# TECHNICAL MEMORANDUM – ALTERNATIVE 1-5 TECHNICAL ANALYSIS

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**Date:** October 9, 2012

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**Pages:** 74 (Not including appendices)
**Project No.:** WBS 300150

**Jurisdiction:** County of San Luis Obispo

**Subject:** Summary of traffic conditions for five 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.) Interchange Alternatives Impacts and Mitigations
9.) Additional Interchange Alternatives

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## 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.

Rick Engineering Company (RICK) has prepared this technical memorandum analyzing the existing and buildout traffic conditions at the US 101 / Main Street Interchange for five (5) improvement alternatives. These alternatives include:

Alternative 1: Widen Bridge to Accommodate Left Turns
Alternative 2: Modified Diamond Interchange
Alternative 3: Fully Compliant Interchange
Alternative 4: Double Hook Ramps
Alternative 5: Double Roundabout

Deliverable 1 evaluated the existing traffic conditions at the interchange (dated July 1, 2011). The second technical memorandum (dated July 1, 2011) evaluated traffic conditions under buildout conditions within the area, with no changes to the existing roadway infrastructure or geometrical layout.

This third memorandum develops and evaluates the peak hour traffic volumes and lane geometrics under existing and buildout conditions for each improvement alternative. The evaluation of traffic conditions included an analysis of Levels of Service (LOS) and vehicle queues at the four (4) 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 each alternative indicated that the following three study intersections do not satisfy the Caltrans LOS threshold criteria, although these intersections do not degrade relative to the existing interchange traffic conditions:

- Main Street / US 101 SB Ramps under Alternative 1, 2 and 3
- Main Street / US 101 NB Ramps under Alternative 2 and 3
- Theatre Drive / US 101 SB Ramps under Alternative 4

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 intersection under each alternative would operate with acceptable overall LOS, satisfying the Caltrans and County's LOS threshold criteria. The only exception was the roundabout at the US 101 northbound ramps/Main Street/Ramada Drive under Alternative 5, which is projected to operate with delays in the range of LOS E-F, thus failing to satisfy the Caltrans and County's threshold criteria. Under each alternative, a few individual movements at the study intersections would experience delays in the range of LOS D-F.

Contrary to the LOS analyses as described above, the 95$^{th}$ percentile queues exceeded the assumed or estimated storage at many individual traffic lanes for all alternatives. The full peak hour demand was not being served due to moderate or severe congestion at downstream locations. Under Alternatives 1 through 4, the 95$^{th}$ percentile queues on the US 101 northbound off-ramp approach are estimated to backup and extend onto the freeway mainline from 22% to 42% of time during the AM or PM peak hours. Under Alternative 1 through 3, the 95$^{th}$ percentile queues are frequently anticipated to spillback into the upstream intersections along Main Street. Generally, excessive queues at the following locations persisted throughout the AM or PM peak hour conditions under all alternatives. Downstream congestion and resultant insufficient discharge capacities were primary reasons for deficient queues at these locations.

- US 101 northbound off-ramp approach
- Westbound Main Street approach at Ramada Drive
- Southbound Ramada Drive at Main Street

RICK investigated and recommended capacity improvements that are likely to create acceptable queuing conditions within the study area. The following recommended improvements were solely based on a traffic operations perspective:

<u>Alternative 1</u>

1. Add a dedicated right-turn lane on the US 101 northbound off-ramp approach at Main Street.
2. Add a dedicated right-turn lane on the southbound Ramada Drive approach at Main Street.
3. Add a shared through/right-turn lane on the westbound Main Street approach at Ramada Drive.

<u>Alternative 2</u>

1. Add a dedicated right-turn lane on the westbound Main Street approach at the US 101 northbound ramps. The recommended right-turn lane should be extended for length of the Main Street segment from the US 101 northbound ramps to Ramada Drive, creating a four-lane cross section.
2. Add a shared through/right-turn lane on the westbound Main Street approach at Ramada Drive.

<u>Alternative 3</u>

1. Add a dedicated right-turn lane on the westbound Main Street approach at the US 101 northbound ramps. The recommended right-turn lane should be extended for length of

the Main Street segment from the US 101 northbound ramps to Ramada Drive, creating a four-lane cross section.
2. Add a second through lane on the westbound Main Street approach at Ramada Drive.

<u>Alternative 4</u>

1. Add a dedicated right-turn lane on the southbound Ramada Drive approach at Main Street.

<u>Alternative 5</u>

1. Add second entry lane on the southbound Ramada Drive approach designated only for the traffic accessing the US 101 northbound on-ramp. This lane will act as a yielding bypass lane.
2. Add second entry lane on the westbound Main Street approach designated only for the traffic heading northbound on Ramada Drive. This lane will act as a yielding bypass lane.

Based on traffic operations analysis for five alternatives, RICK determined that the roundabout (as contained in Alternative 5) would be a viable option for the westerly closely spaced intersections, i.e. US 101 southbound ramps and Theatre Drive. The Type L-6 “hook” ramps (as contained in Alternative 4) for the US 101 northbound is anticipated to result in regulated traffic flow relative to other alternatives studied. This “hybrid” alternative would offer traffic operational benefits of Alternative 4 and 5.

At the easterly intersections, some additional alternatives that may improve traffic flow with minimal physical improvements relative to the five alternatives studied are listed below:

1. The US 101 northbound off-ramp would be relocated to intersect Main Street across from Ramada Drive, which will essentially form a standard four-legged intersection. Remove existing diagonal northbound on-ramp and construct a loop ramp which will be teed up with the Ramada Drive/Main Street intersection and will accommodate all US 101 northbound on-ramp traffic. This configuration would create the Type L-7 configuration for the US 101 northbound ramps.
2. The US 101 northbound off-ramp would be relocated to intersect Main Street across from Ramada Drive, which will essentially form a standard four-legged intersection. Retain existing diagonal on-ramp for the westbound Main Street traffic. Construct a loop for the eastbound Main Street traffic. Both movements would be freely flowing.
3. The US 101 northbound off-ramp would be relocated to intersect Main Street across from Ramada Drive, which will essentially form a standard four-legged intersection. Remove existing diagonal on-ramp and construct a hook on-ramp that would connect Ramada Drive with the Type L-6 ramp (similar to northbound on-ramp under Alternative 5).

## 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 five (5) 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 1: Widen Existing Bridge to Accommodate Side-by-Side Left Turn Lanes
Alternative 2: Modified Diamond Interchange (includes realigning frontage roads)
Alternative 3: Fully Compliant Interchange (includes realigning frontage roads)
Alternative 4: Double Hook Ramps (includes minor realigning frontage roads)
Alternative 5: Double Roundabout (includes minor realigning frontage roads)

This is the third 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).

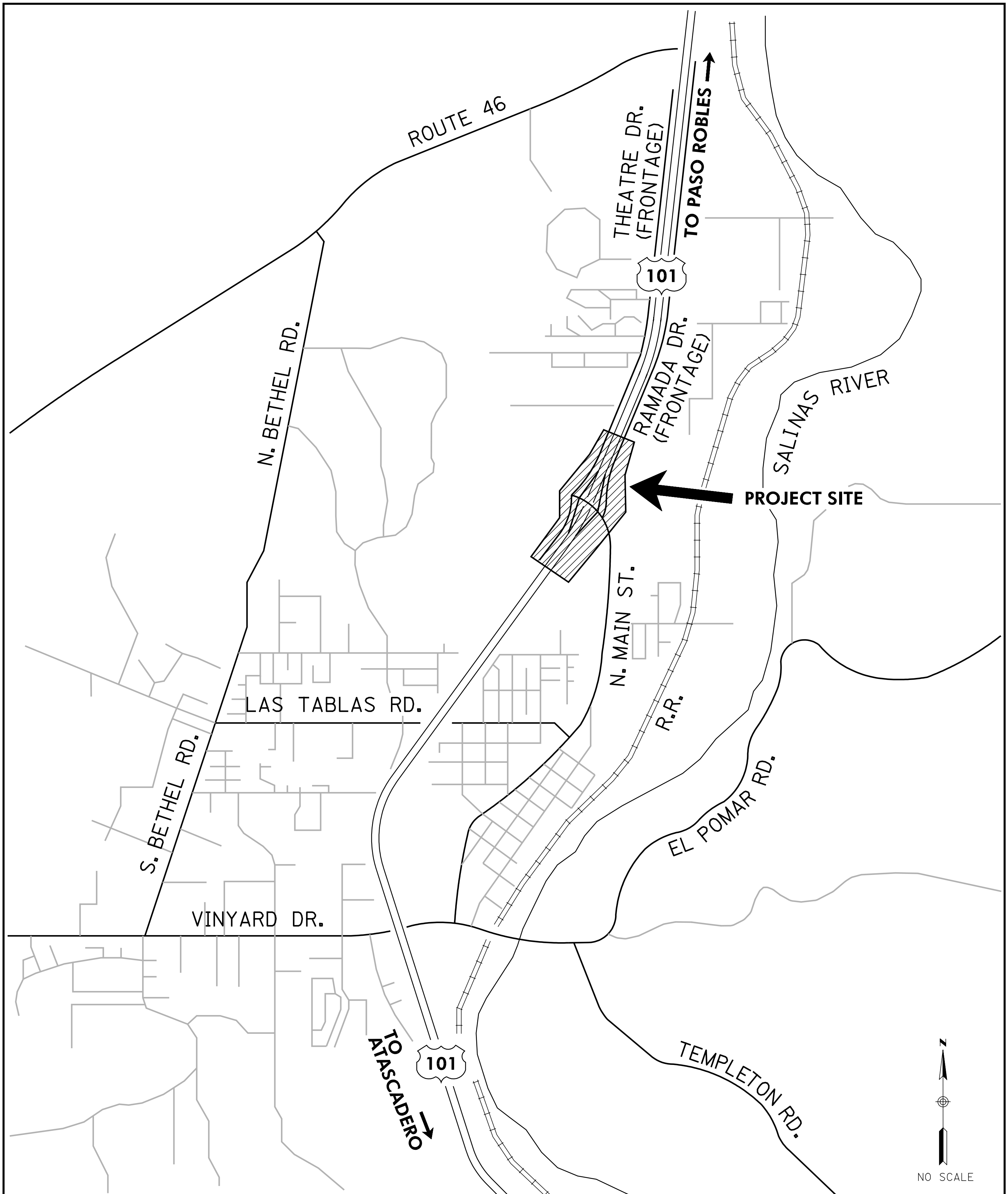
This memorandum includes the development of the AM and PM peak hour traffic volumes under the existing and buildout conditions for each improvement alternative. The existing and buildout traffic conditions for each alternative are evaluated utilizing the proposed layout of roadway and intersection geometrics 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 five (5) 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 1 (Widen Existing Bridge for Left Turn Lanes)

This interchange alternative would widen the existing bridge to provide side-to-side left-turn lanes better accommodating traffic traveling to the US 101 northbound and southbound on-ramps. It is anticipated that the widening would be approximately 35 feet to meet current Caltrans lane width requirements. In order to provide continuity on the east side of bridge, this alternative also includes restriping and/or widening Main Street between the US 101 northbound ramps and Ramada Drive to provide a dedicated left turn lane for northbound Ramada Drive traffic and a dedicated right turn lane for the northbound US 101 on-ramp traffic. On the west side, Main Street between the US 101 southbound ramps and Theater Drive will be restriped and/or widened to provide a dedicated westbound right turn lane for northbound Theatre Drive

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# EXHIBIT 1

PROJECT VICINITY

US 101/MAIN STREET TRAFFIC STUDY



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traffic and a right turn lane for southbound US 101 on-ramp traffic. **Exhibit 2** illustrates the preliminary conceptual layout for Alternative 1. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

Alternative 2 (Modified Diamond)

This alternative would relocate Theatre Drive to the west and Ramada Drive to the east to meet Caltrans current mandatory spacing requirements between intersections. As a result, the spacing between each intersection along Main Street within the study area would be approximately 400 feet. It should be noted that no bridge widening is planned under this Alternative. The new intersection of Main Street and Theatre Drive would be a "T" intersection. **Exhibit 3** depicts the preliminary conceptual layout for Alternative 2. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

Alternative 3 (Fully Compliant Interchange)

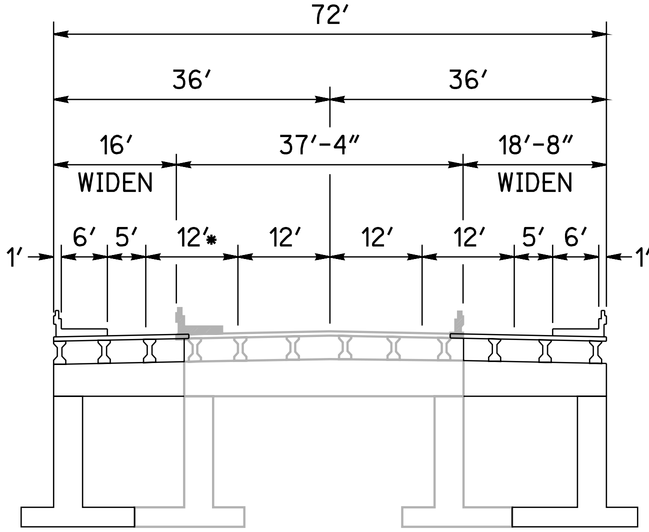
This alternative would relocate US 101 northbound and southbound ramps, Theatre Drive and Ramada Drive to meet the Caltrans current advisory spacing requirements between intersections. As a result, the spacing between each intersection along Main Street within the study area would be 500 feet. It should be noted that no bridge widening is planned under this alternative. The new intersection of Main Street and Theatre Drive would be a "T" intersection. **Exhibit 4** depicts the preliminary conceptual layout for alternative 3. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

Alternative 4 (Double Hook Ramps)

This alternative would re-configure the entire interchange with the Caltrans Type L-6 ramps on the west and east side of US 101. The existing US 101 ramp connections to Main Street would be removed entirely. The Type L-6 "hook" ramps would connect to Ramada Drive and Theater Drive approximately 800 feet north of the Main Street bridge. The existing alignments of Ramada Drive and Theatre Drive with Main Street would remain unchanged. **Exhibit 5** illustrates the preliminary conceptual layout for Alternative 4. It should be noted that no bridge widening is planned for this alternative. Traffic control at the study intersections were determined based on the operations and signal warrant analysis described later in this report.

Alternative 5 (Double Roundabout)

This alternative would eliminate the four existing intersections and construct two roundabouts. One roundabout would serve the US 101 southbound ramps, Main Street and Theater Drive, one roundabout would serve the US 101 NB ramps, Main Street and Ramada Drive. The roundabout on the west side of the interchange would be a six-legged single-lane roundabout, whereas the roundabout on the east side of the interchange would to be a five-legged single-lane roundabout. **Exhibit 6** depicts the preliminary conceptual layout for Alternative 5. It should be noted that no bridge widening is assumed for this alternative.




TYPICAL SECTION

NO SCALE

**NOTES:**

1. THIS DESIGN IS BASED ON THE T.Y. LIN, DATED 11/7/2006. REVISIONS INCLUDE THE ADDITION OF A 6' SIDEWALK ON THE NORTH EDGE OF THE BRIDGE AND SUBSEQUENT NECESSARY WIDENING.

2. ALL STRUCTRAL ELEMENTS SHOWN ON THIS PLAN ARE FOR VISUALIZATION PURPOSES ONLY, AND ARE NOT MEANT FOR CONSTRUCTION.

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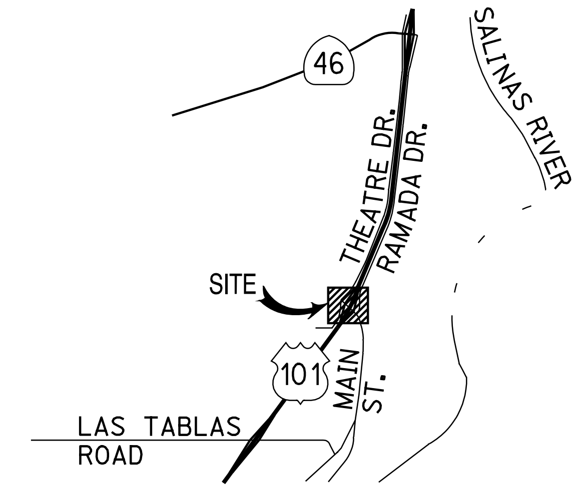
GRAPHIC SCALE 1"= 100'


VICINITY MAP

*NO SCALE*

NOTES:

1. BRIDGE WIDENING TO ACCOMMODATE FOUR 12' LANES, TWO 5' SHOULDERS AND A 6' SIDEWALK.

LEGEND:

(TS) SIGNALIZED INTERSECTION

(U) UNSIGNALIZED INTERSECTION

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

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL.

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# MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS

ALTERNATIVE #1 - WIDEN BRIDGE TO ACCOMMODATE LEFT TURNS

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*Chkd By: DAD*
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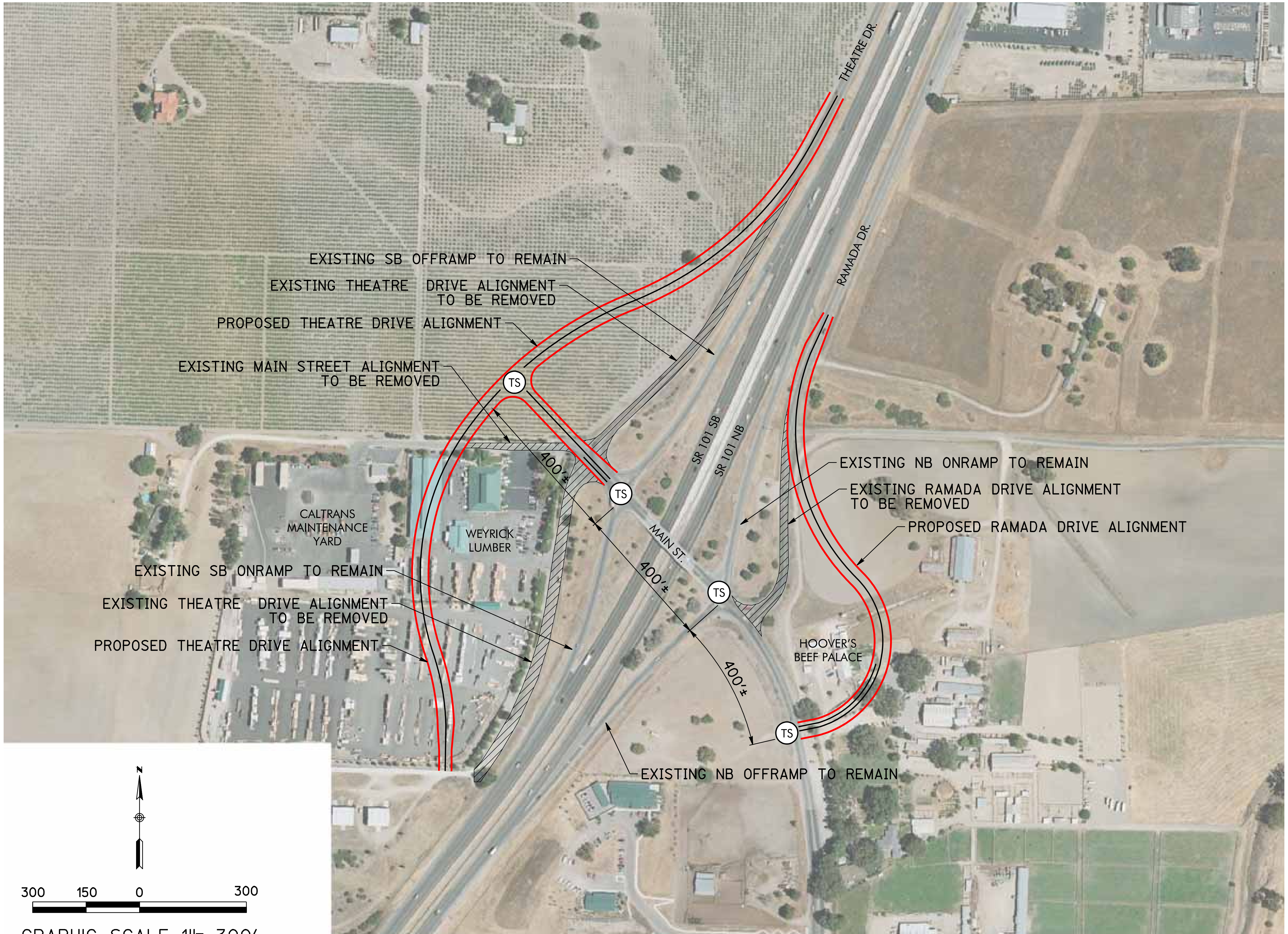


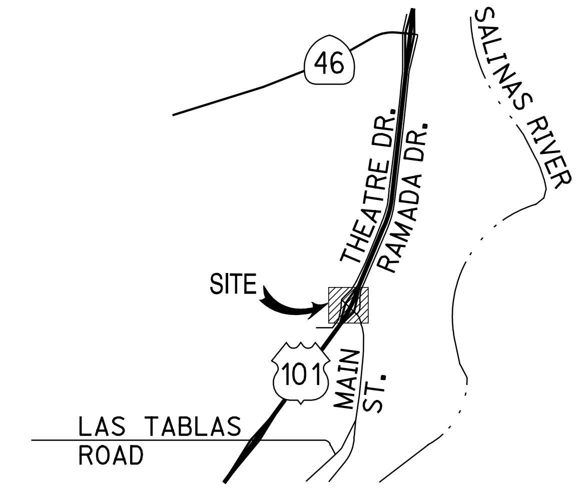


VICINITY MAP
*NO SCALE*

NOTES:

