FEASIBILITY REPORT

BEACH AREA ROUNDABOUT & TRAFFIC SIGNAL PROJECT

(PACIFIC AVENUE / BEACH STREET & PACIFIC AVENUE / CENTER STREET)

Prepared For:

City of Santa Cruz, California

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October 26, 2005

BEACH AREA ROUNDABOUT & TRAFFIC SIGNAL PROJECT FEASIBILITY REPORT

FOR:

CITY OF SANTA CRUZ

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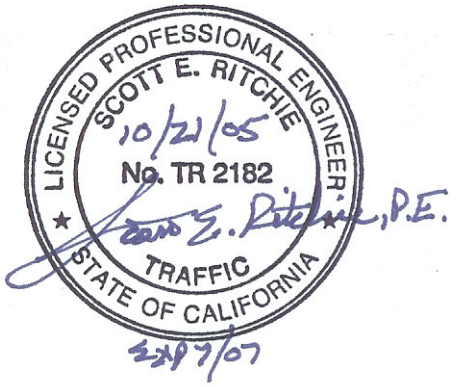

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EXECUTIVE SUMMARY

Kimley-Horn & Associates (KHA) and Roundabouts & Traffic Engineering (RTE) have been retained by the City of Santa Cruz to perform a feasibility study at the intersections of Pacific Avenue/Beach Street and Pacific Avenue/Center Street for a new intersection improvement project in Santa Cruz, California. The purpose of this feasibility study is to provide a comparative analysis of the operational performance of a modern roundabout versus a traffic signal at both of the identified intersections with a final recommendation at each intersection. A comparison between each alternative in terms of capacity, safety, and costs has been analyzed and documented for the future design year of 2025.

The changes in traffic control is proposed to address current and future level of service deficiencies as well as to provide a gateway to the Warf and Monterey Bay areas of the city. The city has concerns with the performance of signals and desires further consideration of roundabouts for their known safety and capacity benefits. The general conclusions of the feasibility study are provided below:

CONCLUSIONS

1. The modern roundabouts provide superior capacity over the signal alternatives with respect to the overall operations, level of service, delay, and queue lengths for the intersection.

2. The "before" and "after" safety statistics conducted in the United States and worldwide provide substantiating evidence of the superior safety performance of modern roundabouts versus traffic signals and other intersection types for both vehicles and pedestrians.

3. The roundabout provides significant safety and operational benefits for the high pedestrian activity in the area. The intersection would operate better, safer, and with near normal operation with the roundabout for vehicles, pedestrians, and trains.

4. The cost estimates of the alternatives (signal and roundabout) identify the signal alternatives with a total cost savings of $120,000 for both intersections combined.

It was determined by nearly all of the contributing factors within the study that the roundabouts are the identified recommended alternatives for both intersections. Please refer to Chapter VIII for additional conclusions and recommendations.

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I. INTRODUCTION

BACKGROUND

Kimley-Horn & Associates (KHA) and Roundabouts & Traffic Engineering (RTE) have been retained by the City of Santa Cruz to perform a feasibility study at the intersections of Pacific Avenue/Beach Street and Pacific Avenue/Center Street for a new intersection improvement project in Santa Cruz, California. The new intersections will either need to be signalized or controlled by modern roundabouts. The project sites are located within the City's right of way at existing intersections currently controlled by stop signs.

The City of Santa Cruz has concerns about the operational performance and safety of traffic signals at these locations and desires further investigation and consideration of modern roundabouts for their known safety and capacity benefits. Hence, the City has requested the consideration of modern roundabouts for these intersections.

The City has also requested general information on roundabouts and supporting evidence of the safety comparisons of traffic signals and modern roundabouts be provided for each alternative. General cost comparisons for the two alternatives at the study intersection have also been requested.

PURPOSE

The purpose of this feasibility study is to provide a comparative analysis of the operational performance of a modern roundabout versus a traffic signal at both of the identified intersections with a final recommendation at each intersection. A comparison between each alternative in terms of capacity, safety, and costs will be analyzed and documented for the future design year of 2025. In addition, this report will determine if the proposed intersections are viable locations for modern roundabouts, depending on the information provided by KHA, RTE, and the City of Santa Cruz. This report documents the existing and future traffic conditions and the recommended alternative for each intersection.

ORGANIZATION

This Roundabout Feasibility Report is organized into the following chapters:

I. Introduction
II. Existing Site Conditions

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III. Traffic Volumes & Future Assumptions
IV. Capacity Analyses & Conceptual Design Alternatives
V. Capacity Comparisons
VI. Safety Comparisons
VII. Cost Comparisons
VIII. Conclusions & Recommendations
IX. Appendix

The report begins with the identification of the existing site conditions for the intersection. The next chapter of the report identifies the existing and future conditions as well as the assumptions used to determine the design volumes in the analyses of the report. Next, the report examines the future capacity and delay requirements for a roundabout and a signal at the intersection location. The following chapter discusses the safety parameters and statistics of signals versus roundabouts. KHA provides rough cost estimates for each traffic control device at each intersection. The cost results are tabulated for comparison.

Finally, the feasibility study provides conclusions based on the results of the comparative analyses conducted for the intersections and recommendations for the selected alternatives.

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II. EXISTING SITE CONDITIONS

SURROUNDING AREA CONDITIONS

There is a need for a higher level traffic control device at the intersections of Pacific Avenue/Beach Street and Pacific Avenue/Center Street due to the increasing congestion with traffic volumes and pedestrian movements as well as the failing operations with the current stop controlled intersections. The changes in traffic control is proposed to address current and future level of service deficiencies as well as to provide a gateway to the Warf and Monterey Bay areas of the city.

The surrounding area consists of a mix of industrial, commercial, and residential zoning as well as public parks and ocean beaches. Growth in the surrounding area city limits is relatively insignificant or very low since the city is nearly fully built out.

A review was performed of the most recent site plans and roadway alignment information as well as a review of the intersections' volumes. The proposed roadways' surrounding topography, centerlines, curb faces, edge of pavement, environmental, and right of way constraints were also reviewed from the information provided.

EXISTING INTERSECTIONS

***Pacific Avenue / Beach Street*:** The existing intersection of Pacific Avenue / Beach Street, also known as the Wharf intersection, is an old intersection currently functioning as a three-way stop controlled intersection since the east leg is a one-way street. The photos of the existing intersection (**Figures 1** through **3**) show the approach configurations, bike lanes, surrounding land uses, proximity to the beach, and the existing rail line. The southbound direction has a three-lane approach and the eastbound direction has a two-lane approach. The northbound approach is from the Wharf pier with three individual approaches just north of the tollbooths. As stated above, there is no westbound approach since the street changes to a one-way configuration east of the intersection.

In general, the Pacific Avenue/Beach Street intersection functions as one of the major gateways to the City of Santa Cruz accommodating large tourist traffic, pedestrians using the beach and boardwalk area, and cyclists recreating along the beachfront roadways.

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Figure 1: Wharf Intersection

Figure 2: Pacific Ave. / Beach St.

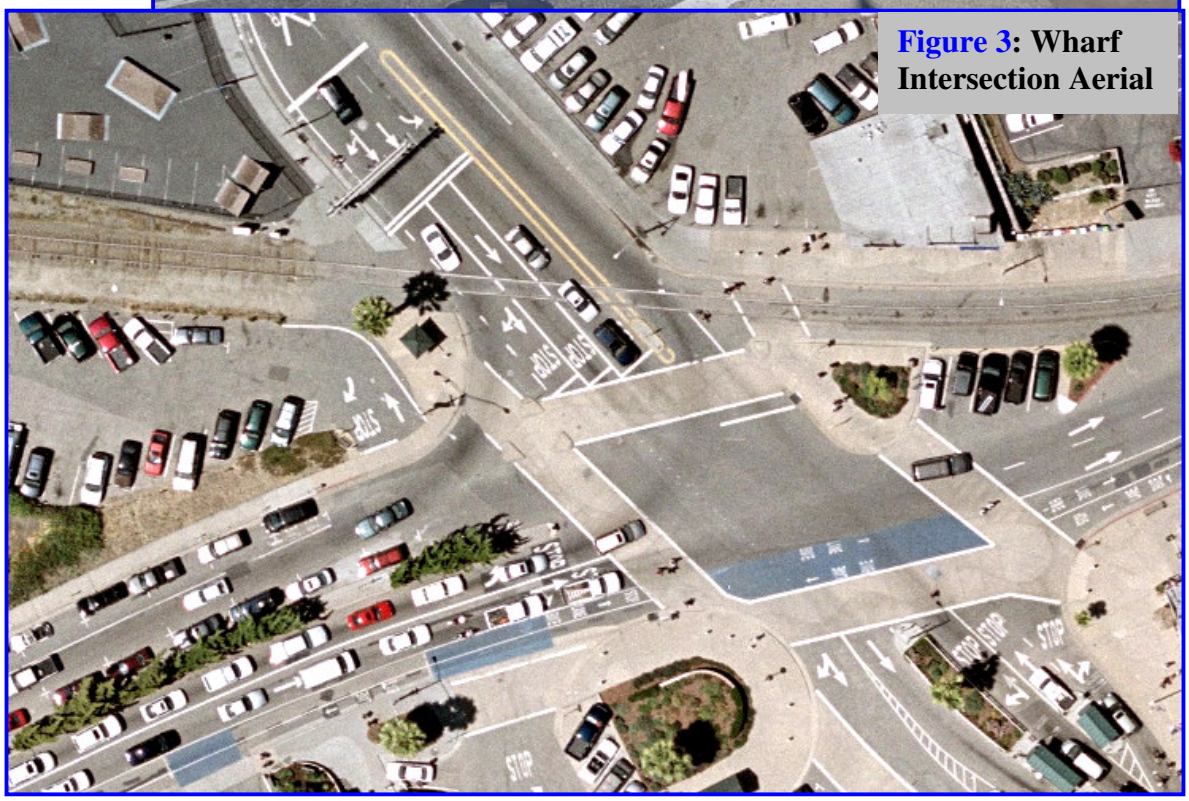

Figure 3: Wharf Intersection Aerial

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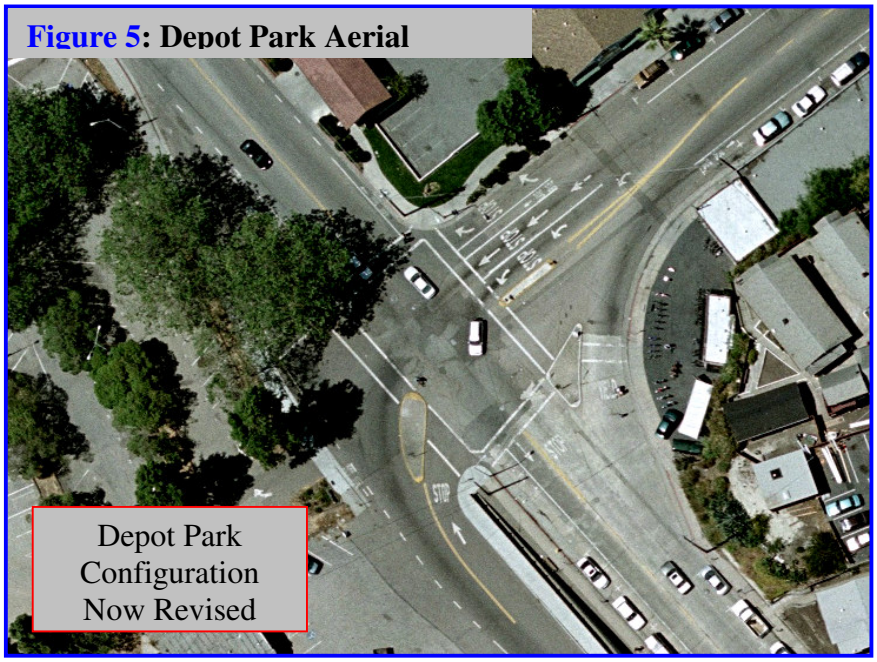
Pacific Avenue / Center Street: The existing intersection of Pacific Avenue / Center Street, also known as the Depot Park intersection, is an old intersection recently updated with the new Depot Park entrance and associated parking lot. It is currently functioning as a four-way stop controlled intersection with an additional exit only lane proceeding up the hill on West Cliff Drive. The photos of the existing intersection (**Figures 4** through **6**) show the approach configurations, bike lanes, surrounding land uses, and the proximity to the park. The southbound direction on Center Street has a shared through left lane and a short right turn lane to the Depot Park or West Cliff Drive. The Depot Park is a single-lane entrance only adjacent to the West Cliff Drive exit only. The westbound direction of Pacific Avenue has a left, a through, and a right turn lane while the northbound direction of Pacific Avenue has a shared left-through, single-lane approach with a short free-right-turn lane at the intersection.

Figure 4: Depot Park Intersection

In general, the Pacific Avenue/Center Street intersection functions as one of the major historical gateways to the City of Santa Cruz that accommodates local and tourist traffic volumes, pedestrians using the park, and cyclists recreating along as well as towards the beachfront roadways.

Figure 5: Depot Park Aerial



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III. TRAFFIC VOLUMES

FUTURE TRAFFIC ASSUMPTIONS & DESIGN VOLUMES

Peak Weekday & Saturday Design Volumes: The City of Santa Cruz worked with KHA and their traffic engineer representative, Ron Marquez, to determine the proper future traffic volumes for this feasibility study's intersections in the future year of 2025 for both the PM peak hours and Saturday peak hours. Existing traffic volumes were also manually surveyed on Friday July 22 and Saturday July 23, 2005 to determine the PM peak weekday and midday peak Saturday traffic volumes at the study intersections. The traffic volumes on Friday were counted from 4:00-6:00 PM and volumes on Saturday were counted from 1:00-3:00 PM. The results were recorded for individual turning movements at fifteen-minute increments. The counts coincided with the weekend of the annual Warf-to-Warf race, which is a 6-mile run from the Santa Cruz Municipal Wharf to the Capitol Wharf. The race is held on Sunday and attracts up to 15,000 runners plus spectators. Because the traffic counts were collected the same weekend (although not the same day) as the race, they resulted in traffic levels that represent common conditions that a roundabout or traffic signal would be expected to accommodate during the peak tourist season.

Results of the traffic counts indicate that peak hour traffic at each intersection was roughly the same on Friday PM and Saturday midday at both intersections, with Saturday being about 25 vehicles greater. Furthermore, the Pacific Avenue/Center Street intersection was noted to have approximately 120 more vehicles during the peak periods than the Pacific Avenue/Beach Street intersection.

During the traffic surveys, vehicles were observed to queue back several hundred feet on the southbound and eastbound approaches to the Pacific Avenue/Beach Street intersection. Backups at the Pacific Avenue/Center Street intersection were observed to be less. A major contributor to the queuing at the Pacific Avenue/Beach Street intersection is the high volume of bicyclists and pedestrians through the all-way stop intersection.

However, the City of Santa Cruz's traffic representative identified that the counts conducted were generally lower than counts performed in the past. Therefore, for the existing weekday PM peak hour counts, the 2005 Citywide Transportation Study data was used. The peak Saturday or "design day" data were based on the Coast Hotel Traffic Analyses Cumulative Scenario without the project since the project was not approved. These estimated 2010 volumes were then balanced and increased by two percent per year to the design year of 2025, as

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recommended by the City of Santa Cruz. This resulted in very conservative (high) estimations of potential traffic volumes in both the PM peak hours and the Saturday peak hours for 2025. Hence, the traffic volumes used in the analyses of this feasibility study were approved and initially provided by the City of Santa Cruz.

Both the 2025 PM peak hours and 2025 Saturday peak hours for both intersections with the growth rates included were provided by KHA as shown in **Figure 6**. It should be noted that the 2025 AM peak hour volumes were not available for the analyses.

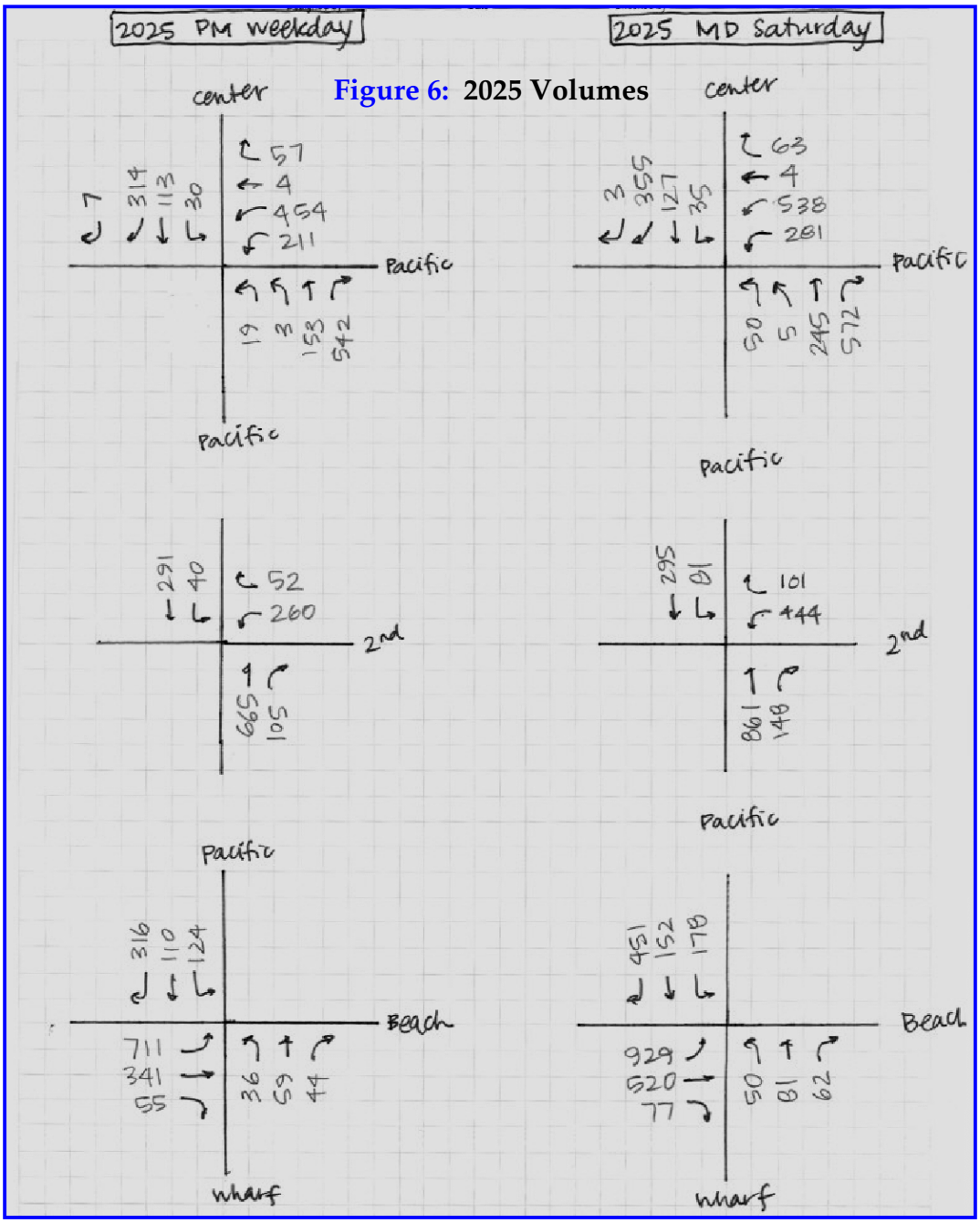

Figure 6: 2025 Volumes

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Bicycle & Pedestrian Volumes: Bicycle and pedestrian volumes were collected during the same time periods as the traffic volume surveys identified above. **Table 1** summarizes the results of the bicycle movement counts. Pedestrian volumes are summarized in **Table 2**. Both tables were provided by KHA. As seen in the tables, both intersections experience a high volume of bicycle activity, particularly at the Pacific Avenue /Beach Street intersection. It should be noted that although the east leg of Beach Street is one-way, bicyclists are permitted to ride in both directions along a bikeway located on the south side of Beach Street. A similar condition exists at the Pacific Avenue/Center Street intersection. West Cliff Drive is one-way for motorized vehicles, but two-way bicycle travel is permitted. **Figure 7** shows the beach street bikeway.

Figure 7: Beach Street Bikeway

When the pedestrian volumes were compared to the bicycle volumes, observed pedestrian volumes ranged from five to eight times greater at the Pacific Avenue/Beach Street intersection and three to five times greater at the Pacific Avenue / Center Street intersection depending on whether it was the weekday or weekend, respectively. **Figures 8** and **9** show the Pacific Avenue bikeway within the intersection and along West Cliff Drive. All of the streets within the study area provide sufficient bike paths along the roadways and within the intersections for a high amount of circulation for the ample quantity of bicyclists in the area.

Figure 8: Pacific Avenue Bikeway

Figure 9: West Cliff Bikeway

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Table 1: Peak Hour Bicycle Turning Movement Volumes

Pacific Avenue/ Beach Street			Pacific Avenue/ Center Street		
Turning Movement	Weekday PM	Saturday Midday	Turning Movement	Weekday PM	Saturday Midday
NBL	1	2	NBL [(a)]	2	7
NBT	19	24	NBL	0	0
NBR	9	15	NBT	0	2
SBL	1	5	NBR	6	2
SBT	28	36	SBL	0	1
SBR	3	5	SBT	10	12
EBL	9	4	SBT [(a)]	10	6
EBT	69	49	SBR	0	0
EBR	2	5	EBL	9	19
WBL	12	13	EBT	0	0
WBT	26	21	EBR	0	0
WBR	5	6	WBL	13	23
			WBL [(a)]	13	12
			WBT	0	0
			WBR	0	2

(a) Volume to W. Cliff Dr.

Table 2: Peak Hour Pedestrian Crossing Volumes

Pacific Avenue/ Beach Street			Pacific Avenue/ Center Street		
Crossing Approach	Weekday PM	Saturday Midday	Turning Movement	Weekday PM	Saturday Midday
North Leg	59	86	North Leg	42	38
South Leg	256	510	South Leg	11	17
East Leg	314	580	East Leg	61	146
West Leg	203	373	West Leg	62	253

Transit Volumes: The study area is served by Santa Cruz Metropolitan Transit District, which operates several bus routes through the two study intersections as shown in **Figure 10**. Routes generally function on a one-hour frequency during the weekday and weekends; however, some routes only operate on weekdays, on limited schedules during the day, or during specific periods of the year.

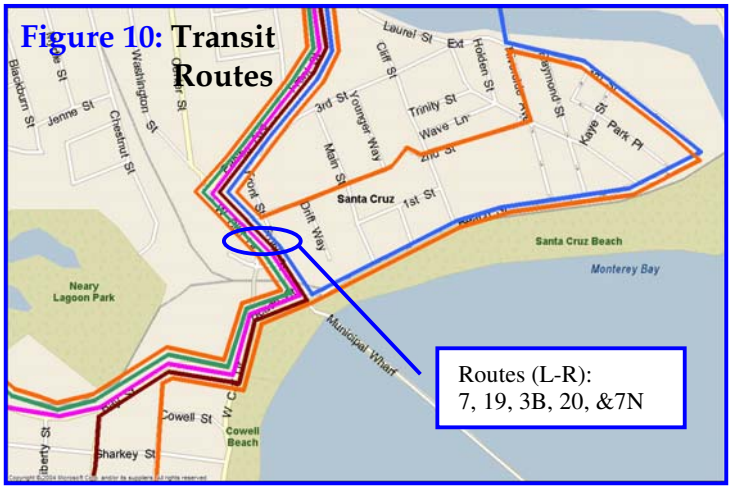

Figure 10: Transit Routes

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***Heavy Vehicle Movements*:** Other heavy vehicles occasionally travel through the two study intersections to make deliveries to commercial establishments in the vicinity. The largest of the vehicles is typically a WB-50 truck with semi-trailer, which is about 55 feet long. Other heavy vehicles (including transit buses) are shorter and would traverse the intersection adequately. The percentage of heavy vehicles at the intersection is typically less than one percent of all traffic volumes. Because of the presence of WB-50 trucks and transit vehicles, the geometry of a roundabout or traffic signal would need to be able to accommodate the large vehicles. The heavy truck percentages for the intersections were assumed to be a conservative one percent for all legs of the intersection based on direction by the City of Santa Cruz. Hence, the intersection designs should be designed primarily for passenger car traffic.

