



APPENDIX G

Traffic Impact Analysis

REVISED FINAL REPORT

Mancara at Robinson Ranch Traffic Impact Analysis City of Santa Clarita

Prepared for

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INTRODUCTION

This traffic impact analysis has been prepared for the proposed 185-acre Mancara at Robinson Ranch development located along Lost Canyon Road in the Sand Canyon area of the City of Santa Clarita.

This analysis evaluates the operation of four selected intersections, agreed to by City of Santa Clarita staff, as potentially being significantly impacted by the proposed project. This report provides key traffic information regarding existing traffic volumes, an analysis of impacts at study intersections and a determination of Levels of Service (LOS) using the *Highway Capacity Manual* (HCM) methodology. Mitigation measures are recommended where appropriate.

Project Description

The proposed project is located in the Sand Canyon region in the City of Santa Clarita. It includes a 99-unit single-family residential development, a 5-acre city park and 30 acres of open space to be utilized as an equestrian exercise area and an emergency evacuation location. The project site is located between the Metrolink / Union Pacific railroad tracks and the Robinson Ranch golf course. The primary access for the project will be from Lost Canyon Road with a second point of access from Oak Springs Canyon Road.

Figure 1 shows the location of the proposed project site in relation to the surrounding street network. **Figure 2** shows the proposed project site plan.

Study Area

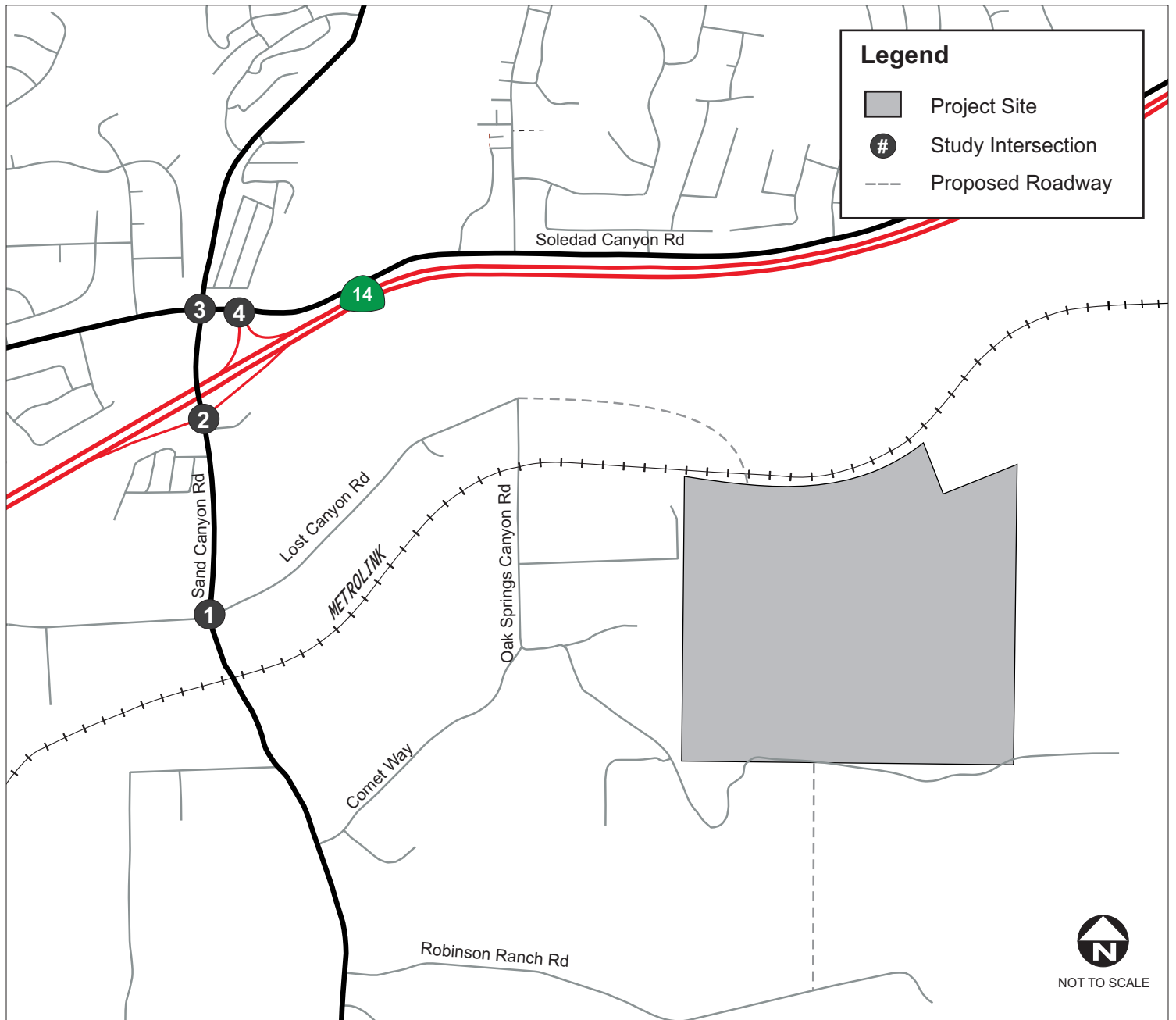
In conjunction with City of Santa Clarita staff, four intersections were selected for level of service (LOS) analysis. The four intersections represent locations that may potentially be impacted by traffic due to the proposed project. The study intersections are as follows:

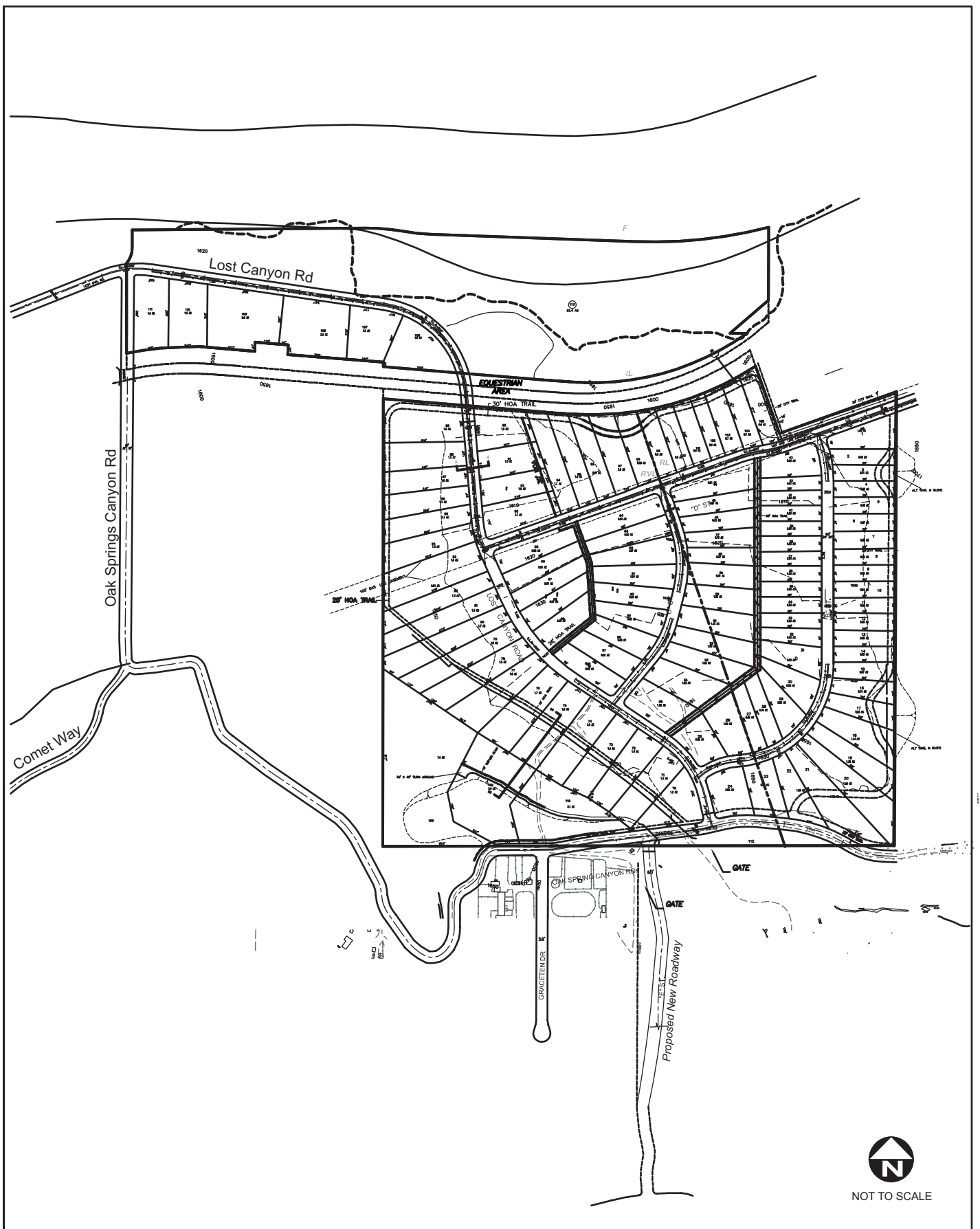
1. Sand Canyon Road at Lost Canyon Road;
2. Sand Canyon Road at State Route 14 Northbound Ramps;
3. Sand Canyon Road at Soledad Canyon Road; and
4. State Route 14 Southbound Ramps at Soledad Canyon Road.

Study Periods

Traffic operations were evaluated for each of the following scenarios during the AM and PM peak hours:

- Existing Conditions;
- Future Without Project Conditions; and
- Future With Project Conditions.





EXISTING CONDITIONS

A site visit was conducted to assess existing conditions at the project site and within the study area. The field inventory included review of intersection geometric layout, traffic control, lane configurations, posted speed limits, transit service, land use, and parking.

Existing Road Network

The following is a description of the existing conditions of the major roadways located within the study area. These descriptions refer to the immediate area surrounding the study intersections.

The Antelope Valley Freeway (SR-14) is a north-south freeway located north of the project site and functions as the primary linkage between the entire Antelope Valley to the north and the San Fernando Valley and the Los Angeles basin to the south. In the immediate vicinity of the project, this facility has two general traffic lanes and one high-occupancy vehicle (HOV) lane in each direction.

Lost Canyon Road is an east-west local residential street that begins at the Santa Clara River, west of Sand Canyon Road, and continues east across Sand Canyon Road to its current terminus just east of Oak Springs Canyon Road. Two school facilities, Pinecrest Elementary and Sulfur Springs Community School, are located west of Sand Canyon Road and utilize Lost Canyon Road as their primary access. This facility consists of one travel lane in each direction. On-street parking is allowed along both sides of the street.

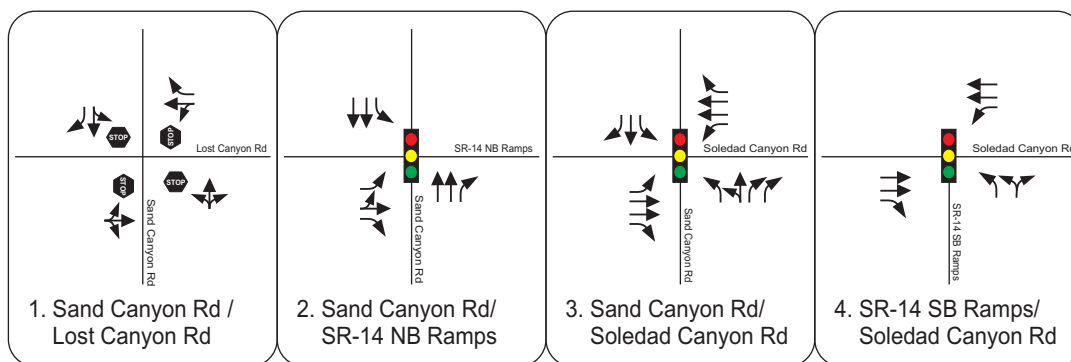
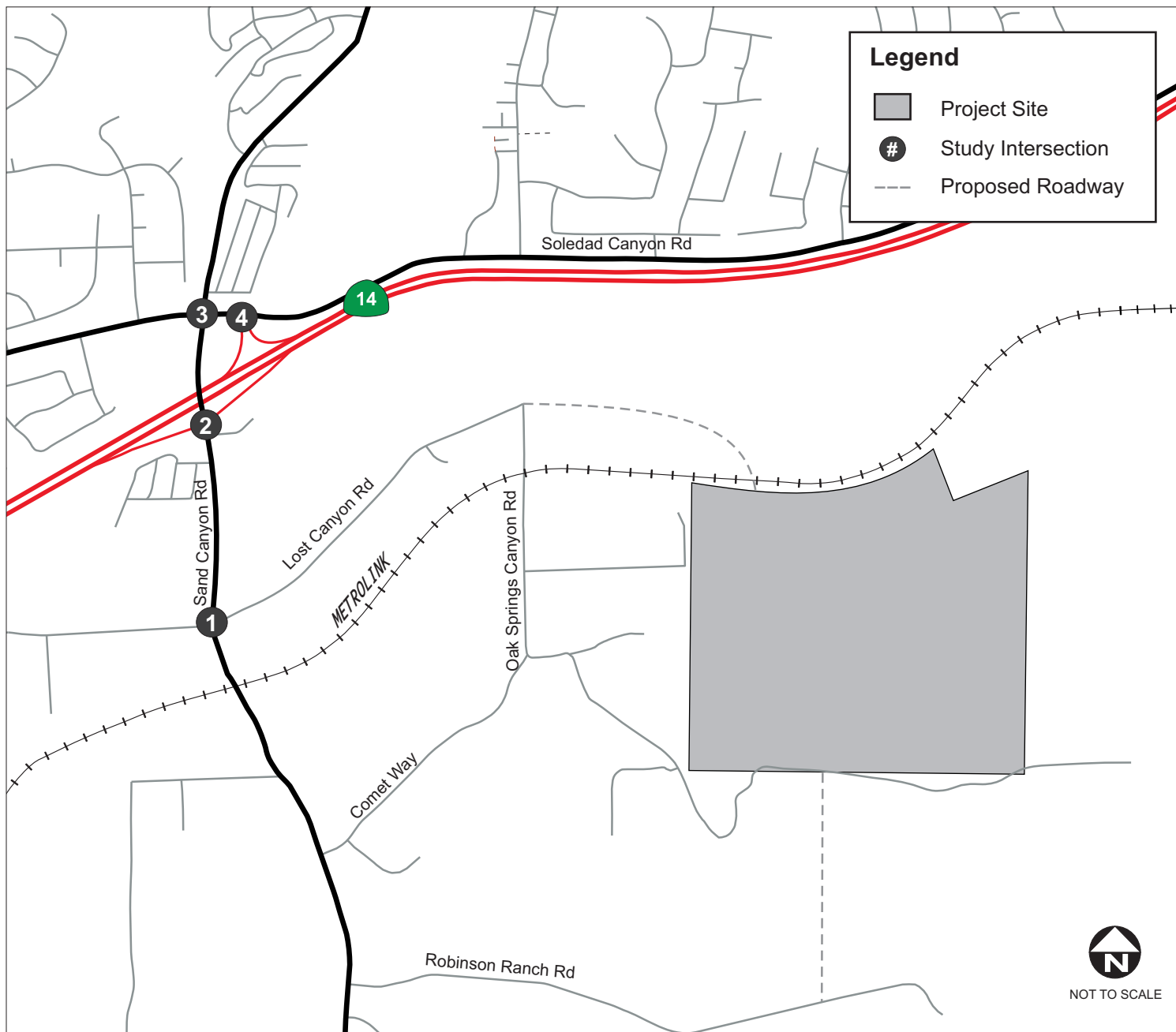
Sand Canyon Road is a north-south major arterial that begins as a T-intersection at Sierra Highway and continues south to Placerita Canyon Road where it becomes Little Tujunga Canyon Road. This facility has an interchange with the Antelope Valley Freeway (SR-14) and provides access to the Sand Canyon region of the City of Santa Clarita. This roadway consists of one travel lane in each direction from Sierra Highway to the northern intersection with Silver Saddle Circle. From Silver Saddle Circle to Soledad Canyon Road, Sand Canyon Road consists of one travel lane in each direction and a continuous two-way left-turn lane. South of Soledad Canyon Road, this facility consists of two travel lanes in each direction until the Santa Clara River. From the Santa Clara River to Placerita Canyon Road, this facility consists of one travel lane in each direction.

Soledad Canyon Road is a major arterial that begins at the Bouquet Canyon Road and Valencia Boulevard intersection and extends east through Santa Clarita and continues to Palmdale. This facility consists of two travel lanes in each direction with a landscaped median and exclusive left-turn pockets at major intersections. Curbside parking is restricted along both sides of the street.

Existing Intersections

Figure 3 illustrates the existing intersection lane configurations for the four analyzed intersections. A brief description of each study intersection follows.

Sand Canyon Road at Lost Canyon Road is a four-way stop-controlled intersection. The southbound and westbound approaches are striped as one shared left-through lane and one right-turn lane. The northbound and eastbound approaches are striped as one all-movement lane.



Sand Canyon Road at SR-14 Northbound Ramps is controlled by a traffic signal with protected / permissive phasing for the southbound left-turn movement. The westbound approach is utilized as the northbound SR-14 on-ramp. The northbound approach is striped as two through lanes and one right-turn lane. The southbound approach is striped as one left-turn lane and two through lanes. The eastbound approach is striped as one left-turn lane, one shared left-through lane and one right-turn lane.

Sand Canyon Road at Soledad Canyon Road is controlled by a traffic signal with split phasing in the northbound and southbound approaches and a right-turn overlap phase in the northbound approach. This signal also operates with protected phasing for left-turn movements in the eastbound and westbound approaches. The northbound approach is striped as one left-turn lane, one shared left-through lane and two right-turn lanes. The southbound approach is striped as one left-turn lane, one through lane and one right-turn lane. The eastbound and westbound approaches are striped as one left-turn lane, two through lanes and one right-turn lane.

SR-14 Southbound Ramps at Soledad Canyon Road is a T-intersection with the southbound approach removed and the northbound approach utilized as the southbound SR-14 ramps. This intersection is controlled by a traffic signal with protected phasing for left-turn movements in the westbound and a right-turn overlap phase for the eastbound movement. The northbound approach is striped as one left-turn lane and one shared left-right lane. The eastbound approach is striped as two through lanes and one right-turn lane. The westbound approach is striped as one left-turn lane and two through lanes.

Existing Public Transit Services

Santa Clarita Transit provides service along one route through the study area. **Route 6** is an east-west local route that travels on Soledad Canyon Road from Shadow Pines Boulevard to Stevenson Ranch via Valencia Town Center and Downtown Newhall. This route operates on weekdays, Saturdays and limited service on Sundays and minor holidays. The closest stop is at Kenroy Avenue, which is located immediately west of Sand Canyon Road along Soledad Canyon Road.

Metrolink's **Antelope Valley Line** provides services from Downtown Los Angeles, through Santa Clarita, to Lancaster. This line travels east-west along the Union Pacific railroad tracks, located on the southern edge of the Santa Clara River and paralleling Soledad Canyon Road. The Via Princessa station is located in the northwest corner of the intersection of Weyerhaeuser Way at Via Princessa. Days of operation are Monday through Sunday with the exception of major holidays.

TRAFFIC OPERATIONS ANALYSIS METHODOLOGY

Traffic operations in the project vicinity were analyzed, as discussed with the City of Santa Clarita staff, using the HCM methodology. The HCM methodology was used to determine average delay per vehicle and service level characteristics for the study intersections.

Level of Service Definitions

Table 1 describes the level of service (LOS) concept and the operating conditions expected under each level of service for signalized and unsignalized intersections.

Table 1: Intersection Level Of Service Definitions

Level of Service	Description	Average Delay (seconds per vehicle)	
		Signalized Intersections	Unsignalized Intersections
A	Excellent operation. All approaches to the intersection appear quite open, turning movements are easily made, and nearly all drivers find freedom of operation.	≤ 10	≤ 10
B	Very good operation. Many drivers begin to feel somewhat restricted within platoons of vehicles. This represents stable flow. An approach to an intersection may occasionally be fully utilized and traffic queues start to form.	>10 and ≤ 20	>10 and ≤ 15
C	Good operation. Occasionally drivers may have to wait more than 60 seconds, and back-ups may develop behind turning vehicles. Most drivers feel somewhat restricted.	>20 and ≤ 35	>15 and ≤ 25
D	Fair operation. Cars are sometimes required to wait more than 60 seconds during short peaks. There are no long-standing traffic queues.	>35 and ≤ 55	>25 and ≤ 35
E	Poor operation. Some long-standing vehicular queues develop on critical approaches to intersections. Delays may be up to several minutes.	>55 and ≤ 80	>35 and ≤ 50
F	Forced flow. Represents jammed conditions. Backups form locations downstream or on the cross street may restrict or prevent movement of vehicles out of the intersection approach lanes; therefore, volumes carried are not predictable. Potential for stop and go type traffic flow.	> 80	> 50
Source: <i>Highway Capacity Manual</i> , Special Report 209, Transportation Research Board, Washington, DC, 2000.			

Level of Service Threshold Criteria

The City of Santa Clarita's significant impact definitions state that a significant impact is deemed to have occurred if the proposed project:

- Worsens an intersection maintained by the City of Santa Clarita from LOS D or better to LOS E or F; or
- Causes an increase in delay of more than 4 seconds at an intersection maintained by the City of Santa Clarita that operates at LOS D with the project; or
- Causes an increase in delay of more than 2 seconds at an intersection maintained by the City of Santa Clarita that operates at LOS E or F with the project.

EXISTING TRAFFIC OPERATIONS ANALYSIS

The morning and evening peak hour level of service analyses were conducted at the four existing study intersections based on the existing traffic volume counts and the methodologies described previously. The level of service analysis was performed using Synchro software, version 6.

New traffic counts were conducted on June 29, 2010 at the four study intersections during the AM peak period (6:30 – 8:30) and PM peak period (4:00 – 6:00). The traffic impact analysis is based on the highest single hour of traffic during each time period at each location. Based on discussions with City

staff, a seasonal increase of 5% was applied to the traffic counts to account for counts taken during off-season months. At the Sand Canyon Road at Lost Canyon Road intersection, 2008 traffic counts obtained from the *Vista Canyon Transit-Oriented Development Transportation Impact Study* (Fehr & Peers, May 2010) were utilized during the a.m. peak hour for turning movements to and away from the Pinecrest School and the Sulphur Springs Community Elementary School along Lost Canyon Road.

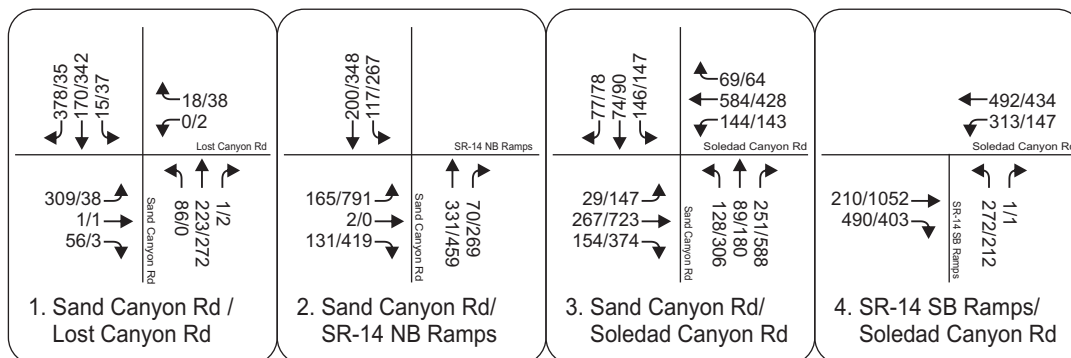
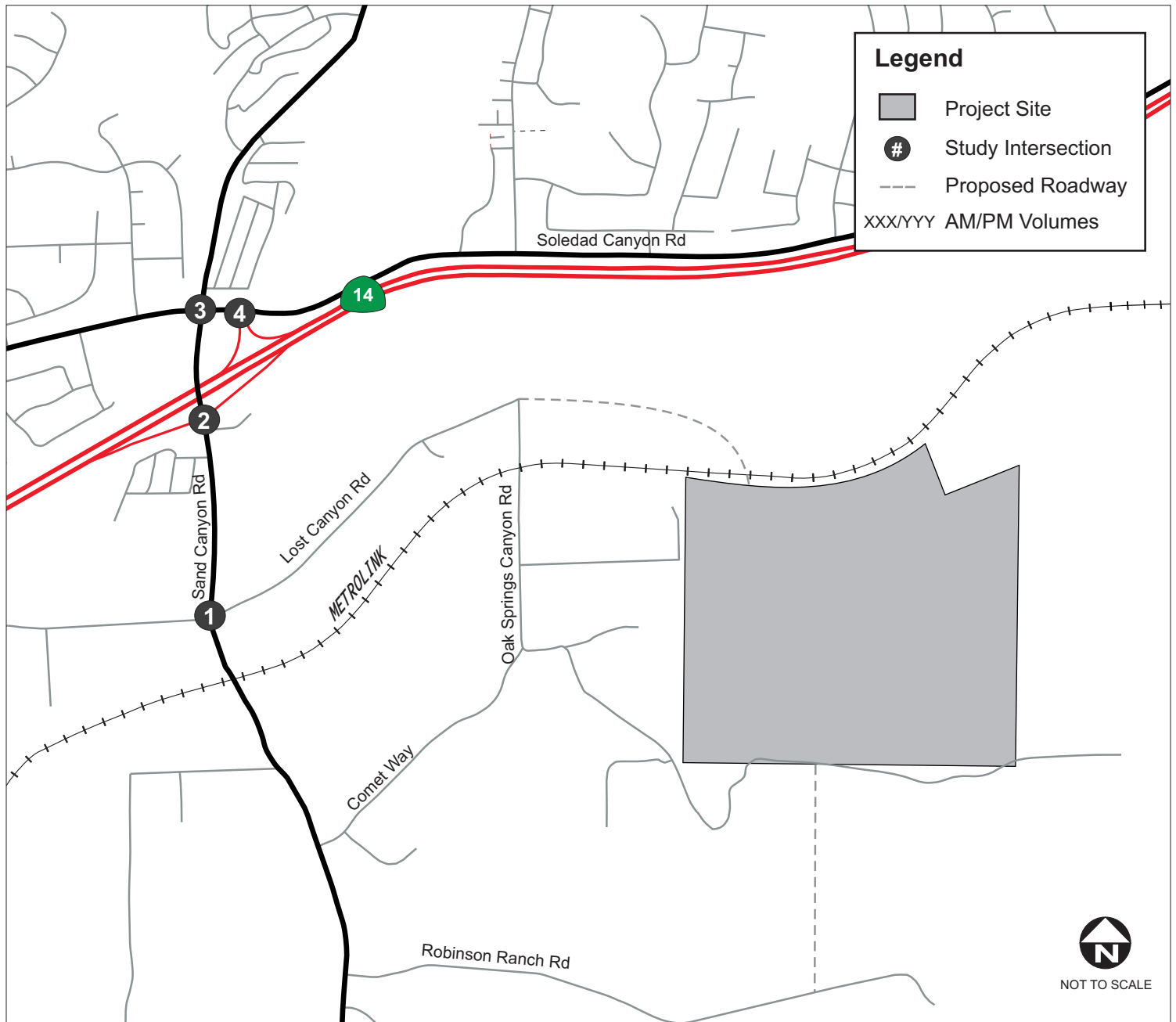
Figure 4 illustrates the existing AM and PM peak hour turning movement volumes at the study intersections. Traffic count sheets are provided in **Appendix A**.

Table 2 summarizes the level of service calculations for the study intersections under existing conditions during the AM and PM peak hours.

TABLE 2: EXISTING CONDITIONS

Intersection		Control	Existing Conditions			
			AM Peak Hour		PM Peak Hour	
			LOS	Delay	LOS	Delay
1	Sand Canyon Rd at Lost Canyon Rd	AWSC	D	29.3	B	12.1
2	Sand Canyon Rd at SR-14 NB Ramps	Signal	B	14.9	C	25.2
3	Sand Canyon Rd at Soledad Canyon Rd	Signal	C	26.8	D	37.2
4	SR-14 SB Ramps at Soledad Canyon Rd	Signal	C	20.2	B	11.5
Notes: HCM 2000 Operations Methodology; AWSC = all-way stop-controlled.						

The results indicate that all four study intersections operate at acceptable levels of service. Level of service analysis worksheets for this scenario are provided in **Appendix B**.



PROPOSED PROJECT

Trip Generation

Trip generation rates for the proposed project were calculated based on those published in the Institute of Transportation Engineers (ITE), *Trip Generation*, 8th Edition. The land use was identified as single-family residential (Land Use Code 210). The results of this calculation are shown in **Table 3**.

Although the project also includes a 5-acre city park and 30 acres of equestrian open space, due to the nature of these uses and their location within the Sand Canyon community, they are not expected to contribute any additional vehicle trips to any of the study intersections during the peak travel periods.

