

TECHNICAL MEMORANDUM – ALTERNATIVE 6&7 TECHNICAL ANALYSIS

Date: January 19, 2015

To: Jeremy Ghent
Operations and Planning Manager
County of San Luis Obispo

Phone No.: 805-781-1406

From: Ryan Hayes

Phone No.: 805-544-0707

Pages: 42 (Not including appendices)

Project No.: WBS 300150

Jurisdiction: County of San Luis Obispo

Subject: Summary of traffic conditions for two selected alternatives under Existing and Buildout conditions at the US 101 / Main Street Interchange in San Luis Obispo County. Memorandum, in addition to Executive Summary includes the following:

- 1.) Introduction
- 2.) Intersection Analysis Methodology
- 3.) Existing Conditions Summary
- 4.) Buildout Conditions Summary
- 5.) Existing Conditions with Interchange Alternatives
- 6.) Buildout Conditions with Interchange Alternatives
- 7.) Intersection ILV Analysis
- 8.) Conclusions and Recommendations



TABLE OF CONTENTS

| | |
|---|----|
| Executive Summary | 1 |
| 1.0 Introduction..... | 4 |
| 1.1 Interchange Alternatives | 4 |
| 2.0 Intersection Analysis Methodology | 6 |
| 2.1 Level of Service Ratings..... | 6 |
| 2.2 Level of Service Threshold Criteria..... | 9 |
| 2.3 Level of Service Analysis | 9 |
| 2.4 Signal Warrant Analysis | 10 |
| 2.5 Queuing Analysis..... | 10 |
| 2.6 Traffic Operation Inputs and Assumptions..... | 11 |
| 2.7 ILV Analysis | 11 |
| 3.0 Existing Conditions..... | 12 |
| 4.0 Buildout Conditions | 16 |
| 5.0 Existing Conditions with Interchange Alternatives | 16 |
| 5.1 Traffic Volumes and Intersection Lane Geometries | 16 |
| 5.2 Traffic Operations Analysis..... | 18 |
| 6.0 Buildout Conditions with Interchange Alternatives | 26 |
| 6.1 Traffic Volumes and Intersection Lane Geometries | 26 |
| 6.2 Traffic Operations Analysis..... | 30 |
| 7.0 Intersection ILV Analysis..... | 38 |
| 8.0 Interchange Alternatives Impacts and Mitigations | 39 |
| 8.1 Alternative 6 | 40 |
| 8.2 Alternative 7 | 40 |

Tables

| | | |
|-----|---|----|
| 1. | Level of Service Characteristics | 9 |
| 2. | ILV Traffic Flow Characteristics..... | 11 |
| 3. | Existing and Buildout LOS Results | 12 |
| 4. | Existing and Buildout Queue Results | 15 |
| 5. | Existing Conditions with Alternative 6 LOS Results | 23 |
| 6. | Existing Conditions with Alternative 6 Queue Results | 24 |
| 7. | Existing Conditions with Alternative 7 LOS Results | 25 |
| 8. | Existing Conditions with Alternative 7 Queue Results | 26 |
| 9. | Buildout Conditions Signal Warrant Analysis Results..... | 27 |
| 10. | Buildout Conditions with Alternative 6 LOS Results | 33 |
| 11. | Buildout Conditions with Alternative 6 Queue Results..... | 34 |
| 12. | Buildout Conditions with Alternative 7 LOS Results | 35 |
| 13. | Buildout Conditions with Alternative 7 Queue Results..... | 37 |
| 14. | Buildout Conditions Intersection ILV Analysis Results | 38 |
| 15. | Existing Conditions LOS Summary..... | 39 |
| 16. | Buildout Conditions LOS Summary..... | 40 |

Exhibits

| | | |
|----|--|----|
| 1. | Project Vicinity | 5 |
| 2. | Alternative 6: Westside Roundabout and Eastside Diamond Interchange | 7 |
| 3. | Alternative 7: Westside Roundabout and Eastside NB Ramps Reconfiguration | 8 |
| 4. | Existing Conditions Intersection Lane Configuration | 13 |
| 5. | Existing Traffic Volumes..... | 14 |
| 6. | Buildout Traffic Volumes..... | 17 |

| | | |
|-----|--|----|
| 7. | Existing Conditions with Alternative 6 Traffic Volumes | 19 |
| 8. | Existing Conditions with Alternative 6 Intersection Lane Configuration | 20 |
| 9. | Existing Conditions with Alternative 7 Traffic Volumes | 21 |
| 10. | Existing Conditions with Alternative 7 Intersection Lane Configuration | 22 |
| 11. | Buildout with Alternative 6 Traffic Volumes | 28 |
| 12. | Buildout with Alternative 6 Intersection Lane Configuration | 29 |
| 13. | Buildout with Alternative 7 Traffic Volumes | 31 |
| 14. | Buildout with Alternative 7 Intersection Lane Configuration | 32 |
| 15. | Buildout with Alternative 7 Recommended Lane Configuration | 42 |

Appendices

Appendix A – HCM 2000 LOS Methodology

Appendix B – Existing Conditions with Alternatives LOS and Queue Analysis Worksheets

Appendix C – Buildout Signal Warrant Analysis Worksheets

Appendix D – Buildout Conditions with Alternatives LOS and Queue Analysis Worksheets

Appendix E – Buildout ILV Analysis Worksheets

EXECUTIVE SUMMARY

Increasing traffic is causing escalations in congestion and safety concerns at the US 101 / Main Street interchange in the Templeton Community of unincorporated San Luis Obispo County. This study is intended to address some of these concerns through detailed traffic operations analysis. Main Street, classified as 2-lane Arterial, currently connects the US 101 via a tight diamond interchange with frontage roads (Ramada Drive and Theatre Drive) intersecting about 40-50 feet from the ramp intersections. The County monitors traffic operations in the Templeton area and documents the results in the Templeton Traffic Circulation Study. The most recent study done in 2009 indicates that the Main Street / US 101 intersections operate at deficient level of service, triggering need for further evaluation.

In October of 2012, Rick Engineering Company (RICK) prepared a traffic study (Deliverable 3) analyzing five improvement alternatives at this intersection. The current Technical Memorandum (Deliverable 4) analyzes two (2) additional improvement alternatives selected by the County under existing and buildout traffic conditions at the US 101 / Main Street Interchange. These alternatives include:

Alternative 6: Roundabout on west side of Hwy 101; Modified Diamond Interchange on east side of Hwy 101.

Alternative 7: Roundabout on west side of Hwy 101; Combined NB Ramps and Ramada Drive intersection on east side of Hwy 101.

Additionally, the lane configuration recommendations in Deliverable 3 were assumed when analyzing Alternative 6 buildout conditions. Based on those recommendations, the following improvements were incorporated:

- A dedicated right turn lane on the westbound Main Street approach at the US 101 northbound ramps, with the right turn lane extended from US 101 northbound ramps to Ramada Drive creating a four-lane cross section.
- An additional shared through-right turn lane on the westbound Main Street approach at Ramada Drive.
- Main Street expanded to a 4-lane roadway between the northbound ramps and Ramada Drive intersections.
- Dual approach lanes for the southbound Ramada Drive approach at Main Street, 300' prior to intersection.

Based on the traffic analysis for Alternatives 6, no additional improvements are necessary to improve queuing or traffic operations within the project area.

For Alternative 7 buildout conditions, the following improvements were incorporated in the analysis:

- Exclusive left turn lanes for all approaches at Main Street and US NB 101 ramps – Ramada Drive.

Based on the traffic analysis for Alternatives 7 the following additional improvements are recommended to improve queuing and traffic operations within the project area:

1. Add a dedicated 200' right turn lane on the southbound approach Ramada Drive at Main Street and increase the left turn lane storage length to 250'.
2. Add a second 300' dedicated left turn lane on the northbound US 101 ramp approach at Main Street. This would require the westbound departure approach on Main Street to provide for two receiving lanes (to accept dual northbound left turn lanes) and the transition back to one lane prior to the existing bridge.
3. Add a dedicated 300' right turn lane on the westbound Main Street approach at Ramada Drive/US 101 northbound ramps, and increase the westbound left turn storage lane to 300'.
4. Increase the eastbound left turn storage lane to 200'.

Deliverable 1 evaluated the existing traffic conditions at the interchange (dated July 1, 2011). The second technical memorandum (Deliverable 2, dated July 1, 2011) evaluated traffic conditions under buildout conditions within the area, with no changes to the existing roadway infrastructure or geometrical layout. As noted above, the third technical memorandum (Deliverable 3, dated October 9, 2012) evaluated traffic conditions under existing and buildout conditions for five improvement alternatives.

This fourth memorandum develops and evaluates the peak hour traffic volumes and lane geometrics under existing and buildout conditions for the two additional selected improvement alternatives. These improvements alternatives were selected by the County based upon analysis and recommendations included in Deliverable 3. The evaluation of traffic conditions included an analysis of Levels of Service (LOS) and vehicle queues at the study intersections. The intersection LOS analysis was based on the Highway Capacity Manual, while queuing analysis was performed using simulation runs.

Existing conditions operations analysis with both alternatives indicated that all of the study intersections satisfy the Caltrans LOS threshold criteria, and traffic conditions were improved relative to the existing interchange traffic conditions.

Existing conditions queuing analysis under each alternative showed no major spillback or backups within the study area.

The buildout conditions operations analysis with the assumed lane geometrics indicated that most of the study intersections under each alternative would operate with acceptable overall LOS, satisfying the Caltrans and County's LOS threshold criteria. The only exception was the intersection of Main Street/US 101 northbound ramps and Ramada Drive under Alternative 7, which is projected to operate at LOS E, thus failing to satisfy County's threshold criteria. Also, under the same alternative, a few individual movements at the study intersections would experience delays in the range of LOS D-F.

Similarly, the 95th percentile queues exceeded the assumed or estimated storage at only a few locations. Under Alternative 6, the estimated 95th percentile queues were reported to be accommodated within the available or assumed storage lengths. Under Alternative 7, the 95th percentile queues on the left turn pockets at all approaches at the Main Street/ US 101 northbound-Ramada Drive intersection would exceed the storage capacity ranging from 1% to 46% of the time during the AM and PM peak hours.

1.0 INTRODUCTION

As requested by the County of San Luis Obispo, Rick Engineering Company (RICK) has prepared this technical memorandum analyzing the existing and buildout traffic conditions at the US 101 / Main Street Interchange for two (2) additional improvement alternatives in the Templeton Community of unincorporated San Luis Obispo County. **Exhibit 1** shows a vicinity map with the study interchange and the surrounding roadway network system. The alternatives include:

Alternative 6: Single Roundabout west of Hwy 101 (includes minor realigning frontage roads); Modified Diamond Interchange east of Hwy 101 (includes realigning frontage roads)

Alternative 7: Single Roundabout west of Hwy 101 (includes minor realigning frontage roads); Combined NB Ramps Interchange east of Hwy 101 (includes new NB hook onramp)

This is the fourth technical memorandum evaluating the traffic conditions at the US 101 / Main Street Interchange. Deliverable 1 was a technical memorandum that evaluated the existing traffic conditions at the interchange (dated July 1, 2011). The second technical memorandum, Deliverable 2 (dated July 1, 2011) evaluated traffic conditions under the buildout scenario within the area, with no changes to the existing roadway infrastructure or geometrical layout. The second technical memorandum also provided an evaluation of various short term traffic control improvement alternatives (i.e.; all-way stop control, signalization, etc). Deliverable 1 evaluated the existing traffic conditions at the interchange (dated July 1, 2011). The third technical memorandum (dated October 9, 2012) evaluated traffic conditions under existing and buildout conditions for five (5) improvement alternatives.

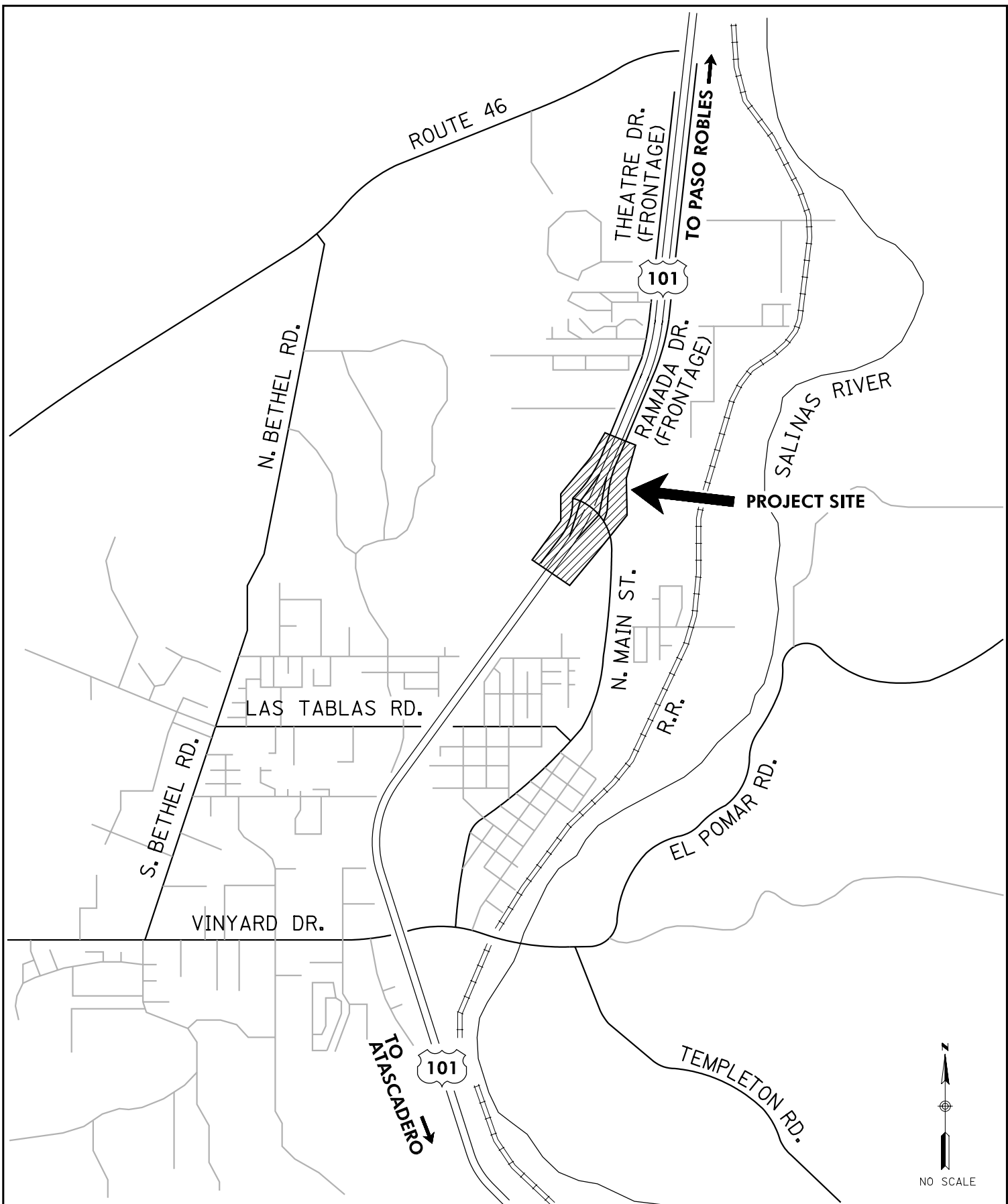
This memorandum includes the development of the AM and PM peak hour traffic volumes under the existing and buildout conditions for both improvement alternatives. The existing and buildout traffic conditions for each alternative are evaluated utilizing the proposed layout of roadway and intersection geometrics defined by the County. The evaluation of traffic conditions includes an analysis of Levels of Service (LOS) and vehicle queues at the various interchange study intersections. Several recommendations are also made to improve traffic flows for each interchange alternative.

1.1 Interchange Alternatives

This traffic analysis evaluates two (2) improvement alternatives to reduce congestion and delays at the US 101 / Main Street interchange under existing and buildout conditions. The following is a description of each alternative considered in this memorandum.

Alternative 6 (Single Roundabout and Fully Compliant Interchange)

This alternative would eliminate the two existing intersections west of Hwy 101 and construct a single roundabout. This roundabout would serve the US 101 southbound ramps, Main Street and Theater Drive. The roundabout would be a six-legged single-lane roundabout, and would require significant reconstruction of both ramps, as well as modifications of all streets at their intersections with the new roundabout.



© 2000 Rick Engineering Company



EXHIBIT 1
 PROJECT VICINITY
 US 101/MAIN STREET TRAFFIC STUDY

East of Hwy 101, this alternative would relocate Ramada Drive approximately 300' to the east to meet the Caltrans current advisory spacing requirements between intersections. As a result, the spacing between each intersection along Main Street (include the Roundabout west of Hwy 101) within the study area would be 500 feet. The new intersection of Main Street and Theatre Drive would be a "T" intersection. It should be noted that no bridge widening is planned under this alternative. **Exhibit 2** depicts the preliminary conceptual layout for alternative 6. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

Alternative 7 (Single Roundabout and NB Hwy 101 Ramps Reconstruction)

This alternative would eliminate the two existing intersections west of Hwy 101 and construct a single roundabout. This roundabout would serve the US 101 southbound ramps, Main Street and Theater Drive. The roundabout would be a six-legged single-lane roundabout, and would require significant reconstruction of both ramps, as well as modifications of all streets at their intersections with the new roundabout.

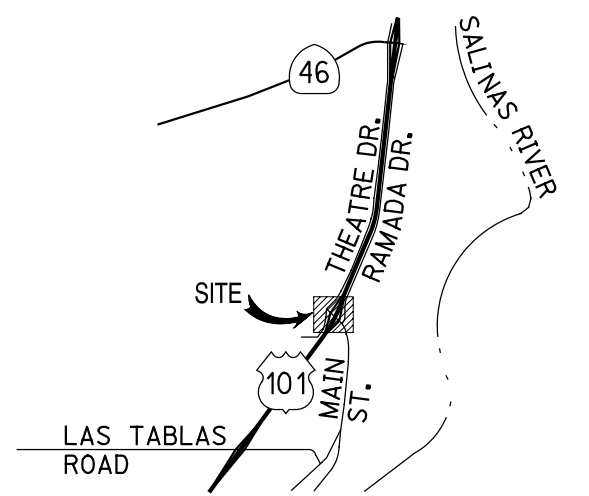
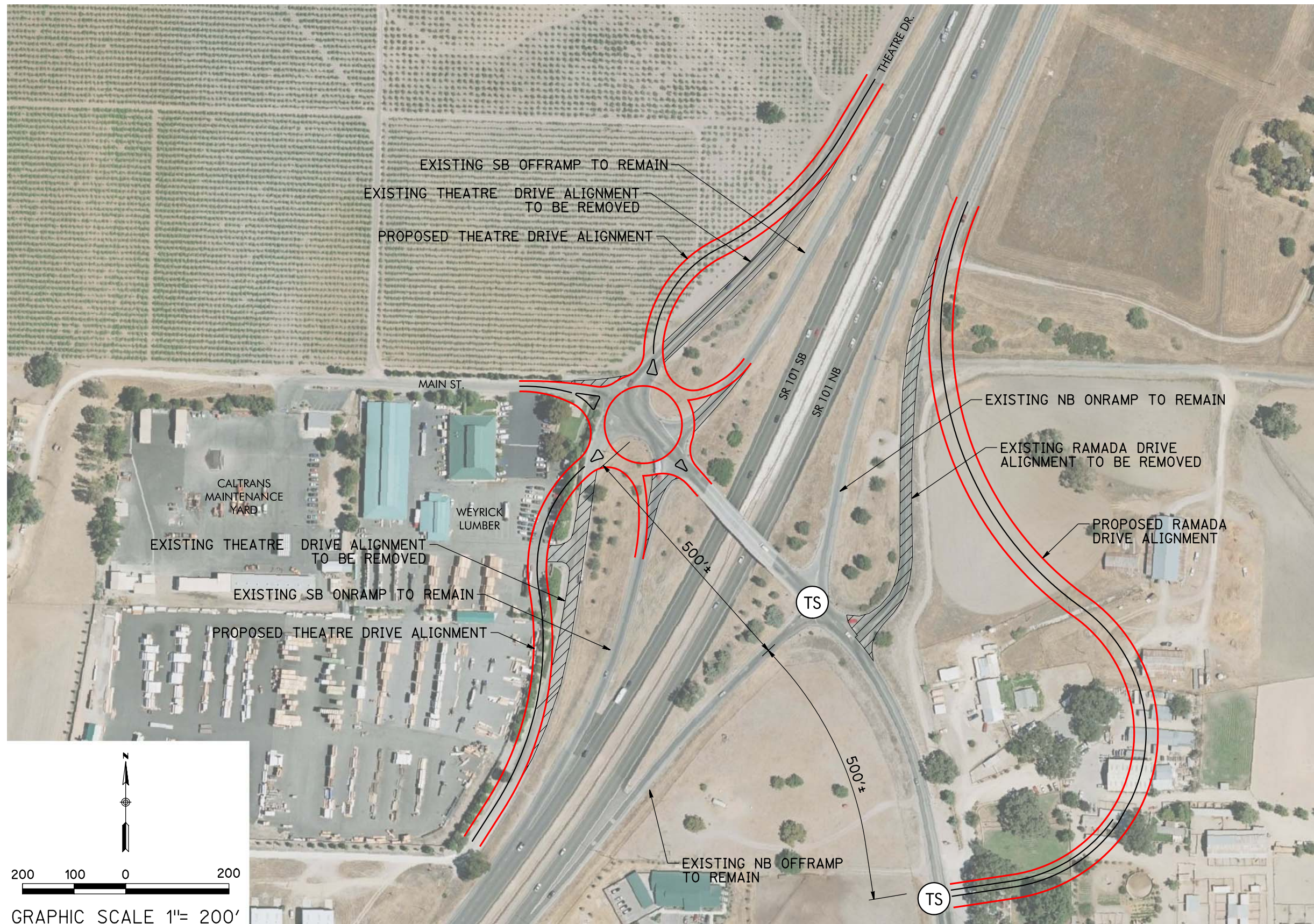
East of Hwy 101, this alternative would combine the dual intersections into a single intersection by reconstructing the northbound freeway ramps. The northbound onramp would be rebuilt as a hook ramp, passing underneath the existing Main St. Overcrossing. This alternative would require work on the existing bridge structure, either by relocating the bridge support or shoring the bridge abutment to allow the new ramp two pass beneath the bridge support and abutment. **Exhibit 3** depicts the preliminary conceptual layout for Alternative 7. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

2.0 INTERSECTION ANALYSIS METHODOLOGY

2.1 Level of Service Ratings

Level of Service (LOS) ratings are quantitative descriptions of intersection operations and are reported using an "A" through "F" letter rating system to describe vehicle delays and congestion. LOS A indicates free-flow conditions with little or no delay and LOS F indicates forced-flow conditions with excessive delays and queues. **Table 1** provides a brief description of the LOS characteristics. **Appendix A** contains the Highway Capacity Manual 2000 (HCM 2000) tables illustrating the LOS-to-delay relationship data for intersection operations (i.e.: two-way stop controlled, all-way stop controlled and signalized intersections).

The peak hour LOS values for the entire intersection operations are based on the estimated "weighted average" vehicle delays. The LOS values are also reported for the various critical movements (i.e.: stop sign approach, main line left-turns, etc.), which are based on the estimated delays for the individual approach and/or movement. Typically, Caltrans uses the "average" control delay for reporting an intersection Measure of Effectiveness (MOE). However, the LOS analyses performed for unsignalized intersections utilizes the lowest performing critical movement LOS for determining when improvements are warranted, consistent with County methodology used in the Templeton Circulation Study.



VICINITY MAP
NO SCALE

NOTES:
1. ROUNDABOUT PRELIMINARY RADIUS OF 75' PER TY LIN DESIGN.

- LEGEND:
- NEW ROADWAY
 - EXISTING ROADWAY TO BE REMOVED

| RECOMMENDED DESIGN STANDARDS* | |
|---------------------------------------|-----------------------------|
| DESIGN SPEED | 70 MPH (FREEWAY) |
| | 45 MPH (LOCAL) |
| | 50 MPH (RAMP AT EXIT NOSE) |
| INTERCHANGE SPACING | 25 MPH (RAMP AT LOCAL ROAD) |
| | MIN. 400' (PREFERRED 500') |
| DISTANCE BETWEEN SUCCESSIVE OFF-RAMPS | 1000' |
| LOOP RAMP RADIUS | 120'-300' |

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL

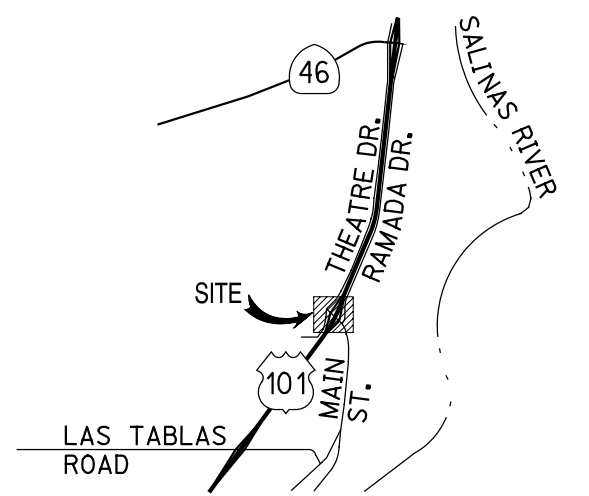
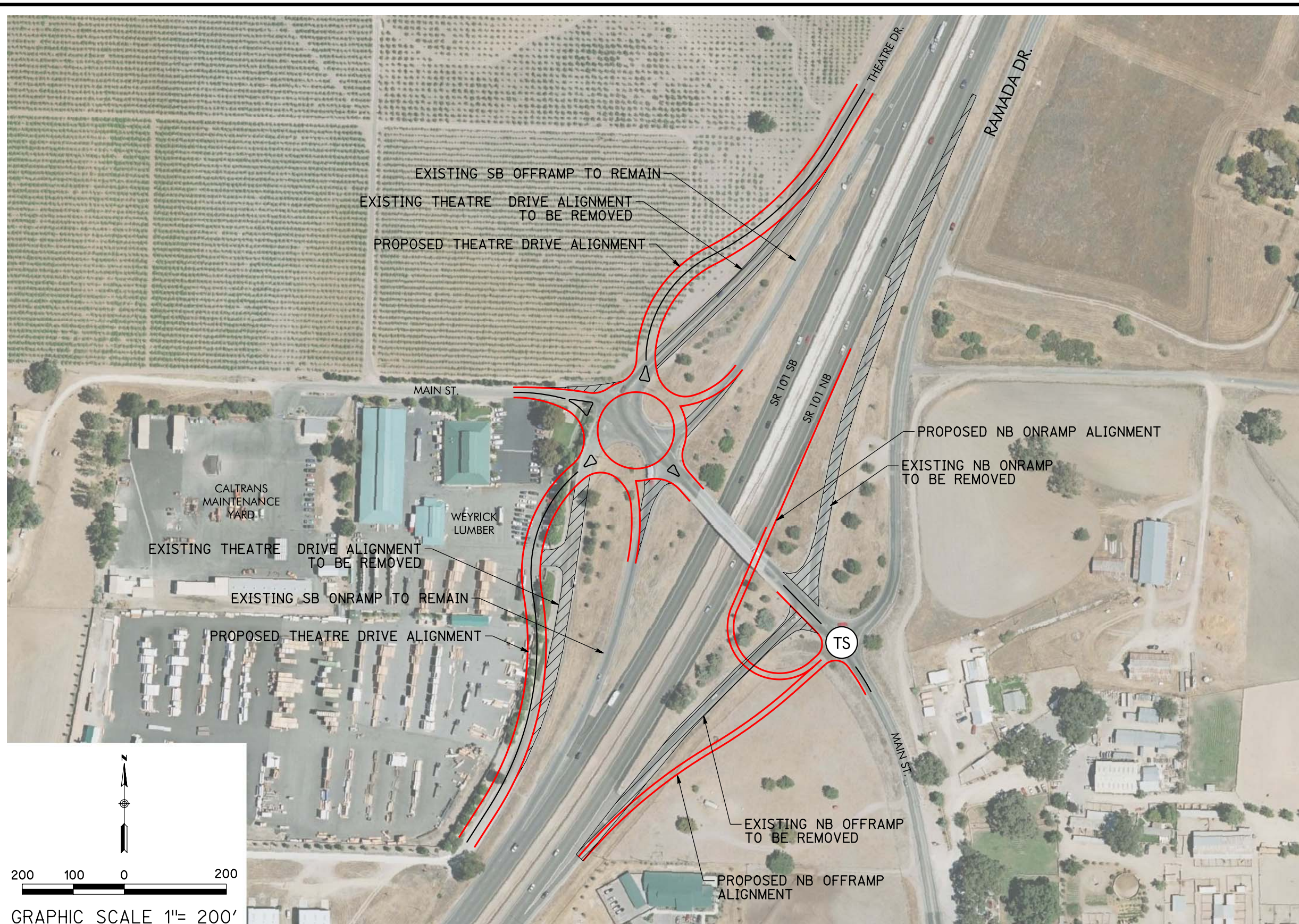
J-16128

RICK ENGINEERING COMPANY

711 TANK FARM ROAD - SUITE 110
SAN LUIS OBISPO, CA 93401
805.544.0707
(FAX)805.544.2052

MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS
ALTERNATIVE #6

| | |
|-------------------|--------------|
| Date: 12-NOV-2014 | Sheet Number |
| Drawn By: RDH | 1 |
| Chkd By: DAD | Of 2 Sheets |
| Scale: 1"=200' | |



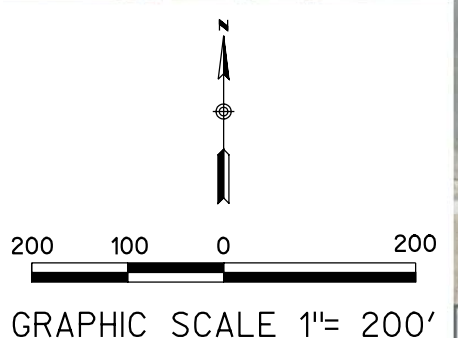
VICINITY MAP
NO SCALE

NOTES:
1. ROUNDABOUT PRELIMINARY RADIUS OF 75' PER TY LIN DESIGN.

- LEGEND:
- NEW ROADWAY
 - EXISTING ROADWAY TO BE REMOVED

| RECOMMENDED DESIGN STANDARDS* | |
|---------------------------------------|-----------------------------|
| DESIGN SPEED | 70 MPH (FREEWAY) |
| | 45 MPH (LOCAL) |
| | 50 MPH (RAMP AT EXIT NOSE) |
| INTERCHANGE SPACING | 25 MPH (RAMP AT LOCAL ROAD) |
| | MIN. 400' (PREFERRED 500') |
| DISTANCE BETWEEN SUCCESSIVE OFF-RAMPS | 1000' |
| LOOP RAMP RADIUS | 120'-300' |

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL



J-16128

RICK
ENGINEERING COMPANY

711 TANK FARM ROAD - SUITE 110
SAN LUIS OBISPO, CA 93401
805.544.0707
(FAX)805.544.2052

MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS
ALTERNATIVE #7

| | |
|-------------------|--------------|
| Date: 12-NOV-2014 | Sheet Number |
| Drawn By: RDH | 2 |
| Chkd By: DAD | Of 2 Sheets |
| Scale: 1"=200' | |

©2014 Rick Engineering Company

TABLE 1
LEVEL OF SERVICE CHARACTERISTICS

| LOS | Characteristics |
|-----|--|
| A | Free flow conditions exist. Each individual driver is virtually unaffected by the presence of others in the traffic stream. |
| B | Stable traffic flow exists. The individual drivers have the freedom to select a desired speed, but encounter a slight decline in the freedom to maneuver. |
| C | Stable and acceptable flow exists, but speed and maneuverability are somewhat restricted due to higher traffic volumes. The individual driver will be significantly affected by the presence of others. |
| D | High density but stable flow will occur. The individual driver will experience a generally poor level of comfort and convenience. Small increases in traffic flow will cause operational problems and restrict driver maneuverability. |
| E | Speeds are low, but relatively uniform. The individual driver's ability to maneuver becomes extremely difficult with high frustration. The traffic volume on the road is near capacity. |
| F | Forced or breakdown flow has occurred. The individual driver is stopped for long periods due to congestion. |

Source: Highway Capacity Manual, Transportation Research Board, 2000 Edition.

2.2 Level of Service Threshold Criteria

The County of San Luis Obispo has adopted LOS C threshold as the minimum standard for rural roadway operations and LOS D or better for roadways within the boundary of the Templeton Urban Reserve Line (URL). Since the US 101 / Main Street interchange is located within the URL, LOS D is the minimum acceptable standard for peak hour operations at the intersection of Main Street with Ramada Drive. When analyzing intersections which include the northbound and southbound ramps, this study uses the standards published in the Caltrans traffic study guidelines (Guide for the Preparation of Traffic Impact Studies, December 2002). These guidelines state that Caltrans endeavors to maintain a target LOS at the transition between LOS C and D range. Therefore, when analyzing the proposed roundabout design in both Alternatives, and the reconstructed intersection of Main Street with the northbound ramps and Ramada Drive in Alternative 7, LOS C will be considered the minimum acceptable standard for peak hour operations.