***Freight and Tourist Rail Frequency*:** The Pacific Avenue/Beach Street intersection has a rail line that runs roughly east-west through the intersection. The line splits west of the intersection with one track leading along the northern coastline to the Davenport Cement Plant. The other line turns inland and leads northward to Felton.

The cement plant line is used by the Union Pacific Railroad primarily to haul coal toward the plant and cement products on the return trip. The freight train operates two to three times per week. Occasionally, the Union Pacific Railroad operates an additional fright train during the week.

The Felton track is used by the Big Trees & Pacific Railway Company, which operates a tourist train between Roaring Camp and the Santa Cruz Boardwalk. The train operates during the year between June and December with the greatest frequency during the summer months. During this period, the Railway runs two round trips through the intersection each day. As illustrated in **Figure 11**, the intersection does not have crossing gates, rather flashing lights and signs only.

Figure 11: Pacific Street Rail Crossing

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IV. CAPACITY ANALYSES & CONCEPTUAL DESIGN ALTERNATIVES

Following KHA and RTE's review of the relevant site plan files and traffic volumes of the proposed project locations and roadways, capacity analyses were commenced for both the traffic signal and modern roundabout design alternatives. The information in this chapter set the initial parameters for the capacity calculations of the proposed roundabout location, geometry, and how they will function as a system with the proposed roadway network. No nearby access locations were identified to incorporate into the design with the exception of the southwest quadrant of the Pacific Street/Beach Street intersection.

GENERAL CAPACITY METHODOLOGY

Both the traffic signal and the modern roundabout capacity analyses are based on the general principles and performance measuring criteria identified in the Highway Capacity Manual. The Highway Capacity Manual[1] evaluates intersections based on vehicular delay as well as their Level of Service.

Traffic operations are assessed in terms of Level of Service (LOS) and delay. The level of service for an intersection is determined by the amount of delay experienced at the intersection. Delay is measured as the average time from when a vehicle stops at the end of the queue until the vehicle departs from the stop or yield line. The numerical value of delay per vehicle (typically in seconds or minutes) of a turning movement, approach, or total intersection is quantified with an assigned letter value or "grade" of measurement called LOS. The LOS is determined from the length of the average delay experienced at the intersection during the peak hour.

LOS is a concept that was developed by transportation engineers to quantify the level of operation of intersections and roadway segments. The LOS for most jurisdictions at intersections is classified in grades "A" through "F." These grades of LOS are the quantified terms that relate to the average delay per vehicle. A LOS "A" reflects full freedom of operation for a driver, while a LOS "F" represents very long delays of operation for a driver, forcing the driver to wait for adequate gaps in conflicting traffic. Under the HCM methodology, an intersection operating at LOS "F" is considered to have failed. Generally, LOS "D" and LOS "E" are considered the thresholds of acceptable operation for signalized and unsignalized intersections, respectively.

[1] Highway Capacity Manual, Transportation Research Board, 2000

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SIGNAL CAPACITY ANALYSES

After obtaining and reviewing all of the pertinent information regarding the roadways, site, and traffic volumes, an analysis of the proposed signal at the intersection using the software program *Traffix For Windows* was conducted to analyze the capacity requirements of the signalized intersection alternative for the design year of 2025. Traffix is based upon the HCM 2000 methodologies described above. Both the PM and Saturday peak hours were analyzed to ensure adequate signal operations during both peak conditions.

Signal Capacity Methodology: For signalized intersections under the Highway Capacity methodology, LOS is primarily measured in terms of average delay. The Volume to Capacity ratio (V/C) is used as an additional measure for quantifying the capacity utilization/design adequacy of the intersection. Typically, an intersection with a v/c ratio over 0.85 indicates the *potential* need for additional capacity on the approach. However, recent research has indicated that an intersection can operate at an acceptable level of service even though the V/C ratio exceeds 1. Therefore, a signalized intersection can operate at an acceptable LOS even if entering traffic volumes at that intersection exceed its theoretical capacity. Such situations occur primarily when unbalanced heavy demands occur on one or two approaches.

Based on the established design criteria for the signalized intersection analyses, the Traffix software program and engineering analyses produced the following results, as shown in **Figures 12** through **15**. The existing levels of service as reported by Traffix does account for pedestrians and bicyclists at the intersection, however, the model of the intersection may not account for real world conditions very accurately with the high pedestrian volumes present, particularly at the Pacific Avenue/Beach Street intersection. **Figures 12** through **15** below provide the following results for the 2025 design year:

- Required Lane Configurations (assumed no change from existing)
- Anticipated Queue Lengths of Each Lane
- Peak Hour Signal Timing
- Peak Hour Cycle Lengths
- LOS Results for PM and Saturday Peak Hours

The signal phasing was analyzed by RTE to determine the most accurate field conditions for each site. It was determined that the Pacific Avenue/Beach Street intersection would need to operate with split phasing (one approach at a time) for the north and south legs with permitted phasing on the west leg since no opposing traffic exists on the east leg. The Pacific Avenue/Center Street intersection could work well with permitted phasing only.

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Figure 12: Pacific/Center Signal Capacity – 2025 PM Peak Hour

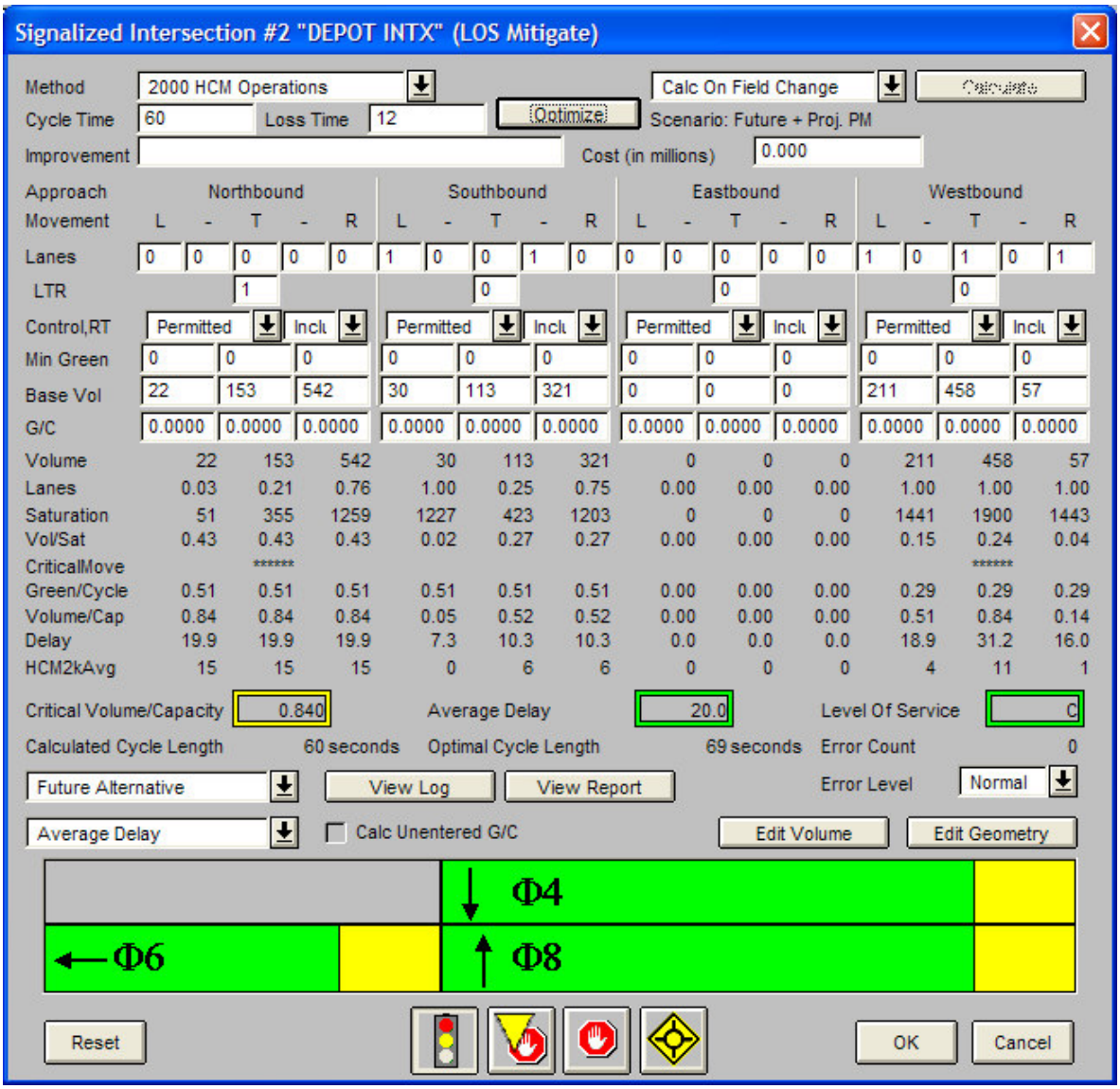

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Figure 13: Pacific/Center Signal Capacity – 2025 Saturday Peak Hour

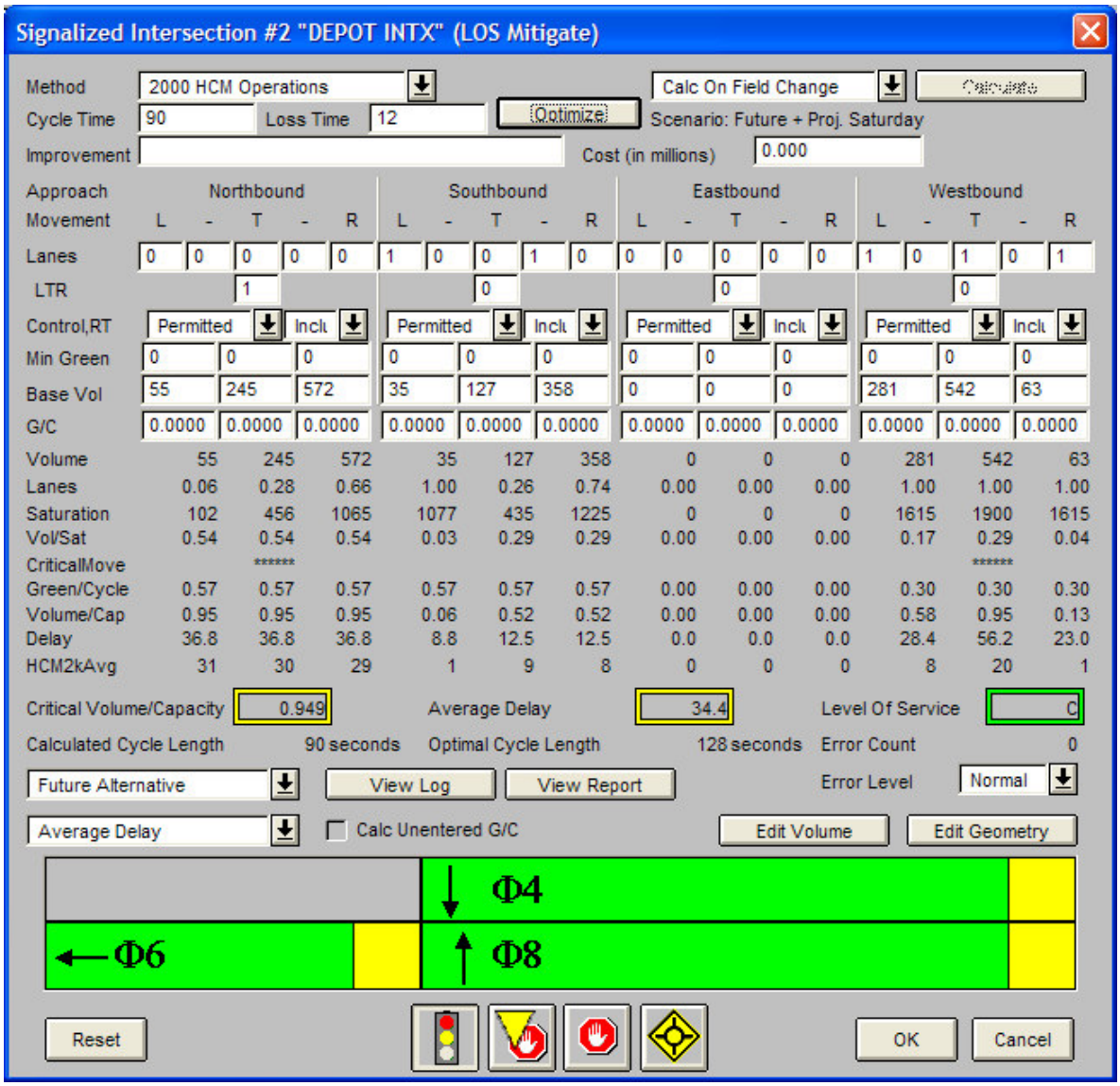

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Figure 14: Pacific/Beach Signal Capacity – 2025 PM Peak Hour

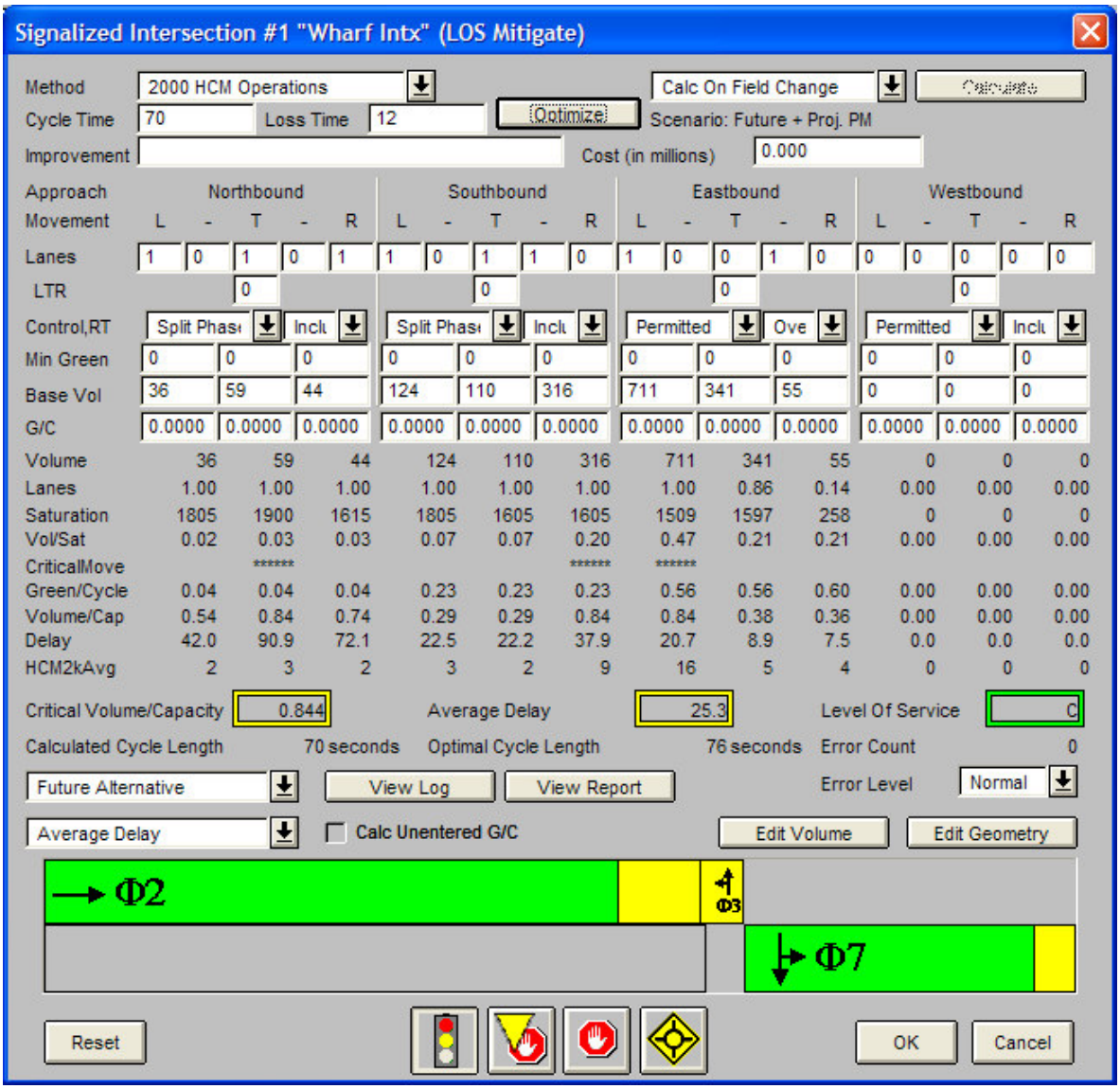

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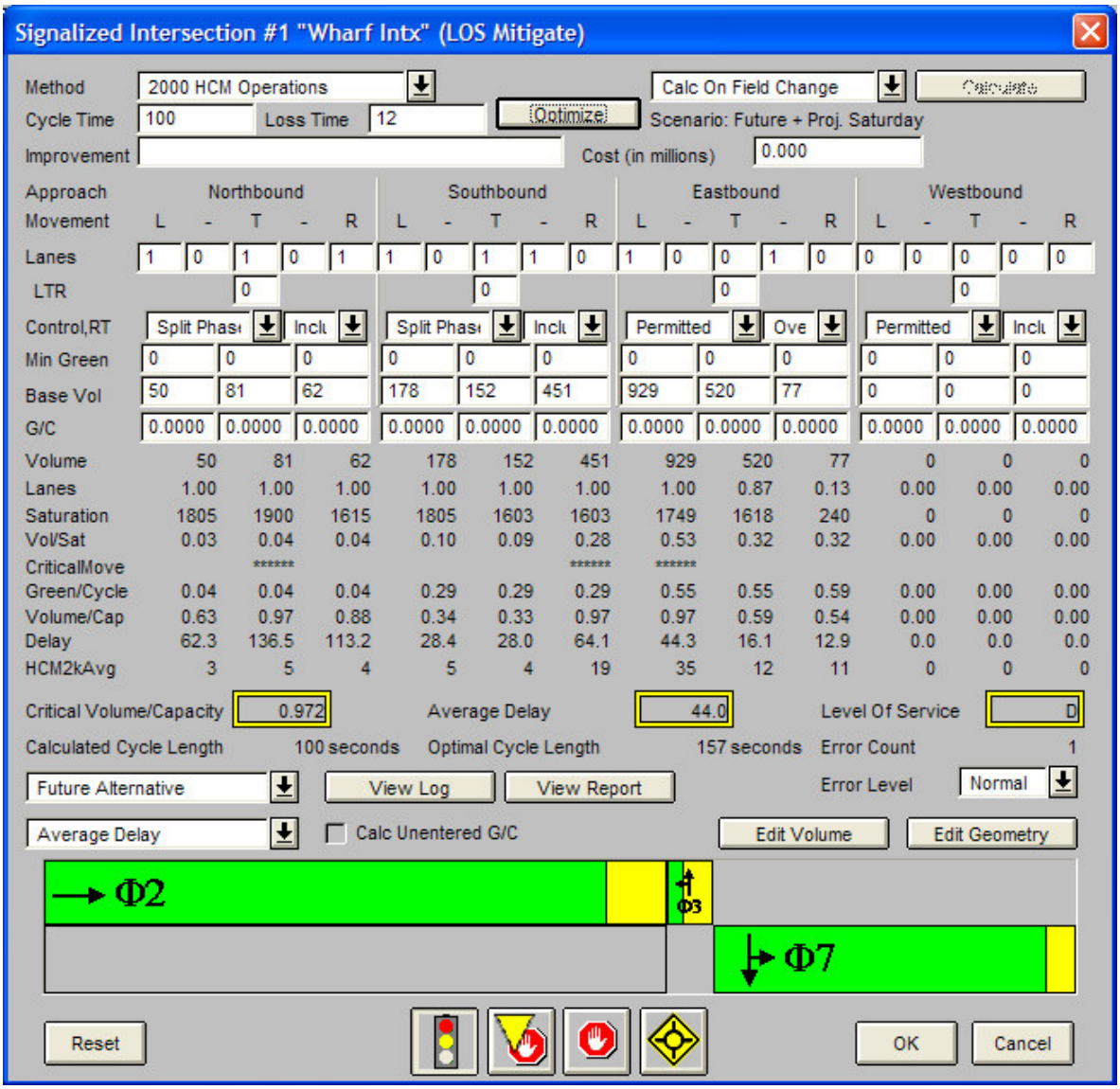
Figure 15: Pacific/Beach Signal Capacity – 2025 Saturday Peak Hour



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As shown in the signal capacity figures, the signalized intersections operate at a LOS C with 20.0 seconds of delay and LOS C with 25.3 seconds of delay in the PM peak hour for the Depot Park and Wharf intersections, respectively. The signals operate at a LOS C (nearly LOS D) with 34.4 seconds of delay and LOS D with 44.0 seconds of delay in the Saturday peak hour for the Depot Park and Wharf intersections, respectively (according to the Traffix Software).

One of the major issues with the signal at the Wharf intersection is the operational ability of the tollbooths on the Wharf pier. If traffic were required to stop and pay for the use of the Wharf pier, the signal may not operate well in allowing traffic to proceed from the tollbooths into the intersection. This topic will need further research with the City of Santa Cruz on how best to handle the pier booth operations (if not eliminate them) if signals are chosen.

The other major issue is the pedestrians and bicyclists actually using the signal's movements at the either intersection. Based on field observations at the Wharf intersection and other intersections near the Boardwalk, pedestrians and cyclists did not follow typical crossings and cautious behavior. They crossed the street at uncontrolled, unmarked locations with general disregard of vehicular traffic. It is questionable whether pedestrians and cyclists at these intersection locations will actually use the pedestrian push buttons and wait up to 100 seconds (1 minute 40 seconds) to cross the street. This issue should be discussed with City Staff and possibly at a City Council meeting.

It should be noted the capacity analyses for the signal reflect relatively the same lane configuration as shown in the field. Significant turn lane lengths are required for some of the left and right turn lanes at the signalized intersections. This is documented in **Table 3** as well as in the comparison analyses between the signals and roundabouts at the end of the next chapter following the roundabout capacity analyses. In addition, KHA performed separate LOS analyses at the intersections using a simulation software program called *Synchro*, which is based on the same *Highway Capacity Manual* methodology. Using the same 2025 traffic volumes, the analyses indicated that the Saturday peak hour dictates the geometric traffic signal timing and operational parameters for the intersections. The advantage of the Synchro software package is the additional capabilities of representing the high pedestrian and cyclist volumes and coordination. **Table 3B** represents the LOS analyses completed by KHA, which may be more accurate than the LOS analyses shown by the Traffix software.

Figures 16 and **17** graphically show the layout of each signal's roadway and lane requirements at each intersection. The signal capacity analysis output may also be found in the **Appendix** of this report. As shown in the layout exhibits, the signals have the least amount of impact on right-of-way with respect to

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reconfiguration of the existing conditions, parking, curb faces, and the like. However, the capacity analyses shown in **Tables 3** show significant queues and traffic impacts to the businesses and properties with respect to daily function or operation.

Table 3: Signalized Intersection Capacity Summary
Depot Park Intersection & Wharf Intersection
PM & Saturday Peak Hour Results

Intersection - Peak Period	Approach NB	SB	EB	WB	Intersection LOS
Depot Park Intersection - PM					
DELAY (in seconds)	19.9	10.1	N/A	26.4	**20.0**
LOS	B	B	-	C	**C**
No. of Cars In Queue / Lane (Avg. max.)	15	6	-	11	
Depot Park Intersection - Saturday					
DELAY (in seconds)	36.8	12.3	N/A	45.0	**34.4**
LOS	D	B	-	D	**C**
No. of Cars In Queue / Lane (Avg. max.)	31	9	-	20	
Wharf Intersection - PM					
DELAY (in seconds)	72.3	31.3	49.4	N/A	**25.3**
LOS	E	C	D	-	**C**
No. of Cars In Queue / Lane (Avg. max.)	3	9	16	-	
Wharf Intersection - Saturday					
DELAY (in seconds)	109.8	48.9	33.1	N/A	**44.0**
LOS	F	D	C	-	**D**
No. of Cars In Queue / Lane (Avg. max.)	5	19	35	-	

Source: RTE SantaCruz Feasibility Tables.xls

The traffic signal alternatives would require signal poles to be installed on most corners and a controller cabinet near the intersection. In addition, at the Pacific Avenue/Beach Street intersection, the existing railroad crossing equipment would be replaced with actuated crossing gates. The addition of a traffic signal would increase traffic speeds through the intersection (when the signal is green) and may increase the likelihood of a vehicle/train collision or conflict. The traffic signal would be interconnected with the railroad gates so that when the gates are in the down position, certain movements at the traffic signal would not be permitted. Coordination with Union Pacific Railroad will be required to install gate arms. A review of base mapping provided by the city indicates that traffic signal poles and other equipment can be located in within the existing right-of-way.