TABLE 3: PROJECT TRIP GENERATION

Land Use	Land Use Code	Size	Units	Trips Ends Generated						
				Weekday AM			Weekday PM			Daily
				In	Out	Total	In	Out	Total	
Mancara at Robinson Ranch Development	210	99	DU	19	56	75	63	37	100	947
Total				19	56	75	63	37	100	947
Source: Institute of Transportation Engineers, <i>Trip Generation</i> , 8 th Edition.										
Note: DU = Dwelling unit										

Trip Distribution and Assignment

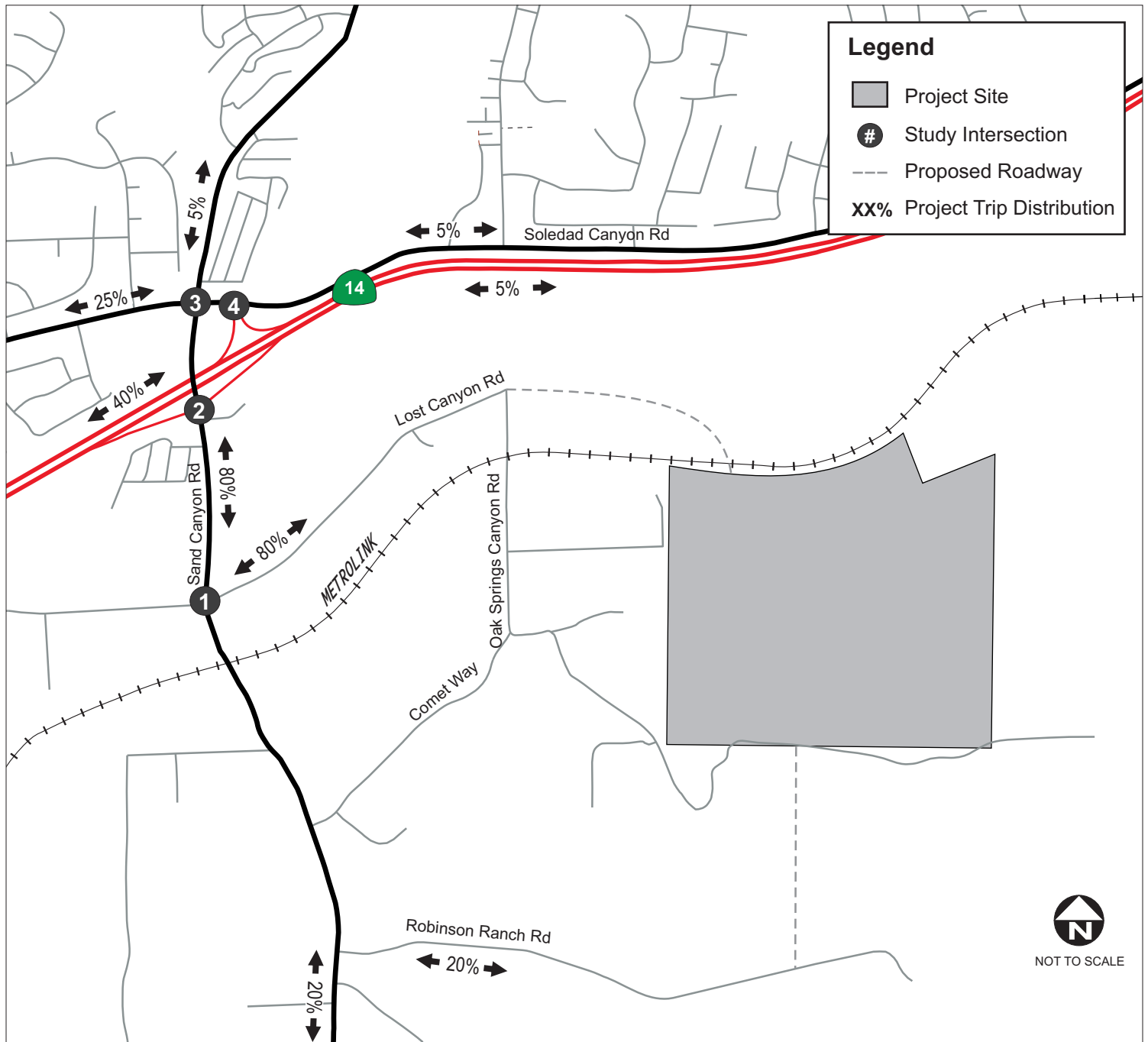
Trip distribution assumptions are used to determine the origin and destination of new vehicle trips associated with the project. The geographic distribution of project trips is based on the locations of local activity centers, the street system that serves the site, and recent traffic data collected in the project study area. The trip distribution utilized for the Future With Project analysis was developed in conjunction with City of Santa Clarita staff. The distribution pattern developed for the project is shown in **Figure 5**.

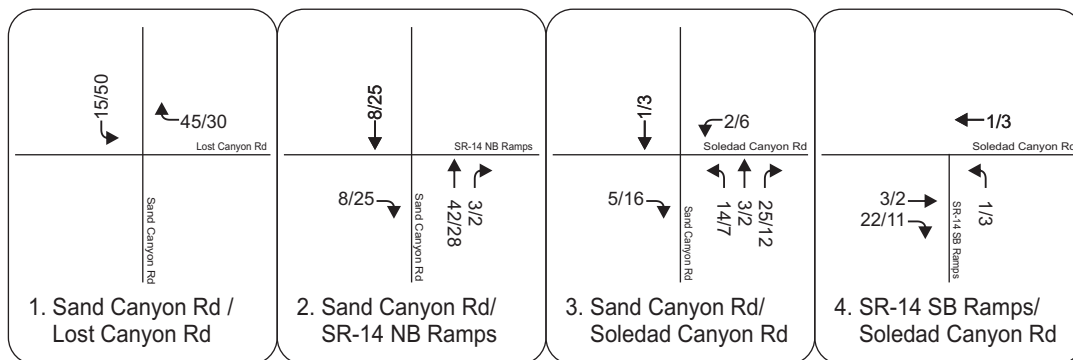
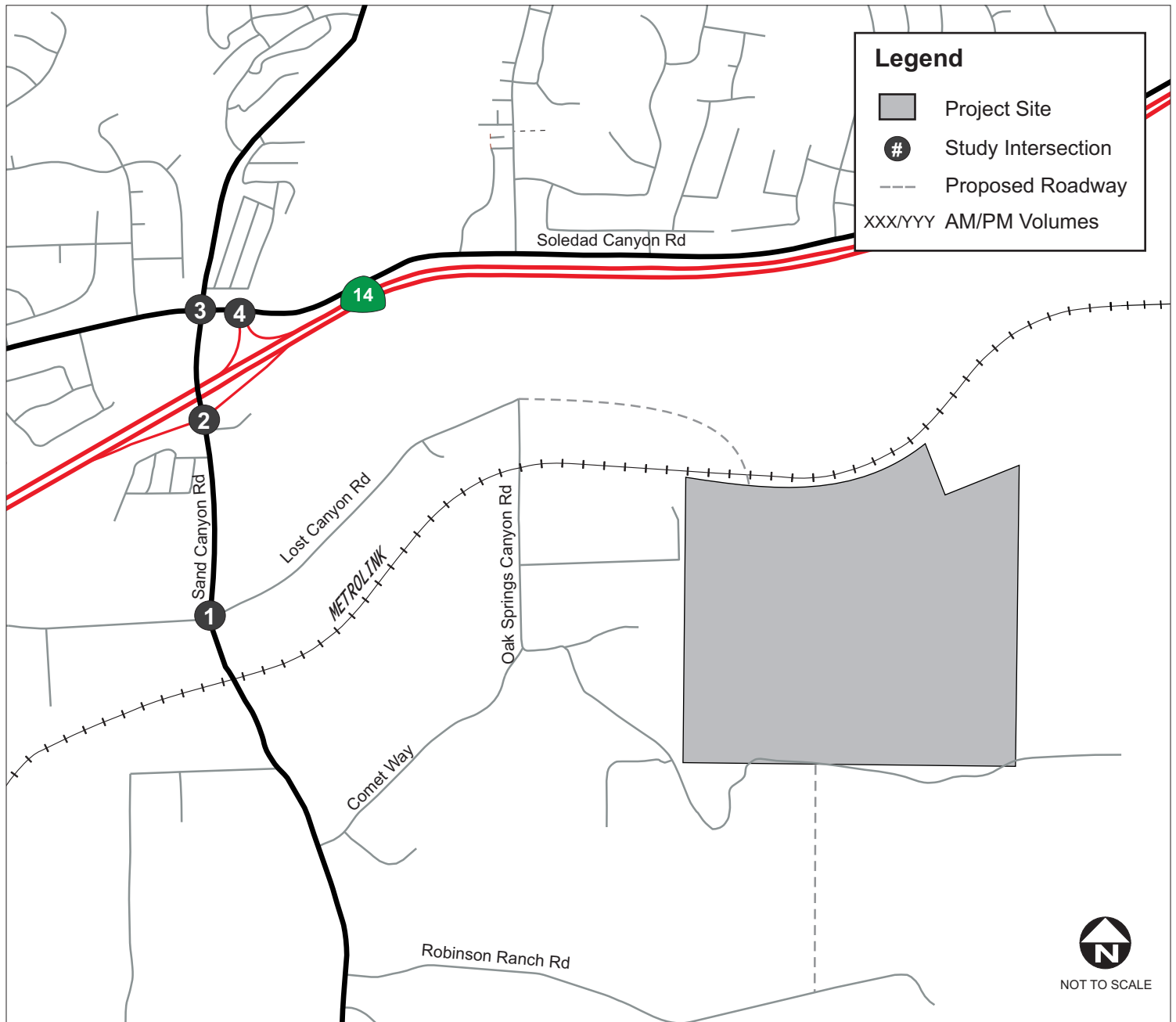
Trips generated by the project, as shown in **Table 3**, were assigned to the surrounding roadway system based on the distribution patterns to estimate the project related peak-hour traffic at each of the study intersections. **Figure 6** illustrates the AM and PM peak hour project trip assignment.

FUTURE VOLUME DEVELOPMENT

Future With Project traffic volumes were obtained from the *Vista Canyon Transit-Oriented Development Transportation Impact Study*, which utilized the Santa Clarita Valley Consolidated Travel Demand Model (SCVCTDM). The model includes related projects located within the Santa Clarita Valley, including the proposed project.

Future Without Project traffic volumes were calculated by subtracting the proposed project trip assignment shown in **Figure 6** from the Future With Project volumes.





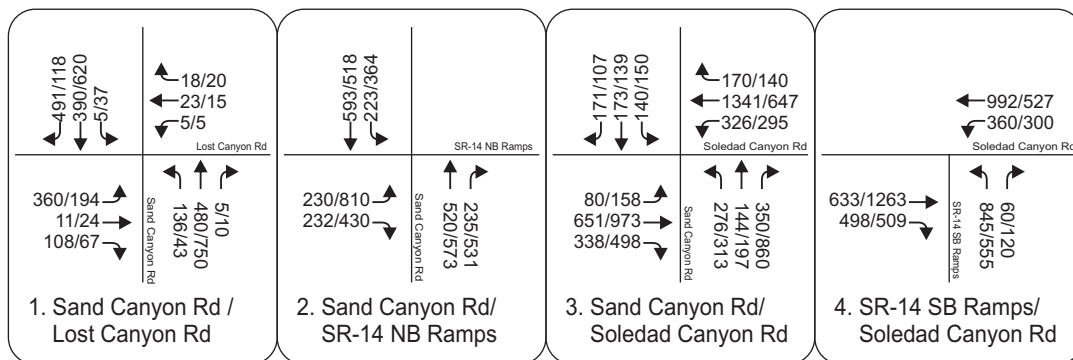
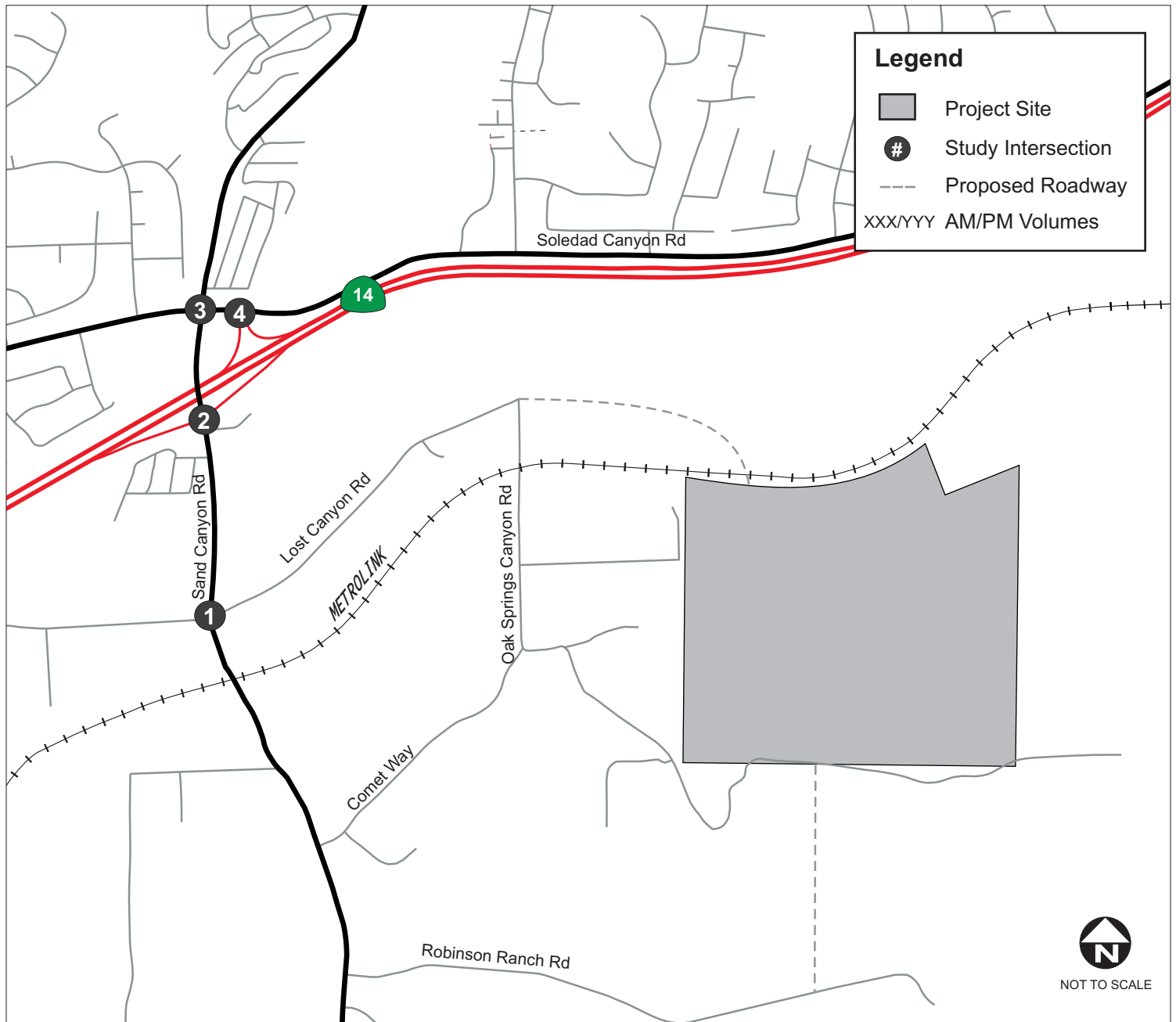
FUTURE WITHOUT PROJECT CONDITIONS

Future Without Project volumes were developed as described above in the “Future Volume Development” section. The resulting traffic volumes were utilized in calculating the levels of service for the study intersections for the Future Without Project conditions during the AM and PM peak hours as summarized in **Table 4** and illustrated in **Figure 7**.

TABLE 4: FUTURE WITHOUT PROJECT CONDITIONS

Intersection		Control	Future Without Project Conditions			
			AM Peak Hour		PM Peak Hour	
			LOS	Delay	LOS	Delay
1	Sand Canyon Rd at Lost Canyon Rd	AWSC	F	135.9	F	223.7
2	Sand Canyon Rd at SR-14 NB Ramps	Signal	B	13.0	C	26.0
3	Sand Canyon Rd at Soledad Canyon Rd	Signal	C	33.8	D	42.9
4	SR-14 SB Ramps at Soledad Canyon Rd	Signal	C	24.9	C	25.6
Notes: HCM 2000 Operations Methodology; AWSC = all-way stop-controlled.						

As can be seen in **Table 4**, three of the four study intersections are expected to operate at acceptable levels of service. The Sand Canyon Road at Lost Canyon Road intersection is expected to operate unacceptably in both the AM and PM peak hour. Level of service analysis worksheets for this scenario are provided in **Appendix B**.



FUTURE WITH PROJECT CONDITIONS

Future With Project volumes were developed as described above in the “Future Volume Development” section. Levels of service for each of the study intersections were then calculated for these volumes and compared against those calculated under the Future Without Project conditions to determine if the proposed project would create any significant traffic impacts, as shown in **Table 5**. The Future With Project traffic volumes for the AM and PM peak hours are illustrated in **Figure 8**.

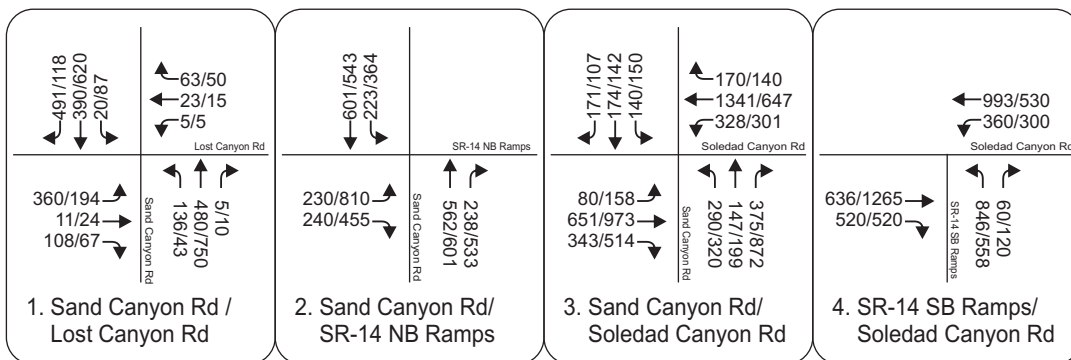
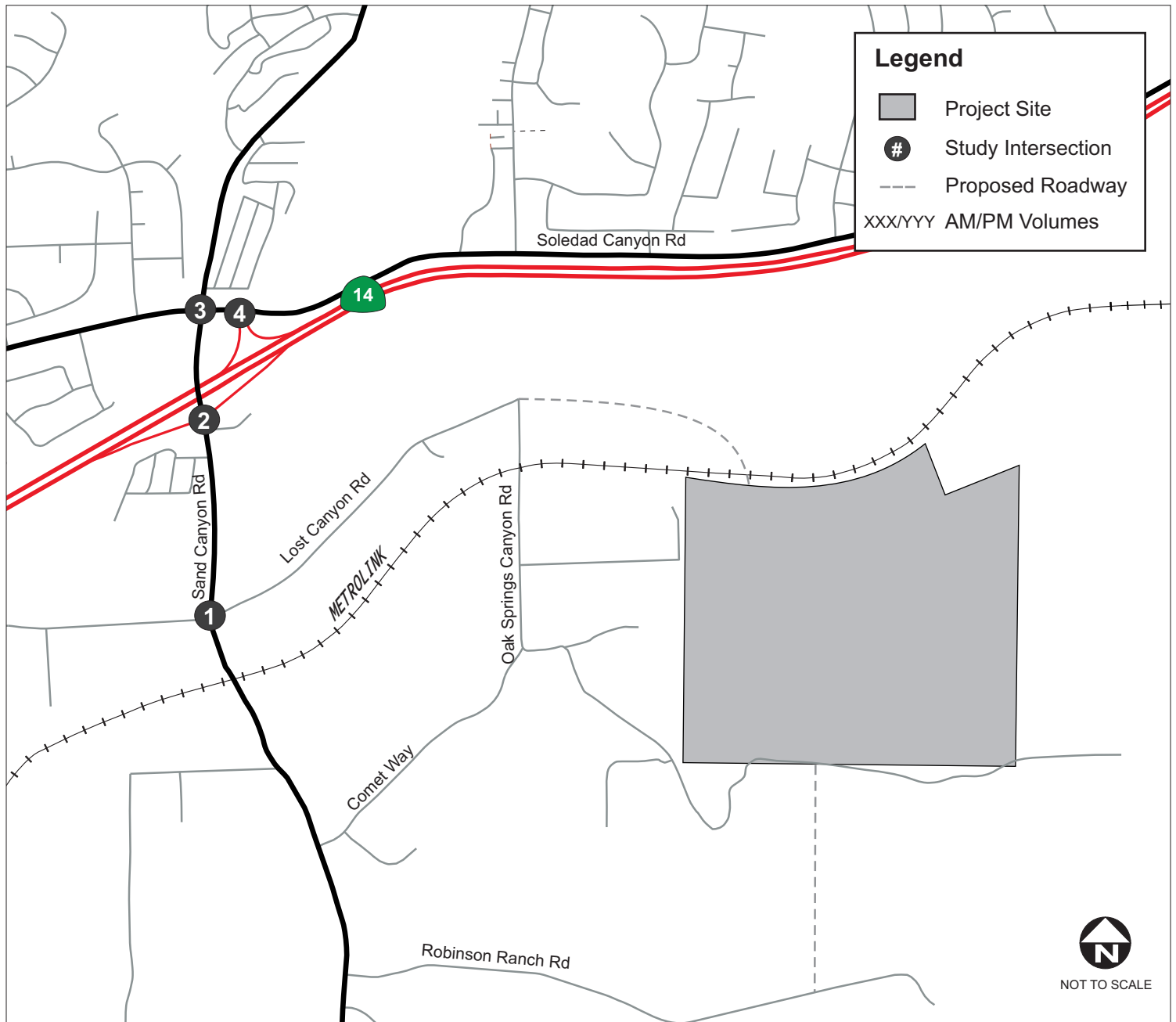
TABLE 5: FUTURE WITH PROJECT CONDITIONS

Intersection		Control	Future Without Project				Future With Project						
			AM Peak Hour		PM Peak Hour		AM Peak Hour			PM Peak Hour			Significant Impact?
			LOS	Delay	LOS	Delay	LOS	Delay	Δ Delay	LOS	Delay	Δ Delay	
1	Sand Canyon Rd at Lost Canyon Rd	AWSC	F	135.9	F	223.7	F	150.3	14.4	F	253.6	29.9	Y Y
2	Sand Canyon Rd at SR-14 NB Ramps	Signal	B	13.0	C	26.0	B	13.1	0.1	C	26.2	0.2	N N
3	Sand Canyon Rd at Soledad Canyon Rd	Signal	C	33.8	D	42.9	C	34.0	0.2	D	43.6	0.7	N N
4	SR-14 SB Ramps at Soledad Canyon Rd	Signal	C	24.9	C	25.6	C	24.8	-0.1	C	25.7	0.1	N N
Notes: HCM 2000 Operations Methodology; AWSC = all-way stop-controlled. Δ represents the difference in the average delay per vehicle between the Future With Project and the Future Without Project scenarios.													

As can be seen in **Table 5**, three of the four study intersections are expected to operate at acceptable levels of service in the AM and PM peak hours. The Sand Canyon Road at Lost Canyon Road intersection is expected to continue to operate at LOS F in both the AM and PM peak hours. Level of service analysis worksheets for this scenario are provided in **Appendix B**.

As the results in **Table 5** show, one significant impact occurs at the Sand Canyon Road at Lost Canyon Road intersection in the AM and PM peak hour.

Although the Sand Canyon Road at Lost Canyon Road intersection is expected to operate at an unacceptable levels of service before project-related traffic is assigned, the incremental increase in average delay exceeds the significant impact threshold that any intersection operating at LOS E or F experiencing a project-related increase of more than 2 seconds of delay is considered to be significantly impacted by project-related traffic.



PROPOSED MITIGATION MEASURES

To reduce the impact associated with project-related traffic to a level of insignificance, the following mitigation measure has been identified:

- **Sand Canyon Road at Lost Canyon Road** - Construction of a Single-Lane Roundabout.

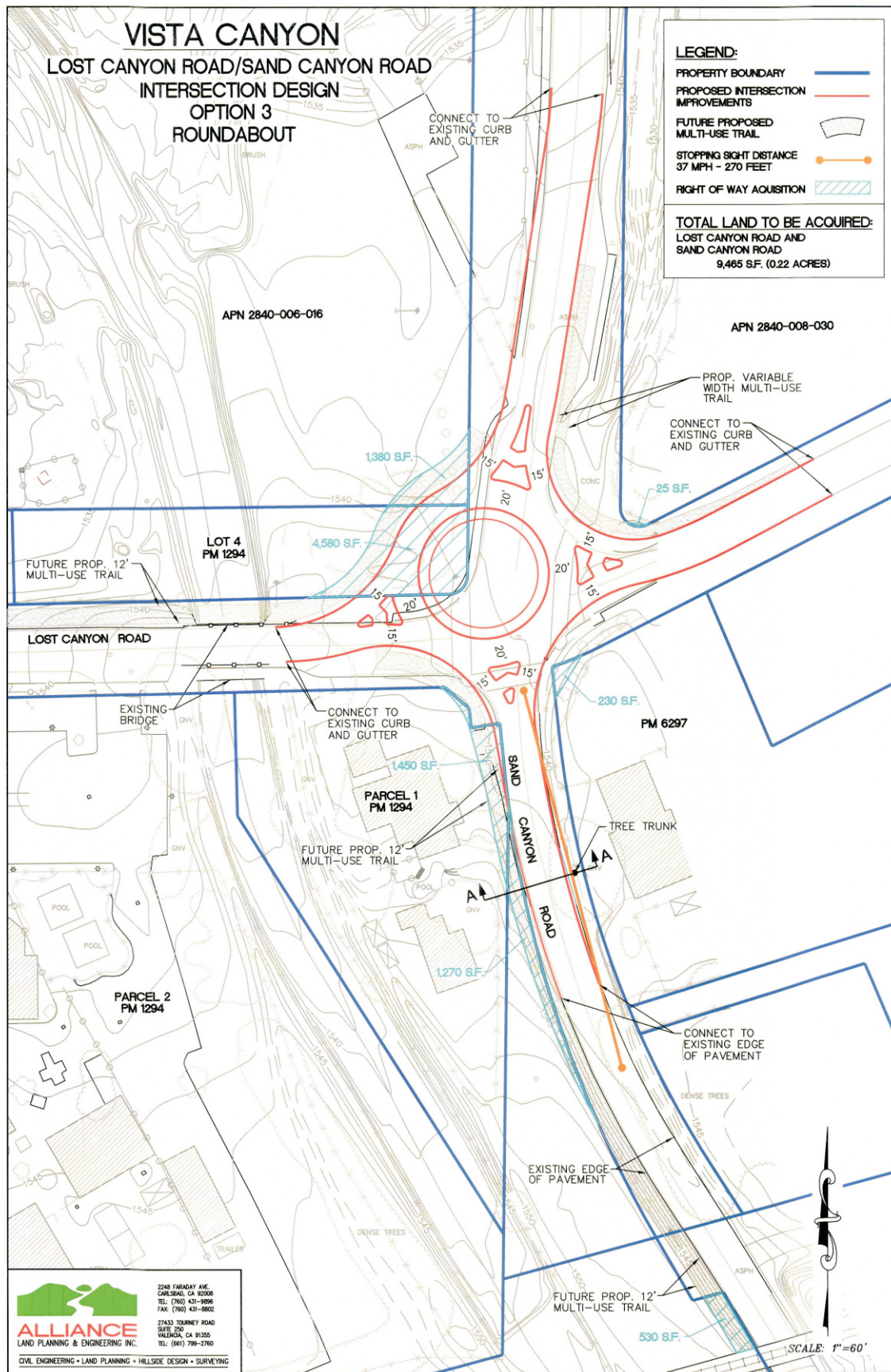
This mitigation measure was recently adopted as the preferred mitigation measure by the City of Santa Clarita Planning Commission to alleviate the impact at this intersection as a result of the Vista Canyon Final Environmental Impact Report (RBF Consulting, February 2011). The single-lane roundabout was analyzed and the results were documented in a feasibility report conducted by Roundabouts & Traffic Engineering (RTE), a traffic engineering company that specializes in roundabout design and analyses, provided in **Appendix D**. RTE's results show that the traffic operations expected at this intersection would improve significantly if a roundabout were installed as a traffic control device at this location. The level of service expected under Future With Project conditions would improve from LOS F in both peak hours to LOS A and B in the AM and PM peak hours, respectively, as shown in **Table 6**. Level of service analysis worksheets are provided in **Appendix B**.

