18	0.28	9.5	A	29
Fairgrounds Rd WB approach	0.86	57.3	F	197	2.38	682.9	F	799
Fairgrounds Rd EB approach	0.63	64.6	F	84	2.28	747.7	F	309
Saturday Midday:								
Surfside Rd NB approach	0.00	8.4	A	0	0.00	8.7	A	0
Surfside Rd SB approach	0.11	8.0	A	10	0.16	8.4	A	14
Fairgrounds Rd WB approach	0.78	36.9	E	170	1.57	305.0	F	680
Fairgrounds Rd EB approach	0.36	28.8	D	38	0.93	131.1	F	148

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).

Old South Road at Fairgrounds Road

As illustrated in Table 19, under both 2004 Existing and 2014 Design Year conditions, left-turns from Old South Road onto Fairgrounds Road operate at acceptable LOS B or better during the weekday AM and PM and Saturday midday peak hours. Left- and right-turning movements from Fairgrounds Road onto Old South Road operate at LOS F under both 2004 Existing and 2014 Design Year conditions, during all three peak hours studied. In addition, the accident records indicate a safety issue at this intersection due to the heavy turning volumes and poorly defined lanes. Improvements are recommended at this location to improve overall intersection operation and safety, as described in the *Findings/Recommendations* section of this report.

Table 19
INTERSECTION CAPACITY AND QUEUE ANALYSIS SUMMARY
Old South Road at Fairgrounds Road

	2004 Existing				2014 Design Year			
Old South Road at Fairgrounds Road	V/C[a]	Delay[b]	LOS[c]	Queue[d]	V/C	Delay	LOS	Queue
Weekday AM:								
Old South Rd NB through-lefts	0.15	9.4	A	13	0.23	10.8	B	23
Fairgrounds Rd EB approach	1.25	180.4	F	402	3.20	NC	F	1,055
Weekday PM:								
Old South Rd NB through-lefts	0.25	10.5	B	24	0.41	13.7	B	51
Fairgrounds Rd EB approach	1.98	507.2	F	648	6.51	NC	F	1,289
Saturday Midday:								
Old South Rd NB through-lefts	0.19	9.8	A	17	0.31	11.8	B	33
Fairgrounds Rd EB approach	1.18	156.3	F	342	3.15	NC	F	945

[a]Volume-to-capacity ratio.
[b]Average stopped delay in seconds per vehicle.
[c]Level of service.
[d]95th percentile queue length in feet per lane (assuming 25 feet per vehicle).
NC = No capacity available.