2.3 Level of Service Analysis

The analysis of existing and buildout peak hour operations at the study intersections was performed using methodologies contained in the Highway Capacity Manual 2000 (HCM 2000), and modeled with the "Synchro" and "SimTraffic" software (Version 8). To model buildout operations a peak hour factor (PHF) of 0.92 and a heavy vehicles proportion of 5% was applied at all intersections. The software estimates vehicle delays for the overall peak hour operations as an "average" and for each "critical" movement (i.e.: stop sign controlled approach, main line left-turns, etc).

The analysis of roundabout operations was performed using methodologies and capacity values contained in the Highway Capacity Manual 2010 (HCM 2010), and modeled with SIDRA software (Version 5.1). The capacity analysis was refined by using the following California-

specific values as recommended in the Caltrans publications *Roundabout Geometric Design Guidelines* (June 2007):

Signal-lane roundabouts: critical headway = 4.8 seconds and follow-up headway = 2.5 seconds

2.4 Signal Warrant Analysis

At each unsignalized intersection, the potential need for a traffic signal was evaluated using the peak hour warrant criteria of the latest version of the California Manual on Uniform Traffic Control Devices (CA MUTCD). The CA MUTCD states that, “*This [peak hour] signal warrant shall be applied only in unusual cases, such as office complexes, manufacturing plants, industrial complexes, or high-occupancy vehicle facilities that attract or discharge large numbers of vehicles over a short time.*” As such, the peak hour warrant is being used in this analysis study as an “indicator” of the likelihood of an unsignalized intersection warranting a traffic signal in the future. A signal may also be warranted by other criteria, some of which cannot be known until the intersection is constructed and operational. The peak hour analysis is not intended to replace a rigorous and complete traffic signal analysis by the responsible jurisdiction.

2.5 Queuing Analysis

Given that “static” analyses used for LOS computation do not explicitly address operations of closely spaced intersections, an intersection queuing analysis was performed using the micro-simulation SimTraffic software. Although simulation does capture the dynamics of queuing and its interaction between adjacent intersections, conjecture over which analysis result is more accurate must be deferred for this study. SimTraffic simulation results generally indicate poorer operations relative to the static Synchro analysis results.

SimTraffic simulation runs were based on a 10-minute seeding interval, a 60-minute simulation interval, and reflect an average of 5 runs. The 95th percentile queues which present maximum back of queues for the 95th percentile traffic volumes were estimated for each movement at the intersection. These queues were compared against the estimated or assumed available storage for the sufficiency analysis. Calculated 95th percentile queues indicate potential for queue spillback conditions onto the freeway mainline (i.e., queues exceed storage capacity of an off-ramp) and/or at upstream intersections (i.e., queues exceed storage between intersections).

It should be noted that 95th percentile queues can represent the worst case scenario and may not even be observed in the field. Given that the SimTraffic simulation results have not been validated against field observations, the 95th percentile queue results should be used with caution. The SimTraffic simulation results generally indicate poorer operations relative to the “static” analyses results, and therefore, the simulation results were used as the basis for facility sizing needs and intersection improvement recommendations. The queuing analysis for roundabouts was based on the results produced by SIDRA.

2.6 Traffic Operation Inputs and Assumptions

When traffic signal control is warranted under the buildout conditions, the minimum pedestrian timing parameters were coded on the appropriate approaches. The Synchro software was allowed to estimate the right-turn on red movements. The timings at the signalized intersections along Main Street were coordinated. The cycle lengths and offsets at each signalized intersection were optimized using the Synchro software.

When determining existing conditions in Section 3.0, it should be noted that the existing Main Street and Theatre Drive intersection has three-way stop sign control, which cannot be modeled correctly using Synchro. RICK determined that modeling the existing intersection as a two-way stop rather than an all-way stop would more closely approximate actual conditions. Since traffic westbound on Main Street currently flows freely, modeling this movement as stop-controlled would inaccurately estimate vehicle delays and queues. Eastbound traffic entering the intersection comprises a relatively small portion of the total intersection volume under existing and Buildout conditions. In addition, conflicting movements between east and westbound traffic will be minimal. Therefore, it was decided that a more accurate representation of actual operations would be obtained by utilizing the two-way stop controlled methodology.

The US 101 southbound and northbound off-ramps approaches, and the southbound Ramada Drive approach are flared at their intersection with Main Street. These flares essentially create a short separate lane that vehicles use to make right turns when the left-through movement queues do not backed up beyond the limits of the flare. Therefore, the analysis of these approaches assumes a single lane approach with a short 50' turn lane for right turn movements.

2.7 ILV Analysis

Caltrans utilizes the Intersection Capacity method contained in Section 406 of the Caltrans Highway Design Manual (HDM), 6th Edition to determine the traffic volume to intersection capacity. The Intersecting Lane Volume (ILV) method is a rough approximation of the functionality of a signalized intersection given traffic volumes. The ILV analysis was used to estimate intersection capacity, identified as being under, at or over capacity. **Table 2** provides values of ILV/hr associated with the various traffic flow thresholds.

**TABLE 2
ILV TRAFFIC FLOW CHARACTERISTICS**

| ILV/hr | Description |
|---------------|---|
| < 1200 | Stable flow with slight, but acceptable delay. Occasional signal loading may develop. Free midblock operations. |
| 1200-1500 | Unstable flow with considerable delays possible. Some vehicles occasionally wait two or more cycles to pass through the intersection. Continuous backup occurs on some approaches. |
| > 1500 | Stop-and-go operation with severe delay and heavy congestion. Traffic volume is limited by maximum discharge rates of each phase. Continuous backup in varying degrees occurs on all approaches. Where downstream capacity is restrictive, mainline congestion can impede orderly discharge through the intersection. |

Source: Highway Design Manual, Table 406, California Department of Transportation.

3.0 EXISTING CONDITIONS

As previously stated, Deliverable 1 included a detailed evaluation of existing conditions at the US 101 / Main Street Interchange (July 1, 2011). Refer to Deliverable 1 for a complete description of the Exiting Roadway Network, Existing Traffic Volumes and Analysis. **Exhibit 4** shows the existing lane geometrics and traffic controls at the study intersections. **Exhibit 5** illustrates the existing peak hour turning movement volumes at the study intersections and Average Daily Traffic (ADT) in the study area. **Table 3** provides a summary of the intersection LOS analysis presented in Deliverable 1.

**TABLE 3
EXISTING AND BUILDOUT LOS RESULTS**

| Study Intersection Main Street at: | Movement | 2009 Existing | | Buildout | |
|---------------------------------------|----------------|---------------|------|------------|-----|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| Theatre Drive (TWSC) | AM Peak | | | | |
| | EB | 1.2 | A | 1.7 | A |
| | NB | 9.3 | A | 10.1 | B |
| | SB | 12.4 | B | 29.8 | D |
| | PM Peak | | | | |
| | EB | 2.2 | A | 1.6 | A |
| NB | 8.8 | A | 11.0 | B | |
| SB | 14.5 | B | > 50 | F | |
| US 101 SB Ramps (TWSC) | AM Peak | | | | |
| | WB | 2.8 | A | 3.6 | A |
| | SB | 24.1 | C | > 50 | F |
| | PM Peak | | | | |
| WB | 4.1 | A | 6.2 | A | |
| SB | 35.5 | E | > 50 | F | |
| US 101 NB Ramps (TWSC) | AM Peak | | | | |
| | EB | 3.2 | A | 5.4 | A |
| | NB | 16.1 | C | > 50 | F |
| | PM Peak | | | | |
| EB | 1.1 | A | 2.4 | A | |
| NB | 26.4 | D | > 50 | F | |
| Ramada Drive (TWSC) | AM Peak | | | | |
| | EB | 3.8 | A | 8.1 | A |
| | SB | 12.8 | B | > 50 | F |
| | PM Peak | | | | |
| EB | 4.0 | A | 7.4 | A | |
| SB | 14.8 | B | > 50 | F | |

LOS = Level of Service; Average Delay in seconds/vehicle

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

Bold indicates that LOS exceeds significance threshold

The data in **Table 3** indicates that average vehicle delays at the study intersections are currently within acceptable limits during the peak hours (LOS C or better at the ramp intersections, and

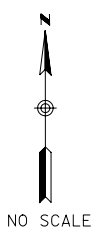
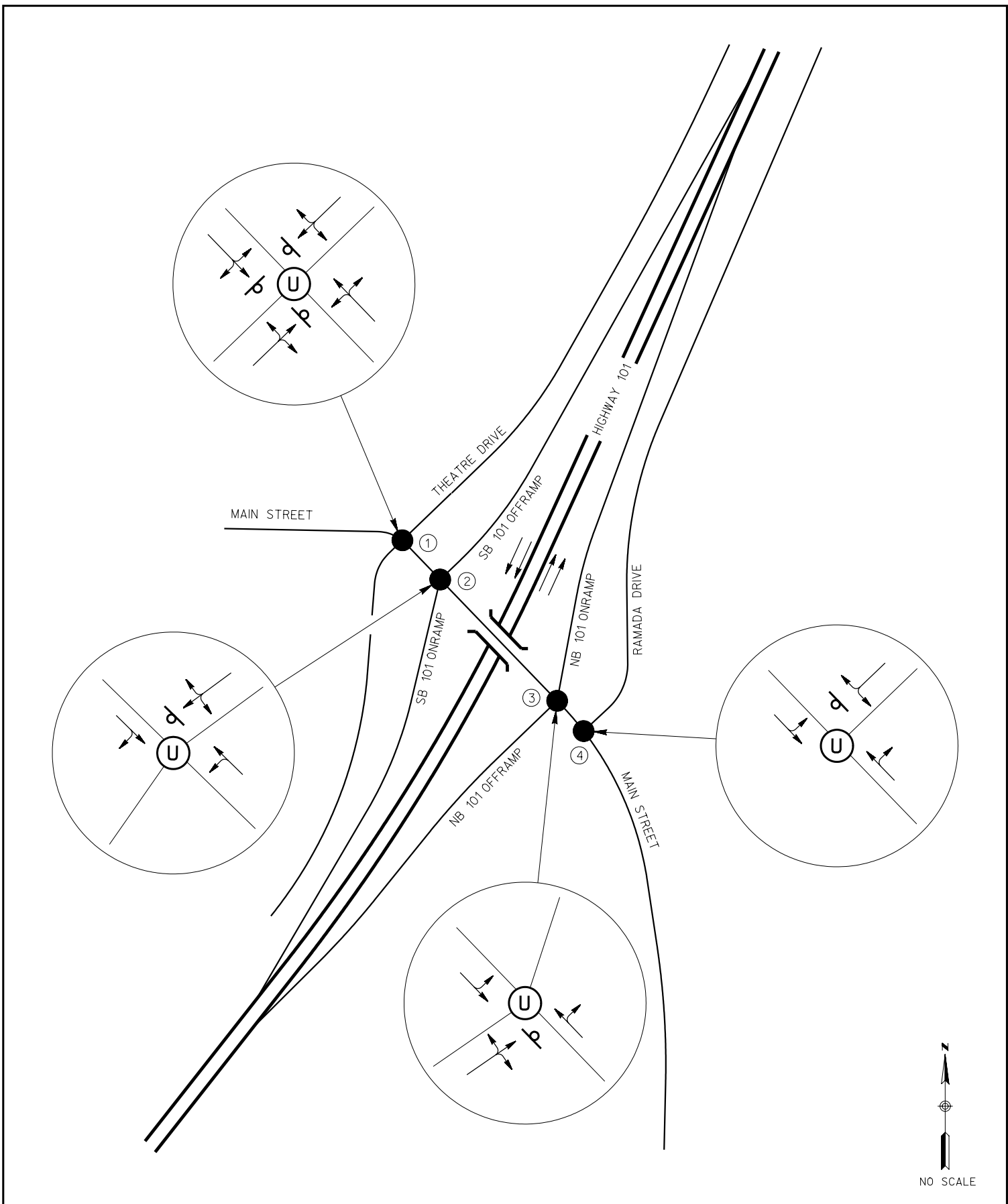


EXHIBIT 4
EXISTING CONDITIONS INTERSECTION
LANE CONFIGURATION
US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - p = STOP-CONTROLLED
 - u = UNSIGNALIZED

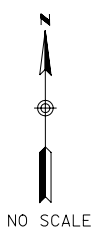
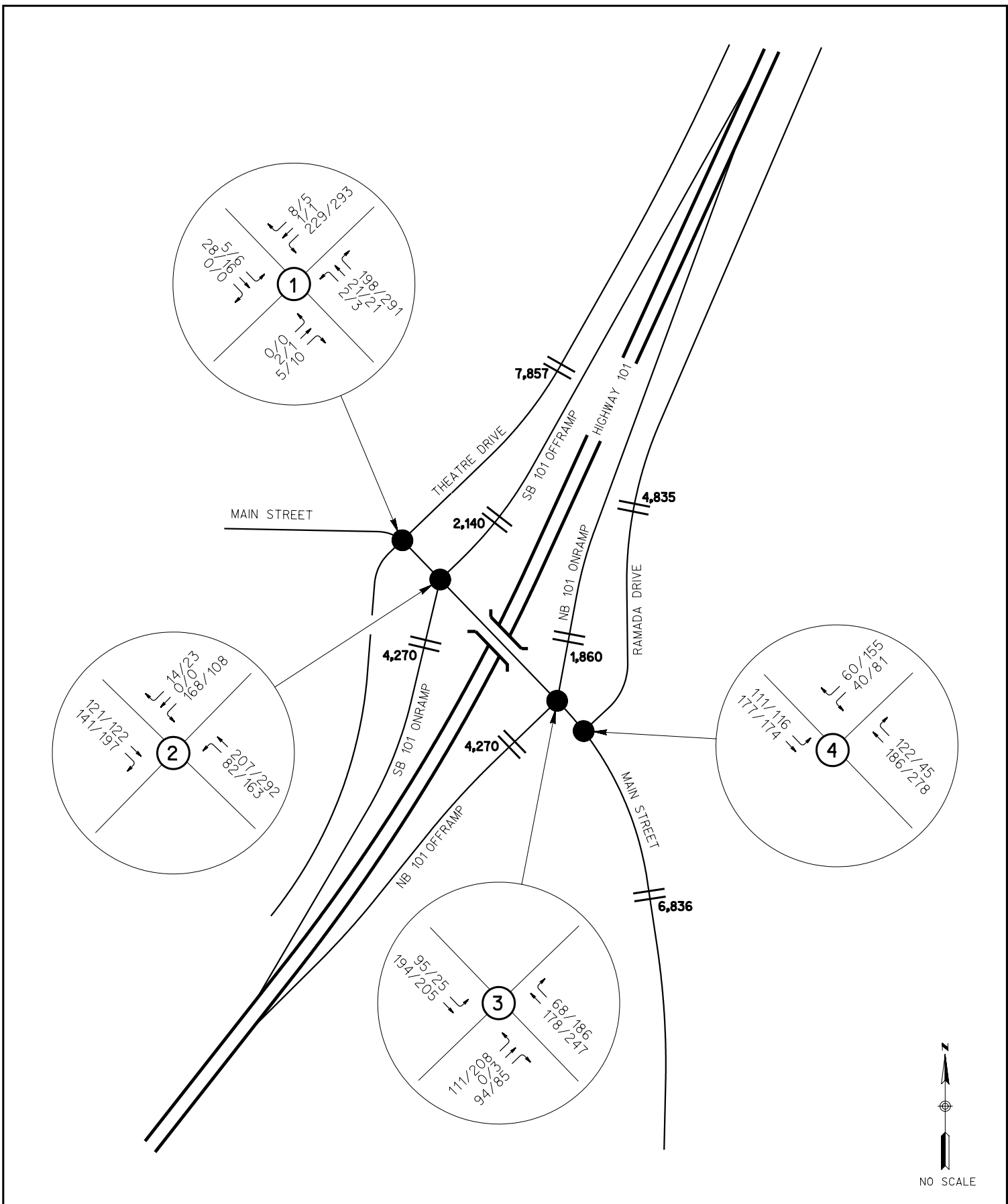


EXHIBIT 5
 EXISTING TRAFFIC VOLUMES
 US 101/NORTH MAIN STREET TRAFFIC STUDY

LEGEND:

- = STUDY INTERSECTION
- XX/XX = AM/PM PEAK HOUR TRAFFIC
- XX = AVERAGE DAILY TRAFFIC

LOS D or better at the frontage road intersections). However, delays for the US 101 north and southbound off-ramps are within the LOS D-E range during the PM peak hour. To analyze queuing lengths under existing conditions, simulations were run using the SimTraffic software within Synchro. **Table 4** summarizes the intersection queuing analysis results under Existing Conditions.

**TABLE 4
EXISTING AND BUILDOUT QUEUE RESULTS**

| Study Intersection Main Street at: | Movement | Existing Storage Length (feet) | 2009 Existing | | Buildout | |
|---------------------------------------|----------------|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| Theatre Drive (TWSC) | AM Peak | | | | | |
| | NB LTR | -- | 34 | Sufficient | 13 | Sufficient |
| | SB LTR | -- | 112 | Sufficient | 262 | Sufficient |
| | PM Peak | | | | | |
| | NB LTR | -- | 24 | Sufficient | 36 | Sufficient |
| | SB LTR | -- | 103 | Sufficient | 594 | Sufficient |
| US 101 SB Ramps (TWSC) | AM Peak | | | | | |
| | WB LT | 300 | 66 | Sufficient | 158 | Sufficient |
| | SB LTR | 1000 | 102 | Sufficient | 1088 | Insufficient |
| | PM Peak | | | | | |
| | WB LT | 300 | 108 | Sufficient | 217 | Sufficient |
| | SB LTR | 1000 | 78 | Sufficient | 1275 | Insufficient |
| US 101 NB Ramps (TWSC) | AM Peak | | | | | |
| | EB LT | 300 | 113 | Sufficient | 395 | Insufficient |
| | NB LTR | 800 | 98 | Sufficient | 1018 | Insufficient |
| | PM Peak | | | | | |
| | EB LT | 300 | 56 | Sufficient | 436 | Insufficient |
| | NB LTR | 800 | 99 | Sufficient | 1017 | Insufficient |
| Ramada Drive (TWSC) | AM Peak | | | | | |
| | EB LT | 40 | 55 | Insufficient | 65 | Insufficient |
| | SB LR | -- | 46 | Sufficient | 1373 | Sufficient |
| | PM Peak | | | | | |
| | EB LT | 40 | 42 | Insufficient | 62 | Insufficient |
| | SB LR | -- | 92 | Sufficient | 1192 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

Bold indicates that the queue spillback may be experienced

The queue analysis demonstrates that the study intersections currently have adequate storage capacity for the 95th percentile queue length on each approach, except the eastbound lane on Main Street at the Ramada Drive intersection. On this approach, traffic waiting to turn left from Main Street to northbound Ramada Drive occasionally blocks the northbound ramps intersection.

However, the analysis in Deliverable 1 concluded that the existing queues do not necessitate any improvements at the study intersections.

4.0 BUILDOUT CONDITIONS

As previously stated, Deliverable 2 included a detailed evaluation of the buildout scenario at the US 101 / Main Street Interchange (July 1, 2011). Buildout roadway traffic volumes were obtained from Templeton Circulation Study, 2009 update, completed by the County Department of Public Works and Omni-Means. The buildout volumes assume “the development of all remaining vacant parcels at maximum allowable densities under the current planning and zoning codes.”

The initial analysis in Deliverable 2 was conducted assuming no changes to the existing interchange geometrics. The analysis also included an evaluation of various short term traffic control improvement alternatives. Refer to Deliverable 2 for a complete description of the Buildout Traffic Volumes and Analysis. **Exhibit 6** illustrates the Buildout ADT and peak hour turning movement volumes at the study intersections. The result of the LOS analysis reflecting no geometric changes at the US 101 / Main Street interchange are presented in **Table 3**.

The data in **Table 3** indicates that average vehicle delays at the study intersections will be within the LOS F range during the PM peak hour. The data also demonstrates that average delays at the US 101 ramp intersections will also be within the LOS F range during the AM peak hour. Excessive delays will be experienced on the US 101 north and southbound off-ramps, and the southbound approaches of Theatre Drive and Ramada Drive.

To analyze queuing lengths under Buildout conditions, simulations were run using the SimTraffic software within Synchro. **Table 4** summarizes the intersection queuing analysis results under Buildout Conditions. The data in **Table 4** indicates that vehicle queues on both the US 101 north and southbound off-ramps will exceed the available storage and possibly backup onto the freeway main-line during the AM and PM peak hours. In addition, queues on the eastbound approach of Main Street at the US 101 northbound ramps will extend west of the US 101 southbound ramps intersection during both peak hour periods. The eastbound queue at the Ramada Drive intersection will also exceed the available storage between the US 101 northbound ramps and Ramada Drive intersections during both peak hour periods.

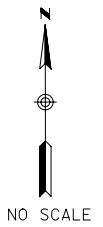
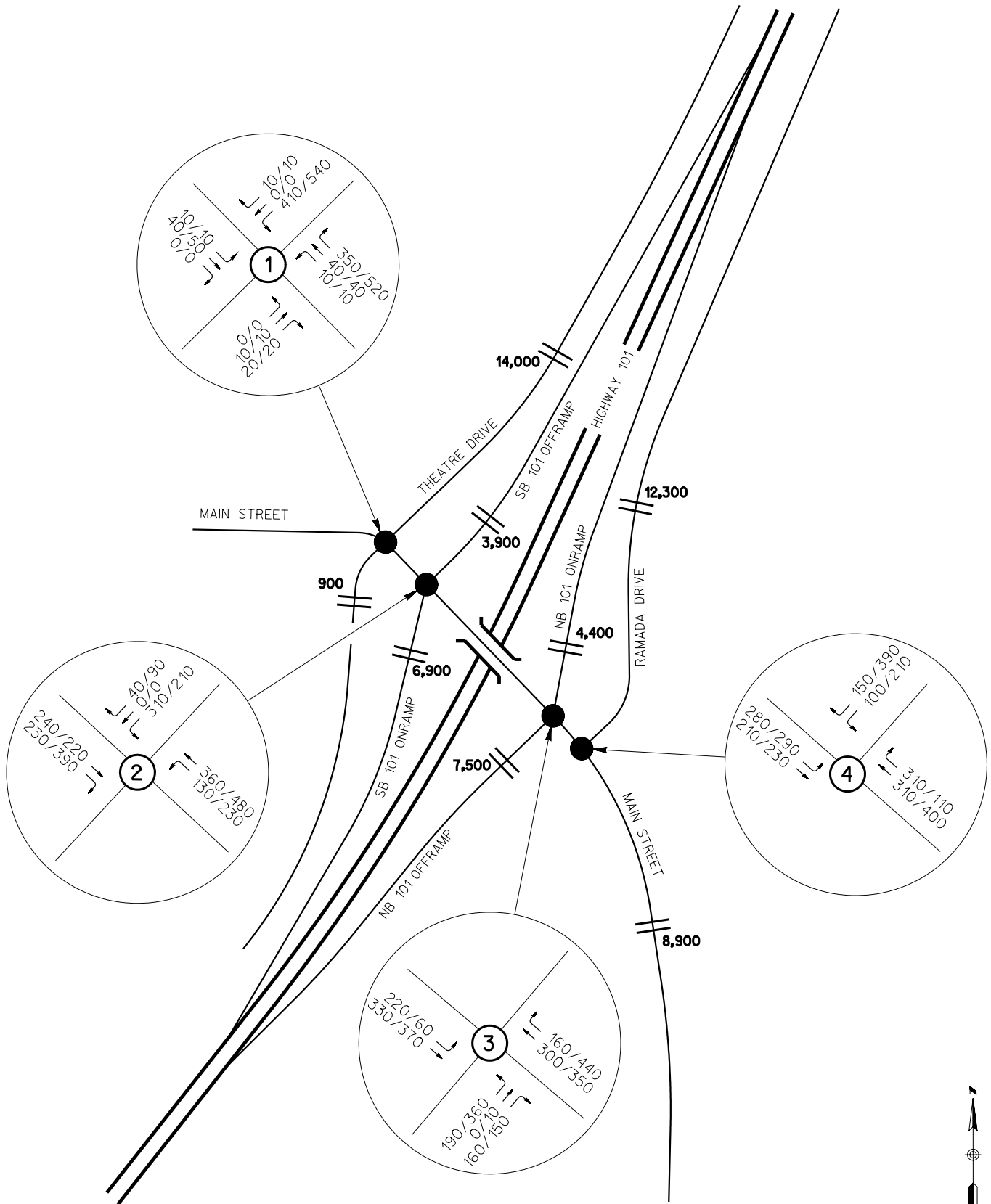
5.0 EXISTING CONDITIONS WITH INTERCHANGE ALTERNATIVES

This section summarizes peak hour traffic volumes and analysis for each alternative under the existing conditions.

This section summarizes the peak hour traffic volumes and analysis for each alternative under the existing conditions.

5.1 Traffic Volumes and Intersection Lane Geometrics

This section summarizes development of traffic volumes and lane geometrics at the study intersections.



© 2000 Rick Engineering Company



EXHIBIT 6
 BUILDOUT TRAFFIC VOLUMES
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - XX/XX = AM/PM PEAK HOUR TRAFFIC
 - XX = AVERAGE DAILY TRAFFIC

Alternative 6

Alternative 6 entails traffic control related improvements on the west side of Hwy 101 without major relocation of roadways, and therefore, peak hour and daily traffic volumes are not anticipated to change significantly as compared to the existing conditions. On the east side of the freeway, Alternative 6 includes the relocation of Ramada Drive by approximately 400 feet along Main Street, although peak hour and daily traffic volumes are not expected to change significantly as compared to the existing conditions. **Exhibit 7** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 6 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 6 are shown on **Exhibit 8**.

Alternative 7

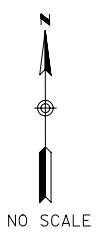
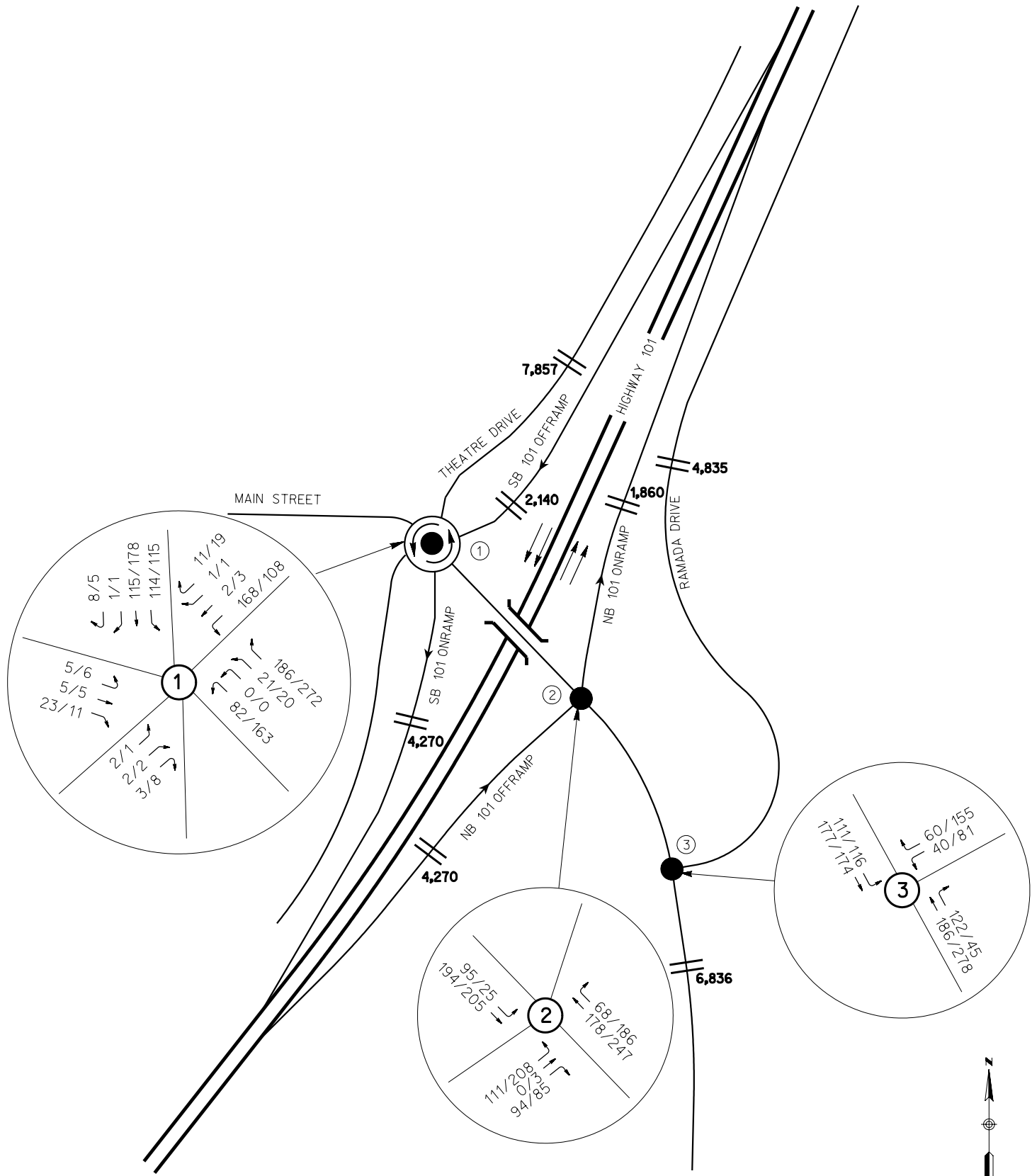
Similar to Alternative 6, Alternative 7 entails traffic control related improvements on the west side of Hwy 101 without major relocation of roadways, and therefore, peak hour and daily traffic volumes are not anticipated to change significantly as compared to the existing conditions. On the east side of the freeway, Alternative 7 includes the reconstruction of the two separate intersections into a single intersection. Additionally, the northbound onramp would be reconfigured to be a hook style ramp. While the turning movements would change, peak hour and daily traffic volumes are not anticipated to change significantly as compared to the existing conditions. **Exhibit 9** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 6 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 6 are shown on **Exhibit 10**.

5.2 Traffic Operations Analysis

This section presents results of traffic operations analysis for each alternative.

Alternative 6

Table 5 presents the results of the intersection and roundabout LOS analysis for Alternative 6. Overall, the intersection operations are projected to improve with this alternative relative to the existing interchange configuration. The study intersections are projected to function at acceptable LOS during both AM and PM peak periods with the exception of the northbound approach at the intersection of Main Street/US 101 northbound ramps which is projected to operate at LOS D during the PM peak period, exceeding Caltrans threshold. Based on the Signal Warrant Analysis included in Deliverable 1 (Section 9.0), a traffic signal would not be warranted under existing conditions at this intersection. It is recommended that peak hour traffic volumes at this intersection be monitored to determine when a traffic signal would be warranted.