1. SPACING BETWEEN ALL INTERSECTIONS IS 400'.

LEGEND:

NEW ROADWAY

EXISTING ROADWAY TO BE REMOVED

(TS) SIGNALIZED INTERSECTION

(U) UNSIGNALIZED INTERSECTION

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

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL



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# MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS

## ALTERNATIVE #2 - MODIFIED DIAMOND INTERCHANGE

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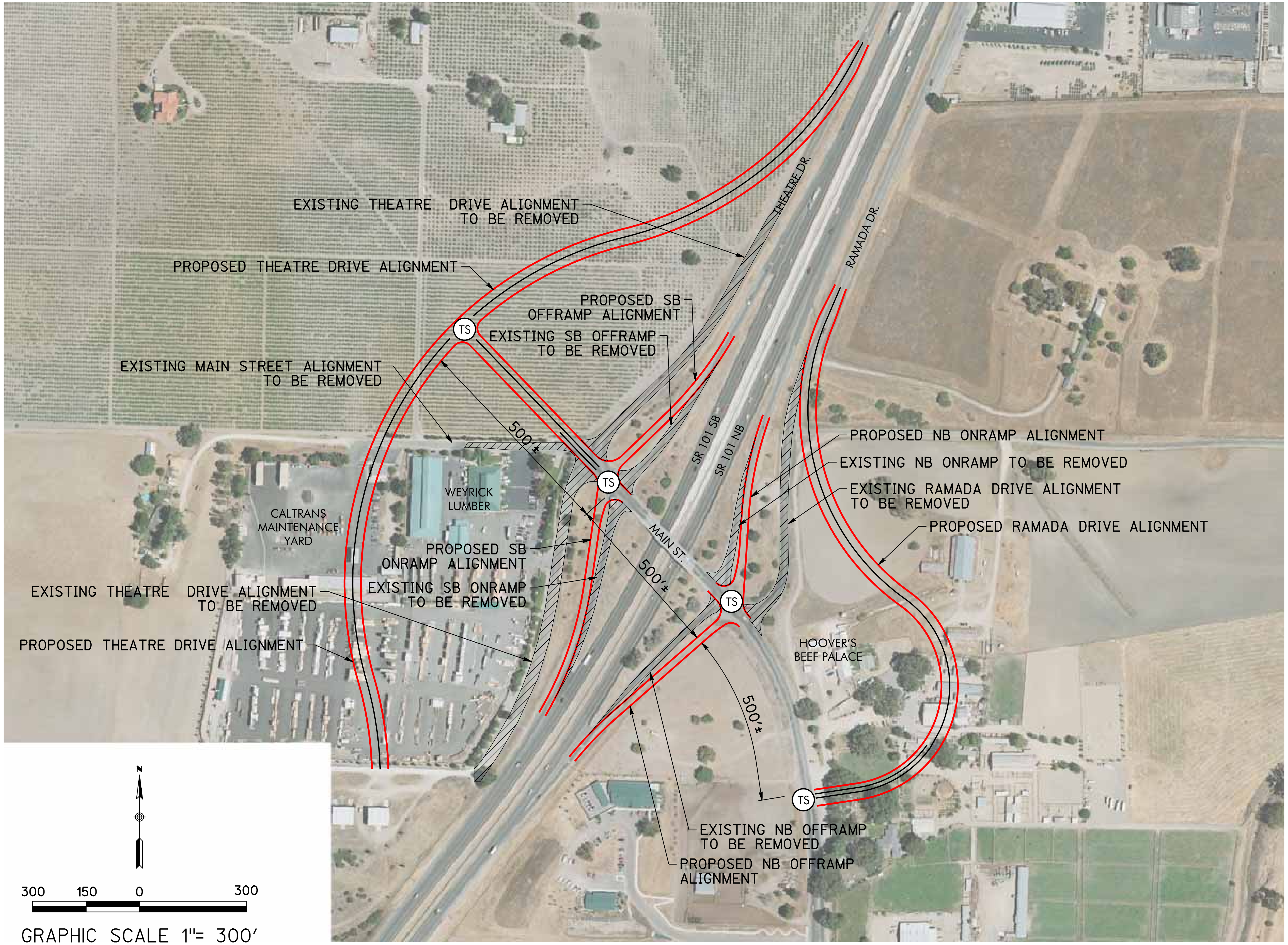


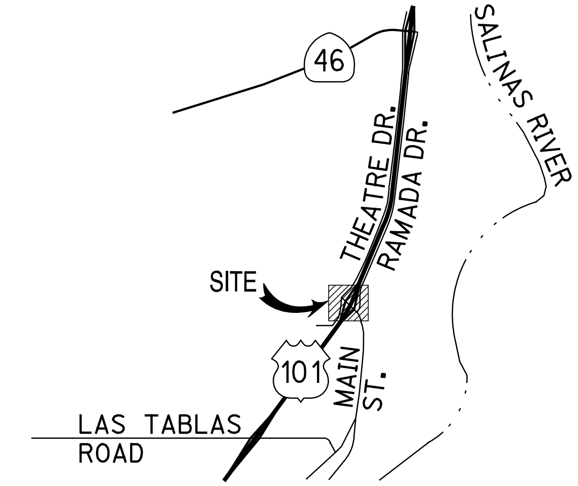


VICINITY MAP
*NO SCALE*

NOTES:

1. SPACING BETWEEN ALL INTERSECTIONS IS 500'.

LEGEND:

NEW ROADWAY

EXISTING ROADWAY TO BE REMOVED

(TS) SIGNALIZED INTERSECTION

(U) UNSIGNALIZED INTERSECTION

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

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL

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# MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS

## ALTERNATIVE #3 - FULLY COMPLIANT INTERCHANGE

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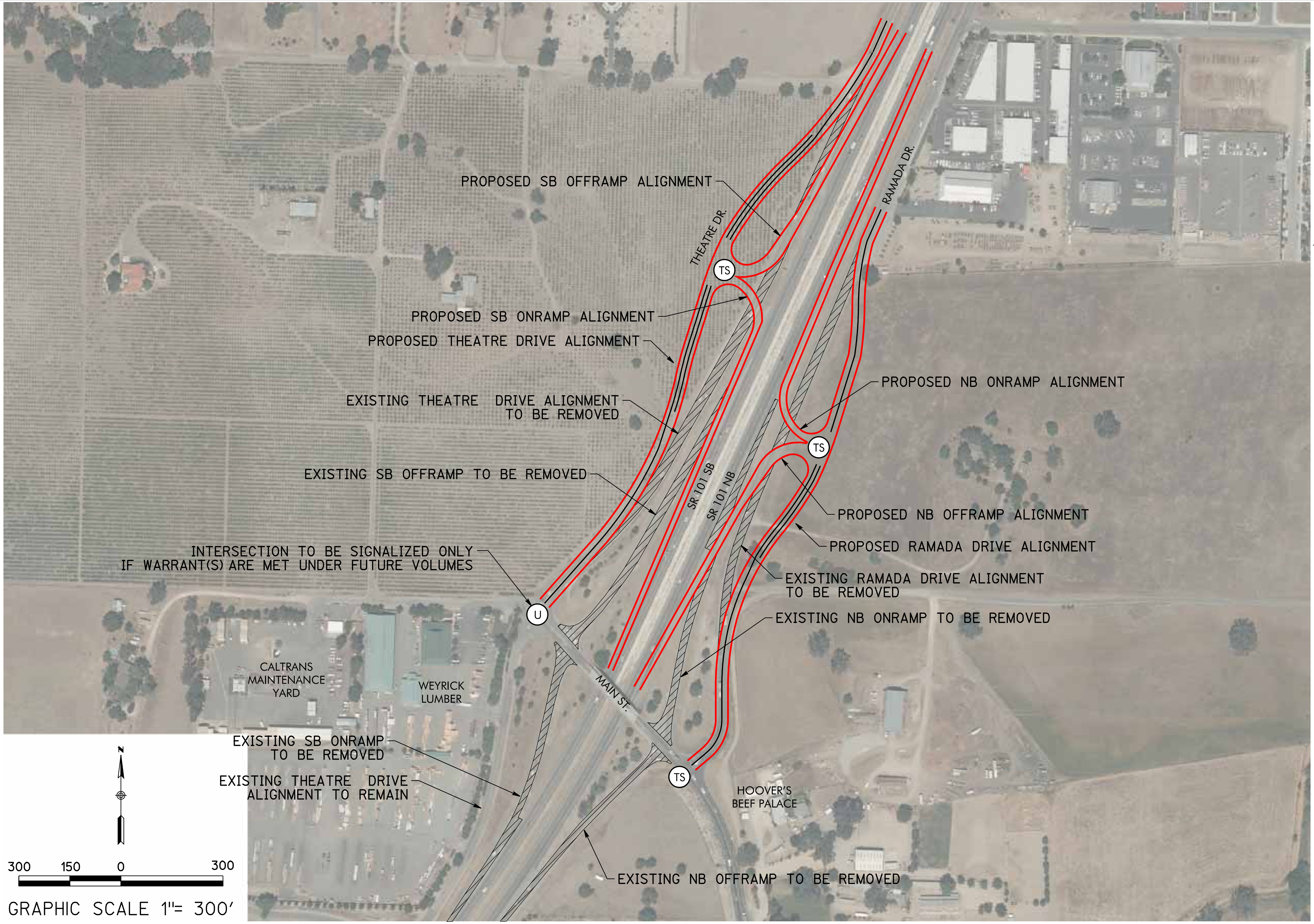


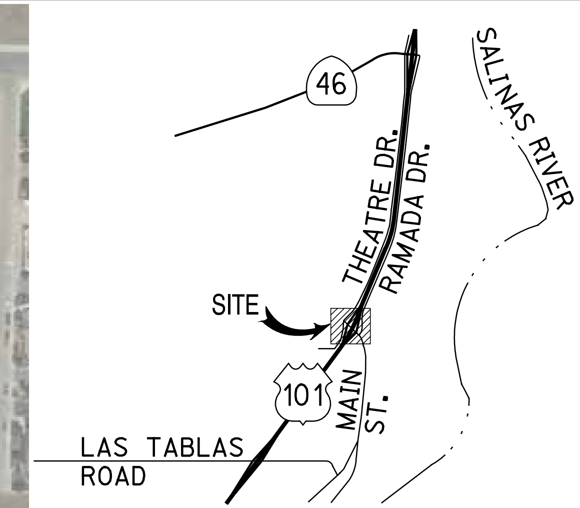


VICINITY MAP
*NO SCALE*

LEGEND:

- NEW ROADWAY
- EXISTING ROADWAY TO BE REMOVED
- TS SIGNALIZED INTERSECTION
- U UNSIGNALIZED INTERSECTION

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

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL.



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# MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS

## ALTERNATIVE #4 - DOUBLE HOOK RAMPS

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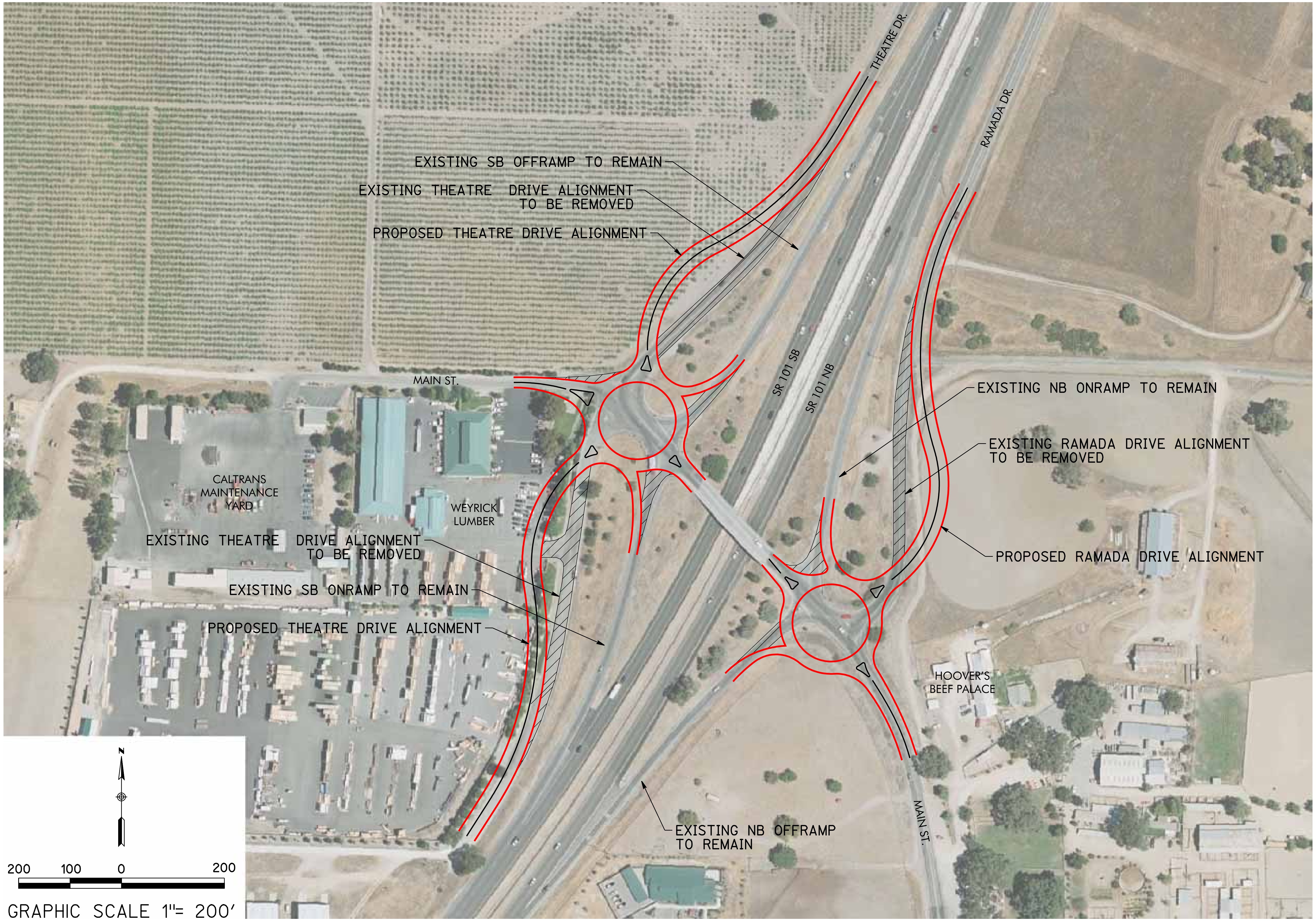


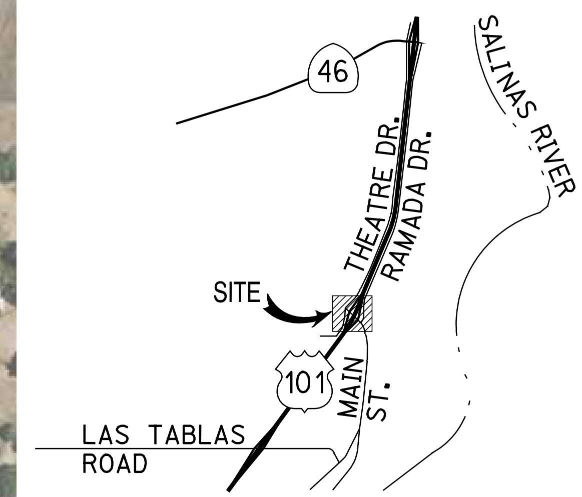


VICINITY MAP
*NO SCALE*

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) |
| | 25 MPH (RAMP AT LOCAL ROAD) |
| INTERCHANGE SPACING | MIN. 400' (PREFERRED 500') |
| DISTANCE BETWEEN SUCCESSIVE OFF-RAMPS | 1000' |
| LOOP RAMP RADIUS | 120'-300' |

*BASED ON THE 2006 CALTRANS HIGHWAY DESIGN MANUAL

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# MAIN STREET / US 101 INTERCHANGE IMPROVEMENTS

ALTERNATIVE #5 - DOUBLE ROUNDABOUT

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## 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.

**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 intersections of Main Street with Ramada Drive and Theatre Drive. For the two intersections of Main Street with the US 101 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, at the intersection of Main Street with the two ramp intersections, 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 internal, and reflect an average of 5 runs. The 95$^{th}$ percentile queues which present maximum back of queues for the 95$^{th}$ 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 95$^{th}$ 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 95$^{th}$ 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 95$^{th}$ 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.

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), 6$^{th}$ 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

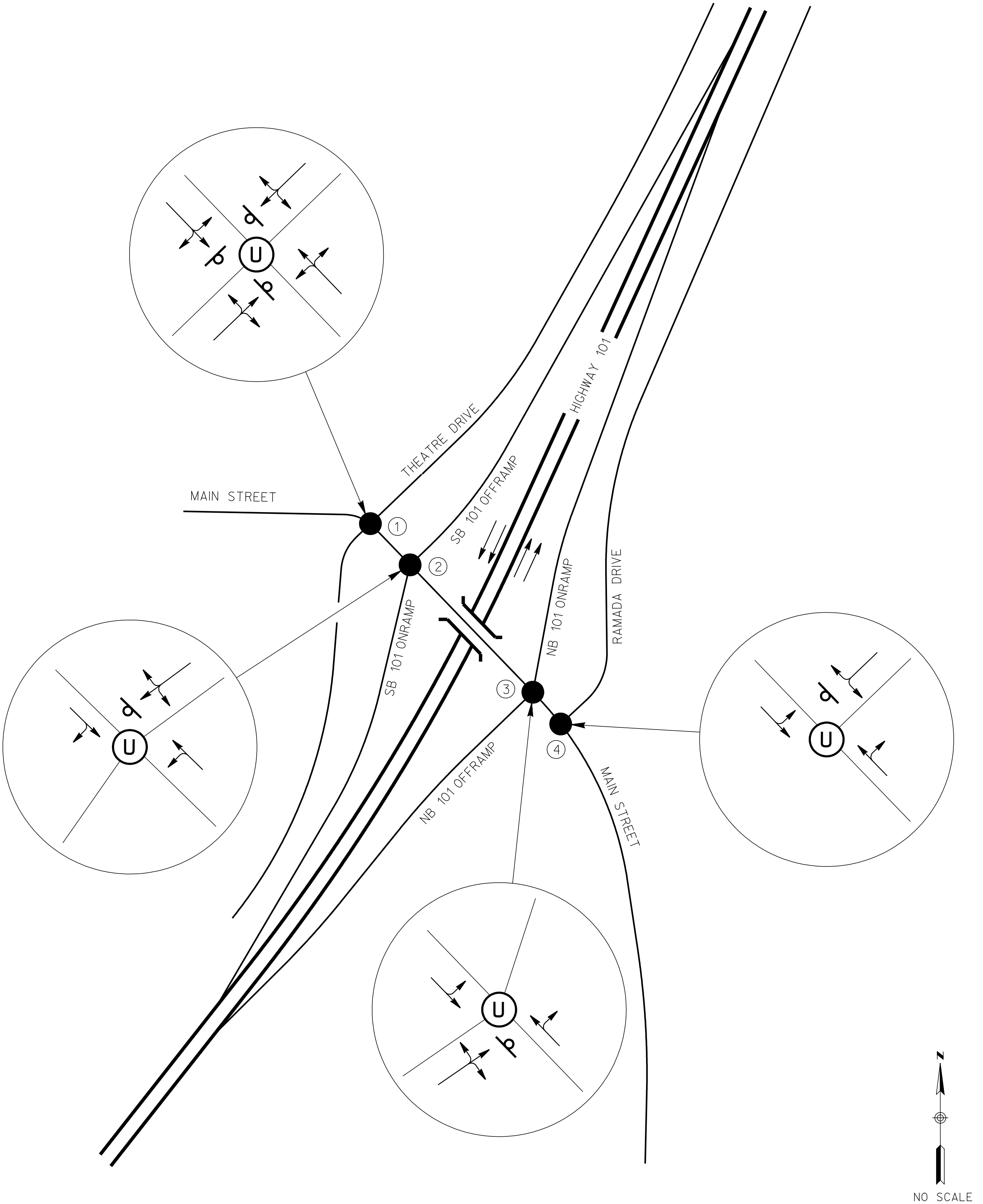
As previously stated, Deliverable 1 included a detailed evaluation of existing conditions at the US 101 / Main Street Interchange (July 1, 2011). Refer to Deliverable 1 for a complete description of the Exiting Roadway Network, Existing Traffic Volumes and Analysis. **Exhibit 7** shows the existing lane geometrics and traffic controls at the study intersections. **Exhibit 8** 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

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# EXHIBIT 7

EXISTING CONDITIONS INTERSECTION
LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- = STUDY INTERSECTION
- = STOP-CONTROLLED
- U = UNSIGNALIZED