Intersection	2025 PM		2025 Sat	
	LOS	Delay	LOS	Delay
Traffic Signal				
Pacific Ave/Beach St	C	26.2	F	128.6
Pacific Ave/Center St	C	31.1	D	46.1

Table 3B: KHA Synchro LOS Calculations

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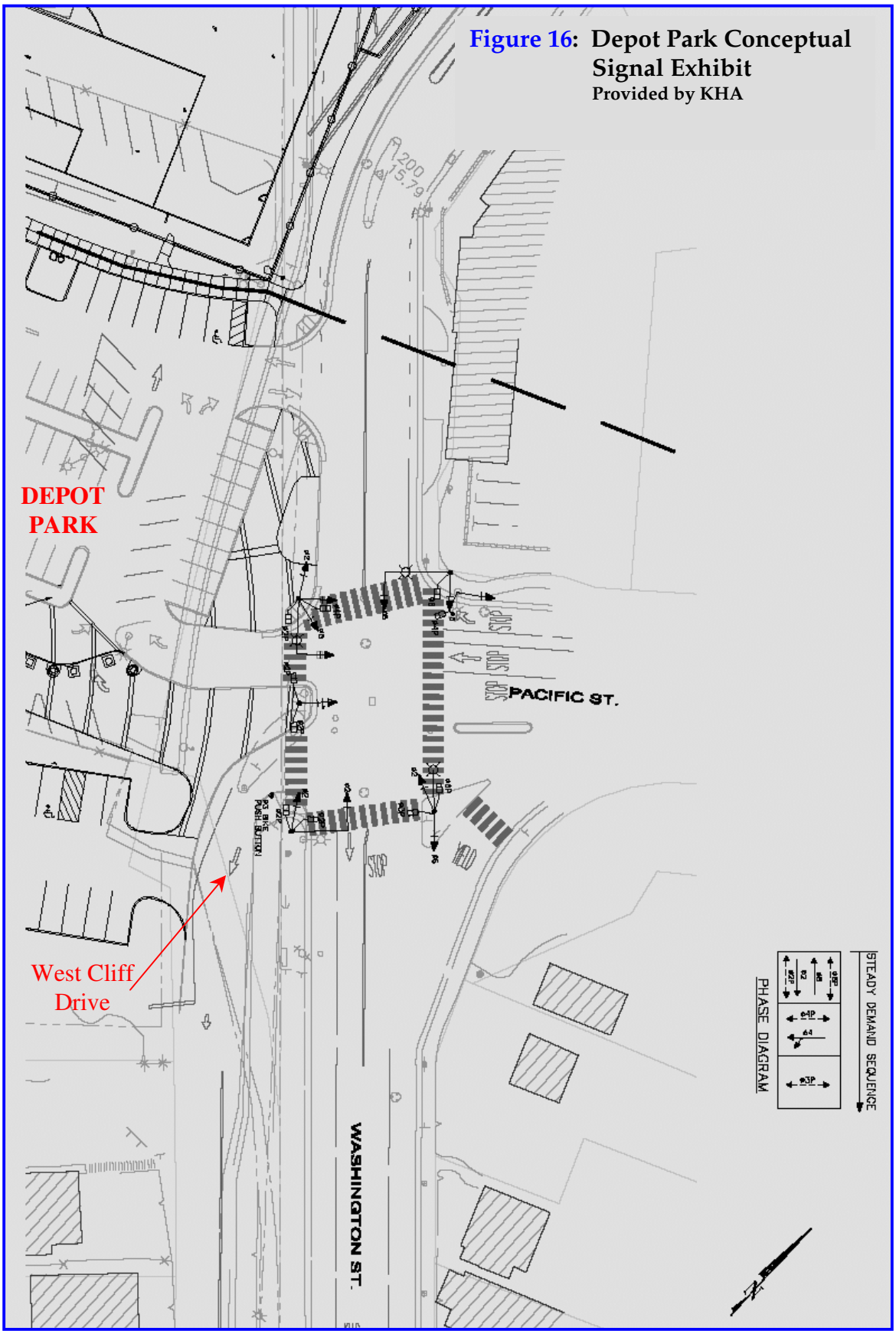

Figure 16: Depot Park Conceptual Signal Exhibit
Provided by KHA

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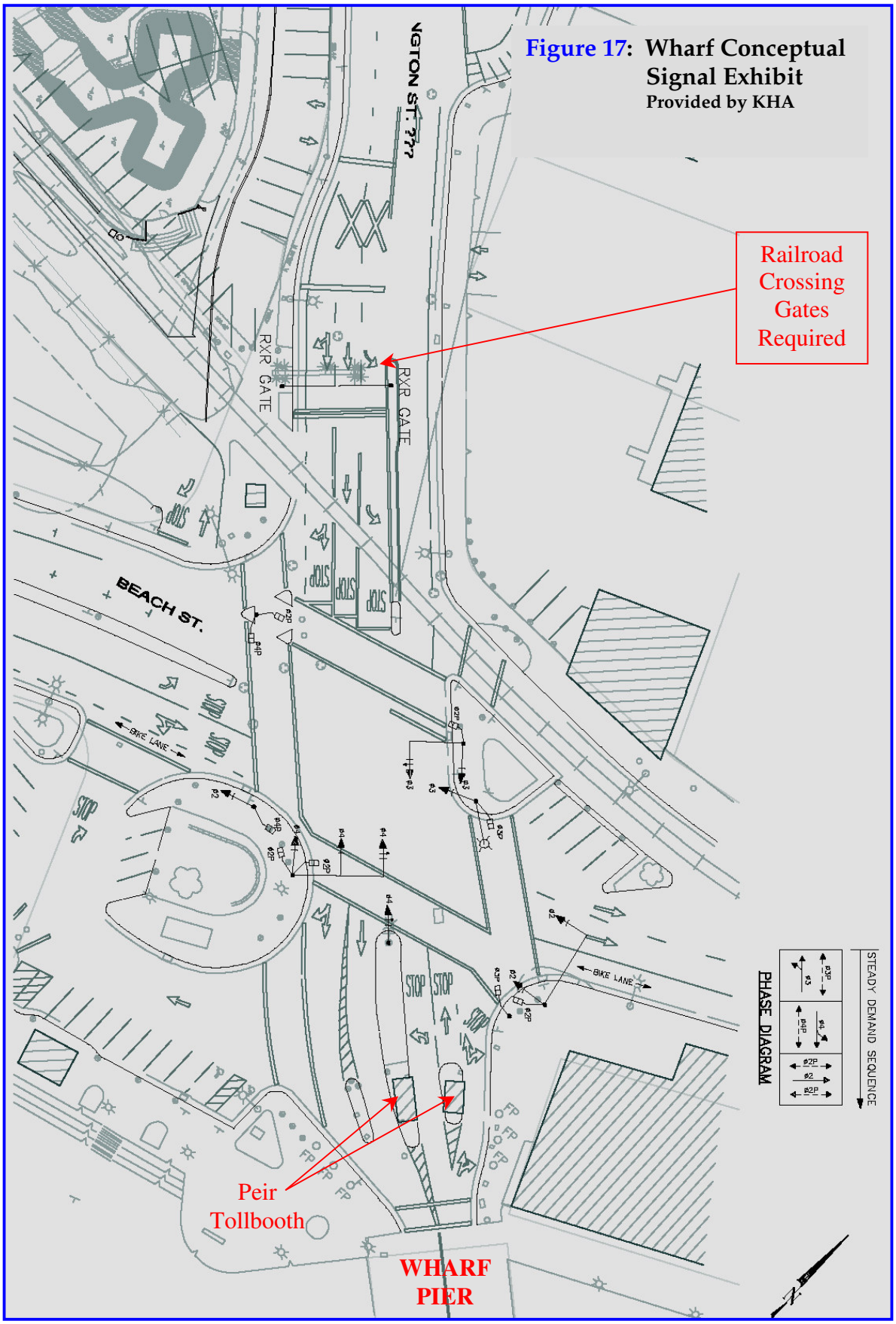

Figure 17: **Wharf Conceptual Signal Exhibit**
Provided by KHA

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ROUNDABOUT CAPACITY ANALYSES

After obtaining and reviewing all of the pertinent information regarding the roadways, site, and traffic volumes, a geometric analysis of the proposed roundabout at the intersection using the roundabout design software tool called RODEL was conducted.

The RODEL calculations provided the initial lane geometry and capacity requirements for the roundabout design alternative based on the design year traffic volumes. RODEL is based on empirical equations (observed and checked from field data) developed by the United Kingdom and utilizes specific geometric relationships to determine the capacity requirements of a roundabout. A further discussion on RODEL software and the geometric factors affecting roundabout capacity is provided in the next section of this report (RODEL Software and Roundabout Geometric Parameters). In general, RODEL (roundabout delay) calculates the required geometry for the roundabout to function within the desired capacity or, alternatively, to determine if the existing/planned geometry will be adequate with respect to capacity and delay. Since both the PM and summer Saturday peak hour volumes are part of the intersection design, separate RODEL calculations were completed for the intersection location to ensure the roundabout will operate appropriately under both peak hour traffic conditions. Since multiple sets of volumes (PM and Saturday) were requested as part of this design, separate RODEL calculations were completed for the design alternative to arrive upon the recommended configuration of the roundabout to ensure it will operate appropriately under both peak hour 2025 traffic conditions.

In addition, separate RODEL calculations were also performed under the peak minutes of the peak hour at an 85th percentile confidence level to ensure the proposed design would be adequate under the recommended geometric recommendations provided herein. Nearly all software programs that analyze traffic volumes with respect to operations and level of service are reported at a 50th percentile confidence level. RODEL offers a "design check" at an 85th percentile confidence level to determine if the roundabout has been designed adequately. This ensures adequate capacity of the roundabout during the peak hour.

***Roundabout Capacity Methodology*:** The predominant consideration in roundabout capacity analyses is the volume of the circulating traffic and the volume of the entering traffic on an approach. Traffic entering a roundabout will look for gaps in the circulating traffic in order to enter the roundabout. This behavior is called gap seeking. In addition to gap seeking, the geometric

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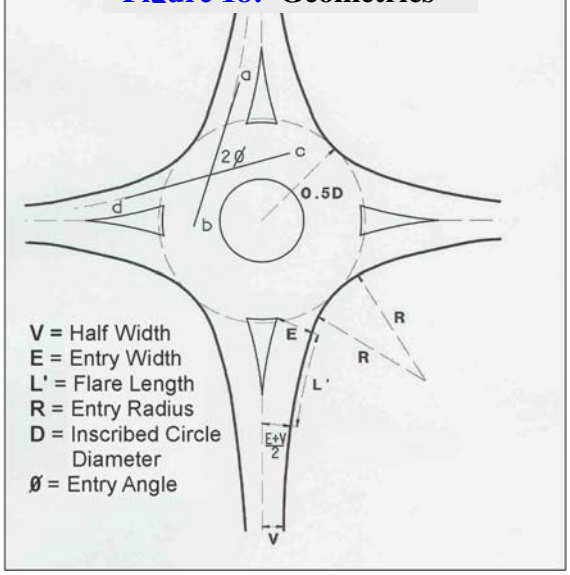
design of the roundabout affects the speeds and comfort level at which drivers will negotiate the roundabout. This also affects the capacity and safety of roundabouts.

The Highway Capacity Manual[2] evaluates roundabouts based on their volume to capacity ratios as well as their level of service. The volume to capacity (v/c) ratio describes the volume of traffic entering the circulating roadway from one approach as compared to the capacity of that approach. The capacity of an approach is dependent on the traffic volume within the circulating roadway at each specific approach. As the traffic within the circulating roadway goes up, the capacity of an approach would be reduced. Because of this, traffic engineers prefer to leave a "reserve capacity" for an approach. Typically, an intersection with a v/c ratio over 0.85 indicates the *potential* need for additional capacity on the approach. However, too much reserve capacity results in an unsafe (too fast or "too loose") roundabout design. Hence, careful and specific ***balance*** is needed in the design of roundabouts for safety and operational capacity purposes.

Roundabout Geometry Parameters / RODEL Software: Empirical studies in England have shown that the following six (6) dimensions collectively control traffic speed, capacity, and safety at a roundabout (see **Figure 18** below):

1. Inscribed Circle Diameter (ICD): the diameter of the outside curb of the circulating roadway. The ICD is established based on the tracking characteristics of the vehicle the roundabout is to accommodate, and the number of circulating lanes required to accommodate the projected traffic volumes. Increasing or decreasing this parameter (and thus increasing or decreasing the central island diameter) has minor effects on the safety of the roundabout (theoretically). However, it can be demonstrated that changing the size (ICD) of the roundabout can substantially change the safety of a roundabout design.

Figure 18: Geometrics



[2] Highway Capacity Manual, Transportation Research Board, 2000

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2. Half Width (V): the width of the approach roadway. This dimension is typically known before the roundabout design process has begun, as it is an element of the upstream roadway cross section. The half width has a significant impact on the capacity of the roundabout and some impact on travel speeds and safety of the roundabout.

3. Entry Width (E): the width of the entering roadway at the point of its intersection with the outside curb of the circulating roadway. Increasing or decreasing the entry width can have large impacts on the safety and capacity of the roundabout.

4. Flare Length (L'): the average *effective* length of flare from the transition between the point where the half width ends and the yield line. Flare length is accident neutral. As the flare gets longer the capacity of the roundabout increases. However, the entry speed increases and the roundabout's deflection decreases. Hence, flare length and entry width are related. If the approaches to the roundabout were parallel, the half width is equal to the entry width, and the flare length is zero (not recommended in modern roundabout designs and proven to increase accidents).

5. Entry Angle (∅) – the mean angle tangential between the direction of entry into the roundabout and tangential to the direction of the adjacent exit (or circulating traffic, depending on the size of the roundabout). The figure above shows the entry angle as half the angle formed by the junction of the tangent line (a-b) projected from the entry and the tangent line (c-d) projected from the adjacent exit. If all other dimensions remain constant, reducing the entry angle will increase the speed at which the roundabout can be entered which, in turn, tends to reduce the safety of the roundabout.

6. Entry Radius (R) – The radius of the outside curb of the entering roadway at its point of intersection with the outside curb of the circulating roadway. The entry radius is a critical component in roundabout design that determines many factors such as entry speed and entry deflection.

After inputting the future traffic turning movement volumes into RODEL for the peak hours, the roundabout was analyzed to verify the appropriate number of lanes at each entry or approach of the roundabout. Specifically, the recommended geometric requirements for the roundabout were analyzed to verify that the entering approach widths, average effective flare lengths, entry angles, entry radii, and roundabout diameter are adequate for the proposed

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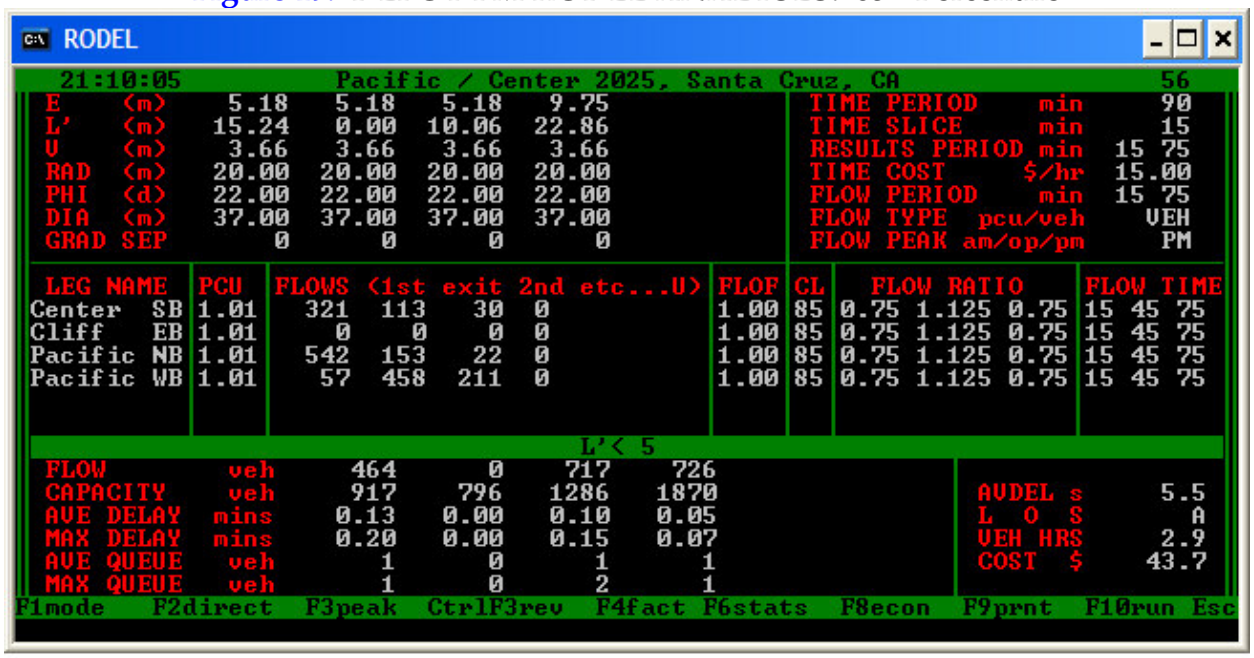
traffic volumes. The results of the RODEL analyses are shown in the following RODEL output in **Figures 19** through **22**:

Figure 19: DEPOT PM RODEL ANALYSES: 85th Percentile

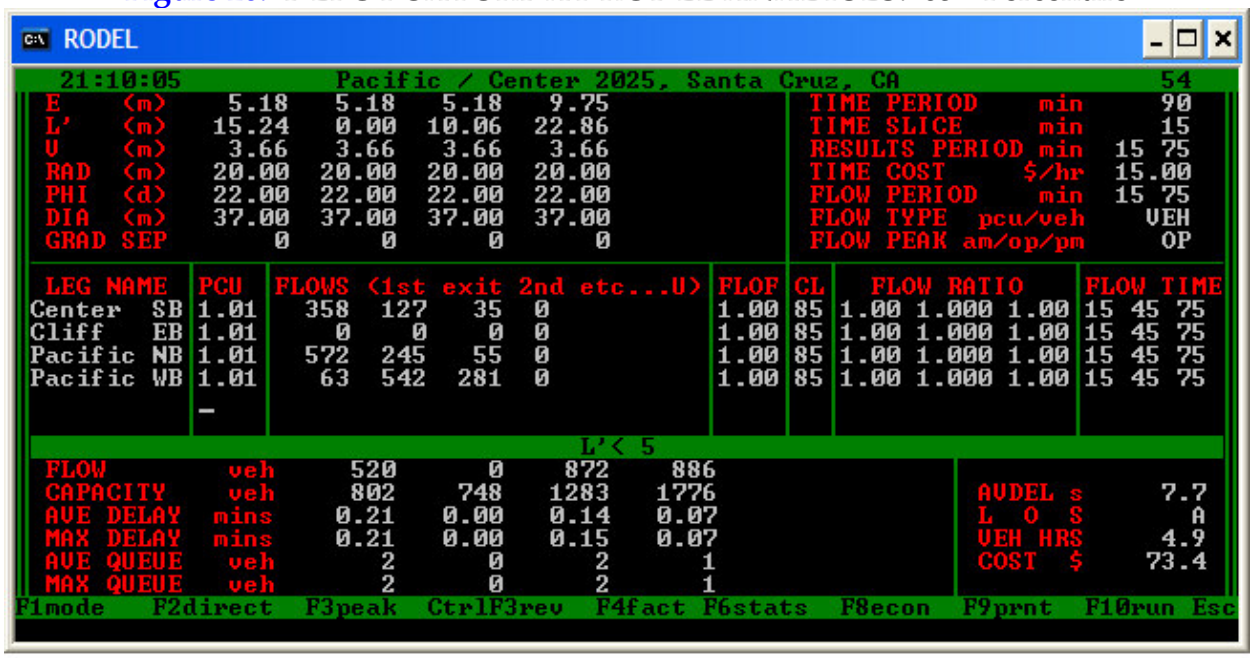


```
RODEL
21:10:05          Pacific / Center 2025, Santa Cruz, CA                    56
E    (m)    5.18    5.18    5.18    9.75          TIME PERIOD     min     90
L'   (m)   15.24    0.00   10.06   22.86          TIME SLICE      min     15
V    (m)    3.66    3.66    3.66    3.66          RESULTS PERIOD min  15 75
RAD  (m)   20.00   20.00   20.00   20.00          TIME COST      $/hr  15.00
PHI  (d)   22.00   22.00   22.00   22.00          FLOW PERIOD     min  15 75
DIA  (m)   37.00   37.00   37.00   37.00          FLOW TYPE  pcu/veh    VEH
GRAD SEP       0       0       0       0          FLOW PEAK am/op/pm     PM

LEG NAME  PCU  FLOWS (1st exit 2nd etc...U)  FLOF CL   FLOW RATIO    FLOW TIME
Center  SB 1.01   321  113   30  0           1.00 85 0.75 1.125 0.75 15 45 75
Cliff   EB 1.01     0    0    0  0           1.00 85 0.75 1.125 0.75 15 45 75
Pacific NB 1.01   542  153   22  0           1.00 85 0.75 1.125 0.75 15 45 75
Pacific WB 1.01    57  458  211  0           1.00 85 0.75 1.125 0.75 15 45 75

                                L'< 5
FLOW        veh     464      0    717    726
CAPACITY    veh     917    796   1286   1870          AVDEL s      5.5
AVE DELAY  mins    0.13   0.00   0.10   0.05          L  O  S        A
MAX DELAY  mins    0.20   0.00   0.15   0.07          VEH HRS      2.9
AVE QUEUE   veh       1      0      1      1          COST  $     43.7
MAX QUEUE   veh       1      0      2      1
F1mode  F2direct  F3peak  CtrlF3rev  F4fact F6stats  F8econ  F9prnt  F10run Esc
```

Figure 20: DEPOT SATURDAY RODEL ANALYSES: 85th Percentile



```
RODEL
21:10:05          Pacific / Center 2025, Santa Cruz, CA                    54
E    (m)    5.18    5.18    5.18    9.75          TIME PERIOD     min     90
L'   (m)   15.24    0.00   10.06   22.86          TIME SLICE      min     15
V    (m)    3.66    3.66    3.66    3.66          RESULTS PERIOD min  15 75
RAD  (m)   20.00   20.00   20.00   20.00          TIME COST      $/hr  15.00
PHI  (d)   22.00   22.00   22.00   22.00          FLOW PERIOD     min  15 75
DIA  (m)   37.00   37.00   37.00   37.00          FLOW TYPE  pcu/veh    VEH
GRAD SEP       0       0       0       0          FLOW PEAK am/op/pm     OP

LEG NAME  PCU  FLOWS (1st exit 2nd etc...U)  FLOF CL   FLOW RATIO    FLOW TIME
Center  SB 1.01   358  127   35  0           1.00 85 1.00 1.000 1.00 15 45 75
Cliff   EB 1.01     0    0    0  0           1.00 85 1.00 1.000 1.00 15 45 75
Pacific NB 1.01   572  245   55  0           1.00 85 1.00 1.000 1.00 15 45 75
Pacific WB 1.01    63  542  281  0           1.00 85 1.00 1.000 1.00 15 45 75
           -
                                L'< 5
FLOW        veh     520      0    872    886
CAPACITY    veh     802    748   1283   1776          AVDEL s      7.7
AVE DELAY  mins    0.21   0.00   0.14   0.07          L  O  S        A
MAX DELAY  mins    0.21   0.00   0.15   0.07          VEH HRS      4.9
AVE QUEUE   veh       2      0      2      1          COST  $     73.4
MAX QUEUE   veh       2      0      2      1
F1mode  F2direct  F3peak  CtrlF3rev  F4fact F6stats  F8econ  F9prnt  F10run Esc
```