TABLE 6: MITIGATED FUTURE WITH PROJECT CONDITIONS

Intersection		Future Without Project				Mitigated Future With Project							
		AM Peak Hour		PM Peak Hour		AM Peak Hour			PM Peak Hour			Significant Impact?	
		LOS	Delay	LOS	Delay	LOS	Delay	Δ Delay	LOS	Delay	Δ Delay		
1[a]	Sand Canyon Rd at Lost Canyon Rd	F	135.9	F	223.7	A	6.5	-129.4	B	16.2	-207.5	N	N
Notes: HCM 2000 Operations Methodology. [a] Mitigated conditions expected if single-lane roundabout were installed as a traffic control device at this location. It should be noted that this analysis was conducted using traffic volumes provided in the draft traffic impact report. These volumes have since been revised but the analysis is still considered valid since the previous traffic volumes were of greater magnitude than the revised volumes. Δ represents the difference in the average delay per vehicle between the Mitigated Future With Project and the Future Without Project scenarios.													

Due to the flexible design parameters of the roundabout, the right-of-way needs associated with the installation of a single-lane roundabout could be significantly less than those needed to install a traffic signal. In addition, intersections with roundabouts have fewer rear-end and broad-side accidents when compared to a standard traffic signal.

A conceptual plan of the single-lane roundabout is shown in **Figure 9**.



CONGESTION MANAGEMENT PROGRAM ANALYSIS

The Congestion Management Program (CMP) was created statewide as a result of Proposition 111 and has been implemented locally by the Los Angeles County Metropolitan Transportation Authority (Metro). The CMP for Los Angeles County requires that the traffic impact of individual development projects of potential regional significance be analyzed. A specific system of arterial roadways plus all freeways comprise the CMP system. A total of 164 intersections are identified for monitoring on the system in Los Angeles County. This section describes the analysis of project-related impacts on the CMP system. The analysis has been conducted according to the guidelines set forth in the 2004 Congestion Management Program for Los Angeles County.

According to the CMP Traffic Impact Analysis (TIA) Guidelines developed by Metro, a CMP traffic impact analysis is required given the following conditions:

- CMP arterial monitoring intersections, including freeway on- or off-ramps, where the proposed project would add 50 or more trips during either the AM or PM weekday peak hours.
- CMP freeway monitoring locations where the proposed project would add 150 or more trips, in either direction, during either the AM or PM weekday peak hours.

According to the CMP guidelines, a significant impact occurs when a proposed project increases traffic demand on a CMP facility by 2% of capacity ($V/C \geq 0.02$ for arterial locations or $D/C \geq 0.02$ for freeway locations), causing LOS F ($V/C > 1.00$ for arterial locations or $D/C > 1.00$ for freeway locations).

CMP Intersection Analysis

None of the proposed study area intersections are part of the 164 CMP Arterial monitoring locations. Therefore, no CMP intersection analysis was conducted in this traffic study report.

CMP Mainline Freeway Segment Analysis

The focus of this analysis is to determine whether project-related trips would significantly impact the freeway system according to CMP guidelines and threshold of significance. For purposes of analyzing the mainline freeway impact of the project, the nearest freeway monitoring stations are located at:

- Antelope Valley Freeway SR-14 south of Angeles Forest Highway (CMP Station 1023)
- Antelope Valley Freeway SR-14 north of Junction of Interstate 5 (CMP Station 1023)

It is projected that the proposed Project will add 1 AM peak hour and 3 PM peak hour southbound trips to the CMP location at Antelope Valley Freeway SR-14 south of Angeles Forest Highway, which are below the CMP analysis guideline threshold of 150 trips; therefore no further CMP analysis is required for this location.

It is projected that the proposed project will add 8 AM peak hour and 25 PM peak hour northbound trips, and 22 AM peak hour and 15 PM peak hour southbound trips to the CMP location at Antelope Valley Freeway SR-14 north of Junction of Interstate 5, which are below the CMP analysis guideline threshold of 150 trips; therefore no further CMP analysis is required for this location.

RESIDENTIAL STREET ANALYSIS

The proposed project is expected to have three points of access from Sand Canyon Road, as shown in **Figure 2**. The primary entrance will be provided by an eastward and southward extension of Lost Canyon Road from its existing terminus just east of Oak Springs Canyon Road. This facility will provide access to the proposed site for approximately 70 percent of project-related traffic. The second point of access will be provided by a new road extending northward from Robinson Ranch Road to Oak Springs Canyon Road, east of Graceton Drive. This entrance is expected to be utilized by approximately 20 percent of project-related traffic. A third point of access is provided by utilizing Oak Springs Canyon Road. This route is expected to accommodate the remaining 10 percent of project-related traffic. All project traffic utilizing Oak Springs Canyon Road will also travel along Lost Canyon Road, resulting in 80 percent of project traffic traveling along Lost Canyon Road to access Sand Canyon Road. The remaining 20 percent of project traffic will access Sand Canyon Road utilizing Robinson Ranch Road.

Daily volumes along Lost Canyon Road between Sand Canyon Road and Oak Springs Canyon Road were calculated from intersection turning movement data at the Sand Canyon Road at Lost Canyon Road intersection. Additionally, average daily traffic counts were conducted along Oak Springs Canyon Road, between Whitewater Canyon Road and Holt Avenue, on Thursday, August 21, 2008. Based on discussions with City staff, base roadway volumes along this segment have not increased since 2008 and are not expected to increase as no new developments are planned along this roadway. Traffic count sheets are provided in **Appendix E**. Existing and Future roadway volumes are provided in **Table 7**.

As shown in **Table 3**, the proposed project is expected to generate approximately 947 new daily trips with 75 and 100 vehicle trips in the AM and PM peak hours, respectively. When these volumes are assigned to the two roadway facilities providing access to the project, the resulting volumes are as shown in **Table 7**.

Based on information provided by City staff, a transportation impact on a local residential street is deemed significant if the volumes are projected to exceed 2,000 vehicles per day.

The addition of project-related traffic to Lost Canyon Road and Oak Springs Canyon Road do not cause either roadway to exceed the City's threshold of 2,000 vehicles per day for a residential street, therefore, neither roadway would be significantly impacted with the addition of project-related traffic.

TABLE 7: TRAFFIC OPERATIONS ALONG PROJECT ACCESS ROADWAYS

Roadway Facility	AM Peak Hour			PM Peak Hour			Daily
	EB/SB	WB/NB	Total	EB/SB	WB/NB	Total	
	Existing Traffic Volumes						
Lost Canyon Road	17	18	35	40	40	80	800
Oak Springs Canyon Road	5	5	10	6	11	17	123
	Future Without Project Traffic Volumes by Roadway						
Lost Canyon Road	21	46	67	71	40	111	1,152
Oak Springs Canyon Road	5	5	10	6	11	17	123
	Proposed Project Traffic Volumes by Roadway						
Lost Canyon Road	15	45	60	50	30	80	758
Oak Springs Canyon Road	2	6	8	6	4	10	95
	Future With Project Traffic Volumes by Roadway						
Lost Canyon Road	36	91	127	121	70	191	1,910
Oak Springs Canyon Road	7	11	18	12	15	27	218
	Project-Related Percent Increase in Traffic Volumes over Existing						
Lost Canyon Road	88.2	250.0	171.4	125.0	75.0	100.0	94.8
Oak Springs Canyon Road	40.0	120.0	80.0	100.0	36.4	58.8	77.2

CONCLUSIONS

The proposed project is located in the Sand Canyon region in the City of Santa Clarita. It includes a 99-unit single-family residential development, a 5-acre city park and 30 acres of open space to be utilized as an equestrian exercise area and an emergency evacuation location. The project site is located between the Metrolink / Union Pacific railroad tracks and the Robinson Ranch golf course. The primary access for the project will be from Lost Canyon Road with a second point of access from Oak Springs Canyon Road.

The proposed project is expected to generate approximately 947 new daily trips with 75 and 100 vehicle trips in the AM and PM peak hours, respectively.

The following intersections were analyzed in this study:

1. Sand Canyon Road at Lost Canyon Road;
2. Sand Canyon Road at State Route 14 Northbound Ramps;
3. Sand Canyon Road at Soledad Canyon Road; and
4. State Route 14 Southbound Ramps at Soledad Canyon Road.

All four study intersections currently operate at acceptable levels of service.

Three of the four study intersections are expected to operate at acceptable levels of service during Future Without Project conditions. The Sand Canyon Road at Lost Canyon Road intersection is expected to operate unacceptably in both the AM and PM peak hour.

Three of the four study intersections are expected to continue to operate at acceptable levels of service in the AM and PM peak hours. One significant impact occurs at the Sand Canyon Road at Lost Canyon Road intersection in the AM and PM peak hour.

To reduce the impact associated with project-related traffic to a level of insignificance, the following mitigation measure has been identified:

- **Sand Canyon Road at Lost Canyon Road** - Construction of a Single-Lane Roundabout.

The level of service expected under Future With Project conditions would improve from LOS F in both peak hours to LOS A and B in the AM and PM peak hours.

The addition of project-related traffic to Lost Canyon Road and Oak Springs Canyon Road do not cause either roadway to exceed the City's threshold of 2,000 vehicles per day for a residential street, therefore, neither roadway would be significantly impacted with the addition of project-related traffic.

Technical Appendices

Mancara at Robinson Ranch Traffic Impact Analysis City of Santa Clarita

Prepared for

Robinson Ranch Residential, LP

Prepared by

Iteris, Inc.

801 South Grand Avenue, Suite 530
Los Angeles, CA 90017

June 23, 2011

J10-1639

APPENDIX A

2010 INTERSECTION TURNING MOVEMENT TRAFFIC COUNTS

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Intersection Turning Movement

Prepared by:



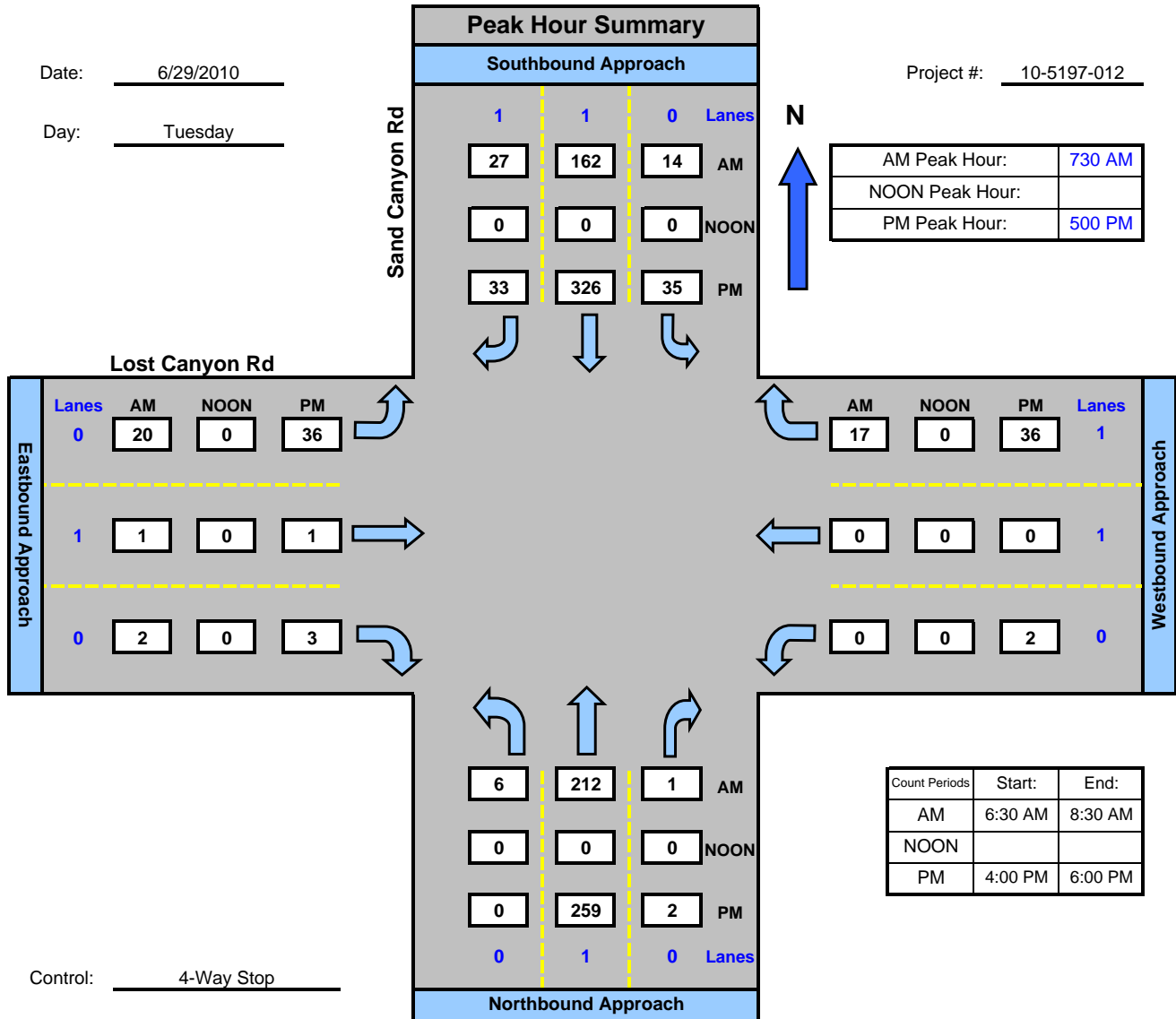
National Data & Surveying Services

Sand Canyon Rd and Lost Canyon Rd , City of Santa Clarita

Date: 6/29/2010

Day: Tuesday

Project #: 10-5197-012



Intersection Turning Movement

Prepared by:



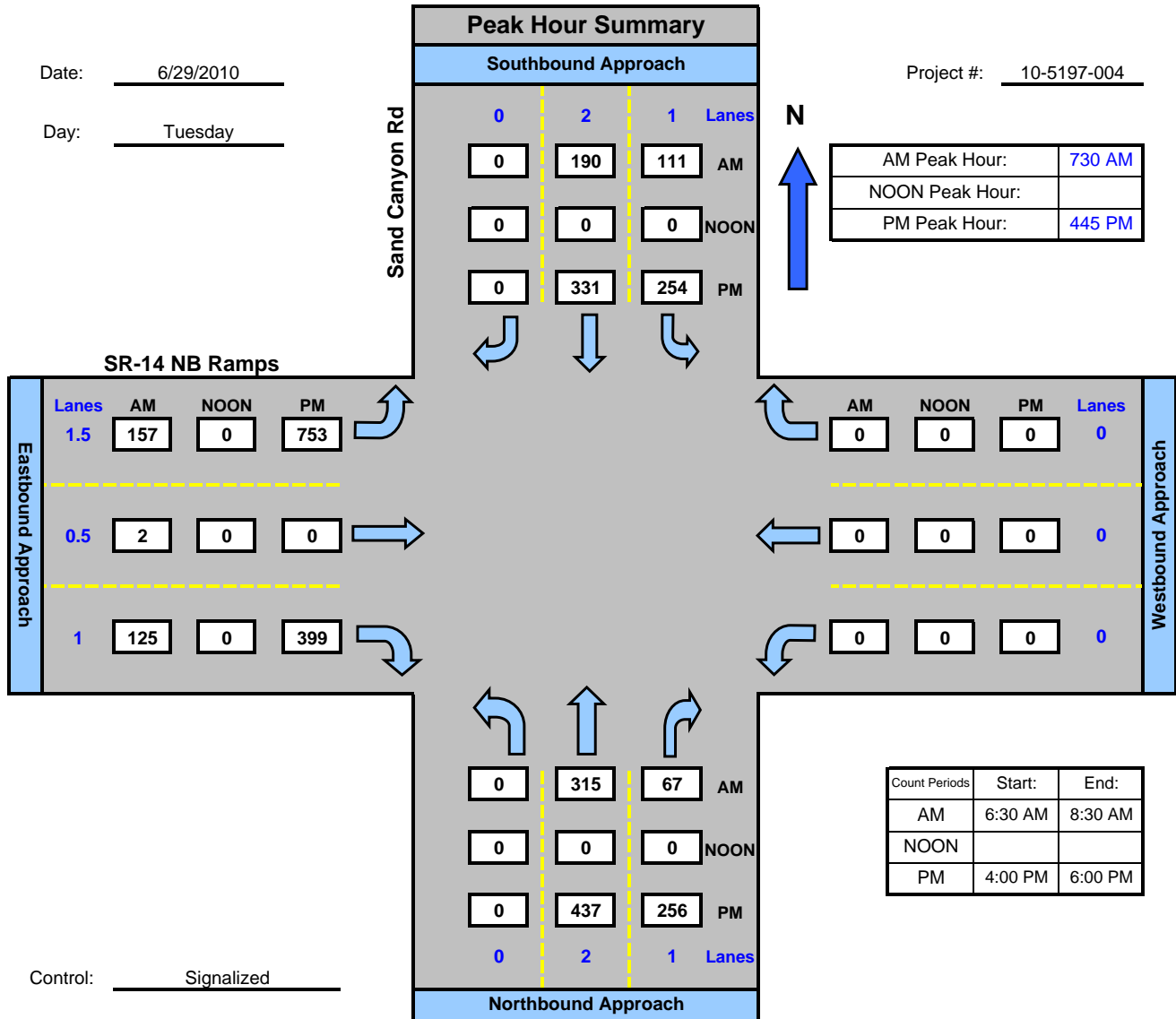
National Data & Surveying Services

Sand Canyon Rd and SR-14 NB Ramps , City of Santa Clarita

Date: 6/29/2010

Day: Tuesday

Project #: 10-5197-004



Intersection Turning Movement

Prepared by:



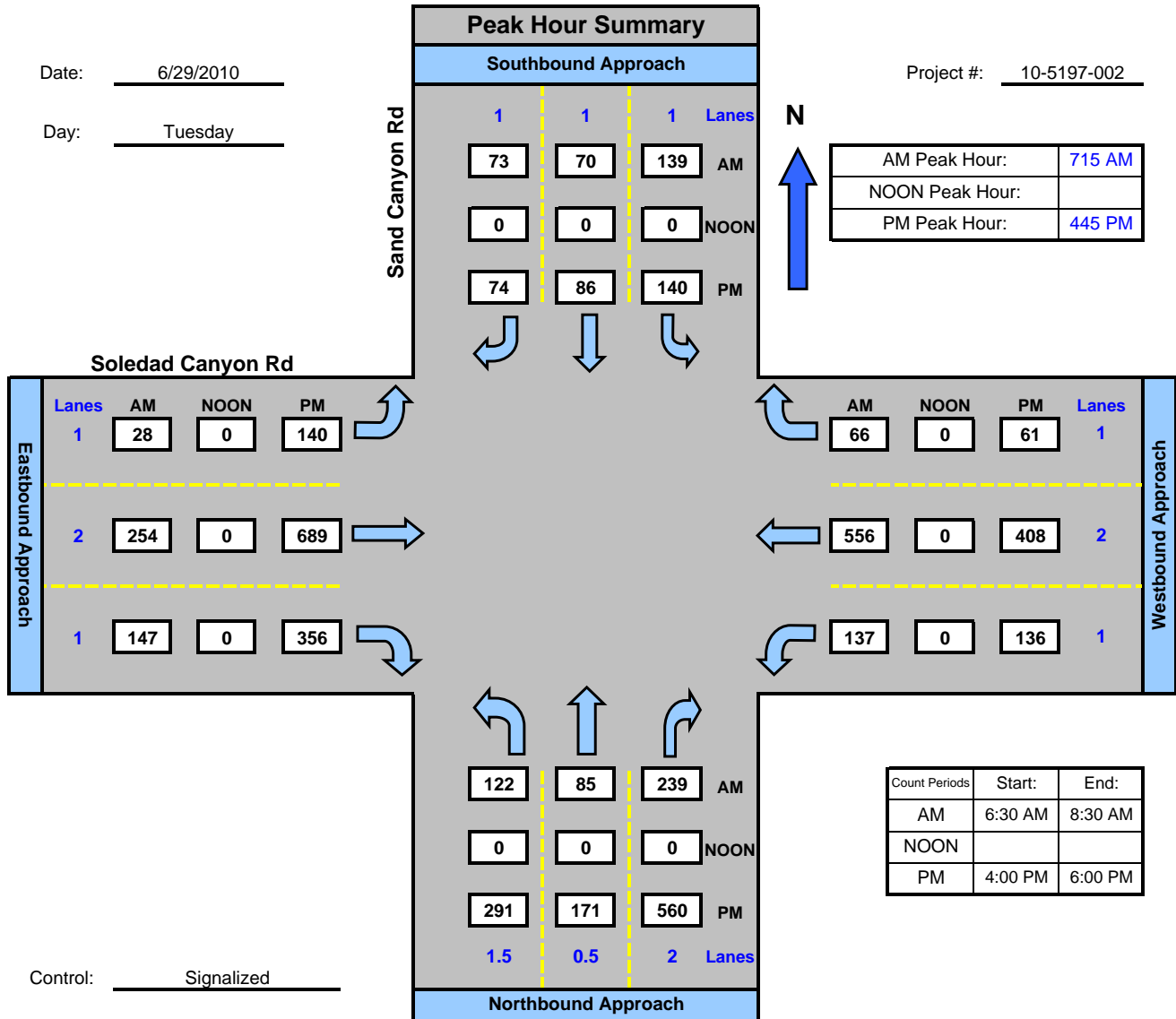
National Data & Surveying Services

Sand Canyon Rd and Soledad Canyon Rd , City of Santa Clarita

Date: 6/29/2010

Day: Tuesday

Project #: 10-5197-002



Intersection Turning Movement

Prepared by:



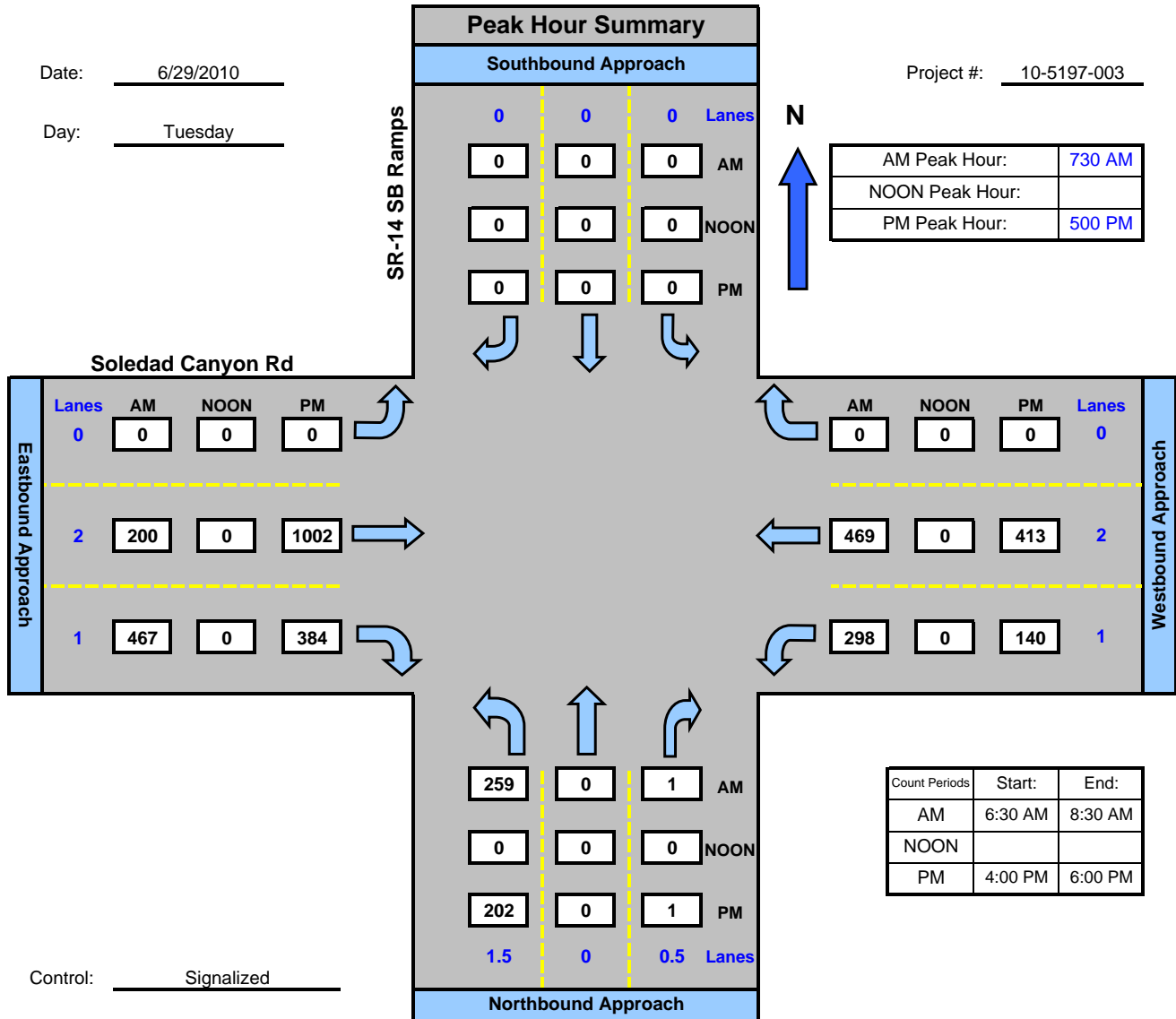
National Data & Surveying Services

SR-14 SB Ramps and Soledad Canyon Rd , City of Santa Clarita

Date: 6/29/2010

Day: Tuesday

Project #: 10-5197-003



APPENDIX B
LEVEL OF SERVICE
CALCULATION WORKSHEETS

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EXISTING CONDITIONS

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HCM Unsignalized Intersection Capacity Analysis





















1: Lost Canyon Rd & Sand Canyon Rd

Existing
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	309	1	56	0	0	18	86	223	1	15	170	378
Peak Hour Factor	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87	0.87
Hourly flow rate (vph)	355	1	64	0	0	21	99	256	1	17	195	434
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	421	0	21	356	213	434						
Volume Left (vph)	355	0	0	99	17	0						
Volume Right (vph)	64	0	21	1	0	434						
Hadj (s)	0.11	0.00	-0.67	0.09	0.07	-0.67						
Departure Headway (s)	7.4	8.8	8.1	7.4	7.2	6.4						
Degree Utilization, x	0.86	0.00	0.05	0.74	0.43	0.78						
Capacity (veh/h)	479	382	396	468	488	545						
Control Delay (s)	40.9	10.6	10.3	28.5	14.2	27.1						
Approach Delay (s)	40.9	10.3		28.5	22.9							
Approach LOS	E	B		D	C							
Intersection Summary												
Delay			29.3									
HCM Level of Service			D									
Intersection Capacity Utilization			56.9%		ICU Level of Service		B					
Analysis Period (min)			15									




























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

Existing
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1687	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.51	1.00	
Satd. Flow (perm)	1681	1687	1583					3539	1583	951	3539	
Volume (vph)	165	2	131	0	0	0	0	331	70	117	200	0
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	177	2	141	0	0	0	0	356	75	126	215	0
RTOR Reduction (vph)	0	0	127	0	0	0	0	0	21	0	0	0
Lane Group Flow (vph)	89	90	14	0	0	0	0	356	54	126	215	0
Turn Type	Perm		Perm						Perm	pm+pt		
Protected Phases		4						2		1	6	
Permitted Phases	4		4						2	6		
Actuated Green, G (s)	10.2	10.2	10.2					71.5	71.5	81.8	81.8	
Effective Green, g (s)	10.2	10.2	10.2					71.5	71.5	81.8	81.8	
Actuated g/C Ratio	0.10	0.10	0.10					0.72	0.72	0.82	0.82	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	171	172	161					2530	1132	830	2895	
v/s Ratio Prot								0.10		c0.01	0.06	
v/s Ratio Perm	0.05	0.05	0.01						0.03	c0.11		
v/c Ratio	0.52	0.52	0.09					0.14	0.05	0.15	0.07	
Uniform Delay, d1	42.6	42.6	40.7					4.5	4.2	1.9	1.8	
Progression Factor	1.00	1.00	1.00					1.00	1.00	0.65	0.64	
Incremental Delay, d2	2.8	2.9	0.2					0.1	0.1	0.1	0.0	
Delay (s)	45.4	45.4	40.9					4.6	4.3	1.3	1.2	
Level of Service	D	D	D					A	A	A	A	
Approach Delay (s)		43.5			0.0			4.6			1.2	
Approach LOS		D			A			A			A	
Intersection Summary												
HCM Average Control Delay			14.9									
HCM Volume to Capacity ratio			0.19									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			30.3%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