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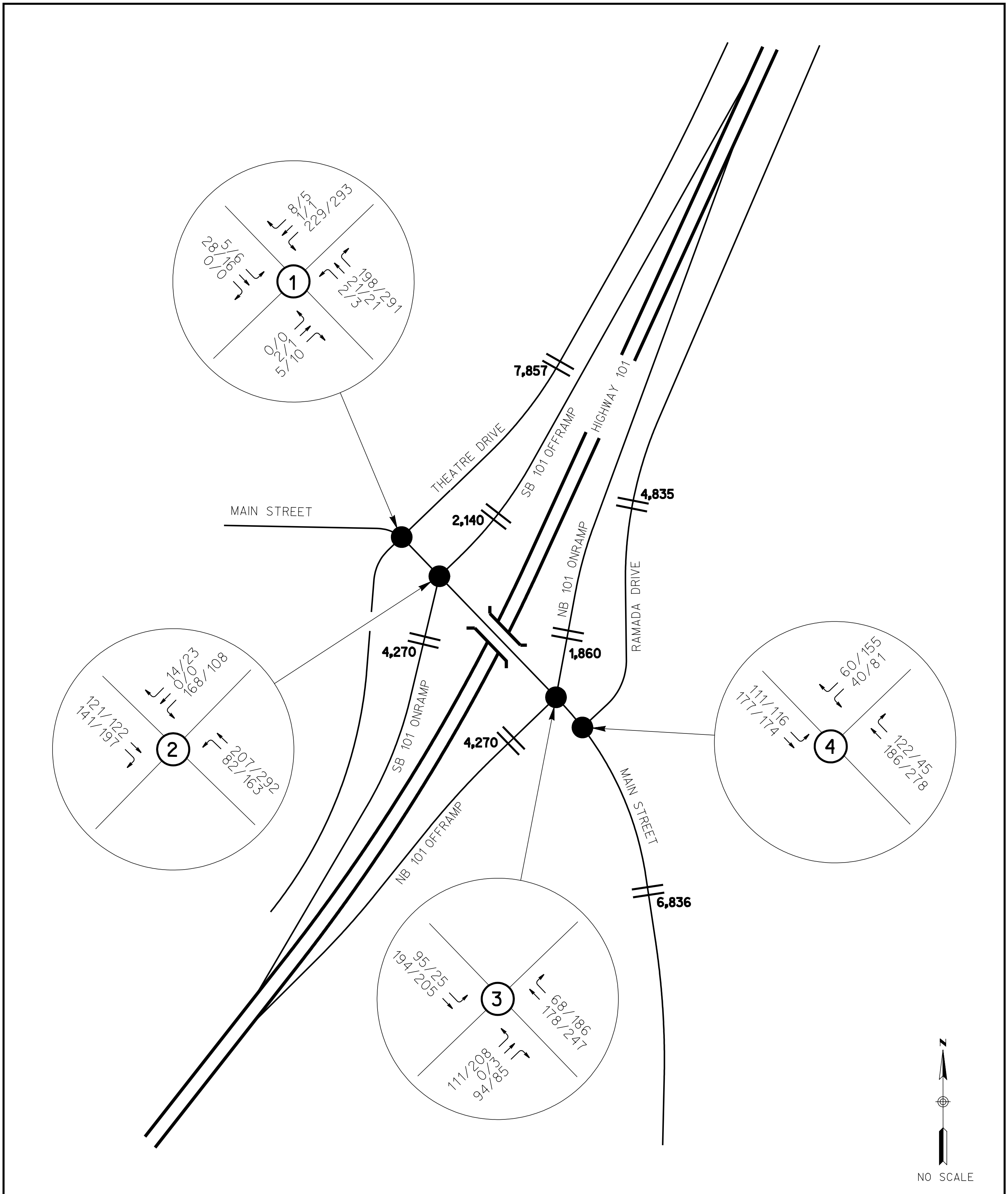


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# EXHIBIT 8

EXISTING TRAFFIC VOLUMES

US 101/NORTH MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

XX/XX = AM/PM PEAK HOUR TRAFFIC

**XX** = AVERAGE DAILY TRAFFIC



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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.

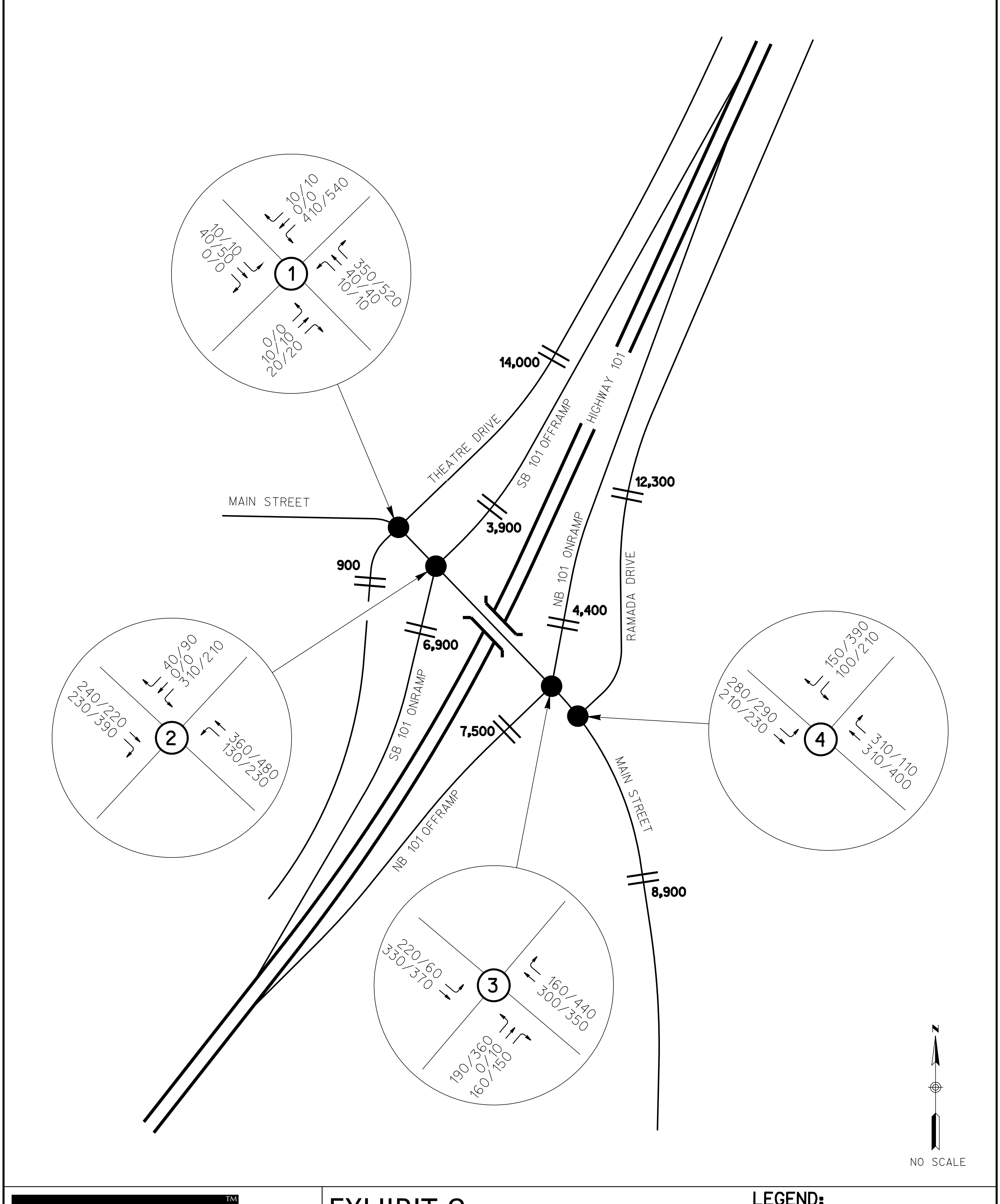
## 4.0 BUILDOUT CONDITIONS

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 9** 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.



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# EXHIBIT 9

BUILDOUT TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

XX/XX = AM/PM PEAK HOUR TRAFFIC

**XX** = AVERAGE DAILY TRAFFIC

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## 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.

Alternative 1

As previously stated, Alternative 1 includes widening the Main Street Bridge without the realignment of Ramada Drive or Theater Drive. Approach improvements on Main Street would also be included to facilitate the bridge widening. Peak hour and daily traffic volumes are not anticipated to change as compared to existing conditions. Therefore, the existing AM and PM peak hour traffic volumes shown on **Exhibit 8** were used for analysis of Alternative 1.

**Exhibit 10** illustrates the assumed lane geometrics and traffic controls for Alternative 1. It was assumed that Main Street between Ramada Drive and the US 101 northbound ramps and between the US 101 southbound ramps and Theatre Drive would be improved to provide a separate right turn lanes for continuity of the bridge widening.

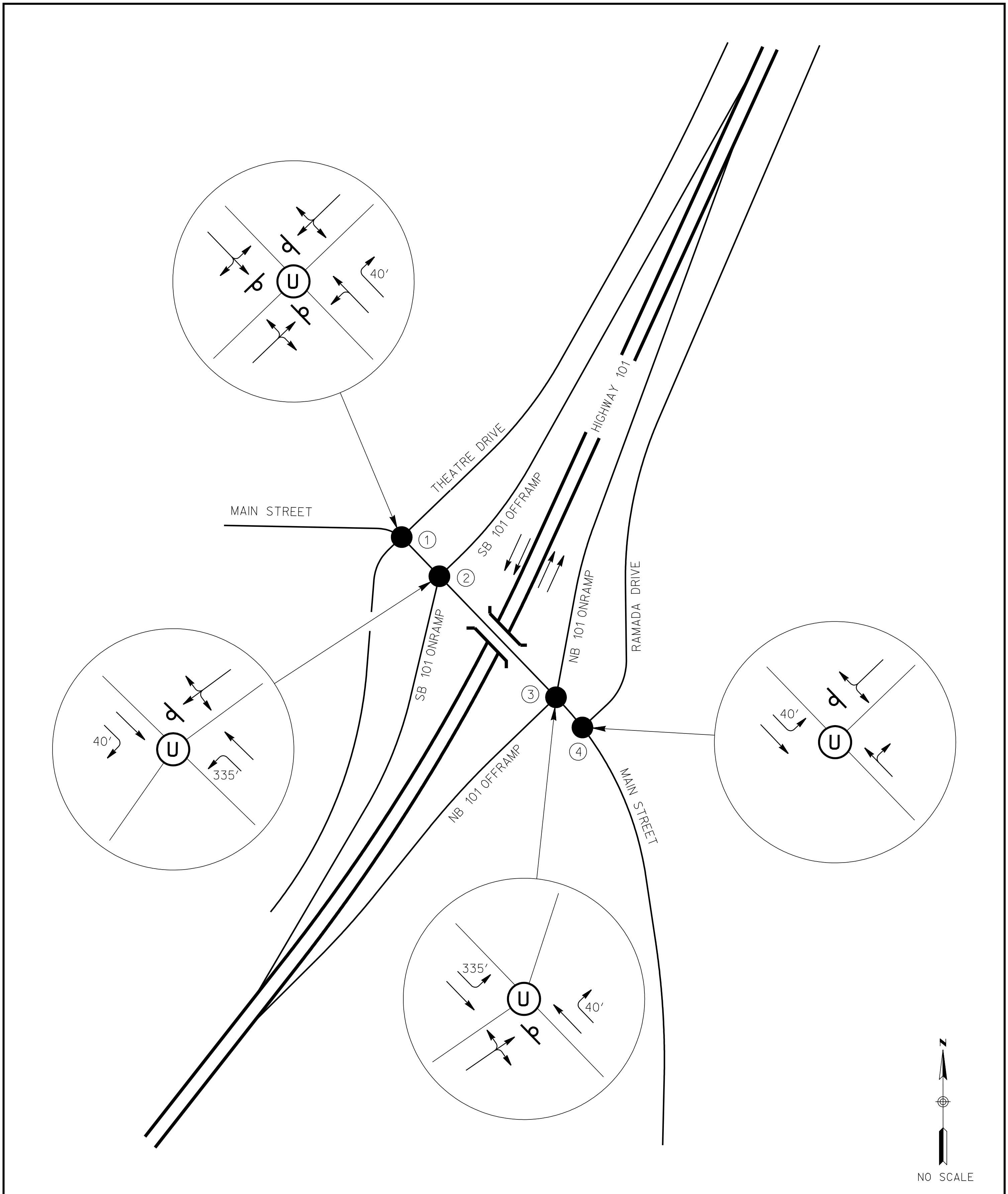
Alternative 2

As previously stated, Alternative 2 includes the relocation of Ramada Drive and Theatre Drive by 400 feet along Main Street. This alternative also eliminates the west leg at the Main Street and Theatre Drive intersection, with the inbound and outbound traffic volumes on this leg being assigned to the south leg. **Exhibit 11** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 2 under the existing conditions.

The assumed lane geometrics and traffic controls for Alternative 2 are shown on **Exhibit 12**. It was assumed that the segment of Main Street west of the US 101 southbound ramps intersection would be improved to provide a separate right turn lane for the southbound on-ramp traffic. Main Street currently provides approximately 55 feet of travel way between the US 101 southbound ramps and Theatre Drive, and therefore, the assumed turn lane, up to Theatre Drive, is anticipated to be accommodated within the existing pavement width without need of widening.

Alternative 3

As previously stated, Alternative 3 includes relocation of Ramada Drive and Theatre Drive by 500 feet along Main Street, and eliminates the west leg of the Main Street and Theatre Drive intersection. The inbound and outbound traffic volumes on this leg were assigned to the south leg. It is anticipated that the minor realignment of the US 101 ramps would also not result any significant change in traffic volumes. **Exhibit 11** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 3 under the existing conditions.

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# EXHIBIT 10

EXISTING CONDITIONS WITH ALTERNATIVE 1
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- ● = STUDY INTERSECTION
- = STOP-CONTROLLED
- (U) = UNSIGNALIZED
- 40' = TRAFFIC LANE & STORAGE



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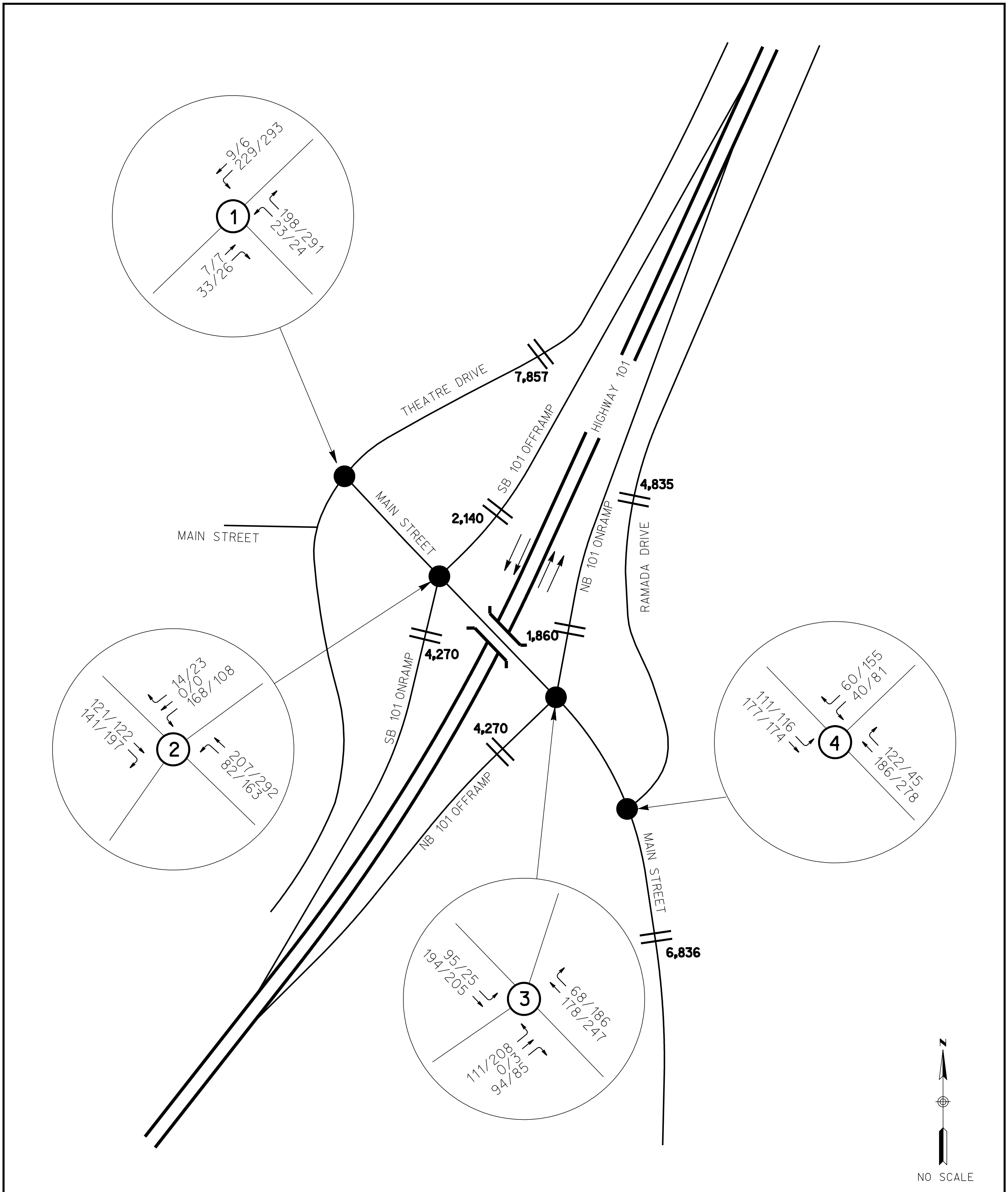


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# EXHIBIT 11

EXISTING CONDITIONS WITH ALTERNATIVE 2 & 3
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

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



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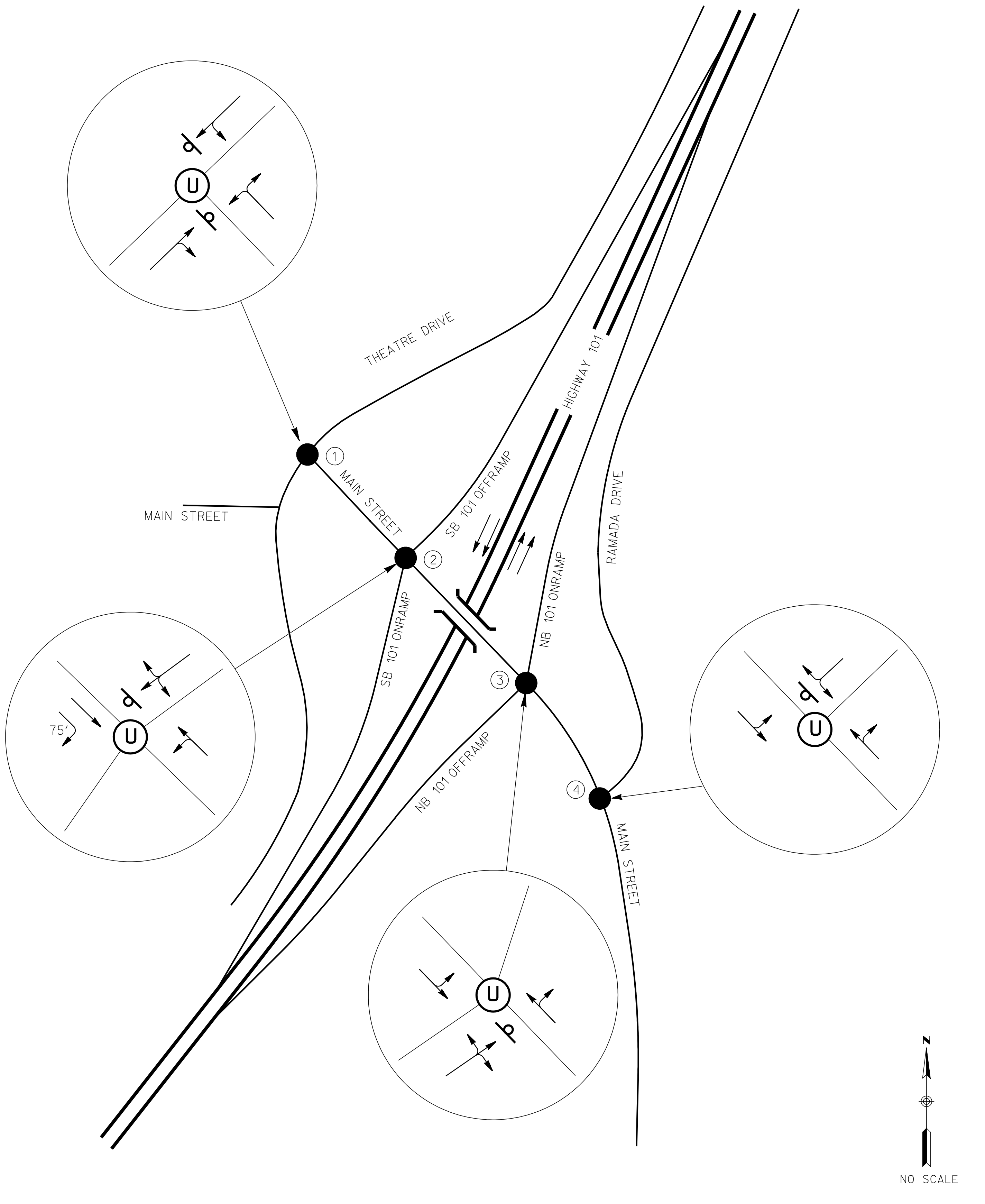




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# EXHIBIT 12

EXISTING CONDITIONS WITH ALTERNATIVE 2 & 3
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION
= STOP-CONTROLLED
Ⓤ = UNSIGNALIZED
40' = TRAFFIC LANE & STORAGE