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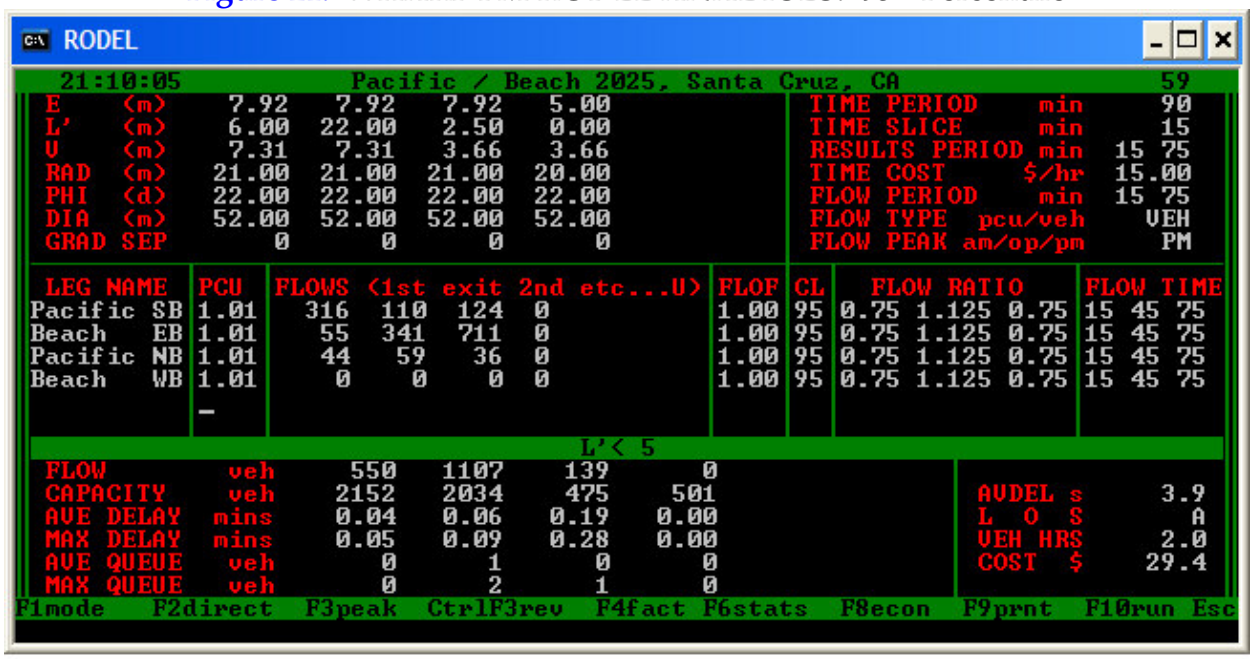
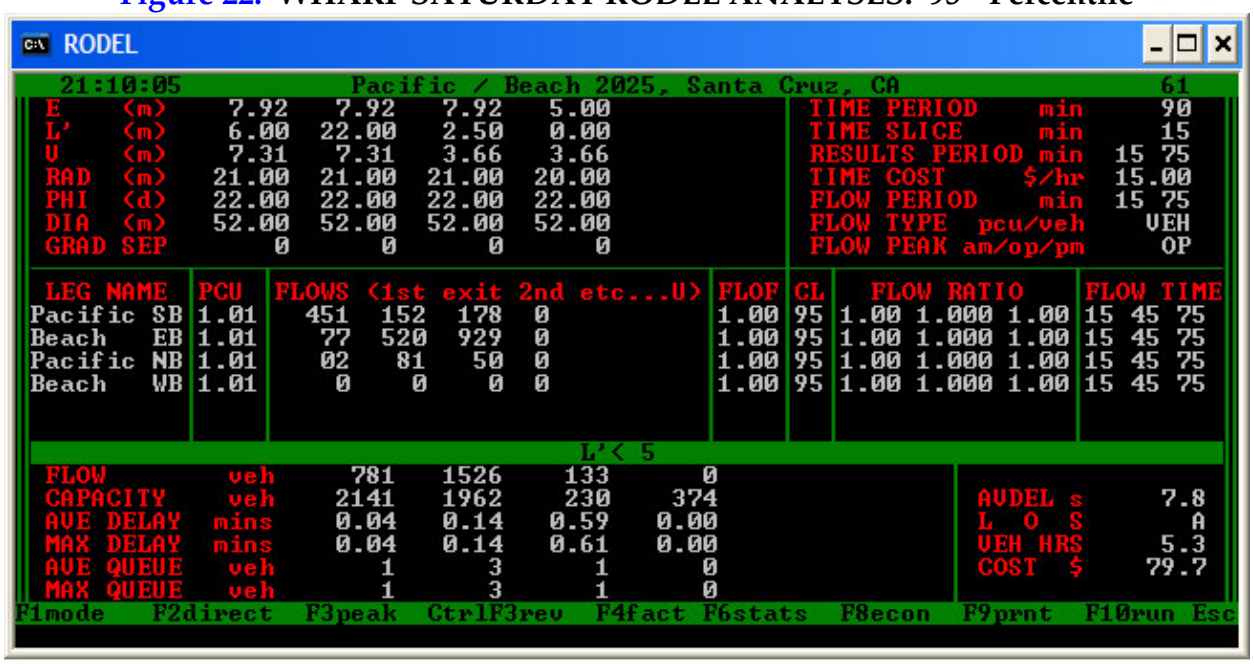
Figure 21: WHARF PM RODEL ANALYSES: 95th Percentile



```
RODEL
21:10:05          Pacific / Beach 2025, Santa Cruz, CA                59
E    (m)    7.92   7.92   7.92   5.00        TIME PERIOD     min      90
L'   (m)    6.00  22.00   2.50   0.00        TIME SLICE      min      15
V    (m)    7.31   7.31   3.66   3.66        RESULTS PERIOD min  15 75
RAD  (m)   21.00  21.00  21.00  20.00        TIME COST      $/hr  15.00
PHI  (d)   22.00  22.00  22.00  22.00        FLOW PERIOD     min  15 75
DIA  (m)   52.00  52.00  52.00  52.00        FLOW TYPE  pcu/veh    VEH
GRAD SEP       0      0      0      0        FLOW PEAK am/op/pm      PM

LEG NAME  PCU  FLOWS (1st exit 2nd etc...U)  FLOF CL   FLOW RATIO    FLOW TIME
Pacific SB 1.01   316  110  124  0          1.00 95 0.75 1.125 0.75 15 45 75
Beach   EB 1.01    55  341  711  0          1.00 95 0.75 1.125 0.75 15 45 75
Pacific NB 1.01    44   59   36  0          1.00 95 0.75 1.125 0.75 15 45 75
Beach   WB 1.01     0    0    0  0          1.00 95 0.75 1.125 0.75 15 45 75
           -
                              L'< 5
FLOW          veh    550   1107   139     0
CAPACITY      veh   2152   2034   475   501          AVDEL s      3.9
AVE DELAY    mins   0.04   0.06  0.19  0.00          L  O  S        A
MAX DELAY    mins   0.05   0.09  0.28  0.00          VEH HRS      2.0
AVE QUEUE     veh      0      1     0     0          COST  $     29.4
MAX QUEUE     veh      0      2     1     0
F1mode   F2direct  F3peak  Ctrl F3rev  F4fact F6stats  F8econ  F9prnt  F10run Esc
```

Figure 22: WHARF SATURDAY RODEL ANALYSES: 95th Percentile



```
RODEL
21:10:05          Pacific / Beach 2025, Santa Cruz, CA                61
E    (m)    7.92   7.92   7.92   5.00        TIME PERIOD     min      90
L'   (m)    6.00  22.00   2.50   0.00        TIME SLICE      min      15
V    (m)    7.31   7.31   3.66   3.66        RESULTS PERIOD min  15 75
RAD  (m)   21.00  21.00  21.00  20.00        TIME COST      $/hr  15.00
PHI  (d)   22.00  22.00  22.00  22.00        FLOW PERIOD     min  15 75
DIA  (m)   52.00  52.00  52.00  52.00        FLOW TYPE  pcu/veh    VEH
GRAD SEP       0      0      0      0        FLOW PEAK am/op/pm      OP

LEG NAME  PCU  FLOWS (1st exit 2nd etc...U)  FLOF CL   FLOW RATIO    FLOW TIME
Pacific SB 1.01   451  152  178  0          1.00 95 1.00 1.000 1.00 15 45 75
Beach   EB 1.01    77  520  929  0          1.00 95 1.00 1.000 1.00 15 45 75
Pacific NB 1.01    02   81   50  0          1.00 95 1.00 1.000 1.00 15 45 75
Beach   WB 1.01     0    0    0  0          1.00 95 1.00 1.000 1.00 15 45 75
                              L'< 5
FLOW          veh    781   1526   133     0
CAPACITY      veh   2141   1962   230   374          AVDEL s      7.8
AVE DELAY    mins   0.04   0.14  0.59  0.00          L  O  S        A
MAX DELAY    mins   0.04   0.14  0.61  0.00          VEH HRS      5.3
AVE QUEUE     veh      1      3     1     0          COST  $     79.7
MAX QUEUE     veh      1      3     1     0
F1mode   F2direct  F3peak  Ctrl F3rev  F4fact F6stats  F8econ  F9prnt  F10run Esc
```

Based on the established design criteria for the roundabout intersection analyses, the RODEL software program and engineering analyses produced the results above. The analyses provide the following results for the 2025 design year:

- Required Lane Configurations
- Anticipated Queue Lengths of Each Approach

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- Roundabout Geometry (E, L', R, Phi, D)
- LOS Results for AM and PM Peak Hours

The pedestrian and bicycle volumes were accounted for by increasing the software's normal 50th percentile confidence level analyses parameter to at least an 85th percentile confidence level analysis parameter for the Depot Park intersection and a 95th percentile for the Wharf intersection. In addition, the Saturday capacity analyses did not use an even distribution of arrival and departure rates (bell curve), rather, a constant flow of steady arrivals and departures was used for conservative purposes and to account for additional peak pedestrian interferences. This is an appropriate way to account for bicyclists and pedestrians and has been recommended by the author/creator of the RODEL software (Barry Crown).

As shown in **Figures 19** and **20**, the Depot Park intersection operates at a LOS A with 5.5 seconds of delay in the PM and LOS A with 7.7 seconds of delay in the summer Saturday peak hours of 2025. As shown in **Figures 21** and **22**, the Wharf intersection operates at a LOS A with 3.9 seconds of delay in the PM and LOS A with 7.8 seconds of delay in the Saturday peak hours of 2025. The recommended roundabout diameters are 110 feet and 130 feet, respectively. These diameters of roundabouts are quite small and not typical, however, the right-of-way constraints require the smaller diameters. This is acceptable due to the existing 25 MPH roadway speeds. Also shown, the required lane configurations of each approach of the intersection are based on the turning movement conflicts and the approaching roadway configuration and are as follows:

Depot Park Intersection:

- Northbound: Single Lane Approach with Snagged Right Turn Lane
- Southbound: Single Lane Approach
- Eastbound: No Approach
- Westbound: Single Lane Approach

Wharf Intersection:

- Northbound: Two Lane Approach with Snagged Right Turn Lane
- Southbound: Single Lane Approach with Snagged Right Turn Lane
- Eastbound: Two Lane Approach
- Westbound: No Approach

In addition, relatively insignificant stopped queue lengths develop at the intersection. This is documented in **Table 4** below as well as in the comparison analyses between the signals and roundabouts at the end of the next chapter following these capacity analyses. **Figures 23** and **24** graphically represent the ***conceptual*** layouts of the roundabouts at each intersection.

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Table 4: Roundabout Intersection Capacity Summary

Depot Park Intersection & Wharf Intersection

PM & Saturday Peak Hour Results

Intersection	Approach NB	Approach SB	Approach EB	Approach WB	Intersection LOS
Depot Park Intersection - PM					
DELAY (in seconds)	6.0	7.8	N/A	3.0	**5.5**
LOS	A	A	-	C	**A**
No. of Cars In Queue / Lane (Avg. max.)	1	1	-	1	
Depot Park Intersection - Saturday					
DELAY (in seconds)	8.4	12.6	N/A	4.2	**7.7**
LOS	A	B	-	A	**A**
No. of Cars In Queue / Lane (Avg. max.)	2	2	-	1	
Wharf Intersection - PM					
DELAY (in seconds)	11.4	2.4	3.6	N/A	**3.9**
LOS	B	A	A	-	**A**
No. of Cars In Queue / Lane (Avg. max.)	0	0	1	-	
Wharf Intersection - Saturday					
DELAY (in seconds)	35.4	2.4	8.4	N/A	**7.8**
LOS	D	A	A	-	**A**
No. of Cars In Queue / Lane (Avg. max.)	1	1	3	-	

Source: RTE — SantaCruz Feasibility Tables.xls

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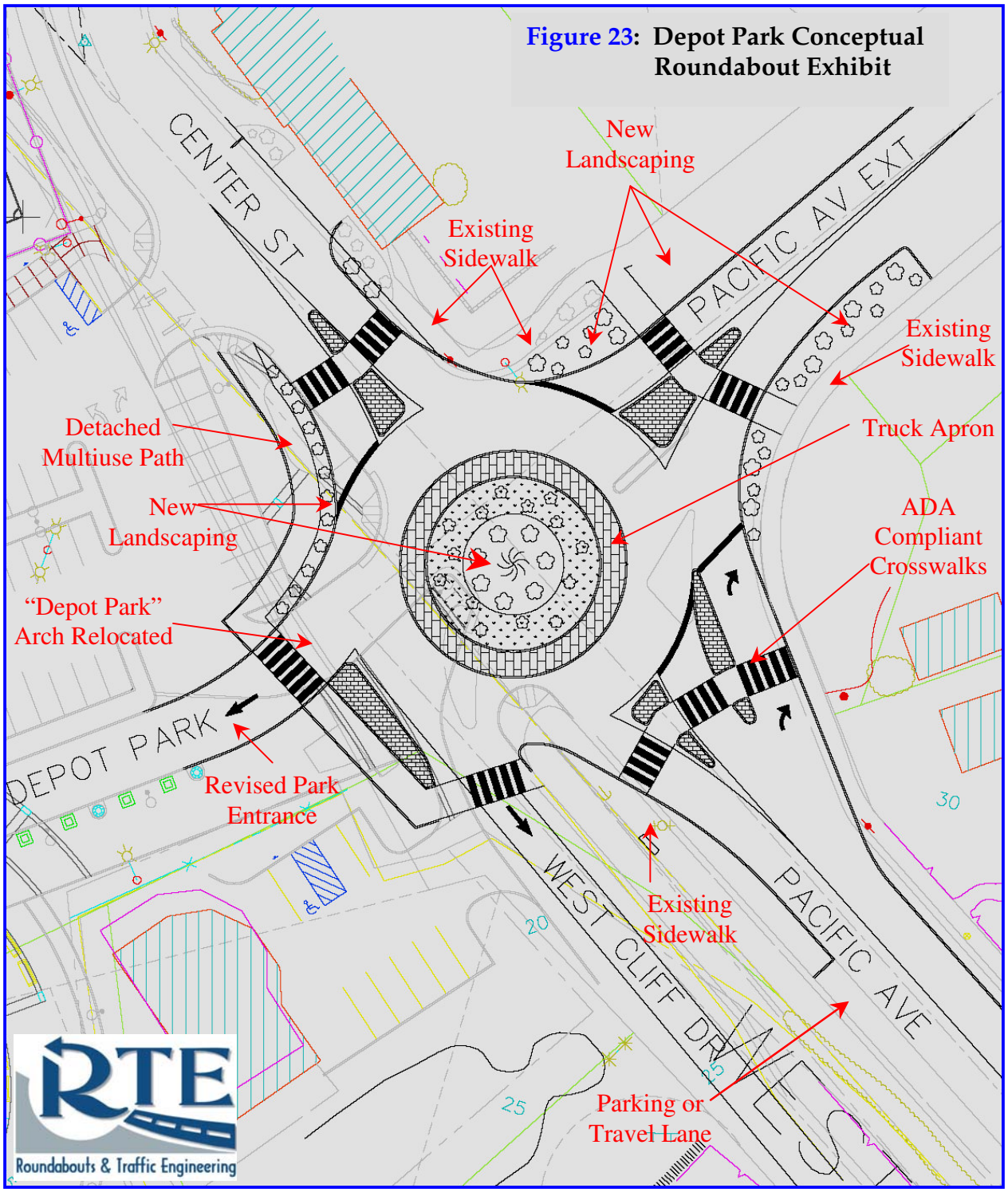

Figure 23: Depot Park Conceptual Roundabout Exhibit

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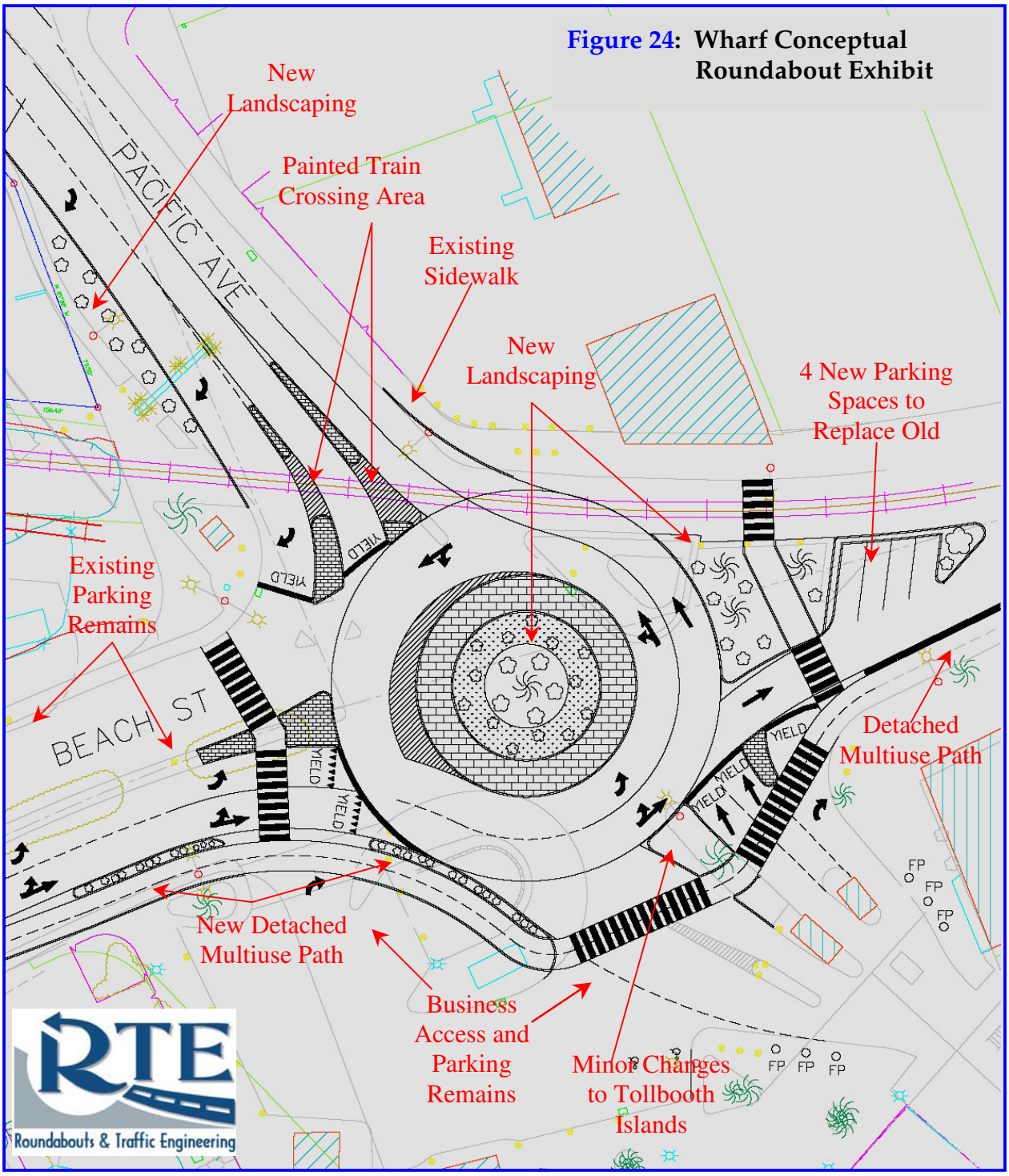

Figure 24: Wharf Conceptual Roundabout Exhibit

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The specific design vehicle, identified as a WB-50, was overlaid on the roundabout designs and preliminarily checked for the truck turning movement capabilities. The software program Autoturn 4.2 was used for the turning movements of the intersection roadways to verify proper truck/bus turning radii through the roundabout for every approach and movement. This determined the width of the truck apron and verified if the turning movements could handle the design vehicle. As a result of all of the above analyses the initial roundabout concepts were developed. As specified above, the intersection is anticipated to only have 1% trucks; hence the designs do not accommodate large trucks traversing the roundabout simultaneously (side by side) in the intersection. This is typical for most roundabouts.

It should be noted that **Figures 23** and **24** are illustrations or conceptual roundabout exhibits at a 30% level of detail that were developed for the two intersections for preliminary discussion purposes only. The sketches simply demonstrate the recommended design lane configurations and initial geometry recommendations with special consideration of pedestrians, the rail line, and traffic flow. The actual design plans (PS&E) to be completed may be slightly different than what is shown in the exhibits. Following this report, a full roundabout design with proper geometrics, signing, striping, lighting, grading, and landscaping design plans will need to be developed if the roundabout alternative is selected by the City of Santa Cruz. The provided conceptual roundabout design will require further modifications for final PS&E plans.

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V. CAPACITY COMPARISONS

This chapter compares the proposed roundabout alternatives to the proposed signal alternatives with respect to a various number of capacity related issues such as the calculated delay and level of service, lane geometry, right of way, and the average queuing for the design year of 2025. It should be noted that the future 2025 Level of Service at the existing stop-controlled intersections would be LOS F (failure).

SIGNALS VS. ROUNDABOUTS

Capacity (Delay & LOS): **Table 5** illustrates the 2025 capacity comparisons of the traffic signals and the modern roundabouts for each intersection. The actual Traffix and RODEL analyses were shown in the previous chapter of this report. As shown in the tables below, the modern roundabouts' level of services operate significantly superior to the signalized intersections.

Table 5: Capacity Comparison
Depot Park Intersection & Wharf Intersection
PM & Saturday Peak Hour Results - 2025

Intersection	Signal[1] PM	Signal[1] SAT	Roundabout[1] PM	Roundabout[1] SAT
Depot Park Intersection				
DELAY (in seconds)	20.0	34.4	5.5	7.7
LOS	C	C / D[2]	A	A
Wharf Intersection				
DELAY (in seconds)	25.3	44.0	3.9	7.8
LOS	C	D / F[2]	A	A

Note1: 2025 LOS Calculations Result in An Approximated LOS Due To Pedestrian and Bicycle Variability

Note2: Synchro Software LOS Results

Source: RTE SantaCruz Feasibility Tables.xls

Lane Geometry: In addition, the required lane geometry should also be compared in order to show the improvements necessary prior to the intersection. The required lane geometry was described in the previous sections of this report. The following major differences between the roundabout and signal for the intersection are as follows:

- The roundabout requires only single lane approaches for the Depot Park intersection. However, an additional snagged right turn lane

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(northbound) on Pacific Avenue was provided since the existing conditions allow for it and reconstruction costs would be lower.

- The signal at the Depot Park intersection utilizes the existing lane configuration for all approaches.
- The roundabout at the Wharf intersection requires less roadway approaches than the signal with only a single southbound lane. A snagged right turn lane has been added due to the available roadway in the existing conditions and additional safety.
- The signal at the Wharf intersection must utilize the existing lane configuration for all approaches.
- The signal would operate with permitted phases (stopped traffic on other legs), whereas the roundabout would operate with yield conditions on all approaches with continuous traffic flow.
- The roundabout provides safer pedestrian crossings with only two lanes of one-way traffic to cross at the largest distances (others are only a single crossing lane) with slow or yielding traffic speeds. The signal requires pedestrians to cross five lanes of two-way traffic while several phases of traffic remain stopped.

The signal would have less of an impact to the existing conditions lane configuration. However, the required lane geometry for the roundabout is less than existing conditions due to a fewer number of required approach lanes, less required storage lengths, less lane changing prior to the intersection, and a smaller amount of turning movement conflicts when compared to signalization of the intersection.

Right-of-Way: Digital base mapping and aerial photography were reviewed to identify constraints for roundabouts and traffic signal equipment. Based on conversations with City staff it was determined that additional right-of-way is generally not available for either traffic control alternative. Additional constraints include existing buildings, railroad tracks, and public improvements associated with the Municipal Warf. On-street parking was determined to be expendable but only in small quantities.