Existing
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 			
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1752	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1752	2787	1770	1863	1583
Volume (vph)	29	267	154	144	584	69	128	89	251	146	74	77
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	31	287	166	155	628	74	138	96	270	157	80	83
RTOR Reduction (vph)	0	0	101	0	0	38	0	0	185	0	0	72
Lane Group Flow (vph)	31	287	65	155	628	36	114	120	85	157	80	11
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	4.2	39.2	39.2	13.5	48.5	48.5	18.0	18.0	31.5	13.3	13.3	13.3
Effective Green, g (s)	4.2	39.2	39.2	13.5	48.5	48.5	18.0	18.0	31.5	13.3	13.3	13.3
Actuated g/C Ratio	0.04	0.39	0.39	0.14	0.48	0.48	0.18	0.18	0.32	0.13	0.13	0.13
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	74	1387	621	239	1716	768	303	315	878	235	248	211
v/s Ratio Prot	c0.02	0.08		c0.09	c0.18		0.07	c0.07	0.01	c0.09	0.04	
v/s Ratio Perm			0.04			0.02			0.02			0.01
v/c Ratio	0.42	0.21	0.10	0.65	0.37	0.05	0.38	0.38	0.10	0.67	0.32	0.05
Uniform Delay, d1	46.7	20.1	19.3	41.0	16.1	13.6	36.1	36.1	24.2	41.2	39.3	37.8
Progression Factor	1.00	1.00	1.00	0.92	1.32	2.07	0.81	0.81	0.50	1.00	1.00	1.00
Incremental Delay, d2	3.8	0.3	0.3	5.7	0.6	0.1	3.5	3.4	0.0	7.0	0.8	0.1
Delay (s)	50.5	20.5	19.6	43.4	21.9	28.1	32.6	32.5	12.2	48.3	40.0	38.0
Level of Service	D	C	B	D	C	C	C	C	B	D	D	D
Approach Delay (s)		22.1			26.3			21.6			43.5	
Approach LOS		C			C			C			D	
Intersection Summary												
HCM Average Control Delay			26.8			HCM Level of Service			C			
HCM Volume to Capacity ratio			0.46									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			16.0			
Intersection Capacity Utilization			44.2%			ICU Level of Service			A			
Analysis Period (min)			15									
c Critical Lane Group												



















Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

Existing
Timing Plan: AM Peak Hour

						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↑	↑↑	↑↑↑	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	1.00	
Flt Protected	1.00	1.00	0.95	1.00	0.95	
Satd. Flow (prot)	3539	1583	1770	3539	3440	
Flt Permitted	1.00	1.00	0.95	1.00	0.95	
Satd. Flow (perm)	3539	1583	1770	3539	3440	
Volume (vph)	210	490	313	492	272	1
Peak-hour factor, PHF	0.97	0.97	0.97	0.97	0.97	0.97
Adj. Flow (vph)	216	505	323	507	280	1
RTOR Reduction (vph)	0	80	0	0	0	0
Lane Group Flow (vph)	216	425	323	507	281	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	53.5	64.9	23.1	80.6	11.4	
Effective Green, g (s)	53.5	64.9	23.1	80.6	11.4	
Actuated g/C Ratio	0.54	0.65	0.23	0.81	0.11	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	1893	1091	409	2852	392	
v/s Ratio Prot	0.06	c0.04	c0.18	0.14	c0.08	
v/s Ratio Perm		0.22				
v/c Ratio	0.11	0.39	0.79	0.18	0.72	
Uniform Delay, d1	11.5	8.2	36.2	2.2	42.7	
Progression Factor	0.70	1.30	1.00	1.00	1.00	
Incremental Delay, d2	0.1	0.2	9.8	0.1	6.1	
Delay (s)	8.2	10.9	45.9	2.3	48.9	
Level of Service	A	B	D	A	D	
Approach Delay (s)	10.1			19.3	48.9	
Approach LOS	B			B	D	
Intersection Summary						
HCM Average Control Delay			20.2		HCM Level of Service	C
HCM Volume to Capacity ratio			0.54			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			54.3%		ICU Level of Service	A
Analysis Period (min)			15			
c Critical Lane Group						





















Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

Existing
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	38	1	3	2	0	38	0	272	2	37	342	35
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Hourly flow rate (vph)	40	1	3	2	0	40	0	283	2	39	356	36
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	44	2	40	285	395	36						
Volume Left (vph)	40	2	0	0	39	0						
Volume Right (vph)	3	0	40	2	0	36						
Hadj (s)	0.17	0.53	-0.67	0.03	0.08	-0.67						
Departure Headway (s)	6.5	6.9	5.7	5.3	5.1	4.4						
Degree Utilization, x	0.08	0.00	0.06	0.42	0.56	0.04						
Capacity (veh/h)	496	465	556	663	689	800						
Control Delay (s)	10.1	8.7	7.8	12.1	13.3	6.4						
Approach Delay (s)	10.1	7.9		12.1	12.7							
Approach LOS	B	A		B	B							
Intersection Summary												
Delay			12.1									
HCM Level of Service			B									
Intersection Capacity Utilization			53.5%		ICU Level of Service				A			
Analysis Period (min)			15									





























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

Existing
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1681	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.38	1.00	
Satd. Flow (perm)	1681	1681	1583					3539	1583	715	3539	
Volume (vph)	791	0	419	0	0	0	0	459	269	267	348	0
Peak-hour factor, PHF	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Adj. Flow (vph)	824	0	436	0	0	0	0	478	280	278	362	0
RTOR Reduction (vph)	0	0	307	0	0	0	0	0	160	0	0	0
Lane Group Flow (vph)	412	412	129	0	0	0	0	478	120	278	362	0
Turn Type	Perm		Perm						Perm	pm+pt		
Protected Phases		4						2		1	6	
Permitted Phases	4		4						2	6		
Actuated Green, G (s)	29.7	29.7	29.7					42.9	42.9	62.3	62.3	
Effective Green, g (s)	29.7	29.7	29.7					42.9	42.9	62.3	62.3	
Actuated g/C Ratio	0.30	0.30	0.30					0.43	0.43	0.62	0.62	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	499	499	470					1518	679	608	2205	
v/s Ratio Prot								0.14		c0.07	0.10	
v/s Ratio Perm	c0.25	0.25	0.08						0.08	c0.21		
v/c Ratio	0.83	0.83	0.28					0.31	0.18	0.46	0.16	
Uniform Delay, d1	32.7	32.7	26.9					18.8	17.6	9.1	7.9	
Progression Factor	1.00	1.00	1.00					1.00	1.00	0.90	0.84	
Incremental Delay, d2	10.7	10.7	0.3					0.5	0.6	0.5	0.1	
Delay (s)	43.4	43.4	27.2					19.4	18.2	8.7	6.8	
Level of Service	D	D	C					B	B	A	A	
Approach Delay (s)		37.8			0.0			19.0			7.6	
Approach LOS		D			A			B			A	
Intersection Summary												
HCM Average Control Delay			25.2									
HCM Volume to Capacity ratio			0.57									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			63.4%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

Existing
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 		 	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1746	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1746	2787	1770	1863	1583
Volume (vph)	147	723	374	143	428	64	306	180	588	147	90	78
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	158	777	402	154	460	69	329	194	632	158	97	84
RTOR Reduction (vph)	0	0	250	0	0	41	0	0	151	0	0	73
Lane Group Flow (vph)	158	777	152	154	460	28	255	268	481	158	97	11
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	13.5	37.7	37.7	16.0	40.2	40.2	17.0	17.0	33.0	13.3	13.3	13.3
Effective Green, g (s)	13.5	37.7	37.7	16.0	40.2	40.2	17.0	17.0	33.0	13.3	13.3	13.3
Actuated g/C Ratio	0.14	0.38	0.38	0.16	0.40	0.40	0.17	0.17	0.33	0.13	0.13	0.13
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	239	1334	597	283	1423	636	286	297	920	235	248	211
v/s Ratio Prot	c0.09	c0.22		0.09	0.13		0.15	c0.15	0.08	c0.09	0.05	
v/s Ratio Perm			0.10			0.02			0.09			0.01
v/c Ratio	0.66	0.58	0.25	0.54	0.32	0.04	0.89	0.90	0.52	0.67	0.39	0.05
Uniform Delay, d1	41.1	24.9	21.5	38.6	20.6	18.2	40.6	40.7	27.1	41.3	39.6	37.9
Progression Factor	1.00	1.00	1.00	0.91	1.36	2.36	1.02	1.02	1.23	1.00	1.00	1.00
Incremental Delay, d2	6.7	1.9	1.0	2.1	0.6	0.1	25.9	26.7	0.4	7.4	1.0	0.1
Delay (s)	47.8	26.7	22.5	37.3	28.6	43.2	67.2	68.1	33.8	48.6	40.7	38.0
Level of Service	D	C	C	D	C	D	E	E	C	D	D	D
Approach Delay (s)		27.9			32.0			49.1			43.7	
Approach LOS		C			C			D			D	
Intersection Summary												
HCM Average Control Delay			37.2			HCM Level of Service			D			
HCM Volume to Capacity ratio			0.66									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			16.0			
Intersection Capacity Utilization			62.6%			ICU Level of Service			B			
Analysis Period (min)			15									
c Critical Lane Group												

Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

Existing
Timing Plan: PM Peak Hour



















						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↑	↑↑	↑↑↑	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	1.00	
Flt Protected	1.00	1.00	0.95	1.00	0.95	
Satd. Flow (prot)	3539	1583	1770	3539	3440	
Flt Permitted	1.00	1.00	0.95	1.00	0.95	
Satd. Flow (perm)	3539	1583	1770	3539	3440	
Volume (vph)	1052	403	147	434	212	1
Peak-hour factor, PHF	0.97	0.97	0.97	0.97	0.97	0.97
Adj. Flow (vph)	1085	415	152	447	219	1
RTOR Reduction (vph)	0	106	0	0	0	0
Lane Group Flow (vph)	1085	309	152	447	220	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	63.3	74.5	13.5	80.8	11.2	
Effective Green, g (s)	63.3	74.5	13.5	80.8	11.2	
Actuated g/C Ratio	0.63	0.74	0.14	0.81	0.11	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	2240	1243	239	2860	385	
v/s Ratio Prot	c0.31	0.03	c0.09	0.13	c0.06	
v/s Ratio Perm		0.17				
v/c Ratio	0.48	0.25	0.64	0.16	0.57	
Uniform Delay, d1	9.7	4.0	40.9	2.1	42.1	
Progression Factor	0.70	0.46	1.00	1.00	1.00	
Incremental Delay, d2	0.6	0.1	5.5	0.1	2.0	
Delay (s)	7.4	1.9	46.4	2.2	44.2	
Level of Service	A	A	D	A	D	
Approach Delay (s)	5.9			13.4	44.2	
Approach LOS	A			B	D	
Intersection Summary						
HCM Average Control Delay			11.5		HCM Level of Service	B
HCM Volume to Capacity ratio			0.52			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			53.3%		ICU Level of Service	A
Analysis Period (min)			15			
c Critical Lane Group						

FUTURE WITHOUT PROJECT CONDITIONS

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















Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

No Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	360	11	108	5	23	18	136	480	5	5	390	491
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	391	12	117	5	25	20	148	522	5	5	424	534
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	521	30	20	675	429	534						
Volume Left (vph)	391	5	0	148	5	0						
Volume Right (vph)	117	0	20	5	0	534						
Hadj (s)	0.05	0.12	-0.67	0.07	0.04	-0.67						
Departure Headway (s)	7.9	9.8	9.0	7.9	7.9	7.2						
Degree Utilization, x	1.15	0.08	0.05	1.49	0.94	1.06						
Capacity (veh/h)	459	363	394	457	450	510						
Control Delay (s)	115.5	12.4	11.2	253.3	55.4	83.6						
Approach Delay (s)	115.5	12.0		253.3	71.1							
Approach LOS	F	B		F	F							
Intersection Summary												
Delay			135.9									
HCM Level of Service			F									
Intersection Capacity Utilization			97.7%		ICU Level of Service				F			
Analysis Period (min)			15									




























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

No Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1681	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.40	1.00	
Satd. Flow (perm)	1681	1681	1583					3539	1583	746	3539	
Volume (vph)	230	0	232	0	0	0	0	520	235	223	593	0
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	250	0	252	0	0	0	0	565	255	242	645	0
RTOR Reduction (vph)	0	0	221	0	0	0	0	0	83	0	0	0
Lane Group Flow (vph)	125	125	32	0	0	0	0	565	172	242	645	0
Turn Type	Perm		Perm						Perm	pm+pt		
Protected Phases		4						2		1	6	
Permitted Phases	4		4						2	6		
Actuated Green, G (s)	12.5	12.5	12.5					67.6	67.6	79.5	79.5	
Effective Green, g (s)	12.5	12.5	12.5					67.6	67.6	79.5	79.5	
Actuated g/C Ratio	0.12	0.12	0.12					0.68	0.68	0.80	0.80	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	210	210	198					2392	1070	674	2814	
v/s Ratio Prot								0.16		c0.03	0.18	
v/s Ratio Perm	c0.07	0.07	0.02						0.11	c0.26		
v/c Ratio	0.60	0.60	0.16					0.24	0.16	0.36	0.23	
Uniform Delay, d1	41.4	41.4	39.1					6.2	5.9	2.7	2.6	
Progression Factor	1.00	1.00	1.00					1.00	1.00	1.38	0.56	
Incremental Delay, d2	4.5	4.5	0.4					0.2	0.3	0.2	0.1	
Delay (s)	45.8	45.8	39.4					6.5	6.2	4.0	1.6	
Level of Service	D	D	D					A	A	A	A	
Approach Delay (s)		42.6			0.0			6.4			2.2	
Approach LOS		D			A			A			A	
Intersection Summary												
HCM Average Control Delay			13.0									
HCM Volume to Capacity ratio			0.39									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			43.3%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

No Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 			
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.98	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1741	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.98	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1741	2787	1770	1863	1583
Volume (vph)	80	651	338	326	1341	170	276	144	350	140	173	171
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	87	708	367	354	1458	185	300	157	380	152	188	186
RTOR Reduction (vph)	0	0	259	0	0	90	0	0	156	0	0	138
Lane Group Flow (vph)	87	708	108	354	1458	95	223	234	224	152	188	48
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	5.6	29.3	29.3	22.8	46.5	46.5	18.0	18.0	40.8	13.9	13.9	13.9
Effective Green, g (s)	5.6	29.3	29.3	22.8	46.5	46.5	18.0	18.0	40.8	13.9	13.9	13.9
Actuated g/C Ratio	0.06	0.29	0.29	0.23	0.46	0.46	0.18	0.18	0.41	0.14	0.14	0.14
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	99	1037	464	404	1646	736	303	313	1137	246	259	220
v/s Ratio Prot	0.05	c0.20		0.20	c0.41		0.13	c0.13	0.04	0.09	c0.10	
v/s Ratio Perm			0.07			0.06			0.04			0.03
v/c Ratio	0.88	0.68	0.23	0.88	0.89	0.13	0.74	0.75	0.20	0.62	0.73	0.22
Uniform Delay, d1	46.9	31.2	26.8	37.2	24.3	15.2	38.8	38.8	19.1	40.5	41.2	38.2
Progression Factor	1.00	1.00	1.00	0.77	0.99	1.05	0.84	0.84	0.35	1.00	1.00	1.00
Incremental Delay, d2	53.0	3.6	1.2	14.5	5.6	0.3	14.2	14.5	0.1	4.6	9.7	0.5
Delay (s)	99.8	34.9	28.0	43.2	29.6	16.2	46.9	47.2	6.7	45.1	50.9	38.7
Level of Service	F	C	C	D	C	B	D	D	A	D	D	D
Approach Delay (s)		37.6			30.8			28.7			44.9	
Approach LOS		D			C			C			D	
Intersection Summary												
HCM Average Control Delay			33.8			HCM Level of Service			C			
HCM Volume to Capacity ratio			0.80									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			12.0			
Intersection Capacity Utilization			75.4%			ICU Level of Service			D			
Analysis Period (min)			15									
c Critical Lane Group												



















Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

No Project
Timing Plan: AM Peak Hour

						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↑	↑↑	↑↑↑	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	0.99	
Flt Protected	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (prot)	3539	1583	1770	3539	3418	
Flt Permitted	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (perm)	3539	1583	1770	3539	3418	
Volume (vph)	633	498	360	992	845	60
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	688	541	391	1078	918	65
RTOR Reduction (vph)	0	60	0	0	5	0
Lane Group Flow (vph)	688	481	391	1078	978	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	29.7	63.1	24.9	58.6	33.4	
Effective Green, g (s)	29.7	63.1	24.9	58.6	33.4	
Actuated g/C Ratio	0.30	0.63	0.25	0.59	0.33	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	1051	1062	441	2074	1142	
v/s Ratio Prot	c0.19	0.15	c0.22	0.30	c0.29	
v/s Ratio Perm		0.15				
v/c Ratio	0.65	0.45	0.89	0.52	0.86	
Uniform Delay, d1	30.7	9.5	36.2	12.3	31.1	
Progression Factor	0.69	0.46	1.00	1.00	1.00	
Incremental Delay, d2	2.7	0.3	18.8	0.9	6.5	
Delay (s)	23.8	4.6	55.0	13.3	37.5	
Level of Service	C	A	E	B	D	
Approach Delay (s)	15.4			24.4	37.5	
Approach LOS	B			C	D	
Intersection Summary						
HCM Average Control Delay			24.9		HCM Level of Service	C
HCM Volume to Capacity ratio			0.80			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			73.4%		ICU Level of Service	D
Analysis Period (min)			15			
c Critical Lane Group						





















Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

No Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	194	24	67	5	15	20	43	750	10	37	620	118
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	211	26	73	5	16	22	47	815	11	40	674	128
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	310	22	22	873	714	128						
Volume Left (vph)	211	5	0	47	40	0						
Volume Right (vph)	73	0	22	11	0	128						
Hadj (s)	0.03	0.16	-0.67	0.04	0.06	-0.67						
Departure Headway (s)	7.8	9.1	8.3	7.1	7.0	6.3						
Degree Utilization, x	0.67	0.06	0.05	1.72	1.39	0.22						
Capacity (veh/h)	449	374	410	515	526	565						
Control Delay (s)	25.6	11.5	10.6	349.4	207.4	9.9						
Approach Delay (s)	25.6	11.0		349.4	177.3							
Approach LOS	D	B		F	F							
Intersection Summary												
Delay			223.7									
HCM Level of Service			F									
Intersection Capacity Utilization			104.7%	ICU Level of Service		G						
Analysis Period (min)			15									




























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

No Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1681	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.27	1.00	
Satd. Flow (perm)	1681	1681	1583					3539	1583	501	3539	
Volume (vph)	810	0	430	0	0	0	0	573	531	364	518	0
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	880	0	467	0	0	0	0	623	577	396	563	0
RTOR Reduction (vph)	0	0	208	0	0	0	0	0	368	0	0	0
Lane Group Flow (vph)	440	440	259	0	0	0	0	623	209	396	563	0
Turn Type	Perm		Perm						Perm pm+pt			
Protected Phases			4						2		1	
Permitted Phases	4		4						2		6	
Actuated Green, G (s)	31.2	31.2	31.2					36.2	36.2	60.8	60.8	
Effective Green, g (s)	31.2	31.2	31.2					36.2	36.2	60.8	60.8	
Actuated g/C Ratio	0.31	0.31	0.31					0.36	0.36	0.61	0.61	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	524	524	494					1281	573	566	2152	
v/s Ratio Prot								0.18		c0.14	0.16	
v/s Ratio Perm	c0.26	0.26	0.16						0.13	c0.28		
v/c Ratio	0.84	0.84	0.52					0.49	0.36	0.70	0.26	
Uniform Delay, d1	32.1	32.1	28.3					24.7	23.4	12.0	9.1	
Progression Factor	1.00	1.00	1.00					1.00	1.00	0.71	0.76	
Incremental Delay, d2	11.3	11.3	1.0					1.3	1.8	2.7	0.2	
Delay (s)	43.4	43.4	29.3					26.0	25.2	11.2	7.2	
Level of Service	D	D	C					C	C	B	A	
Approach Delay (s)		38.5			0.0			25.6			8.8	
Approach LOS		D			A			C			A	
Intersection Summary												
HCM Average Control Delay			26.0									
HCM Volume to Capacity ratio			0.74									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			85.5%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

No Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 			
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1748	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1748	2787	1770	1863	1583
Volume (vph)	158	973	498	295	647	140	313	197	860	150	139	107
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	172	1058	541	321	703	152	340	214	935	163	151	116
RTOR Reduction (vph)	0	0	349	0	0	92	0	0	113	0	0	100
Lane Group Flow (vph)	172	1058	192	321	703	60	270	284	822	163	151	16
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	14.1	32.0	32.0	21.6	39.5	39.5	17.0	17.0	38.6	13.4	13.4	13.4
Effective Green, g (s)	14.1	32.0	32.0	21.6	39.5	39.5	17.0	17.0	38.6	13.4	13.4	13.4
Actuated g/C Ratio	0.14	0.32	0.32	0.22	0.40	0.40	0.17	0.17	0.39	0.13	0.13	0.13
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	250	1132	507	382	1398	625	286	297	1076	237	250	212
v/s Ratio Prot	0.10	c0.30		c0.18	0.20		0.16	c0.16	0.17	c0.09	0.08	
v/s Ratio Perm			0.12			0.04			0.13			0.01
v/c Ratio	0.69	0.93	0.38	0.84	0.50	0.10	0.94	0.96	0.76	0.69	0.60	0.07
Uniform Delay, d1	40.9	33.0	26.3	37.5	22.8	19.0	41.0	41.1	26.7	41.3	40.8	37.9
Progression Factor	1.00	1.00	1.00	0.80	1.37	3.24	1.16	1.16	0.88	1.00	1.00	1.00
Incremental Delay, d2	7.6	15.0	2.2	13.0	1.1	0.3	32.8	34.3	2.3	8.0	4.1	0.1
Delay (s)	48.5	48.0	28.5	42.9	32.3	61.9	80.5	82.0	26.0	49.3	44.9	38.0
Level of Service	D	D	C	D	C	E	F	F	C	D	D	D
Approach Delay (s)		42.1			39.0			46.5			44.7	
Approach LOS		D			D			D			D	
Intersection Summary												
HCM Average Control Delay			42.9			HCM Level of Service			D			
HCM Volume to Capacity ratio			0.87									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			16.0			
Intersection Capacity Utilization			78.7%			ICU Level of Service			D			
Analysis Period (min)			15									
c Critical Lane Group												

Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

No Project
Timing Plan: PM Peak Hour



















						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↓	↑↑	↓↓↓	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	0.97	
Flt Protected	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (prot)	3539	1583	1770	3539	3379	
Flt Permitted	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (perm)	3539	1583	1770	3539	3379	
Volume (vph)	1263	509	300	527	555	120
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	1373	553	326	573	603	130
RTOR Reduction (vph)	0	46	0	0	19	0
Lane Group Flow (vph)	1373	507	326	573	714	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	43.4	67.7	20.3	67.7	24.3	
Effective Green, g (s)	43.4	67.7	20.3	67.7	24.3	
Actuated g/C Ratio	0.43	0.68	0.20	0.68	0.24	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	1536	1135	359	2396	821	
v/s Ratio Prot	c0.39	0.11	c0.18	0.16	c0.21	
v/s Ratio Perm		0.21				
v/c Ratio	0.89	0.45	0.91	0.24	0.87	
Uniform Delay, d1	26.2	7.5	38.9	6.2	36.3	
Progression Factor	0.69	0.37	1.00	1.00	1.00	
Incremental Delay, d2	4.5	0.1	25.7	0.2	9.7	
Delay (s)	22.6	2.9	64.6	6.5	46.1	
Level of Service	C	A	E	A	D	
Approach Delay (s)	16.9			27.5	46.1	
Approach LOS	B			C	D	
Intersection Summary						
HCM Average Control Delay			25.6		HCM Level of Service	C
HCM Volume to Capacity ratio			0.89			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			81.1%		ICU Level of Service	D
Analysis Period (min)			15			
c Critical Lane Group						

FUTURE WITH PROJECT CONDITIONS

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















Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

With Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	360	11	108	5	23	63	136	480	5	20	390	491
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	391	12	117	5	25	68	148	522	5	22	424	534
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	521	30	68	675	446	534						
Volume Left (vph)	391	5	0	148	22	0						
Volume Right (vph)	117	0	68	5	0	534						
Hadj (s)	0.05	0.12	-0.67	0.07	0.06	-0.67						
Departure Headway (s)	8.2	9.8	9.0	8.3	8.1	7.4						
Degree Utilization, x	1.19	0.08	0.17	1.55	1.01	1.10						
Capacity (veh/h)	443	363	395	451	446	498						
Control Delay (s)	133.1	12.5	12.7	279.2	71.8	95.3						
Approach Delay (s)	133.1	12.6		279.2	84.6							
Approach LOS	F	B		F	F							
Intersection Summary												
Delay			150.3									
HCM Level of Service			F									
Intersection Capacity Utilization			98.5%		ICU Level of Service				F			
Analysis Period (min)			15									




























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

With Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1681	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.38	1.00	
Satd. Flow (perm)	1681	1681	1583					3539	1583	705	3539	
Volume (vph)	230	0	240	0	0	0	0	562	238	223	601	0
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	250	0	261	0	0	0	0	611	259	242	653	0
RTOR Reduction (vph)	0	0	228	0	0	0	0	0	85	0	0	0
Lane Group Flow (vph)	125	125	33	0	0	0	0	611	174	242	653	0
Turn Type	Perm		Perm						Perm	pm+pt		
Protected Phases		4						2		1	6	
Permitted Phases	4		4						2	6		
Actuated Green, G (s)	12.5	12.5	12.5					67.2	67.2	79.5	79.5	
Effective Green, g (s)	12.5	12.5	12.5					67.2	67.2	79.5	79.5	
Actuated g/C Ratio	0.12	0.12	0.12					0.67	0.67	0.80	0.80	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	210	210	198					2378	1064	649	2814	
v/s Ratio Prot								0.17		c0.03	0.18	
v/s Ratio Perm	c0.07	0.07	0.02						0.11	c0.27		
v/c Ratio	0.60	0.60	0.16					0.26	0.16	0.37	0.23	
Uniform Delay, d1	41.4	41.4	39.1					6.5	6.0	2.8	2.6	
Progression Factor	1.00	1.00	1.00					1.00	1.00	1.84	0.56	
Incremental Delay, d2	4.5	4.5	0.4					0.3	0.3	0.3	0.1	
Delay (s)	45.8	45.8	39.5					6.8	6.4	5.4	1.6	
Level of Service	D	D	D					A	A	A	A	
Approach Delay (s)		42.6			0.0			6.6			2.6	
Approach LOS		D			A			A			A	
Intersection Summary												
HCM Average Control Delay			13.1									
HCM Volume to Capacity ratio			0.40									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			44.3%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

With Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 			
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.98	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1740	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.98	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1740	2787	1770	1863	1583
Volume (vph)	80	651	343	328	1341	170	290	147	375	140	174	171
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	87	708	373	357	1458	185	315	160	408	152	189	186
RTOR Reduction (vph)	0	0	264	0	0	90	0	0	155	0	0	134
Lane Group Flow (vph)	87	708	109	357	1458	95	231	244	253	152	189	52
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	5.6	29.1	29.1	23.0	46.5	46.5	18.0	18.0	41.0	13.9	13.9	13.9
Effective Green, g (s)	5.6	29.1	29.1	23.0	46.5	46.5	18.0	18.0	41.0	13.9	13.9	13.9
Actuated g/C Ratio	0.06	0.29	0.29	0.23	0.46	0.46	0.18	0.18	0.41	0.14	0.14	0.14
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	99	1030	461	407	1646	736	303	313	1143	246	259	220
v/s Ratio Prot	0.05	c0.20		0.20	c0.41		0.14	c0.14	0.05	0.09	c0.10	
v/s Ratio Perm			0.07			0.06			0.04			0.03
v/c Ratio	0.88	0.69	0.24	0.88	0.89	0.13	0.76	0.78	0.22	0.62	0.73	0.23
Uniform Delay, d1	46.9	31.4	27.0	37.1	24.3	15.2	39.0	39.1	19.1	40.5	41.3	38.3
Progression Factor	1.00	1.00	1.00	0.77	0.98	1.05	0.85	0.85	0.37	1.00	1.00	1.00
Incremental Delay, d2	53.0	3.7	1.2	14.5	5.6	0.3	15.9	16.6	0.1	4.6	9.9	0.6
Delay (s)	99.8	35.2	28.2	43.1	29.5	16.2	49.0	49.8	7.2	45.1	51.1	38.9
Level of Service	F	D	C	D	C	B	D	D	A	D	D	D
Approach Delay (s)		37.7			30.7			29.9			45.1	
Approach LOS		D			C			C			D	
Intersection Summary												
HCM Average Control Delay			34.0			HCM Level of Service			C			
HCM Volume to Capacity ratio			0.81									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			12.0			
Intersection Capacity Utilization			75.9%			ICU Level of Service			D			
Analysis Period (min)			15									
c Critical Lane Group												



















Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

With Project
Timing Plan: AM Peak Hour

						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↑	↑↑	↑↑↑	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	0.99	
Flt Protected	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (prot)	3539	1583	1770	3539	3418	
Flt Permitted	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (perm)	3539	1583	1770	3539	3418	
Volume (vph)	636	520	360	993	846	60
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	691	565	391	1079	920	65
RTOR Reduction (vph)	0	60	0	0	5	0
Lane Group Flow (vph)	691	505	391	1079	980	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	29.4	63.1	24.9	58.3	33.7	
Effective Green, g (s)	29.4	63.1	24.9	58.3	33.7	
Actuated g/C Ratio	0.29	0.63	0.25	0.58	0.34	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	1040	1062	441	2063	1152	
v/s Ratio Prot	c0.20	0.16	c0.22	0.30	c0.29	
v/s Ratio Perm		0.16				
v/c Ratio	0.66	0.48	0.89	0.52	0.85	
Uniform Delay, d1	31.0	9.7	36.2	12.5	30.8	
Progression Factor	0.70	0.47	1.00	1.00	1.00	
Incremental Delay, d2	2.8	0.3	18.8	1.0	6.2	
Delay (s)	24.4	4.9	55.0	13.5	37.0	
Level of Service	C	A	E	B	D	
Approach Delay (s)	15.6			24.5	37.0	
Approach LOS	B			C	D	
Intersection Summary						
HCM Average Control Delay			24.8		HCM Level of Service	C
HCM Volume to Capacity ratio			0.80			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			73.5%		ICU Level of Service	D
Analysis Period (min)			15			
c Critical Lane Group						





















Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

With Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Sign Control		Stop			Stop			Stop			Stop	
Volume (vph)	194	24	67	5	15	50	43	750	10	87	620	118
Peak Hour Factor	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Hourly flow rate (vph)	211	26	73	5	16	54	47	815	11	95	674	128
Direction, Lane #	EB 1	WB 1	WB 2	NB 1	SB 1	SB 2						
Volume Total (vph)	310	22	54	873	768	128						
Volume Left (vph)	211	5	0	47	95	0						
Volume Right (vph)	73	0	54	11	0	128						
Hadj (s)	0.03	0.16	-0.67	0.04	0.10	-0.67						
Departure Headway (s)	8.0	9.2	8.3	7.3	7.2	6.5						
Degree Utilization, x	0.69	0.06	0.13	1.77	1.55	0.23						
Capacity (veh/h)	439	374	410	500	502	548						
Control Delay (s)	27.0	11.5	11.3	372.5	274.4	10.2						
Approach Delay (s)	27.0	11.4		372.5	236.6							
Approach LOS	D	B		F	F							
Intersection Summary												
Delay			253.6									
HCM Level of Service			F									
Intersection Capacity Utilization			104.7%	ICU Level of Service		G						
Analysis Period (min)			15									




























Mancara Robinson Ranch
2: SR-14 NB Ramps & Sand Canyon Rd

With Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	0.95	1.00					0.95	1.00	1.00	0.95	
Frt	1.00	1.00	0.85					1.00	0.85	1.00	1.00	
Flt Protected	0.95	0.95	1.00					1.00	1.00	0.95	1.00	
Satd. Flow (prot)	1681	1681	1583					3539	1583	1770	3539	
Flt Permitted	0.95	0.95	1.00					1.00	1.00	0.25	1.00	
Satd. Flow (perm)	1681	1681	1583					3539	1583	473	3539	
Volume (vph)	810	0	455	0	0	0	0	601	533	364	543	0
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	880	0	495	0	0	0	0	653	579	396	590	0
RTOR Reduction (vph)	0	0	195	0	0	0	0	0	368	0	0	0
Lane Group Flow (vph)	440	440	300	0	0	0	0	653	211	396	590	0
Turn Type	Perm		Perm						Perm	pm+pt		
Protected Phases		4						2		1	6	
Permitted Phases	4		4						2	6		
Actuated Green, G (s)	31.2	31.2	31.2					36.4	36.4	60.8	60.8	
Effective Green, g (s)	31.2	31.2	31.2					36.4	36.4	60.8	60.8	
Actuated g/C Ratio	0.31	0.31	0.31					0.36	0.36	0.61	0.61	
Clearance Time (s)	4.0	4.0	4.0					4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0					3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	524	524	494					1288	576	552	2152	
v/s Ratio Prot								0.18		c0.15	0.17	
v/s Ratio Perm	c0.26	0.26	0.19						0.13	c0.29		
v/c Ratio	0.84	0.84	0.61					0.51	0.37	0.72	0.27	
Uniform Delay, d1	32.1	32.1	29.2					24.8	23.3	12.3	9.2	
Progression Factor	1.00	1.00	1.00					1.00	1.00	0.68	0.75	
Incremental Delay, d2	11.3	11.3	2.1					1.4	1.8	3.1	0.2	
Delay (s)	43.4	43.4	31.3					26.2	25.1	11.4	7.2	
Level of Service	D	D	C					C	C	B	A	
Approach Delay (s)		39.0			0.0			25.7			8.9	
Approach LOS		D			A			C			A	
Intersection Summary												
HCM Average Control Delay			26.2									
HCM Volume to Capacity ratio			0.75									
Actuated Cycle Length (s)			100.0									
Intersection Capacity Utilization			85.6%									
Analysis Period (min)			15									
c Critical Lane Group												







Mancara Robinson Ranch
3: Soledad Canyon Rd & Sand Canyon Rd

With Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		 			 				 			
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Lane Util. Factor	1.00	0.95	1.00	1.00	0.95	1.00	0.95	0.95	0.88	1.00	1.00	1.00
Frt	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (prot)	1770	3539	1583	1770	3539	1583	1681	1748	2787	1770	1863	1583
Flt Permitted	0.95	1.00	1.00	0.95	1.00	1.00	0.95	0.99	1.00	0.95	1.00	1.00
Satd. Flow (perm)	1770	3539	1583	1770	3539	1583	1681	1748	2787	1770	1863	1583
Volume (vph)	158	973	514	301	647	140	320	199	872	150	142	107
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	172	1058	559	327	703	152	348	216	948	163	154	116
RTOR Reduction (vph)	0	0	346	0	0	92	0	0	113	0	0	100
Lane Group Flow (vph)	172	1058	213	327	703	60	275	289	835	163	154	16
Turn Type	Prot		Perm	Prot		Perm	Split		pm+ov	Split		Perm
Protected Phases	7	4		3	8		2	2	3	6	6	
Permitted Phases			4			8			2			6
Actuated Green, G (s)	14.1	32.0	32.0	21.6	39.5	39.5	17.0	17.0	38.6	13.4	13.4	13.4
Effective Green, g (s)	14.1	32.0	32.0	21.6	39.5	39.5	17.0	17.0	38.6	13.4	13.4	13.4
Actuated g/C Ratio	0.14	0.32	0.32	0.22	0.40	0.40	0.17	0.17	0.39	0.13	0.13	0.13
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Lane Grp Cap (vph)	250	1132	507	382	1398	625	286	297	1076	237	250	212
v/s Ratio Prot	0.10	c0.30		c0.18	0.20		0.16	c0.17	0.17	c0.09	0.08	
v/s Ratio Perm			0.13			0.04			0.13			0.01
v/c Ratio	0.69	0.93	0.42	0.86	0.50	0.10	0.96	0.97	0.78	0.69	0.62	0.07
Uniform Delay, d1	40.9	33.0	26.7	37.7	22.8	19.0	41.2	41.3	26.9	41.3	40.9	37.9
Progression Factor	1.00	1.00	1.00	0.80	1.37	3.24	1.16	1.16	0.89	1.00	1.00	1.00
Incremental Delay, d2	7.6	15.0	2.5	14.4	1.1	0.3	36.0	37.6	2.5	8.0	4.5	0.1
Delay (s)	48.5	48.0	29.3	44.5	32.3	61.9	83.7	85.4	26.4	49.3	45.3	38.0
Level of Service	D	D	C	D	C	E	F	F	C	D	D	D
Approach Delay (s)		42.2			39.5			48.1			44.9	
Approach LOS		D			D			D			D	
Intersection Summary												
HCM Average Control Delay			43.6			HCM Level of Service			D			
HCM Volume to Capacity ratio			0.88									
Actuated Cycle Length (s)			100.0			Sum of lost time (s)			16.0			
Intersection Capacity Utilization			79.3%			ICU Level of Service			D			
Analysis Period (min)			15									
c Critical Lane Group												

Mancara Robinson Ranch
4: Soledad Canyon Rd & SR-14 SB Ramps

With Project
Timing Plan: PM Peak Hour



















						
Movement	EBT	EBR	WBL	WBT	NBL	NBR
Lane Configurations	↑↑	↑	↑	↑↑	↑↑↑	
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Total Lost time (s)	4.0	4.0	4.0	4.0	4.0	
Lane Util. Factor	0.95	1.00	1.00	0.95	0.97	
Frt	1.00	0.85	1.00	1.00	0.97	
Flt Protected	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (prot)	3539	1583	1770	3539	3379	
Flt Permitted	1.00	1.00	0.95	1.00	0.96	
Satd. Flow (perm)	3539	1583	1770	3539	3379	
Volume (vph)	1265	520	300	530	558	120
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	1375	565	326	576	607	130
RTOR Reduction (vph)	0	46	0	0	19	0
Lane Group Flow (vph)	1375	519	326	576	718	0
Turn Type	pm+ov		Prot			
Protected Phases	4	2	3	8	2	
Permitted Phases		4				
Actuated Green, G (s)	43.5	67.7	20.3	67.8	24.2	
Effective Green, g (s)	43.5	67.7	20.3	67.8	24.2	
Actuated g/C Ratio	0.44	0.68	0.20	0.68	0.24	
Clearance Time (s)	4.0	4.0	4.0	4.0	4.0	
Vehicle Extension (s)	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	1539	1135	359	2399	818	
v/s Ratio Prot	c0.39	0.11	c0.18	0.16	c0.21	
v/s Ratio Perm		0.22				
v/c Ratio	0.89	0.46	0.91	0.24	0.88	
Uniform Delay, d1	26.1	7.6	38.9	6.2	36.5	
Progression Factor	0.70	0.37	1.00	1.00	1.00	
Incremental Delay, d2	4.4	0.1	25.7	0.2	10.5	
Delay (s)	22.6	2.9	64.6	6.4	47.0	
Level of Service	C	A	E	A	D	
Approach Delay (s)	16.9			27.5	47.0	
Approach LOS	B			C	D	
Intersection Summary						
HCM Average Control Delay			25.7		HCM Level of Service	C
HCM Volume to Capacity ratio			0.89			
Actuated Cycle Length (s)			100.0		Sum of lost time (s)	12.0
Intersection Capacity Utilization			81.3%		ICU Level of Service	D
Analysis Period (min)			15			
c Critical Lane Group						

FUTURE WITH MITIGATED PROJECT CONDITIONS

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













Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

Mitigated With Project
Timing Plan: AM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0			4.0	4.0		4.0			4.0	4.0
Lane Util. Factor		1.00			1.00	1.00		1.00			1.00	1.00
Frt		0.97			1.00	0.85		1.00			1.00	0.85
Flt Protected		0.96			0.99	1.00		0.99			1.00	1.00
Satd. Flow (prot)		1741			1847	1583		1841			1858	1583
Flt Permitted		0.76			0.94	1.00		0.70			0.96	1.00
Satd. Flow (perm)		1373			1751	1583		1303			1787	1583
Volume (vph)	360	11	108	5	23	63	136	480	5	20	390	491
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	391	12	117	5	25	68	148	522	5	22	424	534
RTOR Reduction (vph)	0	13	0	0	0	43	0	0	0	0	0	254
Lane Group Flow (vph)	0	507	0	0	30	26	0	675	0	0	446	280
Turn Type	Perm			Perm		Perm	Perm			Perm		Perm
Protected Phases		4			8			2			6	
Permitted Phases	4			8		8	2			6		6
Actuated Green, G (s)		30.0			30.0	30.0		42.0			42.0	42.0
Effective Green, g (s)		30.0			30.0	30.0		42.0			42.0	42.0
Actuated g/C Ratio		0.38			0.38	0.38		0.52			0.52	0.52
Clearance Time (s)		4.0			4.0	4.0		4.0			4.0	4.0
Vehicle Extension (s)		3.0			3.0	3.0		3.0			3.0	3.0
Lane Grp Cap (vph)		515			657	594		684			938	831
v/s Ratio Prot												
v/s Ratio Perm		c0.37			0.02	0.02		c0.52			0.25	0.18
v/c Ratio		0.98			0.05	0.04		0.99			0.48	0.34
Uniform Delay, d1		24.8			15.9	15.9		18.7			12.0	11.0
Progression Factor		1.00			1.00	1.00		1.00			1.00	1.00
Incremental Delay, d2		35.3			0.0	0.0		31.2			1.7	1.1
Delay (s)		60.1			15.9	15.9		49.9			13.8	12.1
Level of Service		E			B	B		D			B	B
Approach Delay (s)		60.1			15.9			49.9			12.8	
Approach LOS		E			B			D			B	
Intersection Summary												
HCM Average Control Delay			34.8				HCM Level of Service			C		
HCM Volume to Capacity ratio			0.99									
Actuated Cycle Length (s)			80.0				Sum of lost time (s)			8.0		
Intersection Capacity Utilization			98.5%				ICU Level of Service			F		
Analysis Period (min)			15									
c Critical Lane Group												

Mancara Robinson Ranch
1: Lost Canyon Rd & Sand Canyon Rd

Mitigated With Project
Timing Plan: PM Peak Hour

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)		4.0			4.0	4.0		4.0			4.0	4.0
Lane Util. Factor		1.00			1.00	1.00		1.00			1.00	1.00
Frt		0.97			1.00	0.85		1.00			1.00	0.85
Flt Protected		0.97			0.99	1.00		1.00			0.99	1.00
Satd. Flow (prot)		1744			1841	1583		1855			1851	1583
Flt Permitted		0.78			0.93	1.00		0.94			0.83	1.00
Satd. Flow (perm)		1413			1735	1583		1740			1539	1583
Volume (vph)	194	24	67	5	15	50	43	750	10	87	620	118
Peak-hour factor, PHF	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
Adj. Flow (vph)	211	26	73	5	16	54	47	815	11	95	674	128
RTOR Reduction (vph)	0	14	0	0	0	41	0	1	0	0	0	44
Lane Group Flow (vph)	0	296	0	0	21	13	0	872	0	0	769	84
Turn Type	Perm			Perm		Perm	Perm			Perm		Perm
Protected Phases		4			8			2			6	
Permitted Phases	4			8		8	2			6		6
Actuated Green, G (s)		19.7			19.7	19.7		52.3			52.3	52.3
Effective Green, g (s)		19.7			19.7	19.7		52.3			52.3	52.3
Actuated g/C Ratio		0.25			0.25	0.25		0.65			0.65	0.65
Clearance Time (s)		4.0			4.0	4.0		4.0			4.0	4.0
Vehicle Extension (s)		3.0			3.0	3.0		3.0			3.0	3.0
Lane Grp Cap (vph)		348			427	390		1138			1006	1035
v/s Ratio Prot												
v/s Ratio Perm		c0.21			0.01	0.01		c0.50			0.50	0.05
v/c Ratio		0.85			0.05	0.03		0.77			0.76	0.08
Uniform Delay, d1		28.7			23.0	22.9		9.6			9.6	5.1
Progression Factor		1.00			1.00	1.00		1.00			1.00	1.00
Incremental Delay, d2		17.3			0.0	0.0		5.0			5.5	0.2
Delay (s)		46.1			23.1	23.0		14.6			15.1	5.2
Level of Service		D			C	C		B			B	A
Approach Delay (s)		46.1			23.0			14.6			13.7	
Approach LOS		D			C			B			B	
Intersection Summary												
HCM Average Control Delay			19.0				HCM Level of Service			B		
HCM Volume to Capacity ratio			0.79									
Actuated Cycle Length (s)			80.0				Sum of lost time (s)			8.0		
Intersection Capacity Utilization			104.7%				ICU Level of Service			G		
Analysis Period (min)			15									
c Critical Lane Group												

APPENDIX C

FUTURE VOLUME

DEVELOPMENT WORKSHEETS

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INTERSECTION	AM FUTURE WITH PROJECT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	136	480	5	360	11	108	20	390	491	5	23	63
2. Sand Canyon Rd and SR-14 NB Ramps	0	562	238	230	0	240	223	601	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	290	147	375	80	651	343	140	174	171	328	1341	170
4. SR-14 SB Ramps and Soledad Canyon Rd	846	0	60	0	636	520	0	0	0	360	993	0

INTERSECTION	PM FUTURE WITH PROJECT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	43	750	10	194	24	67	87	620	118	5	15	50
2. Sand Canyon Rd and SR-14 NB Ramps	0	601	533	810	0	455	364	543	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	320	199	872	158	973	514	150	142	107	301	647	140
4. SR-14 SB Ramps and Soledad Canyon Rd	558	0	120	0	1265	520	0	0	0	300	530	0

INTERSECTION	AM PROJECT TRIP ASSIGNMENT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	0	0	0	0	0	0	15	0	0	0	0	45
2. Sand Canyon Rd and SR-14 NB Ramps	0	42	3	0	0	8	0	8	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	14	3	25	0	0	5	0	1	0	2	0	0
4. SR-14 SB Ramps and Soledad Canyon Rd	1	0	0	0	3	22	0	0	0	0	1	0

INTERSECTION	PM PROJECT TRIP ASSIGNMENT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	0	0	0	0	0	0	50	0	0	0	0	30
2. Sand Canyon Rd and SR-14 NB Ramps	0	28	2	0	0	25	0	25	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	7	2	12	0	0	16	0	3	0	6	0	0
4. SR-14 SB Ramps and Soledad Canyon Rd	3	0	0	0	2	11	0	0	0	0	3	0

INTERSECTION	AM FUTURE WITHOUT PROJECT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	136	480	5	360	11	108	5	390	491	5	23	18
2. Sand Canyon Rd and SR-14 NB Ramps	0	520	235	230	0	232	223	593	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	276	144	350	80	651	338	140	173	171	326	1341	170
4. SR-14 SB Ramps and Soledad Canyon Rd	845	0	60	0	633	498	0	0	0	360	992	0

INTERSECTION	PM FUTURE WITHOUT PROJECT											
	NORTHBOUND			EASTBOUND			SOUTHBOUND			WESTBOUND		
	L	T	R	L	T	R	L	T	R	L	T	R
1. Sand Canyon Rd and Lost Canyon Rd	43	750	10	194	24	67	37	620	118	5	15	20
2. Sand Canyon Rd and SR-14 NB Ramps	0	573	531	810	0	430	364	518	0	0	0	0
3. Sand Canyon Rd and Soledad Canyon Rd	313	197	860	158	973	498	150	139	107	295	647	140
4. SR-14 SB Ramps and Soledad Canyon Rd	555	0	120	0	1263	509	0	0	0	300	527	0

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APPENDIX D

ROUNDAABOUT ANALYSIS

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ROUNABOUT FEASIBILITY REPORT

LOST CANYON ROAD / SAND CANYON ROAD SANTA CLARITA, CALIFORNIA



Prepared For:

City of Santa Clarita
&
Robinson Ranch Residential, L.P.

Prepared By:

Scott Ritchie, P.E.
Roundabouts & Traffic Engineering



AUGUST 28, 2006

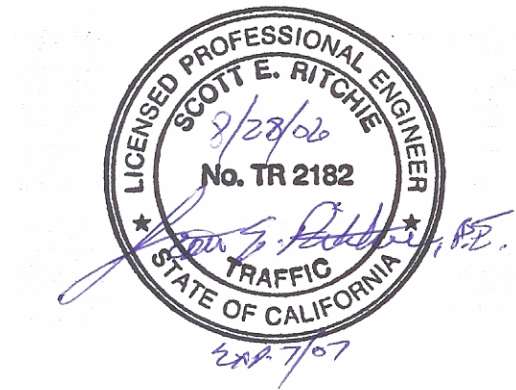
LOST CANYON / SAND CANYON ROUNDAABOUT FEASIBILITY REPORT

FOR:

**CITY OF SANTA CLARITA &
ROBINSON RANCH RESIDENTIAL, L.P.**

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August 28, 2006

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EXECUTIVE SUMMARY

Roundabouts & Traffic Engineering (RTE) has been retained by Robinson Ranch Residential, L.P. and the City of Santa Clarita to perform a Roundabout Feasibility Study at the intersection of Lost Canyon Road/Sand Canyon Road for a signal or modern roundabout project in Santa Clarita, California. The purpose of this study is to provide the feasibility of a modern roundabout as well as a comparative analysis of the operational performance of a modern roundabout versus a traffic signal at the identified intersection with a final recommendation. Comparisons between each alternative in terms of capacity and safety have been analyzed and documented for the future design year of 2010. The initial project site is located within the City's right of way at an existing unsignalized intersection. However, the City of Santa Clarita desires further investigation and consideration of a modern roundabout at this location for their known safety, capacity, and aesthetical benefits. The general conclusions of the feasibility study are provided below:

CONCLUSIONS & RECOMMENDATIONS

1. A modern roundabout is a **feasible and appropriate** traffic control device for this intersection. Due to the right-of-way constraints within the vicinity, the single lane roundabout is the preferred alternative.
2. The roundabout alternatives provide **superior capacity** over the signal alternative with respect to the overall operations, level of service, delay, and queue lengths for the intersection.
3. 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.
4. The **operational characteristics** of the roundabout are superior to the traffic signal. This includes adjacent access operations and emergency vehicle operations.
5. The roundabout and signal will both require **additional right-of-way** under in order to be constructed. The single lane roundabout has the least amount of impacts.
6. The roundabout would **enhance the character** of the City of Santa Clarita near the study intersection with added landscaping and potential ornamental statues for public appeal.

It was determined by all the contributing factors within the study that the roundabout is the identified alternative recommended for this intersection. Please refer to Chapter VII for additional conclusions and recommendations.

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I. INTRODUCTION

BACKGROUND

The City of Santa Clarita has an existing four-way stop controlled intersection at Lost Canyon Road / Sand Canyon Road located on the east side of the City of Santa Clarita, California. Roundabouts & Traffic Engineering (RTE) has been retained by Robinson Ranch Residential, L.P. and the City of Santa Clarita to perform a Roundabout Feasibility Study at the intersection for this new potential roundabout project. The intersection will either need to be signalized or be controlled by a modern roundabout. The City of Santa Clarita desires further investigation and consideration of a modern roundabout at this location for their known safety and capacity benefits. The City has also requested general information on roundabouts and supporting evidence of the safety comparisons of traffic signals and modern roundabouts. The intersection location is shown in [Figure 1](#) in an aerial photograph of the existing area.

PURPOSE

The purpose of this study is to provide a comparative analysis of the operational performance of a modern roundabout versus a traffic signal at the identified intersection with a final recommendation. A comparison between each alternative in terms of capacity and safety has been analyzed and documented for the future design year of 2010, as specified by the City. In addition, this report will determine if the proposed intersection is a viable location for a modern roundabout, depending on the information provided by the City of Santa Clarita, with a provided conceptual roundabout design (not for construction). This report documents the existing and future traffic conditions and a recommended alternative for the intersection.

ORGANIZATION

This Roundabout Feasibility Report is organized into the following chapters:

- I. Introduction
- II. Existing Site Conditions
- III. Traffic Volumes & Future Assumptions
- IV. Capacity Analyses & Conceptual Roundabout Design Alternative
- V. Capacity Comparisons
- VI. Safety Comparisons
- VII. Conclusions & Recommendations

VIII. Appendix

The report begins with the identification of the existing site conditions for the intersection. The next chapter of the report identifies the future conditions and 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.

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

Figure 1: Project Area (Aerial Photo)



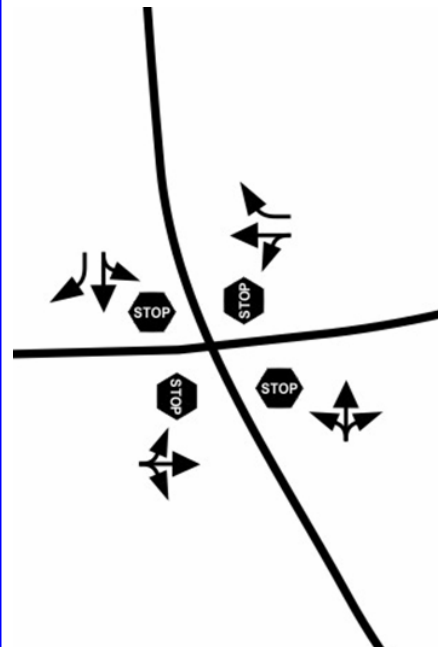
II. EXISTING SITE CONDITIONS

SURROUNDING AREA CONDITIONS

There is a need for a higher-level traffic control device at the intersection of Lost Canyon Road / Sand Canyon Road due to capacity concerns in 2010 and the impacts of surrounding development. Since the identified intersection has skewed approaches, significant right-of-way constraints, is considered a high visibility location for the City, and there is a desire to retain the rural character of the area, the City prefers the construction of a modern roundabout at the intersection as an alternative to a traffic signal.

The land uses surrounding the intersection area are zoned residential and currently consist of residential dwellings on all quadrants with the exception of the northwest quadrant as open land. 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 roadway's surrounding topography, centerlines, striping, edge of pavement, environmental, and right of way constraints were also reviewed from the information provided.

The existing intersection is a relatively old intersection that provides a primary access point to SR 14 and the City of Santa Clarita. The image below shows the existing lane configuration and surrounding area conditions for the intersection (Figure 2). As shown, there is a southbound right turn lane and a westbound right turn lane (flared). The other approaches are shared single lane.



Reviewing the *Traffic Impact Study*, the report documented the following traffic engineering elements pertaining to the City of Santa Clarita's Lost Canyon Road / Sand Canyon Road intersection and this feasibility report's needs.

- Existing year 2005 traffic volumes
- Forecast year 2010 traffic volumes (City model based)
- Calculated LOS Analyses for the intersection (existing and future)

In summary, the existing peak hour turning movement traffic volumes for the intersection were performed in November 2005 with a current LOS of E and F during the AM and PM peak hours, respectively and a future LOS F in both AM and PM peak hours. Signal warrants are met at this intersection as well. However, although the traffic report recommended mitigation for this intersection, it also states the following:

"The existing physical environment surrounding this intersection precludes any intersection improvements without adversely affecting the surrounding property owners. Currently, there are numerous obstacles, both physical and political, that must be overcome to implement any improvements to this intersection:

- *High-voltage powerlines that extend across both the southbound and eastbound approaches;*
- *A Heritage Oak tree located approximately 150 feet south of the intersection and immediately adjacent to Sand Canyon Road;*
- *A two-lane bridge is currently located approximately 150 feet west of the intersection and provides access across the Sand Canyon Creek to two area schools and several households; and*
- *A drainage culvert is located in the northeastern quadrant of the intersection.*

The extremely limited right-of-way and the enormous cost of any possible physical improvements combined with the local resident's adversity to the installation of traffic signal make the impact at this intersection impossible to mitigate. Therefore, the incremental impact created by the proposed project at the Sand Canyon Road at Lost Canyon Road intersection is considered to be an unmitigated project-related impact."

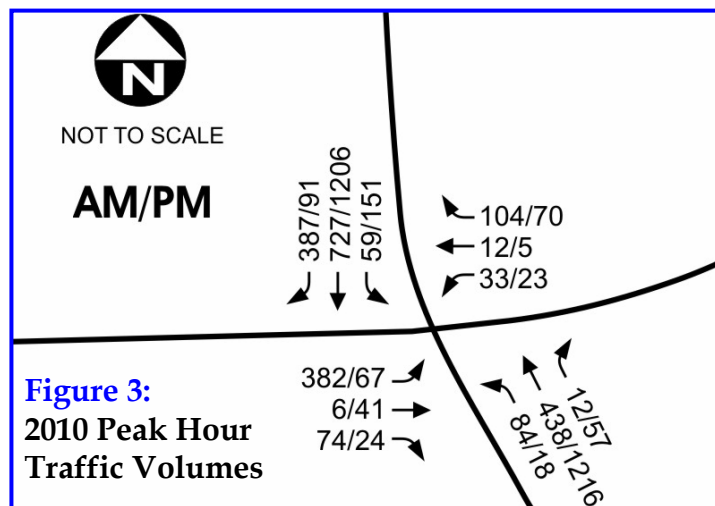
Although RTE agrees with some of the above statements pertaining to existing design constraints, RTE believes that a compromised solution exists as long as the City can be flexible with the constraints listed above, such as the drainage culvert location.

III. TRAFFIC VOLUMES

FUTURE TRAFFIC ASSUMPTIONS & DESIGN VOLUMES

As part of the selected design team for the Robinson Ranch Residential Development project, Meyer Mohaddes Associates completed a traffic study for the Robinson Ranch Residential development dated February 28, 2006. The *Traffic Impact Study* includes the study intersection in the report and identifies future traffic volumes and assumptions included in the analyses. The City of Santa Clarita has accepted the report and recommended Figure 10 in the traffic study for the appropriate 2010 design volumes to use in the analyses herein. Please refer to the original traffic report for further clarification and specific reference material.