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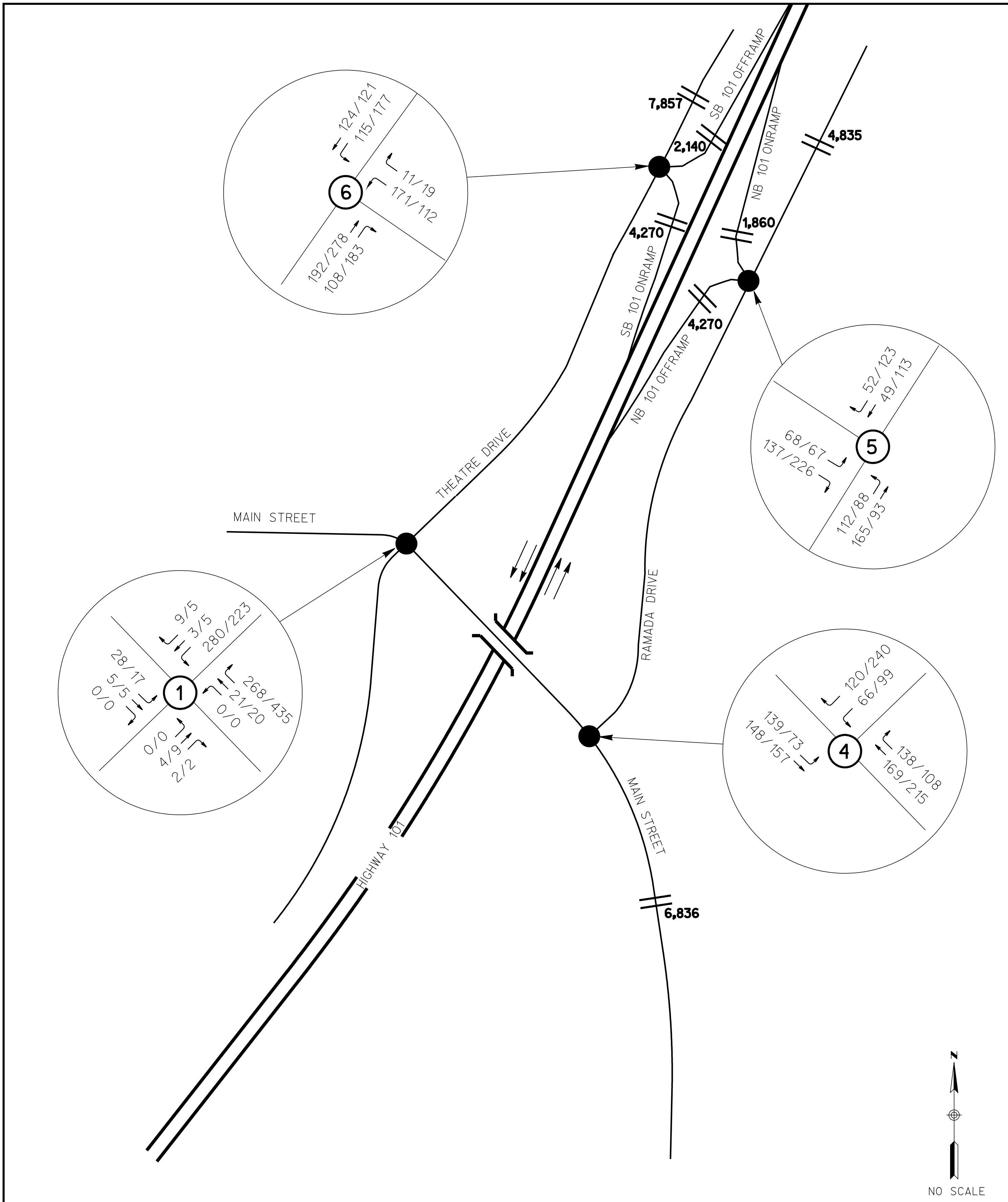
**Exhibit 12** shows the assumed lane geometrics and traffic controls for Alternative 3. It was assumed that the segment of Main Street west of the US 101 southbound ramp intersection would be improved to provide a separate right turn lane for the southbound on ramp traffic. Main Street currently provides approximately 55 feet travel way between the US 101 southbound ramps and Theatre Drive, and therefore, the assumed turn lane, up to Theatre Drive, is anticipated to be accommodated within the existing pavement width without need of widening.

Alternative 4

As previously stated, Alternative 4 includes the removal and relocation of the existing US 101 northbound and southbound ramps. Therefore, the AM and PM peak hour traffic volumes at the study intersections would differ from those under existing conditions as illustrated on Exhibit 8. The existing peak hour and daily traffic volumes were reassigned based on their origin and destination within the study area, as identified in the actual traffic counts. **Exhibit 13** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 4 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 4 are shown on **Exhibit 14**.

Alternative 5

Alternative 5 entails traffic control related improvements 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. **Exhibit 15** illustrated the AM and PM peak hour and daily traffic volumes for Alternative 5 under existing conditions. The assumed lane geometrics and traffic controls for Alternative 5 are shown on **Exhibit 16**.

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# EXHIBIT 13

EXISTING CONDITIONS WITH ALTERNATIVE 4
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

XX/XX = AM/PM PEAK HOUR TRAFFIC

**XX** = AVERAGE DAILY TRAFFIC



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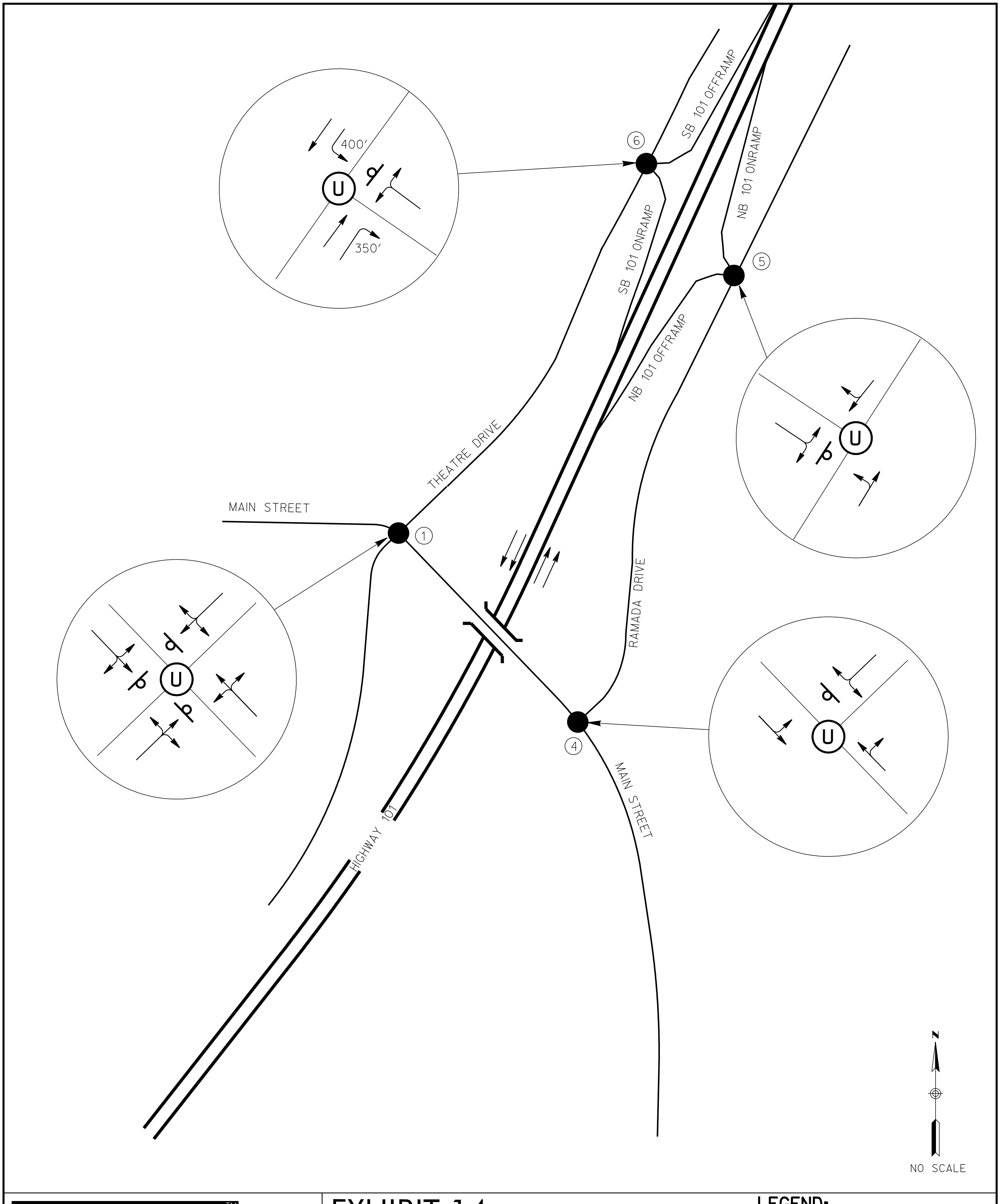


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# EXHIBIT 14

EXISTING CONDITIONS WITH ALTERNATIVE 4
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- ● = STUDY INTERSECTION
- = STOP-CONTROLLED
- (U) = UNSIGNALIZED
- 40' = TRAFFIC LANE & STORAGE



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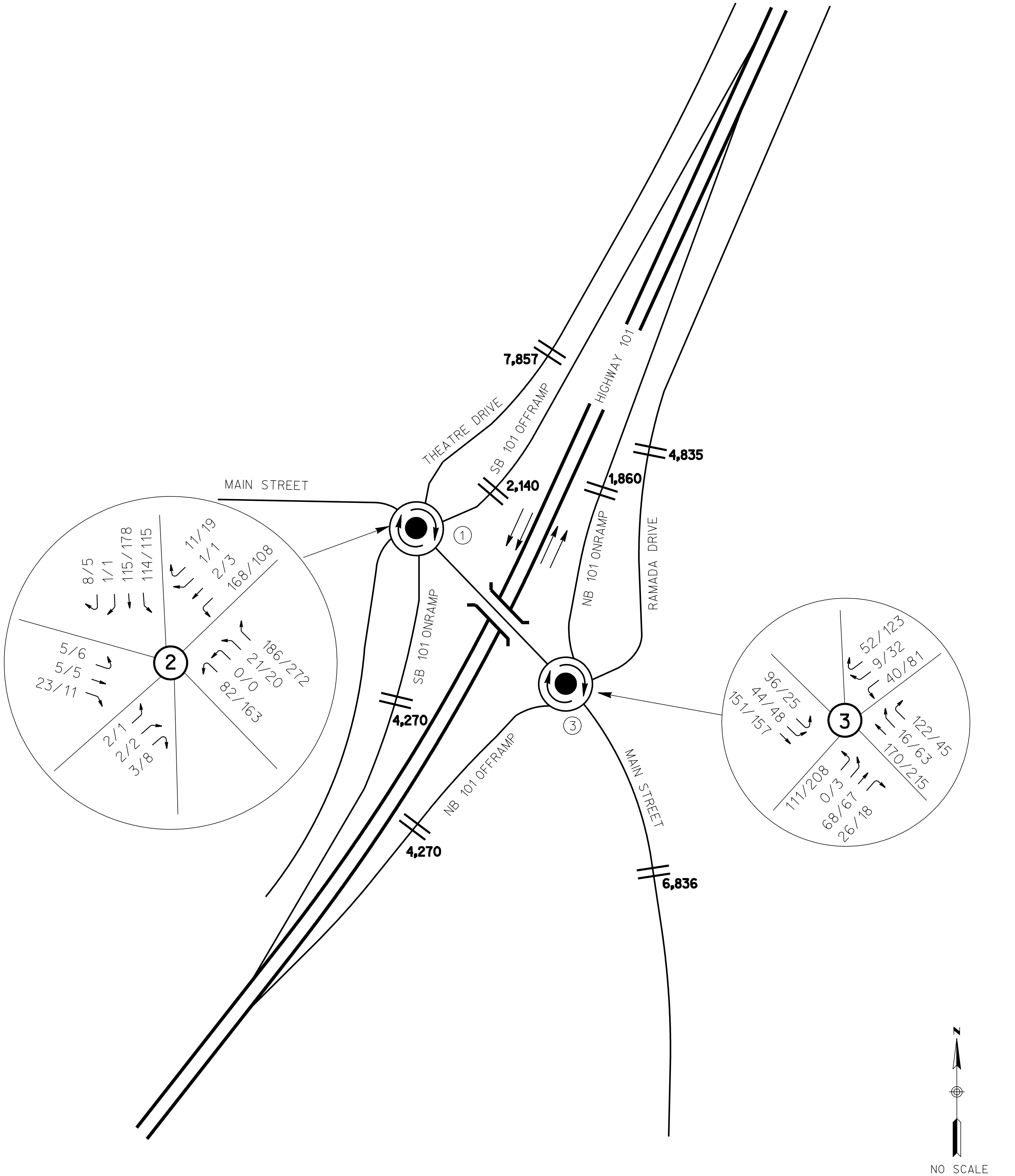


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# EXHIBIT 15

EXISTING CONDITIONS WITH ALTERNATIVE 5
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

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



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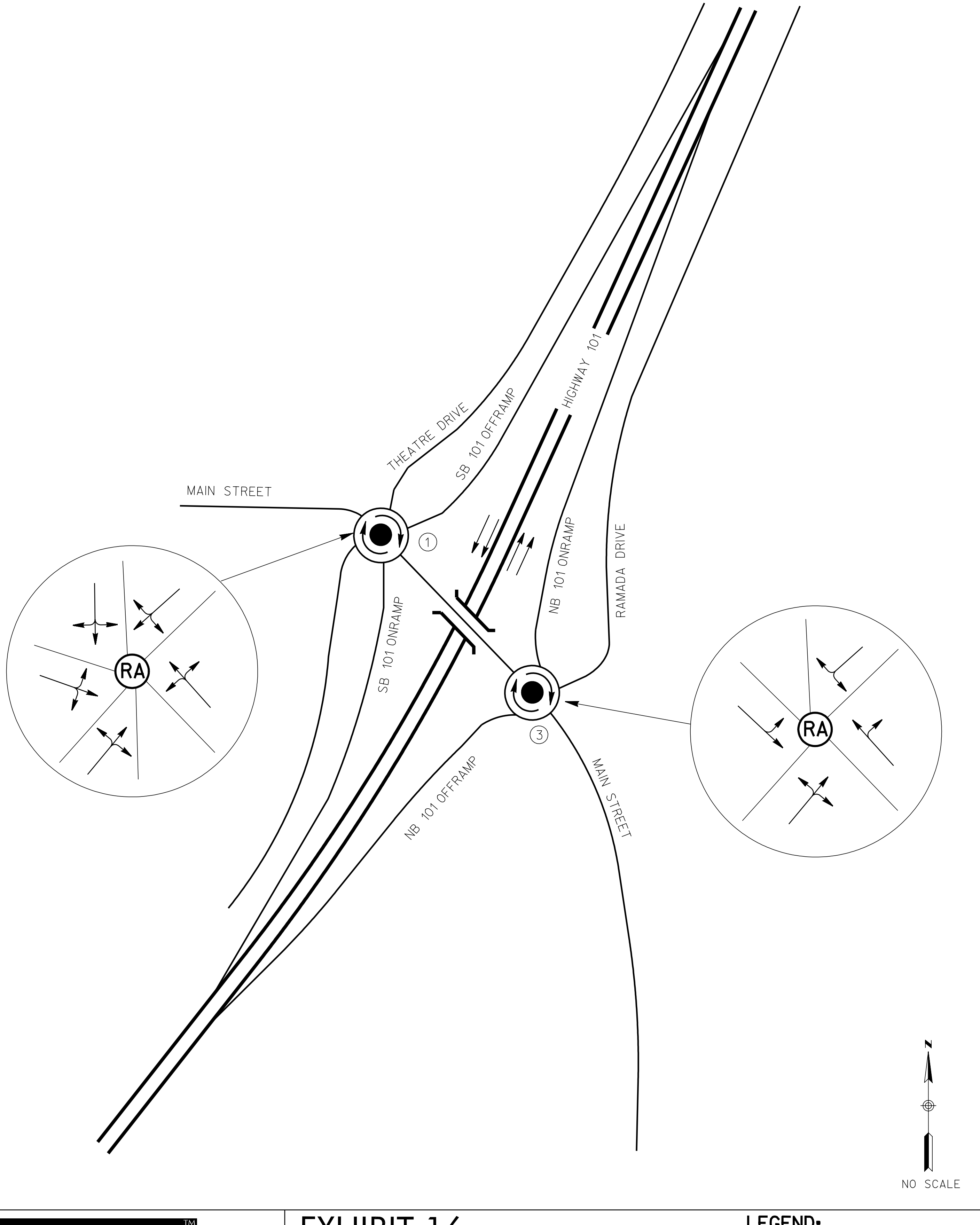


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# EXHIBIT 16

EXISTING CONDITIONS WITH ALTERNATIVE 5
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

(RA) = ROUNDABOUT

40' = TRAFFIC LANE & STORAGE



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## 5.2 Traffic Operations Analysis

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

Alternative 1

**Table 5** presents the results of the intersection LOS analysis for Alternative 1. 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 except the Main Street/US 101 northbound ramps intersection, which is anticipated to operate at LOS D 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 5**
**EXISTING CONDITIONS WITH ALTERNATIVE 1 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 1 | |
|---|---|---|---|
| | | Avg. Delay | LOS |
| Theatre Drive (TWSC) | **AM Peak** | | |
| | EB L | 1.2 | A |
| | WB L | 0.7 | A |
| | NB LTR | 9.3 | A |
| | SB LTR | 10.8 | B |
| | **PM Peak** | | |
| | EB L | 2.2 | A |
| | WB L | 0.9 | A |
| | NB LTR | 8.8 | A |
| | SB LTR | 11.4 | B |
| US 101 SB Ramps (TWSC) | **AM Peak** | | |
| | WB L | 8.2 | A |
| | SB LTR | 20.3 | C |
| | **PM Peak** | | |
| | WB L | 8.6 | A |
| | SB LTR | **27.3** | **D** |
| US 101 NB Ramps (TWSC) | **AM Peak** | | |
| | EB L | 8.1 | A |
| | NB LTR | 15.4 | C |
| | **PM Peak** | | |
| | EB L | 8.4 | A |
| | NB LTR | 19.8 | C |
| Ramada Drive (TWSC) | **AM Peak** | | |
| | EB L | 8.4 | A |
| | SB LTR | 12.9 | B |
| | **PM Peak** | | |
| | EB L | 8.4 | A |
| | SB LTR | 14.8 | B |

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 queuing analysis results under existing conditions with Alternative 1. The estimated 95th percentile queues were reported to be accommodated within the available or assumed storage lengths, with the exception of the eastbound left-turn lane at Main Street and Ramada Drive intersection. 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 1 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
|---|---|---|---|---|
| Theatre Drive (TWSC) | **AM Peak** | | | |
| | EB LT | 500 | 25 | Sufficient |
| | WB L | 40 | 25 | Sufficient |
| | SB LTR | 1000 | 80 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 500 | 25 | Sufficient |
| | WB L | 40 | 25 | Sufficient |
| | SB LTR | 1000 | 93 | Sufficient |
| US 101 SB Ramps (TWSC) | **AM Peak** | | | |
| | WB LT | 300 | 43 | Sufficient |
| | SB LTR | 1000 | 157 | Sufficient |
| | **PM Peak** | | | |
| | WB LT | 300 | 70 | Sufficient |
| | SB LTR | 1000 | 151 | Sufficient |
| US 101 NB Ramps (TWSC) | **AM Peak** | | | |
| | EB LT | 300 | 29 | Sufficient |
| | NB LTR | 800 | 93 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 300 | 25 | Sufficient |
| | NB LTR | 800 | 131 | Sufficient |
| Ramada Drive (TWSC) | **AM Peak** | | | |
| | EB LT | 40 | **66 (3%)** | **Insufficient** |
| | SB LR | 1000 | 50 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 40 | **60 (4%)** | **Insufficient** |
| | SB LR | 1000 | 67 | 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 2

**Table 7** presents the results of intersection LOS analysis for Alternative 2. Overall, the intersection operations are projected to improve with this alternative relative to the existing interchange configuration. The Ramada Drive and Theater Drive intersections are projected to operate within acceptable LOS during the AM and PM peak hours. However, both ramp intersections are anticipated to exceed the Caltrans 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 2 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 2 | |
|---|---|---|---|
| | | Avg. Delay | LOS |
| Theatre Drive (TWSC) | **AM Peak** | | |
| | WB L | 0.9 | A |
| | NB TR | 9.0 | A |
| | SB TR | 13.4 | B |
| | **PM Peak** | | |
| | WB L | 0.7 | A |
| | NB LTR | 9.2 | A |
| | SB LTR | 15.7 | C |
| US 101 SB Ramps (TWSC) | **AM Peak** | | |
| | WB L | 2.8 | A |
| | SB LTR | 20.3 | C |
| | **PM Peak** | | |
| | WB L | 4.1 | A |
| | SB LTR | **27.3** | **D** |
| US 101 NB Ramps (TWSC) | **AM Peak** | | |
| | EB L | 3.2 | A |
| | NB LTR | 16.1 | C |
| | **PM Peak** | | |
| | EB L | 1.1 | A |
| | NB LTR | **26.4** | **D** |
| Ramada Drive (TWSC) | **AM Peak** | | |
| | EB L | 3.8 | A |
| | SB LTR | 14.0 | B |
| | **PM Peak** | | |
| | EB L | 4.0 | A |
| | SB LTR | 19.9 | 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 8** summarizes the intersection queuing analysis results under existing conditions with Alternative 2. The 95th percentile queues were reported to be accommodated within the available or assumed storage lengths at all study intersections.