The signals would have less of an impact on the adjacent right-of-way at the intersections than the roundabouts. However, the traffic volumes that would queue up at the intersections would block business accesses and parking, which would ultimately have a greater impact on the roadways. The signals are able to utilize the existing lane geometry and can be implemented easily, with the exception of the new railroad crossing gates at the Wharf intersection. The roundabouts require minor additional right-of-way from each intersection.

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At the Depot Park intersection, the east and west legs require notable curb and gutter modifications with the addition of landscaping. The existing "Depot Park" overhead sign would need to be relocated further west. The Depot Park parking lot would still function well if not better than the previous design. The east leg would allow for additional landscaping and beautification to the intersection area. The north and south legs are relatively uninterrupted.

At the Wharf intersection, the north, south, and east legs would not require additional right-of-way. However, the west leg impacts a planter box. Only one parking space is lost with the new roundabout on the east side of the intersection. All remaining business owners would not be impacted. The most effective method to compare the right-of-way impacts is to compare the conceptual design exhibit for each alternative. Please refer to **Figures 16** and **17** for the signal layouts and **Figures 23** and **24** for the roundabout layouts.

Queue Lengths: A comparison can also be made between the queue lengths formed from the traffic signals and the modern roundabouts for the 2025 conditions. Typically, roundabouts have significantly less queue buildup than traffic signals since roundabouts provide a continuous flow pattern. The RODEL analyses show the average and maximum peak hour queue lengths (in vehicles). Based on the capacity calculations in the previous chapter of this report, the following average queue length summary information can be provided for the roundabout and signal analyses.

Table 6: Queuing Comparison (Feet)
Depot Park Intersection & Wharf Intersection
PM & Saturday Peak Hour Results

Intersection	Approach NB	SB	EB	WB
DEPOT PARK INTERSECTION	Avg Queue / Lane (max) in Feet			
PM Peak Hour				
Signal	375	150	-	275
Roundabout	25	25	-	25
Saturday Peak Hour				
Signal	775	225	-	500
Roundabout	50	50	-	25
WHARF INTERSECTION				
PM Peak Hour				
Signal	75	225	400	-
Roundabout	0	0	25	-
Saturday Peak Hour				
Signal	125	475	875	-
Roundabout	25	25	75	-

Source: RTE SantaCruz Feasibility Tables.xls

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As shown in **Table 6**, the queue lengths are quite short for the roundabouts compared to the traffic signals with the same traffic volumes. The roundabouts require less roadway widening and less vehicular stacking at the intersection. The roundabouts will also improve driver behavioral characteristics, roadway level of service, environmental impacts, and aesthetical effects to the area.

However, despite the traffic control device chosen, as the traffic volumes increase and the surrounding area experiences additional growth, the intersections should be re-evaluated for proper changes in the projected turning movement volumes. Although not anticipated in the near future, geometric modifications, signal timing changes, or the addition of lanes may be needed if the traffic volumes change significantly from the assumed analysis design year volumes.

Capacity Summary: In summary, there are demonstrated capacity benefits for the modern roundabout operations versus the signalized alternative at both intersections. This can mostly be explained by the basic operational characteristics of a signal versus a roundabout. A signal requires traffic flows to stop and wait for the permission of the traffic signal to move forward. Whereas the roundabout has continuously flowing traffic with yield conditions to approaching vehicles. The approaching traffic flow is only required to search for an available gap in the traffic stream of the roundabout's circulating roadway, which will occur quite frequently with the yielding approaches and since the traffic flows have separate turning movements.

Since the decision making for the driver to enter the roundabout is based on driver judgment, similar to a four-way stop controlled intersection, for only a right turn movement, the natural driver behavioral instinct occurs at the yield line, which is different for every type of driver (aggressive, passive, etcetera). Hence, adequate gaps for conflicting traffic movements automatically form in the roundabout and all traffic continues to flow with minimal delay, relatively slow vehicular speeds, and a high amount of safety. In most studied cases, slowing all traffic at an intersection with continuous flow has been proven to provide faster travel times for a corridor than stopping selected phases or approaches.

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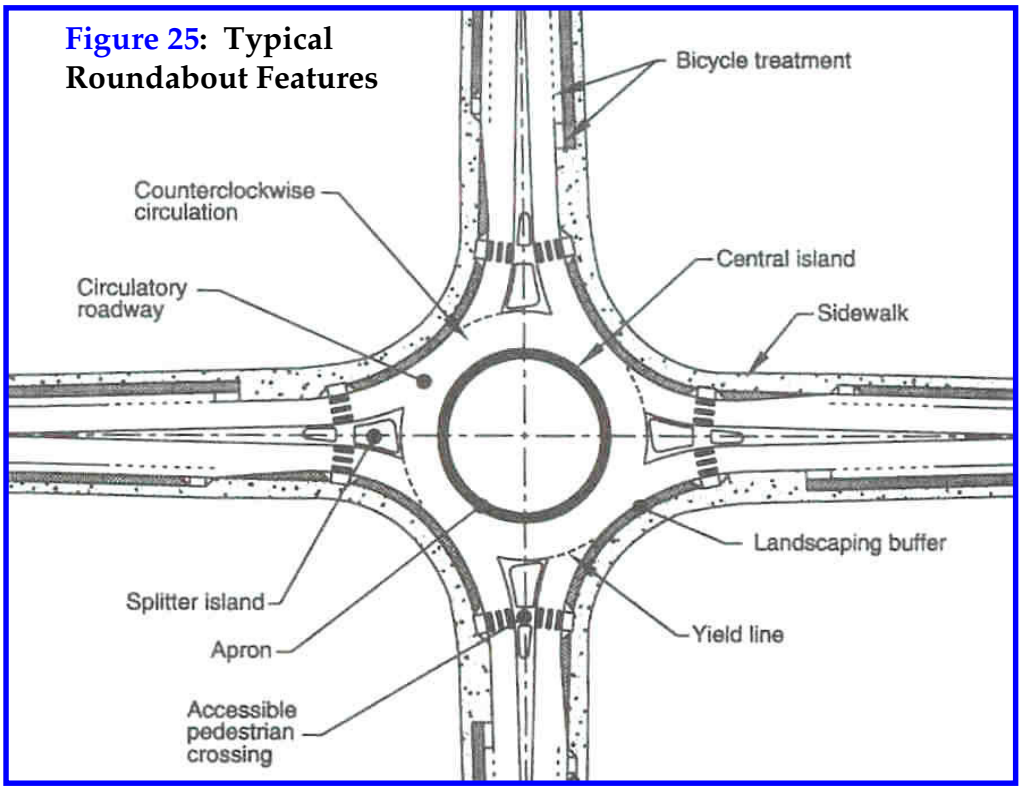
SAFETY COMPARISONS

The previous chapters analyzed the capacity requirements of both a traffic signal and a modern roundabout at each intersection and provided conceptual design illustrations. The previous section demonstrated the capacity comparisons between the two alternatives. This chapter discusses the safety considerations and comparisons between roundabouts and signalized intersections as well as the function of the high amount of pedestrians at the intersections.

GENERAL ROUNDABOUT INFORMATION

Modern roundabouts are a type of circular intersection with specific design and traffic control features to control driver behavior. **Figure 25** identifies key modern roundabout features[3] required in roundabout design. Some of these features include yield control for entering traffic, channelized approaches, and a geometric design that ensures travel speeds are relatively low and safe. Modern roundabouts are unique from other circular intersections in that they use *splitter islands* (or curved medians) and physical geometry (raised concrete curb) to control and slow the speeds of vehicles entering the roundabout and traveling

Figure 25: Typical Roundabout Features



[3] U.S. Department of Transportation, Federal Highway Administration, *Roundabouts: An Information Guide*, 2000

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through the roundabout. The splitter islands help control speeds, guides drivers into the roundabout, physically separate entering and exiting traffic streams, significantly increases intersection safety, deters wrong-way movements, and provides safe pedestrian crossings. Modern roundabouts are designed and sized to accommodate specific design speeds, traffic flows, and large design vehicles.

Roundabouts improve the safety of an intersection through the introduction of a raised island in the center of the intersection and the conversion of all movements through the intersection to right turns thus eliminating vehicle-to-vehicle crossing conflicts.

The horizontal and vertical geometry of a roundabout is crucial to the operation and safety of the roundabout. Since the capacity of a roundabout is dependent on the turning movement volumes at each approach, the capacity or RODEL analyses completed above identified the required lane geometry and the number of entries required for the design. As depicted in the RODEL analyses, the correct geometric design is identified only with respect to the capacity. The safety factors of each design's geometry now become the primary concerns for the operational adequacy of the roundabout. The "body language" of the roundabout directly relates how comfortable and safe drivers will use the roundabout. The body language of the roundabout must adequately communicate to the driver in order to avoid accident problems.

The geometric analysis of a roundabout evaluates the geometric parameters that affect roundabout *capacity and safety*. However, for the purposes of this feasibility study, the capacity and safety of the roundabout have been divided into separate sections for ease of reader comprehension. The geometric safety design includes the design of fast path speeds and speed consistency within the roundabout design. The roundabout designs also consider other safety parameters such as vehicle deflection into the roundabout, splitter island design, crosswalk locations and the ability of the design vehicle to negotiate the roundabout.

In addition, a large part of roundabout design involves specific non-geometric details such as the roundabout's signing, striping, and lighting of the roundabout. These intersection locations have not progressed to this level of detail yet. However, many other proposed roundabout features were analyzed during the roundabout designs.

The design of roundabout entries and exits is an intricate and complicated procedure that involves numerous variables that need to be addressed to ensure a safe design and adequate capacity. Some of these variables include the following:

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- Entry Width
- Entry Flare
- Entry Angle
- Entry Radius
- Entry Deflection
- Entry Path Curvature
- Entry Path Overlap
- Entry Speeds
- Fast Path Speeds
- Speed Consistency
- Sight Distance
- Exit Path Overlap
- Entry and Circulating Visibility
- Splitter Island Design
- Exit Lanes and Geometry
- Pedestrian Crossings/Crosswalks
- Maneuverability of Large Trucks
- Vertical Design Parameters

SAFETY COMPARISONS (RESEARCH FACTS & STATISTICS)

The best method of comparing traffic signals to roundabouts is through "before" and "after" case study results with respect to roundabouts compared to other types of stop controlled and signalized intersections. The Insurance Institute for Highway Safety (IIHS) performed a study[4] titled *Crash Reductions Following Installation of Roundabouts in the United States* in 2000 on 24 U.S. intersections that had converted both signalized intersections and stop-controlled intersections to modern roundabouts. Similarly, the Institute of Transportation Engineers (ITE) also completed a related study[5] in 2002. The US Department of Transportation, Federal Highway Administration (FHWA) also produced *Roundabouts: An Information Guide* in 2000 with safety statistics contained. All of these studies revealed very consistent "before" and "after" results with respect to the safety of modern roundabouts compared to other types of stop controlled and signalized intersections. The following is a brief summary of these results with regard to the extent to which modern roundabout conversions improved the accident safety of the intersections:

- 38 - 40% average reduction in all crash types
- 74 - 78% average decrease in injury accidents
- 90% average decrease in fatalities or incapacitating injuries
- 30 - 40% average decrease in pedestrian accidents (depending on the roundabout location and existing pedestrian volumes)
- As much as a 75% reduction in delay where roundabouts replaced traffic signals

[4] IIHS, Status Report, 5/13/2000

[5] ITE Journal, September 2002

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The FHWA information guide on roundabouts states that accident frequency and severity is less for a roundabout than a traffic signal. These study results replicate the results of numerous other studies conducted on roundabouts in Europe and Australia and provide quantitative evidence that the selection of a roundabout over the more conventional intersection geometrics and traffic control can have significantly positive traffic safety implications. Studies completed in England have revealed that the total number of pedestrian accidents with vehicles at roundabouts is lower than that of other intersection types by 33 to 54 percent. Norway has also indicated in several studies over the years that roundabouts have provided a 73 percent reduction in pedestrian crashes at intersections converted to roundabouts.

The unaware person typically asks why roundabouts are safer than traffic signals. The following bulleted list of items provides these answers as well as further discussions and illustrations below:

- Roundabouts have fewer conflict points for vehicles, pedestrians, and cyclists. The potential for many hazardous conflicts, such as right-angle accidents and conflicting left turn head-on crashes, are eliminated with modern roundabouts.
- Speeds at roundabouts are significantly lower (average of 22 mph) than other types of crossings, which allows drivers more time to react to potential conflicts.
- There is a lower speed ***differential*** between the users of roundabouts (e.g. vehicles to pedestrians to cyclists) since the road users travel at similar speeds through the roundabout.
- Lower speeds and speed differentials between users of roundabouts significantly reduces the accident severity if an accident occurs.
- Pedestrian crossings at roundabouts are much shorter in distance and entails interruption in only one direction of the traffic stream at a time. Since conflicting vehicles arrive in one direction only to the pedestrians, the pedestrians need only to check to their left for conflicting vehicles. In addition, the speed of the vehicles in the roundabout at entry and exit are reduced with a proper roundabout design.

The following are some facts on traffic signals, red light running, and roundabouts:

1. In 2002, more than 1.8 million intersection crashes occurred throughout the nation. Of those crashes, about 219,000 are due to red light running; resulting in about 1,000 deaths and 181,000 injuries. (*Insurance Institute for Highway Safety, IIHS, and Federal Highway Administration, FHWA, 2003*)

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2. A study conducted by the Insurance Institute for Highway Safety (IIHS) in 2003 found that at a busy intersection in Virginia, a motorist ran a red light every 20 minutes. During peak commuting times red light running was more frequent.

3. Researchers at the IIHS studied police reports of crashes on public roads in four urban areas. Of thirteen crash types identified, violating traffic control devices accounted for 22 percent of all crashes. Of those, 24 percent were attributed to red-light-running.

4. According to a survey conducted by the U.S. Department of Transportation and the American Trauma Society, two out of three Americans see someone running a red light at least a few times a week and, at most, once a day. (1998)

5. One in three Americans knows someone who has been injured or killed in a red light running crash. (FHWA, 2002)

6. Research from the IIHS illustrates far fewer crashes occur at intersections with roundabouts than at intersections with signals or stop signs. Modern roundabouts are substantially safer than intersections controlled by stop signs, traffic signals or traffic circles.

7. Compared to the former traffic circle or rotary, the majority of modern roundabouts have excellent safety performance mostly due to their small diameter, slower circulating speeds, flared approach, deflection, and yield control entrances. Studies from around the world have shown modern roundabouts typically reduce crashes by 40 to 60 percent compared to stop signs and traffic signals. They also typically reduce injury crashes by 35 to 80 percent and almost completely eliminate fatal and incapacitating crashes.

Roundabouts are self-regulating traffic control devices that automatically control driver speeds. Lower speeds at roundabouts, compared to traffic signals, directly relates to intersection safety. To elaborate on this concept, lower speeds on a roadway or at an intersection equate to shorter braking distances. The following bar chart (**Figure 26**) demonstrates a comparison of traffic signals to roundabouts based on braking distance and driver perception/reaction distances for braking.

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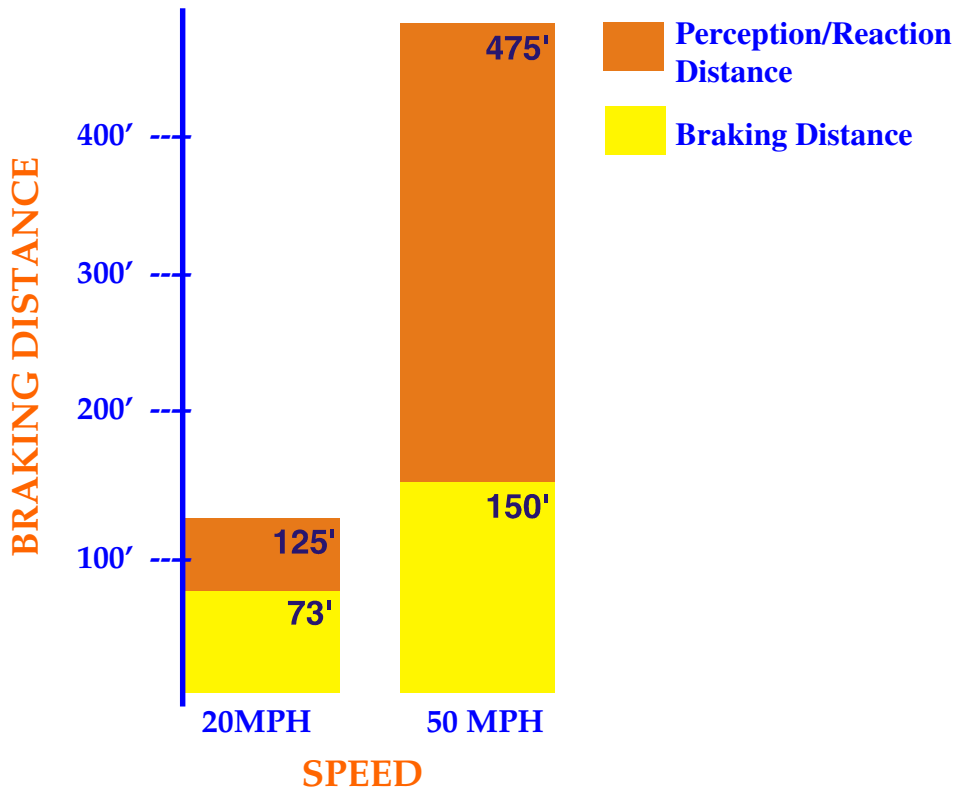

Figure 26: Braking Distances & Speeds

As mentioned above, since the speeds at roundabouts are significantly lower with a lower speed differential between the users of roundabouts, this significantly reduces the accident severity of collisions at roundabouts. Figure 27 illustrates the accident severity of collisions at roundabouts versus traffic signals based upon vehicle speeds. As shown in the chart below, roundabouts will have a lower accident severity rate than that of traffic signals. Hence, there will be less injuries and fatalities at roundabouts than signals as well as other types of intersections. The statistics discussed above or the "before" and "after" field studies verify this reality.

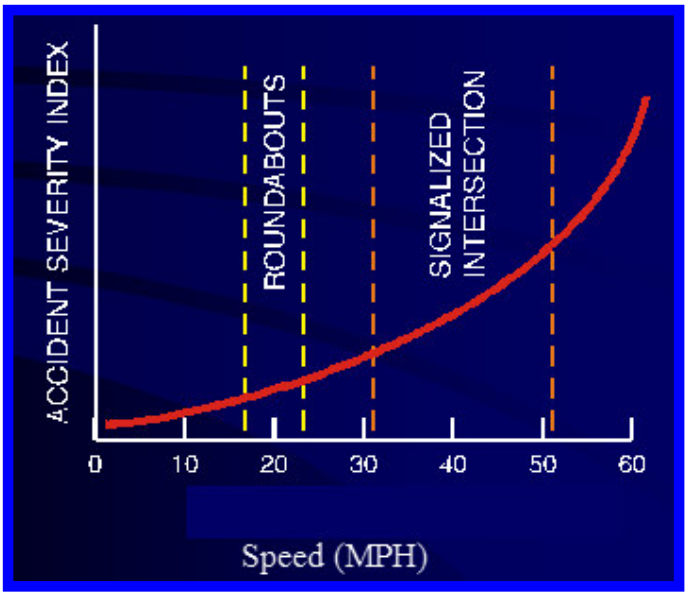

Figure 27: Accident Severity & Speeds

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Another reason why roundabouts are safer types of intersections are the reduced number of conflict points at a roundabout versus a signal. The following illustrations (**Figures 28** and **29**) show the number of vehicle-to-vehicle (black dots) and vehicle-to-pedestrian (white dots) conflicts at a roundabout and signal.

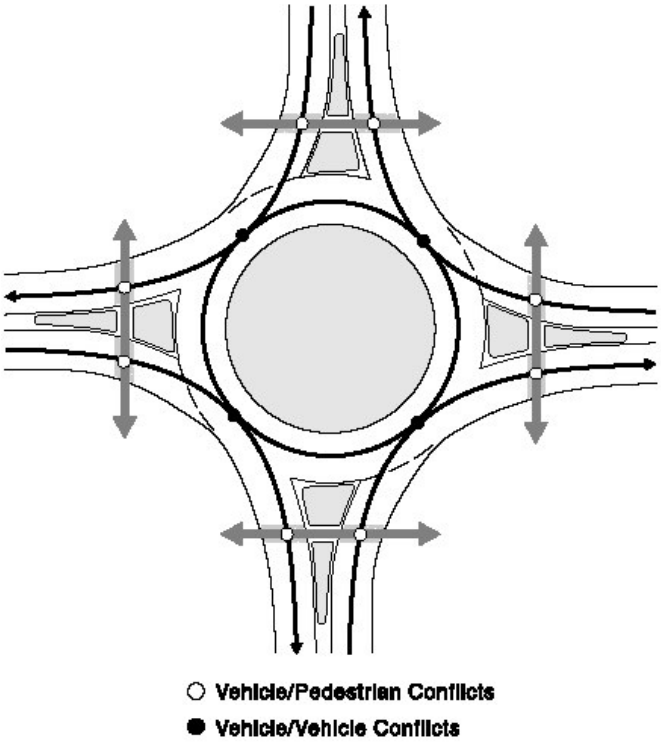

Figure 28: Roundabout Points of Conflict

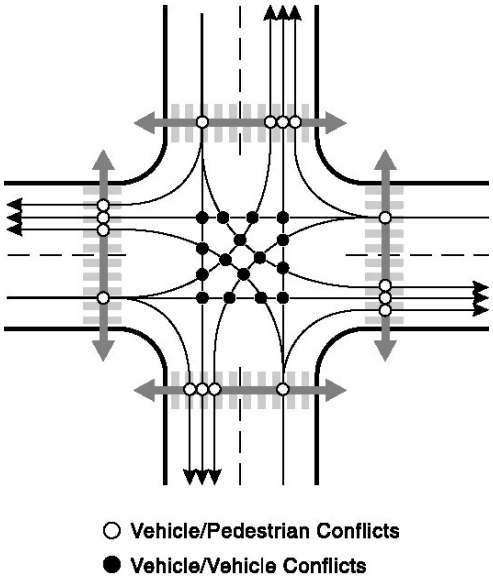

Figure 29: Signal Points of Conflict

As shown above, there are more vehicular and pedestrian points of conflicts at a signalized intersection than a roundabout. This solves the question in a very basic way of why roundabouts are safer than a signalized intersection.

In addition to a significant reduction in traffic accidents, roundabout installation can generate reductions in delays and associated air emissions, improve intersection capacity and pedestrian travel, reduce intersection improvement costs and associated operation and maintenance costs, and can be a key element in improving the visual quality of roadway corridors and town centers.

In general, if roundabouts are designed by a qualified roundabout specialist, the modern roundabout will function as a self-regulating traffic control device that offers numerous capacity, safety, aesthetic, and often cost benefits to a community and/or public jurisdiction.

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ACCIDENT ANALYSES

The accidents at the two intersections were briefly analyzed to determine if the types of accidents that were occurring at the intersections would be mitigated with the intersection improvements. In a two-year period between 2002 and 2004, thirteen collisions were reported at the Pacific Avenue/Beach Street intersection (i.e. 6.5/yr.) and nine were reported at the Pacific Avenue/Center Street intersection (i.e. 4.5/yr.). A breakdown of the collisions is summarized below in **Table 7**.

Table 7: Accident History

Date	Type of Collision	w/ Bike or Ped.	Day or Night	Injury or Non-Injury
Pacific Avenue/Beach Street				
2/13/2002	Other	Bike	Night	Non-Injury
9/12/2002	Broadside	No	Day	Unknown
9/14/2002	Other	No	Day	Non-Injury
9/24/2002	Other	Pedestrian	Day	Non-Injury
12/2/2002	Other	No	Day	Unknown
12/21/2002	Other	Pedestrian	Night	Injury
1/12/2003	Other	Unknown	Day	Unknown
2/16/2003	Angle	No	Day	Non-Injury
3/2/2003	Rear-End	No	Day	Unknown
6/18/2003	Sideswipe	Unknown	Day	Unknown
8/8/2003	Other	No	Day	Unknown
4/27/2004	Broadside	No	Day	Non-Injury
12/18/2004	Broadside	No	Day	Injury
Pacific Avenue/Center Street				
1/15/2002	Other	Bike	Night	Injury
4/5/2002	Other	Pedestrian	Day	Injury
5/27/2002	Rear-End	No	Day	Injury
7/17/2002	Rear-End	No	Day	Injury
8/7/2003	Rear-End	No	Day	Unknown
9/27/2003	Rear-End	No	Day	Non-Injury
2/11/2004	Sideswipe	No	Day	Unknown
9/20/2004	Other	No	Day	Unknown
5/21/2005	Other	No	Night	Injury

With the installation of a higher level of traffic control device (signals or roundabout), the type and frequency of collisions are expected to change.