© 2000 Rick Engineering Company



EXHIBIT 7
 EXISTING CONDITIONS WITH ALTERNATIVE 6
 TRAFFIC VOLUMES
 US 101/MAIN STREET TRAFFIC STUDY

LEGEND:
 ● = STUDY INTERSECTION
 XX/XX = AM/PM PEAK HOUR TRAFFIC
 xx = AVERAGE DAILY TRAFFIC

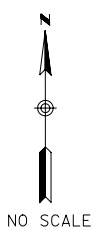
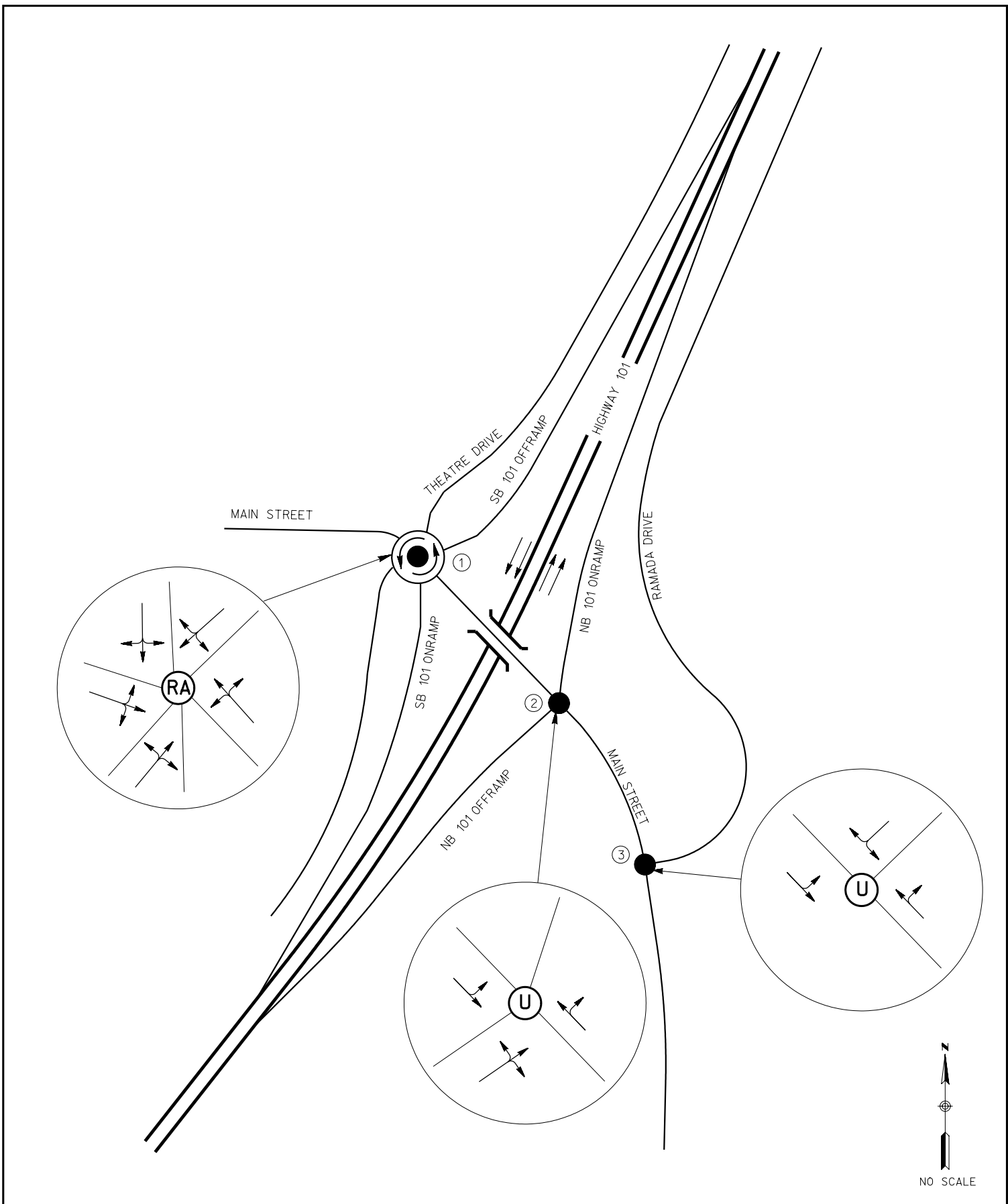


EXHIBIT 8
 EXISTING CONDITIONS WITH ALTERNATIVE 6
 INTERSECTION LANE CONFIGURATION
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - Ⓡ = ROUNDABOUT
 - Ⓢ = UNSIGNALIZED INTERSECTION
 - ↔ = TRAFFIC LANE & STORAGE

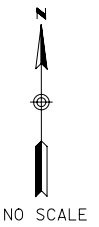
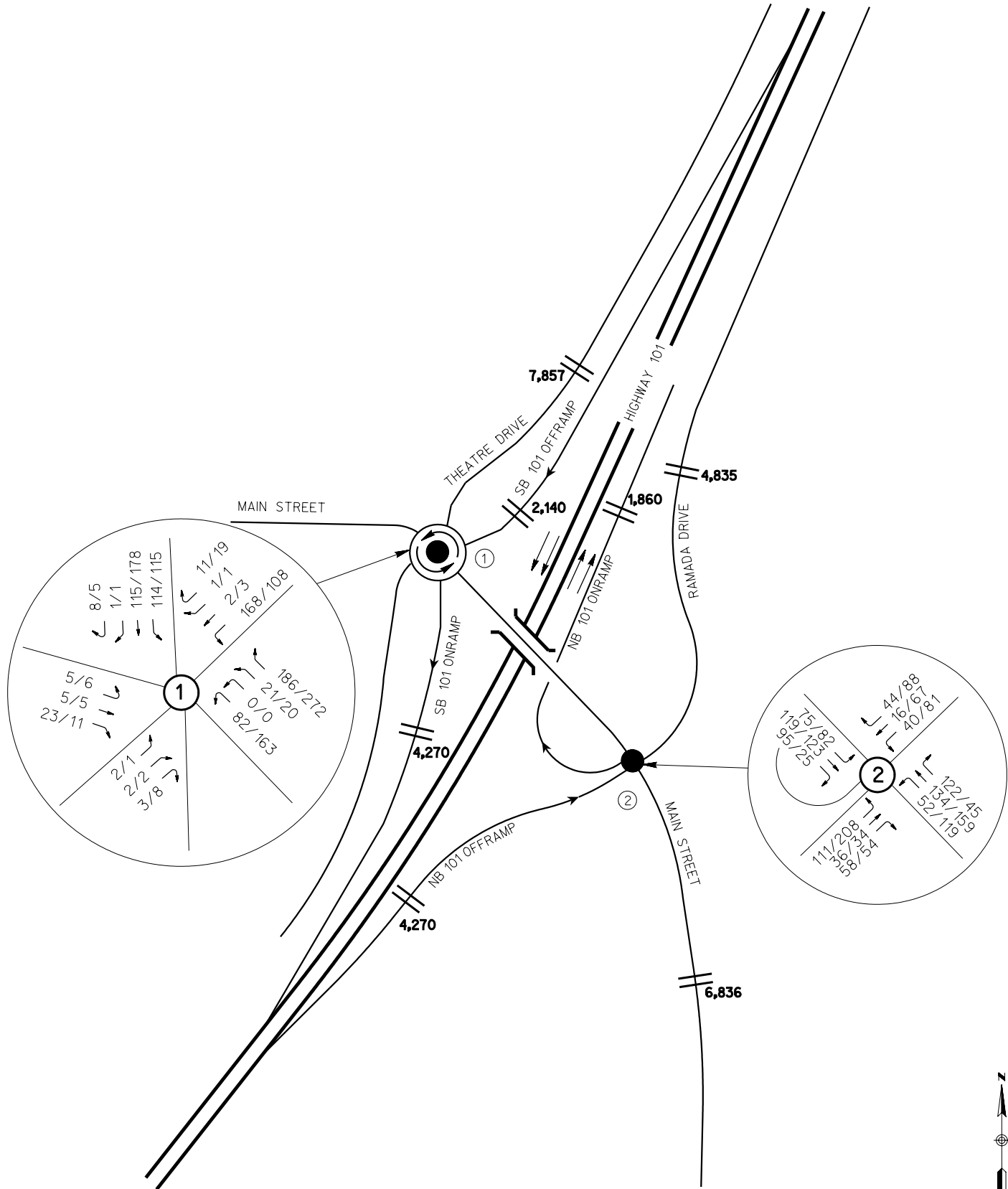
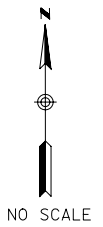
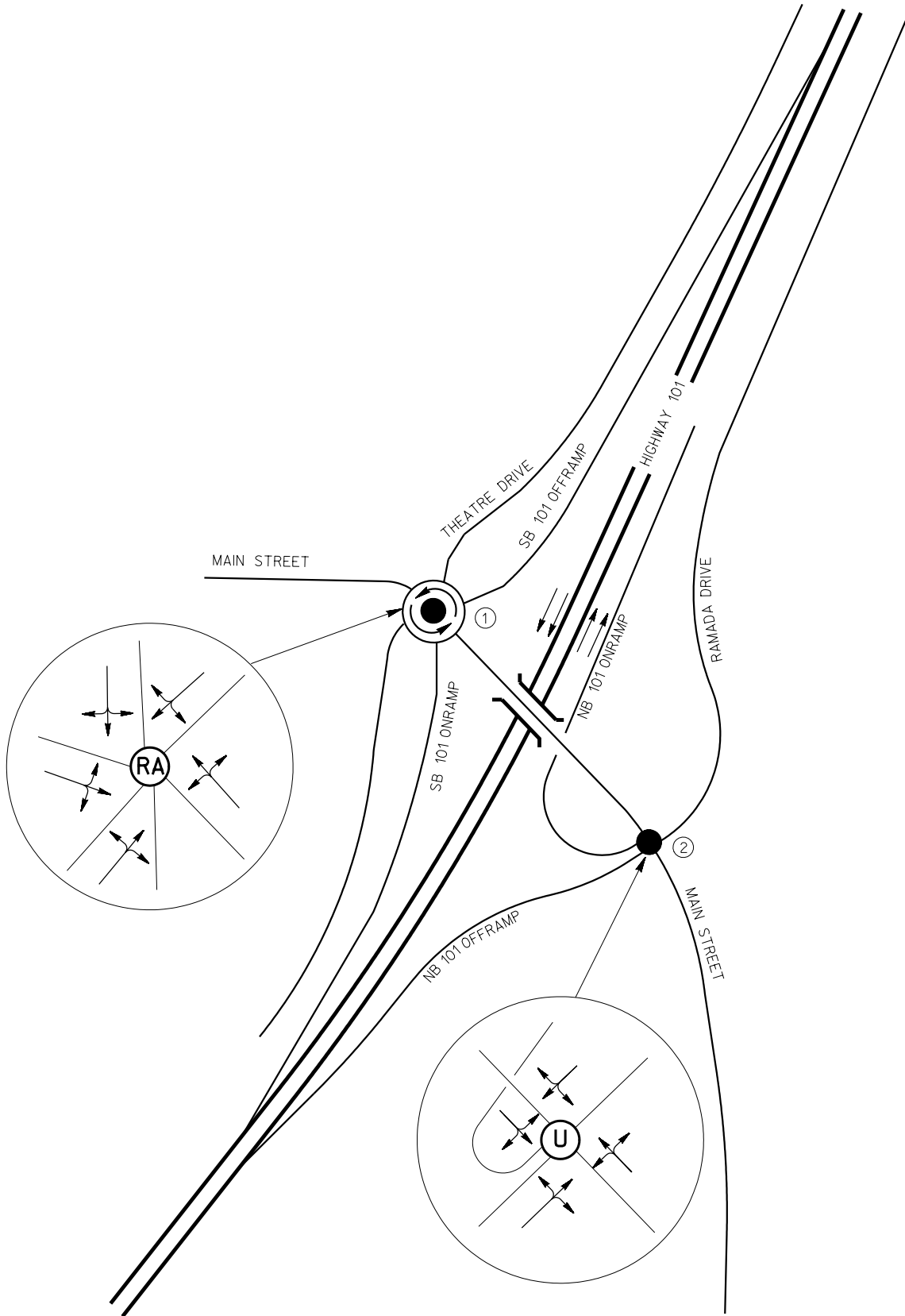


EXHIBIT 9
 EXISTING CONDITIONS WITH ALTERNATIVE 7
 TRAFFIC VOLUMES
 US 101/MAIN STREET TRAFFIC STUDY

LEGEND:
 ● = STUDY INTERSECTION
 XX/XX = AM/PM PEAK HOUR TRAFFIC
 XX = AVERAGE DAILY TRAFFIC



© 2000 Rick Engineering Company



EXHIBIT 10
 EXISTING CONDITIONS WITH ALTERNATIVE 7
 INTERSECTION LANE CONFIGURATION
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - Ⓡ = ROUNDABOUT
 - Ⓢ = UNSIGNALIZED INTERSECTION
 - 40' ↗ = TRAFFIC LANE & STORAGE

**TABLE 5
EXISTING CONDITIONS WITH ALTERNATIVE 6 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 6 | |
|---------------------------------------|----------------|---|----------|
| | | Avg. Delay | LOS |
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | |
| | Average | 6.1 | A |
| | WB LTR | 5.1 | A |
| | SB LTR | 6.4 | A |
| | SE LTR | 7.1 | A |
| | EB LTR | 5.7 | A |
| | NW LTR | 5.5 | A |
| | PM Peak | | |
| | Average | 6.5 | A |
| | WB LTR | 6.1 | A |
| | SB LTR | 6.4 | A |
| | SE LTR | 7.3 | A |
| | EB LTR | 5.4 | A |
| NW LTR | 5.3 | A | |
| US 101 NB Ramps (TWSC) | AM Peak | | |
| | EB L | 3.2 | A |
| | NB LTR | 16.1 | C |
| | PM Peak | | |
| | EB L | 1.2 | A |
| | NB LTR | 27.5 | D |
| Ramada Drive (TWSC) | AM Peak | | |
| | EB L | 3.8 | A |
| | SB LR | 14.0 | B |
| | PM Peak | | |
| | EB L | 4.0 | A |
| | SB LR | 20.5 | C |

LOS = Level of Service; Average Delay in seconds/vehicle

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

Bold indicates that LOS exceeds significance threshold

To analyze vehicular queue lengths, simulations were run using the SimTraffic software within Synchro. **Table 6** summarizes the intersection and roundabout queuing analysis results under existing conditions with Alternative 6. The estimated 95th percentile queues were reported to be accommodated within the available or assumed storage lengths. The 95th percentile queues would exceed the available storage and potentially block traffic at the US 101 northbound ramps intersection.

TABLE 6
EXISTING CONDITIONS WITH ALTERNATIVE 6 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient/Insufficient |
|--|----------------|-------------------------------------|---|---|
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | | |
| | WB LTR | 300 | 34 | Sufficient |
| | SB LTR | 800 | 26 | Sufficient |
| | SE LTR | 1000 | 35 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| | PM Peak | | | |
| | WB LTR | 300 | 60 | Sufficient |
| | SB LTR | 800 | 25 | Sufficient |
| | SE LTR | 1000 | 44 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| US 101 NB Ramps (TWSC) | AM Peak | | | |
| | EB LT | 470 | 56 | Sufficient |
| | NB LTR | 800 | 99 | Sufficient |
| | PM Peak | | | |
| | EB LT | 470 | 41 | Sufficient |
| NB LTR | 800 | 128 | Sufficient | |
| Ramada Drive (TWSC) | AM Peak | | | |
| | EB LT | 500 | 88 | Sufficient |
| | SB LR | 1000 | 70 | Sufficient |
| | PM Peak | | | |
| | EB LT | 500 | 74 | Sufficient |
| SB LR | 1000 | 134 | Sufficient | |

Storage length based on measured or estimated clear distance between intersections or turning bay

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

(12%) indicates % of time the upstream end of the lane is blocked during the peak hour

Bold indicates that the queue spillback may be experienced

Alternative 7

Table 7 presents the results of intersection and roundabout LOS analysis for Alternative 7. Overall, the intersection operations are projected to improve with this alternative relative to the existing interchange configuration. The US 101 NB ramps/Ramada Drive intersection is projected to operate within acceptable LOS during the AM and PM peak hours. However, the northbound ramp and Ramada Drive intersection at Main Street is anticipated to exceed the County's LOS D threshold during the PM peak hour. Based on the Signal Warrant Analysis included in Deliverable 1 (Section 9.0), a traffic signal would not be warranted under existing conditions at this intersection. It is recommended that peak hour traffic volumes at this intersection be monitored to determine when a traffic signal would be warranted.

TABLE 7
EXISTING CONDITIONS WITH ALTERNATIVE 7 LOS RESULTS

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 7 | |
|---------------------------------------|----------------|---|----------|
| | | Avg. Delay | LOS |
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | |
| | Average | 6.1 | A |
| | WB LTR | 5.1 | A |
| | SB LTR | 6.4 | A |
| | SE LTR | 7.1 | A |
| | EB LTR | 5.7 | A |
| | NW LTR | 5.5 | A |
| | PM Peak | | |
| | Average | 6.5 | A |
| | WB LTR | 6.1 | A |
| | SB LTR | 6.4 | A |
| | SE LTR | 7.3 | A |
| | EB LTR | 5.4 | A |
| | NW LTR | 5.3 | A |
| US 101 NB Ramps & Ramada Drive (TWSC) | AM Peak | | |
| | EB LTR | 2.6 | A |
| | WB LTR | 1.7 | A |
| | SB LTR | 16.9 | C |
| | NB LTR | 22.5 | C |
| | PM Peak | | |
| | EB LTR | 3.2 | A |
| | WB LTR | 3.5 | A |
| | SB LTR | 26.3 | D |
| NB LTR | >50 | F | |

LOS = Level of Service; Average Delay in seconds/vehicle
 TWSC = Two-Way Stop Control, TS = Traffic Signal
 NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound
 L = Left turn movement, T = Through movement, R = Right turn movement
Bold indicates that LOS exceeds significance threshold

To analyze vehicular queue lengths at the northbound ramps/Main Street/Ramada Drive intersection, simulations were run using the SimTraffic software within Synchro. **Table 8** summarizes the intersection queuing analysis results under existing conditions with Alternative 7. The 95th percentile queues were reported to be accommodated within the available or assumed storage lengths at all study intersections.

To analyze vehicular queue lengths for the west roundabout, SIDRA software was used. **Table 8** summarizes the roundabout queuing analysis results under the existing conditions with Alternative 7. The 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements.

TABLE 8
EXISTING CONDITIONS WITH ALTERNATIVE 7 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient/Insufficient |
|--|----------------|-------------------------------------|---|---|
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | | |
| | WB LTR | 300 | 34 | Sufficient |
| | SB LTR | 800 | 26 | Sufficient |
| | SE LTR | 1000 | 35 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| | PM Peak | | | |
| | WB LTR | 300 | 60 | Sufficient |
| | SB LTR | 800 | 25 | Sufficient |
| | SE LTR | 1000 | 44 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| NW LTR | 500 | 25 | Sufficient | |
| US 101 NB Ramps & Ramada Drive (TWSC) | AM Peak | | | |
| | EB LTR | 470 | 70 | Sufficient |
| | WB LTR | 1000 | 69 | Sufficient |
| | SB LTR | 1000 | 62 | Sufficient |
| | NB LTR | 800 | 86 | Sufficient |
| | PM Peak | | | |
| | EB LTR | 470 | 459 | Sufficient |
| | WB LTR | 1000 | 70 | Sufficient |
| | SB LTR | 1000 | 113 | Sufficient |
| NB LTR | 800 | 265 | Sufficient | |

Storage length based on measured or estimated clear distance between intersections or turning bay
 TWSC = Two-Way Stop Control, TS = Traffic Signal
 NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound
 L = Left turn movement, T = Through movement, R = Right turn movement
 (12%) indicates % of time the upstream end of the lane is blocked during the peak hour
Bold indicates that the queue spillback may be experienced

6.0 BUILDOUT CONDITIONS WITH INTERCHANGE ALTERNATIVES

This section summarizes peak hour traffic volumes and analysis for each alternative under the buildout conditions.

6.1 Traffic Volumes and Intersection Lane Geometrics

This section describes development of traffic volumes and lane geometrics under each alternative.

Alternative 6

As previously stated, Alternative 6 includes construction of a six-legged roundabout west of Hwy 101, and realignment of Ramada Drive so that the intersection of Ramada Drive at Main

Street meets minimum intersection separation standards. **Exhibit 11** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 6 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 6 are shown on **Exhibit 12**.

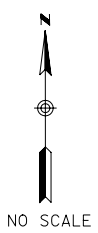
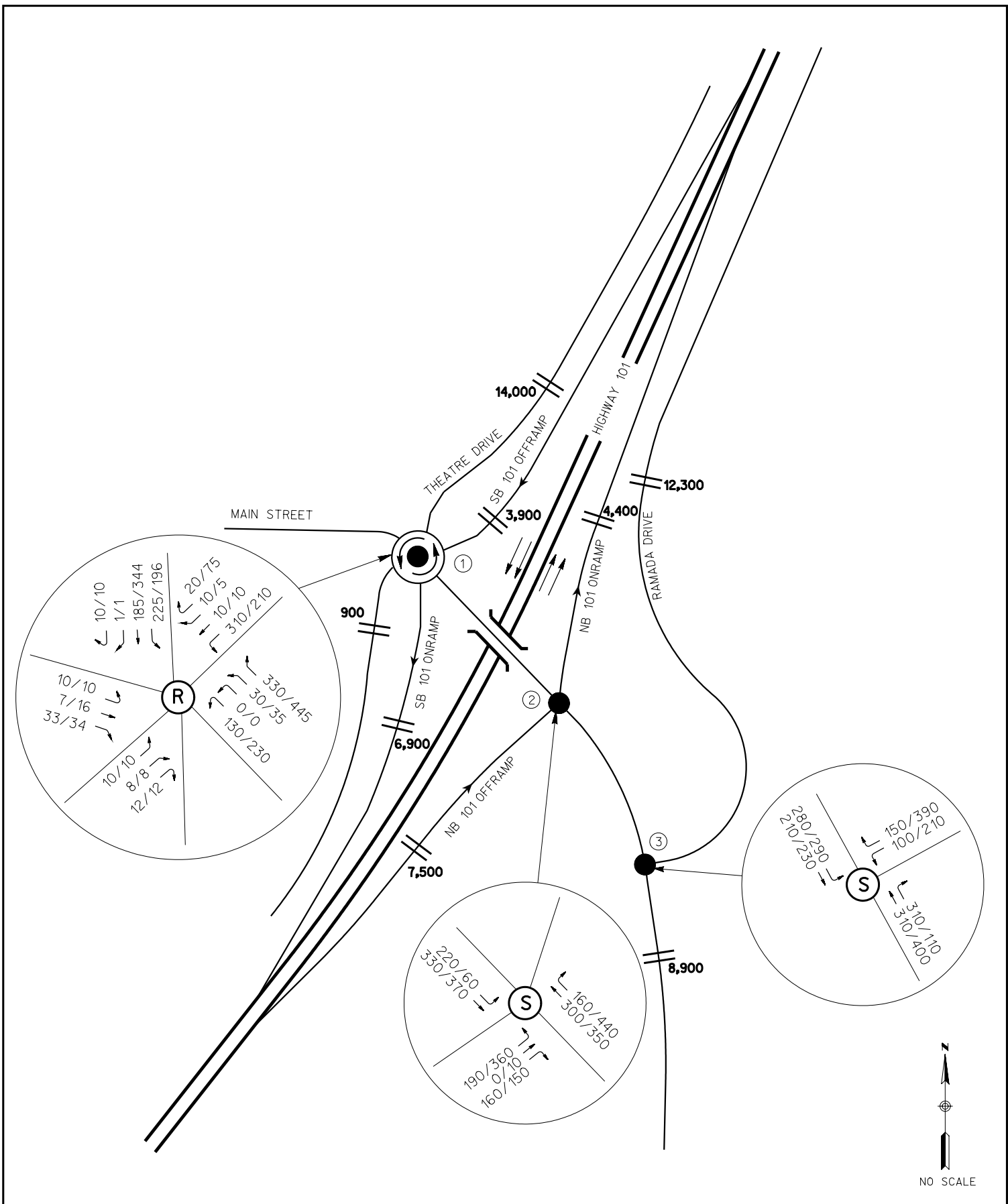
Under the buildout conditions, both the peak hour and average daily traffic signal warrants would be satisfied for the US 101 northbound ramp and Ramada Drive intersections. Refer to Section 8.0 in Deliverable for the complete description and signal warrant analysis. Similar to the analysis conducted under existing conditions, each intersection was evaluated to determine the appropriate traffic control device. The potential need for a traffic signal was evaluated using the peak hour volume and delay (Warrant #3) warrant criteria in the latest version of the CA MUTCD. These warrants are being used as an “indicator” to identify the likelihood of an unsignalized intersection warranting traffic signal control. The results of the traffic signal warrant analysis for each alternative are displayed in **Table 9**. All signal warrant analysis worksheets are contained in **Appendix C**. The buildout peak hour traffic volumes at the Main Street/Ramada Drive and intersection would satisfy the minimum volume signal warrant during one or both peak hour periods. Therefore, the two study intersections (not including the roundabout) were assumed to be signalized for the analysis of Alternative 6.

Exhibit 12 illustrates the assumed lane geometrics and traffic controls for Alternative 6.

**TABLE 9
BUILDOUT CONDITIONS SIGNAL WARRANT ANALYSIS RESULTS**

| Study Intersection | Alternative 6 | | Alternative 7 | |
|--------------------------------|--------------------|---------------------|--------------------|---------------------|
| | Delay Warrant Met? | Volume Warrant Met? | Delay Warrant Met? | Volume Warrant Met? |
| Main Street & US 101 NB Ramps | | | | |
| AM Peak | Yes | Yes | NA | NA |
| PM Peak | Yes | Yes | NA | NA |
| Ramada Drive & US 101 NB Ramps | | | | |
| AM Peak | NA | NA | Yes | Yes |
| PM Peak | NA | NA | Yes | Yes |
| Main Street & Ramada Drive | | | | |
| AM Peak | Yes | Yes | NA | NA |
| PM Peak | Yes | Yes | NA | NA |

NA = Not Applicable

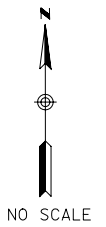
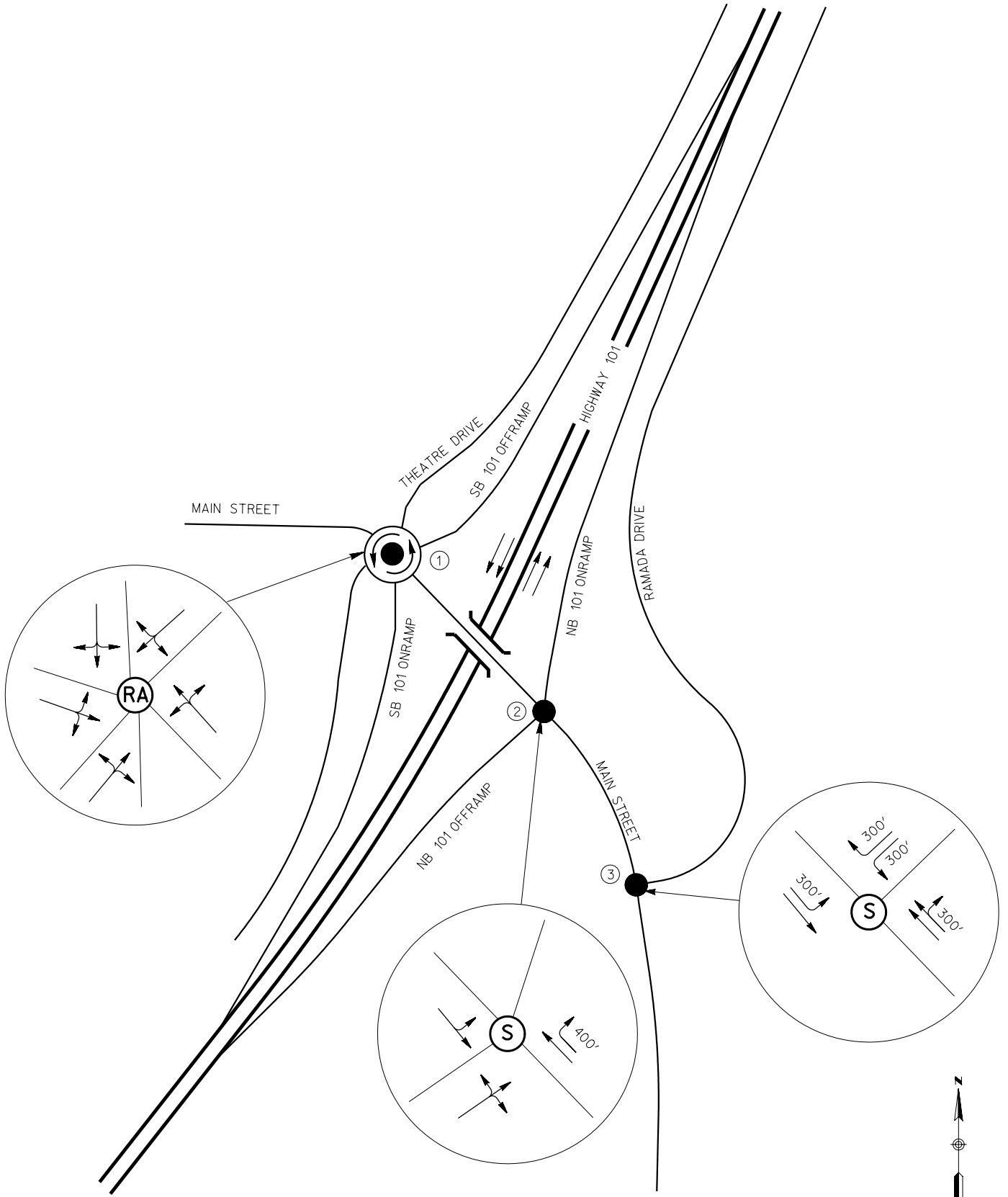


© 2000 Rick Engineering Company



EXHIBIT 11
 BUILDOUT CONDITIONS WITH ALTERNATIVE 6
 TRAFFIC VOLUMES
 US 101/MAIN STREET TRAFFIC STUDY

LEGEND:
 ● = STUDY INTERSECTION
 XX/XX = AM/PM PEAK HOUR TRAFFIC
 xx = AVERAGE DAILY TRAFFIC



© 2000 Rick Engineering Company



EXHIBIT 12
 BUILDOUT CONDITIONS WITH ALTERNATIVE 6
 INTERSECTION LANE CONFIGURATION
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - Ⓡ = ROUNDABOUT
 - Ⓢ = SIGNALIZED INTERSECTION
 - 40' = TRAFFIC LANE & STORAGE

Alternative 7

As previously stated, Alternative 7 includes construction of a six-legged roundabout west of Hwy 101, and realignment of the NB ramps to create a single intersection with Ramada Drive. **Exhibit 13** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 7 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 7 are shown on **Exhibit 14**.

Under the buildout conditions, both the peak hour and the average daily traffic signal warrants would be satisfied for the US 101 northbound ramp intersections. In addition, the buildout peak hour traffic volumes at the Main Street/Ramada Drive # US 101 NB Ramps intersection would satisfy the minimum peak hour volume signal warrant criteria during one or both peak hour periods. Therefore, the reconstructed study intersection east of the freeway was assumed to be signalized under the buildout with Alternative 7 conditions.

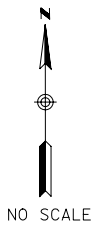
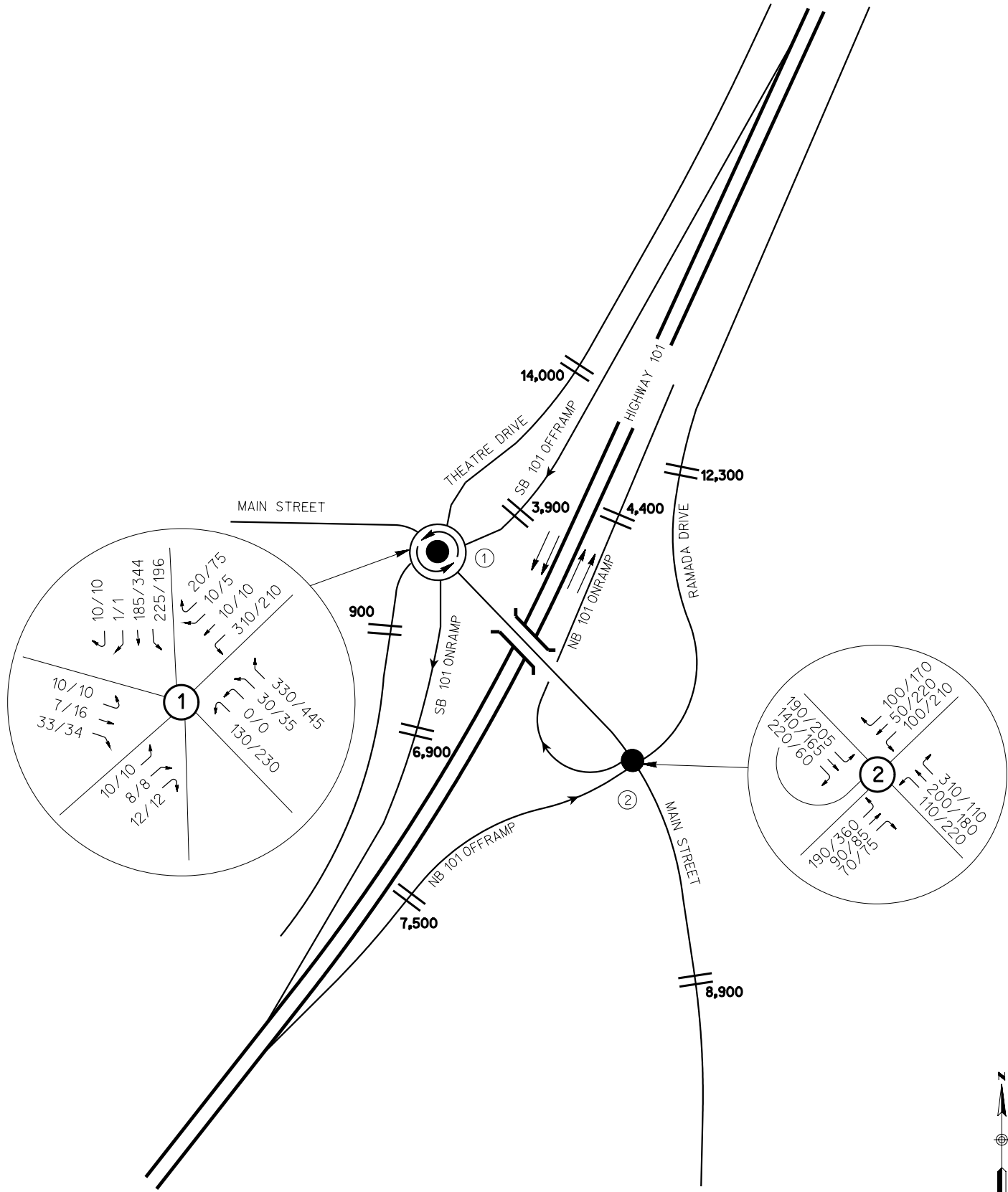
6.2 Traffic Operations Analysis

Alternative 6

Table 10 presents the results of the intersection LOS analysis for Alternative 6. Average delays at the northbound ramp intersection is projected to be within acceptable limits (LOS C or better) during both peak hour periods, meeting the Caltrans threshold criteria. The Main Street intersection with Ramada Drive is also projected to operate at overall LOS C or better during both peak hours, meeting the County's LOS criteria. US 101 southbound ramps/Main Street/Theatre Drive roundabout is projected to function at an overall LOS B during the AM and PM peak hours, thus satisfying the Caltrans and County's threshold criteria.