**TABLE 8**

**EXISTING CONDITIONS WITH ALTERNATIVE 2 QUEUE RESULTS**

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
|---|---|---|---|---|
| Theatre Drive (TWSC) | **AM Peak** | | | |
| | WB TR | 400 | 25 | Sufficient |
| | NB TR | 500 | 25 | Sufficient |
| | SB LT | 1000 | 96 | Sufficient |
| | **PM Peak** | | | |
| | WB TR | 400 | 44 | Sufficient |
| | NB TR | 500 | 25 | Sufficient |
| | SB LT | 1000 | 99 | Sufficient |
| US 101 SB Ramps (TWSC) | **AM Peak** | | | |
| | WB LT | 300 | 55 | Sufficient |
| | SB LTR | 1000 | 127 | Sufficient |
| | **PM Peak** | | | |
| | WB LT | 300 | 103 | Sufficient |
| | SB LTR | 1000 | 109 | Sufficient |
| US 101 NB Ramps (TWSC) | **AM Peak** | | | |
| | EB LT | 300 | 58 | Sufficient |
| | NB LTR | 800 | 96 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 300 | 31 | Sufficient |
| | NB LTR | 800 | 133 | Sufficient |
| Ramada Drive (TWSC) | **AM Peak** | | | |
| | EB LT | 400 | 73 | Sufficient |
| | SB LR | 1000 | 66 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 400 | 75 | 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 3

**Table 9** presents the results of the intersection LOS analysis for Alternative 3. Overall, the intersection operations are projected to improve with this alternative relative to the existing interchange configuration. The Ramada Drive and Theater Drive intersections are projected to operate within acceptable LOS during the AM and PM peak hours. However, both ramp intersections are anticipated to exceed the Caltrans 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 9**

**EXISTING CONDITIONS WITH ALTERNATIVE 3 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 3 | |
|---|---|---|---|
| | | Avg. Delay | LOS |
| Theatre Drive (TWSC) | **AM Peak** | | |
| | WB L | 0.9 | A |
| | NB TR | 9.0 | A |
| | SB TR | 13.4 | B |
| | **PM Peak** | | |
| | WB L | 0.7 | A |
| | NB LTR | 9.2 | A |
| | SB LTR | 15.7 | C |
| US 101 SB Ramps (TWSC) | **AM Peak** | | |
| | WB L | 2.8 | A |
| | SB LTR | 20.3 | C |
| | **PM Peak** | | |
| | WB L | 4.1 | A |
| | SB LTR | **27.3** | **D** |
| US 101 NB Ramps (TWSC) | **AM Peak** | | |
| | EB L | 3.2 | A |
| | NB LTR | 16.1 | C |
| | **PM Peak** | | |
| | EB L | 1.1 | A |
| | NB LTR | **26.4** | **D** |
| Ramada Drive (TWSC) | **AM Peak** | | |
| | EB L | 3.8 | A |
| | SB LTR | 14.0 | B |
| | **PM Peak** | | |
| | EB L | 4.0 | A |
| | SB LTR | 19.9 | 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 10** summarizes the intersection queuing analysis results under existing conditions with Alternative 3. The 95th percentile queues were reported to be accommodated within the available or assumed storage lengths at all study intersections.

**TABLE 10**

**EXISTING CONDITIONS WITH ALTERNATIVE 3 QUEUE RESULTS**

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
|---|---|---|---|---|
| Theatre Drive (TWSC) | **AM Peak** | | | |
| | WB TR | 500 | 25 | Sufficient |
| | NB TR | 500 | 55 | Sufficient |
| | SB LT | 1000 | 93 | Sufficient |
| | **PM Peak** | | | |
| | WB TR | 500 | 45 | Sufficient |
| | NB TR | 500 | 25 | Sufficient |
| | SB LT | 1000 | 91 | Sufficient |
| US 101 SB Ramps (TWSC) | **AM Peak** | | | |
| | WB LT | 470 | 51 | Sufficient |
| | SB LTR | 1000 | 126 | Sufficient |
| | **PM Peak** | | | |
| | WB LT | 470 | 95 | Sufficient |
| | SB LTR | 1000 | 100 | Sufficient |
| US 101 NB Ramps (TWSC) | **AM Peak** | | | |
| | EB LT | 470 | 45 | Sufficient |
| | NB LTR | 800 | 102 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 470 | 34 | Sufficient |
| | NB LTR | 800 | 142 | Sufficient |
| Ramada Drive (TWSC) | **AM Peak** | | | |
| | EB LT | 500 | 72 | Sufficient |
| | SB LR | 1000 | 63 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 500 | 75 | Sufficient |
| | SB LR | 1000 | 126 | 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 4

**Table 11** presents the results of the intersection LOS analysis for Alternative 4. Overall, the intersection operations are projected to improve with this alternative relative to the existing interchange configuration. The study intersections are projected to operate at acceptable LOS, except delays on the stop-controlled approach at the US 101 southbound ramps which are anticipated to exceed the Caltrans LOS D threshold during the PM peak hour. It is recommended that peak hour traffic volumes at this intersection be monitored to determine when a traffic signal would be warranted.

**TABLE 11**

**EXISTING CONDITIONS WITH ALTERNATIVE 4 LOS RESULTS**

| Study Intersection | Movement | Existing Conditions with Alternative 4 | |
|---|---|---|---|
| | | Avg. Delay | LOS |
| Main Street & Theatre Drive (TWSC) | **AM Peak** | | |
| | EB L | 6.9 | A |
| | WB L | 0.1 | A |
| | NB LTR | 10.8 | B |
| | SB LTR | 15.1 | C |
| | **PM Peak** | | |
| | EB L | 6.6 | A |
| | WB L | 0.1 | A |
| | NB LTR | 12.6 | B |
| | SB LTR | 15.0 | B |
| Main Street & Ramada Drive (TWSC) | **AM Peak** | | |
| | EB L | 4.8 | A |
| | SB LTR | 16.6 | C |
| | **PM Peak** | | |
| | EB L | 3.0 | A |
| | SB LTR | 20.8 | C |
| Ramada Drive & US 101 NB Ramps (TWSC) | **AM Peak** | | |
| | EB LR | 12.5 | B |
| | NB L | 3.6 | A |
| | **PM Peak** | | |
| | EB LR | 14.0 | B |
| | NB L | 4.2 | A |
| Theatre Drive & US 101 SB Ramps (TWSC) | **AM Peak** | | |
| | WB LR | 21.7 | C |
| | SB L | 8.4 | A |
| | **PM Peak** | | |
| | WB LR | **28.0** | **D** |
| | SB L | 9.3 | A |

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 12** summarizes the intersection queuing analysis results under existing conditions with Alternative 4. The 95$^{th}$ percentile queues were reported to be accommodated within the available or assumed storage lengths at all study intersections.

## TABLE 12
## EXISTING CONDITIONS WITH ALTERNATIVE 4 QUEUE RESULTS

| Study Intersection | Movement | Assumed Storage Length (feet) | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
|---|---|---|---|---|
| Main Street & Theatre Drive (TWSC) | **AM Peak** | | | |
| | EB LTR | 500 | 37 | Sufficient |
| | WB LTR | 600 | 25 | Sufficient |
| | NB LTR | 500 | 25 | Sufficient |
| | SB LTR | 1100 | 97 | Sufficient |
| | **PM Peak** | | | |
| | EB LTR | 500 | 31 | Sufficient |
| | WB LTR | 645 | 25 | Sufficient |
| | NB LTR | 500 | 27 | Sufficient |
| | SB LTR | 1100 | 95 | Sufficient |
| Main Street & Ramada Drive (TWSC) | **AM Peak** | | | |
| | EB LT | 600 | 86 | Sufficient |
| | SB LR | 1000 | 135 | Sufficient |
| | **PM Peak** | | | |
| | EB LT | 600 | 57 | Sufficient |
| | SB LR | 1000 | 184 | Sufficient |
| Ramada Drive & US 101 NB Ramps (TWSC) | **AM Peak** | | | |
| | EB LR | 800 | 120 | Sufficient |
| | NB LT | 1000 | 55 | Sufficient |
| | **PM Peak** | | | |
| | EB LR | 800 | 133 | Sufficient |
| | NB LT | 1000 | 54 | Sufficient |
| Theatre Drive & US 101 SB Ramps (TWSC) | **AM Peak** | | | |
| | WB LR | 800 | 155 | Sufficient |
| | SB L | 400 | 63 | Sufficient |
| | **PM Peak** | | | |
| | WB LR | 800 | 131 | Sufficient |
| | SB L | 400 | 87 | 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 5

**Table 13** presents the results of the intersection LOS analysis for Alternative 5. Average delays for each movement at both roundabouts are projected to be within LOS A range during the AM and PM peak hours.

**TABLE 13**

**EXISTING CONDITIONS WITH ALTERNATIVE 5 LOS RESULTS**

| Study Intersection Main Street at: | Movement | Existing Conditions with Alternative 5 | |
|---|---|---|---|
| | | 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 (RAB) | **AM Peak** | | |
| | Average | 6.5 | A |
| | NB LTR | 6.6 | A |
| | WB LTR | 8.1 | A |
| | SB LTR | 5.6 | A |
| | EB LTR | 5.1 | A |
| | **PM Peak** | | |
| | Average | 7.2 | A |
| | NB LTR | 7.1 | A |
| | WB LTR | 8.1 | A |
| | SB LTR | 8.6 | A |
| | EB LTR | 4.5 | A |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 14** summarizes the roundabout queuing analysis results under existing conditions with Alternative 5. The 95th percentile queues were reported to be accommodated within the available or assumed storage lengths at both study intersections.

## TABLE 14
## EXISTING CONDITIONS WITH ALTERNATIVE 5 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 (RAB) | **AM Peak** | | | |
| | NB LTR | 800 | 30 | Sufficient |
| | WB LTR | 1000 | 48 | Sufficient |
| | SB LTR | 1000 | 25 | Sufficient |
| | EB LTR | 300 | 34 | Sufficient |
| | **PM Peak** | | | |
| | NB LTR | 800 | 43 | Sufficient |
| | WB LTR | 1000 | 50 | Sufficient |
| | SB LTR | 1000 | 39 | Sufficient |
| | EB LTR | 300 | 25 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 LOS and queue analysis calculation worksheets are provided in **Appendix B**.

## 6.0 BUILDOUT CONDITIONS WITH INTERCHANGE ALTERNATIVES

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

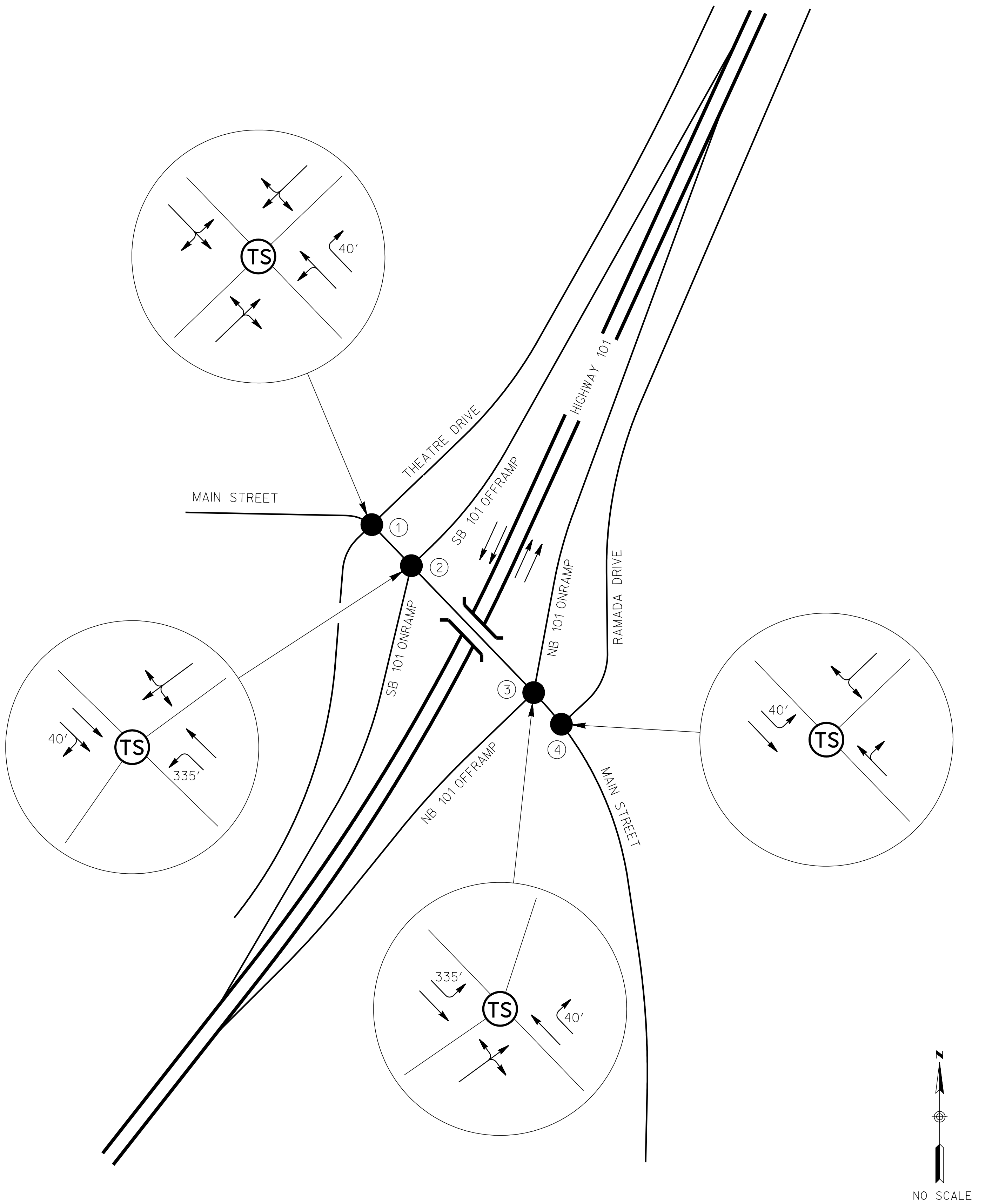
### 6.1 Traffic Volumes and Intersection Lane Geometrics

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

Alterative 1
As previously stated, Alternative 1 includes widening the Main Street Bridge without the realignment of Ramada Drive or Theater Drive. Approach improvements on Main Street would also be included to facilitate the bridge widening. Peak hour and daily traffic volumes are not anticipated to change as compared to the buildout conditions illustrated on **Exhibit 9**. Therefore, the buildout AM and PM peak hour traffic volumes shown on **Exhibit 9** were used for analysis of Alternative 1.

Under the buildout conditions, both the peak hour and average daily traffic signal warrants would be satisfied for the US 101 southbound and northbound ramp 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 15**. All signal warrant analysis worksheets are contained in **Appendix C**. The buildout peak hour traffic volumes at the Main Street/Ramada Drive and Main Street/Theatre Drive intersections would satisfy the minimum volume signal warrant during one or both peak hour periods. Therefore, the four study intersections were assumed to be signalized for the analysis of Alternative 1.

**Exhibit 17** illustrates the assumed lane geometrics and traffic controls for Alternative 1. It was assumed that Main Street between Ramada Drive and the US 101 northbound ramps and between the US 101 southbound ramps and Theatre Drive would be widened to provide a separate right turn lane for continuity of the bridge widening.



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# EXHIBIT 17

BUILDOUT WITH ALTERNATIVE 1
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- ● = STUDY INTERSECTION
- (TS) = TRAFFIC SIGNAL
- 40' = TRAFFIC LANE & STORAGE

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**TABLE 15**
**BUILDOUT CONDITIONS SIGNAL WARRANT ANALYSIS RESULTS**

| Study Intersection | Alternative 1 | | Alternative 2 | | Alternative 3 | | Alternative 4 | |
|---|---|---|---|---|---|---|---|---|
| | Delay Warrant Met? | Volume Warrant Met? | Delay Warrant Met? | Volume Warrant Met? | Delay Warrant Met? | Volume Warrant Met? | Delay Warrant Met? | Volume Warrant Met? |
| Main Street & Theatre Drive | | | | | | | | |
| **AM Peak** | **No** | **No** | Yes | **No** | Yes | **No** | Yes | Yes |
| **PM Peak** | **No** | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| Main Street & Ramada Drive | | | | | | | | |
| **AM Peak** | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| **PM Peak** | Yes | Yes | Yes | Yes | Yes | Yes | Yes | Yes |
| Ramada Drive & US 101 NB Ramps | | | | | | | | |
| **AM Peak** | NA | NA | NA | NA | NA | NA | Yes | Yes |
| **PM Peak** | NA | NA | NA | NA | NA | NA | Yes | Yes |
| Theatre Drive & US 101 SB Ramps | | | | | | | | |
| **AM Peak** | NA | NA | NA | NA | NA | NA | Yes | Yes |
| **PM Peak** | NA | NA | NA | NA | NA | NA | Yes | Yes |

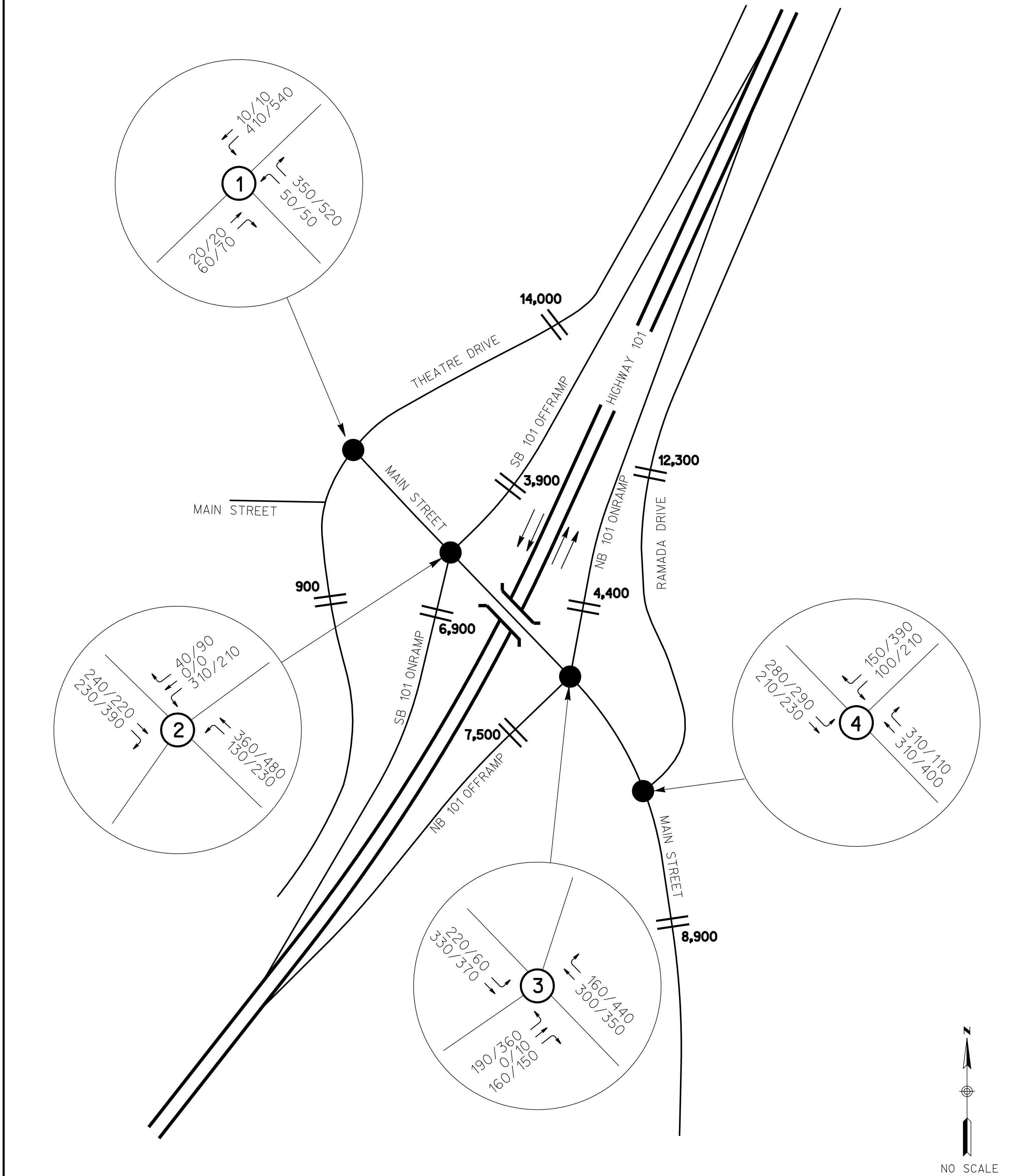
NA = Not Applicable

<u>Alternative 2</u>

As previously stated, Alternative 2 includes the relocation of Ramada Drive and Theatre Drive by 400 feet along Main Street. This alternative also eliminates the west leg at the Main Street and Theatre Drive intersection, with the inbound and outbound traffic volumes on this leg being assigned to the south leg. **Exhibit 18** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 2 under the buildout conditions.

Under the buildout conditions, both the peak hour and the average daily traffic signal warrants would be satisfied for the US 101 southbound and northbound ramp intersections. Refer to Section 8.0 in Deliverable 2 for the complete description and analysis. In addition, the buildout peak hour traffic volumes at the Main Street/Ramada Drive and Main Street/Theatre Drive intersections would satisfy the minimum peak hour volume signal warrant criteria during one or both peak hour periods. Therefore, the four study intersections were assumed to be signalized under the buildout with Alternative 2 conditions.