According to data published in National Cooperative Highway Research Program (NCHRP) Report 491 – Crash Experience Warrant for Traffic Signals, right angle (i.e. broadside) crashes would be expected to decrease, whereas rear-end collision may increase with traffic signals installed. Other collision types would also decrease slightly with signals. Based on one source of data in the report, overall collisions would be expected to be reduced by approximately 29%

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which would result in 4.6 collisions per year at Pacific Avenue/Beach Street and 3.2 per year at the Pacific Avenue/Center Street intersection.

Currently both intersections are all-way stop controlled. Hence, vehicle speeds are generally slow. If changed to traffic signals, vehicles traveling on the green light can pass though the intersections at higher speed which is expected to increase the collision severity; however, the degree of change is unknown.

In addition to the general reduction in expected collisions, it was noted that 60% of the bike/pedestrian collisions with motorized vehicles occurred at night compared with 6% for vehicle only collisions. This suggests that drivers may be having difficulty seeing pedestrians and cyclists during nighttime hours. Therefore, it is assumed that with the traffic signal alternative, additional intersection safety lighting would be added to the intersections.

With the installation of modern roundabouts at the intersections it is anticipated that 75 to 80% of the injury accidents would be eliminated, and 30 to 70% of pedestrian accidents would be eliminated. This results in roughly 1.3 collisions per year at Pacific Avenue/Beach Street and 0.9 per year at the Pacific Avenue/Center Street intersection.

EMERGENCY VEHICLES

With respect to operational safety, the traffic signal would require pre-emption for emergency vehicles to pass through the intersection. Once the intersection clears of traffic (usually taking a few moments to turn from green to yellow to red followed by vehicles needing to exit the intersection) and assuming no traffic run the red light or stop in the middle of the intersection, the emergency vehicle could pass through the intersection with a relatively high degree of safety with respect to traffic signals. The emergency vehicle could then make any turn necessary to proceed to an incident.

A notable concern is when an emergency vehicle decides to create the new green phase (stopping other traffic). It is common, with the sudden phase change tripped seconds after a green light has been shown to a stopped phase of traffic waiting at the signal, that confusion to pedestrians and drivers occurs with their sudden termination of phase or drivers being unaware that their phase has turned red in such a short time. However, the sirens, lights, and horn of the emergency vehicle usually are sufficient warning regardless of the sudden change from green to red at the signal to stop drivers from running a red light with the short time of a green light. It is also required by law to stop at all red lights (despite the statistics showing 24% of all accidents at signals are red light runners).

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There are operational concerns if traffic stops in the intersection as emergency vehicles enter. Stopped traffic in the intersection may hinder the emergency vehicle's ability to maneuver through the intersection since the traffic may not be aware of which direction the emergency vehicle needs to travel.

However, the roundabout would not require any special phasing, pedestrian or design modifications, or special traffic control features at the intersection. All traffic and drivers are already anticipating to yield to circulating traffic in the roundabout and thus are anticipating a reduced speed if not a brief stop condition at the intersection. Similar to the signal, the emergency vehicle would sound its sirens, horn, and lights while approaching the roundabout. By law, all approaching traffic at the roundabout must yield to vehicles in the roundabout (circulating). However, unlike the signal, the geometry of the roundabout (right curb faces, splitter islands, and the central island) enforces and forces the speed for all traffic to slow down at the intersection to a maximum of 25 miles per hour at entry and 15 miles per hour circulating the roundabout. This significantly decreases the likelihood of accidents with vehicles and the entering emergency vehicle. The emergency vehicle would traverse the roundabout or truck apron in the same direction as traffic (counterclockwise).

The only conflicting movements with the emergency vehicle after entering the roundabout are any remaining traffic in the circulating roadway, with a driver choice to either exit the roundabout or pull over. All approaching traffic to the roundabout would be required to yield to the emergency vehicle now in the circulating roadway. The emergency vehicle would enter the roundabout with the same movements as normal traffic using the intersection and proceed around the roundabout (counterclockwise) to whichever exit or direction the emergency requires.

The roundabout would operate safer than the signal with respect to the emergency vehicle making the same anticipated movements as a vehicle using the intersection. This reduces driver and pedestrian confusion and allows traffic to proceed around the roundabout as normal. The addition of sirens and lights increases traffic safety with a stopped/yielded condition of other traffic at the yield line and nearly guarantees emptying of the roundabout.

In the event of traffic stopping within the circulating roadway of the roundabout (uncommon), the emergency vehicle may also use the truck apron of the roundabout to bypass any stopped traffic or incidents. This can be shown in video clips taken by RTE. Vehicular traffic is relatively undisturbed with little driver frustration once the emergency vehicle passes and can continue to operate

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normally. The emergency vehicle has access anywhere to any direction, including u-turn options within the roundabout.

Other Site Comparison: Since emergency vehicle operations are a common question for communities with a new roundabout, additional field data is provided below. For data comparison, RTE has provided an example of a fire station in a similar city setting located approximately 180 feet from a modern roundabout. Initially, the City and fire personnel were concerned of installing a traffic signal at the intersection of Hamilton Drive / Wilson Street due to the same issues identified above (the short distance between the fire station driveway and the signalized intersection causing potential problems with stopped vehicles and long queues blocking the driveway access to the fire station). However, the City of Hamilton (Village of Ancaster) decided to install a modern roundabout to eliminate and reduce the potential safety and access issues with the fire station. The images below (**Figures 30** and **31**) show illustrations of the intersection before and after construction. The after construction photo was taken from the roof of the fire station.

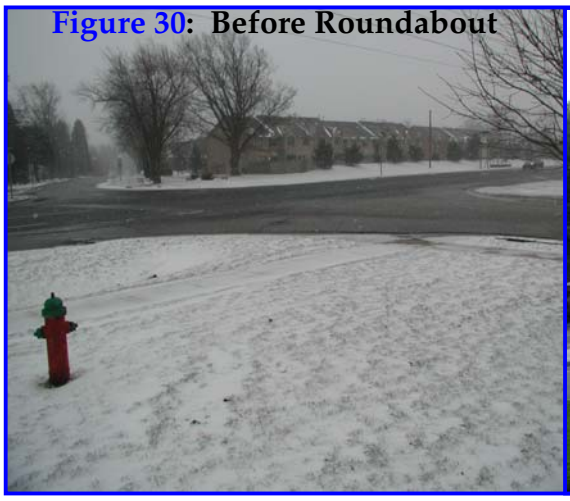

Figure 30: Before Roundabout

Figure 31: After Roundabout

The roundabout opened in 2002 with extremely positive comments from the city and fire department on the operations of the intersection as a roundabout. The fire department has commended the easy emergency vehicle operations, lack of delay at the intersection, and fast emergency response times. In addition, the fire station has installed a web-cam on the Internet of live operations of the roundabout on the radio tower pole on top of the fire station. An aerial photo of the site is shown in **Figure 32**.

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Figure 32: Fire Station Near Roundabout

The roundabout allows the emergency vehicles to proceed in any direction, as needed. Please contact Scott Ritchie with RTE to obtain more information on emergency vehicles at roundabouts.

Speed studies[6] were also performed along the Wilson Street Corridor (fire station's main access roadway) as well in the roundabout itself to test the operational performance of the effects of the roundabout before and after installation as well as to test the predicted design speeds of the roundabout before construction to the actual speeds of traffic after construction.

As a result of these studies it was shown that the 85th percentile speeds as well as the highest speeds through the roundabout and along the Wilson Street corridor (see **Figures 33** and **34** for the locations of the speed survey points) were all reduced substantially and lower than the fastest path design speeds predicted. Please refer to Scott Ritchie's *High Speed Approaches At Roundabouts* published by the Transportation Research Board in 2005 for more detailed results and data.

[6] High Speed Approaches At Roundabouts, Scott Ritchie, P.E., Roundabouts & Traffic Engineering, 2005, Published By the Transportation Research Board and Presented At The International Roundabout Conference in Vail, 2005.

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Figure 33: Speed Study Locations at Roundabout

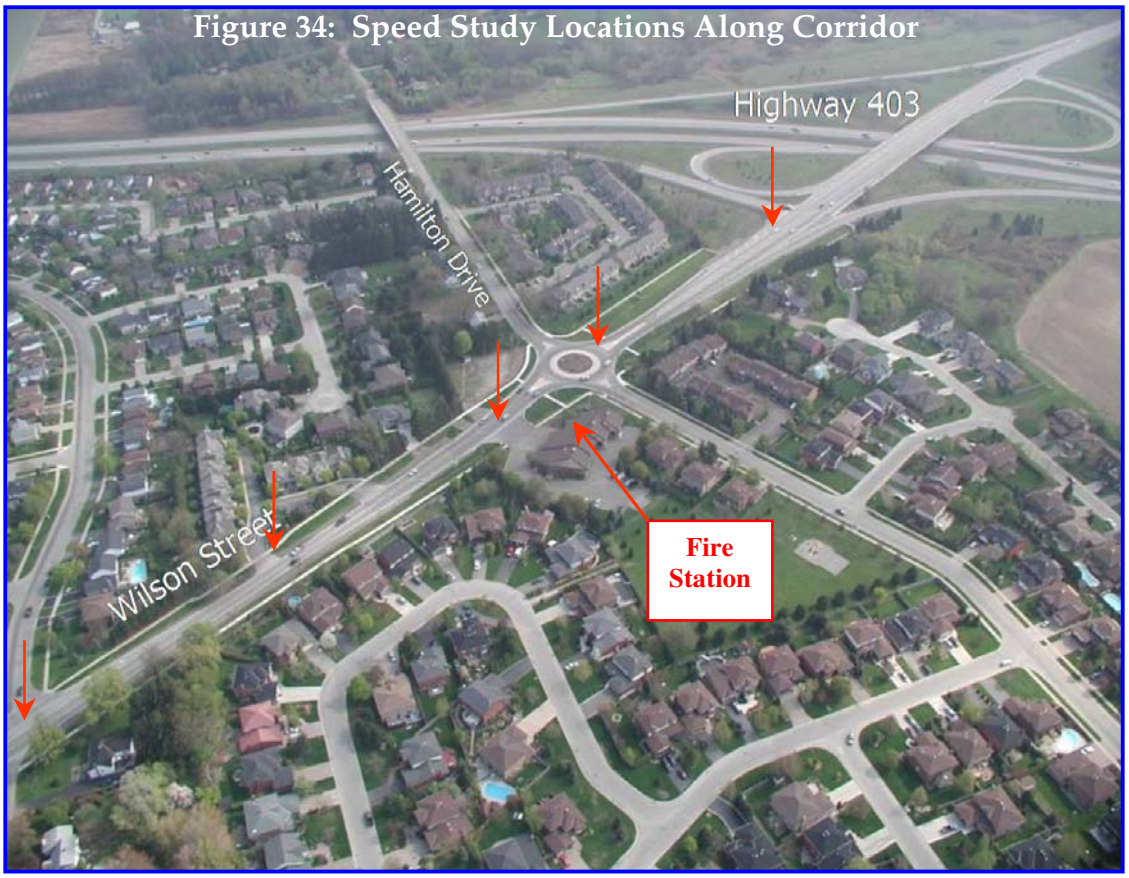

Figure 34: Speed Study Locations Along Corridor

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VII. COST COMPARISONS

This chapter compares the proposed roundabout alternatives to the proposed signal alternatives with respect to the cost related issues for each intersection. These costs are not detailed engineering cost estimates, rather initial costs assuming typical construction.

CONSTRUCTION COST ESTIMATES OF ALTERNATIVES

Both the signal and roundabout alternative cost estimates are based on the capacity calculations derived above, the conceptual intersection exhibits of each intersection, and current unit costs. KHA provided the cost estimates for both intersection alternatives. **Table 8** summarizes the total construction costs for both alternatives for each intersection. As shown, the project cost for the roundabout alternative at the Depot Park intersection is estimated at $220,000 whereas the signal cost is $170,000. Also shown, the project cost for the roundabout alternative at the Wharf intersection is estimated at $330,000 whereas the signal cost is $260,000.

Table 8: Cost Performance

Depot Park Intersection & Wharf Intersection

Comparison of Construction Cost Estimates (Conceptual)

INTERSECTION	SIGNAL	ROUNDABOUT
Pacific Avenue / Center Street (Depot Park)	$170,000	$220,000
Pacific Avenue / Beach Street (Wharf)	$260,000	$330,000

Note: These estimates were prepared for the sole purpose of this report and represent conceptual costs only. Do not use these costs for project financing or budgeting.

Source: KHA — SantaCruz Feasibility Tables.xls

The signal alternatives would provide a slight cost savings of $120,000. It should be noted that additional costs might occur for both estimates.

MAINTENANCE COSTS

Traffic signals require ongoing maintenance costs for the signal poles, controller cabinet, loop or video detectors, signal heads, or the like. These costs typically add up to an annual average of $5,500 per year per signal. Roundabouts typically do not incur such maintenance costs unless annual flowers or foliage need replacement or upkeep in the central island or outside the roundabout. The

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required obstructions in the roundabout designs can usually be accommodated with perennial foliage, statues, native plants/trees, or rocks that requires little to no maintenance. Other costs, such as curb/pavement repair and lighting maintenance are similar between the two alternatives.

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VIII. CONCLUSIONS & RECOMMENDATIONS

CONCLUSIONS

The preceding three sections of this report compared the capacity, safety, and cost of each intersection with either a roundabout or signal alternative. The following summarizes the comparative analysis sections of this report.

- The roundabouts provide superior capacity at both intersections over the signal alternatives with respect to the overall operations, level of service, delay, and queue lengths for the intersection.

- The "before" and "after" safety statistics conducted in the United States and worldwide provide substantiating evidence of the superior safety performance of roundabouts versus signals and other intersection types for both vehicles and pedestrians.

- The construction cost estimates of the roundabout alternatives illustrate a slightly higher cost versus the traffic signals. The combined additional cost for the roundabouts versus the signals totaled $120,000. However, the modern roundabouts may also require less annual maintenance costs.

In addition to the capacity, safety, and costs of each alternative, the importance of a proper functioning traffic control device for the existing site constraints (pedestrians, Wharf pier, business accesses, etc.) are critical to the safety operations and public acceptance of either traffic control device. As a result of the analyses herein as well as the conceptual designs, the proposed modern roundabouts would function superior to the traffic signals.

Therefore, it can be unanimously determined by all the contributing factors within this feasibility study (except the slight additional cost for the roundabouts) that the modern roundabout is the recommended alternative for these two intersections.

Although this report provides comparisons and information primarily focusing on the capacity analyses, safety analyses, and cost analyses of each alternative, a number of other comparisons could be made between the two alternatives for each intersection. However, this report does not provide additional comparisons or explanation on these additional issues such as aesthetics, driver behavioral characteristics, benefit-to-cost ratios, predicted accident safety costs, predicted societal accident costs, life-cycle maintenance costs, and delay costs to road users.

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However, a simplified comparison matrix is provided below that accounts for decision-making factors between the signal and roundabout alternative.

Comparison Matrix: The Comparison Matrix is designed for the internal use for the project development team. It assists in a comparative analysis that measures and weighs a various number of major design decision options. RTE has compiled the results of a variety of comparison factors into a matrix that includes key decision measures, assigned percentages, and weighted values based on the capacity, safety, special design vehicle (fire station trucks), and cost results completed in this feasibility study. The comparison matrix merely provides a tool for the design team to aid in the selection of a preferred alternative. **Table 6** provides the items considered and the summary results of the analyses.

Table 9: Decision Matrix

Comparison of Key Elements of Alternatives

Intersection / Alternative	Delay / LOS	Vehicle Safety	Const. Cost	Ped Safety	O & M Costs	Aes-thetics	Total Score: Higher is Better
Weight	30%	30%	10%	20%	5%	5%	
DEPOT PARK							
Signalized Intersection	4	3	5	2	3	2	**3.25**
Modern Roundabout	5	5	4	5	4	5	**4.85**
WHARF							
Signalized Intersection	4	3	5	1	2	2	**3.00**
Modern Roundabout	5	5	3	5	4	5	**4.75**

Legend: 0=Very Poor, 1=Poor, 2=Below Avg, 3=Average, 4=Above Avg, 5=Excellent

Source: RTE SantaCruz Feasibility Tables.xls

As shown above, the modern roundabouts have significantly higher ratings than the signal alternatives. It is understood that each jurisdiction has slightly differing weights and factors included for each design project, however, the intent of the comparison matrix is to provide a general insight and basic rational behind the conclusions of this report. The results of each numeric factor are based on the results of this report, available averages, nationwide statistics, and RTE's professional judgment for this particular project. The provided weights of each key element or factor is a derivative and average of public decision makers with similar roundabout versus signal projects RTE has dealt with throughout

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North America. As shown in the comparison matrix, the total score for the roundabout alternative is much better than the score of the signal alternative.

Other Comments: As noted in previous chapters, there is a concern of pedestrians and cyclists actually using the signals' pedestrian facilities in these beach locations. It is anticipated that pedestrians will ignore the "Walk" and "Don't Walk" pedestrian crossings at the signals, especially at the Wharf intersection. This creates a safety concern and operational concerns with the traffic and pedestrians with signals installed. The modern roundabouts have safe, unrestricted pedestrian movements at any time, which would not create a concern of pedestrians having to wait for "Walk" symbol or sign.

There is also a concern with the queues that would stack with signals on the northbound approach of the Wharf intersection. Even though the light may turn green for the vehicles at the stop bar to proceed, this would allow only two cars to proceed per lane since the Wharf pier requires paid parking at the pier toll booths. Hence, the cars stacked in the queue would need to stop and pay even though a green light is shown ahead. With nearly a two-minute cycle length of the signal, this would negatively impact the departure rate of vehicles using the Wharf pier, and consequentally may reduce pier use by vehicles. The roundabout at the Wharf pier would allow vehicles to proceed after paying at the tollbooth and proceed into the intersection seconds later for a more continuous traffic flow.

The roadway network between intersections is also a concern with signals unless the signals can be connected or timed together. Since the signals do not operate the most efficiently on the same cycle lengths, additional delay may occur at the intersections' side streets. This is of particular concern at the intersection of Pacific Avenue/Second Street. Currently, there is a three-way stop at the intersection to allow vehicles/drivers to enter the congested Pacific Avenue. As a result, drivers can re-circulate into the Beach Area if they missed the parking areas or Wharf as they proceeded down Beach Street (common occurrence). With traffic signals installed, the Pacific Avenue/Second Street intersection may be blocked during peak periods.

With signals installed at the Depot Park and Wharf intersections, this creates a "platooning effect" or large groups of traffic that may increase the amount of gaps (space in between vehicles for side street traffic to enter) at the Pacific Avenue/Second Street intersection to allow Second Street traffic to enter the intersection. However, the gaps are not expected to be sufficient to completely eliminate the need for the stop signs, particularly in the future. This may also be true with the continuous traffic flow of roundabouts, if roundabouts are installed. Therefore, the city may wish to observe traffic operations after the

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installation of either traffic control device and make the final determination whether the stops on Pacific Avenue/Second Street can be removed. The city may also consider the installation of a roundabout at the Pacific Avenue/Second Street intersection to allow Second Street traffic to enter regardless of the traffic control devices chosen for the Depot Park and Wharf intersections.

It is also worth noting that the modern roundabouts would operate similar to the existing all-way stop control. Whereas the traffic signals would require stopped phases of traffic and the signal at the Wharf would require new railroad crossing gates. These stopped phases and new railroad crossing arms are often frustrating transitions for the public. Railroad gate control should not be required due to the slow speeds of traffic, similar to existing conditions.

However, a first roundabout in a community also requires a "learning period" for the public. This typically lasts 90 days. Places like Vail, Colorado or Truckee, California with a high amount of tourism and visitors with roundabouts in operation since 1993 and 1997 have not experienced any notable problems or accidents with their roundabouts. In fact, the first Truckee roundabout has not had a single injury accident after final construction in over 12 years.

Emergency vehicle response times are also worth noting in the conclusions. Discussions with fire department and police department chiefs in jurisdictions throughout the nation where RTE has roundabouts constructed or where modern roundabouts have replaced traffic signals or stop control have reported either a decrease in emergency response times or no reported problems with roundabouts implemented. RTE has observed and videotaped the traffic behavior of emergency vehicles in route to an incident where little to no hindrances to the emergency vehicle were experienced. In general, traffic moves to the curb near or within the roundabout or exits the roundabout before pulling over. Emergency officials state that drivers infrequently pull over in a manner that does not permit the emergency vehicle to proceed through the intersection. In these infrequent cases where vehicles block the circulating roadway, the emergency vehicle utilizes the truck apron or the adjacent exit to bypass traffic. This ease of emergency response and reduced response times is due to the continuous traffic flows, wider entry lanes at roundabouts, and wide circulating lanes for large trucks to maneuver in the roundabout, which provides enough room for an emergency vehicle to pass by passenger vehicles.

Air emissions and construction traffic impacts are additional topics that could be discussed at great length where roundabouts provide positive results over traffic signals. In addition, the aesthetic benefit of roundabouts is understated in most instances. As a civic feature, roundabouts provide a gateway to a town entry or

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city focal point. The local environment at the intersections could be significantly improved with proper landscaping at and around roundabouts.

Summary Conclusion: As a final conclusion, **Table 10** provides specific items from the above analyses for comparison between the proposed roundabouts and traffic signals. As shown in the contents of this feasibility study, the modern roundabout alternative is superior to the traffic signal at both intersection locations.

Table 10: Summary Comparison
Modern Roundabouts versus Traffic Signals (2025)

MAJOR FACTOR	SIGNAL	ROUNDABOUT
DEPOT PARK INTERSECTION		
Capacity (Level of Service - PM/Sat)	LOS C / LOS C	LOS A / LOS A
Capacity (Delay in seconds - PM/Sat)	20.0 / 34.4	5.5 / 7.7
Lane Geometry	Existing Lane Configuration	Reduced Lane Configuration
Operations	Stopped Phases	Free Flow / Yield
Stopped Queue Lengths (Feet)	Up to 775'	Up to 50'
Air Emissions	Stopped Idling Pollution / Stop/Go Pollution	Reduced Air Emissions With Reduced Delays
Safety (Injury Accidents)	24% of All Signal Injuries Are Red Light Runners	78% Reduction in Injuries / 90% Fatalities
Safety (Speeds)	50 MPH Average	20 MPH AVG
Safety (Pedestrian / Cycles)	Nationwide Statistics Find High Accidents	Substantial Reduction in Accident Rates
Construction Costs	$170,000	$220,000
Maintenance	Electrical	Landscaping
WHARF INTERSECTION		
Capacity (Level of Service - AM / PM)	LOS C / LOS D	LOS A / LOS A
Capacity (Delay in seconds - AM/PM)	25.3 / 44.0	3.9 / 7.8
Lane Geometry	Existing Lane Configuration	Reduced Lane Configuration
Operations	Stopped Phases	Free Flow / Yield
Stopped Queue Lengths (Feet)	Up to 875'	Up to 75'
Noise Pollution	Stop/Go Noise Pollution	Reduced Pollution with Free Flow Traffic
Safety (Injury Accidents)	Higher Speeds With Signals = Injuries	Similar Ped/Veh/Cycle Speeds = No Injuries
Safety (Speeds)	475' Braking Distance	125' Braking Distance
Safety (Pedestrian / Cycles)	Peds / Cyclists May Not Use Signal Properly	No Waiting / Safer & Slow Vehicular Speeds
Construction Costs	$260,000	$330,000
Emergency Vehicle Operations	Detectors Recommended	Reduced Response Times Proven

Source: RTE — SantaCruz Feasibility Tables.xls

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RECOMMENDATIONS

The key points this study established were that the roundabout alternative provides the most amounts of capacity and operational safety for this project location. This was demonstrated in the capacity analyses, the safety discussions, the proposed roundabout conceptual designs, and the comparative analyses between the signals and the roundabouts. In addition, the roundabouts function superior to the signal for pedestrians, cyclists, and emergency vehicles. As determined in the conclusions, the roundabout alternative is recommended for both of these intersection locations.