To analyze vehicular queue lengths at the northbound ramps intersection and the Main Street intersection with Ramada Drive, simulations were run using the SimTraffic software within Synchro. **Table 11** summarizes the intersection 95th percentile queues under the buildout with Alternative 6 conditions. Including the proposed improvements, the 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements.

To analyze vehicular queue lengths for the west roundabout, SIDRA software was used. **Table 11** summarizes the roundabout queuing analysis results under the buildout conditions with Alternative 6. The 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements.

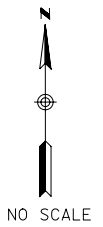
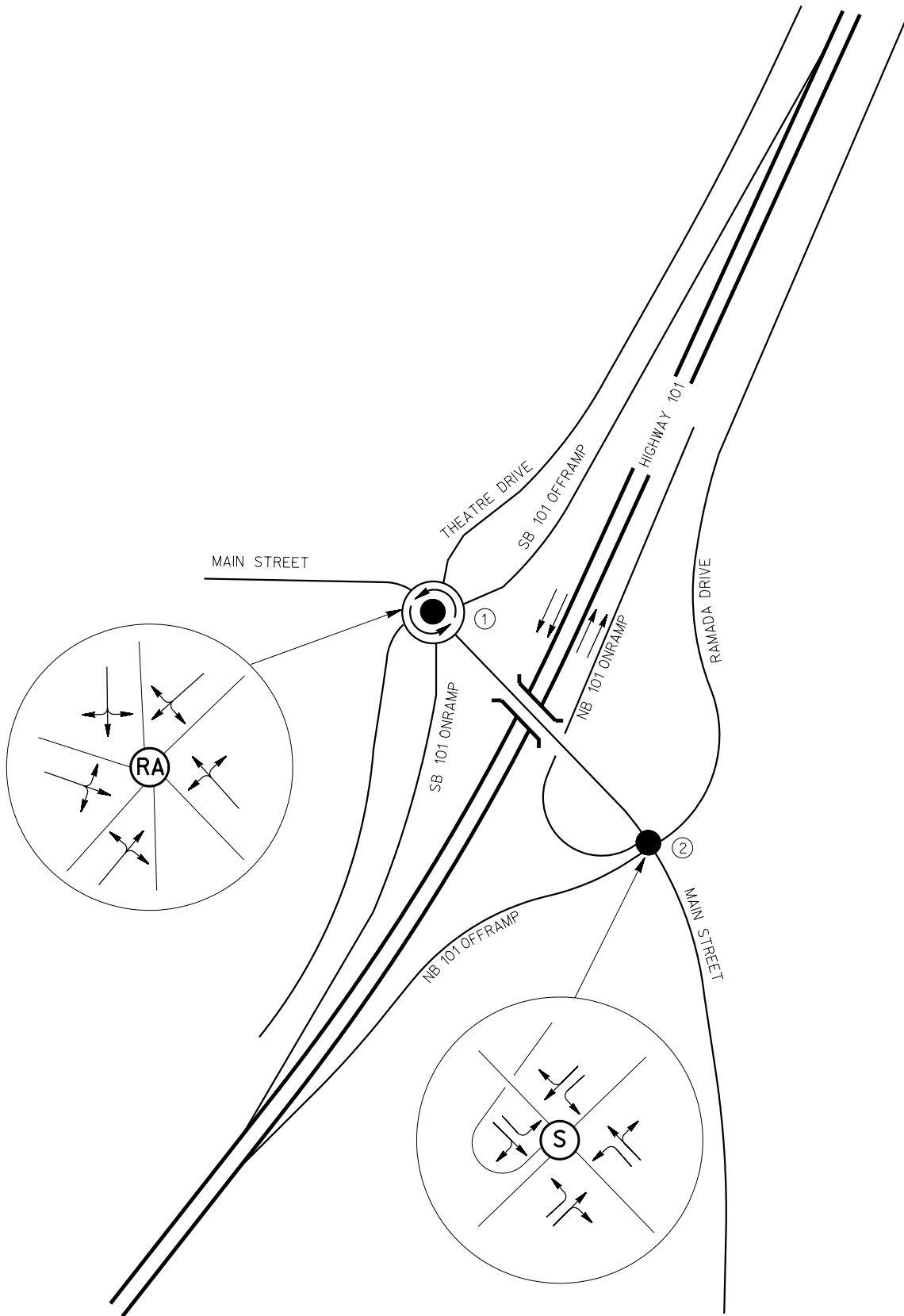


© 2000 Rick Engineering Company



EXHIBIT 13
 BUILDOUT CONDITIONS WITH ALTERNATIVE 7
 TRAFFIC VOLUMES
 US 101/MAIN STREET TRAFFIC STUDY

LEGEND:
 ● = STUDY INTERSECTION
 XX/XX = AM/PM PEAK HOUR TRAFFIC
 xx = AVERAGE DAILY TRAFFIC



© 2000 Rick Engineering Company



EXHIBIT 14
 BUILDOUT CONDITIONS WITH ALTERNATIVE 7
 INTERSECTION LANE CONFIGURATION
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - Ⓡ = ROUNDABOUT
 - Ⓢ = SIGNALIZED INTERSECTION
 - 40' ↗ = TRAFFIC LANE & STORAGE

TABLE 10

BUILDOUT CONDITIONS WITH ALTERNATIVE 6 LOS RESULTS

| Study Intersection Main Street at: | Movement | Buildout Conditions with Alternative 6 | |
|---------------------------------------|-------------------|---|-----|
| | | Avg. Delay | LOS |
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | |
| | Average | 10.1 | B |
| | WB LTR | 6.4 | A |
| | SB LTR | 11.6 | B |
| | SE LTR | 13.5 | B |
| | EB LTR | 8.2 | A |
| | NW LTR | 8.1 | A |
| | PM Peak | | |
| | Average | 14.3 | B |
| | WB LTR | 9.1 | A |
| | SB LTR | 15.5 | C |
| | SE LTR | 21.1 | C |
| | EB LTR | 10.1 | B |
| NW LTR | 9.6 | A | |
| US 101 NB Ramps (TS) | AM Peak | | |
| | Average | 11.0 | B |
| | EB LT | 6.7 | A |
| | WB T | 4.6 | A |
| | WB R | 6.3 | A |
| | NB LTR | 26.3 | C |
| | PM Peak | | |
| | Average | 15.3 | B |
| | EB LT | 7.6 | A |
| | WB T | 13.8 | B |
| | WB R | 15.2 | B |
| | NB LTR | 24.0 | C |
| | Ramada Drive (TS) | AM Peak | |
| Average | | 18.3 | B |
| EB L | | 32.4 | C |
| EB T | | 2.1 | A |
| WB | | 14.1 | B |
| SB L | | 28.2 | C |
| SB R | | 26.0 | C |
| PM Peak | | | |
| Average | | 20.0 | B |
| EB L | | 34.5 | C |
| EB T | | 2.7 | A |
| WB | | 16.1 | B |
| SB L | | 26.9 | C |
| SB R | 20.8 | C | |

LOS = Level of Service; Average Delay in seconds/vehicle

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

Bold indicates that LOS exceeds significance threshold

TABLE 11
BUILDOUT CONDITIONS WITH ALTERNATIVE 6 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient/Insufficient |
|--|----------------|-------------------------------------|---|---|
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | | |
| | WB LTR | 300 | 66 | Sufficient |
| | SB LTR | 800 | 74 | Sufficient |
| | SE LTR | 1000 | 101 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| | PM Peak | | | |
| | WB LTR | 300 | 130 | Sufficient |
| | SB LTR | 800 | 79 | Sufficient |
| | SE LTR | 1000 | 190 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| US 101 NB Ramps (TS) | AM Peak | | | |
| | EB LT | 470 | 361 | Sufficient |
| | WB T | 500 | 78 | Sufficient |
| | WB R | 500 | 34 | Sufficient |
| | NB LT | 800 | 178 | Sufficient |
| | PM Peak | | | |
| | EB LT | 470 | 258 | Sufficient |
| | WB T | 500 | 333 | Sufficient |
| | WB R | 500 | 249 | Sufficient |
| | NB LT | 800 | 625 | Sufficient |
| Ramada Drive (TS) | AM Peak | | | |
| | EB L | 500 | 243 | Sufficient |
| | EB T | 500 | 44 | Sufficient |
| | WB T | 1000 | 149 | Sufficient |
| | WB TR | 300 | 170 | Sufficient |
| | SB L | 1000 | 109 | Sufficient |
| | SB R | 300 | 64 | Sufficient |
| | PM Peak | | | |
| | EB L | 500 | 231 | Sufficient |
| | EB T | 500 | 72 | Sufficient |
| | WB T | 1000 | 196 | Sufficient |
| | WB TR | 300 | 168 | Sufficient |
| | SB L | 1000 | 158 | Sufficient |
| | SB R | 300 | 159 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

(12%) indicates % of time the upstream end of the lane is blocked during the peak hour

Bold indicates that the queue spillback may be experienced

The data in **Table 11** demonstrates that the 95th percentile queues to be accommodated within the available or estimated storage at all movements.

Alternative 7

Table 12 presents the results of the intersection LOS analysis for Alternative 7. Average delays at the northbound ramps/Main Street/Ramada Drive intersection are projected to operate with deficient LOS E for the PM peak hour, thus exceeding the Caltrans and County’s threshold criteria. US 101 southbound ramps/Main Street/Theatre Drive roundabout is projected to function at an overall LOS B during the AM and PM peak hours, thus satisfying the Caltrans and County’s threshold criteria.

**TABLE 12
BUILDOUT CONDITIONS WITH ALTERNATIVE 7 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Buildout Conditions with Alternative 7 | |
|---------------------------------------|----------------|---|----------|
| | | Avg. Delay | LOS |
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | |
| | Average | 10.1 | B |
| | WB LTR | 6.4 | A |
| | SB LTR | 11.6 | B |
| | SE LTR | 13.5 | B |
| | EB LTR | 8.2 | A |
| | NW LTR | 8.1 | A |
| | PM Peak | | |
| | Average | 14.3 | B |
| | WB LTR | 9.1 | A |
| | SB LTR | 15.5 | C |
| | SE LTR | 21.1 | C |
| | EB LTR | 10.1 | B |
| NW LTR | 9.6 | A | |
| US 101 NB Ramps & Ramada Drive (TS) | AM Peak | | |
| | Average | 36.8 | D |
| | EB L | 40.0 | D |
| | WB L | 40.9 | D |
| | SB L | 40.0 | D |
| | NB L | 37.7 | D |
| | PM Peak | | |
| | Average | 66.8 | E |
| | EB L | 44.7 | D |
| | WB L | 47.5 | D |
| | SB L | 38.9 | D |
| NB L | 59.6 | E | |

LOS = Level of Service; Average Delay in seconds/vehicle
 TWSC = Two-Way Stop Control, TS = Traffic Signal
 NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound
 L = Left turn movement, T = Through movement, R = Right turn movement
Bold indicates that LOS exceeds significance threshold

To analyze vehicular queue lengths at the northbound ramps /Main Street/Ramada Drive intersection, simulations were run using the SimTraffic software within Synchro. **Table 13** summarizes the intersection 95th percentile queues under the buildout with Alternative 7 conditions. Including the proposed improvements, the 95th percentile queues were estimated to exceed the available or estimated storage at all approach lanes at the Main Street/northbound ramps and Ramada Drive intersection during both the AM and PM peak hour periods.

To analyze vehicular queue lengths for the west roundabout, SIDRA software was used. **Table 13** summarizes the roundabout queuing analysis results under the buildout conditions with Alternative 7. The 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements.

TABLE 13
BUILDOUT CONDITIONS WITH ALTERNATIVE 7 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient/Insufficient |
|--|----------------|-------------------------------------|---|---|
| US 101 SB Ramps & Theatre Drive (RAB) | AM Peak | | | |
| | WB LTR | 300 | 66 | Sufficient |
| | SB LTR | 800 | 74 | Sufficient |
| | SE LTR | 1000 | 101 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| | PM Peak | | | |
| | WB LTR | 300 | 130 | Sufficient |
| | SB LTR | 800 | 79 | Sufficient |
| | SE LTR | 1000 | 190 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient |
| US 101 NB Ramps & Ramada Drive (TS) | AM Peak | | | |
| | EB L | 100 | 167 (22%) | Insufficient |
| | EB TR | 470 | 311 | Sufficient |
| | WBL | 200 | 279 (1%) | Insufficient |
| | WB TR | 1000 | 508 | Sufficient |
| | SB L | 200 | 115 | Sufficient |
| | SB TR | 1000 | 149 | Sufficient |
| | NB L | 200 | 195 | Sufficient |
| | NB TR | 800 | 176 | Sufficient |
| | PM Peak | | | |
| | EB L | 100 | 173 (26%) | Insufficient |
| | EB TR | 470 | 278 | Sufficient |
| | WB L | 200 | 267 (6%) | Insufficient |
| | WB TR | 1000 | 369 | Sufficient |
| | SB L | 200 | 311 (3%) | Insufficient |
| | SBTR | 1000 | 806 | Sufficient |
| | NB L | 200 | 290 (46%) | Insufficient |
| | NBTR | 800 | 757 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay

TWSC = Two-Way Stop Control, TS = Traffic Signal

NB = Northbound, SB = Southbound, EB = Eastbound, WB = Westbound

L = Left turn movement, T = Through movement, R = Right turn movement

(12%) indicates % of time the upstream end of the lane is blocked during the peak hour

Bold indicates that the queue spillback may be experienced

The data in **Table 13** demonstrates that the 95th percentile queues on the following lanes would exceed the available or assumed storage:

- Eastbound Main Street approach at US NB Ramps and Ramada Drive - The left turn queues (167 feet and 173 feet) are reported to exceed the estimated storage (100 feet) for

22% and 26% of time during the AM and PM peak hours, respectively. This intersection is projected to experience heavy peak hour volumes for all conflicting movements which would result in insufficient allocation of green times.

- Westbound Main Street approach at US NB Ramps and Ramada Drive - The left turn queues (279 feet and 267 feet) are reported to exceed the estimated storage (200 feet) for only 1% and 6% of time during the AM and PM peak hours, respectively. This spillback is considered very minor and is not anticipated to degrade the overall operations of ramp intersections.
- Southbound Main Street approach at US NB Ramps and Ramada Drive - The left turn queue (311 feet) is reported to exceed the estimated storage (200 feet) for only 3% of time during the PM peak hour. This spillback is considered very minor and is not anticipated to degrade the overall operations of ramp intersections.
- Northbound Main Street approach at US NB Ramps and Ramada Drive - The left turn queue (290 feet) is reported to exceed the estimated storage (200 feet) for 46% of time during the PM peak hour. The longer queues persisted at this approach is due to downstream congestion.

7.0 INTERSECTION ILV ANALYSIS

The buildout peak hour volumes were used to perform an Intersection Lane Vehicles (ILV) capacity analysis at the intersections. **Table 14** presents the results of the ILV analysis. **Appendix E** contains the ILV method calculation sheets.

TABLE 14
BUILDOUT CONDITIONS INTERSECTION ILV ANALYSIS RESULTS

| Study Intersection | Peak Hour | Buildout Alternative 6 | |
|---|-----------|------------------------|-----------|
| | | ILV per hour | Capacity |
| Main Street / 101 NB Ramps | AM | 1,040 | Under |
| | PM | 1,240 | At |
| Main Street / Ramada Drive. | AM | 895 | Under |
| | PM | 990 | Under |
| | | | |
| Study Intersection | Peak Hour | Buildout Alternative 7 | |
| | | ILV per hour | Capacity |
| Main Street / 101 NB Ramps / Ramada Drive | AM | 1,050 | Under |
| | PM | 1,260 | At |

The data in **Table 14** indicates that under Alternative 6, the 101 northbound ramps at Main Street intersection is estimated to have an ILV/hr in the range of 1,240 during the PM peak hour and in Alternative 7, the 101 northbound ramps at Main Street and Ramada Drive intersection is also estimated to have an ILV/hr in the range of 1,260 during the PM peak hour. This is considered to

be “unstable flow” conditions and some traffic congestion is expected during this time period for both Alternatives.

8.0 INTERCHANGE ALTERNATIVES IMPACTS AND MITIGATIONS

This section describes traffic operations related constraints or impacts at the deficient locations, and recommends mitigation measures to improve the anticipated traffic congestion under each interchange alternative. **Table 15** presents a summary of the LOS analysis for the existing “no build” and each alternative improvement (refer to Tables 3, 5 and 7).

**TABLE 15
EXISTING CONDITIONS LOS SUMMARY**

| ID | Study Intersection Main Street at: | Peak Hour | Vehicle Delay - LOS Value | | |
|----|---------------------------------------|--------------|---------------------------|-----------------|-------------------|
| | | | No Build | Alt. 6 | Alt. 7 |
| 1 | Theatre Drive | AM | 12.4 - B | NA | NA |
| | | PM | 14.5 - B | NA | NA |
| 2 | US 101 SB Ramps | AM | 24.1 - C | NA | NA |
| | | PM | 35.5 - E | NA | NA |
| 3 | US 101 NB Ramps | AM | 16.1 - C | 16.1 - C | NA |
| | | PM | 26.4 - D | 27.5 - D | NA |
| 4 | Ramada Drive | AM | 12.8 - B | 14.0 - B | NA |
| | | PM | 14.8 - B | 20.5 - C | NA |
| 5 | Ramada Drive & US 101 NB Ramps | AM | NA | NA | 22.5 - C |
| | | PM | NA | NA | >50 - F |
| 6 | Theatre Drive & US 101 SB Ramps | AM | NA | 7.1 - A | 7.1 - A |
| | | PM | NA | 7.3 - A | 7.3 - A |

Delays and LOS calculated based on the methodologies described in Chapters 16 and 17 of the HCM 2000
 LOS = Level of Service; Average Delay in seconds/vehicle for signalized intersection;
 Worse movement Delay in seconds/vehicle for two-way stop control intersections
 Average Delay in seconds/vehicle for roundabout intersections
Bold indicates that LOS exceeds significance threshold

The data in Table 15 indicated that delays and LOS under each interchange alternatives with the assumed intersection geometrics and traffic controls are projected to improve peak hour traffic operations within the study area relative to the existing interchange analysis as provided in Deliverable 3 with the exception of Main Street/101 Northbound Ramps-Ramada Drive intersection (Alternative 7).

Based on the analysis of the 2030 buildout conditions, it can be inferred that the assumed lane geometrics and traffic control would be sufficient to achieve acceptable LOS at the study intersections, with the exception of the Main Street/101 Northbound Ramps-Ramada Drive

intersection (Alternative 7). **Table 16** presents a summary of the LOS analysis for the 2030 “no build” and each alternative improvement (refer to Tables 3, 10 and 12).

TABLE 16
2030 BUILDOUT CONDITIONS LOS SUMMARY

| ID | Study Intersection Main Street at: | Peak Hour | Vehicle Delay - LOS Value | | |
|----|---------------------------------------|--------------|---------------------------|----------|-----------------|
| | | | No Build | Alt. 6 | Alt. 7 |
| 1 | Theatre Drive | AM | 29.8 - D | NA | NA |
| | | PM | > 50 - F | NA | NA |
| 2 | US 101 SB Ramps | AM | > 50 - F | NA | NA |
| | | PM | > 50 - F | NA | NA |
| 3 | US 101 NB Ramps | AM | > 50 - F | 11.0 - B | NA |
| | | PM | > 50 - F | 15.3 - B | NA |
| 4 | Ramada Drive | AM | > 50 - F | 18.3 - B | NA |
| | | PM | > 50 - F | 20.0 - B | NA |
| 5 | Ramada Drive & US 101 NB Ramps | AM | NA | NA | 36.8 - D |
| | | PM | NA | NA | 66.8 - E |
| 6 | Theatre Drive & US 101 SB Ramps | AM | NA | 10.1 - B | 10.1 - B |
| | | PM | NA | 14.3 - C | 14.3 - C |

Delays and LOS calculated based on the methodologies described in Chapters 16 and 17 of the HCM 2000
 LOS = Level of Service; Average Delay in seconds/vehicle for signalized intersection;
 Worse movement Delay in seconds/vehicle for two-way stop control intersections
 Average Delay in seconds/vehicle for roundabout intersections
Bold indicates that LOS exceeds significance threshold

8.1 Alternative 6

With the assumption of Main Street expanding to a 4-lane roadway between the northbound ramps and Ramada Drive intersections and the recommendations from Alternative 2 included under the 2030 buildout conditions, the “static” LOS analysis results as presented earlier show that all study intersections operating at acceptable overall LOS during both peak hours. Based on the SimTraffic simulation, the 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements during both AM and PM peak hours (listed in Section 6.2). No mitigations are needed for this Alternative.

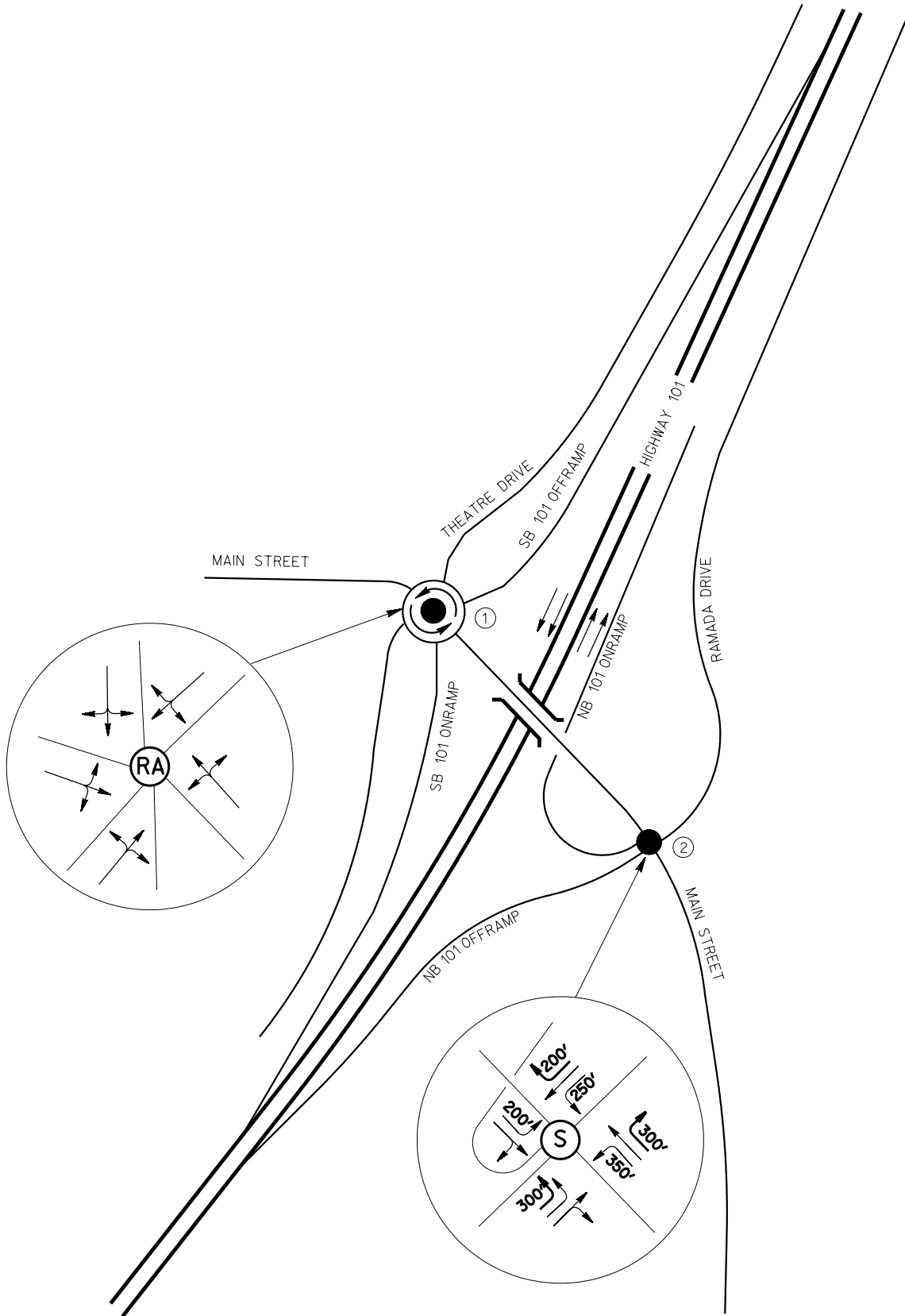
8.2 Alternative 7

Under the 2030 buildout conditions, the “static” LOS analysis results as presented earlier show that all study intersections operating at acceptable overall LOS during both peak hours with the exception of Main Street at US 101 northbound ramps-Ramada Drive which operated at LOS D

and E during the AM and PM peak hours respectively. Based on the SimTraffic simulation, some locations were identified experiencing queuing issues during both AM and PM peak hours (listed in Section 6.2). The following mitigations are recommended to achieve acceptable LOS and queuing conditions in simulation analysis:

1. Add a dedicated 200' right turn lane on the southbound approach Ramada Drive at Main Street and increase the left turn lane storage length to 250'.
2. Add a second 300' dedicated left turn lane on the northbound US 101 ramp approach at Main Street. This would require the westbound departure approach on Main Street to provide for two receiving lanes (to accept dual northbound left turn lanes) and the transition back to one lane prior to the existing bridge.
3. Add a dedicated 300' right turn lane on the westbound Main Street approach at Ramada Drive/US 101 northbound ramps, and increase the westbound left turn storage lane to 300'.
4. Increase the eastbound left turn storage lane to 200'.

Exhibit 15 depicts the recommended lane configurations for this alternative. It should be noted that these recommendations were simply based on traffic operations perspective, and do not take account of any right of way and design related limitations.



© 2000 Rick Engineering Company

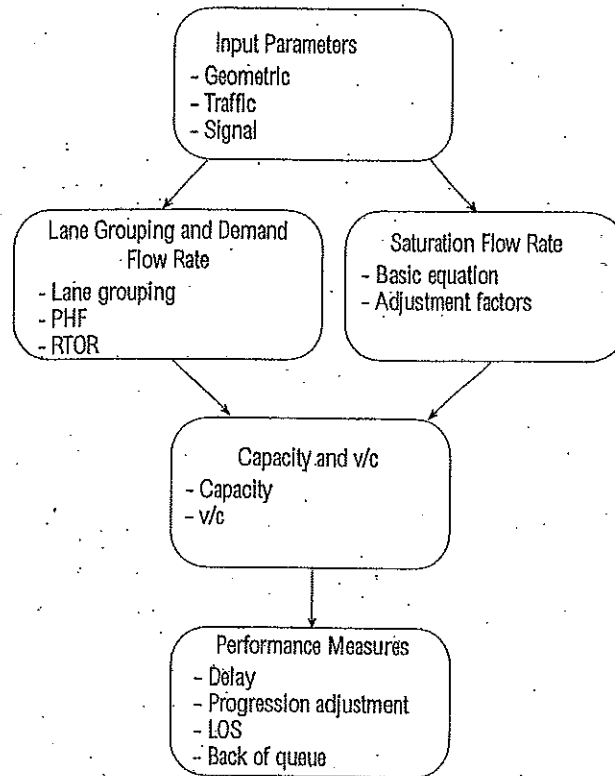


EXHIBIT 15
 BUILDOUT WITH ALTERNATIVE 7
 RECOMMENDED LANE CONFIGURATION
 US 101/MAIN STREET TRAFFIC STUDY

- LEGEND:**
- = STUDY INTERSECTION
 - (TS) = TRAFFIC SIGNAL
 - 40' ↗ = TRAFFIC LANE & STORAGE
 - 40' ↘ = REC. TRAFFIC LANE & STORAGE

APPENDIX A
HCM 2000 LOS METHODOLOGY

EXHIBIT 16-1. SIGNALIZED INTERSECTION METHODOLOGY



LOS

The average control delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection as a whole. LOS is directly related to the control delay value. The criteria are listed in Exhibit 16-2.

EXHIBIT 16-2. LOS CRITERIA FOR SIGNALIZED INTERSECTIONS

| LOS | Control Delay per Vehicle (s/veh) |
|-----|-----------------------------------|
| A | ≤ 10 |
| B | > 10–20 |
| C | > 20–35 |
| D | > 35–55 |
| E | > 55–80 |
| F | > 80 |

LOS criteria

EXHIBIT 17-1. TWSC UNSIGNALIZED INTERSECTION METHODOLOGY

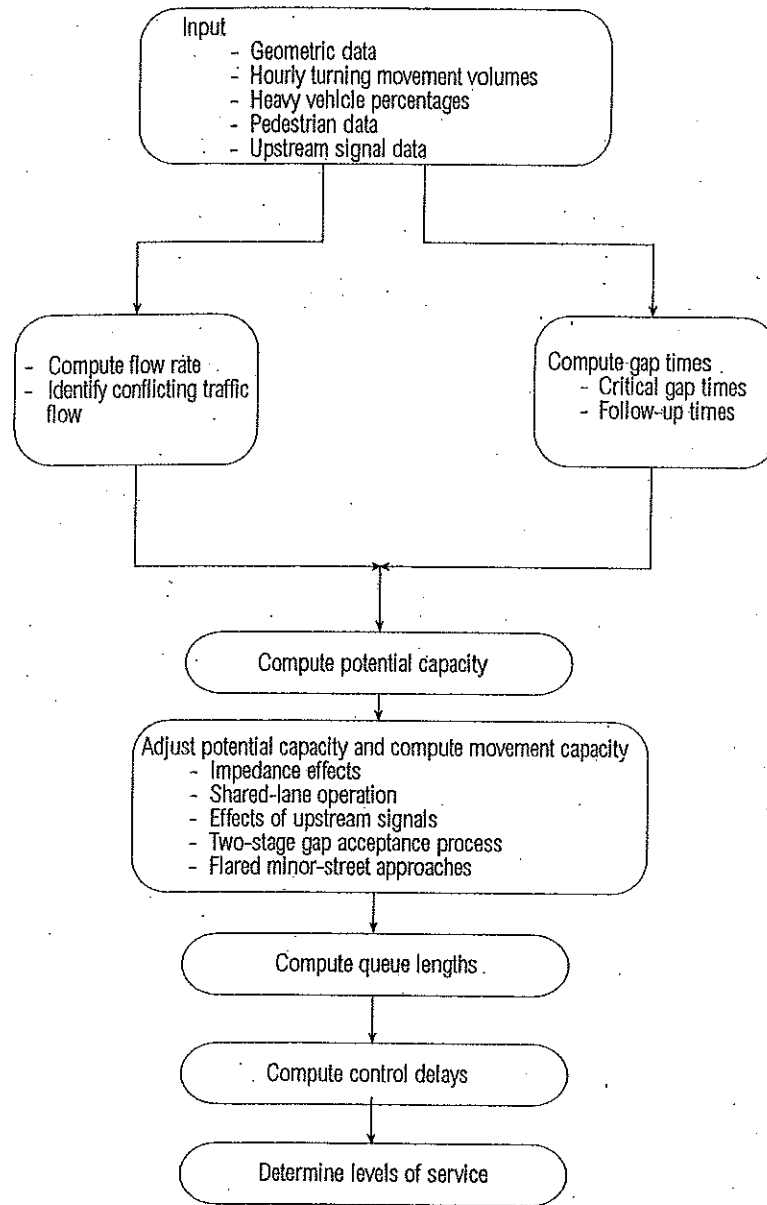
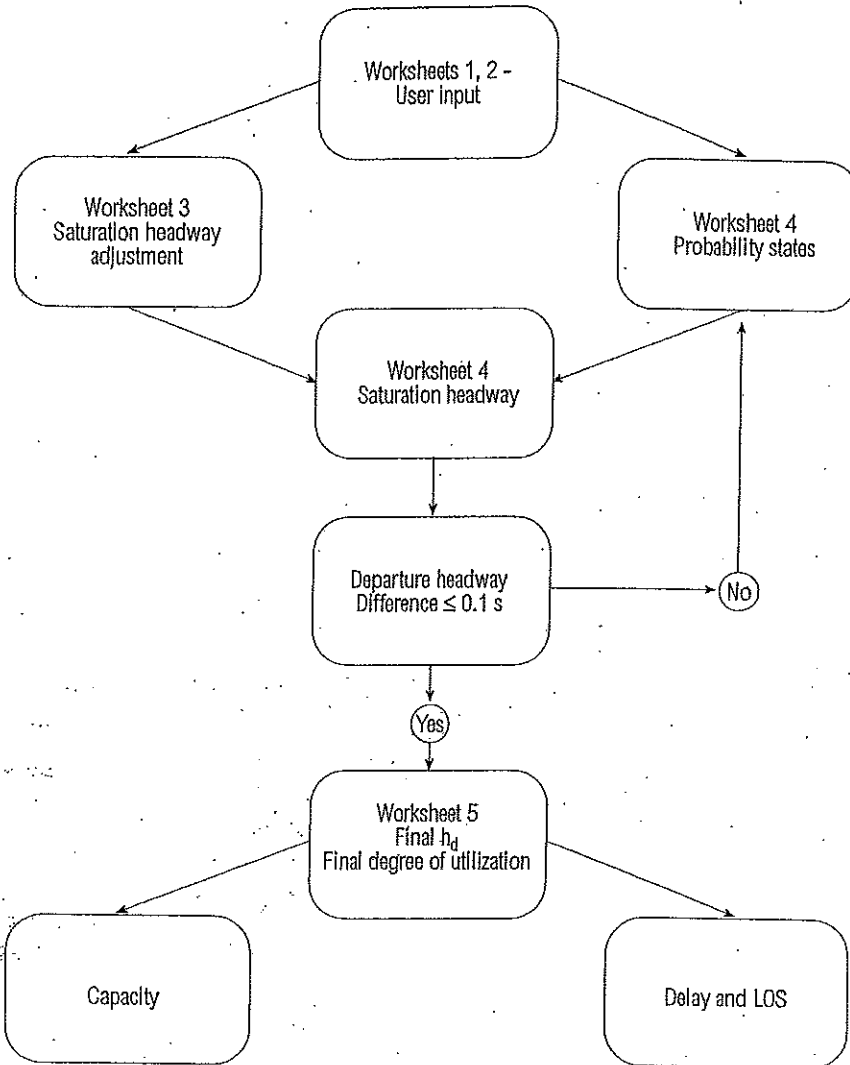


EXHIBIT 17-2. LEVEL-OF-SERVICE CRITERIA FOR TWSC INTERSECTIONS

| Level of Service | Average Control Delay (s/veh) |
|------------------|-------------------------------|
| A | 0-10 |
| B | > 10-15 |
| C | > 15-25 |
| D | > 25-35 |
| E | > 35-50 |
| F | > 50 |

EXHIBIT 17-31. AWSC INTERSECTION METHODOLOGY



LEVEL-OF-SERVICE CRITERIA

The level-of-service criteria are given in Exhibit 17-22. The criteria for AWSC intersections have different threshold values than do those for signalized intersections primarily because drivers expect different levels of performance from distinct types of transportation facilities. The expectation is that a signalized intersection is designed to carry higher traffic volumes than an AWSC intersection. Thus a higher level of control delay is acceptable at a signalized intersection for the same LOS.