The assumed lane geometrics and traffic controls for Alternative 2 are shown on **Exhibit 19**. It was assumed that the segment of Main Street west of the US 101 southbound ramps intersection would be improved to provide a separate right turn lane for the southbound on-ramp traffic. Main Street currently provides approximately 55 feet travel way between the US 101 southbound ramps and Theatre Drive, and therefore, the assumed turn lane, up to Theatre Drive, is anticipated to be accommodated within the existing pavement width without need of widening. Main Street was assumed to have a separate left turn lane on the eastbound approach at Ramada

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# EXHIBIT 18

BUILDOUT WITH ALTERNATIVE 2 & 3
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

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



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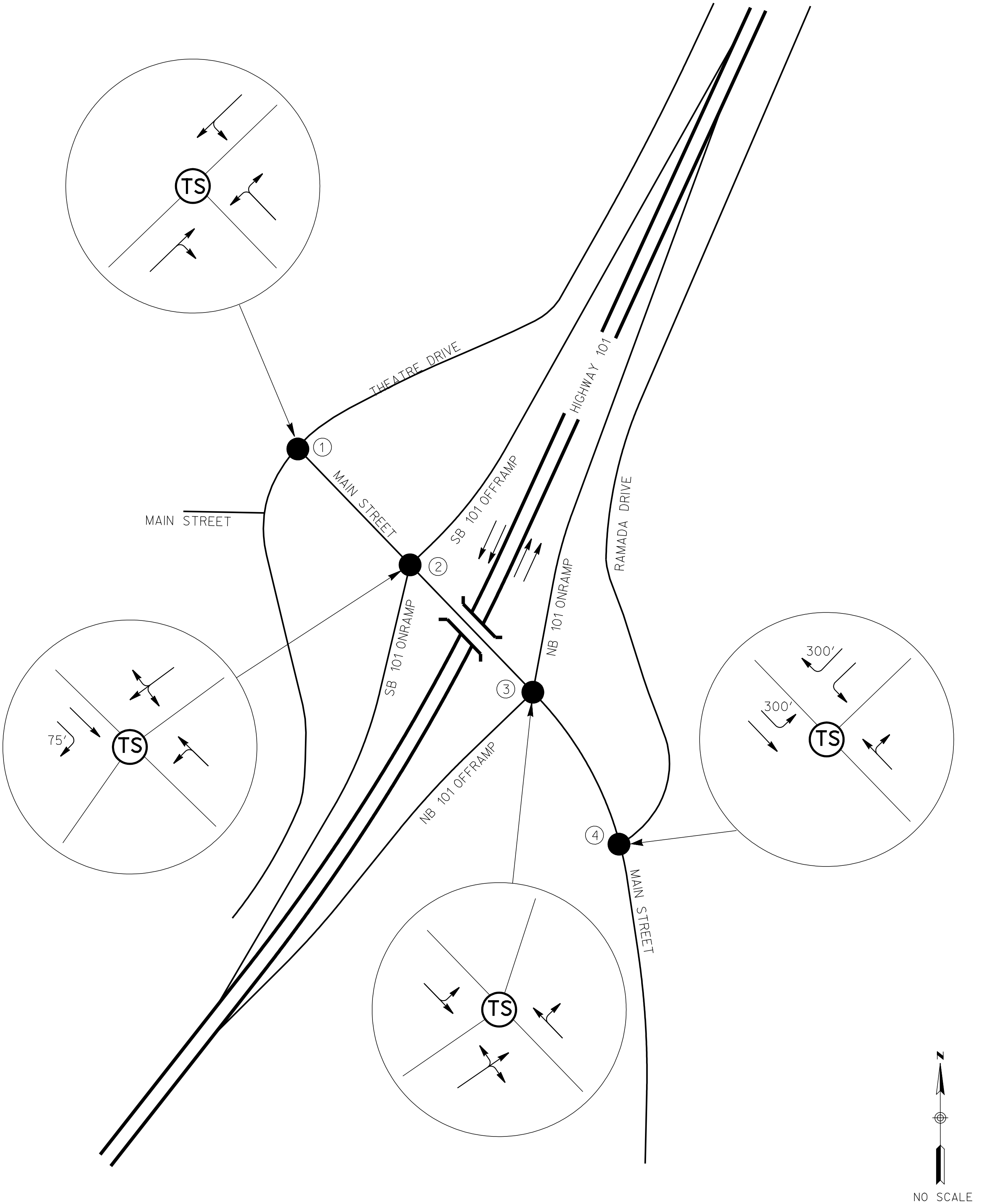


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# EXHIBIT 19

BUILDOUT WITH ALTERNATIVE 2 & 3
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

TS = TRAFFIC SIGNAL

40' = TRAFFIC LANE & STORAGE

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Drive, and Ramada Drive would have a separate left and right turn lane. It should be noted the assumed lane geometrics on Main Street may require widening.

Alternative 3

As previously stated, Alternative 3 includes the relocation of Ramada Drive and Theatre Drive by 500 feet along Main Street, and eliminates the west leg of the Main Street and Theatre Drive intersection. The inbound and outbound traffic volumes on this leg were assigned to the south leg. It is anticipated that the minor realignment of the US 101 northbound and southbound ramps would also not result in any significant change in traffic volumes. **Exhibit 18** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 3 under the buildout conditions.

Under the buildout conditions, both the peak hour and the average daily traffic signal warrants were satisfied for the US 101 southbound and northbound ramp intersections. Refer to Section 8.0 in Deliverable 2 for the complete description and analysis. In addition, the buildout peak hour traffic volumes at the Main Street/Ramada Drive and Main Street/Theatre Drive intersections would satisfy the minimum peak hour volume signal warrant criteria during one or both peak hour periods. Therefore, the four study intersections were assumed to be signalized under the buildout with Alternative 3 conditions.

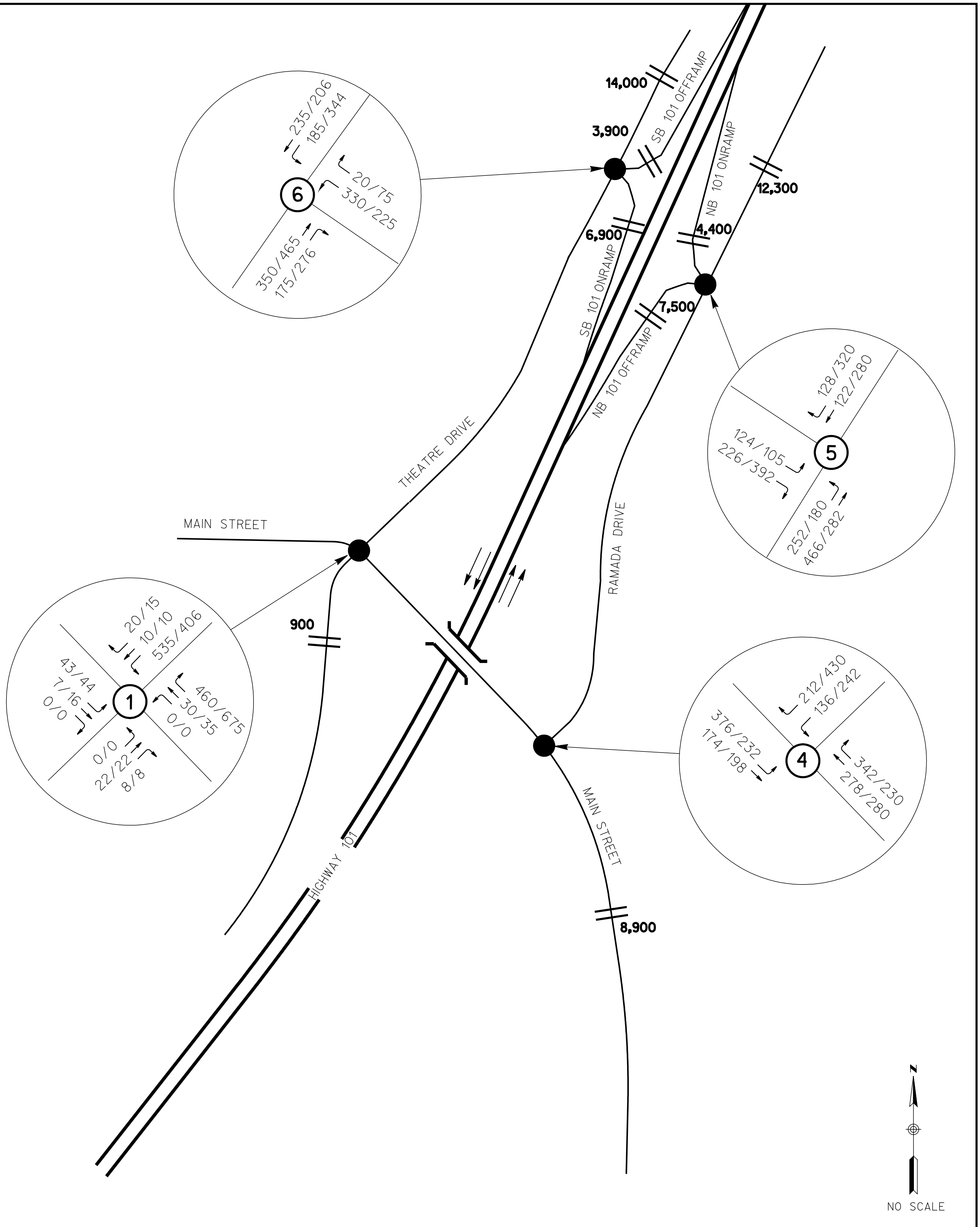
**Exhibit 19** shows the assumed lane geometrics and traffic controls for Alternative 3. It was assumed that the segment of Main Street west of the US 101 southbound ramp intersection would be improved to provide a separate right turn lane for the southbound on ramp traffic. Main Street currently provides approximately 55 feet travel way between the US 101 southbound ramps and Theatre Drive, and therefore, the assumed turn lane, up to Theatre Drive, is anticipated to be accommodated within the existing pavement width without need of widening. Main Street was assumed to have a separate left turn lane on the eastbound approach at Ramada Drive, and Ramada Drive would have a separate left and right turn lane. It should be noted the assumed lane geometrics on Main Street may require widening.

Alternative 4

As previously stated, Alternative 4 includes the removal and relocation of the existing US 101 northbound and southbound ramps. Therefore, the AM and PM peak hour traffic volumes at the study intersections would differ from those under buildout conditions as illustrated on **Exhibit 9**. The buildout peak hour and daily traffic volumes were reassigned based on their origin and destination within the study area, as identified in the actual traffic counts and derived from the County of San Luis Obispo and Templeton Travel Demand Model Update. **Exhibit 20** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 4 under the buildout conditions.

The buildout peak hour traffic volumes shown on **Exhibit 20** would satisfy the minimum peak hour volume and delay signal warrant criteria at the four study intersections during both peak hour periods. Therefore, the four study intersections were assumed to be signalized under the buildout with Alternative 4 conditions.

**Exhibit 21** illustrates the assumed lane geometrics and traffic controls for Alternative 4. It was assumed that the Main Street segment east of the overpass bridge would be improved to provide

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# EXHIBIT 20

BUILDOUT WITH ALTERNATIVE 4
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

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



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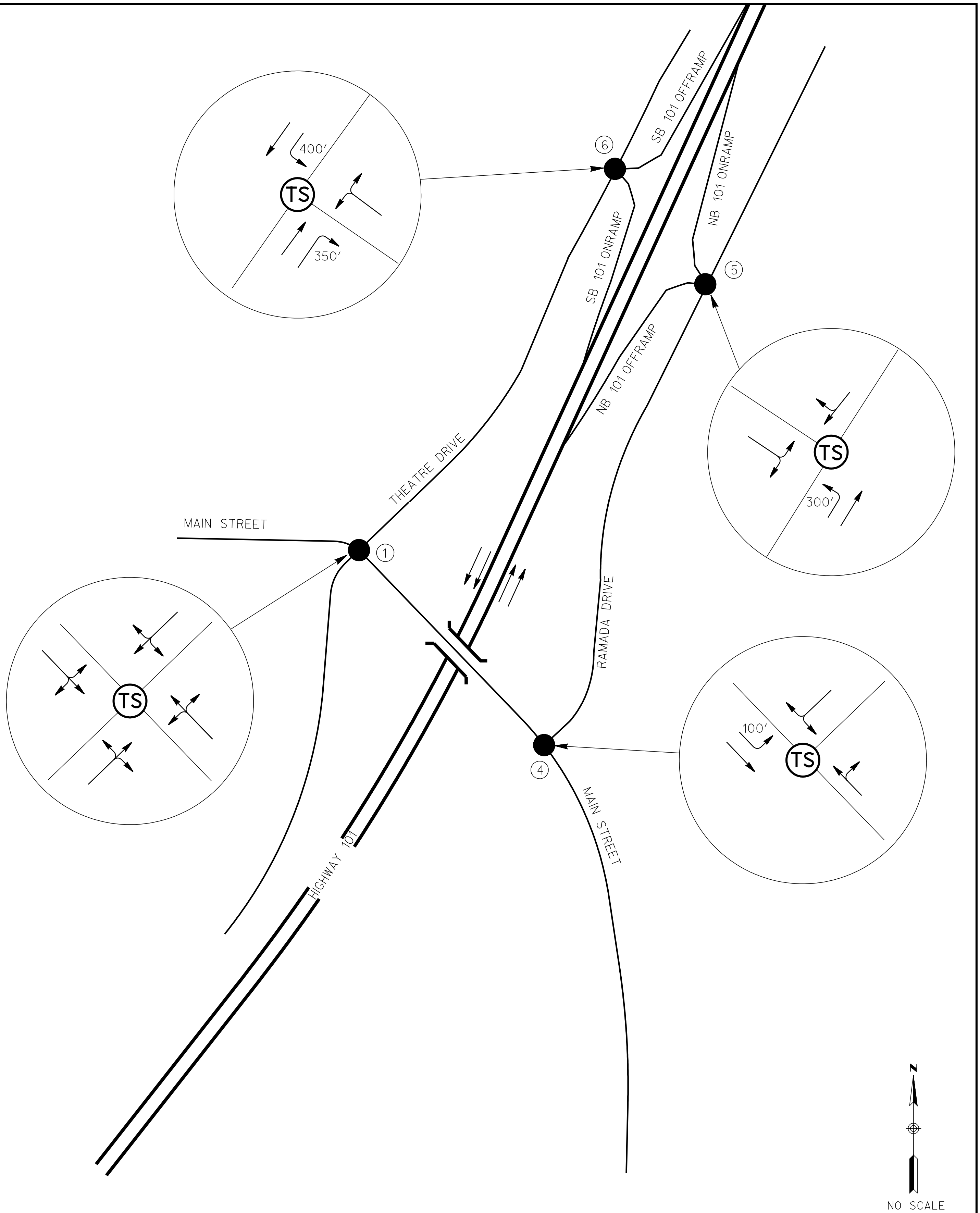


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# EXHIBIT 21

BUILDOUT WITH ALTERNATIVE 4
INTERSECTION LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

● = STUDY INTERSECTION

TS = TRAFFIC SIGNAL

40' = TRAFFIC LANE & STORAGE

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a 100 feet eastbound left turn lane at Ramada Drive. Main Street currently has a width of approximately 45 feet between the US 101 northbound ramps and Ramada Drive, and therefore, the assumed turn lane up to the Ramada Drive is anticipated to be accommodated without significant widening of Main Street. However, Main Street segment immediately east of Ramada Drive may require widening.

Alternative 5

Alternative 5 entails traffic control related improvements without major relocation of roadways, and therefore, peak hour and daily traffic volumes are not anticipated to change significantly as compared to the buildout conditions. **Exhibit 22** illustrates the AM and PM peak hour and daily traffic volumes for Alternative 5 under buildout conditions.

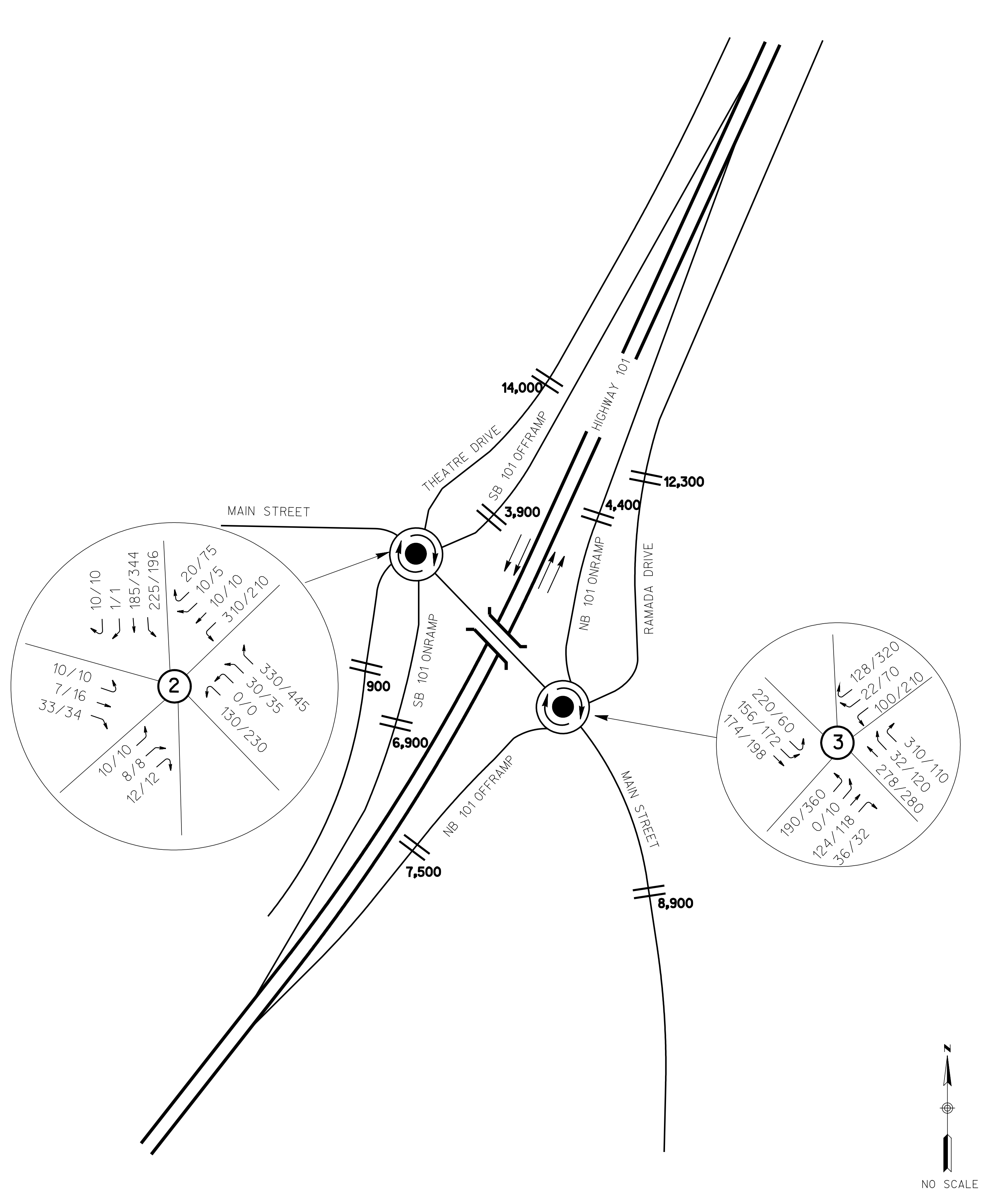
Lane geometrics for the study intersections under the Buildout conditions were assumed to be identical to those under existing conditions. **Exhibit 16** presents lane geometrics and traffic controls for Alternative 5.

## 6.2 Traffic Operations Analysis

Alternative 1

**Table 16** presents the results of the intersection LOS analysis for Alternative 1. Average delays at both ramp intersections are projected to be within acceptable limits (LOS C or better) during both peak hour periods, meeting the Caltrans threshold criteria. The Main Street intersections with Theatre Drive and Ramada Drive are also projected to operate at overall LOS D or better during both peak hours, meeting the County’s LOS criteria.

To analyze vehicular queue lengths, simulations were run using the SimTraffic software within Synchro. **Table 17** summarizes the intersection 95$^{th}$ percentile queues under the buildout with Alternative 1 conditions. **Table 17** also depicts % of time within the peak hour the upstream end of lane would be blocked. It can be surmised that the 95$^{th}$ percentile queues on many individual lanes would cause storage deficiencies and upstream blockage, primarily due to the closely spaced intersections.