It is recommended to proceed with the final geometric layout of the roundabout, including geometric design modifications and details still to complete, all of the non-geometric essentials (such as signing, lighting, striping, and landscaping), and the remaining civil components of the design plans including grading, utilities, drainage, and survey information. It is recommended that the geometric layouts of the roundabout designs remain relatively unchanged with the exception of final modifications to ensure right-of-way impacts, turn lanes into nearby access points, and additional information requested for the exact points to tie into the existing roadway.

As stated in the introduction of the report, the modern roundabout, coupled with good design practices and additional geometric and non-geometric design measures such as proper signing and landscaping, are the traffic control devices of choice for intersections in most countries. The self-regulating traffic control device creates an environment controlled by roadway and intersection geometric layouts with roadway widths, curves, medians, lighting, signing, striping, and landscaping to regulate traffic speeds.

As shown in the conceptual roundabout designs (**Figure 23** and **24**), the entries are visible to drivers from a safe stopping distance, safe design speeds promote yielding at entry with slow entry and circulating speeds, the splitter islands have been designed properly, ADA and bike lane appurtenances are present, as well as many other design features.

***Additional Implementation Recommendations*:** The following additional items not shown in the conceptual roundabout designs are also recommended:

- A four-inch rolled curb is recommended for the roundabout's truck apron.
- Provide at least a 3-5% slope on the truck apron sloping downward towards the circulating roadway with textured or stamped concrete. The concrete should be constructed with color (preferably red/brown or

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black/white) with large chevrons in the concrete (as separate sections in the concrete) to discourage pedestrian usage and driver awareness.

- Provide highly visible and obstructing landscaping in the central islands according to sight distance requirements for each entry and circulating points within the roundabouts.
- Landscaping on the medians and splitter islands approximately 75 feet prior to the yield line to create a tunnel effect for approaching vehicles is recommended. No landscaping over 24 inches should be provided between the yield line and 75 prior to the yield line.
- Provide post mounted maptype signs as shown in the *Roundabout Signing Guide, A Recommended Practice, 1st Edition*,[7] for driver comprehension of destination and repeated display and understanding of a roundabout ahead.
- Provide an internally illuminated bollard (MUTCD compliant[8]) on the splitter islands of all approaches to assist in nighttime visibility of the roadway geometry ahead. RTE has an MUTCD compliant sample of the product available for illustration and discussion.
- Provide roadway, approach, and exit lighting at the roundabouts at least 200 feet prior to the yield line for all approaches. RTE can identify the specific locations for proper positive contrast lighting at the roundabout.
- Provide detached sidewalks with landscaping between the back of curb and face of walk to provide a tunnel effect or constrained environment for the driver to slow down prior to entry. A four to five-foot detached sidewalk with an eight to ten-foot multiuse path is already shown on the conceptual roundabout plans depending on the location (**Figures 23** and **24**).
- If possible, the use of internally illuminated exit signs is a highly visible method of displaying an intersection with a roundabout. RTE has illustrations of the internally illuminated signs used in Vail, Colorado.
- General conformance to the recommendations found in the *Roundabout Signing Guide, A Recommended Practice* such as the arrow shaped exit signs are recommended for all approaches at both roundabouts.[8]
- Conformance to the DRAFT 2008 MUTCD manual is recommended. Scott Ritchie is a member of the board for the new MUTCD 2008 manual on signing and striping at roundabouts and can provide recommendations on the latest federal and ADA recommendations.
- Provide highly visible crosswalks with the use of recessed thermoplastic in an international style stripe design (a.k.a. "ladder" stripes) or a stamped and colored concrete for high visibility.

[7] Roundabout Signing Guide, A Recommended Practice, 1st Edition, Scott Ritchie, P.E., Roundabouts & Traffic Engineering and Phil Weber, P.Eng. Roundabouts Canada, 2005, Published by the Transportation Research Board 2005

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- Provide proper advanced and intersection signing and markings to advise of the appropriate speed and lane for approaching drivers.

Advance signage combined with a visible driving situation with appropriate landscaping and a well-illuminated intersection all contribute to the good safety performance currently being observed at roundabout sites. The consequences of an inconspicuous central island and/or splitter islands is mainly loss of control crashes as motorists unfamiliar with the roundabout are not given sufficient visual information to elicit a change in speed and path.

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APPENDIX

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Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Computation Report
2000 HCM Operations Method (Future Volume Alternative)

Intersection #2 DEPOT INTX

Cycle (sec): 60 Critical Vol./Cap. (X): 0.840
Loss Time (sec): 12 (Y+R = 4 sec) Average Delay (sec/veh): 20.0
Optimal Cycle:OPTIMIZED Level Of Service: C

Street Name:	Center						Pacific					
Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L - T - R			L - T - R			L - T - R			L - T - R		
Control:	Permitted			Permitted			Permitted			Permitted		
Rights:	Include			Include			Include			Include		
Min. Green:	0	0	0	0	0	0	0	0	0	0	0	0
Lanes:	0 0	1! 0	0	1 0	0 1	0	0 0	0 0	0	1 0	1 0	1
Volume Module: PM												
Base Vol:	22	153	542	30	113	321	0	0	0	211	458	57
Growth Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Initial Bse:	22	153	542	30	113	321	0	0	0	211	458	57
Added Vol:	0	0	0	0	0	0	0	0	0	0	0	0
In-Process:	0	0	0	0	0	0	0	0	0	0	0	0
Initial Fut:	22	153	542	30	113	321	0	0	0	211	458	57
User Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Volume:	22	153	542	30	113	321	0	0	0	211	458	57
Reduct Vol:	0	0	0	0	0	0	0	0	0	0	0	0
Reduced Vol:	22	153	542	30	113	321	0	0	0	211	458	57
PCE Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
MLF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Final Vol.:	22	153	542	30	113	321	0	0	0	211	458	57
Saturation Flow Module:												
Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.88	0.88	0.87	0.65	0.89	0.84	1.00	1.00	1.00	0.76	1.00	0.76
Lanes:	0.03	0.21	0.76	1.00	0.25	0.75	0.00	0.00	0.00	1.00	1.00	1.00
Final Sat.:	51	355	1259	1227	423	1203	0	0	0	1441	1900	1443
Capacity Analysis Module:												
Vol/Sat:	0.43	0.43	0.43	0.02	0.27	0.27	0.00	0.00	0.00	0.15	0.24	0.04
Crit Moves:		****									****	
Green/Cycle:	0.51	0.51	0.51	0.51	0.51	0.51	0.00	0.00	0.00	0.29	0.29	0.29
Volume/Cap:	0.84	0.84	0.84	0.05	0.52	0.52	0.00	0.00	0.00	0.51	0.84	0.14
Delay/Veh:	19.9	19.9	19.9	7.3	10.3	10.3	0.0	0.0	0.0	18.9	31.2	16.0
User DelAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
AdjDel/Veh:	19.9	19.9	19.9	7.3	10.3	10.3	0.0	0.0	0.0	18.9	31.2	16.0
HCM2kAvg:	15	15	15	0	6	6	0	0	0	4	11	1

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Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound				South Bound				East Bound				West Bound			
Movement:	L	-	T	- R	L	-	T	- R	L	-	T	- R	L	-	T	- R
HCM Ops Adjusted Lane Utilization Module:																
Lanes:	0	0	1!	0 0	1	0	0	1 0	0	0	0	0 0	1	0	1	0 1
Lane Group:	LTR	LTR		LTR	L	RT		RT	xxxx	xxxx		xxxx	L		T	R
#LnsInGrps:	1	1		1	1	1		1	0	0		0	1		1	1
HCM Ops Input Saturation Adj Module:																
Lane Width:	12	12		12	12	12		12	12	12		12	12	12		12
CrosswalkWid	8				8				8				8			
% Hev Veh:	0				0				0				0			
Grade:	0%				0%				0%				0%			
Parking/Hr:	No				No				No				No			
Bus Stp/Hr:	0				0				0				0			
Area Type:	< < < < < < < < < < < < < < < < Other > > > > > > > > > > > > > > > >															
Cnft Ped/Hr:	11				42				62				61			
ExclusiveRT:	Include				Include				Include				Include			
% RT Prtct:	0				0				0				0			
HCM Ops f(lt) Adj Case Module:																
f(lt) Case:	5	5		5	2	xxxx	xxxx	xxxx	xxxx	xxxx		2r	xxxx	xxxx		
HCM Ops Saturation Adj Module:																
Ln Wid Adj:	1.00	1.00		1.00	1.00	1.00		1.00	xxxx	xxxx	xxxxx		1.00	1.00		1.00
Hev Veh Adj:	1.00	1.00		1.00	1.00	1.00		1.00	xxxx	xxxx	xxxxx		1.00	1.00		1.00
Grade Adj:	1.00	1.00		1.00	1.00	1.00		1.00	xxxx	xxxx	xxxxx		1.00	1.00		1.00
Parking Adj:	1.00	1.00		1.00	xxxx	1.00		1.00	xxxx	xxxx	xxxxx		xxxx	xxxx		1.00
Bus Stp Adj:	1.00	1.00		1.00	xxxx	1.00		1.00	xxxx	xxxx	xxxxx		xxxx	xxxx		1.00
Area Adj:	1.00	1.00		1.00	1.00	1.00		1.00	xxxx	xxxx	xxxxx		1.00	1.00		1.00
RT Adj:	0.90	0.90		0.90	xxxx	0.89		0.89	xxxx	xxxx	xxxxx		xxxx	xxxx		0.85
LT Adj:	0.98	0.98		0.98	0.65	xxxx		xxxxx	xxxx	xxxx	xxxxx		0.85	xxxx		xxxxx
PedBike Adj:	1.00	1.00		0.99	1.00	1.00		0.95	1.00	1.00		1.00	0.89	1.00		0.89
HCM Sat Adj:	0.88	0.88		0.87	0.65	0.89		0.84	1.00	1.00		1.00	0.76	1.00		0.76
Usr Sat Adj:	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00
MLF Sat Adj:	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00
Fnl Sat Adj:	0.88	0.88		0.87	0.65	0.89		0.84	1.00	1.00		1.00	0.76	1.00		0.76
Delay Adjustment Factor Module:																
Coordinated:	< < < < < < < < < < < < < < < < No > > > > > > > > > > > > > > > >															
Signal Type:	< < < < < < < < < < < < < < Actuated > > > > > > > > > > > > > >															
DelAdjFctr:	1.00	1.00		1.00	1.00	1.00		1.00	0.00	0.00		0.00	1.00	1.00		1.00

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Permitted Left Turn Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North	South	East	West
Cycle Length, C:	60	60	xxxxxx	xxxxxx
Actual Green Time Per Lane Group, G:	32.77	32.77	xxxxxx	xxxxxx
Effective Green Time Per Lane Group, g:	30.77	30.77	xxxxxx	xxxxxx
Opposing Effective Green Time, go:	30.77	30.77	xxxxxx	xxxxxx
Number Of Opposing Lanes, No:	1	1	xxxxxx	xxxxxx
Number Of Lanes In Lane Group, N:	1	1	xxxxxx	xxxxxx
Adjusted Left-Turn Flow Rate, Vlt:	22	30	xxxxxx	xxxxxx
Proportion of Left Turns in Lane Group, Plt:	0.03	1.00	xxxxxx	xxxxxx
Proportion of Left Turns in Opp Flow, Plto:	xxxxxx	1.00	xxxxxx	xxxxxx
Left Turns Per Cycle, LTC:	0.37	0.50	xxxxxx	xxxxxx
Adjusted Opposing Flow Rate, Vo:	434	717	xxxxxx	xxxxxx
Opposing Flow Per Lane Per Cycle, Volc:	7.23	11.95	xxxxxx	xxxxxx
Opposing Platoon Ratio, Rpo:	1.00	1.00	xxxxxx	xxxxxx
Lost Time Per Phase, tl:	6.00	6.00	xxxxxx	xxxxxx
Eff grn until arrival of left-turn car, gf:	14.74	13.16	xxxxxx	xxxxxx
Opposing Queue Ratio, qro:	0.49	0.49	xxxxxx	xxxxxx
Eff grn blocked by opposing queue, gq:	3.29	9.26	xxxxxx	xxxxxx
Eff grn while left turns filter thru, gu:	16.03	17.61	xxxxxx	xxxxxx
Max opposing cars arriving during gq-gf, n:	xxxxxx	0.00	xxxxxx	xxxxxx
Proportion of Opposing Thru & RT cars, ptho:	xxxxxx	0.00	xxxxxx	xxxxxx
Left-turn Saturation Factor, fs:	0.60	xxxxxx	xxxxxx	xxxxxx
Proportion of Left Turns in Shared Lane, pl:	0.03	1.00	xxxxxx	xxxxxx
Through-car Equivalents, el1:	2.17	2.59	xxxxxx	xxxxxx
Single Lane Through-car Equivalents, el2:	xxxxxx	1.00	xxxxxx	xxxxxx
Minimum Left Turn Adjustment Factor, fmin:	0.07	0.13	xxxxxx	xxxxxx
Single Lane Left Turn Adjustment Factor, fm:	0.98	0.65	xxxxxx	xxxxxx
Left Turn Adjustment Factor, flt:	0.98	0.65	xxxxxx	xxxxxx

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Ped/Bike Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	T	R	L	T	R	L	T	R	L	T	R
CrsswalkWid:	8.00		8.00	8.00		8.00	0.00		0.00	8.00		8.00
CrsswalkLen:	12.00		36.00	36.00		12.00	0.00		0.00	24.00		36.00
MinPedGrn:	6.82		13.54	13.54		6.82	0.00		0.00	10.34		13.76
PedGrn:	30.77		30.77	30.77		30.77	0.00		0.00	17.23		17.23
PedVol:	42		11	11		42	0		0	62		61
PedFlowRate:	82		21	21		82	0		0	216		212
BikeVol:	10		0	0		10	0		0	0		0
BikeFlwRate:	0		0	0		20	0		0	0		0
PedOcc:	0.041		0.011	0.011		0.041	0.000		0.000	0.108		0.106
BikeOcc:	0.000		0.000	0.000		0.027	0.000		0.000	0.000		0.000
PedAfterOcc:	0.039		0.000	0.009		0.000	0.000		0.000	0.108		0.000
rOcc:	0.021		0.011	0.003		0.067	0.000		0.000	0.108		0.106
TurnVehAdj:	0.979		0.989	0.997		0.933	0.000		0.000	0.892		0.894
Prt:	0.000		0.756	0.000		0.740	0.000		0.000	0.000		1.000
Prta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
Plt:	0.031		0.000	1.000		0.000	0.000		0.000	0.000		0.000
Plta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
PedBikeAdj:	0.999		0.992	0.997		0.950	1.000		1.000	0.892		0.894

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (HCM2000 Queue Method)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L -	T -	R	L -	T -	R	L -	T -	R	L -	T -	R
Green/Cycle:	0.51	0.51	0.51	0.51	0.51	0.51	0.00	0.00	0.00	0.29	0.29	0.29
ArrivalType:		3			3			3			3	
ProgFactor:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Q1:	10.6	10.6	10.4	0.4	5.0	4.7	0.0	0.0	0.0	3.5	7.2	0.7
UpstreamVC:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
UpstreamAdj:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EarlyArrAdj:	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	1.00	1.00	1.00
Q2:	4.2	4.2	4.2	0.1	1.1	1.1	0.0	0.0	0.0	1.0	3.9	0.2
HCM2KQueue:	14.9	14.8	14.6	0.4	6.1	5.8	0.0	0.0	0.0	4.5	11.0	0.9
70th%Factor:	1.17	1.17	1.17	1.20	1.19	1.19	1.20	1.20	1.20	1.19	1.18	1.20
70th%HCM2kQ:	17.4	17.3	17.0	0.5	7.2	6.9	0.0	0.0	0.0	5.3	13.0	1.0
85th%Factor:	1.48	1.48	1.48	1.60	1.55	1.55	1.60	1.60	1.60	1.56	1.51	1.59
85th%HCM2kQ:	22.1	22.0	21.6	0.7	9.4	9.0	0.0	0.0	0.0	7.0	16.6	1.4
90th%Factor:	1.59	1.59	1.59	1.79	1.70	1.70	1.80	1.80	1.80	1.72	1.63	1.78
90th%HCM2kQ:	23.7	23.5	23.2	0.8	10.3	9.9	0.0	0.0	0.0	7.7	18.0	1.5
95th%Factor:	1.76	1.76	1.77	2.09	1.93	1.93	2.10	2.10	2.10	1.97	1.82	2.07
95th%HCM2kQ:	26.2	26.1	25.7	0.9	11.7	11.2	0.0	0.0	0.0	8.8	20.1	1.8
98th%Factor:	2.02	2.02	2.03	2.67	2.33	2.34	2.70	2.70	2.70	2.41	2.13	2.64
98th%HCM2kQ:	30.0	29.9	29.5	1.2	14.1	13.6	0.0	0.0	0.0	10.8	23.5	2.3

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Computation Report
2000 HCM Operations Method (Future Volume Alternative)

Intersection #1 Wharf Intx

Cycle (sec): 70 Critical Vol./Cap. (X): 0.843
Loss Time (sec): 12 (Y+R = 4 sec) Average Delay (sec/veh): 25.3
Optimal Cycle:OPTIMIZED Level Of Service: C

Street Name:	Pacific									Beach		
Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L - T - R			L - T - R			L - T - R			L - T - R		
Control:	Split Phase			Split Phase			Permitted			Permitted		
Rights:	Include			Include			Ovl			Include		
Min. Green:	0	0	0	0	0	0	0	0	0	0	0	0
Lanes:	1 0 1 0 1			1 0 1 1 0			1 0 0 1 0			0 0 0 0 0		
Volume Module: PM												
Base Vol:	36	59	44	124	110	316	711	341	55	0	0	0
Growth Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Initial Bse:	36	59	44	124	110	316	711	341	55	0	0	0
Added Vol:	0	0	0	0	0	0	0	0	0	0	0	0
In-Process:	0	0	0	0	0	0	0	0	0	0	0	0
Initial Fut:	36	59	44	124	110	316	711	341	55	0	0	0
User Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Volume:	36	59	44	124	110	316	711	341	55	0	0	0
Reduct Vol:	0	0	0	0	0	0	0	0	0	0	0	0
Reduced Vol:	36	59	44	124	110	316	711	341	55	0	0	0
PCE Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
MLF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Final Vol.:	36	59	44	124	110	316	711	341	55	0	0	0
Saturation Flow Module:												
Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.95	1.00	0.85	0.95	0.84	0.84	0.79	0.98	0.96	1.00	1.00	1.00
Lanes:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.86	0.14	0.00	0.00	0.00
Final Sat.:	1805	1900	1615	1805	1605	1605	1510	1597	258	0	0	0
Capacity Analysis Module:												
Vol/Sat:	0.02	0.03	0.03	0.07	0.07	0.20	0.47	0.21	0.21	0.00	0.00	0.00
Crit Moves:		****				****	****					
Green/Cycle:	0.04	0.04	0.04	0.23	0.23	0.23	0.56	0.56	0.60	0.00	0.00	0.00
Volume/Cap:	0.54	0.84	0.74	0.29	0.29	0.84	0.84	0.38	0.36	0.00	0.00	0.00
Delay/Veh:	41.9	90.8	72.0	22.5	22.2	37.9	20.7	8.9	7.5	0.0	0.0	0.0
User DelAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
AdjDel/Veh:	41.9	90.8	72.0	22.5	22.2	37.9	20.7	8.9	7.5	0.0	0.0	0.0
HCM2kAvg:	2	3	2	3	2	9	16	5	4	0	0	0

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Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound				South Bound				East Bound				West Bound			
Movement:	L	- T	- R		L	- T	- R		L	- T	- R		L	- T	- R	
HCM Ops Adjusted Lane Utilization Module:																
Lanes:	1	0 1	0 1		1	0 1	1 0		1	0 0	1 0		0	0 0	0 0	
Lane Group:	L	T	R		L	RT	RT		L	RT	RT		xxxx	xxxx	xxxx	
#LnsInGrps:	1	1	1		1	2	2		1	1	1		0	0	0	
HCM Ops Input Saturation Adj Module:																
Lane Width:	12	12	12	12	12	12	12	12	12	12	12	12				
CrosswalkWid	8				8				8				8			
% Hev Veh:	0				0				0				0			
Grade:	0%				0%				0%				0%			
Parking/Hr:	No				No				No				No			
Bus Stp/Hr:	0				0				0				0			
Area Type:	< < < < < < < < < < < < < < < < Other > > > > > > > > > > > > > > > >															
Cnft Ped/Hr:	256				59				203				314			
ExclusiveRT:	Include				Include				Include				Include			
% RT Prtct:	0				0				0				0			
HCM Ops f(lt) Adj Case Module:																
f(lt) Case:	1	xxxx	xxxx		1	xxxx	xxxx		5r	xxxx	xxxx		xxxx	xxxx	xxxx	
HCM Ops Saturation Adj Module:																
Ln Wid Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Hev Veh Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Grade Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Parking Adj:	xxxx	xxxx	1.00		xxxx	1.00	1.00		xxxx	1.00	1.00		xxxx	xxxx	xxxxx	
Bus Stp Adj:	xxxx	xxxx	1.00		xxxx	1.00	1.00		xxxx	1.00	1.00		xxxx	xxxx	xxxxx	
Area Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
RT Adj:	xxxx	xxxx	0.85		xxxx	0.89	0.89		xxxx	0.98	0.98		xxxx	xxxx	xxxxx	
LT Adj:	0.95	xxxx	xxxxx		0.95	xxxx	xxxxx		0.98	xxxx	xxxxx		xxxx	xxxx	xxxxx	
PedBike Adj:	1.00	1.00	1.00		1.00	1.00	1.00		0.81	1.00	0.98		1.00	1.00	1.00	
HCM Sat Adj:	0.95	1.00	0.85		0.95	0.89	0.89		0.79	0.98	0.96		1.00	1.00	1.00	
Usr Sat Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00	
MLF Sat Adj:	1.00	1.00	1.00		1.00	0.95	0.95		1.00	1.00	1.00		1.00	1.00	1.00	
Fnl Sat Adj:	0.95	1.00	0.85		0.95	0.84	0.84		0.79	0.98	0.96		1.00	1.00	1.00	
Delay Adjustment Factor Module:																
Coordinated:	< < < < < < < < < < < < < < < < No > > > > > > > > > > > > > > > >															
Signal Type:	< < < < < < < < < < < < < < Actuated > > > > > > > > > > > > > >															
DelAdjFctr:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		0.00	0.00	0.00	

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Ped/Bike Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	- T	- R	L	- T	- R	L	- T	- R	L	- T	- R
CrsswalkWid:	0.00		0.00	0.00		0.00	8.00		8.00	0.00		0.00
CrsswalkLen:	0.00		0.00	0.00		0.00	48.00		60.00	0.00		0.00
MinPedGrn:	0.00		0.00	0.00		0.00	16.85		19.27	0.00		0.00
PedGrn:	0.00		0.00	0.00		0.00	37.50		37.50	0.00		0.00
PedVol:	0		0	0		0	314		203	0		0
PedFlowRate:	0		0	0		0	586		379	0		0
BikeVol:	0		0	0		0	26		69	0		0
BikeFlwRate:	0		0	0		0	0		129	0		0
PedOcc:	0.000		0.000	0.000		0.000	0.293		0.189	0.000		0.000
BikeOcc:	0.000		0.000	0.000		0.000	0.000		0.068	0.000		0.000
PedAfterOcc:	0.000		0.000	0.000		0.000	0.293		0.000	0.000		0.000
rOcc:	0.000		0.000	0.000		0.000	0.293		0.244	0.000		0.000
TurnVehAdj:	0.000		0.000	0.000		0.000	0.707		0.853	0.000		0.000
Prt:	0.000		0.000	0.000		0.000	0.000		0.139	0.000		0.000
Prta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
Plt:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
Plta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
PedBikeAdj:	1.000		1.000	1.000		1.000	0.812		0.980	1.000		1.000