EXHIBIT 17-22. LEVEL-OF-SERVICE CRITERIA FOR AWSC INTERSECTIONS

| Level of Service | Control Delay (s/veh) |
|------------------|-----------------------|
| A | 0-10 |
| B | > 10-15 |
| C | > 15-25 |
| D | > 25-35 |
| E | > 35-50 |
| F | > 50 |

















APPENDIX B

EXISTING CONDITIONS WITH ALTERNATIVES LOS AND QUEUE ANALYSIS WORKSHEETS

HCM Unsignalized Intersection Capacity Analysis

3: NB 101 Offramp/NB 101 Onramp & Main St.

Existing Conditions Alt 6
Timing Plan: AM Peak

| |  |  |  |  |  |  |  |  |  |  |  |  |
|-----------------------------------|---|---|---|---|---|---|--|---|---|---|---|---|
| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
| Lane Configurations | |  | | |  | | |  |  | | | |
| Volume (veh/h) | 95 | 194 | 0 | 0 | 178 | 68 | 111 | 0 | 94 | 0 | 0 | 0 |
| Sign Control | | Free | | | Free | | | Stop | | | Stop | |
| Grade | | 0% | | | 0% | | | 0% | | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 106 | 216 | 0 | 0 | 198 | 76 | 123 | 0 | 104 | 0 | 0 | 0 |
| Pedestrians | | | | | | | | | | | | |
| Lane Width (ft) | | | | | | | | | | | | |
| Walking Speed (ft/s) | | | | | | | | | | | | |
| Percent Blockage | | | | | | | | | | | | |
| Right turn flare (veh) | | | | | | | | | 2 | | | |
| Median type | | None | | | None | | | | | | | |
| Median storage (veh) | | | | | | | | | | | | |
| Upstream signal (ft) | | | | | | | | | | | | |
| pX, platoon unblocked | | | | | | | | | | | | |
| vC, conflicting volume | 273 | | | 216 | | | 662 | 700 | 216 | 714 | 662 | 236 |
| vC1, stage 1 conf vol | | | | | | | | | | | | |
| vC2, stage 2 conf vol | | | | | | | | | | | | |
| vCu, unblocked vol | 273 | | | 216 | | | 662 | 700 | 216 | 714 | 662 | 236 |
| tC, single (s) | 4.2 | | | 4.2 | | | 7.2 | 6.6 | 6.3 | 7.2 | 6.6 | 6.3 |
| tC, 2 stage (s) | | | | | | | | | | | | |
| tF (s) | 2.3 | | | 2.3 | | | 3.6 | 4.1 | 3.4 | 3.6 | 4.1 | 3.4 |
| p0 queue free % | 92 | | | 100 | | | 64 | 100 | 87 | 100 | 100 | 100 |
| cM capacity (veh/h) | 1256 | | | 1319 | | | 343 | 326 | 809 | 276 | 343 | 789 |
| Direction, Lane # | EB 1 | WB 1 | NB 1 | | | | | | | | | |
| Volume Total | 321 | 273 | 228 | | | | | | | | | |
| Volume Left | 106 | 0 | 123 | | | | | | | | | |
| Volume Right | 0 | 76 | 104 | | | | | | | | | |
| cSH | 1256 | 1700 | 634 | | | | | | | | | |
| Volume to Capacity | 0.08 | 0.16 | 0.36 | | | | | | | | | |
| Queue Length 95th (ft) | 7 | 0 | 41 | | | | | | | | | |
| Control Delay (s) | 3.2 | 0.0 | 16.1 | | | | | | | | | |
| Lane LOS | A | | C | | | | | | | | | |
| Approach Delay (s) | 3.2 | 0.0 | 16.1 | | | | | | | | | |
| Approach LOS | | | C | | | | | | | | | |
| Intersection Summary | | | | | | | | | | | | |
| Average Delay | | | 5.7 | | | | | | | | | |
| Intersection Capacity Utilization | | | 45.1% | | ICU Level of Service | | | | A | | | |
| Analysis Period (min) | | | 15 | | | | | | | | | |

HCM Unsignalized Intersection Capacity Analysis
 4: Main St. & Ramada Dr.

Existing Conditions Alt 6
 Timing Plan: AM Peak



| Movement | EBL | EBT | WBT | WBR | SBL | SBR |
|------------------------|------|------|------|------|------|------|
| Lane Configurations | | ↔ | ↔ | | ↔ | |
| Volume (veh/h) | 111 | 177 | 186 | 122 | 40 | 60 |
| Sign Control | | Free | Free | | Stop | |
| Grade | | 0% | 0% | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 123 | 197 | 207 | 136 | 44 | 67 |
| Pedestrians | | | | | | |
| Lane Width (ft) | | | | | | |
| Walking Speed (ft/s) | | | | | | |
| Percent Blockage | | | | | | |
| Right turn flare (veh) | | | | | | |
| Median type | | None | None | | | |
| Median storage (veh) | | | | | | |
| Upstream signal (ft) | | | | | | |
| pX, platoon unblocked | | | | | | |
| vC, conflicting volume | 342 | | | | 718 | 274 |
| vC1, stage 1 conf vol | | | | | | |
| vC2, stage 2 conf vol | | | | | | |
| vCu, unblocked vol | 342 | | | | 718 | 274 |
| tC, single (s) | 4.2 | | | | 6.5 | 6.3 |
| tC, 2 stage (s) | | | | | | |
| tF (s) | 2.3 | | | | 3.6 | 3.4 |
| p0 queue free % | 90 | | | | 87 | 91 |
| cM capacity (veh/h) | 1184 | | | | 347 | 750 |

| Direction, Lane # | EB 1 | WB 1 | SB 1 |
|------------------------|------|------|------|
| Volume Total | 320 | 342 | 111 |
| Volume Left | 123 | 0 | 44 |
| Volume Right | 0 | 136 | 67 |
| cSH | 1184 | 1700 | 512 |
| Volume to Capacity | 0.10 | 0.20 | 0.22 |
| Queue Length 95th (ft) | 9 | 0 | 20 |
| Control Delay (s) | 3.8 | 0.0 | 14.0 |
| Lane LOS | A | | B |
| Approach Delay (s) | 3.8 | 0.0 | 14.0 |
| Approach LOS | | | B |

| Intersection Summary | | | |
|-----------------------------------|-------|-----|------------------------|
| Average Delay | | 3.6 | |
| Intersection Capacity Utilization | 48.6% | | ICU Level of Service A |
| Analysis Period (min) | 15 | | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|------|------|------|------|------|------|
| Start Time | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 |
| End Time | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 1200 | 1231 | 1166 | 1188 | 1263 | 1211 |
| Vehs Exited | 1211 | 1241 | 1172 | 1188 | 1260 | 1214 |
| Starting Vehs | 15 | 20 | 13 | 12 | 12 | 12 |
| Ending Vehs | 4 | 10 | 7 | 12 | 15 | 9 |
| Denied Entry Before | 0 | 1 | 0 | 1 | 0 | 0 |
| Denied Entry After | 0 | 0 | 0 | 1 | 0 | 0 |
| Travel Distance (mi) | 361 | 372 | 355 | 362 | 380 | 366 |
| Travel Time (hr) | 14.6 | 15.2 | 14.2 | 14.4 | 15.5 | 14.8 |
| Total Delay (hr) | 3.0 | 3.2 | 2.9 | 2.9 | 3.3 | 3.1 |
| Total Stops | 557 | 572 | 487 | 509 | 582 | 541 |
| Fuel Used (gal) | 18.4 | 19.1 | 17.9 | 18.1 | 19.0 | 18.5 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 6:50 |
| End Time | 7:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 7:00 |
| End Time | 8:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|------|------|------|------|------|------|
| Vehs Entered | 1200 | 1231 | 1166 | 1188 | 1263 | 1211 |
| Vehs Exited | 1211 | 1241 | 1172 | 1188 | 1260 | 1214 |
| Starting Vehs | 15 | 20 | 13 | 12 | 12 | 12 |
| Ending Vehs | 4 | 10 | 7 | 12 | 15 | 9 |
| Denied Entry Before | 0 | 1 | 0 | 1 | 0 | 0 |
| Denied Entry After | 0 | 0 | 0 | 1 | 0 | 0 |
| Travel Distance (mi) | 361 | 372 | 355 | 362 | 380 | 366 |
| Travel Time (hr) | 14.6 | 15.2 | 14.2 | 14.4 | 15.5 | 14.8 |
| Total Delay (hr) | 3.0 | 3.2 | 2.9 | 2.9 | 3.3 | 3.1 |
| Total Stops | 557 | 572 | 487 | 509 | 582 | 541 |
| Fuel Used (gal) | 18.4 | 19.1 | 17.9 | 18.1 | 19.0 | 18.5 |

Intersection: 3: NB 101 Ramps/Ramada Drive & Main Street

| Movement | EB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|
| Directions Served | LTR | LTR | L | TR | L | TR |
| Maximum Queue (ft) | 102 | 98 | 122 | 97 | 78 | 81 |
| Average Queue (ft) | 26 | 25 | 47 | 41 | 26 | 31 |
| 95th Queue (ft) | 70 | 69 | 86 | 76 | 55 | 62 |
| Link Distance (ft) | 509 | 406 | | 579 | | 638 |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | | | 100 | | 100 | |
| Storage Blk Time (%) | | | 1 | 0 | 0 | 0 |
| Queuing Penalty (veh) | | | 1 | 0 | 0 | 0 |

Zone Summary

Zone wide Queuing Penalty: 1

HCM Unsignalized Intersection Capacity Analysis
3: NB 101 Offramp/NB 101 Onramp & Main St.

Existing Conditions Alt 6
Timing Plan: PM Peak



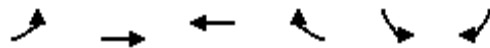
| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|------|------|------|------|------|------|------|------|------|------|------|------|
| Lane Configurations | | ↖ | | | ↗ | | | ↖ | ↗ | | | |
| Volume (veh/h) | 25 | 205 | 0 | 0 | 247 | 186 | 208 | 3 | 85 | 0 | 0 | 0 |
| Sign Control | | Free | | | Free | | | Stop | | | Stop | |
| Grade | | 0% | | | 0% | | | 0% | | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 28 | 228 | 0 | 0 | 274 | 207 | 231 | 3 | 94 | 0 | 0 | 0 |
| Pedestrians | | | | | | | | | | | | |
| Lane Width (ft) | | | | | | | | | | | | |
| Walking Speed (ft/s) | | | | | | | | | | | | |
| Percent Blockage | | | | | | | | | | | | |
| Right turn flare (veh) | | | | | | | | | 2 | | | |
| Median type | | None | | | None | | | | | | | |
| Median storage (veh) | | | | | | | | | | | | |
| Upstream signal (ft) | | | | | | | | | | | | |
| pX, platoon unblocked | | | | | | | | | | | | |
| vC, conflicting volume | 481 | | | 228 | | | 661 | 764 | 228 | 710 | 661 | 378 |
| vC1, stage 1 conf vol | | | | | | | | | | | | |
| vC2, stage 2 conf vol | | | | | | | | | | | | |
| vCu, unblocked vol | 481 | | | 228 | | | 661 | 764 | 228 | 710 | 661 | 378 |
| tC, single (s) | 4.2 | | | 4.2 | | | 7.2 | 6.6 | 6.3 | 7.2 | 6.6 | 6.3 |
| tC, 2 stage (s) | | | | | | | | | | | | |
| tF (s) | 2.3 | | | 2.3 | | | 3.6 | 4.1 | 3.4 | 3.6 | 4.1 | 3.4 |
| p0 queue free % | 97 | | | 100 | | | 36 | 99 | 88 | 100 | 100 | 100 |
| cM capacity (veh/h) | 1051 | | | 1306 | | | 360 | 318 | 797 | 292 | 365 | 656 |

| Direction, Lane # | EB 1 | WB 1 | NB 1 |
|------------------------|------|------|------|
| Volume Total | 256 | 481 | 329 |
| Volume Left | 28 | 0 | 231 |
| Volume Right | 0 | 207 | 94 |
| cSH | 1051 | 1700 | 479 |
| Volume to Capacity | 0.03 | 0.28 | 0.69 |
| Queue Length 95th (ft) | 2 | 0 | 129 |
| Control Delay (s) | 1.2 | 0.0 | 27.5 |
| Lane LOS | A | | D |
| Approach Delay (s) | 1.2 | 0.0 | 27.5 |
| Approach LOS | | | D |

| Intersection Summary | | |
|-----------------------------------|-------|----------------------|
| Average Delay | | 8.8 |
| Intersection Capacity Utilization | 50.1% | ICU Level of Service |
| Analysis Period (min) | | 15 |
| | | A |

HCM Unsignalized Intersection Capacity Analysis
4: Main St. & Ramada Dr.

Existing Conditions Alt 6
Timing Plan: PM Peak



| Movement | EBL | EBT | WBT | WBR | SBL | SBR |
|------------------------|------|------|------|------|------|------|
| Lane Configurations | | ↶ | ↷ | | ↘ | |
| Volume (veh/h) | 116 | 174 | 278 | 45 | 81 | 155 |
| Sign Control | | Free | Free | | Stop | |
| Grade | | 0% | 0% | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 129 | 193 | 309 | 50 | 90 | 172 |
| Pedestrians | | | | | | |
| Lane Width (ft) | | | | | | |
| Walking Speed (ft/s) | | | | | | |
| Percent Blockage | | | | | | |
| Right turn flare (veh) | | | | | | |
| Median type | | None | None | | | |
| Median storage veh | | | | | | |
| Upstream signal (ft) | | | | | | |
| pX, platoon unblocked | | | | | | |
| vC, conflicting volume | 359 | | | | 785 | 334 |
| vC1, stage 1 conf vol | | | | | | |
| vC2, stage 2 conf vol | | | | | | |
| vCu, unblocked vol | 359 | | | | 785 | 334 |
| tC, single (s) | 4.2 | | | | 6.5 | 6.3 |
| tC, 2 stage (s) | | | | | | |
| tF (s) | 2.3 | | | | 3.6 | 3.4 |
| p0 queue free % | 89 | | | | 71 | 75 |
| cM capacity (veh/h) | 1167 | | | | 314 | 694 |

| Direction, Lane # | EB 1 | WB 1 | SB 1 |
|------------------------|------|------|------|
| Volume Total | 322 | 359 | 262 |
| Volume Left | 129 | 0 | 90 |
| Volume Right | 0 | 50 | 172 |
| cSH | 1167 | 1700 | 491 |
| Volume to Capacity | 0.11 | 0.21 | 0.53 |
| Queue Length 95th (ft) | 9 | 0 | 78 |
| Control Delay (s) | 4.0 | 0.0 | 20.5 |
| Lane LOS | A | | C |
| Approach Delay (s) | 4.0 | 0.0 | 20.5 |
| Approach LOS | | | C |

| Intersection Summary | | | |
|-----------------------------------|--|-------|------------------------|
| Average Delay | | 7.1 | |
| Intersection Capacity Utilization | | 57.0% | ICU Level of Service B |
| Analysis Period (min) | | 15 | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|------|------|------|------|------|------|
| Start Time | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 |
| End Time | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 1458 | 1469 | 1391 | 1501 | 1499 | 1463 |
| Vehs Exited | 1444 | 1475 | 1393 | 1491 | 1496 | 1460 |
| Starting Vehs | 14 | 22 | 21 | 15 | 12 | 13 |
| Ending Vehs | 28 | 16 | 19 | 25 | 15 | 19 |
| Denied Entry Before | 0 | 1 | 0 | 1 | 0 | 0 |
| Denied Entry After | 0 | 0 | 1 | 0 | 0 | 0 |
| Travel Distance (mi) | 435 | 448 | 429 | 464 | 463 | 448 |
| Travel Time (hr) | 19.1 | 21.0 | 19.7 | 23.5 | 21.5 | 20.9 |
| Total Delay (hr) | 5.5 | 6.9 | 6.3 | 9.0 | 7.0 | 6.9 |
| Total Stops | 808 | 849 | 814 | 878 | 882 | 846 |
| Fuel Used (gal) | 22.3 | 23.3 | 22.0 | 24.5 | 23.9 | 23.2 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 3:50 |
| End Time | 4:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 4:00 |
| End Time | 5:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|------|------|------|------|------|------|
| Vehs Entered | 1458 | 1469 | 1391 | 1501 | 1499 | 1463 |
| Vehs Exited | 1444 | 1475 | 1393 | 1491 | 1496 | 1460 |
| Starting Vehs | 14 | 22 | 21 | 15 | 12 | 13 |
| Ending Vehs | 28 | 16 | 19 | 25 | 15 | 19 |
| Denied Entry Before | 0 | 1 | 0 | 1 | 0 | 0 |
| Denied Entry After | 0 | 0 | 1 | 0 | 0 | 0 |
| Travel Distance (mi) | 435 | 448 | 429 | 464 | 463 | 448 |
| Travel Time (hr) | 19.1 | 21.0 | 19.7 | 23.5 | 21.5 | 20.9 |
| Total Delay (hr) | 5.5 | 6.9 | 6.3 | 9.0 | 7.0 | 6.9 |
| Total Stops | 808 | 849 | 814 | 878 | 882 | 846 |
| Fuel Used (gal) | 22.3 | 23.3 | 22.0 | 24.5 | 23.9 | 23.2 |

Intersection: 3: NB 101 Ramps/Ramada Drive & Main Street



















| Movement | EB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|
| Directions Served | LTR | LTR | L | TR | L | TR |
| Maximum Queue (ft) | 80 | 92 | 149 | 352 | 118 | 139 |
| Average Queue (ft) | 21 | 25 | 98 | 93 | 44 | 62 |
| 95th Queue (ft) | 57 | 70 | 163 | 265 | 89 | 113 |
| Link Distance (ft) | 509 | 406 | | 579 | | 638 |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | | | 100 | | 100 | |
| Storage Blk Time (%) | | | 24 | 0 | 1 | 3 |
| Queuing Penalty (veh) | | | 23 | 0 | 2 | 3 |

Zone Summary

Zone wide Queuing Penalty: 28

HCM Unsignalized Intersection Capacity Analysis
 3: NB 101 Ramps/Ramada Drive & Main Street

Existing Conditions Alt 7
 Timing Plan: AM Peak

| |  |  |  |  |  |  |  |  |  |  |  |  |
|-----------------------------------|---|---|---|---|---|---|---|---|---|---|---|---|
| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
| Lane Configurations | |  | | |  | |  |  | |  |  | |
| Volume (veh/h) | 75 | 119 | 95 | 52 | 134 | 122 | 111 | 36 | 58 | 40 | 16 | 44 |
| Sign Control | | Free | | | Free | | | Stop | | | Stop | |
| Grade | | 0% | | | 0% | | | 0% | | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 83 | 132 | 106 | 58 | 149 | 136 | 123 | 40 | 64 | 44 | 18 | 49 |
| Pedestrians | | | | | | | | | | | | |
| Lane Width (ft) | | | | | | | | | | | | |
| Walking Speed (ft/s) | | | | | | | | | | | | |
| Percent Blockage | | | | | | | | | | | | |
| Right turn flare (veh) | | | | | | | | | | | | |
| Median type | | None | | | None | | | | | | | |
| Median storage (veh) | | | | | | | | | | | | |
| Upstream signal (ft) | | | | | | | | | | | | |
| pX, platoon unblocked | | | | | | | | | | | | |
| vC, conflicting volume | 284 | | | 238 | | | 742 | 752 | 185 | 768 | 737 | 217 |
| vC1, stage 1 conf vol | | | | | | | | | | | | |
| vC2, stage 2 conf vol | | | | | | | | | | | | |
| vCu, unblocked vol | 284 | | | 238 | | | 742 | 752 | 185 | 768 | 737 | 217 |
| tC, single (s) | 4.2 | | | 4.2 | | | 7.2 | 6.6 | 6.3 | 7.2 | 6.6 | 6.3 |
| tC, 2 stage (s) | | | | | | | | | | | | |
| tF (s) | 2.3 | | | 2.3 | | | 3.6 | 4.1 | 3.4 | 3.6 | 4.1 | 3.4 |
| p0 queue free % | 93 | | | 96 | | | 54 | 86 | 92 | 81 | 94 | 94 |
| cM capacity (veh/h) | 1244 | | | 1295 | | | 267 | 296 | 842 | 237 | 302 | 808 |
| Direction, Lane # | EB 1 | WB 1 | NB 1 | NB 2 | SB 1 | SB 2 | | | | | | |
| Volume Total | 321 | 342 | 123 | 104 | 44 | 67 | | | | | | |
| Volume Left | 83 | 58 | 123 | 0 | 44 | 0 | | | | | | |
| Volume Right | 106 | 136 | 0 | 64 | 0 | 49 | | | | | | |
| cSH | 1244 | 1295 | 267 | 494 | 237 | 559 | | | | | | |
| Volume to Capacity | 0.07 | 0.04 | 0.46 | 0.21 | 0.19 | 0.12 | | | | | | |
| Queue Length 95th (ft) | 5 | 4 | 57 | 20 | 17 | 10 | | | | | | |
| Control Delay (s) | 2.6 | 1.7 | 29.5 | 14.2 | 23.7 | 12.3 | | | | | | |
| Lane LOS | A | A | D | B | C | B | | | | | | |
| Approach Delay (s) | 2.6 | 1.7 | 22.5 | | 16.9 | | | | | | | |
| Approach LOS | | | C | | C | | | | | | | |
| Intersection Summary | | | | | | | | | | | | |
| Average Delay | | | 8.4 | | | | | | | | | |
| Intersection Capacity Utilization | | | 45.2% | | ICU Level of Service | | | | A | | | |
| Analysis Period (min) | | | 15 | | | | | | | | | |

Summary of All Intervals

| | |
|-------------------------|------|
| Start Time | 6:50 |
| End Time | 8:00 |
| Total Time (min) | 70 |
| Time Recorded (min) | 60 |
| # of Intervals | 2 |
| # of Recorded Intervals | 1 |
| Vehs Entered | 1224 |
| Vehs Exited | 1210 |
| Starting Vehs | 8 |
| Ending Vehs | 22 |
| Travel Distance (mi) | 362 |
| Travel Time (hr) | 14.6 |
| Total Delay (hr) | 3.1 |
| Total Stops | 522 |
| Fuel Used (gal) | 18.4 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 6:50 |
| End Time | 7:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 7:00 |
| End Time | 8:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |
| Vehs Entered | 1224 |
| Vehs Exited | 1210 |
| Starting Vehs | 8 |
| Ending Vehs | 22 |
| Travel Distance (mi) | 362 |
| Travel Time (hr) | 14.6 |
| Total Delay (hr) | 3.1 |
| Total Stops | 522 |
| Fuel Used (gal) | 18.4 |

Intersection: 3: NB 101 Ramps/Ramada Drive & Main Street

| Movement | EB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|
| Directions Served | LTR | LTR | L | TR | L | TR |
| Maximum Queue (ft) | 114 | 77 | 76 | 76 | 66 | 119 |
| Average Queue (ft) | 21 | 22 | 43 | 39 | 26 | 38 |
| 95th Queue (ft) | 67 | 62 | 71 | 66 | 49 | 81 |
| Link Distance (ft) | 509 | 406 | | 579 | | 638 |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | | | 100 | | 100 | |
| Storage Blk Time (%) | | | | | | 1 |
| Queuing Penalty (veh) | | | | | | 1 |

HCM Unsignalized Intersection Capacity Analysis

3: NB 101 Ramps/Ramada Drive & Main Street

Existing Conditions Alt 7
Timing Plan: PM Peak



| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|------|------|------|------|------|------|------|------|------|------|------|------|
| Lane Configurations | | ↕ | | | ↕ | | ↗ | ↘ | | ↗ | ↘ | |
| Volume (veh/h) | 82 | 123 | 25 | 119 | 159 | 45 | 208 | 34 | 54 | 81 | 67 | 88 |
| Sign Control | | Free | | | Free | | | Stop | | | Stop | |
| Grade | | 0% | | | 0% | | | 0% | | | 0% | |
| Peak Hour Factor | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 | 0.90 |
| Hourly flow rate (vph) | 91 | 137 | 28 | 132 | 177 | 50 | 231 | 38 | 60 | 90 | 74 | 98 |
| Pedestrians | | | | | | | | | | | | |
| Lane Width (ft) | | | | | | | | | | | | |
| Walking Speed (ft/s) | | | | | | | | | | | | |
| Percent Blockage | | | | | | | | | | | | |
| Right turn flare (veh) | | | | | | | | | | | | |
| Median type | | None | | | None | | | | | | | |
| Median storage (veh) | | | | | | | | | | | | |
| Upstream signal (ft) | | | | | | | | | | | | |
| pX, platoon unblocked | | | | | | | | | | | | |
| vC, conflicting volume | 227 | | | 164 | | | 934 | 824 | 151 | 878 | 813 | 202 |
| vC1, stage 1 conf vol | | | | | | | | | | | | |
| vC2, stage 2 conf vol | | | | | | | | | | | | |
| vCu, unblocked vol | 227 | | | 164 | | | 934 | 824 | 151 | 878 | 813 | 202 |
| tC, single (s) | 4.2 | | | 4.2 | | | 7.2 | 6.6 | 6.3 | 7.2 | 6.6 | 6.3 |
| tC, 2 stage (s) | | | | | | | | | | | | |
| tF (s) | 2.3 | | | 2.3 | | | 3.6 | 4.1 | 3.4 | 3.6 | 4.1 | 3.4 |
| p0 queue free % | 93 | | | 90 | | | 0 | 85 | 93 | 53 | 71 | 88 |
| cM capacity (veh/h) | 1307 | | | 1378 | | | 145 | 254 | 880 | 191 | 257 | 824 |

| Direction, Lane # | EB 1 | WB 1 | NB 1 | NB 2 | SB 1 | SB 2 |
|------------------------|------|------|-------|------|------|------|
| Volume Total | 256 | 359 | 231 | 98 | 90 | 172 |
| Volume Left | 91 | 132 | 231 | 0 | 90 | 0 |
| Volume Right | 28 | 50 | 0 | 60 | 0 | 98 |
| cSH | 1307 | 1378 | 145 | 450 | 191 | 422 |
| Volume to Capacity | 0.07 | 0.10 | 1.59 | 0.22 | 0.47 | 0.41 |
| Queue Length 95th (ft) | 6 | 8 | 403 | 20 | 57 | 49 |
| Control Delay (s) | 3.2 | 3.5 | 351.3 | 15.2 | 39.7 | 19.3 |
| Lane LOS | A | A | F | C | E | C |
| Approach Delay (s) | 3.2 | 3.5 | 251.4 | | 26.3 | |
| Approach LOS | | | F | | D | |

Intersection Summary

| | |
|-----------------------------------|-------|
| Average Delay | 76.0 |
| Intersection Capacity Utilization | 53.8% |
| ICU Level of Service | A |
| Analysis Period (min) | 15 |

Summary of All Intervals

| | |
|-------------------------|------|
| Start Time | 3:50 |
| End Time | 5:00 |
| Total Time (min) | 70 |
| Time Recorded (min) | 60 |
| # of Intervals | 2 |
| # of Recorded Intervals | 1 |
| Vehs Entered | 1489 |
| Vehs Exited | 1490 |
| Starting Vehs | 20 |
| Ending Vehs | 19 |
| Travel Distance (mi) | 455 |
| Travel Time (hr) | 21.2 |
| Total Delay (hr) | 7.0 |
| Total Stops | 826 |
| Fuel Used (gal) | 23.7 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 3:50 |
| End Time | 4:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

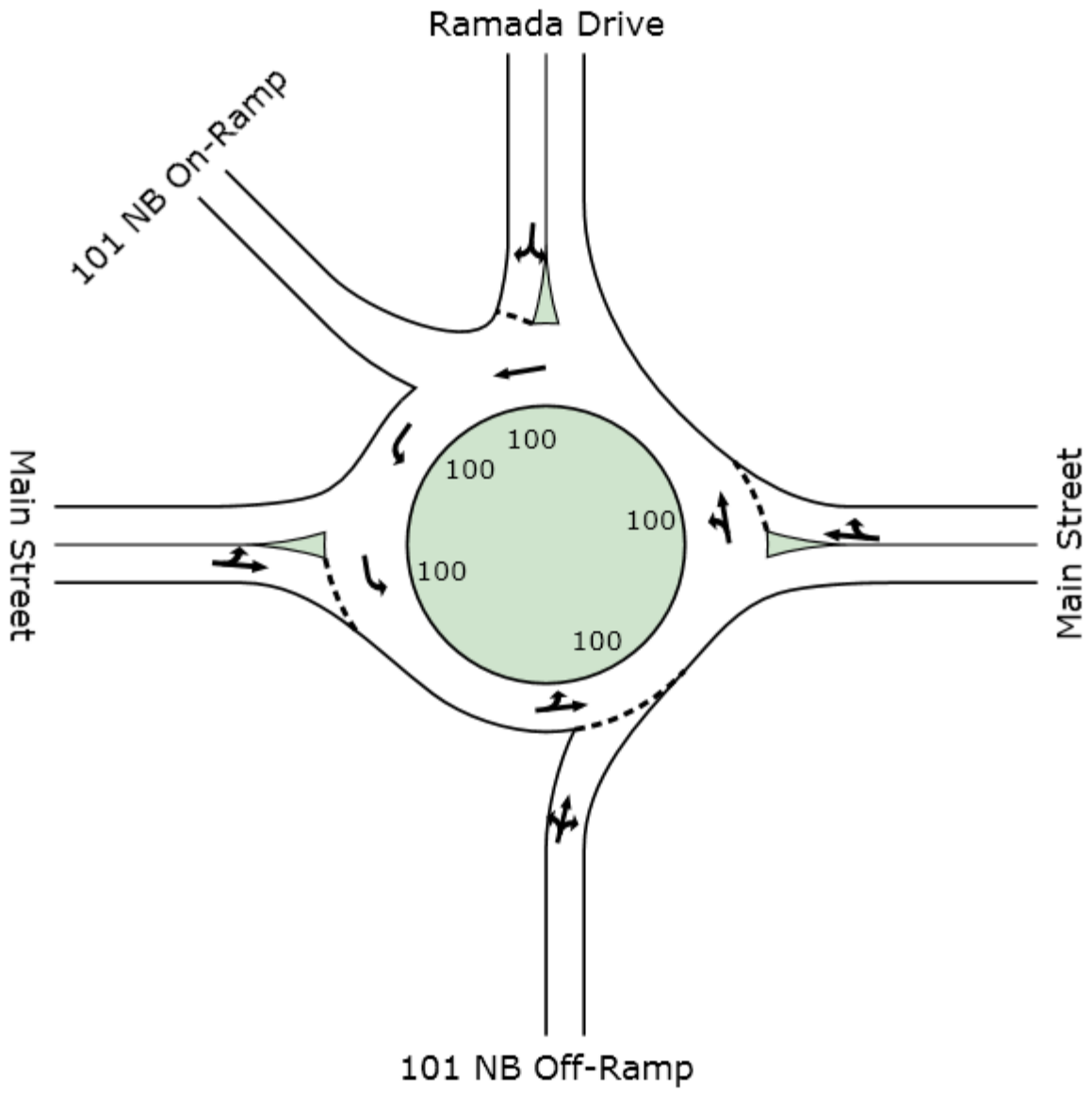
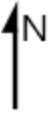
| | |
|--|------|
| Start Time | 4:00 |
| End Time | 5:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |
| Vehs Entered | 1489 |
| Vehs Exited | 1490 |
| Starting Vehs | 20 |
| Ending Vehs | 19 |
| Travel Distance (mi) | 455 |
| Travel Time (hr) | 21.2 |
| Total Delay (hr) | 7.0 |
| Total Stops | 826 |
| Fuel Used (gal) | 23.7 |

Intersection: 3: NB 101 Ramps/Ramada Drive & Main Street

| Movement | EB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|
| Directions Served | LTR | LTR | L | TR | L | TR |
| Maximum Queue (ft) | 52 | 95 | 150 | 397 | 71 | 117 |
| Average Queue (ft) | 24 | 30 | 104 | 98 | 41 | 64 |
| 95th Queue (ft) | 48 | 75 | 168 | 283 | 68 | 102 |
| Link Distance (ft) | 509 | 406 | | 579 | | 638 |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | | | 100 | | 100 | |
| Storage Blk Time (%) | | | 30 | 0 | | 2 |
| Queuing Penalty (veh) | | | 29 | 0 | | 2 |

Zone Summary

Zone wide Queuing Penalty: 31



MOVEMENT SUMMARY

Site: 101 NB Ramps_AM

Highway 101/Main Street Interchange Analysis
 Existing Conditions with Alternative 5
 AM Peak Hour
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|---------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| South: 101 NB Off-Ramp | | | | | | | | | | | |
| 3 | L | 123 | 8.0 | 0.252 | 6.6 | LOS A | 1.1 | 29.5 | 0.53 | 0.84 | 28.4 |
| 8 | T | 76 | 8.0 | 0.252 | 6.6 | LOS A | 1.1 | 29.5 | 0.53 | 0.66 | 31.4 |
| 18 | R | 29 | 8.0 | 0.252 | 6.6 | LOS A | 1.1 | 29.5 | 0.53 | 0.71 | 30.9 |
| Approach | | 228 | 8.0 | 0.252 | 6.6 | LOS A | 1.1 | 29.5 | 0.53 | 0.77 | 29.6 |
| East: Main Street | | | | | | | | | | | |
| 6 | T | 189 | 8.0 | 0.374 | 8.1 | LOS A | 1.8 | 48.3 | 0.57 | 0.71 | 30.7 |
| 16 | R | 153 | 8.0 | 0.374 | 8.1 | LOS A | 1.8 | 48.3 | 0.57 | 0.74 | 30.3 |
| Approach | | 342 | 8.0 | 0.374 | 8.1 | LOS A | 1.8 | 48.3 | 0.57 | 0.72 | 30.5 |
| North: Ramada Drive | | | | | | | | | | | |
| 7 | L | 44 | 8.0 | 0.134 | 5.6 | LOS A | 0.5 | 14.1 | 0.52 | 0.82 | 28.8 |
| 14 | R | 68 | 8.0 | 0.134 | 5.6 | LOS A | 0.5 | 14.1 | 0.52 | 0.72 | 31.2 |
| Approach | | 112 | 8.0 | 0.134 | 5.6 | LOS A | 0.5 | 14.1 | 0.52 | 0.76 | 30.1 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 154 | 8.0 | 0.253 | 5.1 | LOS A | 1.3 | 33.8 | 0.18 | 0.80 | 28.7 |
| 2 | T | 168 | 8.0 | 0.253 | 5.1 | LOS A | 1.3 | 33.8 | 0.18 | 0.44 | 32.8 |
| Approach | | 322 | 8.0 | 0.253 | 5.1 | LOS A | 1.3 | 33.8 | 0.18 | 0.61 | 30.6 |
| All Vehicles | | 1004 | 8.0 | 0.374 | 6.5 | LOS A | 1.8 | 48.3 | 0.43 | 0.70 | 30.3 |

Level of Service (LOS) Method: Delay (HCM 2000).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay per movement

Intersection and Approach LOS values are based on average delay for all vehicle movements.