Based on data from the City's traffic model, future projected traffic volumes were established and verified by City of Santa Clarita Staff. The weekday AM and PM peak hour traffic volumes provided are shown in **Figure 3**, which is also shown in Figure 10 of the original traffic report.



No specific design vehicle was identified in the *Traffic Impact Study* report. However, the City specified a Class A Emergency Response Fire Truck for all turning movements, similar to the illustration provided in **Figure 4**.

The heavy truck percentages for the intersection were assumed to be a typical percentage of 1.5% for all legs of the intersection. A 1.5% volume of trucks for all approaches of the intersection is considered low in traffic terms. Hence, the roundabout will be designed primarily for passenger cars and local fire trucks. In addition, RTE assumed a secondary larger design vehicle of a WB-50 for all turning movements to ensure truck traffic could negotiate the proposed conceptual roundabout.



IV. CAPACITY ANALYSES & CONCEPTUAL DESIGN ALTERNATIVES

Following 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 existing roadway network. No nearby access locations were identified to incorporate into the design with the exception of the existing driveways near the quadrants of the 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 Manual¹ 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. The City of Santa Clarita's threshold policy refers to the V/C ratio from specific project impacts.

¹ Highway Capacity Manual, Transportation Research Board, 2000

SIGNAL CAPACITY ANALYSES

After obtaining and reviewing all of the pertinent information regarding the roadways, site, and traffic volumes, an analysis of the existing signal at the intersection using the software program *Traffix For Windows* was conducted to analyze the capacity requirements of the signalized intersection for the design year of 2010. *Traffix* is based upon the HCM 2000 methodologies described above. Although it was clearly evident that the PM peak hour volumes were substantially higher than the AM peak hour volumes, both the AM and PM peak hours were analyzed to ensure adequate signal operations during all future 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 5** and **6**. Please note that the lane configurations are set a minimum for a signal layout. The figures below provide the following results for the 2010 design year:

- Lane Configurations
- Anticipated Queue Lengths of Each Lane
- Peak Hour Signal Timing
- Peak Hour Cycle Lengths
- LOS Results for AM & PM Peak Hours

As shown in the figures below, the signalized intersection operates at a LOS D with 38.6 seconds of delay in the AM and a LOS D with 44.0 seconds of delay in the PM peak hours of 2010. Also shown, the lane configurations of each approach of the intersection are as follows:

Signalized Intersection #1 "Lost/Sand" (LOS Mitigate)

Method: 2000 HCM Operations | Calc On Field Change | Calculate

Cycle Time: 125 | Loss Time: 16 | Optimize | Scenario: 2010 Signal AM

Improvement: Install Traffic Signal | Cost (in millions): 0.000

Approach	Northbound					Southbound					Eastbound					Westbound				
	L	-	T	-	R	L	-	T	-	R	L	-	T	-	R	L	-	T	-	R
Lanes	1	0	0	1	0	1	0	0	1	0	1	0	0	1	0	1	0	0	1	0
LTR	0					0					0					0				
Control,RT	Protected					Protected					Protected					Protected				
Min Green	0					0					0					0				
Base Vol	84					59					67					33				
G/C	0.0000					0.0000					0.0000					0.0000				
Volume	88					62					71					35				
Lanes	1.00					1.00					1.00					1.00				
Saturation	1823					1823					1787					1841				
Vol/Sat	0.05					0.03					0.04					0.02				
CriticalMove	*****					*****					*****					*****				
Green/Cycle	0.05					0.09					0.04					0.04				
Volume/Cap	0.92					0.38					0.92					0.47				
Delay	126.4					54.9					136.8					63.4				
HCM2kAvg	6					3					5					2				
Critical Volume/Capacity	0.924					Average Delay					38.6					Level Of Service				
Calculated Cycle Length	125 seconds					Optimal Cycle Length					146 seconds					Error Count				
Future Alternative	View Log					View Report					Error Level					Normal				
Average Delay	Calc Unentered G/C					Edit Volume					Edit Geometry									
Reset										OK					Cancel					

Figure 5: Sand Canyon / Lost Canyon Capacity Analyses - 2010 AM Peak Hour

Signalized Intersection #1 "Lost/Sand" (LOS Mitigate)

Method: 2000 HCM Operations | Calc On Field Change | Calculate

Cycle Time: 170 | Loss Time: 16 | Optimize | Scenario: 2010 Signal PM

Improvement: Install Traffic Signal | Cost (in millions): 0.000

Approach	Northbound			Southbound			Eastbound			Westbound		
	L	-	R	L	-	R	L	-	R	L	-	R
Movement	L	-	R	L	-	R	L	-	R	L	-	R
Lanes	1	0	0	1	0	0	1	0	0	1	0	0
LTR												
Control,RT	Protected		Incl	Protected		Incl	Protected		Incl	Protected		Incl
Min Green	0	0	0	0	0	0	0	0	0	0	0	0
Base Vol	18	1216	57	151	1206	91	67	41	24	23	5	70
G/C	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Volume	19	1280	60	159	1269	96	71	43	25	24	5	74
Lanes	1.00	0.96	0.04	1.00	0.93	0.07	1.00	0.63	0.37	1.00	0.07	0.93
Saturation	1823	1820	85	1823	1767	133	1787	1121	656	1841	111	1556
Vol/Sat	0.01	0.70	0.70	0.09	0.72	0.72	0.04	0.04	0.04	0.01	0.05	0.05
CriticalMove		*****			*****			*****			*****	
Green/Cycle	0.01	0.73	0.73	0.09	0.80	0.80	0.04	0.07	0.07	0.02	0.05	0.05
Volume/Cap	0.89	0.97	0.97	0.97	0.89	0.89	0.97	0.58	0.58	0.58	0.97	0.97
Delay	229.7	38.6	38.6	137.7	18.6	18.6	175.7	83.8	83.8	100.4	169.4	169.4
HCM2kAvg	2	69	76	12	51	51	6	4	4	2	6	7

Critical Volume/Capacity: 0.968 | Average Delay: 44.0 | Level Of Service: D

Calculated Cycle Length: 170 seconds | Optimal Cycle Length: 180 seconds | Error Count: 1

Future Alternative: | View Log | View Report | Error Level: Normal

Average Delay: | Calc Unentered G/C | Edit Volume | Edit Geometry

Reset | [Traffic Signal Icons] | OK | Cancel

Figure 6: Sand Canyon / Lost Canyon Capacity Analyses - 2010 PM Peak Hour

- Northbound: Left Turn Lane, Shared Through/Right Lane
- Southbound: Left Turn Lane, Through Lane, Right Turn Lane
- Eastbound: Left Turn Lane, Shared Through/Right Lane
- Westbound: Left Turn Lane, Shared Through/Right Lane

It should be noted the capacity analyses for the signal reflect two lane approaches on all four quadrants of the intersection with the needed left turn lanes. This will require significant right of way takes at the intersection. In addition, significant turn lane lengths are required for most of the lanes at the intersection in 2010. The extension of the turn lanes will require significant roadway widening and right-of-way for the required improvements. This most likely will have a detrimental impact on adjacent properties. An additional through lane would be strongly recommended in the northbound and southbound directions due to the higher PM peak hour volumes. This would equate to a five lane cross section northbound/southbound. However, in order to make the signal comparable to the roundabout with respect to the number of northbound and southbound lanes, no additional lanes were added at this time.

The capacity results are documented in [Table 1](#) as well as in the comparison analyses between the signals and roundabouts at the end of the next chapter following the roundabout capacity analyses.

Table 1: Signalized Intersection Capacity Summary					
Lost Canyon Rd / Sand Canyon Rd					
Peak Hour Results - 2010					
Intersection	Approach				Intersection LOS
	NB	SB	EB	WB	
<u>AM PEAK HOUR</u>					
DELAY (in seconds)	28.1	28.8	97.6	101.7	38.6
LOS	C	C	F	F	D
AVG QUEUE (ft / lane)	200'	1050'	125'	200'	
<u>PM PEAK HOUR</u>					
DELAY (in seconds)	41.3	31.0	130.4	153.2	44.0
LOS	D	C	F	F	D
AVG QUEUE (ft / lane)	1900'	1275'	150'	175'	
Source: RTE					
Lost-Sand Feasibility Tables.xls					

ROUNDBOUT 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 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 AM and PM 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 (AM and PM) 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 AM and PM 2010 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 allows for 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 minutes of the peak hour.

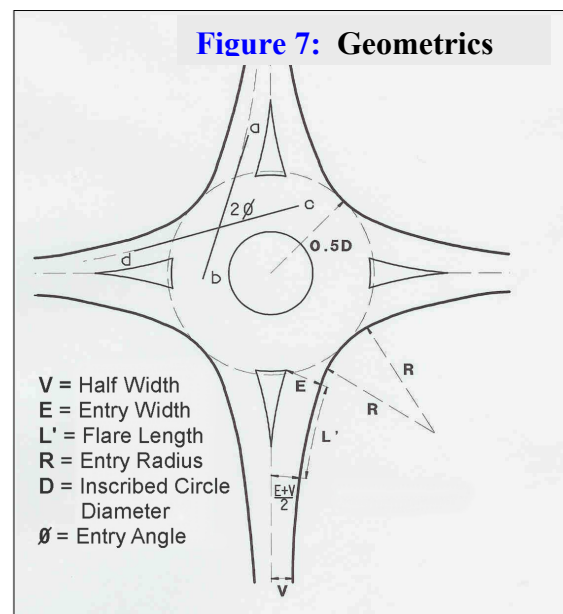
Roundabout Capacity Methodology: The predominant consideration in roundabout capacity analyses is the volume of the circulating traffic and the volume of the entering traffic on an approach. Traffic entering a roundabout will look for gaps in the circulating traffic in order to enter the roundabout. This behavior is called gap seeking. In addition to gap seeking, the geometric 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 Manual² 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 7](#) 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.

² Highway Capacity Manual, Transportation Research Board, 2000

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 (\emptyset) - 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

traffic volumes. The results of the RODEL analyses are shown in the following RODEL output in **Figures 8 and 9**:

Figure 8: AM RODEL ANALYSES: Two Lane Rbt and 1.5% Trucks

Shortcut to RODEL												
8:8:06 LostCyn/SandCyn, 2010, Santa Clarita, CA 17												
E	<m>	7.92	5.00	7.92	5.00	TIME PERIOD		min	90			
L'	<m>	35.00	6.00	25.00	6.00	TIME SLICE		min	15			
U	<m>	4.26	4.26	4.26	4.26	RESULTS PERIOD		min	15	75		
RAD	<m>	28.00	28.00	28.00	28.00	TIME COST		\$/hr	15.00			
PHI	<d>	20.00	20.00	20.00	20.00	FLOW PERIOD		min	15	75		
DIA	<m>	45.72	45.72	45.72	45.72	FLOW TYPE		pcu/veh	VEH			
GRAD SEP		0	0	0	0	FLOW PEAK		am/op/pm	AM			
LEG NAME	PCU	VEH TURNS <1st exit, 2nd..U>				FLOF	CL	FLOW RATIO			FLOW TIME	
LEG1	1.01	387	727	59	0	1.00	50	0.75	1.125	0.75	15	45 75
LEG2	1.01	382	6	74	0	1.00	50	0.75	1.125	0.75	15	45 75
LEG3	1.01	12	438	84	0	1.00	50	0.75	1.125	0.75	15	45 75
LEG4	1.01	104	12	33	0	1.00	50	0.75	1.125	0.75	15	45 75
MODE 2												
FLOW	veh	1173	462	534	149	AVEDEL		s	4.0			
CAPACITY	veh	2107	1012	2023	1147	LOS		SIG	A			
AUE DELAY	mins	0.06	0.11	0.04	0.06	LOS		UNSIG	A			
MAX DELAY	mins	0.09	0.16	0.05	0.08	VEHIC HRS			2.6			
AUE QUEUE	veh	1	1	0	0	COST		\$	39			
MAX QUEUE	veh	2	1	0	0							
F1mode F2direct F3peak CtrlF3rev F4fact F6stats F8econ F9prnt F10run Esc												

Figure 9: PM RODEL ANALYSES: Two Lane Rbt and 1.5% Trucks

Shortcut to RODEL														
8:8:06		LostCyn/SandCyn, 2010, Santa Clarita, CA										15		
E	<m>	7.92	5.00	7.92	5.00	TIME PERIOD				min	90			
L'	<m>	35.00	6.00	25.00	6.00	TIME SLICE				min	15			
U	<m>	4.26	4.26	4.26	4.26	RESULTS PERIOD				min	15 75			
RAD	<m>	28.00	28.00	28.00	28.00	TIME COST				\$/hr	15.00			
PHI	<d>	20.00	20.00	20.00	20.00	FLOW PERIOD				min	15 75			
DIA	<m>	45.72	45.72	45.72	45.72	FLOW TYPE				pcu/veh	VEH			
GRAD SEP		0	0	0	0	FLOW PEAK				am/op/pm	PM			
LEG NAME		PCU	VEH TURNS <1st exit, 2nd..U>			FLOF	CL	FLOW RATIO			FLOW TIME			
LEG1		1.01	91	1206	151	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG2		1.01	24	41	67	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG3		1.01	57	1216	18	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG4		1.01	70	5	23	0	1.00	50	0.75	1.125	0.75	15	45	75
MODE 2														
FLOW		veh	1448	132	1291	98	AVEDEL				s	5.5		
CAPACITY		veh	2169	672	1936	720	LOS				SIG	A		
AVE DELAY		mins	0.09	0.11	0.10	0.10	LOS				UNSIG	A		
MAX DELAY		mins	0.13	0.16	0.15	0.14	VEHIC HRS					4.5		
AVE QUEUE		veh	2	0	2	0	COST				\$	68		
MAX QUEUE		veh	3	0	3	0								
F1mode F2direct F3peak CtrlF3rev 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 preliminary results above. The analyses provide the following results for the 2010 design year:

- Required Lane Configurations
- Anticipated Queue Lengths of Each Approach
- Roundabout Geometry (E, L', R, Phi, D)
- LOS Results for AM and PM Peak Hours

As shown in [Figures 8](#) and [9](#), the intersection operates at a LOS A with 4.0 seconds of delay in the AM and LOS A with 5.5 seconds of delay in the PM peak hours of 2010. The recommended minimum roundabout diameter for the two-lane design is 150 feet. However, a single lane roundabout could also be used in this location with a worse level of service (this is discussed in a moment). Also shown, the required lane configurations of each approach of the intersection are based on the turning movement conflicts and the approaching existing roadway configuration. Based, on the provided traffic volumes, the minimum required number of lanes per approach to meet these traffic volumes with an excellent LOS is as follows:

- Northbound: Two Lane Approach with Single Left Turn
- Southbound: Two Lane Approach with Single Left Turn
- Eastbound: Single Lane Approach
- Westbound: Single Lane Approach

In addition, relatively insignificant stopped queue lengths develop at the intersection. This is documented in [Table 2A](#) below as well as in the comparison analyses between the signals and roundabouts at the end of the next chapter following these capacity analyses.

As noted above, the roundabout requires two lane approaches northbound and southbound and single lane approaches eastbound and westbound, which will require significant right of way takes at the intersection. The roundabout itself as well as the two lane approaches will require significant roadway widening and right-of-way for the required improvements to function properly. RTE has developed a conceptual roundabout design for this intersection that will satisfy the capacity requirements at a LOS A, which is shown in [Figure 12](#).

However, it should also be noted that this roundabout just exceeds the capacity requirements for a single lane roundabout. A single lane roundabout would have significantly less right-of-way takes associated with the design implementation. The City should thoroughly review the 2010 traffic volumes to


Table 2A: Roundabout Intersection Capacity Summary					
Lost Canyon Rd / Sand Canyon Rd					
Peak Hour Results - 2010 (TWO LANE ROUNDABOUT)					
Intersection	Approach				Intersection LOS
	SB	EB	NB	WB	
<u>AM PEAK HOUR</u>					
DELAY (in seconds)	3.6	6.6	2.4	3.6	4.0
LOS	A	A	A	A	A
AVG QUEUE (ft / lane)	25'	25'	0'	0'	
<u>PM PEAK HOUR</u>					
DELAY (in seconds)	5.4	6.6	6.0	6.0	5.5
LOS	A	A	A	A	A
AVG QUEUE (ft / lane)	50'	0'	50'	0'	
Source: RTE			Lost-Sand Feasibility Tables.xls		

confirm their validity and ensure they have not been over-inflated in the future years. RTE questions the growth rate and validity of these volumes since they are significantly higher than the existing volumes shown in the traffic report.

A single lane roundabout would be preferred for this intersection due to the sheer nature of the residential area with little available right-of-way, the existing bridges, the river, and the relocation of utility poles. Although development of multiple roundabout scenarios is out of scope for this report, RTE used the hours from other sections of the scope to develop a rough sketch of a potential single lane roundabout design as well as a two-lane roundabout design for comparison purposes of the potential right of way takes between a two-lane roundabout and a single lane roundabout. The single lane roundabout alternative does not meet 2010 capacity needs but fits within the intersection area much better without major impacts to adjacent properties. The single lane roundabout would fail (LOS F) only during the PM peak hour if the provided traffic volumes are realized (conservatively speaking). However, if the traffic volumes did not quite reach 100% of the future volumes provided **or** if the assumed truck percentages were lower, the single lane roundabout would function at a LOS B or better, which is adequate.

Figures 10 and **11** show the capacity analyses with the same future volumes as the two-lane roundabout, a 1.5% lower assumed truck traffic during the PM peak hour, and wider single lane approaches for all legs of the roundabout with a 120 foot diameter. Without the 1.5% reduction in truck traffic during the PM peak hour, the capacity analyses show a LOS F during the PM peak hour.

Figure 10: AM RODEL ANALYSES: Single Lane Rbt and 1.5% Trucks



Shortcut to RODEL

28:8:06

LostCyn/SandCyn, 2010, Santa Clarita, CA

36

E	<m>	5.48	5.00	5.48	5.00	TIME PERIOD		min	90
L'	<m>	75.00	6.00	75.00	6.00	TIME SLICE		min	15
U	<m>	4.27	4.27	4.27	4.27	RESULTS PERIOD		min	15 75
RAD	<m>	36.00	23.00	36.00	22.00	TIME COST		\$/hr	15.00
PHI	<d>	20.00	20.00	20.00	20.00	FLOW PERIOD		min	15 75
DIA	<m>	36.60	36.60	36.60	36.60	FLOW TYPE		pcu/veh	VEH
GRAD	SEP	0	0	0	0	FLOW PEAK		am/op/pm	AM

LEG NAME	PCU	VEH	URNS	<1st exit, 2nd..U>	FLOF	CL	FLOW RATIO			FLOW TIME
LEG1	1.01	387	727	59 0	1.00	50	0.75	1.125	0.75	15 45 75
LEG2	1.01	382	6	74 0	1.00	50	0.75	1.125	0.75	15 45 75
LEG3	1.01	12	438	84 0	1.00	50	0.75	1.125	0.75	15 45 75
LEG4	1.01	104	12	33 0	1.00	50	0.75	1.125	0.75	15 45 75
-										

MODE 2

FLOW	veh	1173	462	534	149	AUDEL		s	6.5
CAPACITY	veh	1631	987	1624	1124	LOS		SIG	A
AUE DELAY	mins	0.14	0.12	0.05	0.06	LOS		UNSIG	A
MAX DELAY	mins	0.21	0.17	0.07	0.08				
AUE QUEUE	veh	3	1	0	0	VEHIC HRS			4.2
MAX QUEUE	veh	4	1	1	0	COST		\$	63

F1mode

F2direct

F3peak

CtrlF3rev

F4fact

F6stats

F8econ

F9prnt

F10run

Esc

Figure 11: PM RODEL ANALYSES: Single Lane Rbt and 0% Trucks

Shortcut to RODEL														
28:8:06		LostCyn/SandCyn, 2010, Santa Clarita,CA								34				
E	<m>	5.48	5.00	5.48	5.00	TIME PERIOD		min	90					
L'	<m>	25.00	6.00	75.00	6.00	TIME SLICE		min	15					
U	<m>	4.27	4.27	4.27	4.27	RESULTS PERIOD		min	15	75				
RAD	<m>	36.00	23.00	36.00	22.00	TIME COST		\$/hr	15.00					
PHI	<d>	20.00	20.00	20.00	20.00	FLOW PERIOD		min	15	75				
DIA	<m>	36.60	36.60	36.60	36.60	FLOW TYPE		pcu/veh	VEH					
GRAD	SEP	0	0	0	0	FLOW PEAK		am/op/pm	PM					
LEG NAME		PCU	VEH TURNS <1st exit, 2nd..U>			FLOF	CL	FLOW RATIO		FLOW TIME				
LEG1		1.00	91	1206	151	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG2		1.00	24	41	67	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG3		1.00	57	1216	18	0	1.00	50	0.75	1.125	0.75	15	45	75
LEG4		1.00	70	5	23	0	1.00	50	0.75	1.125	0.75	15	45	75
MODE 2														
FLOW		veh	1448	132	1291	98	AUDEL		s	16.2				
CAPACITY		veh	1704	653	1561	701	LOS		SIG	B				
AUE DELAY		mins	0.29	0.12	0.27	0.10	LOS		UNSIG	C				
MAX DELAY		mins	0.54	0.17	0.50	0.14								
AUE QUEUE		veh	7	0	6	0	VEHIC HRS			13.3				
MAX QUEUE		veh	12	0	10	0	COST		\$	200				
F1mode F2direct F3peak CtrlF3rev F4fact F6stats F8econ F9prnt F10run Esc														

As shown above, the queue lengths that form during the PM peak hour are beginning to be unacceptable for a “good roundabout design” with queues of 6-7 vehicles. The future design year volumes were not adjusted in the analyses above with the exception of the percentage of trucks. With the added delay large trucks create by slowing or stopping at intersections, the capacity of the single lane roundabout quickly diminishes as the percentage of truck traffic increases. Hence, this option may not be the “best” option for capacity purposes if the City

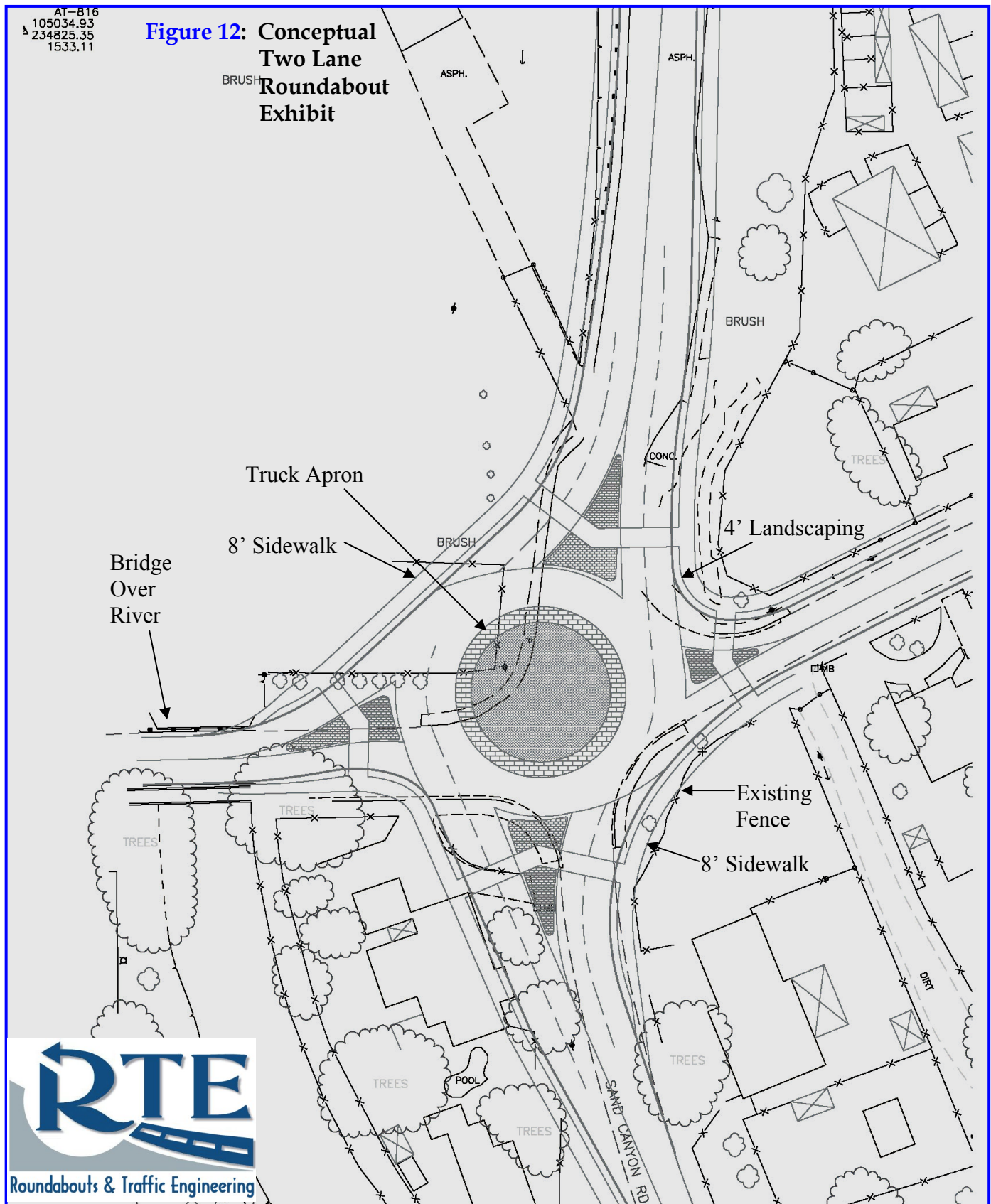
believes the traffic volumes and truck percentages are accurate. The results of the above analyses are shown in [Table 2B](#) below.

Table 2B: Roundabout Intersection Capacity Summary					
Lost Canyon Rd / Sand Canyon Rd					
Peak Hour Results - 2010 (SINGLE LANE ROUNDABOUT)					
Intersection	Approach				Intersection LOS
	SB	EB	NB	WB	
<u>AM PEAK HOUR</u>					
DELAY (in seconds)	8.4	7.2	3.0	3.6	6.5
LOS	A	A	A	A	A
AVG QUEUE (ft / lane)	75'	25'	0'	0'	
<u>PM PEAK HOUR</u>[†]					
DELAY (in seconds)	17.4	7.2	16.2	6.0	16.2
LOS	B	A	B	A	B
AVG QUEUE (ft / lane)	175'	0'	150'	0'	
Note: The PM peak hour truck percentages were reduced 1.5% to 1.00 in the analyses.					
Source: RTE			Lost-Sand Feasibility Tables.xls		

Regardless, at the request of the applicant and City staff, RTE developed a single lane roundabout concept for this intersection as well to compare the impacts of both roundabout alternatives. [Figure 13](#) illustrates only one variation of an adequate single lane roundabout design.

The City should keep in mind that modern roundabouts have a high amount of “flexibility” on the placement of the circle. Both [Figures 12](#) and [13](#) below graphically represent the *conceptual* layout of (1) the partial two-lane modern roundabout and (2) a single lane roundabout at the intersection. Specific fastest path entry design speeds (prior to yield line) at approximately between 20 and 25 miles per hour have been incorporated into the roundabout design for proper deflection and safety (on a conceptual level).

The specific design vehicle, identified as a fire truck and a WB-50, was overlaid on the roundabout design and briefly checked for the truck turning movement capabilities. The software program AutoTurn 5.0 was used for the turning movements of the intersection roadways to verify proper truck turning radii through the roundabout for every approach and movement on a conceptual level. This determined the theoretical 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 less than 1.5% trucks; hence the design does not accommodate large trucks traversing the roundabout simultaneously (side by side) in the intersection. This is typical for most roundabouts.



**Figure 13: Conceptual
Single Lane
Roundabout
Exhibit**



It should be noted that both **Figures 12** and **13** are abstract illustrations or conceptual roundabout exhibits at a low level of detail that was developed for the intersection for preliminary discussion purposes only. The sketch simply demonstrates the recommended design lane configurations and initial geometry recommendations with special consideration of approach speeds, the existing bridge, and the right-of-way constraints.

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 Clarita. The provided conceptual roundabout design will require further modifications for final PS&E plans.

V. CAPACITY COMPARISONS

This chapter compares the proposed roundabout alternative to the proposed signal alternative 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 2010.

SIGNALS VS. ROUNDABOUTS

Capacity (Delay & LOS): **Table 3** illustrates the 2010 capacity comparisons of the traffic signal and the two-lane modern roundabout. The actual Traffix and RODEL output was shown in the previous chapter of this report. As shown in the tables below, the two-lane modern roundabout's level of service operates significantly superior to the signalized intersection assuming similar lane configurations.