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# EXHIBIT 22

BUILDOUT WITH ALTERNATIVE 5
TRAFFIC VOLUMES

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

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



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## TABLE 16
## BUILDOUT CONDITIONS WITH ALTERNATIVE 1 LOS RESULTS

| Study Intersection Main Street at: | Movement | AM Peak | | PM Peak | |
|---|---|---|---|---|---|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| Theatre Drive (TS) | **Average** | 46.7 | D | 50.2 | D |
| | EB LTR | 28.1 | C | 22.9 | C |
| | WB LT | 34.4 | C | 29.2 | C |
| | WB R | **104.2** | **F** | **101.4** | **F** |
| | NB LTR | 2.7 | A | 3.2 | A |
| | SB LTR | 5.6 | A | 9.3 | A |
| US 101 SB Ramps (TS) | **Average** | 15.4 | B | 11.8 | B |
| | EB T | 13.8 | B | 15.7 | B |
| | EB R | 14.7 | B | 18.1 | B |
| | WB L | 25.2 | C | 8.0 | A |
| | WB T | 0.8 | A | 0.7 | A |
| | SB LTR | 28.4 | C | 21.5 | C |
| US 101 NB Ramps (TS) | **Average** | 14.7 | B | 15.5 | C |
| | EB L | 21.1 | C | 9.3 | A |
| | EB T | 0.8 | A | 3.5 | A |
| | WB T | 4.2 | A | 6.1 | A |
| | WB R | 20.2 | C | 18.3 | B |
| | NB LTR | 30.3 | C | 28.6 | C |
| Ramada Drive (TS) | **Average** | 48.3 | D | 47.6 | D |
| | EB L | 19.3 | B | 19.7 | B |
| | EB T | 0.7 | A | 1.0 | A |
| | WB TR | **84.9** | **F** | **108.1** | **F** |
| | SB LTR | 29.8 | C | 27.4 | C |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 17
## BUILDOUT CONDITIONS WITH ALTERNATIVE 1 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | AM Peak | | PM Peak | |
|---|---|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| Theatre Drive (TS) | EB LTR | 500 | 81 | Sufficient | 79 | Sufficient |
| | WB LT | 40 | **51 (11%)** | **Insufficient** | **45 (5%)** | **Insufficient** |
| | WB R | 40 | **67 (17%)** | **Insufficient** | **64 (18%)** | **Insufficient** |
| | NB LTR | 500 | 35 | Sufficient | 33 | Sufficient |
| | SB LTR | 1000 | 316 | Sufficient | 983 | Sufficient |
| US 101 SB Ramps (TS) | EB T | 40 | **90 (47%)** | **Insufficient** | **77 (31%)** | **Insufficient** |
| | EB R | 40 | **74 (17%)** | **Insufficient** | **61 (8%)** | **Insufficient** |
| | WB L | 300 | 95 | Sufficient | 104 | Sufficient |
| | WB T | 335 | 170 | Sufficient | 183 | Sufficient |
| | SB LTR | 1000 | 263 | Sufficient | 213 | Sufficient |
| US 101 NB Ramps (TS) | EB L | 300 | 140 | Sufficient | 48 | Sufficient |
| | EB T | 335 | 211 | Sufficient | 178 | Sufficient |
| | WB T | 40 | **63 (18%)** | **Insufficient** | **78 (33%)** | **Insufficient** |
| | WB R | 40 | **56 (4%)** | **Insufficient** | **73 (17%)** | **Insufficient** |
| | NB LTR | 800 | **1008 (35%)** | **Insufficient** | **914 (16%)** | **Insufficient** |
| Ramada Drive (TS) | EB L | 40 | **69 (53%)** | **Insufficient** | **67 (50%)** | **Insufficient** |
| | EB T | 40 | 33 | Sufficient | 33 | Sufficient |
| | WB TR | 1000 | **1308 (53%)** | **Insufficient** | **1240 (63%)** | **Insufficient** |
| | SB LTR | 1000 | 138 | Sufficient | **1439 (17%)** | **Insufficient** |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 17 indicates that the widened four-lane bridge is anticipated to accommodate the 95$^{th}$ percentile queues without any storage spillbacks or backups. However, excessive queues are reported at four of the "entry" points to the study area (i.e., southbound Theatre Drive, northbound off-ramp, southbound Ramada Drive, westbound Main Street at Ramada Drive, which indicates that the projected traffic demands would be unable to reach the bridge without major delays). In other words, these "entry" locations act as "constraint points". Moreover, starvation (i.e., wasted green time) would be experienced due to downstream blockage or insufficient capacities. The reduction in capacity due to starvation was led to the spillback upstream. If the capacity constraints at four of the "entry" locations are released, an increased level of green time utilization may be achieved at intersections. The 95$^{th}$ percentile queues at the US 101 northbound off-ramp approach are estimated to backup and block the freeway mainline for 35% and 22% of time during the AM and PM peak hour periods, respectively. The eastbound Main Street left turn lane at Ramada Drive would exceed the available storage approximately 50% of time (i.e., every other cycle length).

Alternative 2

**Table 18** presents the results of the intersection LOS analysis for Alternative 2. Average delays at both ramp intersections are projected to be within acceptable limits (LOS C or better) during both peak hour periods, meeting the Caltrans threshold criteria. The Main Street intersections with Theatre Drive and Ramada Drive are also projected to operate at overall LOS D or better during both peak hours, meeting the County's LOS criteria.

**TABLE 18**

**BUILDOUT CONDITIONS WITH ALTERNATIVE 2 LOS RESULTS**

| Intersection Main Street at: | Movement | AM Peak | | PM Peak | |
|---|---|---|---|---|---|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| Theatre Drive (TS) | **Average** | 29.7 | C | 19.1 | B |
| | WB LR | **59.8** | **E** | 30.4 | C |
| | NB TR | 2.8 | A | 2.9 | A |
| | SB LT | 6.3 | A | 10.0 | A |
| US 101 SB Ramps (TS) | **Average** | 18.1 | B | 30.8 | C |
| | EB T | 18.4 | B | 28.3 | C |
| | EB R | **35.2** | **D** | **48.0** | **D** |
| | WB LT | 1.3 | A | 23.8 | C |
| | SB LTR | 19.7 | B | 27.0 | C |
| US 101 NB Ramps (TS) | **Average** | 13.6 | B | 34.4 | C |
| | EB LT | 8.9 | A | 25.8 | C |
| | WB TR | 9.0 | A | 34.1 | C |
| | NB LTR | 27.2 | C | **42.0** | **D** |
| Ramada Drive (TS) | **Average** | 22.1 | C | 26.1 | C |
| | EB L | 29.7 | C | 41.1 | D |
| | EB T | 1.9 | A | 2.7 | A |
| | WB TR | 23.3 | C | 21.4 | C |
| | SB L | 28.7 | C | 42.8 | D |
| | SB R | 26.3 | C | 26.1 | C |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 19** summarizes the intersection 95$^{th}$ percentile queues under the buildout with Alternative 2 conditions. **Table 19** also depicts % of time within the peak hour the upstream end of lane would be blocked.

## TABLE 19
## BUILDOUT CONDITIONS WITH ALTERNATIVE 2 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | AM Peak | | PM Peak | |
|---|---|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| Theatre Drive (TS) | WB LR | 400 | 251 | Sufficient | **425 (3%)** | **Insufficient** |
| | NB TR | 500 | 49 | Sufficient | 48 | Sufficient |
| | SB LT | 1000 | 220 | Sufficient | 357 | Sufficient |
| US 101 SB Ramps (TS) | EB T | 400 | 190 | Sufficient | 353 | Sufficient |
| | EB R | 75 | **120 (8%)** | **Insufficient** | **152 (27%)** | **Insufficient** |
| | WB LT | 300 | 226 | Sufficient | **385 (4%)** | **Insufficient** |
| | SB LTR | 1000 | 259 | Sufficient | 375 | Sufficient |
| US 101 NB Ramps (TS) | EB LT | 300 | 261 | Sufficient | **376 (4%)** | **Insufficient** |
| | WB TR | 400 | 371 | Sufficient | **497 (22%)** | **Insufficient** |
| | NB LTR | 800 | 235 | Sufficient | **1012 (29%)** | **Insufficient** |
| Ramada Drive (TS) | EB L | 300 | 260 | Sufficient | 277 | Sufficient |
| | EB T | 400 | 80 | Sufficient | 70 | Sufficient |
| | WB TR | 1000 | 620 | Sufficient | **1257 (56%)** | **Insufficient** |
| | SB L | 1000 | 123 | Sufficient | **1178 (3%)** | **Insufficient** |
| | SB R | 300 | 93 | Sufficient | **430 (35%)** | **Insufficient** |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 19** demonstrates that the 95th percentile queues on the following lanes would exceed the available or assumed storage:

- Westbound Main Street approach at Theatre Drive - The queues (425 feet) are reported to exceed the estimated storage (400 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.

- Eastbound Main Street right turn lane at the US 101 southbound ramps - The queues (152 feet) are reported to exceed a 75 feet right-turn for 27% of time during the PM peak hour. However, the queues (353 feet) in the adjacent through lane would be accommodated within the assumed storage of 400 feet. This spillback is considered minor and is not anticipated to degrade the overall operations of ramp intersections.

- Westbound Main Street approach at the US 101 southbound ramps - The queues (385 feet) are reported to exceed assumed storage (335 feet) for only 4% of time during the PM peak hour. The primary reason for this spillback is a shared left and through lane, and associated signal phasing. However, an existing 15 feet wide lane at this approach will encourage through traffic to sneak past the waiting left-turn traffic (waiting for gaps or a protected phase). Synchro cannot model this operation accurately. Based on anticipated traffic operations behavior, this spillback would not be experienced during the PM peak hour.

- Eastbound Main Street approach at the US 101 northbound ramps - The queues (376 feet) are reported to exceed assumed storage (335 feet) for only 4% of time during the PM peak hour. The primary reason for this spillback is a shared left and through lane, and associated signal phasing. However, an existing 15 feet wide lane at this approach will encourage through traffic to sneak past the waiting left-turn traffic (waiting for gaps or a protected phase). Synchro cannot model this operation accurately. Based on anticipated traffic operations behavior, this spillback would not be experienced during the PM peak hour.

- Westbound Main Street approach at the US 101 northbound ramps - The queues (497 feet) are reported to exceed assumed storage (400 feet) for 22% of time during the PM peak hour. A single lane westbound Main Street between the US 101 northbound ramps and Ramada Drive is projected to serve nearly 800 vehicles during PM peak hour. Further, the downstream storage spillback results in inefficient use of green time at this approach. The combined effect of insufficient capacity and downstream congestion trigger longer queue backups on this approach.

- US 101 northbound off-ramp at Main Street - The 95$^{th}$ percentile queues on this approach are estimated to backup and extend onto the freeway mainline for 29% of time during the PM peak hour.

- Westbound Main Street approach at Ramada Drive - The longer queues are reported primarily due to congested downstream intersections at the US 101 northbound and southbound ramps.

- Southbound Ramada Drive approach at Main Street - The longer queues are reported as a result of congestion in the westbound direction on Main Street at the US 101 ramp intersections.

Alternative 3

**Table 20** presents the results of the intersection LOS analysis for Alternative 3. Average delays at both ramp intersections are projected to be within acceptable limits (LOS C or better) during both peak hour periods, meeting the Caltrans threshold criteria. The Main Street intersections with Theatre Drive and Ramada Drive are also projected to operate at overall LOS D or better during both peak hours, meeting the County's LOS criteria.

**TABLE 20**

**BUILDOUT CONDITIONS WITH ALTERNATIVE 3 LOS RESULTS**

| Study Intersection Main Street at: | Movement | AM Peak | | PM Peak | |
|---|---|---|---|---|---|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| Theatre Drive (TS) | **Average** | 25.9 | C | 21.9 | C |
| | WB LR | 51.3 | D | 36.4 | D |
| | NB TR | 2.8 | A | 2.9 | A |
| | SB LT | 6.3 | A | 10.0 | A |
| US 101 SB Ramps (TS) | **Average** | 18.5 | B | 19.8 | B |
| | EB T | 18.4 | B | 15.4 | B |
| | EB R | 34.0 | C | **20.9** | **D** |
| | WB LT | 10.5 | B | 17.5 | B |
| | SB LTR | 19.7 | B | 27.0 | C |
| US 101 NB Ramps (TS) | **Average** | 14.1 | B | 32.3 | C |
| | EB LT | 9.1 | A | 24.2 | C |
| | WB TR | 10.1 | B | 27.4 | C |
| | NB LTR | 27.2 | C | **46.4** | **D** |
| Ramada Drive (TS) | **Average** | 22.2 | C | 25.7 | C |
| | EB L | 30.3 | C | 37.7 | D |
| | EB T | 2.0 | A | 3.4 | A |
| | WB TR | 23.3 | C | 21.4 | C |
| | SB L | 28.7 | C | 42.8 | D |
| | SB R | 26.3 | C | 26.1 | C |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 21** summarizes the intersection 95$^{th}$ percentile queues under the buildout with Alternative 3 conditions. **Table 21** also depicts **Table 17** also depicts % of time within the peak hour the upstream end of lane would be blocked.

## TABLE 21
## BUILDOUT CONDITIONS WITH ALTERNATIVE 3 QUEUE RESULTS

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | AM Peak | | PM Peak | |
|---|---|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| Theatre Drive (TS) | WB LR | 500 | 284 | Sufficient | 479 | Sufficient |
| | NB TR | 500 | 48 | Sufficient | 52 | Sufficient |
| | SB LT | 1000 | 208 | Sufficient | 367 | Sufficient |
| US 101 SB Ramps (TS) | EB T | 500 | 185 | Sufficient | 387 | Sufficient |
| | EB R | 75 | **106 (6%)** | **Insufficient** | **147 (16%)** | **Insufficient** |
| | WB LT | 470 | 262 | Sufficient | **530 (3%)** | **Insufficient** |
| | SB LTR | 1000 | 249 | Sufficient | 214 | Sufficient |
| US 101 NB Ramps (TS) | EB LT | 470 | 198 | Sufficient | 470 | Sufficient |
| | WB TR | 500 | 363 | Sufficient | **620 (18%)** | **Insufficient** |
| | NB LTR | 800 | 235 | Sufficient | **1038 (42%)** | **Insufficient** |
| Ramada Drive (TS) | EB L | 300 | 257 | Sufficient | 293 | Sufficient |
| | EB T | 500 | 65 | Sufficient | 132 | Sufficient |
| | WB TR | 1000 | 630 | Sufficient | **1207 (34%)** | **Insufficient** |
| | SB L | 1000 | 123 | Sufficient | **1158 (3%)** | **Insufficient** |
| | SB R | 300 | 98 | Sufficient | **432 (36%)** | **Insufficient** |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 21** indicates that the 95$^{th}$ percentile queues on the following lanes would exceed the available or assumed storage:

- Eastbound Main Street right-turn lane at the US 101 southbound ramps - The queues (147 feet) are reported to exceed the 75 feet right turn lane storage for 16% of time during the PM peak hour. However, the queues (387 feet) in the adjacent through lane would be accommodated within the estimated storage of 500 feet. This spillback is considered very minor and is not anticipated to degrade the overall operations of ramp intersections.

- Westbound Main Street approach at the US 101 southbound ramps - The queues (530 feet) are reported to exceed the assumed storage (470 feet) for only 3% of time during the PM peak hour. The primary reason for this spillback is a shared left and through lane, and associated signal phasing. However, an existing 15 feet wide lane at this approach will encourage through traffic to sneak past the waiting left-turn traffic (waiting for a gap in opposite direction traffic or a protected phase). Synchro cannot model this operation accurately. Based on the anticipated traffic operations behavior, this spillback would not be experienced during the PM peak hour.

- Westbound Main Street approach at the US 101 northbound ramps - The queues (620 feet) are reported to exceed the assumed storage (500 feet) for 18% of time during the PM peak hour. A single lane westbound Main Street between the US 101 northbound ramps and Ramada Drive is projected to serve nearly 800 vehicles during the PM peak hour. Further, the downstream storage spillback results in inefficient use of green time at this approach. The combined effect of insufficient capacity and downstream congestion trigger longer queue backups on this approach.

- US 101 northbound off-ramp at Main Street - The 95$^{th}$ percentile queues on this approach are estimated to backup and extend onto the freeway mainline for 42% of time during the PM peak hour.

- Westbound Main Street approach at Ramada Drive - The longer queues are reported primarily due to congested downstream intersections at the US 101 northbound and southbound ramps.

- Southbound Ramada Drive approach at Main Street - The longer queues reported as a result of congestion in the westbound direction on Main Street at the US 101 ramp intersections.

Alternative 4

**Table 22** presents the results of the intersection LOS analysis for Alternative 3. Average delays at both ramp intersections are projected to be within acceptable limits (LOS C or better) during both peak hour periods, meeting the Caltrans threshold criteria. The Main Street intersections with Theatre Drive and Ramada Drive are also projected to operate at overall LOS D or better during both peak hours, meeting the County's LOS criteria.

**TABLE 22**

**BUILDOUT CONDITIONS WITH ALTERNATIVE 4 LOS RESULTS**

| Study Intersection | Movement | AM Peak | | PM Peak | |
|---|---|---|---|---|---|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| Main Street & Theatre Drive (TS) | **Average** | 21.4 | C | 28.3 | C |
| | EB LTR | 46.3 | D | 47.4 | D |
| | WB LTR | 33.5 | C | 39.4 | D |
| | NB LTR | 3.4 | A | 4.4 | A |
| | SB LTR | 9.7 | A | 9.1 | A |
| Main Street & Ramada Drive (TS) | **Average** | 41.6 | D | 54.5 | D |
| | EB L | 51.1 | D | **64.6** | **E** |
| | EB T | 1.6 | A | 3.3 | A |
| | WB TR | 41.7 | D | **60.7** | **E** |
| | SB LR | 51.1 | D | **61.4** | **E** |
| Ramada Drive & US 101 NB Ramps (TS) | **Average** | 14.4 | B | 26.2 | C |
| | EB LR | 18.9 | B | 28.5 | C |
| | NB L | 20.1 | C | **37.6** | **D** |
| | NB T | 6.6 | A | 5.4 | A |
| | SB TR | 17.1 | B | 30.6 | C |
| Theatre Drive & US 101 SB Ramps (TS) | **Average** | 18.0 | B | 27.2 | C |
| | WB LR | 23.5 | C | 31.8 | C |
| | NB T | 19.4 | B | 32.6 | C |
| | NB R | 12.8 | B | 15.3 | B |
| | SB L | 25.2 | C | **39.0** | **D** |
| | SB T | 5.9 | A | 4.7 | A |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 23** summarizes the intersection 95$^{th}$ percentile queues under the buildout Conditions with Alternative 4. **Table 23** also depicts % of time within the peak hour the upstream end of lane would be blocked.

## TABLE 23
## BUILDOUT CONDITIONS WITH ALTERNATIVE 4 QUEUE RESULTS

| Study Intersection | Movement | Assumed Storage Length (feet) | AM Peak | | PM Peak | |
|---|---|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| Main Street & Theatre Drive (TS) | EB LTR | 500 | 142 | Sufficient | 163 | Sufficient |
| | WB LTR | 600 | 233 | Sufficient | 296 | Sufficient |
| | NB LTR | 500 | 35 | Sufficient | 29 | Sufficient |
| | SB LTR | 1100 | 991 | Sufficient | 517 | Sufficient |
| Main Street & Ramada Drive (TS) | EB L | 100 | **153 (66%)** | **Insufficient** | **170 (55%)** | **Insufficient** |
| | EB T | 600 | **750 (14%)** | **Insufficient** | 576 | Sufficient |
| | WB TR | 1000 | **1261 (19%)** | **Insufficient** | **1208 (18%)** | **Insufficient** |
| | SB LR | 1000 | 479 | Sufficient | **1256 (11%)** | **Insufficient** |
| Ramada Drive & US 101 NB Ramps (TS) | EB LR | 800 | 271 | Sufficient | **921 (22%)** | **Insufficient** |
| | NB L | 300 | 171 | Sufficient | 203 | Sufficient |
| | NB T | 1000 | 143 | Sufficient | 107 | Sufficient |
| | SB TR | 700 | 162 | Sufficient | **1488 (31%)** | **Insufficient** |
| Theatre Drive & US 101 SB Ramps (TS) | WB LR | 800 | 386 | Sufficient | 348 | Sufficient |
| | NB T | 1100 | 165 | Sufficient | 307 | Sufficient |
| | NB R | 350 | 89 | Sufficient | 149 | Sufficient |
| | SB L | 400 | 175 | Sufficient | 346 | Sufficient |
| | SB T | 1000 | 100 | Sufficient | 163 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 23** demonstrates that the 95$^{th}$ percentile queues at the following individual lanes exceeded the available or estimated storage:

- Eastbound Main Street approach at Ramada Drive - The left turn lane queues (153 feet) are reported to exceed the assumed storage and block the adjacent through lane for 66% of time during the AM peak hour. As a consequence, the through queues (750 feet) are also reported to exceed the available storage (600 feet) for 14% of time during the AM peak hour. This intersection is projected to experience heavy peak hour traffic volumes for all conflicting movements which would result in insufficient allocation of green times.

- Westbound Main Street approach at Ramada Drive - The longer queues were reported primarily due to the same reasons mentioned above for the eastbound approach.

- Southbound Ramada Drive approach at Main Street - The queues were reported to extend and spillback into the upstream intersection of the US 101 northbound ramps for 11% of time during the PM peak hour due to the same reasons mentioned above for the eastbound and westbound approaches.

- US 101 northbound off-ramps at Ramada Drive - The 95th percentile queues on this approach are estimated to backup and extend onto the freeway mainline for 22% of time during the PM peak hour. The congestion on southbound Ramada Drive results in inefficient use of green time on this approach.

- Southbound Ramada Drive approach at US 101 northbound ramps - The longer queues persisted at this approach due to downstream congestion.

Alternative 5

**Table 24** presents the results of the roundabout LOS analysis for Alternative 5. The 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. However, the US 101 northbound ramps/Main Street/Ramada Drive roundabout is estimated to operate with deficient overall LOS E and LOS F during the AM and PM peak hours respectively, thus exceeding the Caltrans and County's threshold criteria.