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

Processed: Thursday, April 19, 2012 3:03:55 PM

SIDRA INTERSECTION 5.1.11.2079

Project: T:\16128\Traffic\Synchro\Deliverable3_Alternatives\Alt_5_DoubleRoundabout\Existing

\101_NB_Ramps_Main_Ramada.sip

8001416, RICK ENGINEERING COMPANY, SINGLE

Copyright © 2000-2011 Akcelik and Associates Pty Ltd

www.sidrasolutions.com

SIDRA
INTERSECTION

MOVEMENT SUMMARY

Site: 101 NB Ramps_PM

Highway 101/Main Street Interchange Analysis
 Existing Conditions with Alternative 5
 PM Peak Hour
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | | |
|---------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|--|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph | |
| South: 101 NB Off-Ramp | | | | | | | | | | | | |
| 3 | L | 234 | 3.0 | 0.334 | 7.1 | LOS A | 1.7 | 43.2 | 0.55 | 0.82 | 28.0 | |
| 8 | T | 74 | 3.0 | 0.334 | 7.1 | LOS A | 1.7 | 43.2 | 0.55 | 0.66 | 30.9 | |
| 18 | R | 20 | 3.0 | 0.334 | 7.1 | LOS A | 1.7 | 43.2 | 0.55 | 0.70 | 30.5 | |
| Approach | | 329 | 3.0 | 0.334 | 7.1 | LOS A | 1.7 | 43.2 | 0.55 | 0.78 | 28.7 | |
| East: Main Street | | | | | | | | | | | | |
| 6 | T | 239 | 3.0 | 0.381 | 8.1 | LOS A | 2.0 | 50.3 | 0.60 | 0.73 | 30.8 | |
| 16 | R | 120 | 3.0 | 0.381 | 8.1 | LOS A | 2.0 | 50.3 | 0.60 | 0.75 | 30.6 | |
| Approach | | 359 | 3.0 | 0.381 | 8.1 | LOS A | 2.0 | 50.3 | 0.60 | 0.73 | 30.7 | |
| North: Ramada Drive | | | | | | | | | | | | |
| 7 | L | 90 | 3.0 | 0.335 | 8.6 | LOS A | 1.5 | 39.4 | 0.65 | 0.93 | 27.3 | |
| 14 | R | 172 | 3.0 | 0.335 | 8.6 | LOS A | 1.5 | 39.4 | 0.65 | 0.85 | 29.3 | |
| Approach | | 262 | 3.0 | 0.335 | 8.6 | LOS A | 1.5 | 39.4 | 0.65 | 0.88 | 28.6 | |
| West: Main Street | | | | | | | | | | | | |
| 5 | L | 80 | 3.0 | 0.199 | 4.5 | LOS A | 1.0 | 25.1 | 0.25 | 0.81 | 29.3 | |
| 2 | T | 174 | 3.0 | 0.199 | 4.5 | LOS A | 1.0 | 25.1 | 0.25 | 0.47 | 33.2 | |
| Approach | | 254 | 3.0 | 0.199 | 4.5 | LOS A | 1.0 | 25.1 | 0.25 | 0.58 | 31.8 | |
| All Vehicles | | 1204 | 3.0 | 0.381 | 7.2 | LOS A | 2.0 | 50.3 | 0.52 | 0.75 | 29.9 | |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

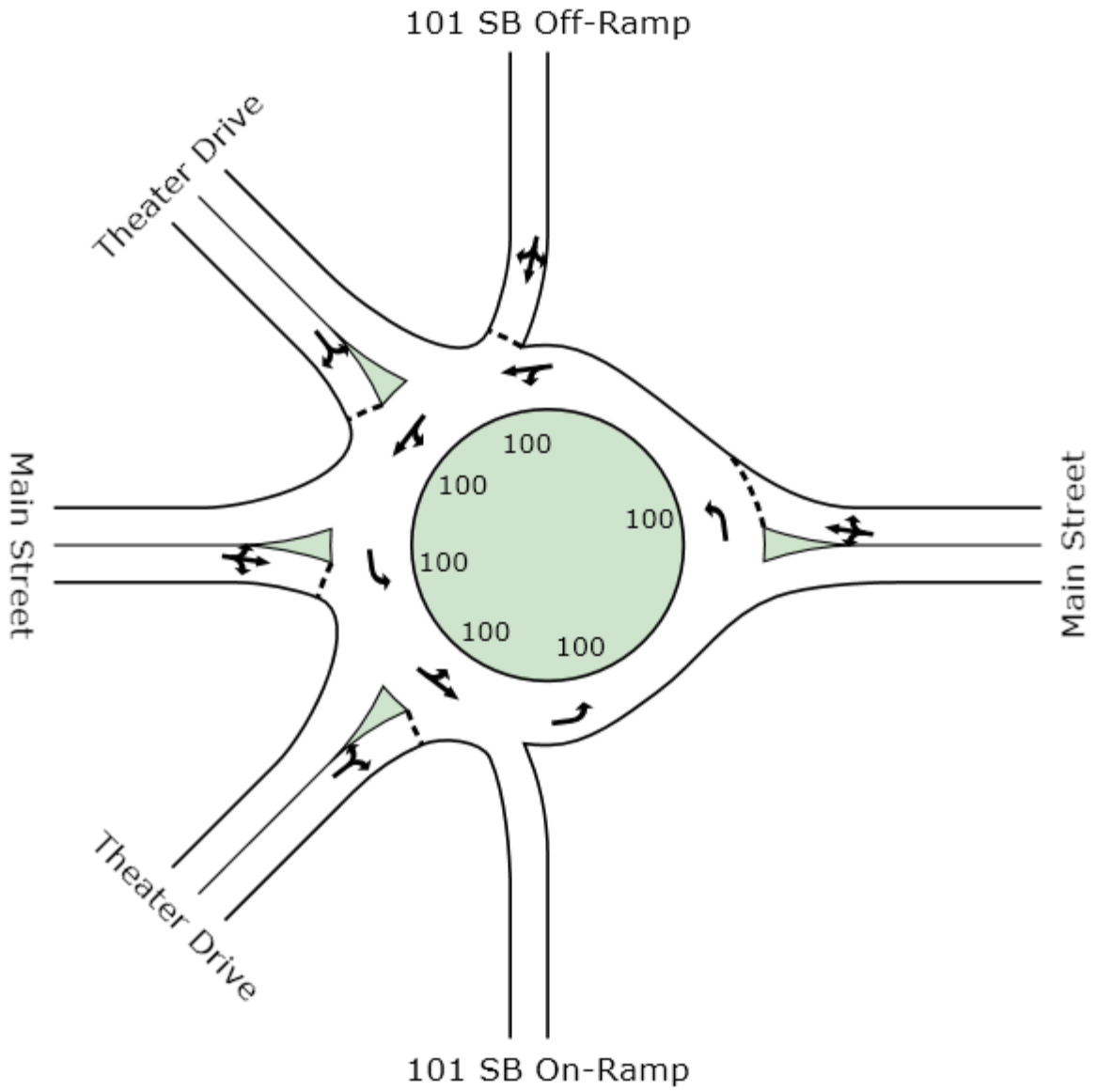
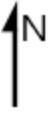
Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.



MOVEMENT SUMMARY

Site: 101 SB Ramps_AM

Highway 101/Main Street Interchange Analysis
 Existing Conditions with Alternative 5
 AM Peak
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | | |
|---------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|--|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph | |
| East: Main Street | | | | | | | | | | | | |
| 1 | L | 91 | 13.0 | 0.254 | 5.1 | LOS A | 1.2 | 33.8 | 0.07 | 0.87 | 29.0 | |
| 6 | T | 23 | 13.0 | 0.254 | 5.1 | LOS A | 1.2 | 33.8 | 0.07 | 0.45 | 32.9 | |
| 16 | R | 207 | 13.0 | 0.254 | 5.1 | LOS A | 1.2 | 33.8 | 0.07 | 0.45 | 32.9 | |
| Approach | | 321 | 13.0 | 0.254 | 5.1 | LOS A | 1.2 | 33.8 | 0.07 | 0.57 | 31.6 | |
| North: 101 SB Off-Ramp | | | | | | | | | | | | |
| 7 | L | 187 | 13.0 | 0.230 | 6.4 | LOS A | 0.9 | 26.2 | 0.50 | 0.78 | 28.2 | |
| 4 | T | 1 | 13.0 | 0.230 | 6.4 | LOS A | 0.9 | 26.2 | 0.50 | 0.62 | 31.2 | |
| 14 | R | 16 | 13.0 | 0.230 | 6.4 | LOS A | 0.9 | 26.2 | 0.50 | 0.69 | 30.6 | |
| Approach | | 203 | 13.0 | 0.230 | 6.4 | LOS A | 0.9 | 26.2 | 0.50 | 0.78 | 28.4 | |
| North West: Theater Drive | | | | | | | | | | | | |
| 7X | L | 127 | 13.0 | 0.292 | 7.1 | LOS A | 1.3 | 35.1 | 0.51 | 0.82 | 28.7 | |
| 14X | R | 138 | 13.0 | 0.292 | 7.1 | LOS A | 1.3 | 35.1 | 0.51 | 0.65 | 31.1 | |
| Approach | | 264 | 13.0 | 0.292 | 7.1 | LOS A | 1.3 | 35.1 | 0.51 | 0.73 | 29.8 | |
| West: Main Street | | | | | | | | | | | | |
| 5 | L | 6 | 13.0 | 0.052 | 5.7 | LOS A | 0.2 | 5.1 | 0.55 | 0.87 | 28.5 | |
| 2 | T | 6 | 13.0 | 0.052 | 5.7 | LOS A | 0.2 | 5.1 | 0.55 | 0.67 | 32.1 | |
| 12 | R | 26 | 13.0 | 0.052 | 5.7 | LOS A | 0.2 | 5.1 | 0.55 | 0.71 | 31.7 | |
| Approach | | 37 | 13.0 | 0.052 | 5.7 | LOS A | 0.2 | 5.1 | 0.55 | 0.73 | 31.2 | |
| South West: Theater Drive | | | | | | | | | | | | |
| 5X | L | 2 | 13.0 | 0.012 | 5.5 | LOS A | 0.0 | 1.1 | 0.55 | 0.78 | 29.0 | |
| 12X | R | 6 | 13.0 | 0.012 | 5.5 | LOS A | 0.0 | 1.1 | 0.55 | 0.65 | 31.7 | |
| Approach | | 8 | 13.0 | 0.012 | 5.5 | LOS A | 0.0 | 1.1 | 0.55 | 0.69 | 30.8 | |
| All Vehicles | | 833 | 13.0 | 0.292 | 6.1 | LOS A | 1.3 | 35.1 | 0.34 | 0.68 | 30.2 | |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

MOVEMENT SUMMARY

Site: 101 SB Ramps_PM

Highway 101/Main Street Interchange Analysis
 Existing Conditions with Alternative 5
 PM Peak Hour
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|----------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| East: Main Street | | | | | | | | | | | |
| 1 | L | 181 | 5.0 | 0.372 | 6.1 | LOS A | 2.3 | 60.0 | 0.08 | 0.85 | 28.5 |
| 6 | T | 22 | 5.0 | 0.372 | 6.1 | LOS A | 2.3 | 60.0 | 0.08 | 0.45 | 32.2 |
| 16 | R | 302 | 5.0 | 0.372 | 6.1 | LOS A | 2.3 | 60.0 | 0.08 | 0.45 | 32.2 |
| Approach | | 506 | 5.0 | 0.372 | 6.1 | LOS A | 2.3 | 60.0 | 0.08 | 0.59 | 30.7 |
| North: 101 SB Off-Ramp | | | | | | | | | | | |
| 7 | L | 120 | 5.0 | 0.182 | 6.4 | LOS A | 0.8 | 19.6 | 0.57 | 0.85 | 28.3 |
| 4 | T | 1 | 5.0 | 0.182 | 6.4 | LOS A | 0.8 | 19.6 | 0.57 | 0.71 | 31.2 |
| 14 | R | 26 | 5.0 | 0.182 | 6.4 | LOS A | 0.8 | 19.6 | 0.57 | 0.76 | 30.6 |
| Approach | | 147 | 5.0 | 0.182 | 6.4 | LOS A | 0.8 | 19.6 | 0.57 | 0.83 | 28.7 |
| North West: Theater Drive | | | | | | | | | | | |
| 7X | L | 128 | 5.0 | 0.341 | 7.3 | LOS A | 1.7 | 44.1 | 0.54 | 0.84 | 28.6 |
| 14X | R | 204 | 5.0 | 0.341 | 7.3 | LOS A | 1.7 | 44.1 | 0.54 | 0.67 | 31.0 |
| Approach | | 332 | 5.0 | 0.341 | 7.3 | LOS A | 1.7 | 44.1 | 0.54 | 0.73 | 30.0 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 7 | 5.0 | 0.034 | 5.4 | LOS A | 0.1 | 3.3 | 0.56 | 0.86 | 28.6 |
| 2 | T | 6 | 5.0 | 0.034 | 5.4 | LOS A | 0.1 | 3.3 | 0.56 | 0.67 | 32.3 |
| 12 | R | 12 | 5.0 | 0.034 | 5.4 | LOS A | 0.1 | 3.3 | 0.56 | 0.71 | 31.9 |
| Approach | | 24 | 5.0 | 0.034 | 5.4 | LOS A | 0.1 | 3.3 | 0.56 | 0.74 | 30.9 |
| South West: Theater Drive | | | | | | | | | | | |
| 5X | L | 1 | 5.0 | 0.017 | 5.3 | LOS A | 0.1 | 1.7 | 0.57 | 0.81 | 29.1 |
| 12X | R | 11 | 5.0 | 0.017 | 5.3 | LOS A | 0.1 | 1.7 | 0.57 | 0.69 | 31.7 |
| Approach | | 12 | 5.0 | 0.017 | 5.3 | LOS A | 0.1 | 1.7 | 0.57 | 0.70 | 31.4 |
| All Vehicles | | 1021 | 5.0 | 0.372 | 6.5 | LOS A | 2.3 | 60.0 | 0.32 | 0.68 | 30.2 |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

APPENDIX C
BUILDOUT SIGNAL WARRANT ANALYSIS
WORKSHEETS

 US 101 / Main Street Traffic Study
 2030 Buildout Alt 2 & 3
 PM Peak

Scenario Report

Scenario: 2030 PM Alt 2 & 3
 Command: 2030 PM Alt 2 & 3
 Volume: 2030 PM Alt 2 & 3
 Geometry: 2030 Alt 2 & 3
 Impact Fee: Default Impact Fee
 Trip Generation: Default Trip Generation
 Trip Distribution: Default Trip Distribution
 Paths: Default Path
 Routes: Default Route
 Configuration: Default Configuration

 US 101 / Main Street Traffic Study
 2030 Buildout Alt 2 & 3
 PM Peak

Signal Warrant Summary Report

| Intersection | Base Met | | Future Met | |
|-----------------------------------|-------------|-----------|-------------|-----------|
| | [Del / Vol] | Yes / Yes | [Del / Vol] | Yes / Yes |
| # 1 Main Street and Theatre Drive | Yes / Yes | Yes / Yes | ??? | ??? |
| # 4 Main Street and Ramada Drive | Yes / Yes | Yes / Yes | ??? | ??? |

US 101 / Main Street Traffic Study
2030 Buildout Alt 2 & 3
PM Peak

Peak Hour Delay Signal Warrant Report

Intersection #1 Main Street and Theatre Drive

Base Volume Alternative: Peak Hour Warrant Met

Table with 4 columns: Approach, Movement, Control, Lanes. Rows include North Bound, South Bound, East Bound, West Bound with various traffic signal details.

Approach[northbound] [lanes=1] [control=Stop Sign]
Signal Warrant Rule #1: [vehicle-hours=0.3]
FAIL - Vehicle-hours less than 4 for one lane approach.
Signal Warrant Rule #2: [approach volume=90]
FAIL - Approach volume less than 100 for one lane approach.
Signal Warrant Rule #3: [approach count=3] [total volume=1210]
SUCCEED - Total volume greater than or equal to 650 for intersection with less than four approaches.

Approach[southbound] [lanes=1] [control=Stop Sign]
Signal Warrant Rule #1: [vehicle-hours=23.7]
SUCCEED - Vehicle-hours greater than or equal to 4 for one lane approach.
Signal Warrant Rule #2: [approach volume=550]
SUCCEED - Approach volume greater than or equal to 100 for one lane approach.
Signal Warrant Rule #3: [approach count=3] [total volume=1210]
SUCCEED - Total volume greater than or equal to 650 for intersection with less than four approaches.

SIGNAL WARRANT DISCLAIMER
This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future. Intersections that exceed this warrant are probably more likely to meet one or more of the other volume based signal warrant (such as the 4-hour or 8-hour warrants).

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction. Consideration of the other signal warrants, which is beyond the scope of this software, may yield different results.

US 101 / Main Street Traffic Study
2030 Buildout Alt 2 & 3
PM Peak

Peak Hour Volume Signal Warrant Report [Urban]

Intersection #1 Main Street and Theatre Drive

Base Volume Alternative: Peak Hour Warrant Met

Table with 4 columns: Approach, Movement, Control, Lanes. Rows include North Bound, South Bound, East Bound, West Bound with various traffic signal details.

Major Street Volume: 570
Minor Approach Volume: 550
Minor Approach Volume Threshold: 369

SIGNAL WARRANT DISCLAIMER
This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future. Intersections that exceed this warrant are probably more likely to meet one or more of the other volume based signal warrant (such as the 4-hour or 8-hour warrants).

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction. Consideration of the other signal warrants, which is beyond the scope of this software, may yield different results.

US 101 / Main Street Traffic Study
2030 Buildout Alt 2 & 3
PM Peak

Peak Hour Delay Signal Warrant Report

Intersection #4 Main Street and Ramada Drive

Base Volume Alternative: Peak Hour Warrant Met

Table with 4 columns: Approach (North Bound, South Bound, East Bound, West Bound), Movement (L-T-R), Control (Stop Sign, Uncontrolled), and Initial Vol. Values include 0, 210, 290, 400.

Approach[southbound] [lanes=1] [control=Stop Sign]

Signal Warrant Rule #1: [vehicle-hours=142.6]

SUCCEED - Vehicle-hours greater than or equal to 4 for one lane approach.

Signal Warrant Rule #2: [approach volume=600]

SUCCEED - Approach volume greater than or equal to 100 for one lane approach.

Signal Warrant Rule #3: [approach count=3] [total volume=1630]

SUCCEED - Total volume greater than or equal to 650 for intersection with less than four approaches.

SIGNAL WARRANT DISCLAIMER

This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future.

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction.

US 101 / Main Street Traffic Study
2030 Buildout Alt 2 & 3
PM Peak

Peak Hour Volume Signal Warrant Report [Urban]

Intersection #4 Main Street and Ramada Drive

Base Volume Alternative: Peak Hour Warrant Met

Table with 4 columns: Approach (North Bound, South Bound, East Bound, West Bound), Movement (L-T-R), Control (Stop Sign, Uncontrolled), and Initial Vol. Values include 0, 210, 290, 400.

Major Street Volume: 1030

Minor Approach Volume: 600

Minor Approach Volume Threshold: 212

SIGNAL WARRANT DISCLAIMER

This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future.

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction.

Peak Hour Delay Signal Warrant Report

Intersection #5 Ramada Drive and US 101 NB Ramps

Future Volume Alternative: Peak Hour Warrant Met

| Approach: | North Bound | | | | South Bound | | | | East Bound | | | | West Bound | | | | | | | |
|--------------|-------------|----|----|---|-------------|-----|-----|-----|--------------|---|-----|-----|--------------|---|---|-----|-----|-----|---|---|
| Movement: | L | T | R | | L | T | R | | L | T | R | | L | T | R | | | | | |
| Control: | Stop Sign | | | | Stop Sign | | | | Uncontrolled | | | | Uncontrolled | | | | | | | |
| Lanes: | 1 | 0 | 0 | 1 | 0 | 1 | 0 | 0 | 1 | 0 | 1 | 0 | 0 | 1 | 0 | 1 | 0 | 0 | 1 | 0 |
| Initial Vol: | 360 | 85 | 75 | | | 210 | 220 | 170 | | | 205 | 165 | 60 | | | 220 | 180 | 110 | | |
| ApproachDel: | +Inf | | | | +Inf | | | | xxxxxx | | | | xxxxxx | | | | | | | |

Approach[northbound] [lanes=2] [control=Stop Sign]

Signal Warrant Rule #1: [vehicle-hours=OVERFLOW]

SUCCEED - Vehicle-hours >= 5 for two or more lane approach.

Signal Warrant Rule #2: [approach volume=520]

SUCCEED - Approach volume >= 150 for two or more lane approach.

Signal Warrant Rule #3: [approach count=4][total volume=2060]

SUCCEED - Total volume greater than or equal to 800 for intersection with four or more approaches.

Approach[southbound] [lanes=2] [control=Stop Sign]

Signal Warrant Rule #1: [vehicle-hours=OVERFLOW]

SUCCEED - Vehicle-hours >= 5 for two or more lane approach.

Signal Warrant Rule #2: [approach volume=600]

SUCCEED - Approach volume >= 150 for two or more lane approach.

Signal Warrant Rule #3: [approach count=4][total volume=2060]

SUCCEED - Total volume greater than or equal to 800 for intersection with four or more approaches.

SIGNAL WARRANT DISCLAIMER

This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future. Intersections that exceed this warrant are probably more likely to meet one or more of the other volume based signal warrant (such as the 4-hour or 8-hour warrants).

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction. Consideration of the other signal warrants, which is beyond the scope of this software, may yield different results.

Peak Hour Volume Signal Warrant Report [Urban]

Intersection #5 Ramada Drive and US 101 NB Ramps

Future Volume Alternative: Peak Hour Warrant Met

| Approach: | North Bound | | | South Bound | | | East Bound | | | West Bound | | |
|----------------------------------|-------------|----|----|-------------|-----|-----|--------------|-----|----|--------------|-----|-----|
| Movement: | L | T | R | L | T | R | L | T | R | L | T | R |
| Control: | Stop Sign | | | Stop Sign | | | Uncontrolled | | | Uncontrolled | | |
| Lanes: | 1 | 0 | 0 | 1 | 0 | 0 | 1 | 0 | 0 | 1 | 0 | 0 |
| Initial Vol: | 360 | 85 | 75 | 210 | 220 | 170 | 205 | 165 | 60 | 220 | 180 | 110 |
| Major Street Volume: | 940 | | | | | | | | | | | |
| Minor Approach Volume: | 600 | | | | | | | | | | | |
| Minor Approach Volume Threshold: | 401 | | | | | | | | | | | |

SIGNAL WARRANT DISCLAIMER

This peak hour signal warrant analysis should be considered solely as an "indicator" of the likelihood of an unsignalized intersection warranting a traffic signal in the future. Intersections that exceed this warrant are probably more likely to meet one or more of the other volume based signal warrant (such as the 4-hour or 8-hour warrants).

The peak hour warrant analysis in this report is not intended to replace a rigorous and complete traffic signal warrant analysis by the responsible jurisdiction. Consideration of the other signal warrants, which is beyond the scope of this software, may yield different results.

APPENDIX D

BUILDOUT CONDITIONS WITH ALTERNATIVES

LOS AND QUEUE ANALYSIS WORKSHEETS

HCM Signalized Intersection Capacity Analysis
3: NB 101 Offramp/NB 101 Onramp & Main St.

2030 Build-Out Alt 6
Timing Plan: AM Peak



| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|-------|-------|------|------|------|------|-------|-------|------|------|------|------|
| Lane Configurations | | ↕ | | | ↑ | ↗ | | ↕ | ↗ | | | |
| Volume (vph) | 220 | 330 | 0 | 0 | 300 | 160 | 190 | 0 | 160 | 0 | 0 | 0 |
| Ideal Flow (vphpl) | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 |
| Lane Width | 12 | 15 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 |
| Total Lost time (s) | | 4.0 | | | 4.0 | 4.0 | | 4.0 | 4.0 | | | |
| Lane Util. Factor | | 1.00 | | | 1.00 | 1.00 | | 1.00 | 1.00 | | | |
| Frt | | 1.00 | | | 1.00 | 0.85 | | 1.00 | 0.85 | | | |
| Flt Protected | | 0.98 | | | 1.00 | 1.00 | | 0.95 | 1.00 | | | |
| Satd. Flow (prot) | | 1951 | | | 1810 | 1538 | | 1719 | 1538 | | | |
| Flt Permitted | | 0.73 | | | 1.00 | 1.00 | | 0.95 | 1.00 | | | |
| Satd. Flow (perm) | | 1462 | | | 1810 | 1538 | | 1719 | 1538 | | | |
| 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) | 239 | 359 | 0 | 0 | 326 | 174 | 207 | 0 | 174 | 0 | 0 | 0 |
| RTOR Reduction (vph) | 0 | 0 | 0 | 0 | 0 | 54 | 0 | 0 | 140 | 0 | 0 | 0 |
| Lane Group Flow (vph) | 0 | 598 | 0 | 0 | 326 | 120 | 0 | 207 | 34 | 0 | 0 | 0 |
| Turn Type | D.P+P | NA | | | NA | Perm | Split | NA | Perm | | | |
| Protected Phases | 5 | 2 | | | 6 | | 8 | 8 | | | | |
| Permitted Phases | 6 | | | | | 6 | | | 8 | | | |
| Actuated Green, G (s) | | 48.2 | | | 48.2 | 48.2 | | 13.8 | 13.8 | | | |
| Effective Green, g (s) | | 48.2 | | | 48.2 | 48.2 | | 13.8 | 13.8 | | | |
| Actuated g/C Ratio | | 0.69 | | | 0.69 | 0.69 | | 0.20 | 0.20 | | | |
| 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) | | 1006 | | | 1246 | 1059 | | 338 | 303 | | | |
| v/s Ratio Prot | | | | | 0.18 | | | c0.12 | | | | |
| v/s Ratio Perm | | c0.41 | | | | 0.08 | | | 0.02 | | | |
| v/c Ratio | | 0.59 | | | 0.26 | 0.11 | | 0.61 | 0.11 | | | |
| Uniform Delay, d1 | | 5.7 | | | 4.1 | 3.7 | | 25.7 | 23.1 | | | |
| Progression Factor | | 1.00 | | | 0.77 | 1.66 | | 1.00 | 1.00 | | | |
| Incremental Delay, d2 | | 0.9 | | | 0.5 | 0.2 | | 3.3 | 0.2 | | | |
| Delay (s) | | 6.7 | | | 3.6 | 6.3 | | 28.9 | 23.2 | | | |
| Level of Service | | A | | | A | A | | C | C | | | |
| Approach Delay (s) | | 6.7 | | | 4.6 | | | 26.3 | | | 0.0 | |
| Approach LOS | | A | | | A | | | C | | | A | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 11.0 | HCM 2000 Level of Service | B |
| HCM 2000 Volume to Capacity ratio | 0.64 | | |
| Actuated Cycle Length (s) | 70.0 | Sum of lost time (s) | 12.0 |
| Intersection Capacity Utilization | 65.9% | ICU Level of Service | C |
| Analysis Period (min) | 15 | | |

c Critical Lane Group

HCM Signalized Intersection Capacity Analysis
4: Main St. & Ramada Dr.