Table 3: Capacity Comparison Lost Canyon Rd / Sand Canyon Rd Peak Hour Results - 2010						
Intersection	Single Lane Rbt		Two Lane Rbt		Signal	
	AM	PM ¹	AM	PM	AM	PM
DELAY (in seconds)	6.5	16.2	4.0	5.5	38.6	44.0
LOS	A	B	A	A	D	D
<i>Note: The PM peak hour truck percentages were reduced 1.5% to 1.00 in the single lane rbt analyses.</i>						
Source: RTE			Lost-Sand 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 on the eastbound and westbound approaches; whereas the traffic signal requires left turn lanes at all four approaches. The City may need to review the traffic signals impact on the existing bridge and its width constraints for implementing a left turn lane for the traffic signal.
- The signal would operate with protected (stopped) phases, whereas the roundabout would operate with yield conditions on all approaches with relatively continuous traffic flow.

- The roundabout provides safer pedestrian crossings with only one (eastbound/westbound) or two lanes (northbound/southbound) of one-way traffic to cross at the largest distances with slow or yielding traffic speeds. The signal requires a pedestrian to cross at least three lanes of two-way traffic while several phases of traffic remain stopped.
- The roundabout could be constructed as a single lane roundabout as an interim measure until the higher traffic volumes are realized (if ever) in the future.

As a result, the required lane geometry for the roundabout has fewer number of required approach lanes, significantly less storage lengths required, less lane changing prior to the intersection, and a smaller amount of turning movement conflicts when compared to the traffic signal at the intersection.

Right-of-Way: Even though the two lane modern roundabout requires a minimum of a 150-foot outside diameter (120 feet for the single lane roundabout), the right of way impacts between the two-lane roundabout and the signal are relatively similar since the traffic signal also requires roadway widening and most of the roundabout's circle can be placed in unoccupied land. The southwest and southeast quadrants will require minor right of way acquisition despite the alternative chosen. However, if a single lane roundabout were constructed as an interim measure, the right-of-way impacts are relatively small.

Queue Lengths: A significant comparison can also be made between the queue lengths formed from the traffic signal and the modern roundabout for the 2010 design year conditions. Typically, roundabouts have significantly less queue buildup than traffic signals since roundabouts provide a relatively continuous traffic flow pattern. The RODEL output shows 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.

As shown in [Table 4](#) below, the queue lengths are quite short for the two-lane roundabout compared to the traffic signal with the same traffic volumes. The roundabout requires less vehicular stacking at the intersection, which will improve the operations of nearby private driveways. The roundabout 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.

Table 4: Queuing Comparison				
Lost Canyon Rd / Sand Canyon Rd				
Peak Hour Results - 2010				
Intersection	Approach			
	SB	EB	NB	WB
<u>AM PEAK HOUR</u>				
Traffic Signal	200'	1050'	125'	200'
Single Lane Roundabout	75'	25'	0'	0'
Two Lane Roundabout	25'	25'	0'	0'
<u>PM PEAK HOUR</u>				
Traffic Signal	1900'	1275'	150'	175'
Single Lane Roundabout	175'	0'	150'	0'
Two Lane Roundabout	50'	0'	50'	0'
Source: RTE		Lost-Sand Feasibility Tables.xls		

One primary point illustrated in Table 4 are the extremely long queues that would form in the northbound and southbound directions with the traffic signal. In addition, these queues are *longer* than the queues that would form at the single lane roundabout alternative. Another major issue with the traffic signal's eastbound queue lengths is the lack of space for a left turn lane due to the presence of the existing bridge. The actual length of this turn lane would be substantially shorter than the average queue in the PM peak hours.

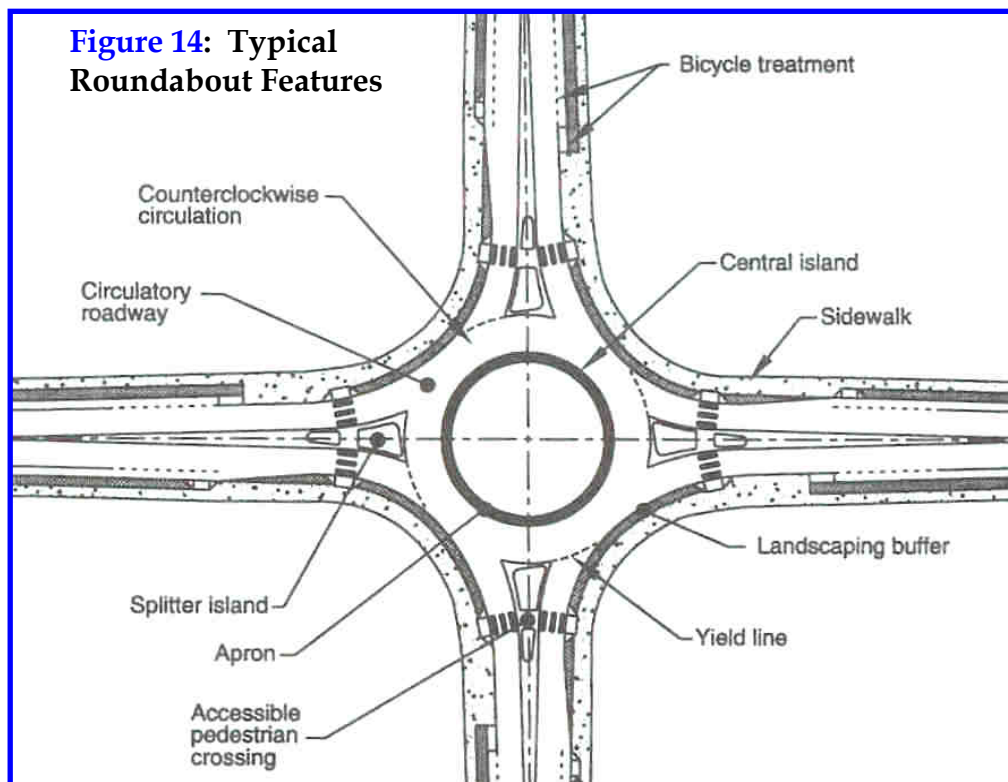
Capacity Summary: In summary, there are demonstrated capacity benefits for the roundabout operations (single or two-lane) versus the signal. This can mostly be explained by the basic operational characteristics of a signal versus a roundabout. A signal requires traffic flows to stop and wait for the 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.

VI. SAFETY COMPARISONS

The previous chapters analyzed the capacity requirements of both a traffic signal and a modern roundabout at the 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.

GENERAL ROUNDABOUT INFORMATION

Modern roundabouts are a type of circular intersection with specific design and traffic control features to control driver behavior. **Figure 14** identifies key modern roundabout features³ 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



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

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. This intersection location has not progressed to this level of detail yet. However, many other proposed roundabout features were analyzed during the roundabout design.

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 design components:

- 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 study⁴ 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 study⁵ 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

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*)

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 15](#)) demonstrates a comparison of traffic signals to roundabouts based on braking distance and driver perception/reaction distances for braking.

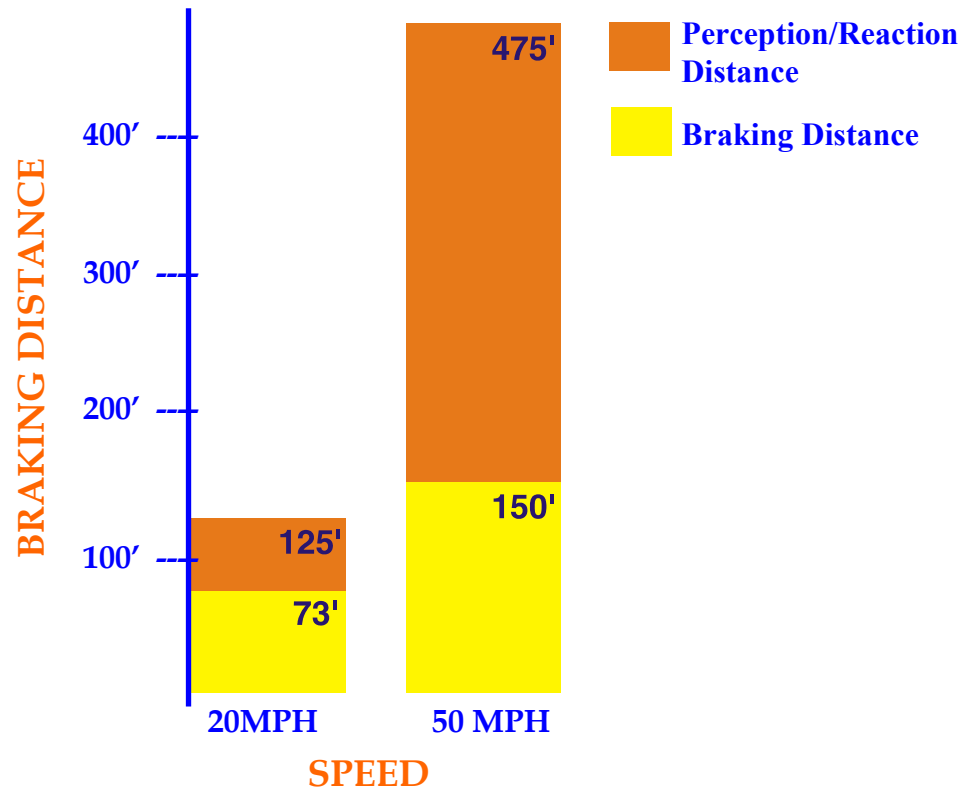


Figure 15: Braking Distances & Speeds

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 16** 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.

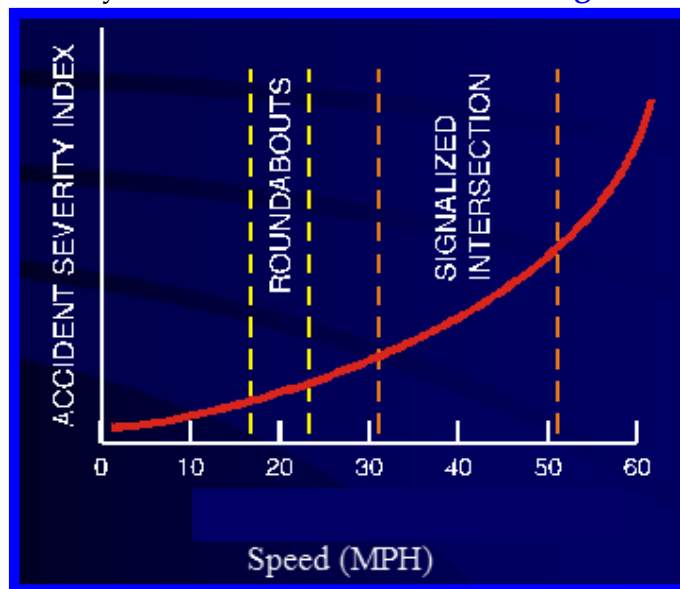
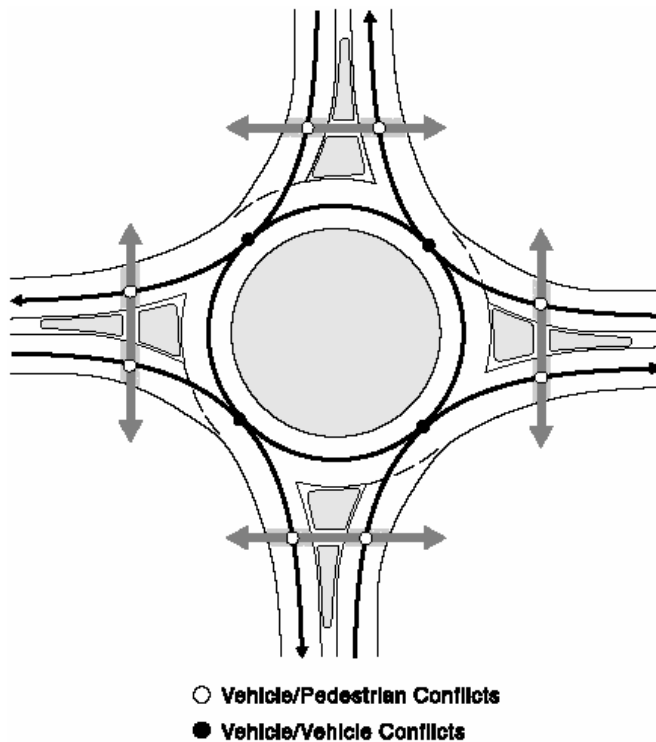
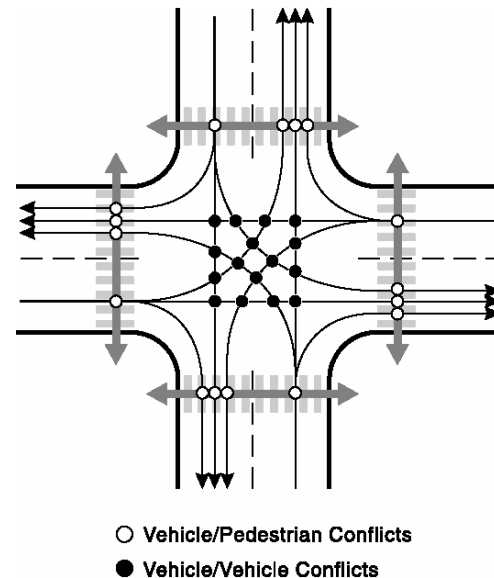


Figure 16: Accident Severity & Speeds

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 17** and **18**) show the number of vehicle-to-vehicle (black dots) and vehicle-to-pedestrian (white dots) conflicts at a roundabout and signal.



**Figure 17: Roundabout
 Points of Conflict**



**Figure 18: 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.

ACCESS OPERATIONS

Both the traffic signal and roundabout alternative would maintain ingress and egress access to all adjacent properties. However, the issue arises with vehicles blocking these driveways with the traffic signal as more traffic congestion occurs in the future. The signal's queue lengths far exceed the access points to many driveways along all four approaches/roadways. The single lane roundabout also has queues that may block access in the PM peak hour for a short period.

Although the roundabout designs propose to extend the median or splitter island west towards the bridge over the river, access to all properties can still be maintained from any direction of travel. However, further information is required to analyze all access points near the roundabout, which should be accomplished under final design if the City decides to proceed with the roundabout alternative. Hence, one primary difference between the traffic signal and the modern roundabout is the operational characteristics of the two intersections and the benefits the roundabout provides for access operations.

EMERGENCY VEHICLE OPERATIONS

With respect to emergency vehicle operations, the traffic signal would need to be equipped with an emergency detection system that actuates the signal in favor of the emergency vehicle in order to have similar response times to the roundabout. Although the sirens, lights, and horn of the emergency vehicle should be sufficient warning the concern for this intersection is the ability of the emergency vehicle to pass through a long queue of vehicles waiting at the signal (similar to the access issue above).

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 entering the roundabout's circulating roadway. 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 about 25 miles per hour at entry and about 18 miles per hour circulating the roundabout. This significantly decreases the likelihood of accidents with vehicles and the entering emergency vehicle.

The only conflicting movements with the emergency vehicle are any remaining traffic in the circulating roadway, with a driver choice to either exit the roundabout and pull over or pull over immediately within the roundabout (not common). All approaching traffic to the roundabout would be required to yield to the emergency vehicle now in the circulating roadway. The emergency vehicle could 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 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 incident. This can be shown in video clips taken by RTE in the field. Vehicular traffic is relatively undisturbed with little driver frustration once the emergency vehicle passes and can continue to operate normally. The emergency vehicle has access anywhere to any direction, including u-turn options within the roundabout.

Emergency vehicle response times are also worth noting. 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.

VII. CONCLUSIONS & RECOMMENDATIONS

CONCLUSIONS

The preceding sections of this report analyzed the feasibility of a modern roundabout at the intersection of Sand Canyon Road / Lost Canyon Road as well as compared the capacity and safety of either a modern roundabout or a traffic signal alternative. The following summarizes the analysis sections of this report.

- A modern roundabout is a feasible and appropriate traffic control device for this intersection.
- The single lane roundabout is the preferred alternative between the two-lane roundabout and the single-lane roundabout due to the tight right-of-way, property impacts, bridge constraints, etcetera. However, the two-lane roundabout is the “best” choice for capacity considerations only (based on the traffic volumes developed in the traffic study).
- Both roundabout alternatives provide superior capacity over the signal alternative 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 roundabout provides self-enforcing geometry and forces traffic speeds to be slow for the entire intersection. This significantly decreases the likelihood of accidents with vehicles as well as with emergency vehicles.
- The operational characteristics of the roundabout are superior to the traffic signal. This includes adjacent access operations and emergency vehicle operations.
- The roundabout and signal will both require additional right-of-way in future conditions in order to maintain City thresholds of LOS C and the additional roadway width for the signal’s required turn lane lengths.
- The roundabout would reduce vehicle emissions (see [Appendix](#)).

- The roundabout would enhance the character of the City of Santa Clarita at and near the study intersection with added landscaping and potential ornamental statues for public appeal.

In addition to the feasibility, capacity, and safety of each alternative, the importance of a proper functioning traffic control device that increases aesthetical beauty and community character for the City of Santa Clarita is also significant.

Therefore, it can be unanimously determined by all the contributing factors within this feasibility study except the need for additional right-of-way that the modern roundabout is recommended for this intersection.

Although this report provides information and comparisons primarily focusing on the feasibility, capacity requirements, and safety analyses of each alternative, a number of other comparisons could also be made between the two alternatives for the 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. 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 and the City of Santa Clarita. 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, and other 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 5](#) provides the items considered and the summary results of the analyses.

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 North America. As shown in the comparison matrix, the total score for both roundabout alternatives is much

better than the score of the signal alternative. Also shown, the single lane roundabout has a slightly higher score due to the lower impacts of the project despite the lower LOS and increased delay at the intersection.

Table 5: Decision Matrix <i>Comparison of Key Elements of Alternatives</i>							
Alternative	Delay / LOS	Vehicle Safety	Const. Cost	EMS & Ped Safety	O & M Costs	Aes- thetics	Total Score: Higher is Better
Weight	30%	30%	10%	15%	5%	10%	
Signalized Intersection	2	3	3	1	3	2	2.30
Multi-Lane Roundabout	5	4	2	5	4	4	4.25
Single-Lane Roundabout	3	5	4	5	5	5	4.30
Legend: 0=Very Poor, 1=Poor, 2=Below Avg, 3=Average, 4=Above Avg, 5=Excellent							
Source: RTE				Lost-Sand Feasibility Tables.xls			

Cost Comments: Although the costs of the roundabout are not addressed in this feasibility report, it should be noted that a traffic signal requires 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 range of \$5,500 to \$22,000 per year depending on the signal. A roundabout does not incur such maintenance costs unless annual flowers or foliage need replacement or upkeep in the central island or outside the roundabout. The required obstructions in the roundabout design can usually be accommodated with perennial foliage, statues, or rocks that requires little to no maintenance. Other costs, such as curb and gutter, drainage maintenance, pavement repair, and lighting maintenance are similar between the two alternatives.

Other Comments: Air emissions and construction traffic impacts are additional topics that could be discussed at great length where roundabouts provide positive results over traffic signals. With respect to the reduction of air emissions, a recent report published by Gene Russell with Kansas State University is attached to this report (see [Appendix](#)) for additional vehicle emission reduction information from roundabouts. Essentially, the report documents through several case studies, "The research concludes that modern roundabout can be used as a viable alternative to cut down vehicular emissions and thereby making intersections more environmentally friendly". 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 city focal point. The local environment at the intersections could be significantly improved with proper landscaping.

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 design, and the comparative analyses between the signal and the roundabout. As determined in the conclusions, the roundabout alternative is recommended for this intersection location. Specifically, the single lane roundabout would be the preferred alternative for this location due to the many design constraints despite the reduced capacity benefits. However, if the City is very confident that the future volumes will reach and surpass the volumes identified in the traffic study in the near future, then the multi-lane roundabout would be most beneficial for the capacity of the intersection.

It is recommended to proceed with the final geometric layout of the single lane 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 staking information. It is recommended that the geometric layout of the roundabout design be revisited with a redesign and potential slight relocation of the roundabout's circle to best fit the public and city's concerns.

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 design ([Figures 12 and 13](#)), the entries are visible to drivers from a safe stopping distance, safe design speeds corresponding to the fastest paths exist to 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 design 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 black/white) with chevrons (as separate sections).
- Provide highly visible and obstructing landscaping in the central island according to sight distance requirements for each entry and circulating points within the roundabout.
- Provide obstructing landscaping in the planter areas outside the roundabout to reduce driver - pedestrian side friction.
- Provide post mounted maptype signs as shown in the *Roundabout Signing Guide, A Recommended Practice, 1st Edition*,⁶ for driver comprehension of destination and repeated display and understanding of a roundabout ahead.
- Provide an internally illuminated bollard (MUTCD compliant⁸) 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-foot planted for the detached sidewalk with an eight-foot multiuse path is already shown on the conceptual roundabout exhibits. However, these dimensions could be reduced to a two-foot planter if necessary.
- 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.

⁶ 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

- Provide highly visible crosswalks with the use of thermoplastic in an international style stripe design (a.k.a. “ladder” stripes) or a stamped and colored concrete for high visibility.
- 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.

APPENDIX

Roundabout Feasibility Report
 Traffic Impact Analysis Data
 RTE: Lost Cyn / Sand Cyn Intersection

Level Of Service Computation Report

2000 HCM Operations Method (Future Volume Alternative)

Intersection #1 Lost/Sand

Cycle (sec): 125 Critical Vol./Cap. (X): 0.924

Loss Time (sec): 16 (Y+R = 4 sec) Average Delay (sec/veh): 38.6

Optimal Cycle: OPTIMIZED Level Of Service: D

Street Name:	Sand Cyn			Lost Cyn								
Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	T	R	L	T	R	L	T	R	L	T	R
Control:	Protected			Protected			Protected			Protected		
Rights:	Include			Include			Include			Include		
Min. Green:	0	0	0	0	0	0	0	0	0	0	0	0
Lanes:	1	0	0	1	0	0	1	0	0	1	0	0

Volume Module:	>>	Count	Date:	1 Jan 2010	<<	AM
Base Vol:	84	438	12	59	727	387
Growth Adj:	1.00	1.00	1.00	1.00	1.00	1.00
Initial Bse:	84	438	12	59	727	387
Added Vol:	0	0	0	0	0	0
PasserByVol:	0	0	0	0	0	0
Initial Fut:	84	438	12	59	727	387
User Adj:	1.00	1.00	1.00	1.00	1.00	1.00
PHF Adj:	0.95	0.95	0.95	0.95	0.95	0.95
PHF Volume:	88	461	13	62	765	407
Reduct Vol:	0	0	0	0	0	0
Reduced Vol:	88	461	13	62	765	407
PCE Adj:	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
Final Vol.:	88	461	13	62	765	407

Saturation Flow Module:	Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.96	1.01	1.01	0.96	0.96	0.96	0.94	0.94	0.94	0.97	0.88	0.88
Lanes:	1.00	0.97	0.03	1.00	0.65	0.35	1.00	0.63	0.37	1.00	0.10	0.90
Final Sat.:	1823	1860	51	1823	1187	632	1787	1121	656	1841	173	1503

Capacity Analysis Module:	Vol/Sat:	0.05	0.25	0.25	0.03	0.64	0.64	0.04	0.04	0.04	0.02	0.07	0.07
Crit Moves:	****				****			****				****	
Green/Cycle:	0.05	0.66	0.66	0.09	0.70	0.70	0.04	0.08	0.08	0.04	0.08	0.08	0.08
Volume/Cap:	0.92	0.38	0.38	0.38	0.92	0.92	0.92	0.47	0.47	0.47	0.92	0.92	0.92
Delay/Veh:	126.4	9.8	9.8	54.9	27.4	27.4	136.8	57.2	57.2	63.4	113	112.6	112.6
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	1.00
AdjDel/Veh:	126.4	9.8	9.8	54.9	27.4	27.4	136.8	57.2	57.2	63.4	113	112.6	112.6
HCM2kAvg:	6	8	7	3	42	42	5	3	3	2	8	8	8

Roundabout Feasibility Report
 Traffic Impact Analysis Data
 RTE: Lost Cyn / Sand Cyn Intersection

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

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*****
Intersection #1 Lost/Sand
*****
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 0 1 0      1 0 0 1 0      1 0 0 1 0      1 0 0 1 0
Lane Group:     L  RT  RT      L  RT  RT      L  RT  RT      L  RT  RT
#LnsInGrps:     1  1      1  1      1  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:          -2%           -2%           +2%           -4%
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:     1 xxxx  xxxx      1 xxxx  xxxx      1 xxxx  xxxx      1 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  1.00 1.00  1.00
Hev Veh 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
Grade Adj:      1.01 1.01  1.01  1.01 1.01  1.01  0.99 0.99  0.99  1.02 1.02  1.02
Parking Adj:    xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00
Bus Stp Adj:    xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00
Area 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
RT Adj:         xxxx 1.00  1.00  xxxx 0.95 0.95  xxxx 0.95 0.95  xxxx 0.87 0.87
LT Adj:         0.95 xxxx xxxxxx 0.95 xxxx xxxxxx 0.95 xxxx xxxxxx 0.95 xxxx xxxxxx
PedBike 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
HCM Sat Adj:    0.96 1.01  1.01  0.96 0.96  0.96  0.94 0.94  0.94  0.97 0.88  0.88
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:    0.96 1.01  1.01  0.96 0.96  0.96  0.94 0.94  0.94  0.97 0.88  0.88
-----|-----|-----|-----|
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  1.00 1.00  1.00
*****

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Roundabout Feasibility Report
 Traffic Impact Analysis Data
 RTE: Lost Cyn / Sand Cyn Intersection

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

Intersection #1 Lost/Sand

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	T	R	L	T	R	L	T	R	L	T	R
Green/Cycle:	0.05	0.66	0.66	0.09	0.70	0.70	0.04	0.08	0.08	0.04	0.08	0.08
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:	3.2	7.5	6.6	2.1	34.7	34.4	2.6	2.3	2.3	1.2	4.4	4.2
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	1.00	1.00	1.00
Q2:	3.0	0.6	0.6	0.6	7.5	7.5	2.7	0.8	0.8	0.8	3.4	3.3
HCM2KQueue:	6.2	8.1	7.2	2.7	42.2	41.8	5.3	3.1	3.1	2.0	7.7	7.5
70th%Factor:	1.19	1.18	1.18	1.19	1.13	1.14	1.19	1.19	1.19	1.20	1.18	1.18
70th%HCM2kQ:	7.3	9.5	8.5	3.2	47.9	47.5	6.2	3.7	3.7	2.4	9.1	8.9
85th%Factor:	1.54	1.53	1.54	1.57	1.37	1.37	1.55	1.57	1.57	1.58	1.53	1.53
85th%HCM2kQ:	9.5	12.3	11.1	4.3	58.0	57.5	8.2	4.9	4.8	3.2	11.8	11.5
90th%Factor:	1.69	1.67	1.68	1.75	1.45	1.45	1.71	1.74	1.74	1.76	1.67	1.67
90th%HCM2kQ:	10.5	13.4	12.1	4.7	61.2	60.6	9.0	5.4	5.4	3.6	12.9	12.6
95th%Factor:	1.93	1.88	1.90	2.02	1.56	1.56	1.95	2.01	2.01	2.04	1.89	1.90
95th%HCM2kQ:	11.9	15.2	13.7	5.5	65.8	65.2	10.2	6.2	6.2	4.1	14.6	14.2
98th%Factor:	2.32	2.24	2.27	2.51	1.74	1.74	2.37	2.49	2.49	2.56	2.25	2.26
98th%HCM2kQ:	14.4	18.0	16.4	6.8	73.4	72.8	12.4	7.7	7.7	5.2	17.4	17.0

Roundabout Feasibility Report
Traffic Impact Analysis Data
RTE: Lost Cyn / Sand Cyn Intersection

Level Of Service Computation Report

2000 HCM Operations Method (Future Volume Alternative)

Intersection #1 Lost/Sand

Cycle (sec): 170 Critical Vol./Cap. (X): 0.968

Loss Time (sec): 16 (Y+R = 4 sec) Average Delay (sec/veh): 44.0

Optimal Cycle: OPTIMIZED Level Of Service: D

Street Name:	Sand Cyn				Lost Cyn				
Approach:	North Bound		South Bound		East Bound		West Bound		
Movement:	L	T	R	L	T	R	L	T	R
Control:	Protected		Protected		Protected		Protected		
Rights:	Include		Include		Include		Include		
Min. Green:	0	0	0	0	0	0	0	0	0
Lanes:	1	0	0	1	0	0	1	0	0