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (HCM2000 Queue Method)
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L -	T -	R	L -	T -	R	L -	T -	R	L -	T -	R
Green/Cycle:	0.04	0.04	0.04	0.23	0.23	0.23	0.56	0.56	0.60	0.00	0.00	0.00
ArrivalType:		3			3			3			3	
ProgFactor:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Q1:	0.7	1.1	0.8	2.1	1.8	5.9	11.8	4.3	3.9	0.0	0.0	0.0
UpstreamVC:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
UpstreamAdj:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EarlyArrAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00
Q2:	1.0	2.1	1.6	0.4	0.4	3.6	4.3	0.6	0.6	0.0	0.0	0.0
HCM2KQueue:	1.7	3.3	2.4	2.5	2.2	9.5	16.1	4.9	4.5	0.0	0.0	0.0
70th%Factor:	1.20	1.19	1.19	1.19	1.19	1.18	1.17	1.19	1.19	1.20	1.20	1.20
70th%HCM2kQ:	2.0	3.9	2.9	3.0	2.6	11.2	18.8	5.9	5.3	0.0	0.0	0.0
85th%Factor:	1.58	1.57	1.58	1.58	1.58	1.52	1.48	1.55	1.56	1.60	1.60	1.60
85th%HCM2kQ:	2.7	5.1	3.8	3.9	3.4	14.4	23.7	7.7	7.0	0.0	0.0	0.0
90th%Factor:	1.77	1.74	1.75	1.75	1.76	1.65	1.58	1.71	1.72	1.80	1.80	1.80
90th%HCM2kQ:	3.0	5.7	4.2	4.4	3.8	15.6	25.4	8.5	7.7	0.0	0.0	0.0
95th%Factor:	2.05	2.00	2.02	2.02	2.03	1.85	1.75	1.96	1.97	2.10	2.10	2.10
95th%HCM2kQ:	3.4	6.5	4.9	5.1	4.4	17.6	28.1	9.7	8.8	0.0	0.0	0.0
98th%Factor:	2.58	2.48	2.53	2.52	2.55	2.18	1.99	2.38	2.41	2.70	2.70	2.70
98th%HCM2kQ:	4.3	8.1	6.1	6.3	5.5	20.7	32.0	11.8	10.8	0.0	0.0	0.0

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Computation Report
2000 HCM Operations Method (Future Volume Alternative)

Intersection #2 DEPOT INTX

Cycle (sec): 90 Critical Vol./Cap. (X): 0.949
Loss Time (sec): 12 (Y+R = 4 sec) Average Delay (sec/veh): 34.4
Optimal Cycle:OPTIMIZED Level Of Service: C

Street Name:	Center						Pacific					
Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L - T - R			L - T - R			L - T - R			L - T - R		
Control:	Permitted			Permitted			Permitted			Permitted		
Rights:	Include			Include			Include			Include		
Min. Green:	0	0	0	0	0	0	0	0	0	0	0	0
Lanes:	0 0	1! 0	0	1 0	0 1	0	0 0	0 0	0	1 0	1 0	1
Volume Module: SATURDAY												
Base Vol:	55	245	572	35	127	358	0	0	0	281	542	63
Growth Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Initial Bse:	55	245	572	35	127	358	0	0	0	281	542	63
Added Vol:	0	0	0	0	0	0	0	0	0	0	0	0
In-Process:	0	0	0	0	0	0	0	0	0	0	0	0
Initial Fut:	55	245	572	35	127	358	0	0	0	281	542	63
User Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Volume:	55	245	572	35	127	358	0	0	0	281	542	63
Reduct Vol:	0	0	0	0	0	0	0	0	0	0	0	0
Reduced Vol:	55	245	572	35	127	358	0	0	0	281	542	63
PCE Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
MLF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Final Vol.:	55	245	572	35	127	358	0	0	0	281	542	63
Saturation Flow Module:												
Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.86	0.86	0.85	0.57	0.89	0.87	1.00	1.00	1.00	0.85	1.00	0.85
Lanes:	0.06	0.28	0.66	1.00	0.26	0.74	0.00	0.00	0.00	1.00	1.00	1.00
Final Sat.:	102	456	1065	1077	435	1225	0	0	0	1615	1900	1615
Capacity Analysis Module:												
Vol/Sat:	0.54	0.54	0.54	0.03	0.29	0.29	0.00	0.00	0.00	0.17	0.29	0.04
Crit Moves:		****									****	
Green/Cycle:	0.57	0.57	0.57	0.57	0.57	0.57	0.00	0.00	0.00	0.30	0.30	0.30
Volume/Cap:	0.95	0.95	0.95	0.06	0.52	0.52	0.00	0.00	0.00	0.58	0.95	0.13
Delay/Veh:	36.8	36.8	36.8	8.8	12.5	12.5	0.0	0.0	0.0	28.4	56.2	23.0
User DelAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
AdjDel/Veh:	36.8	36.8	36.8	8.8	12.5	12.5	0.0	0.0	0.0	28.4	56.2	23.0
HCM2kAvg:	31	30	29	1	9	8	0	0	0	8	20	1

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Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound				South Bound				East Bound				West Bound			
Movement:	L	- T	- R		L	- T	- R		L	- T	- R		L	- T	- R	
HCM Ops Adjusted Lane Utilization Module:																
Lanes:	0	0	1!	0	0	1	0	0	1	0	0	0	0	0	1	0
Lane Group:	LTR	LTR	LTR		L	RT	RT		xxxx	xxxx	xxxx		L	T	R	
#LnsInGrps:	1	1	1		1	1	1		0	0	0		1	1	1	
HCM Ops Input Saturation Adj Module:																
Lane Width:	12	12	12	12	12	12	12	12	12	12	12	12				
CrosswalkWid	8				8				8				8			
% Hev Veh:	0				0				0				0			
Grade:	0%				0%				0%				0%			
Parking/Hr:	No				No				No				No			
Bus Stp/Hr:	0				0				0				0			
Area Type:	< < < < < < < < < < < < < < < < Other > > > > > > > > > > > > > > > >															
Cnft Ped/Hr:	0				0				0				0			
ExclusiveRT:	Include				Include				Include				Include			
% RT Prtct:	0				0				0				0			
HCM Ops f(lt) Adj Case Module:																
f(lt) Case:	5	5	5		2	xxxx	xxxx	xxxx	xxxx	xxxx		2r	xxxx	xxxx		
HCM Ops Saturation Adj Module:																
Ln Wid Adj:	1.00	1.00	1.00	1.00	1.00	1.00	xxxx	xxxx	xxxxx	1.00	1.00	1.00				
Hev Veh Adj:	1.00	1.00	1.00	1.00	1.00	1.00	xxxx	xxxx	xxxxx	1.00	1.00	1.00				
Grade Adj:	1.00	1.00	1.00	1.00	1.00	1.00	xxxx	xxxx	xxxxx	1.00	1.00	1.00				
Parking Adj:	1.00	1.00	1.00	xxxx	1.00	1.00	xxxx	xxxx	xxxxx	xxxx	xxxx	1.00				
Bus Stp Adj:	1.00	1.00	1.00	xxxx	1.00	1.00	xxxx	xxxx	xxxxx	xxxx	xxxx	1.00				
Area Adj:	1.00	1.00	1.00	1.00	1.00	1.00	xxxx	xxxx	xxxxx	1.00	1.00	1.00				
RT Adj:	0.91	0.91	0.91	xxxx	0.89	0.89	xxxx	xxxx	xxxxx	xxxx	xxxx	0.85				
LT Adj:	0.95	0.95	0.95	0.57	xxxx	xxxxx	xxxx	xxxx	xxxxx	0.85	xxxx	xxxxx				
PedBike Adj:	1.00	1.00	0.99	1.00	1.00	0.98	1.00	1.00	1.00	1.00	1.00	1.00				
HCM Sat Adj:	0.86	0.86	0.85	0.57	0.89	0.87	1.00	1.00	1.00	0.85	1.00	0.85				
Usr Sat Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00				
MLF Sat Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00				
Fnl Sat Adj:	0.86	0.86	0.85	0.57	0.89	0.87	1.00	1.00	1.00	0.85	1.00	0.85				
Delay Adjustment Factor Module:																
Coordinated:	< < < < < < < < < < < < < < < < No > > > > > > > > > > > > > > > >															
Signal Type:	< < < < < < < < < < < < < < Actuated > > > > > > > > > > > > > >															
DelAdjFctr:	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	1.00	1.00	1.00				

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Permitted Left Turn Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North	South	East	West
Cycle Length, C:	90	90	xxxxxx	xxxxxx
Actual Green Time Per Lane Group, G:	52.95	52.95	xxxxxx	xxxxxx
Effective Green Time Per Lane Group, g:	50.95	50.95	xxxxxx	xxxxxx
Opposing Effective Green Time, go:	50.95	50.95	xxxxxx	xxxxxx
Number Of Opposing Lanes, No:	1	1	xxxxxx	xxxxxx
Number Of Lanes In Lane Group, N:	1	1	xxxxxx	xxxxxx
Adjusted Left-Turn Flow Rate, Vlt:	55	35	xxxxxx	xxxxxx
Proportion of Left Turns in Lane Group, Plt:	0.06	1.00	xxxxxx	xxxxxx
Proportion of Left Turns in Opp Flow, Plto:	xxxxxx	1.00	xxxxxx	xxxxxx
Left Turns Per Cycle, LTC:	1.38	0.88	xxxxxx	xxxxxx
Adjusted Opposing Flow Rate, Vo:	485	872	xxxxxx	xxxxxx
Opposing Flow Per Lane Per Cycle, Volc:	12.13	21.80	xxxxxx	xxxxxx
Opposing Platoon Ratio, Rpo:	1.00	1.00	xxxxxx	xxxxxx
Lost Time Per Phase, tl:	6.00	6.00	xxxxxx	xxxxxx
Eff grn until arrival of left-turn car, gf:	12.51	17.75	xxxxxx	xxxxxx
Opposing Queue Ratio, qro:	0.43	0.43	xxxxxx	xxxxxx
Eff grn blocked by opposing queue, gq:	8.40	15.34	xxxxxx	xxxxxx
Eff grn while left turns filter thru, gu:	38.44	33.20	xxxxxx	xxxxxx
Max opposing cars arriving during gq-gf, n:	xxxxxx	0.00	xxxxxx	xxxxxx
Proportion of Opposing Thru & RT cars, ptho:	xxxxxx	0.00	xxxxxx	xxxxxx
Left-turn Saturation Factor, fs:	0.57	xxxxxx	xxxxxx	xxxxxx
Proportion of Left Turns in Shared Lane, pl:	0.06	1.00	xxxxxx	xxxxxx
Through-car Equivalents, el1:	2.27	2.98	xxxxxx	xxxxxx
Single Lane Through-car Equivalents, el2:	xxxxxx	1.00	xxxxxx	xxxxxx
Minimum Left Turn Adjustment Factor, fmin:	0.04	0.08	xxxxxx	xxxxxx
Single Lane Left Turn Adjustment Factor, fm:	0.95	0.57	xxxxxx	xxxxxx
Left Turn Adjustment Factor, flt:	0.95	0.57	xxxxxx	xxxxxx

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Ped/Bike Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	T	R	L	T	R	L	T	R	L	T	R
CrsswalkWid:	8.00		8.00	8.00		8.00	0.00		0.00	8.00		8.00
CrsswalkLen:	12.00		36.00	36.00		12.00	0.00		0.00	24.00		36.00
MinPedGrn:	6.63		13.49	13.49		6.63	0.00		0.00	10.06		13.49
PedGrn:	50.95		50.95	50.95		50.95	0.00		0.00	27.05		27.05
PedVol:	0		0	0		0	0		0	0		0
PedFlowRate:	0		0	0		0	0		0	0		0
BikeVol:	18		2	2		18	0		0	0		0
BikeFlwRate:	0		4	0		32	0		0	0		0
PedOcc:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
BikeOcc:	0.000		0.021	0.000		0.032	0.000		0.000	0.000		0.000
PedAfterOcc:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
rOcc:	0.000		0.021	0.000		0.032	0.000		0.000	0.000		0.000
TurnVehAdj:	1.000		0.979	1.000		0.968	0.000		0.000	1.000		1.000
Prt:	0.000		0.656	0.000		0.738	0.000		0.000	0.000		1.000
Prta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
Plt:	0.063		0.000	1.000		0.000	0.000		0.000	0.000		0.000
Plta:	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000
PedBikeAdj:	1.000		0.986	1.000		0.977	1.000		1.000	1.000		1.000

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (HCM2000 Queue Method)
2000 HCM Operations Method
Future Volume Alternative

Intersection #2 DEPOT INTX

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L -	T -	R	L -	T -	R	L -	T -	R	L -	T -	R
Green/Cycle:	0.57	0.57	0.57	0.57	0.57	0.57	0.00	0.00	0.00	0.30	0.30	0.30
ArrivalType:		3			3			3			3	
ProgFactor:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Q1:	22.7	21.7	21.4	0.7	7.5	7.4	0.0	0.0	0.0	7.0	13.3	1.1
UpstreamVC:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
UpstreamAdj:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EarlyArrAdj:	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00	1.00	1.00	1.00
Q2:	8.2	8.1	8.0	0.1	1.0	1.0	0.0	0.0	0.0	1.3	6.6	0.1
HCM2KQueue:	30.9	29.7	29.5	0.8	8.5	8.5	0.0	0.0	0.0	8.3	19.9	1.3
70th%Factor:	1.15	1.15	1.15	1.20	1.18	1.18	1.20	1.20	1.20	1.18	1.16	1.20
70th%HCM2kQ:	35.4	34.1	33.8	0.9	10.1	10.0	0.0	0.0	0.0	9.8	23.1	1.5
85th%Factor:	1.41	1.41	1.41	1.59	1.53	1.53	1.60	1.60	1.60	1.53	1.45	1.59
85th%HCM2kQ:	43.5	41.9	41.6	1.2	13.0	12.9	0.0	0.0	0.0	12.7	28.9	2.1
90th%Factor:	1.49	1.49	1.49	1.79	1.66	1.66	1.80	1.80	1.80	1.66	1.55	1.77
90th%HCM2kQ:	45.9	44.3	44.0	1.3	14.2	14.1	0.0	0.0	0.0	13.8	30.7	2.3
95th%Factor:	1.61	1.62	1.62	2.08	1.87	1.87	2.10	2.10	2.10	1.88	1.70	2.06
95th%HCM2kQ:	49.7	48.0	47.7	1.6	16.0	15.9	0.0	0.0	0.0	15.6	33.7	2.7
98th%Factor:	1.79	1.80	1.80	2.64	2.22	2.22	2.70	2.70	2.70	2.23	1.92	2.61
98th%HCM2kQ:	55.4	53.5	53.2	2.0	18.9	18.8	0.0	0.0	0.0	18.5	38.1	3.4

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Computation Report
2000 HCM Operations Method (Future Volume Alternative)

Intersection #1 Wharf Intx

Cycle (sec): 100 Critical Vol./Cap. (X): 0.972
Loss Time (sec): 12 (Y+R = 4 sec) Average Delay (sec/veh): 44.0
Optimal Cycle:OPTIMIZED Level Of Service: D

Street Name:	Pacific									Beach		
Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L - T - R			L - T - R			L - T - R			L - T - R		
Control:	Split Phase			Split Phase			Permitted			Permitted		
Rights:	Include			Include			Ovl			Include		
Min. Green:	0	0	0	0	0	0	0	0	0	0	0	0
Lanes:	1 0 1	0 1		1 0 1	1 0		1 0 0	1 0		0 0 0	0 0	
Volume Module: SATURDAY												
Base Vol:	50	81	62	178	152	451	929	520	77	0	0	0
Growth Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Initial Bse:	50	81	62	178	152	451	929	520	77	0	0	0
Added Vol:	0	0	0	0	0	0	0	0	0	0	0	0
In-Process:	0	0	0	0	0	0	0	0	0	0	0	0
Initial Fut:	50	81	62	178	152	451	929	520	77	0	0	0
User Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
PHF Volume:	50	81	62	178	152	451	929	520	77	0	0	0
Reduct Vol:	0	0	0	0	0	0	0	0	0	0	0	0
Reduced Vol:	50	81	62	178	152	451	929	520	77	0	0	0
PCE Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
MLF Adj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Final Vol.:	50	81	62	178	152	451	929	520	77	0	0	0
Saturation Flow Module:												
Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.95	1.00	0.85	0.95	0.84	0.84	0.92	0.98	0.95	1.00	1.00	1.00
Lanes:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.87	0.13	0.00	0.00	0.00
Final Sat.:	1805	1900	1615	1805	1603	1603	1749	1618	240	0	0	0
Capacity Analysis Module:												
Vol/Sat:	0.03	0.04	0.04	0.10	0.09	0.28	0.53	0.32	0.32	0.00	0.00	0.00
Crit Moves:		****				****	****					
Green/Cycle:	0.04	0.04	0.04	0.29	0.29	0.29	0.55	0.55	0.59	0.00	0.00	0.00
Volume/Cap:	0.63	0.97	0.88	0.34	0.33	0.97	0.97	0.59	0.54	0.00	0.00	0.00
Delay/Veh:	62.3	137	113.2	28.4	28.0	64.1	44.3	16.1	12.9	0.0	0.0	0.0
User DelAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
AdjDel/Veh:	62.3	137	113.2	28.4	28.0	64.1	44.3	16.1	12.9	0.0	0.0	0.0
HCM2kAvg:	3	5	4	5	4	19	35	12	11	0	0	0

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Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound				South Bound				East Bound				West Bound			
Movement:	L	- T	- R		L	- T	- R		L	- T	- R		L	- T	- R	
HCM Ops Adjusted Lane Utilization Module:																
Lanes:	1	0 1	0 1		1	0 1	1 0		1	0 0	1 0		0	0 0	0 0	
Lane Group:	L	T	R		L	RT	RT		L	RT	RT		xxxx	xxxx	xxxx	
#LnsInGrps:	1	1	1		1	2	2		1	1	1		0	0	0	
HCM Ops Input Saturation Adj Module:																
Lane Width:	12	12	12	12	12	12	12	12	12	12	12	12				
CrosswalkWid	8				8				8				8			
% Hev Veh:	0				0				0				0			
Grade:	0%				0%				0%				0%			
Parking/Hr:	No				No				No				No			
Bus Stp/Hr:	0				0				0				0			
Area Type:	< < < < < < < < < < < < < < < < Other > > > > > > > > > > > > > > > >															
Cnft Ped/Hr:	510				86				373				580			
ExclusiveRT:	Include				Include				Include				Include			
% RT Prtct:	0				0				0				0			
HCM Ops f(lt) Adj Case Module:																
f(lt) Case:	1	xxxx	xxxx		1	xxxx	xxxx		5r	xxxx	xxxx		xxxx	xxxx	xxxx	
HCM Ops Saturation Adj Module:																
Ln Wid Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Hev Veh Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Grade Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
Parking Adj:	xxxx	xxxx	1.00		xxxx	1.00	1.00		xxxx	1.00	1.00		xxxx	xxxx	xxxxx	
Bus Stp Adj:	xxxx	xxxx	1.00		xxxx	1.00	1.00		xxxx	1.00	1.00		xxxx	xxxx	xxxxx	
Area Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		xxxx	xxxx	xxxxx	
RT Adj:	xxxx	xxxx	0.85		xxxx	0.89	0.89		xxxx	0.98	0.98		xxxx	xxxx	xxxxx	
LT Adj:	0.95	xxxx	xxxxx		0.95	xxxx	xxxxx		0.98	xxxx	xxxxx		xxxx	xxxx	xxxxx	
PedBike Adj:	1.00	1.00	1.00		1.00	1.00	1.00		0.94	1.00	0.97		1.00	1.00	1.00	
HCM Sat Adj:	0.95	1.00	0.85		0.95	0.89	0.89		0.92	0.98	0.95		1.00	1.00	1.00	
Usr Sat Adj:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00	
MLF Sat Adj:	1.00	1.00	1.00		1.00	0.95	0.95		1.00	1.00	1.00		1.00	1.00	1.00	
Fnl Sat Adj:	0.95	1.00	0.85		0.95	0.84	0.84		0.92	0.98	0.95		1.00	1.00	1.00	
Delay Adjustment Factor Module:																
Coordinated:	< < < < < < < < < < < < < < < < No > > > > > > > > > > > > > > > >															
Signal Type:	< < < < < < < < < < < < < < Actuated > > > > > > > > > > > > > >															
DelAdjFctr:	1.00	1.00	1.00		1.00	1.00	1.00		1.00	1.00	1.00		0.00	0.00	0.00	

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (Ped/Bike Sat Adj)
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound				South Bound				East Bound				West Bound			
Movement:	L	-	T	- R	L	-	T	- R	L	-	T	- R	L	-	T	- R
CrsswalkWid:	0.00		0.00		0.00		0.00		8.00		8.00		0.00		0.00	
CrsswalkLen:	0.00		0.00		0.00		0.00		48.00		60.00		0.00		0.00	
MinPedGrn:	0.00		0.00		0.00		0.00		19.55		21.00		0.00		0.00	
PedGrn:	0.00		0.00		0.00		0.00		57.42		57.42		0.00		0.00	
PedVol:		0		0		0		0		580		373		0		0
PedFlowRate:		0		0		0		0		1010		650		0		0
BikeVol:		0		0		0		0		21		49		0		0
BikeFlwRate:		0		0		0		0		0		85		0		0
PedOcc:	0.000		0.000		0.000		0.000		0.101		0.325		0.000		0.000	
BikeOcc:	0.000		0.000		0.000		0.000		0.000		0.052		0.000		0.000	
PedAfterOcc:	0.000		0.000		0.000		0.000		0.101		0.000		0.000		0.000	
rOcc:	0.000		0.000		0.000		0.000		0.101		0.360		0.000		0.000	
TurnVehAdj:	0.000		0.000		0.000		0.000		0.899		0.784		0.000		0.000	
Prt:	0.000		0.000		0.000		0.000		0.000		0.129		0.000		0.000	
Prta:	0.000		0.000		0.000		0.000		0.000		0.000		0.000		0.000	
Plt:	0.000		0.000		0.000		0.000		0.000		0.000		0.000		0.000	
Plta:	0.000		0.000		0.000		0.000		0.000		0.000		0.000		0.000	
PedBikeAdj:	1.000		1.000		1.000		1.000		0.938		0.972		1.000		1.000	

Feasibility Report
Traffic Impact Analysis Data
Roundabouts & Traffic Engineering (RTE)

Level Of Service Detailed Computation Report (HCM2000 Queue Method)
2000 HCM Operations Method
Future Volume Alternative

Intersection #1 Wharf Intx

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L -	T -	R	L -	T -	R	L -	T -	R	L -	T -	R
Green/Cycle:	0.04	0.04	0.04	0.29	0.29	0.29	0.55	0.55	0.59	0.00	0.00	0.00
ArrivalType:		3			3			3			3	
ProgFactor:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Q1:	1.4	2.2	1.7	4.1	3.3	12.4	25.4	11.1	9.9	0.0	0.0	0.0
UpstreamVC:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
UpstreamAdj:	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
EarlyArrAdj:	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	0.00	0.00
Q2:	1.3	3.0	2.3	0.5	0.5	6.7	9.3	1.4	1.2	0.0	0.0	0.0
HCM2KQueue:	2.7	5.3	4.0	4.6	3.8	19.1	34.7	12.5	11.1	0.0	0.0	0.0
70th%Factor:	1.19	1.19	1.19	1.19	1.19	1.16	1.14	1.17	1.18	1.20	1.20	1.20
70th%HCM2kQ:	3.2	6.3	4.8	5.5	4.5	22.2	39.7	14.6	13.1	0.0	0.0	0.0
85th%Factor:	1.57	1.55	1.56	1.56	1.56	1.46	1.39	1.50	1.51	1.60	1.60	1.60
85th%HCM2kQ:	4.3	8.2	6.2	7.2	5.9	27.9	48.4	18.7	16.7	0.0	0.0	0.0
90th%Factor:	1.75	1.71	1.73	1.72	1.73	1.55	1.47	1.61	1.63	1.80	1.80	1.80
90th%HCM2kQ:	4.8	9.0	6.9	7.9	6.6	29.7	51.1	20.2	18.1	0.0	0.0	0.0
95th%Factor:	2.02	1.95	1.98	1.96	1.99	1.71	1.59	1.80	1.82	2.10	2.10	2.10
95th%HCM2kQ:	5.5	10.3	7.9	9.1	7.5	32.6	55.1	22.5	20.3	0.0	0.0	0.0
98th%Factor:	2.51	2.37	2.44	2.40	2.45	1.93	1.77	2.08	2.13	2.70	2.70	2.70
98th%HCM2kQ:	6.8	12.5	9.7	11.1	9.3	36.9	61.4	26.0	23.6	0.0	0.0	0.0