2030 Build-Out Alt 6
Timing Plan: AM Peak



| Movement | EBL | EBT | WBT | WBR | SBL | SBR |
|------------------------|-------|------|-------|------|-------|------|
| Lane Configurations | | | | | | |
| Volume (vph) | 280 | 210 | 310 | 310 | 100 | 150 |
| 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 | 1.00 | 1.00 | 0.95 | | 1.00 | 1.00 |
| Frt | 1.00 | 1.00 | 0.93 | | 1.00 | 0.85 |
| Flt Protected | 0.95 | 1.00 | 1.00 | | 0.95 | 1.00 |
| Satd. Flow (prot) | 1719 | 1810 | 3180 | | 1719 | 1538 |
| Flt Permitted | 0.95 | 1.00 | 1.00 | | 0.95 | 1.00 |
| Satd. Flow (perm) | 1719 | 1810 | 3180 | | 1719 | 1538 |
| Peak-hour factor, PHF | 0.92 | 0.92 | 0.92 | 0.92 | 0.92 | 0.92 |
| Adj. Flow (vph) | 304 | 228 | 337 | 337 | 109 | 163 |
| RTOR Reduction (vph) | 0 | 0 | 192 | 0 | 0 | 139 |
| Lane Group Flow (vph) | 304 | 228 | 482 | 0 | 109 | 24 |
| Turn Type | Prot | NA | NA | | Prot | Perm |
| Protected Phases | 5 | 2 | 6 | | 4 | |
| Permitted Phases | | | | | | 4 |
| Actuated Green, G (s) | 17.4 | 51.6 | 30.2 | | 10.4 | 10.4 |
| Effective Green, g (s) | 17.4 | 51.6 | 30.2 | | 10.4 | 10.4 |
| Actuated g/C Ratio | 0.25 | 0.74 | 0.43 | | 0.15 | 0.15 |
| 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) | 427 | 1334 | 1371 | | 255 | 228 |
| v/s Ratio Prot | c0.18 | 0.13 | c0.15 | | c0.06 | |
| v/s Ratio Perm | | | | | | 0.02 |
| v/c Ratio | 0.71 | 0.17 | 0.35 | | 0.43 | 0.11 |
| Uniform Delay, d1 | 24.0 | 2.8 | 13.3 | | 27.1 | 25.8 |
| Progression Factor | 1.14 | 0.67 | 1.00 | | 1.00 | 1.00 |
| Incremental Delay, d2 | 5.0 | 0.2 | 0.7 | | 1.2 | 0.2 |
| Delay (s) | 32.4 | 2.1 | 14.1 | | 28.2 | 26.0 |
| Level of Service | C | A | B | | C | C |
| Approach Delay (s) | | 19.4 | 14.1 | | 26.9 | |
| Approach LOS | | B | B | | C | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 18.3 | HCM 2000 Level of Service | B |
| HCM 2000 Volume to Capacity ratio | 0.47 | | |
| Actuated Cycle Length (s) | 70.0 | Sum of lost time (s) | 12.0 |
| Intersection Capacity Utilization | 52.4% | ICU Level of Service | A |
| Analysis Period (min) | 15 | | |
| c Critical Lane Group | | | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|------|------|------|------|------|------|
| Start Time | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 |
| End Time | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 2248 | 2284 | 2284 | 2307 | 2254 | 2277 |
| Vehs Exited | 2214 | 2276 | 2276 | 2288 | 2233 | 2258 |
| Starting Vehs | 28 | 39 | 39 | 39 | 27 | 32 |
| Ending Vehs | 62 | 47 | 47 | 58 | 48 | 49 |
| Denied Entry Before | 0 | 1 | 1 | 0 | 0 | 0 |
| Denied Entry After | 2 | 0 | 0 | 2 | 2 | 0 |
| Travel Distance (mi) | 903 | 925 | 925 | 927 | 915 | 919 |
| Travel Time (hr) | 46.2 | 49.3 | 49.3 | 49.4 | 47.0 | 48.2 |
| Total Delay (hr) | 18.5 | 20.9 | 20.9 | 20.9 | 18.9 | 20.0 |
| Total Stops | 2493 | 2769 | 2769 | 2827 | 2546 | 2679 |
| Fuel Used (gal) | 44.6 | 45.9 | 45.9 | 46.2 | 45.2 | 45.5 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 6:50 |
| End Time | 7:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 7:00 |
| End Time | 8:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|------|------|------|------|------|------|
| Vehs Entered | 2248 | 2284 | 2284 | 2307 | 2254 | 2277 |
| Vehs Exited | 2214 | 2276 | 2276 | 2288 | 2233 | 2258 |
| Starting Vehs | 28 | 39 | 39 | 39 | 27 | 32 |
| Ending Vehs | 62 | 47 | 47 | 58 | 48 | 49 |
| Denied Entry Before | 0 | 1 | 1 | 0 | 0 | 0 |
| Denied Entry After | 2 | 0 | 0 | 2 | 2 | 0 |
| Travel Distance (mi) | 903 | 925 | 925 | 927 | 915 | 919 |
| Travel Time (hr) | 46.2 | 49.3 | 49.3 | 49.4 | 47.0 | 48.2 |
| Total Delay (hr) | 18.5 | 20.9 | 20.9 | 20.9 | 18.9 | 20.0 |
| Total Stops | 2493 | 2769 | 2769 | 2827 | 2546 | 2679 |
| Fuel Used (gal) | 44.6 | 45.9 | 45.9 | 46.2 | 45.2 | 45.5 |

Intersection: 3: NB 101 Offramp/NB 101 Onramp & Main St.

| Movement | EB | WB | WB | NB | NB |
|-----------------------|-----|-----|-----|-----|-----|
| Directions Served | LT | T | R | LT | R |
| Maximum Queue (ft) | 415 | 116 | 50 | 224 | 121 |
| Average Queue (ft) | 191 | 32 | 12 | 102 | 49 |
| 95th Queue (ft) | 361 | 78 | 34 | 178 | 90 |
| Link Distance (ft) | 405 | 419 | 419 | 924 | |
| Upstream Blk Time (%) | 1 | | | | |
| Queuing Penalty (veh) | 5 | | | | |
| Storage Bay Dist (ft) | | | | | 200 |
| Storage Blk Time (%) | | | | 0 | |
| Queuing Penalty (veh) | | | | 1 | |

Intersection: 4: Main St. & Ramada Dr.

| Movement | EB | EB | WB | WB | SB | SB |
|-----------------------|-----|-----|-----|-----|------|-----|
| Directions Served | L | T | T | TR | L | R |
| Maximum Queue (ft) | 302 | 99 | 200 | 196 | 124 | 77 |
| Average Queue (ft) | 162 | 24 | 78 | 95 | 61 | 39 |
| 95th Queue (ft) | 243 | 69 | 149 | 170 | 109 | 64 |
| Link Distance (ft) | | 419 | 946 | | 1274 | |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | 300 | | | 300 | | 300 |
| Storage Blk Time (%) | 0 | | | | | |
| Queuing Penalty (veh) | 1 | | | | | |

Zone Summary

| |
|------------------------------|
| Zone wide Queuing Penalty: 7 |
|------------------------------|

HCM Signalized Intersection Capacity Analysis
 3: NB 101 Offramp/NB 101 Onramp & Main St.

2030 Build-Out Alt 6
 Timing Plan: PM Peak



| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|-------|-------|------|------|-------|------|-------|-------|------|------|------|------|
| Lane Configurations | | ↖ | | | ↑ | ↗ | | ↖ | ↗ | | | |
| Volume (vph) | 60 | 370 | 0 | 0 | 350 | 440 | 360 | 10 | 150 | 0 | 0 | 0 |
| Ideal Flow (vphpl) | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 | 1900 |
| Lane Width | 12 | 15 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 | 12 |
| Total Lost time (s) | | 4.0 | | | 4.0 | 4.0 | | 4.0 | 4.0 | | | |
| Lane Util. Factor | | 1.00 | | | 1.00 | 1.00 | | 1.00 | 1.00 | | | |
| Frt | | 1.00 | | | 1.00 | 0.85 | | 1.00 | 0.85 | | | |
| Flt Protected | | 0.99 | | | 1.00 | 1.00 | | 0.95 | 1.00 | | | |
| Satd. Flow (prot) | | 1977 | | | 1810 | 1538 | | 1726 | 1538 | | | |
| Flt Permitted | | 0.92 | | | 1.00 | 1.00 | | 0.95 | 1.00 | | | |
| Satd. Flow (perm) | | 1830 | | | 1810 | 1538 | | 1726 | 1538 | | | |
| 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) | 65 | 402 | 0 | 0 | 380 | 478 | 391 | 11 | 163 | 0 | 0 | 0 |
| RTOR Reduction (vph) | 0 | 0 | 0 | 0 | 0 | 292 | 0 | 0 | 115 | 0 | 0 | 0 |
| Lane Group Flow (vph) | 0 | 467 | 0 | 0 | 380 | 186 | 0 | 402 | 48 | 0 | 0 | 0 |
| Turn Type | D.P+P | NA | | | NA | Perm | Split | NA | Perm | | | |
| Protected Phases | 5 | 2 | | | 6 | | 8 | 8 | | | | |
| Permitted Phases | 6 | | | | | 6 | | | 8 | | | |
| Actuated Green, G (s) | | 34.3 | | | 23.3 | 23.3 | | 17.7 | 17.7 | | | |
| Effective Green, g (s) | | 34.3 | | | 23.3 | 23.3 | | 17.7 | 17.7 | | | |
| Actuated g/C Ratio | | 0.57 | | | 0.39 | 0.39 | | 0.29 | 0.29 | | | |
| 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) | | 1063 | | | 702 | 597 | | 509 | 453 | | | |
| v/s Ratio Prot | | c0.05 | | | c0.21 | | | c0.23 | | | | |
| v/s Ratio Perm | | 0.20 | | | | 0.12 | | | 0.03 | | | |
| v/c Ratio | | 0.44 | | | 0.54 | 0.31 | | 0.79 | 0.11 | | | |
| Uniform Delay, d1 | | 7.4 | | | 14.2 | 12.8 | | 19.4 | 15.4 | | | |
| Progression Factor | | 1.00 | | | 0.66 | 1.10 | | 1.00 | 1.00 | | | |
| Incremental Delay, d2 | | 0.3 | | | 2.6 | 1.1 | | 8.0 | 0.1 | | | |
| Delay (s) | | 7.6 | | | 12.0 | 15.2 | | 27.4 | 15.5 | | | |
| Level of Service | | A | | | B | B | | C | B | | | |
| Approach Delay (s) | | 7.6 | | | 13.8 | | | 24.0 | | | 0.0 | |
| Approach LOS | | A | | | B | | | C | | | A | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 15.3 | HCM 2000 Level of Service | B |
| HCM 2000 Volume to Capacity ratio | 0.63 | | |
| Actuated Cycle Length (s) | 60.0 | Sum of lost time (s) | 12.0 |
| Intersection Capacity Utilization | 80.5% | ICU Level of Service | D |
| Analysis Period (min) | 15 | | |

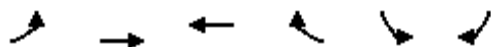
c Critical Lane Group

HCM Signalized Intersection Capacity Analysis

4: Main St. & Ramada Dr.

2030 Build-Out Alt 6

Timing Plan: PM Peak



| Movement | EBL | EBT | WBT | WBR | SBL | SBR |
|------------------------|-------|------|-------|------|-------|------|
| Lane Configurations | | | | | | |
| Volume (vph) | 290 | 230 | 400 | 110 | 210 | 390 |
| 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 | 1.00 | 1.00 | 0.95 | | 1.00 | 1.00 |
| Frt | 1.00 | 1.00 | 0.97 | | 1.00 | 0.85 |
| Flt Protected | 0.95 | 1.00 | 1.00 | | 0.95 | 1.00 |
| Satd. Flow (prot) | 1719 | 1810 | 3327 | | 1719 | 1538 |
| Flt Permitted | 0.95 | 1.00 | 1.00 | | 0.95 | 1.00 |
| Satd. Flow (perm) | 1719 | 1810 | 3327 | | 1719 | 1538 |
| Peak-hour factor, PHF | 0.92 | 0.92 | 0.92 | 0.92 | 0.92 | 0.92 |
| Adj. Flow (vph) | 315 | 250 | 435 | 120 | 228 | 424 |
| RTOR Reduction (vph) | 0 | 0 | 38 | 0 | 0 | 339 |
| Lane Group Flow (vph) | 315 | 250 | 517 | 0 | 228 | 85 |
| Turn Type | Prot | NA | NA | | Prot | Perm |
| Protected Phases | 5 | 2 | 6 | | 4 | |
| Permitted Phases | | | | | | 4 |
| Actuated Green, G (s) | 14.8 | 40.0 | 21.2 | | 12.0 | 12.0 |
| Effective Green, g (s) | 14.8 | 40.0 | 21.2 | | 12.0 | 12.0 |
| Actuated g/C Ratio | 0.25 | 0.67 | 0.35 | | 0.20 | 0.20 |
| 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) | 424 | 1206 | 1175 | | 343 | 307 |
| v/s Ratio Prot | c0.18 | 0.14 | c0.16 | | c0.13 | |
| v/s Ratio Perm | | | | | | 0.06 |
| v/c Ratio | 0.74 | 0.21 | 0.44 | | 0.66 | 0.28 |
| Uniform Delay, d1 | 20.8 | 3.9 | 14.9 | | 22.1 | 20.3 |
| Progression Factor | 1.34 | 0.60 | 1.00 | | 1.00 | 1.00 |
| Incremental Delay, d2 | 6.6 | 0.4 | 1.2 | | 4.8 | 0.5 |
| Delay (s) | 34.5 | 2.7 | 16.1 | | 26.9 | 20.8 |
| Level of Service | C | A | B | | C | C |
| Approach Delay (s) | | 20.4 | 16.1 | | 23.0 | |
| Approach LOS | | C | B | | C | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 20.0 | HCM 2000 Level of Service | B |
| HCM 2000 Volume to Capacity ratio | 0.59 | | |
| Actuated Cycle Length (s) | 60.0 | Sum of lost time (s) | 12.0 |
| Intersection Capacity Utilization | 52.3% | ICU Level of Service | A |
| Analysis Period (min) | 15 | | |
| c Critical Lane Group | | | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|------|------|------|------|------|------|
| Start Time | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 |
| End Time | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 2688 | 2713 | 2771 | 2772 | 2787 | 2747 |
| Vehs Exited | 2684 | 2629 | 2773 | 2706 | 2741 | 2707 |
| Starting Vehs | 73 | 47 | 85 | 55 | 43 | 60 |
| Ending Vehs | 77 | 131 | 83 | 121 | 89 | 97 |
| Denied Entry Before | 3 | 0 | 0 | 1 | 1 | 0 |
| Denied Entry After | 1 | 26 | 2 | 14 | 3 | 9 |
| Travel Distance (mi) | 1066 | 1059 | 1107 | 1087 | 1099 | 1084 |
| Travel Time (hr) | 61.3 | 89.7 | 64.4 | 80.0 | 74.4 | 73.9 |
| Total Delay (hr) | 28.2 | 56.9 | 30.0 | 46.4 | 40.3 | 40.4 |
| Total Stops | 3714 | 4149 | 3863 | 4188 | 4013 | 3983 |
| Fuel Used (gal) | 53.3 | 58.7 | 54.8 | 57.5 | 56.8 | 56.2 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 3:50 |
| End Time | 4:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 4:00 |
| End Time | 5:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|------|------|------|------|------|------|
| Vehs Entered | 2688 | 2713 | 2771 | 2772 | 2787 | 2747 |
| Vehs Exited | 2684 | 2629 | 2773 | 2706 | 2741 | 2707 |
| Starting Vehs | 73 | 47 | 85 | 55 | 43 | 60 |
| Ending Vehs | 77 | 131 | 83 | 121 | 89 | 97 |
| Denied Entry Before | 3 | 0 | 0 | 1 | 1 | 0 |
| Denied Entry After | 1 | 26 | 2 | 14 | 3 | 9 |
| Travel Distance (mi) | 1066 | 1059 | 1107 | 1087 | 1099 | 1084 |
| Travel Time (hr) | 61.3 | 89.7 | 64.4 | 80.0 | 74.4 | 73.9 |
| Total Delay (hr) | 28.2 | 56.9 | 30.0 | 46.4 | 40.3 | 40.4 |
| Total Stops | 3714 | 4149 | 3863 | 4188 | 4013 | 3983 |
| Fuel Used (gal) | 53.3 | 58.7 | 54.8 | 57.5 | 56.8 | 56.2 |

Intersection: 3: NB 101 Offramp/NB 101 Onramp & Main St.

| Movement | EB | WB | WB | NB | NB |
|-----------------------|-----|-----|-----|-----|-----|
| Directions Served | LT | T | R | LT | R |
| Maximum Queue (ft) | 341 | 300 | 222 | 540 | 222 |
| Average Queue (ft) | 123 | 129 | 89 | 246 | 78 |
| 95th Queue (ft) | 258 | 333 | 249 | 625 | 209 |
| Link Distance (ft) | 405 | 419 | 419 | 924 | |
| Upstream Blk Time (%) | 0 | 2 | 1 | 4 | |
| Queuing Penalty (veh) | 1 | 10 | 6 | 0 | |
| Storage Bay Dist (ft) | | | | | 200 |
| Storage Blk Time (%) | | | | 16 | |
| Queuing Penalty (veh) | | | | 25 | |

Intersection: 4: Main St. & Ramada Dr.

| Movement | EB | EB | WB | WB | SB | SB |
|-----------------------|-----|-----|-----|-----|------|-----|
| Directions Served | L | T | T | TR | L | R |
| Maximum Queue (ft) | 276 | 97 | 214 | 208 | 173 | 191 |
| Average Queue (ft) | 145 | 28 | 99 | 84 | 98 | 96 |
| 95th Queue (ft) | 231 | 72 | 196 | 168 | 158 | 159 |
| Link Distance (ft) | | 419 | 946 | | 1274 | |
| Upstream Blk Time (%) | | | | | | |
| Queuing Penalty (veh) | | | | | | |
| Storage Bay Dist (ft) | 300 | | | 300 | | 300 |
| Storage Blk Time (%) | 0 | | 0 | | | |
| Queuing Penalty (veh) | 1 | | 1 | | | |

Zone Summary

Zone wide Queuing Penalty: 43

HCM Signalized Intersection Capacity Analysis
 3: N. 101 Ramps/Ramada Drive & Main Street

2030 Build-Out Alt 7
 Timing Plan: AM Peak



| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|-------|------|------|------|-------|------|-------|-------|------|------|------|------|
| Lane Configurations | | | | | | | | | | | | |
| Volume (vph) | 190 | 140 | 220 | 110 | 200 | 310 | 190 | 90 | 70 | 100 | 50 | 100 |
| 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 | |
| Lane Util. Factor | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | |
| Frt | 1.00 | 0.91 | | 1.00 | 0.91 | | 1.00 | 0.93 | | 1.00 | 0.90 | |
| Flt Protected | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | |
| Satd. Flow (prot) | 1719 | 1644 | | 1719 | 1644 | | 1719 | 1691 | | 1719 | 1628 | |
| Flt Permitted | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | |
| Satd. Flow (perm) | 1719 | 1644 | | 1719 | 1644 | | 1719 | 1691 | | 1719 | 1628 | |
| 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) | 207 | 152 | 239 | 120 | 217 | 337 | 207 | 98 | 76 | 109 | 54 | 109 |
| RTOR Reduction (vph) | 0 | 49 | 0 | 0 | 53 | 0 | 0 | 33 | 0 | 0 | 93 | 0 |
| Lane Group Flow (vph) | 207 | 342 | 0 | 120 | 501 | 0 | 207 | 141 | 0 | 109 | 70 | 0 |
| Turn Type | Prot | NA | | Prot | NA | | Prot | NA | | Prot | NA | |
| Protected Phases | 5 | 2 | | 1 | 6 | | 3 | 8 | | 7 | 4 | |
| Permitted Phases | | | | | | | | | | | | |
| Actuated Green, G (s) | 14.7 | 33.9 | | 9.8 | 29.0 | | 15.4 | 15.9 | | 9.4 | 9.9 | |
| Effective Green, g (s) | 14.7 | 33.9 | | 9.8 | 29.0 | | 15.4 | 15.9 | | 9.4 | 9.9 | |
| Actuated g/C Ratio | 0.17 | 0.40 | | 0.12 | 0.34 | | 0.18 | 0.19 | | 0.11 | 0.12 | |
| Clearance Time (s) | 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 | |
| Lane Grp Cap (vph) | 297 | 655 | | 198 | 560 | | 311 | 316 | | 190 | 189 | |
| v/s Ratio Prot | c0.12 | 0.21 | | 0.07 | c0.30 | | c0.12 | c0.08 | | 0.06 | 0.04 | |
| v/s Ratio Perm | | | | | | | | | | | | |
| v/c Ratio | 0.70 | 0.52 | | 0.61 | 0.89 | | 0.67 | 0.45 | | 0.57 | 0.37 | |
| Uniform Delay, d1 | 33.1 | 19.4 | | 35.8 | 26.5 | | 32.4 | 30.7 | | 35.9 | 34.7 | |
| Progression Factor | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | |
| Incremental Delay, d2 | 7.0 | 3.0 | | 5.2 | 19.3 | | 5.3 | 1.0 | | 4.1 | 1.2 | |
| Delay (s) | 40.0 | 22.4 | | 40.9 | 45.8 | | 37.7 | 31.7 | | 40.0 | 35.9 | |
| Level of Service | D | C | | D | D | | D | C | | D | D | |
| Approach Delay (s) | | 28.5 | | | 45.0 | | | 34.9 | | | 37.6 | |
| Approach LOS | | C | | | D | | | C | | | D | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 36.8 | HCM 2000 Level of Service | D |
| HCM 2000 Volume to Capacity ratio | 0.75 | | |
| Actuated Cycle Length (s) | 85.0 | Sum of lost time (s) | 16.0 |
| Intersection Capacity Utilization | 72.7% | ICU Level of Service | C |
| Analysis Period (min) | 15 | | |
| c Critical Lane Group | | | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|------|------|------|------|------|------|
| Start Time | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 | 6:50 |
| End Time | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 | 8:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 2167 | 2211 | 2204 | 2202 | 2206 | 2198 |
| Vehs Exited | 2148 | 2198 | 2203 | 2219 | 2204 | 2194 |
| Starting Vehs | 32 | 26 | 44 | 52 | 38 | 38 |
| Ending Vehs | 51 | 39 | 45 | 35 | 40 | 40 |
| Denied Entry Before | 1 | 1 | 0 | 0 | 2 | 0 |
| Denied Entry After | 9 | 0 | 11 | 1 | 7 | 5 |
| Travel Distance (mi) | 652 | 664 | 663 | 673 | 658 | 662 |
| Travel Time (hr) | 46.3 | 48.5 | 47.3 | 46.2 | 46.2 | 46.9 |
| Total Delay (hr) | 25.2 | 27.0 | 25.8 | 24.4 | 24.9 | 25.5 |
| Total Stops | 2312 | 2269 | 2293 | 2457 | 2441 | 2353 |
| Fuel Used (gal) | 38.0 | 39.5 | 39.0 | 39.3 | 38.7 | 38.9 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 6:50 |
| End Time | 7:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 7:00 |
| End Time | 8:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|------|------|------|------|------|------|
| Vehs Entered | 2167 | 2211 | 2204 | 2202 | 2206 | 2198 |
| Vehs Exited | 2148 | 2198 | 2203 | 2219 | 2204 | 2194 |
| Starting Vehs | 32 | 26 | 44 | 52 | 38 | 38 |
| Ending Vehs | 51 | 39 | 45 | 35 | 40 | 40 |
| Denied Entry Before | 1 | 1 | 0 | 0 | 2 | 0 |
| Denied Entry After | 9 | 0 | 11 | 1 | 7 | 5 |
| Travel Distance (mi) | 652 | 664 | 663 | 673 | 658 | 662 |
| Travel Time (hr) | 46.3 | 48.5 | 47.3 | 46.2 | 46.2 | 46.9 |
| Total Delay (hr) | 25.2 | 27.0 | 25.8 | 24.4 | 24.9 | 25.5 |
| Total Stops | 2312 | 2269 | 2293 | 2457 | 2441 | 2353 |
| Fuel Used (gal) | 38.0 | 39.5 | 39.0 | 39.3 | 38.7 | 38.9 |

Intersection: 3: N. 101 Ramps/Ramada Drive & Main Street

| Movement | EB | EB | WB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|-----|-----|
| Directions Served | L | TR | L | TR | L | TR | L | TR |
| Maximum Queue (ft) | 149 | 374 | 250 | 453 | 236 | 229 | 133 | 188 |
| Average Queue (ft) | 122 | 166 | 136 | 301 | 119 | 88 | 65 | 78 |
| 95th Queue (ft) | 167 | 311 | 279 | 508 | 195 | 176 | 115 | 149 |
| Link Distance (ft) | | 512 | | 405 | | 573 | | 632 |
| Upstream Blk Time (%) | | 0 | | 17 | | | | |
| Queuing Penalty (veh) | | 1 | | 0 | | | | |
| Storage Bay Dist (ft) | 100 | | 200 | | 200 | | 200 | |
| Storage Blk Time (%) | 22 | 14 | 0 | 33 | 1 | 0 | | 0 |
| Queuing Penalty (veh) | 88 | 30 | 1 | 40 | 2 | 0 | | 0 |

Zone Summary

Zone wide Queuing Penalty: 161

HCM Signalized Intersection Capacity Analysis
 3: N. 101 Ramps/Ramada Drive & Main Street

2030 Build-Out Alt 7

1/16/2015



| Movement | EBL | EBT | EBR | WBL | WBT | WBR | NBL | NBT | NBR | SBL | SBT | SBR |
|------------------------|------|------|------|-------|-------|------|-------|------|------|------|-------|------|
| Lane Configurations | | | | | | | | | | | | |
| Volume (vph) | 205 | 165 | 60 | 220 | 180 | 110 | 360 | 85 | 75 | 210 | 220 | 170 |
| 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 | |
| Lane Util. Factor | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | |
| Frt | 1.00 | 0.96 | | 1.00 | 0.94 | | 1.00 | 0.93 | | 1.00 | 0.93 | |
| Flt Protected | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | |
| Satd. Flow (prot) | 1719 | 1737 | | 1719 | 1706 | | 1719 | 1682 | | 1719 | 1691 | |
| Flt Permitted | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | | 0.95 | 1.00 | |
| Satd. Flow (perm) | 1719 | 1737 | | 1719 | 1706 | | 1719 | 1682 | | 1719 | 1691 | |
| 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) | 223 | 179 | 65 | 239 | 196 | 120 | 391 | 92 | 82 | 228 | 239 | 185 |
| RTOR Reduction (vph) | 0 | 15 | 0 | 0 | 25 | 0 | 0 | 35 | 0 | 0 | 32 | 0 |
| Lane Group Flow (vph) | 223 | 229 | 0 | 239 | 291 | 0 | 391 | 139 | 0 | 228 | 392 | 0 |
| Turn Type | Prot | NA | | Prot | NA | | Prot | NA | | Prot | NA | |
| Protected Phases | 5 | 2 | | 1 | 6 | | 3 | 8 | | 7 | 4 | |
| Permitted Phases | | | | | | | | | | | | |
| Actuated Green, G (s) | 14.5 | 17.2 | | 14.8 | 17.5 | | 20.7 | 20.9 | | 16.1 | 16.3 | |
| Effective Green, g (s) | 14.5 | 17.2 | | 14.8 | 17.5 | | 20.7 | 20.9 | | 16.1 | 16.3 | |
| Actuated g/C Ratio | 0.17 | 0.20 | | 0.17 | 0.21 | | 0.24 | 0.25 | | 0.19 | 0.19 | |
| Clearance Time (s) | 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 | |
| Lane Grp Cap (vph) | 293 | 351 | | 299 | 351 | | 418 | 413 | | 325 | 324 | |
| v/s Ratio Prot | 0.13 | 0.13 | | c0.14 | c0.17 | | c0.23 | 0.08 | | 0.13 | c0.23 | |
| v/s Ratio Perm | | | | | | | | | | | | |
| v/c Ratio | 0.76 | 0.65 | | 0.80 | 0.83 | | 0.94 | 0.34 | | 0.70 | 1.21 | |
| Uniform Delay, d1 | 33.6 | 31.1 | | 33.7 | 32.3 | | 31.5 | 26.3 | | 32.2 | 34.4 | |
| Progression Factor | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | | 1.00 | 1.00 | |
| Incremental Delay, d2 | 11.1 | 9.1 | | 13.8 | 19.7 | | 28.1 | 0.5 | | 6.7 | 119.3 | |
| Delay (s) | 44.7 | 40.2 | | 47.5 | 52.0 | | 59.6 | 26.8 | | 38.9 | 153.7 | |
| Level of Service | D | D | | D | D | | E | C | | D | F | |
| Approach Delay (s) | | 42.4 | | | 50.1 | | | 49.5 | | | 113.5 | |
| Approach LOS | | D | | | D | | | D | | | F | |

Intersection Summary

| | | | |
|-----------------------------------|-------|---------------------------|------|
| HCM 2000 Control Delay | 66.8 | HCM 2000 Level of Service | E |
| HCM 2000 Volume to Capacity ratio | 0.95 | | |
| Actuated Cycle Length (s) | 85.0 | Sum of lost time (s) | 16.0 |
| Intersection Capacity Utilization | 82.8% | ICU Level of Service | E |
| Analysis Period (min) | 15 | | |
| c Critical Lane Group | | | |

Summary of All Intervals

| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|-------------------------|-------|-------|------|------|-------|-------|
| Start Time | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 | 3:50 |
| End Time | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 | 5:00 |
| Total Time (min) | 70 | 70 | 70 | 70 | 70 | 70 |
| Time Recorded (min) | 60 | 60 | 60 | 60 | 60 | 60 |
| # of Intervals | 2 | 2 | 2 | 2 | 2 | 2 |
| # of Recorded Intervals | 1 | 1 | 1 | 1 | 1 | 1 |
| Vehs Entered | 2709 | 2681 | 2654 | 2677 | 2675 | 2678 |
| Vehs Exited | 2690 | 2636 | 2637 | 2657 | 2638 | 2651 |
| Starting Vehs | 76 | 79 | 71 | 62 | 54 | 68 |
| Ending Vehs | 95 | 124 | 88 | 82 | 91 | 97 |
| Denied Entry Before | 2 | 1 | 0 | 2 | 0 | 0 |
| Denied Entry After | 79 | 120 | 47 | 60 | 157 | 91 |
| Travel Distance (mi) | 808 | 796 | 798 | 795 | 799 | 799 |
| Travel Time (hr) | 140.7 | 151.3 | 75.6 | 99.1 | 156.4 | 124.6 |
| Total Delay (hr) | 115.6 | 126.6 | 50.9 | 74.4 | 131.5 | 99.8 |
| Total Stops | 4561 | 4375 | 3610 | 4033 | 4412 | 4198 |
| Fuel Used (gal) | 65.0 | 66.8 | 50.0 | 55.4 | 68.3 | 61.1 |

Interval #0 Information Seeding

| | |
|-------------------------------------|------|
| Start Time | 3:50 |
| End Time | 4:00 |
| Total Time (min) | 10 |
| Volumes adjusted by Growth Factors. | |
| No data recorded this interval. | |

Interval #1 Information Recording

| | |
|--|------|
| Start Time | 4:00 |
| End Time | 5:00 |
| Total Time (min) | 60 |
| Volumes adjusted by PHF, Growth Factors. | |

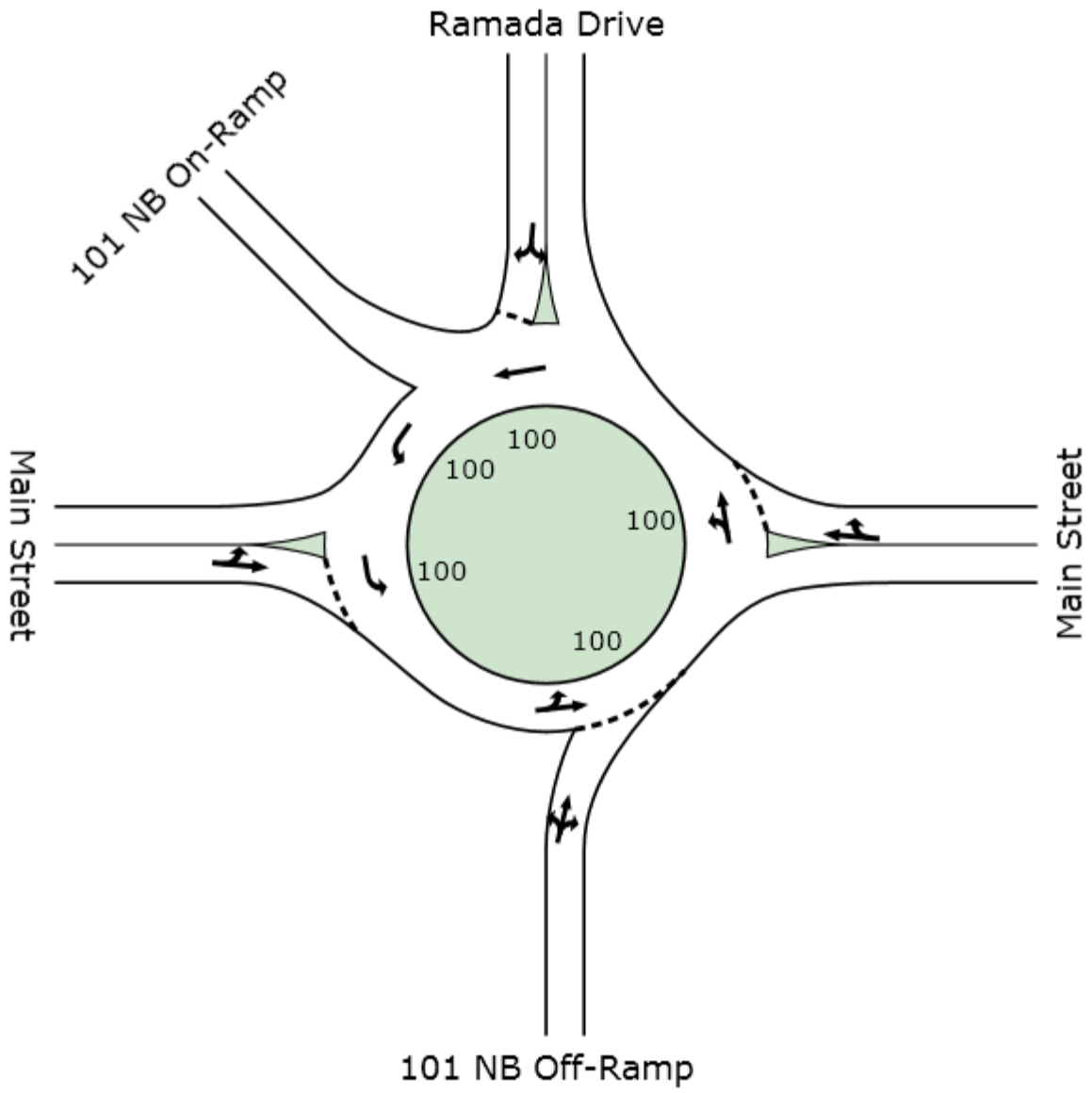
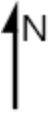
| Run Number | 1 | 2 | 3 | 4 | 5 | Avg |
|----------------------|-------|-------|------|------|-------|-------|
| Vehs Entered | 2709 | 2681 | 2654 | 2677 | 2675 | 2678 |
| Vehs Exited | 2690 | 2636 | 2637 | 2657 | 2638 | 2651 |
| Starting Vehs | 76 | 79 | 71 | 62 | 54 | 68 |
| Ending Vehs | 95 | 124 | 88 | 82 | 91 | 97 |
| Denied Entry Before | 2 | 1 | 0 | 2 | 0 | 0 |
| Denied Entry After | 79 | 120 | 47 | 60 | 157 | 91 |
| Travel Distance (mi) | 808 | 796 | 798 | 795 | 799 | 799 |
| Travel Time (hr) | 140.7 | 151.3 | 75.6 | 99.1 | 156.4 | 124.6 |
| Total Delay (hr) | 115.6 | 126.6 | 50.9 | 74.4 | 131.5 | 99.8 |
| Total Stops | 4561 | 4375 | 3610 | 4033 | 4412 | 4198 |
| Fuel Used (gal) | 65.0 | 66.8 | 50.0 | 55.4 | 68.3 | 61.1 |