Volume Module: >> Count Date: 1 Jan 2025 << PM

Base Vol:	18	1216	57	151	1206	91	67	41	24	23	5	70
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:	18	1216	57	151	1206	91	67	41	24	23	5	70
Added Vol:	0	0	0	0	0	0	0	0	0	0	0	0
PasserByVol:	0	0	0	0	0	0	0	0	0	0	0	0
Initial Fut:	18	1216	57	151	1206	91	67	41	24	23	5	70
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:	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
PHF Volume:	19	1280	60	159	1269	96	71	43	25	24	5	74
Reduct Vol:	0	0	0	0	0	0	0	0	0	0	0	0
Reduced Vol:	19	1280	60	159	1269	96	71	43	25	24	5	74
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.:	19	1280	60	159	1269	96	71	43	25	24	5	74

Saturation Flow Module:

Sat/Lane:	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Adjustment:	0.96	1.00	1.00	0.96	1.00	1.00	0.94	0.94	0.94	0.97	0.88	0.88
Lanes:	1.00	0.96	0.04	1.00	0.93	0.07	1.00	0.63	0.37	1.00	0.07	0.93
Final Sat.:	1823	1820	85	1823	1767	133	1787	1121	656	1841	111	1556

Capacity Analysis Module:

Vol/Sat:	0.01	0.70	0.70	0.09	0.72	0.72	0.04	0.04	0.04	0.01	0.05	0.05
Crit Moves:	****		****		****		****		****		****	
Green/Cycle:	0.01	0.73	0.73	0.09	0.80	0.80	0.04	0.07	0.07	0.02	0.05	0.05
Volume/Cap:	0.89	0.97	0.97	0.97	0.89	0.89	0.97	0.58	0.58	0.58	0.97	0.97
Delay/Veh:	229.7	38.6	38.6	137.7	18.6	18.6	175.7	83.8	83.8	100.4	169	169.4
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:	229.7	38.6	38.6	137.7	18.6	18.6	175.7	83.8	83.8	100.4	169	169.4
HCM2kAvg:	2	69	76	12	51	51	6	4	4	2	6	7

Roundabout Feasibility Report
Traffic Impact Analysis Data
RTE: Lost Cyn / Sand Cyn Intersection

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

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*****
Intersection #1 Lost/Sand
*****
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 0 1 0      1 0 0 1 0      1 0 0 1 0      1 0 0 1 0
Lane Group:     L  RT  RT      L  RT  RT      L  RT  RT      L  RT  RT
#LnsInGrps:     1  1      1  1      1  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:          -2%           -2%           +2%           -4%
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:     1 xxxx  xxxx      1 xxxx  xxxx      1 xxxx  xxxx      1 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  1.00 1.00  1.00
Hev Veh 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
Grade Adj:      1.01 1.01  1.01  1.01 1.01  1.01  0.99 0.99  0.99  1.02 1.02  1.02
Parking Adj:    xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00
Bus Stp Adj:    xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00  xxxx 1.00  1.00
Area 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
RT Adj:        xxxx 0.99  0.99  xxxx 0.99  0.99  xxxx 0.95  0.95  xxxx 0.86  0.86
LT Adj:         0.95 xxxx xxxxxx 0.95 xxxx xxxxxx 0.95 xxxx xxxxxx 0.95 xxxx xxxxxx
PedBike 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
HCM Sat Adj:    0.96 1.00  1.00  0.96 1.00  1.00  0.94 0.94  0.94  0.97 0.88  0.88
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:    0.96 1.00  1.00  0.96 1.00  1.00  0.94 0.94  0.94  0.97 0.88  0.88
-----|-----|-----|-----|
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  1.00 1.00  1.00
*****
```

Roundabout Feasibility Report
 Traffic Impact Analysis Data
 RTE: Lost Cyn / Sand Cyn Intersection

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

Intersection #1 Lost/Sand

Approach:	North Bound			South Bound			East Bound			West Bound		
Movement:	L	-	T - R	L	-	T - R	L	-	T - R	L	-	T - R
Green/Cycle:	0.01	0.73	0.73	0.09	0.80	0.80	0.04	0.07	0.07	0.02	0.05	0.05
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:	0.9	58.1	65.4	7.9	44.8	44.9	3.5	3.1	3.1	1.2	3.5	3.7
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	1.00	1.00	1.00	1.00	1.00	1.00
Q2:	1.4	10.5	11.0	4.2	6.4	6.4	2.9	1.1	1.1	1.0	2.9	3.0
HCM2KQueue:	2.4	68.6	76.3	12.1	51.1	51.3	6.4	4.3	4.3	2.2	6.5	6.7
70th%Factor:	1.19	1.12	1.11	1.17	1.13	1.13	1.19	1.19	1.19	1.19	1.19	1.18
70th%HCM2kQ:	2.8	76.7	85.1	14.2	57.7	57.8	7.6	5.1	5.1	2.6	7.7	8.0
85th%Factor:	1.58	1.33	1.32	1.50	1.35	1.35	1.54	1.56	1.56	1.58	1.54	1.54
85th%HCM2kQ:	3.7	91.2	101.0	18.2	69.3	69.4	9.9	6.7	6.7	3.4	10.0	10.4
90th%Factor:	1.76	1.41	1.41	1.62	1.43	1.43	1.69	1.72	1.72	1.76	1.69	1.69
90th%HCM2kQ:	4.2	96.9	107.5	19.6	73.2	73.3	10.8	7.4	7.4	3.8	10.9	11.3
95th%Factor:	2.03	1.51	1.51	1.81	1.54	1.53	1.92	1.97	1.97	2.03	1.92	1.91
95th%HCM2kQ:	4.8	104	115.1	21.9	78.5	78.7	12.3	8.5	8.4	4.4	12.4	12.9
98th%Factor:	2.53	1.71	1.70	2.09	1.72	1.72	2.31	2.42	2.42	2.55	2.31	2.30
98th%HCM2kQ:	6.0	117	130.0	25.4	87.9	88.1	14.8	10.4	10.3	5.5	14.9	15.4

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Impact of Modern Roundabouts on Vehicular Emissions

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ABSTRACT

Vehicular emissions have increased considerably over the years with the increase in traffic. Modern roundabouts can improve traffic flow as well as cut down vehicular emissions and fuel consumption by reducing the vehicle idle time at intersections and thereby creating a positive impact on the environment.

The primary focus of this research is to study the impact of modern roundabouts in cutting down vehicular emissions. Six sites with different traffic volume ranges, where a modern roundabout has replaced Stop controlled intersection, have been chosen for the study. The operation of the roadways at the intersection was videotaped and the traffic flow data was extracted from these tapes and analyzed using Signalized and Unsignalized Intersection Design and Research Aid (SIDRA) software. The version used is a.a. SIDRA 2.0. The software produces many measures of effectiveness (MOEs) of which five were chosen for analyzing the environmental impact of roundabouts. The chosen five outputs give rate of emission of HC, CO, NO_x, and CO₂ in (kg/hr).

All the MOEs were statistically compared to determine which intersection control performed better. After comparing all the MOEs at all locations for the before and after traffic volumes, it was found that the modern roundabout performed better than the existing intersection control (i.e., stop signs) in cutting down vehicular emissions thereby creating a positive impact on the environment. The research concludes that a modern roundabout can be used, as a viable alternative to cut down vehicular emissions and thereby making intersections more environment friendly.

Key words: modern roundabouts—reducing air pollution—vehicular emissions

INTRODUCTION

With the increase in traffic over the years, one of the major threats to clean air in many of the developed countries like the U.S. is vehicular emissions. Problems posed by the environmental impact of traffic are growing and are a challenge for traffic engineers. Vehicular emissions are dependent on the total amount of traffic, intersection control type, driving patterns and vehicular characteristics.

Vehicular emissions contain a wide variety of pollutants, principally carbon monoxide (CO), carbon dioxide (CO₂), oxides of nitrogen (NO_x), particulate matter (PM₁₀) and hydrocarbons (HC) or volatile organic compounds (VOCs) which have a major long term impact on air quality. These emissions vary with the engine design, the air-to-fuel ratio, and vehicle operating characteristics. With increasing vehicle speed there is an increase in NO_x emissions and decrease in CO, PM₁₀ and HC or VOC emissions. The emissions of (CO₂) and oxides of sulfur (SO_x) vary directly with fuel consumption and for any given vehicle and fuel combination, aggregate emission levels vary according to the distance traveled and the driving patterns (1).

Road and street intersections force vehicular traffic to slow down and stop in varying patterns of interruption of ideal, constant traffic flow at an ideal speed. The longer the stops, the more fuel that is consumed and vehicular emissions increase. With the vehicular emissions problems worsening, it has become prudent to choose effective traffic control devices (TCDs) that can improve traffic flow on the roads and reduce emissions per vehicle kilometer traveled while enhancing mobility.

Modern roundabouts in the U.S., which are functioning as one of the safest and most efficient forms of intersection control (4, 14, 15) and improving traffic flow at intersections, have the additional advantage of cutting down vehicular emissions and fuel consumption by reducing vehicle idling time at the intersections and thereby having a positive affect on the environment.

The primary focus of this research is to study the impact of modern roundabouts in cutting down vehicular emissions at intersections. This research focuses on six sites with different traffic volume ranges where a modern roundabout has replaced stop-controlled intersection. The emissions at the intersections were compared for the before (stop controlled) and after (modern roundabout) conditions to assess the impact of a modern roundabout at these intersections.

LITERATURE REVIEW

Vehicle exhaust fumes played a major role in the deterioration of air quality in urban areas since 1950s and as a result the Clean Air Act (CAA) was passed in 1970. The CAA gives the Environmental Protection Agency (EPA) the authority to set limits on emission standards. The EPA estimates that over 5,000 tons of VOCs from transportation sources were emitted in 1999 and that approximately 62 million people living in areas that do not meet health based standards. EPA also estimated that in 1999 the transportation sector, including on-road and non-road vehicles, contributed to 47 percent of hydrocarbon (HC) emissions, 55 percent of nitrogen oxides (NO_x) emissions, 77 percent of carbon monoxide (CO) emissions, and 25 percent of particulate matter (PM) emissions (2).

Roundabouts are being implemented throughout the U.S. in a variety of situations. Many states and cities are considering roundabouts as a viable alternative to other TCD's, and, in some cases, complex freeway interchanges. Modern roundabouts are becoming popular in the US for more than just safety reasons. As stated in an article by the Insurance Institute for Highway Safety (IIHS) they reduce fuel consumption and vehicular emissions by reducing stopping at intersections, and also reduce noise levels by making the traffic flow orderly. Modern roundabouts can enhance the aesthetics of the place and create visual

gateways to communities or neighborhoods. In commercial areas they can improve access to adjacent properties (3).

Vehicles stopping at traffic signals and stop signs emit more carbon dioxide (CO₂) when compared to roundabouts as the delay and queuing are greater. Even if the delays are similar to that of roundabout, traffic signals always queue traffic at a red light and hence emissions are usually greater. The average delays at roundabouts have to be significantly larger than at traffic signals for the emissions to be equal. When traffic volumes are low, traffic rarely stops at a roundabout and the emissions are very small (5, 6).

When roundabouts become very congested with large queues, the emissions equal those at traffic signals. During off-peak hours roundabouts do not experience long queues and delays and the emissions are low. Traffic signals and stop signs stop vehicles even during off-peak hours and thereby experience higher delays and emissions. United Kingdom (UK) engineers believe that traffic signals have lower emissions only in exceptional cases (5, 6).

As stated by Barry Crown, a roundabout expert from the UK: “When vehicles are idle in a queue they emit about 7 times as much carbon monoxide (CO) as vehicles traveling at 10 mph. The emissions from a stopped vehicle are about 4.5 times greater than a vehicle moving at 5 MPH” (5).

The Bärenkreuzung/Zollikofen project undertaken in Bern, Switzerland, replaced two important signalized intersections by roundabouts and the result was a reduction of emissions and fuel savings by about 17 percent. The roundabouts also steadied the driving patterns (7).

On a microscale there have been studies conducted on the effect that different traffic flows have on emissions at an intersection. Of the studies that reported quantitative results, roundabouts reduced vehicle emissions for hydrocarbons (HC) in 5 studies by an average of 33 percent, carbon monoxide (CO) in 6 studies by an average of 36 percent, and nitric oxides (NO_x) in 6 studies by an average of 21 percent. The regional scale air quality benefits of roundabouts would depend on their percent contribution to regional mobile source emissions (8, 9).

In a study conducted by Mustafa et.al (1993), the authors concluded that there exists a direct relationship between vehicle emissions and traffic volumes at urban intersections regardless of traffic control. Their simulation results showed that traffic signals generate more emissions (almost 50 percent higher) than a roundabout. In case of higher traffic volumes the HC generated by traffic signals is twice as high as that generated at roundabouts (10).

In another study conducted by Varhelyi in Sweden, he found that replacing a signalized intersection with a roundabout resulted in an average decrease in CO emissions by 29 percent and NO_x emissions by 21 percent and fuel consumption by 28 percent per car within the influence of the junction (11).

Results of a study conducted by Jarkko Niittymäki show fuel consumption reductions of 30 percent in an intersection designed as a roundabout instead of using traffic signals and environmentally optimized traffic control systems have proved an energy saving potential of 10 percent to 20 percent in different cases (12).

METHODOLOGY

Description of Study Sites

Six study sites were selected for this research. Five of the sites are in Kansas and one in Nevada. Of the sites studied in Kansas two were in Olathe, one in Lawrence, one in Hutchinson, and one in Paola. Data from these sites was available from previous roundabout studies at Kansas State University (KSU) (4, 14, 15).

The sites in Olathe are (1) the intersection of the Ridgeview Road and Sheridan Avenue and (2) the intersection of Rogers Road and Sheridan Avenue. Sheridan Avenue runs in the East-West direction while the Ridgeview and Rogers Roads run in the North-South direction, roughly parallel to Interstate 35 (I-35).

The site in Lawrence is the T-intersection of the Harvard Road and Monterey Way. Harvard Road runs in the east-west direction while and ends at Monterey Way, which runs in the north-south direction.

The site in Hutchinson is the intersection of 23rd Street and Severance Avenue. Severance Avenue runs in the north-south direction and 23rd Street runs in the east-west direction.

The site in Paola is the intersection of the Old KC Road, State Route K68, and Hedge Lane. The Old KC Road runs in the north-south direction. The K68 runs in the east-west direction. Hedge Lane runs in Southeast- northwest direction, and intersects K68 just east of the K-68 and Old KC Road intersection.

The site in Nevada is the intersection of the Wedekind Road and ClearAcre Lane. Wedekind Road runs in the east-west direction while ClearAcre Lane runs in the north-south direction.

All the sites except Hutchinson and Nevada (which had a two-way stop control, TWSC) were controlled by stop signs on all approaches (all-way stop control, AWSC) prior to the installation of the modern roundabout. The major drawback of AWSC is that the presence of vehicles on all the approaches of the intersection will result in longer departure headways and longer driver decision times that reduce the capacity of the intersection. The major drawback of TWSC is that congestion on the minor street caused by a demand that exceeds capacity, and queues that form on the major street because of inadequate capacity for left turning vehicles yielding to opposing traffic. In the after condition a single-lane modern roundabout was built at all sites. The Paola roundabout is different from the others because it has five legs, and is an intersection on the state highway (4). See Table 1 for the intersection hourly traffic volume ranges and the percentage of left turn for the intersections studied.

TABLE 1. Intersection Hourly Traffic Volume Ranges and Percentages of Left Turns

PAOLA DATA			
AM (AWSC)	AM (Roundabout)	PM (AWSC)	PM (Roundabout)
257-594 (veh/hr)	235-559 (veh/hr)	192-690 (veh/hr)	156-663 (veh/hr)
28% left turns	29% left turns	38% left turns	40% left turns
LAWRENCE DATA			
AM (AWSC)	AM (Roundabout)	PM (AWSC)	PM (Roundabout)
227-536 (veh/hr)	263-447 (veh/hr)	412-733 (veh/hr)	442-692 (veh/hr)
30% left turns	17% left turns	26% left turns	21% left turns
OLATHE: ROGERS/SHERIDAN DATA			
AM (AWSC)	AM (Roundabout)	PM (AWSC)	PM (Roundabout)
926-1625 (veh/hr)	931-1738 (veh/hr)	1220-1994 (veh/hr)	1244-2024 (veh/hr)
28% left turns	28% left turns	21% left turns	22% left turns
OLATHE: RIDGEVIEW/SHERIDAN DATA			
AM (AWSC)	AM (Roundabout)	PM (AWSC)	PM (Roundabout)
708-1110 (veh/hr)	776-1124 (veh/hr)	1140-1626 (veh/hr)	1119-1784 (veh/hr)
33% left turns	33% left turns	35% left turns	38% left turns
HUTCHINSON DATA			
AM (TWSC)	AM (Roundabout)	PM (TWSC)	PM (Roundabout)
449-983 (veh/hr)	415-864 (veh/hr)	514-1204 (veh/hr)	501-1110 (veh/hr)
13% left turns	12% left turns	13% left turns	15% left turns
NEVADA DATA			
AM (TWSC)	AM (Roundabout)	PM (TWSC)	PM (Roundabout)
423-718 (veh/hr)	372-691 (veh/hr)	619-893 (veh/hr)	547-881 (veh/hr)
31% left turns	32% left turns	28% left turns	27% left turns

Data Collection

The available data had been collected in two phases. The first phase was videotaping intersection traffic movements with a video camera and the second phase was obtaining traffic counts visually from the videotapes (4, 14, 15).

Phase 1: Video Data Collection

The benefit of using this method for data collection is that all the data is recorded on videotapes and can be accessed and retrieved at a later time. In this method, all the information recorded on the tapes can be accessed for evaluation at any time and serves as a permanent record for re- verification of data, or reuse for other purposes. A specially designed 360-degree omni directional, video camera and videocassette recorder were used for data collection at each location. The camera was designed to provide a full 360 degrees view when mounted above the intersection. The camera was placed near the intersection to see the traffic flow coming toward and leaving the intersection. The camera was installed on existing poles and mounted perpendicular to the ground. The perpendicular mounting allowed the video image to be relatively distortion free to the horizon in all directions. The camera was mounted approximately 6 meters (20 feet) above the ground. This mounting height provides a focal plane of approximately 40.5 meters by 54.0 meters (133 feet by 177 feet). The camera feed went in to a TV/VCR unit placed in a recycled traffic signal controller cabinet. All the equipment was mounted on a single pole. The video images were recorded on standard VHS videotapes. See Figure 1 for details (13, 14). The traffic counts from the

intersection were video taped for two six-hour sessions from 7:00 AM to 1:00 PM and from 1:00 PM to 7:00 PM on normal week days for the before and after conditions. A normal day in this study refers to a day with no adverse environmental/weather or any external factor(s), such as special events in the nearby locality of the study intersection that would impact the flow of traffic through the study intersection.



FIGURE 1. Camera and TV/VCR Units Used in Data Collection

Phase 2: Visual Data Collection

In this phase the data was visually collected from the videotapes. All the videotapes were studied visually to extract the traffic volumes and turning movements for the analysis. Various graduate student research assistants in the Department of Civil Engineering at KSU did the data extraction from the videotapes. Every vehicle coming from all the approaches for a period of 15 minutes was recorded on pre-prepared data collection sheets. Hourly counts were used as input data for analysis using the computer program Signalized and Unsignalized Intersection Design and Research Aid (aaSIDRA) (15).

Software Selection

The software used for data analysis is aaSIDRA, Version 2.0. The Australian Road Research Board (ARRB), Transport Research Ltd., originally developed the SIDRA package as an aid for design and evaluation of intersections such as signalized intersections; roundabouts, two-way stop control, and yield-sign control intersections. SIDRA was taken over by a private company that now supports the software. aaSIDRA 2.0 is the latest version.

In evaluating and computing the performance of intersection controls there are some advantages that the SIDRA model has over any other software model. The SIDRA method emphasizes the consistency of capacity and performance analysis methods for roundabouts, sign-controlled, and signalized intersections through the use of an integrated modeling framework. Another strength of SIDRA is that it is based on the *U.S. Highway Capacity Manual* as well as Australian Road Research Board research results. (16)

The input to the software includes the road geometry, traffic counts, turning movements, and speed of the vehicles. The SIDRA software analyzes the data and the output provides measures of effectiveness from which the performance of the roadway can be determined. There are 19 measures of effectiveness given in SIDRA output but only four of them were considered relevant to the project. The four measures of effectiveness (MOEs) used in evaluating the performance are as follows:

- Carbon Monoxide (CO)
- Carbon Dioxide (CO₂)
- Nitrogen Oxides (NO_x)
- Hydrocarbons (HC) or VOCs

SIDRA uses a four-mode elemental model for estimating fuel consumption, operating cost and pollutant emissions for all types of traffic facilities. This helps with estimation of air quality, energy and cost implications of alternative intersection design. For this purpose, a unique vehicle drive-cycle model (acceleration, deceleration, idling, cruise) is used. See Figure 2 for details (17).

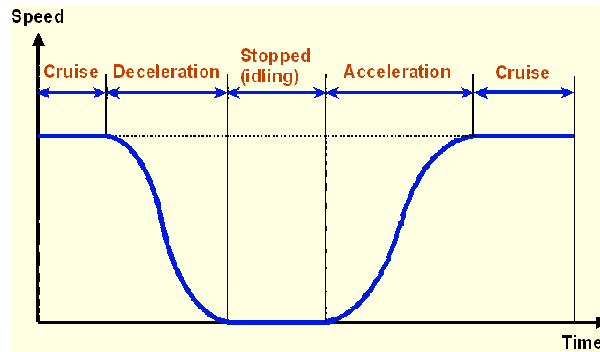


FIGURE 2. Graphical Representation of Drive-Cycle Model Used by SIDRA

Fuel consumption and emission rates are calculated from a set of equations which use such vehicle parameters as mass and fuel emission efficiency rates, as well as road grade and relevant speeds (cruise, initial, final).

Data Analysis

The data collected from videotapes for the AM and PM periods was recorded manually in 15-minute periods, and hourly data was then input to the SIDRA software for analysis. All the MOEs were statistically compared using standard statistical procedures. Minitab 13 was the software used to perform the statistical tests. The data analysis was done separately for the AM and PM hourly volumes but the procedure followed was the same for both sets of data. This was done to see whether the results differed due to the differences in before and after traffic volumes for both AM and PM traffic counts, as there was more traffic during the PM period than during the AM period.

RESULTS

The statistical analysis of the MOEs helps determine if and how the Stop controlled Intersections and the Roundabout controlled Intersections differed in cutting down vehicular emissions. The analysis provides information to assess characteristics of the Stop Controls and the Roundabout. The statistical testing was done separately for the AM and PM periods for all the locations in order to evaluate the operation of the intersection during these separate periods. The results obtained for each site are then averaged and the overall results are given in Table 2.

TABLE 2. Overall Emissions Results

EMISSIONS RESULTS				
AM Results				
Measures Of Effectiveness	SC	R.A	% Diff.	Statistically Different
Carbon Monoxide (CO) Kg/Hr	9.77	7.67	-21%	Yes
Carbon Dioxide (CO2) Kg/Hr	138.91	117.18	-16%	Yes
Oxides Of Nitrogen (NOX) Kg/Hr	0.31	0.25	-20%	Yes
HydroCarbons (HC) Kg/Hr	0.23	0.19	-18%	Yes
PM Results				
Measures Of Effectiveness	SC	R.A	% Diff.	Statistically Different
Carbon Monoxide (CO) Kg/Hr	11.8225	6.855	-42%	Yes
Carbon Dioxide (CO2) Kg/Hr	335.7	138	-59%	Yes
Oxides Of Nitrogen (NOX) Kg/Hr	0.3875	0.2015	-48%	Yes
HydroCarbons (HC) Kg/Hr	0.662375	0.23	-65%	Yes

Note: SC: AWSC or TWSC, RA: Roundabout

- The average Carbon Monoxide (CO) emissions (Kg/hr) for the intersection locations studied are 21 percent and 42 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in CO emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Carbon Dioxide (CO₂) emissions (Kg/hr) for the intersection locations studied are 16 percent and 59 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in CO₂ emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Oxides of Nitrogen (Nox) emissions (Kg/hr) for the intersection locations studied are 20 percent and 48 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in NOx emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The average Hydrocarbons (HC) emissions (Kg/hr) for the intersection locations studied are 18 percent and 65 percent less for the AM period and PM periods respectively for the case of a modern roundabout. Statistical tests showed that the decrease in HC emissions after a roundabout was installed is statistically different from the emissions that occurred in case of AWSC for both AM and PM conditions.
- The results from SIDRA analysis also showed that there was a statistically significant decrease in delay, queuing and stopping after the modern roundabout was installed when compared to the before (AWSC/TWSC) because, as previous studies have concluded, the modern roundabouts have less delay, queuing and stopping than an AWSC/TWSC. This is reflected in the decrease in vehicular emissions shown above.

CONCLUSIONS

- The modern roundabouts in Kansas operated more effectively than the before intersection control (AWSC/TWSC) in reducing vehicular emissions at all locations studied.
- There was a (21 percent to 42 percent) decrease in the Carbon Monoxide (CO) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (16 percent to 59 percent) decrease in the Carbon Dioxide (CO₂) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (20 percent to 48 percent) decrease in the Oxides of Nitrogen (NO_x) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- There was a (18 percent to 65 percent) decrease in the Hydrocarbons (HC) emissions (Kg/hr) for the AM and PM periods after the installation of modern roundabout. The decrease was observed to be statistically significant for both periods.
- Reduction in delays, queues and proportion of vehicle stopped at the intersection in the case of roundabouts suggest that roundabouts enhanced the operational performance of the intersections and account for the reduction in vehicular emissions.
- Since all the locations had a range of different traffic conditions, it is reasonable to suggest that a modern roundabout may be the best intersection alternative to reduce vehicular emissions for several other locations in Kansas with similar ranges of traffic volumes.

Overall Conclusion

Considering the above summary, it is concluded that at the intersections studied the modern roundabouts studied significantly reduced the vehicular emissions of the intersections studied by making the traffic flow orderly.

Further Study

Further studies should be conducted in other locations in United States with different traffic conditions, particularly those where volumes are high enough that a multi-lane roundabout is required, in order to get a much clearer picture. Also, field studies should be conducted using emissions detection equipment to further verify the results obtained from SIDRA.

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APPENDIX E

RESIDENTIAL STREET ANALYSIS

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City Traffic Counters
626.256.4171

Site Code : 000000000000
Start Date: 08/21/2008
File I.D. : E:\ALL FOLDER
Page : 1

Street name :Oak Springs Canyon Rd Cross street:Bt. Whitewater Canyon & Comet Way ,

Begin Time	<----- North		-----><----- South		-----><----- Combined		----->		Thursday
	A.M.	P.M.	A.M.	P.M.	A.M.	P.M.	A.M.	P.M.	
12:00 08/21	0	1	0	1	0	2			
12:15	0	2	0	1	0	3			
12:30	0	3	0	2	0	5			
12:45	0	*	1	7	0	*	3	13	
01:00	0	1	0	0	0	1			
01:15	1	3	0	0	1	3			
01:30	0	0	0	0	0	0			
01:45	0	1	0	4	0	*	1	5	
02:00	0	2	0	0	0	2			
02:15	0	1	0	0	0	1			
02:30	0	1	0	2	0	3			
02:45	0	*	4	8	0	*	8	14	
03:00	0	3	0	0	0	3			
03:15	0	1	0	0	0	1			
03:30	0	3	0	0	0	3			
03:45	1	1	3	10	0	*	1	11	
04:00	0	1	0	0	0	1			
04:15	1	3	0	0	1	3			
04:30	0	2	0	2	0	4			
04:45	1	2	2	8	0	*	2	12	
05:00	0	0	0	0	0	0			
05:15	0	1	0	0	0	1			
05:30	1	1	0	0	1	1			
05:45	0	1	0	2	0	*	1	3	
06:00	0	3	2	0	2	3			
06:15	1	3	0	0	1	3			
06:30	3	4	0	0	3	4			
06:45	0	4	0	10	1	3	*	7	10
07:00	0	0	1	0	1	0			
07:15	1	1	0	0	1	1			
07:30	1	0	0	0	1	0			
07:45	1	3	0	1	1	4	0	1	
08:00	1	1	1	0	2	1			
08:15	1	0	0	2	1	2			
08:30	2	1	0	0	2	1			
08:45	0	4	0	2	3	8	0	4	
09:00	2	1	0	0	2	1			
09:15	0	0	0	1	0	1			
09:30	1	2	0	0	1	2			
09:45	1	4	1	4	0	*	1	5	
10:00	1	2	0	0	1	2			
10:15	1	0	0	0	1	0			
10:30	0	1	0	0	0	1			
10:45	1	3	1	4	0	*	1	4	
11:00	0	0	0	0	0	0			
11:15	1	1	1	0	2	1			
11:30	0	0	3	0	3	0			
11:45	2	3	1	2	3	8	1	2	
Totals	26	62	13	22	39	84			
Day Totals		88		35		123			
Split %	66.6%	73.8%	33.3%	26.1%					

Peak Hour	07:45	02:45	11:00	12:00	08:00	02:15
Volume	5	11	5	6	8	15
P.H.F.	.62	.68	.41	.75	.66	.46

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