Intersection: 3: N. 101 Ramps/Ramada Drive & Main Street

| Movement | EB | EB | WB | WB | NB | NB | SB | SB |
|-----------------------|-----|-----|-----|-----|-----|-----|-----|-----|
| Directions Served | L | TR | L | TR | L | TR | L | TR |
| Maximum Queue (ft) | 150 | 305 | 250 | 433 | 250 | 554 | 250 | 686 |
| Average Queue (ft) | 127 | 165 | 165 | 203 | 224 | 367 | 232 | 581 |
| 95th Queue (ft) | 173 | 278 | 267 | 369 | 290 | 757 | 311 | 806 |
| Link Distance (ft) | | 512 | | 405 | | 573 | | 632 |
| Upstream Blk Time (%) | | | | 3 | | 33 | | 53 |
| Queuing Penalty (veh) | | | | 0 | | 0 | | 0 |
| Storage Bay Dist (ft) | 100 | | 200 | | 200 | | 200 | |
| Storage Blk Time (%) | 26 | 23 | 6 | 11 | 46 | 1 | 3 | 71 |
| Queuing Penalty (veh) | 63 | 51 | 19 | 26 | 81 | 3 | 11 | 162 |

Zone Summary

Zone wide Queuing Penalty: 416



MOVEMENT SUMMARY

Site: 101 NB Ramps_AM

Highway 101/Main Street Interchange Analysis
 2030 Build-Out Conditions with Alternative 5
 AM Peak Hour
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|---------------------------------|------|----------------------|---------|------------------|----------------------|------------------|--------------------------------------|----------------|--------------|--------------------------------|----------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| South: 101 NB Off-Ramp | | | | | | | | | | | |
| 3 | L | 207 | 5.0 | 0.577 | 15.5 | LOS C | 3.5 | 91.8 | 0.79 | 1.05 | 24.4 |
| 8 | T | 135 | 5.0 | 0.577 | 15.5 | LOS C | 3.5 | 91.8 | 0.79 | 0.97 | 26.1 |
| 18 | R | 39 | 5.0 | 0.577 | 15.5 | LOS C | 3.5 | 91.8 | 0.79 | 0.99 | 25.9 |
| Approach | | 380 | 5.0 | 0.577 | 15.5 | LOS C | 3.5 | 91.8 | 0.79 | 1.02 | 25.1 |
| East: Main Street | | | | | | | | | | | |
| 6 | T | 302 | 5.0 | 1.068 | 80.7 | LOS F | 29.7 | 771.0 | 1.00 | 2.18 | 12.0 |
| 16 | R | 372 | 5.0 | 1.068 | 80.7 | LOS F | 29.7 | 771.0 | 1.00 | 2.18 | 12.0 |
| Approach | | 674 | 5.0 | 1.068 | 80.7 | LOS F | 29.7 | 771.0 | 1.00 | 2.18 | 12.0 |
| North: Ramada Drive | | | | | | | | | | | |
| 7 | L | 109 | 5.0 | 0.436 | 12.3 | LOS B | 2.1 | 55.9 | 0.73 | 0.99 | 25.6 |
| 14 | R | 163 | 5.0 | 0.436 | 12.3 | LOS B | 2.1 | 55.9 | 0.73 | 0.94 | 27.2 |
| Approach | | 272 | 5.0 | 0.436 | 12.3 | LOS B | 2.1 | 55.9 | 0.73 | 0.96 | 26.5 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 409 | 5.0 | 0.488 | 8.1 | LOS A | 3.3 | 86.0 | 0.40 | 0.73 | 27.2 |
| 2 | T | 189 | 5.0 | 0.488 | 8.1 | LOS A | 3.3 | 86.0 | 0.40 | 0.49 | 30.4 |
| Approach | | 598 | 5.0 | 0.488 | 8.1 | LOS A | 3.3 | 86.0 | 0.40 | 0.65 | 28.1 |
| All Vehicles | | 1924 | 5.0 | 1.068 | 35.6 | LOS E | 29.7 | 771.0 | 0.73 | 1.30 | 18.9 |

Level of Service (LOS) Method: Delay (HCM 2000).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay per movement

Intersection and Approach LOS values are based on average delay for all vehicle movements.

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

Processed: Thursday, April 19, 2012 3:32:44 PM

SIDRA INTERSECTION 5.1.11.2079

Project: T:\16128\Traffic\Synchro\Deliverable3_Alternatives\Alt_5_DoubleRoundabout\2030

\101_NB_Ramps_Main_Ramada.sip

8001416, RICK ENGINEERING COMPANY, SINGLE

Copyright © 2000-2011 Akcelik and Associates Pty Ltd

www.sidrasolutions.com

SIDRA
INTERSECTION

MOVEMENT SUMMARY

Site: 101 NB Ramps_PM

Highway 101/Main Street Interchange Analysis
 2030 Build-Out Volumes with Alternative 5
 PM Peak Hour
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|---------------------------------|------|----------------------|---------|------------------|----------------------|------------------|--------------------------------------|----------------|--------------|--------------------------------|----------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| South: 101 NB Off-Ramp | | | | | | | | | | | |
| 3 | L | 402 | 4.9 | 0.812 | 27.6 | LOS D | 8.2 | 213.0 | 0.93 | 1.22 | 20.4 |
| 8 | T | 128 | 5.0 | 0.812 | 27.6 | LOS D | 8.2 | 213.0 | 0.93 | 1.20 | 21.2 |
| 18 | R | 35 | 5.0 | 0.812 | 27.6 | LOS D | 8.2 | 213.0 | 0.93 | 1.20 | 21.1 |
| Approach | | 565 | 4.9 | 0.812 | 27.6 | LOS D | 8.2 | 213.0 | 0.93 | 1.22 | 20.6 |
| East: Main Street | | | | | | | | | | | |
| 6 | T | 304 | 5.0 | 0.908 | 43.1 | LOS E | 11.4 | 297.2 | 0.97 | 1.42 | 17.6 |
| 16 | R | 250 | 5.0 | 0.908 | 43.1 | LOS E | 11.4 | 297.2 | 0.97 | 1.42 | 17.5 |
| Approach | | 554 | 5.0 | 0.908 | 43.1 | LOS E | 11.4 | 297.2 | 0.97 | 1.42 | 17.5 |
| North: Ramada Drive | | | | | | | | | | | |
| 7 | L | 228 | 5.0 | 1.210 | 135.4 | LOS F | 46.9 | 1219.2 | 1.00 | 2.97 | 8.4 |
| 14 | R | 424 | 5.0 | 1.210 | 135.4 | LOS F | 46.9 | 1219.2 | 1.00 | 2.97 | 8.1 |
| Approach | | 652 | 5.0 | 1.210 | 135.4 | LOS F | 46.9 | 1219.2 | 1.00 | 2.97 | 8.2 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 252 | 5.0 | 0.414 | 7.5 | LOS A | 2.4 | 62.5 | 0.47 | 0.78 | 27.8 |
| 2 | T | 215 | 5.0 | 0.414 | 7.5 | LOS A | 2.4 | 62.5 | 0.47 | 0.57 | 30.8 |
| Approach | | 467 | 5.0 | 0.414 | 7.5 | LOS A | 2.4 | 62.5 | 0.47 | 0.68 | 29.1 |
| All Vehicles | | 2239 | 5.0 | 1.210 | 58.6 | LOS F | 46.9 | 1219.2 | 0.86 | 1.67 | 14.5 |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

Processed: Thursday, April 19, 2012 3:32:44 PM

SIDRA INTERSECTION 5.1.11.2079

Project: T:\16128\Traffic\Synchro\Deliverable3_Alternatives\Alt_5_DoubleRoundabout\2030

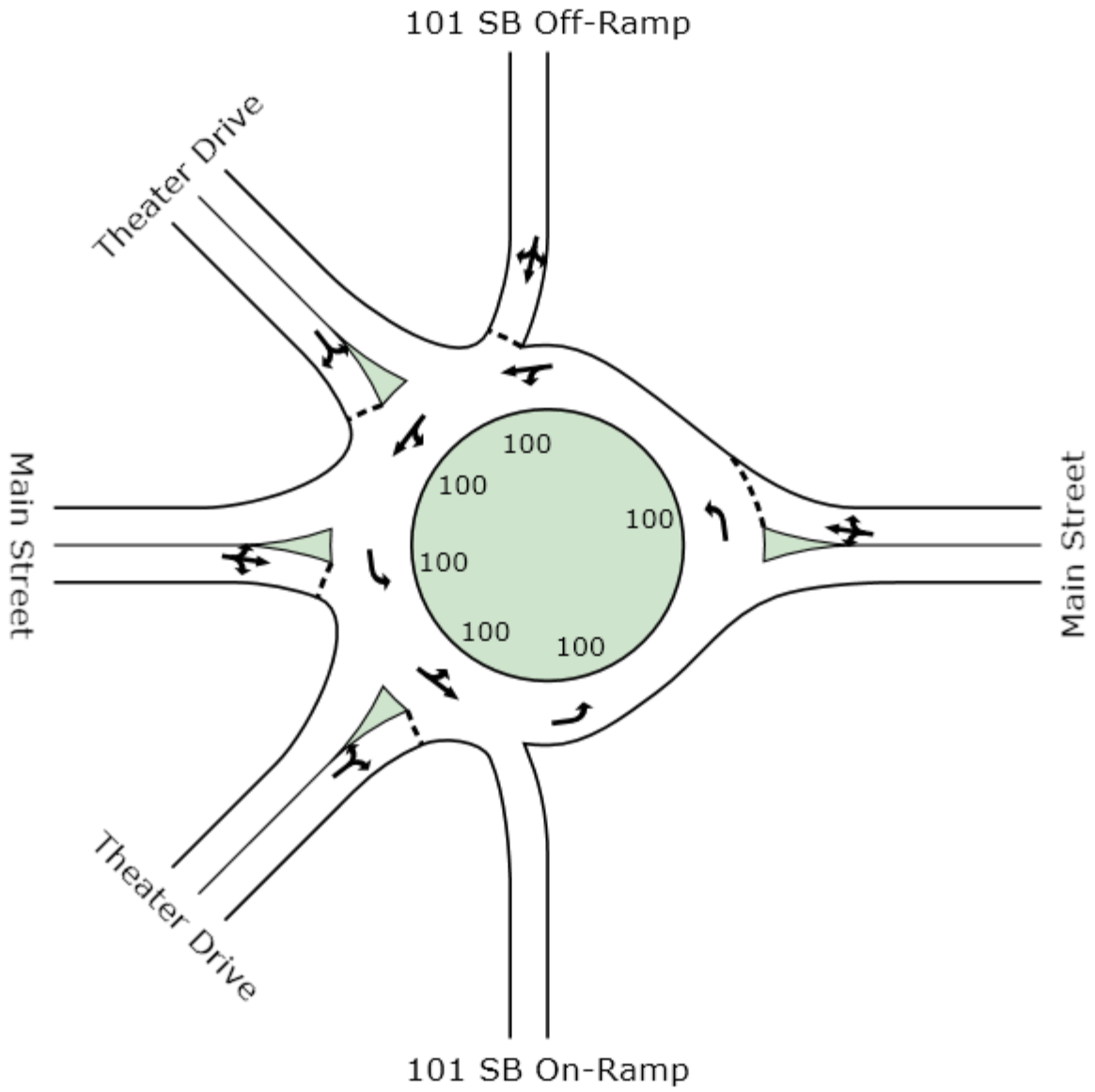
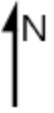
\101_NB_Ramps_Main_Ramada.sip

8001416, RICK ENGINEERING COMPANY, SINGLE

Copyright © 2000-2011 Akcelik and Associates Pty Ltd

www.sidrasolutions.com

SIDRA
INTERSECTION



MOVEMENT SUMMARY

Site: 101 SB Ramps_AM Peak

Highway 101/Main Street Interchange Analysis
 2030 Build-Out Conditions with Alternative 5
 AM Peak
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|---------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| East: Main Street | | | | | | | | | | | |
| 1 | L | 141 | 5.0 | 0.397 | 6.4 | LOS A | 2.5 | 65.9 | 0.15 | 0.83 | 28.3 |
| 6 | T | 33 | 5.0 | 0.397 | 6.4 | LOS A | 2.5 | 65.9 | 0.15 | 0.45 | 31.9 |
| 16 | R | 359 | 5.0 | 0.397 | 6.4 | LOS A | 2.5 | 65.9 | 0.15 | 0.45 | 31.9 |
| Approach | | 533 | 5.0 | 0.397 | 6.4 | LOS A | 2.5 | 65.9 | 0.15 | 0.55 | 30.8 |
| North: 101 SB Off-Ramp | | | | | | | | | | | |
| 7 | L | 337 | 5.0 | 0.494 | 11.6 | LOS B | 2.9 | 74.1 | 0.72 | 0.98 | 25.8 |
| 4 | T | 1 | 3.0 | 0.494 | 11.6 | LOS B | 2.9 | 74.1 | 0.72 | 0.88 | 27.8 |
| 14 | R | 43 | 5.0 | 0.494 | 11.6 | LOS B | 2.9 | 74.1 | 0.72 | 0.91 | 27.6 |
| Approach | | 382 | 5.0 | 0.494 | 11.6 | LOS B | 2.9 | 74.1 | 0.72 | 0.97 | 26.0 |
| North West: Theater Drive | | | | | | | | | | | |
| 7X | L | 245 | 5.0 | 0.578 | 13.5 | LOS B | 3.9 | 100.8 | 0.76 | 1.03 | 25.6 |
| 14X | R | 212 | 5.0 | 0.578 | 13.5 | LOS B | 3.9 | 100.8 | 0.76 | 0.94 | 27.1 |
| Approach | | 457 | 5.0 | 0.578 | 13.5 | LOS B | 3.9 | 100.8 | 0.76 | 0.99 | 26.2 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 11 | 5.0 | 0.104 | 8.2 | LOS A | 0.4 | 9.8 | 0.66 | 0.97 | 27.2 |
| 2 | T | 8 | 5.0 | 0.104 | 8.2 | LOS A | 0.4 | 9.8 | 0.66 | 0.82 | 30.3 |
| 12 | R | 36 | 5.0 | 0.104 | 8.2 | LOS A | 0.4 | 9.8 | 0.66 | 0.85 | 30.0 |
| Approach | | 54 | 5.0 | 0.104 | 8.2 | LOS A | 0.4 | 9.8 | 0.66 | 0.87 | 29.4 |
| South West: Theater Drive | | | | | | | | | | | |
| 5X | L | 11 | 5.0 | 0.066 | 8.1 | LOS A | 0.2 | 6.0 | 0.66 | 0.94 | 27.7 |
| 12X | R | 22 | 5.0 | 0.066 | 8.1 | LOS A | 0.2 | 6.0 | 0.66 | 0.84 | 29.9 |
| Approach | | 33 | 5.0 | 0.066 | 8.1 | LOS A | 0.2 | 6.0 | 0.66 | 0.88 | 29.1 |
| All Vehicles | | 1458 | 5.0 | 0.578 | 10.1 | LOS B | 3.9 | 100.8 | 0.52 | 0.82 | 27.8 |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

MOVEMENT SUMMARY

Site: 101 SB Ramps_PM Peak

Highway 101/Main Street Interchange Analysis
 2030 Build-Out Conditions with Alternative 5
 PM Peak
 Roundabout

| Movement Performance - Vehicles | | | | | | | | | | | |
|----------------------------------|------|-------------------|------|---------------|-------------------|------------------|--------------------------------|-------------|--------------|-----------------------------|-------------------|
| Mov ID | Turn | Demand Flow veh/h | HV % | Deg. Satn v/c | Average Delay sec | Level of Service | 95% Back of Queue Vehicles veh | Distance ft | Prop. Queued | Effective Stop Rate per veh | Average Speed mph |
| East: Main Street | | | | | | | | | | | |
| 1 | L | 250 | 5.0 | 0.576 | 9.1 | LOS A | 5.0 | 129.6 | 0.20 | 0.79 | 27.0 |
| 6 | T | 38 | 5.0 | 0.576 | 9.1 | LOS A | 5.0 | 129.6 | 0.20 | 0.44 | 30.1 |
| 16 | R | 484 | 5.0 | 0.576 | 9.1 | LOS A | 5.0 | 129.6 | 0.20 | 0.44 | 30.1 |
| Approach | | 772 | 5.0 | 0.576 | 9.1 | LOS A | 5.0 | 129.6 | 0.20 | 0.55 | 29.0 |
| North: 101 SB Off-Ramp | | | | | | | | | | | |
| 7 | L | 228 | 5.0 | 0.542 | 15.5 | LOS C | 3.0 | 79.0 | 0.78 | 1.03 | 24.2 |
| 4 | T | 1 | 3.0 | 0.542 | 15.5 | LOS C | 3.0 | 79.0 | 0.78 | 0.96 | 25.9 |
| 14 | R | 98 | 5.0 | 0.542 | 15.5 | LOS C | 3.0 | 79.0 | 0.78 | 0.99 | 25.6 |
| Approach | | 327 | 5.0 | 0.542 | 15.5 | LOS C | 3.0 | 79.0 | 0.78 | 1.02 | 24.6 |
| North West: Theater Drive | | | | | | | | | | | |
| 7X | L | 213 | 5.0 | 0.757 | 21.1 | LOS C | 7.3 | 189.5 | 0.89 | 1.15 | 22.8 |
| 14X | R | 385 | 5.0 | 0.757 | 21.1 | LOS C | 7.3 | 189.5 | 0.89 | 1.10 | 23.8 |
| Approach | | 598 | 5.0 | 0.757 | 21.1 | LOS C | 7.3 | 189.5 | 0.89 | 1.12 | 23.4 |
| West: Main Street | | | | | | | | | | | |
| 5 | L | 11 | 5.0 | 0.145 | 10.1 | LOS B | 0.5 | 13.4 | 0.70 | 0.98 | 26.5 |
| 2 | T | 17 | 5.0 | 0.145 | 10.1 | LOS B | 0.5 | 13.4 | 0.70 | 0.84 | 29.2 |
| 12 | R | 37 | 5.0 | 0.145 | 10.1 | LOS B | 0.5 | 13.4 | 0.70 | 0.87 | 28.9 |
| Approach | | 65 | 5.0 | 0.145 | 10.1 | LOS B | 0.5 | 13.4 | 0.70 | 0.88 | 28.5 |
| South West: Theater Drive | | | | | | | | | | | |
| 5X | L | 11 | 5.0 | 0.077 | 9.6 | LOS A | 0.3 | 6.9 | 0.71 | 0.95 | 26.9 |
| 12X | R | 22 | 5.0 | 0.077 | 9.6 | LOS A | 0.3 | 6.9 | 0.71 | 0.87 | 29.0 |
| Approach | | 33 | 5.0 | 0.077 | 9.6 | LOS A | 0.3 | 6.9 | 0.71 | 0.89 | 28.3 |
| All Vehicles | | 1795 | 5.0 | 0.757 | 14.3 | LOS B | 7.3 | 189.5 | 0.56 | 0.84 | 26.0 |

Level of Service (LOS) Method: Delay & v/c (HCM 2010).

Roundabout LOS Method: Same as Sign Control.

Vehicle movement LOS values are based on average delay and v/c ratio (degree of saturation) per movement

LOS F will result if v/c > 1 irrespective of movement delay value (does not apply for approaches and intersection).

Intersection and Approach LOS values are based on average delay for all movements (v/c not used as specified in HCM 2010).

Roundabout Capacity Model: US HCM 2010.

HCM Delay Model used. Geometric Delay not included.

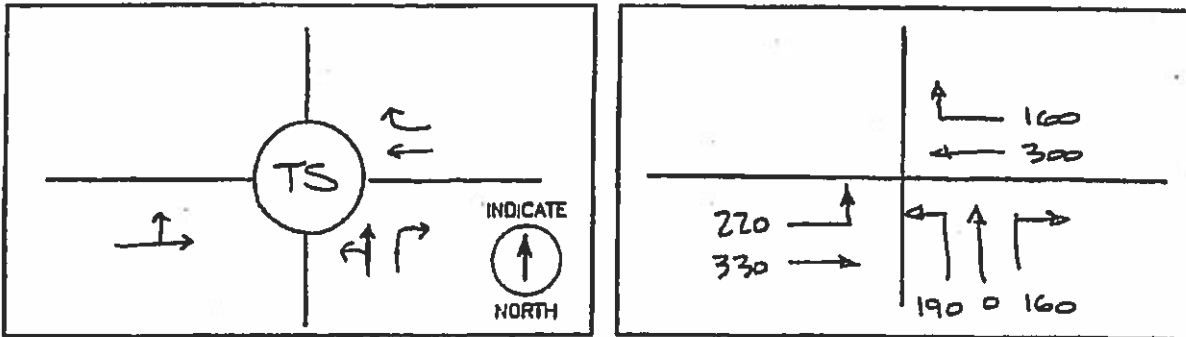
APPENDIX E
BUILDOUT ILV ANALYSIS WORKSHEETS

INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN S / 101 NB RAMP DIST. CO. RTE P.M. 05-SLO-101-52.5
 (ALT. 6 - BUILDOUT AM PEAK-HR) BY A.E. DATE 1/8/15
 TIME AM/PM

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|----------------|--------------------|---------|
| ↑ → | ↶ 160 ← 300 | ↶ ↑ ↷ 190 0 160 | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 550 | 300 | 190 | |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 1040 |

IS ... < 1200 ILV/HR.

> 1200 BUT < 1500 ILV/HR.

> 1500 ILV/HR (CAPACITY)

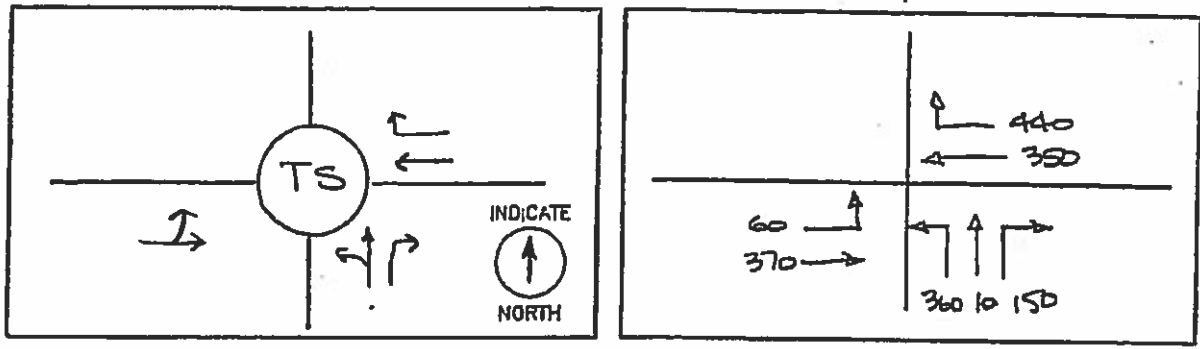
REMARKS: STABLE FLOW.

INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN 5 / 101 NB RAMP DIST. CO. RTE PM. 05-50-101-52.5
 (ALT. 6 - BUILDOUT PM PEAK HOUR) BY A.E. DATE 1/8/15
 TIME _____ AM/PM

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| | | | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 430 | 440 | 370 | |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 1240 |

IS ... < 1200 ILV/HR.
 > 1200 BUT < 1500 ILV/HR.
 > 1500 ILV/HR (CAPACITY)

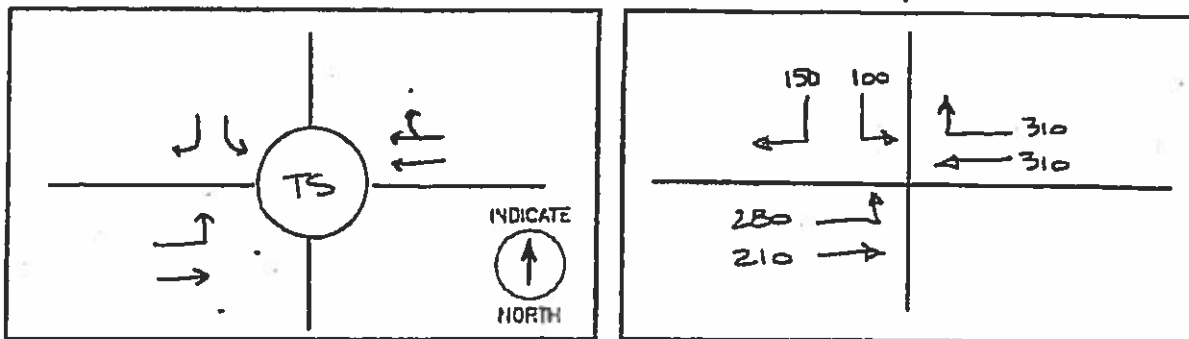
REMARKS: APPROACHING INTERSECTION CAPACITY

INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN ST / RAMADA DR. DIST. CO. RTE. PM. 05-SLO-101-57.5
 (ALT 6 - BUILDOUT AM PEAK HOUR) BY A.E. DATE 4/8/15
 TIME AM/PM

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| | | | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 280 | 465 | 150 | |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 895 |

- IS . . .
- < 1200 ILV/HR.
 - > 1200 BUT < 1500 ILV/HR.
 - > 1500 ILV/HR (CAPACITY)

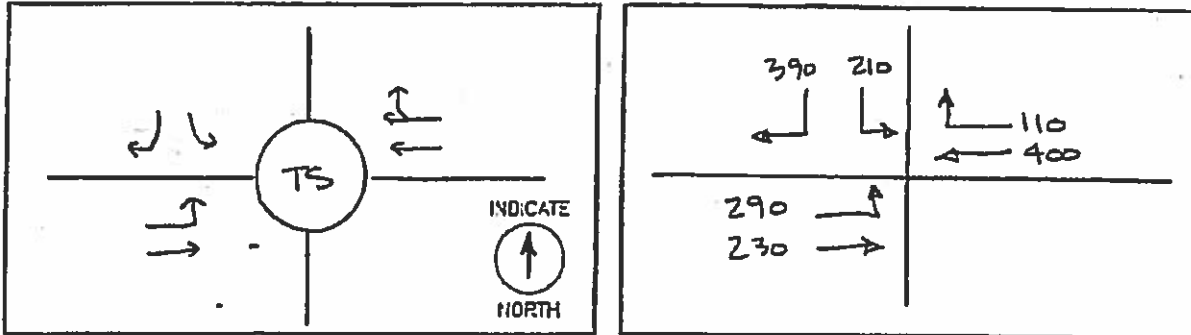
REMARKS: SIGNAL NECESSARY.

INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN S / RAMADA DR. DIST. CO. RTE P.M. 05-510-101-57.5
 (ALT 6 - BUILDOUT PM PEAK HOUR) BY A.E. DATE 1/8/15
 TIME _____ AM/PM (PM)

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| | | | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 290 | 310 | 390 | |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 990 |

IS . . . < 1200 ILV/HR.
 > 1200 BUT < 1500 ILV/HR.
 > 1500 ILV/HR (CAPACITY)

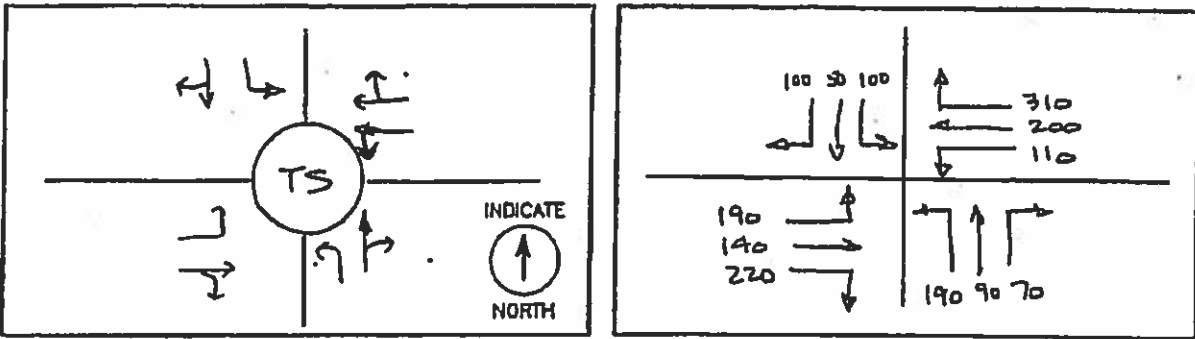
REMARKS: STABLE FLOW

INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN ST/101 NB RAMPS/RAMP DIST. CO. RTE. PM. 05-SLO-101-52.5
 (ALT 7 - BUILDOUT AM PEAK HR) BY A.E. DATE 1/8/15
 TIME AM/PM

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| | | | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 190 | 510 | 190 | 160 |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 1050 |

- IS . . .
- < 1200 ILV/HR.
 - > 1200 BUT < 1500 ILV/HR.
 - > 1500 ILV/HR (CAPACITY)

REMARKS: STABLE FLOW.

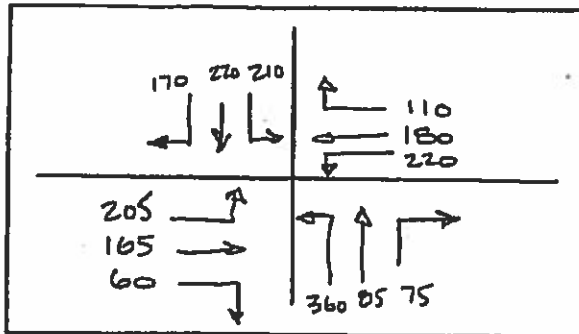
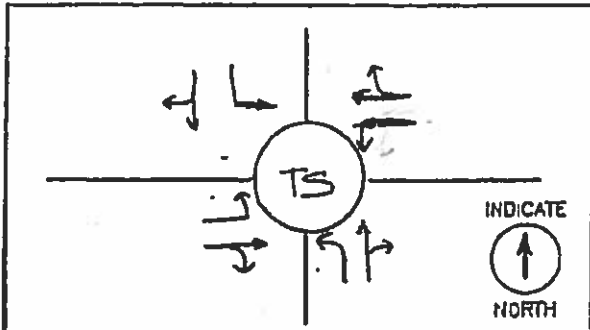
INTERSECTION

Signalized Intersection CAPACITY ANALYSIS

INTERSECTION MAIN ST / NB DL RAMP / CAMADA DIST. CO. RTE P.M. 05-50-101-57.5
 (ALT 7 - BUILDOUT PM PEAK HOUR) BY A.E. DATE 1/8/15

TIME AM/PM

DIAGRAM AND TRAFFIC FLOWS:



LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| | | | |

CRITICAL LANE VOLUMES (ILV/HR)

| PHASE 1 | PHASE 2 | PHASE 3 | PHASE 4 |
|---------|---------|---------|---------|
| 220 | 290 | 360 | 390 |

TOTAL OPERATING LEVEL (ILV/HR)

| |
|----------|
| Σ |
| 1260 |

- IS . . .
- < 1200 ILV/HR.
 - > 1200 BUT < 1500 ILV/HR.
 - > 1500 ILV/HR (CAPACITY)

REMARKS: APPROACHING INTERSECTION CAPACITY