**TABLE 24**

**BUILDOUT CONDITIONS WITH ALTERNATIVE 5 LOS RESULTS**

| Study Intersection Main Street at: | Movement | AM Peak | | PM Peak | |
|---|---|---|---|---|---|
| | | Avg. Delay | LOS | Avg. Delay | LOS |
| US 101 SB Ramps & Theatre Drive (RAB) | **Average** | 10.1 | B | 14.3 | B |
| | WB LTR | 6.4 | A | 9.1 | A |
| | SB LTR | 11.6 | B | 15.5 | C |
| | SE LTR | 13.5 | B | 21.1 | C |
| | EB LTR | 8.2 | A | 10.1 | B |
| | NW LTR | 8.1 | A | 9.6 | A |
| US 101 NB Ramps & Ramada Drive (RAB) | **Average** | **35.6** | **E** | **58.6** | **F** |
| | NB LTR | 15.5 | C | **27.6** | **D** |
| | WB LTR | **80.7** | **F** | **43.1** | **E** |
| | SB LTR | 12.3 | B | **135.4** | **F** |
| | EB LTR | 8.1 | A | 7.5 | A |

LOS = Level of Service; Average Delay in seconds/vehicle
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 25** summarizes the roundabout queuing analysis results under the buildout conditions with Alternative 5. The 95th percentile queues were estimated to be accommodated within the available or estimated storage at all movements, except for the southbound Ramada Drive approach at the Main Street/US 101 northbound ramps, where excessive queues are reported.

**TABLE 25**
**BUILDOUT CONDITIONS WITH ALTERNATIVE 5 QUEUE RESULTS**

| Study Intersection Main Street at: | Movement | Assumed Storage Length (feet) | AM Peak | | PM Peak | |
|---|---|---|---|---|---|---|
| | | | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient | 95th Percentile Queue Length (feet) | Storage Length Sufficient / Insufficient |
| US 101 SB Ramps & Theatre Drive (RAB) | | | | | | |
| | WB LTR | 300 | 66 | Sufficient | 130 | Sufficient |
| | SB LTR | 800 | 74 | Sufficient | 79 | Sufficient |
| | SE LTR | 1000 | 101 | Sufficient | 190 | Sufficient |
| | EB LTR | 500 | 25 | Sufficient | 25 | Sufficient |
| | NW LTR | 500 | 25 | Sufficient | 25 | Sufficient |
| US 101 NB Ramps & Ramada Drive (RAB) | | | | | | |
| | NB LTR | 800 | 92 | Sufficient | 213 | Sufficient |
| | WB LTR | 1000 | 771 | Sufficient | 297 | Sufficient |
| | SB LTR | 1000 | 56 | Sufficient | **1219** | **Insufficient** |
| | EB LTR | 300 | 86 | Sufficient | 63 | Sufficient |

Storage length based on measured or estimated clear distance between intersections or turning bay
TWSC = Two-Way Stop Control, TS = Traffic Signal, RAB = Roundabout
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 LOS and queuing analysis calculation worksheets are provided in **Appendix D**.

## 7.0 INTERSECTION ILV ANALYSIS

The buildout peak hour volumes were used to perform an Intersection Lane Vehicles (ILV) capacity analysis at the ramp intersections. **Table 26** presents the results of the ILV analysis. **Appendix E** contains the ILV method calculation sheets.

**TABLE 26**

**BUILDOUT CONDITIONS INTERSECTION ILV ANLAYSIS RESULTS**

| Study Intersection | Peak Hour | Buildout Alternative 1 | |
|---|---|---|---|
| | | ILV per hour | Capacity |
| Main Street / 101 NB Ramps | AM | 1,030 | Under |
| | PM | 1,370 | **At** |
| Main Street / 101 SB Ramps | AM | 950 | Under |
| | PM | 1,140 | Under |
| | | | |
| Study Intersection | Peak Hour | Buildout Alternative 2 | |
| | | ILV per hour | Capacity |
| Main Street / 101 NB Ramps | AM | 1,030 | Under |
| | PM | 1,370 | **At** |
| Main Street / 101 SB Ramps | AM | 950 | Under |
| | PM | 1,140 | Under |
| | | | |
| Study Intersection | Peak Hour | Buildout Alternative 3 | |
| | | ILV per hour | Capacity |
| Main Street / 101 NB Ramps | AM | 1,030 | Under |
| | PM | 1,370 | **At** |
| Main Street / 101 SB Ramps | AM | 950 | Under |
| | PM | 1,140 | Under |
| | | | |
| Study Intersection | Peak Hour | Buildout Alternative 4 | |
| | | ILV per hour | Capacity |
| Ramada Drive / 101 NB Ramps | AM | 1,068 | Under |
| | PM | 1,277 | **At** |
| Theatre Drive / 101 SB Ramps | AM | 1,060 | Under |
| | PM | 1,385 | **At** |

The data in **Table 26** indicates that under Alternative 1 through 3, the Main Street/US 101 northbound ramps intersection is expected to have an ILV/hr close to 1,400 during the PM peak hour, which is considered to be approaching “unstable flow” conditions or operating at capacity. Under alternative 4, both US 101 northbound and southbound ramp intersections are estimated to have an ILV/hr in the range of 1,300-1,400 during the PM peak hour, which is considered to be approaching “unstable flow” conditions or operating at capacity.

## 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 27** presents a summary of the LOS analysis for the existing "no build" and each alternative improvement (refer to Tables 3, 5, 7, 9, 11 and 13).

**TABLE 27**

**EXISTING CONDITIONS LOS SUMMARY**

| ID | Study Intersection Main Street at: | Peak Hour | Vehicle Delay - LOS Value | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | No Build | Alt. 1 | Alt. 2 | Alt. 3 | Alt. 4 | Alt. 5 |
| 1 | Theatre Drive | AM | 12.4 - B | 10.8 - B | 13.4 - B | 13.4 - B | 15.1 - C | See ID 2 |
| | | PM | 14.5 - B | 11.4 - B | 15.7 - C | 15.7 - C | 15.0 - B | See ID 2 |
| 2 | US 101 SB Ramps | AM | 24.1 - C | 20.3 - C | 20.3 - C | 20.3 - C | NA | 6.1 - A |
| | | PM | **35.5 - E** | **27.3 - D** | **27.3 - D** | 27.3 - D | NA | 6.5 - A |
| 3 | US 101 NB Ramps | AM | 16.1 - C | 15.4 - C | 16.1 - C | 16.1 - C | NA | 6.5 - A |
| | | PM | **26.4 - D** | 19.8 - C | **26.4 - D** | **26.4 - D** | NA | 7.2 - A |
| 4 | Ramada Drive | AM | 12.8 - B | 12.9 - B | 14.0 - B | 14.0 - B | 16.6 - C | See ID 3 |
| | | PM | 14.8 - B | 14.8 - B | 19.9 - C | 19.9 - C | 20.8 - C | See ID 3 |
| 5 | Ramada Drive & US 101 NB Ramps | AM | NA | NA | NA | NA | 12.5 - B | NA |
| | | PM | NA | NA | NA | NA | 14.0 - B | NA |
| 6 | Theatre Drive & US 101 SB Ramps | AM | NA | NA | NA | NA | 21.7 - C | NA |
| | | PM | NA | NA | NA | NA | **28.0 - D** | NA |

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 27 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 1.

Based on the analysis of the 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 roundabout (Alterative 5). **Table 28** presents a summary of the LOS analysis for the buildout "no build" and each alternative improvement (refer to Tables 3, 16, 18, 20, 22 and 24).

**TABLE 28**
**BUILDOUT CONDITIONS LOS SUMMARY**

| ID | Study Intersection Main Street at: | Peak Hour | Vehicle Delay - LOS Value | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | No Build | Alt. 1 | Alt. 2 | Alt. 3 | Alt. 4 | Alt. 5 |
| 1 | Theatre Drive | AM | 29.8 - D | 46.7 - D | 29.7 - C | 25.9 - C | 21.4 - C | See ID 2 |
| | | PM | > **50 - F** | 50.2 - D | 19.1 - B | 21.9 - C | 28.3 - C | See ID 2 |
| 2 | US 101 SB Ramps | AM | > **50 - F** | 15.4 - B | 18.1 - B | 18.5 - B | NA | 10.1 – B |
| | | PM | > **50 - F** | 11.8 - B | 30.8 - C | 19.8 - B | NA | 14.3 – B |
| 3 | US 101 NB Ramps | AM | > **50 - F** | 14.7 - B | 13.6 - B | 14.1 - B | NA | **35.6 – E** |
| | | PM | > **50 - F** | 15.5 - C | 34.4 - C | 32.3 - C | NA | **58.6 – F** |
| 4 | Ramada Drive | AM | > **50 - F** | 48.3 - D | 22.1 - C | 22.2 - C | 41.6 - D | See ID 3 |
| | | PM | > **50 - F** | 47.6 - D | 26.1 - C | 25.7 - C | 54.5 - D | See ID 3 |
| 5 | Ramada Drive & US 101 NB Ramps | AM | NA | NA | NA | NA | 14.4 - B | NA |
| | | PM | NA | NA | NA | NA | 26.2 - C | NA |
| 6 | Theatre Drive & US 101 SB Ramps | AM | NA | NA | NA | NA | 18.0 - B | NA |
| | | PM | NA | NA | NA | NA | 27.2 - C | NA |

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 1

Based on the buildout "static" LOS analysis results, the study intersections are projected to operate within acceptable overall LOS with a few critical movements experiencing delays in the range of LOS E-F. Contrary to the overall acceptable conditions in "static" LOS analyses, the SimTraffic simulation analysis indicates that excessive queues would occur at various locations resulting in severe congestion at four of the "entry" points (listed in Section 6.2). This is primarily due to the downstream spillbacks and closely spaced intersections. As mentioned earlier, the "static" analyses do not explicitly address operations at the closely spaced signalized intersections, and therefore, acceptable LOS grades using the "static" technique can represent more optimistic operations. RICK believes that the simulation analysis, in this case, would represent more accurate and realistic operations at the study intersections.

Based on simulation observations and reported collisions (refer Deliverable 1), this interchange alternative is considered "fatally flawed" from an operational perspective. If this alternative continues to be a viable candidate for consideration without further improvements, it is recommended that a more sophisticated simulation package (i.e. VISSIM) be used that can address queue spillback conditions resulting from closely spaced intersections.

In order to improve traffic flows within the study area, it is imperative to release capacity related constraints at four of the "entry" points, especially at the easterly cluster. Therefore, the following mitigations are recommended to achieve acceptable queuing conditions in simulation analysis:

1. Add a dedicated right-turn lane with 300 feet storage on the US 101 northbound off-ramp approach at Main Street.
2. Add a dedicated right-turn lane with 300 feet storage on the southbound Ramada Drive approach at Main Street.
3. Add a second through lane with 300 feet storage on the westbound Main Street approach at Ramada Drive.

**Exhibit 23** depicts the recommended lane configurations for this alternative. It should be noted that above recommendations were simply based on traffic operations perspective, and do not take account of any right of way and design related limitations.

## 8.2 Alternative 2

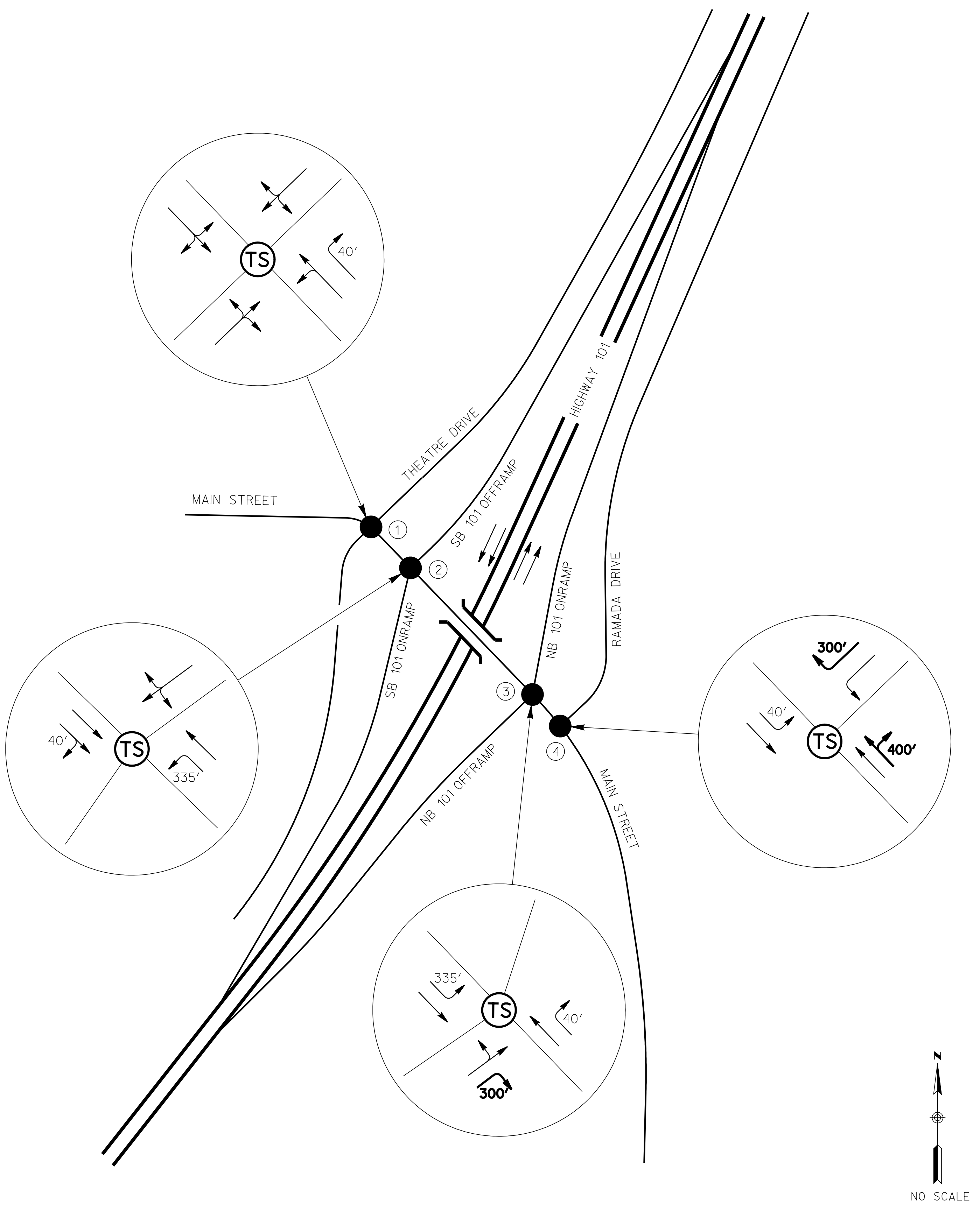
Under the buildout conditions, the "static" LOS analysis results as presented earlier show that all study intersections operating at acceptable overall LOS during both peak hours. Based on the SimTraffic simulation, some locations were identified experiencing queuing issues during the PM peak hour (listed in Section 6.2). The queue spillbacks are expected to decrease for Alternative 2 compared to those of Alternative 1 due to increased spacing between the study intersections along Main Street. However, similar to the Alternative 1, it is imperative to release capacity constraints in order to improve queue discharge capabilities and to serve full peak hour demand at key locations. The following mitigations are recommended to achieve acceptable queuing conditions in simulation analysis:

1. Add a dedicated right-turn lane on the westbound Main Street approach at the US 101 northbound ramps. The recommended right-turn lane should be extended for length of the Main Street segment from the US 101 northbound ramps to Ramada Drive, creating a four-lane cross section.
2. Add a second through lane with 300 feet storage on the westbound Main Street approach at Ramada Drive.

**Exhibit 24** depicts the recommended lane configurations for this alternative. The alternative mitigation strategy would be to widen the Main Street bridge to accommodate left-turn lanes, which will facilitate more flexibility with signal timings in terms of phasing allocation. The alternative mitigation strategy would also require widening Main Street between the US 101 northbound ramps and Ramada Drive as indicated above. 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.

## 8.3 Alternative 3

Under the buildout conditions, the "static" LOS analysis results as presented earlier show that all study intersections would operate at acceptable overall LOS during both peak hours. Similar to Alternative 3, some locations were identified experiencing queuing issues based on simulation analysis (listed in Section 6.2). However, level of congestion is anticipated to decrease for Alternative 3 as compared to Alternative 2 since the spacing between intersections has increased. However, similar to the Alternative 2, it is imperative to release capacity constraints in order to

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# EXHIBIT 23

BUILDOUT WITH ALTERNATIVE 1
RECOMMENDED LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- = STUDY INTERSECTION
- TS = TRAFFIC SIGNAL
- 40' = TRAFFIC LANE & STORAGE
- 40' = REC. TRAFFIC LANE & STORAGE



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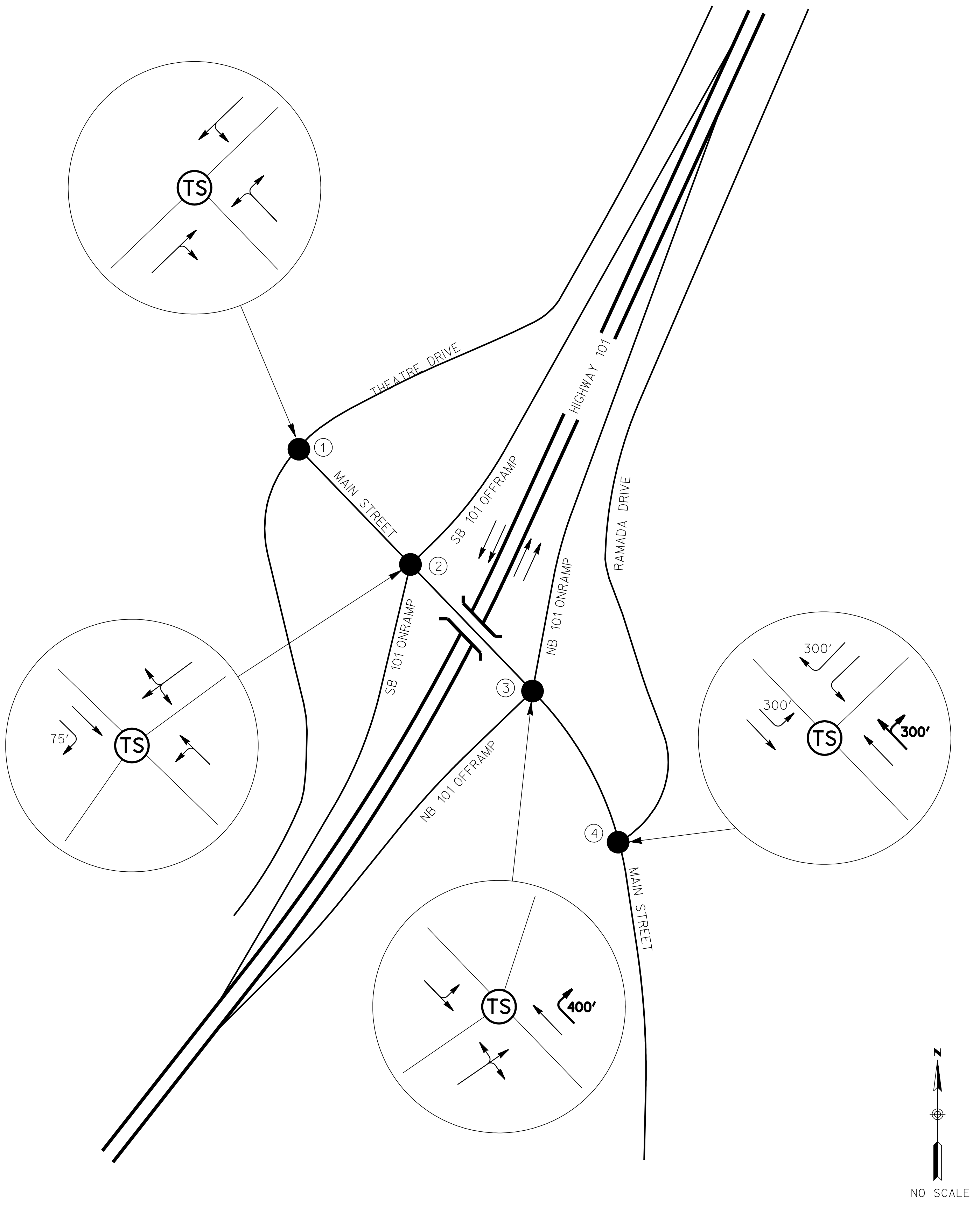


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# EXHIBIT 24

BUILDOUT WITH ALTERNATIVE 2
RECOMMENDED LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- = STUDY INTERSECTION
- TS = TRAFFIC SIGNAL
- 40' = TRAFFIC LANE & STORAGE
- 40' = REC. TRAFFIC LANE & STORAGE



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improve queue discharge capabilities and to serve full peak hour demand at some locations. The following mitigations are recommended to achieve acceptable queuing conditions in simulation analysis:

1. Add a dedicated right-turn lane on the westbound Main Street approach at the US 101 northbound ramps. The recommended right-turn lane should be extended for length of the Main Street segment from the US 101 northbound ramps to Ramada Drive, creating a four-lane cross section.
2. Add a shared through/right-turn lane with 300 feet storage on the westbound Main Street approach at Ramada Drive.

**Exhibit 25** depicts the recommended lane configurations for this alternative. The alternative mitigation strategy would be to widen the Main Street bridge to accommodate left-turn lanes, which will facilitate more flexibility with signal timings in terms of phasing allocation. The alternative mitigation strategy would also require widening Main Street segment between the US 101 northbound ramps and Ramada Drive as indicated above. In summary, Alternative 3 is anticipated to operationally perform similar to Alternative 2 with the same recommended mitigations. The only additional benefit that Alternative 3 offers is the increased spacing between intersections. It should be noted that above recommendations were simply based on traffic operations perspective, and do not take account of any right of way and design related limitations.

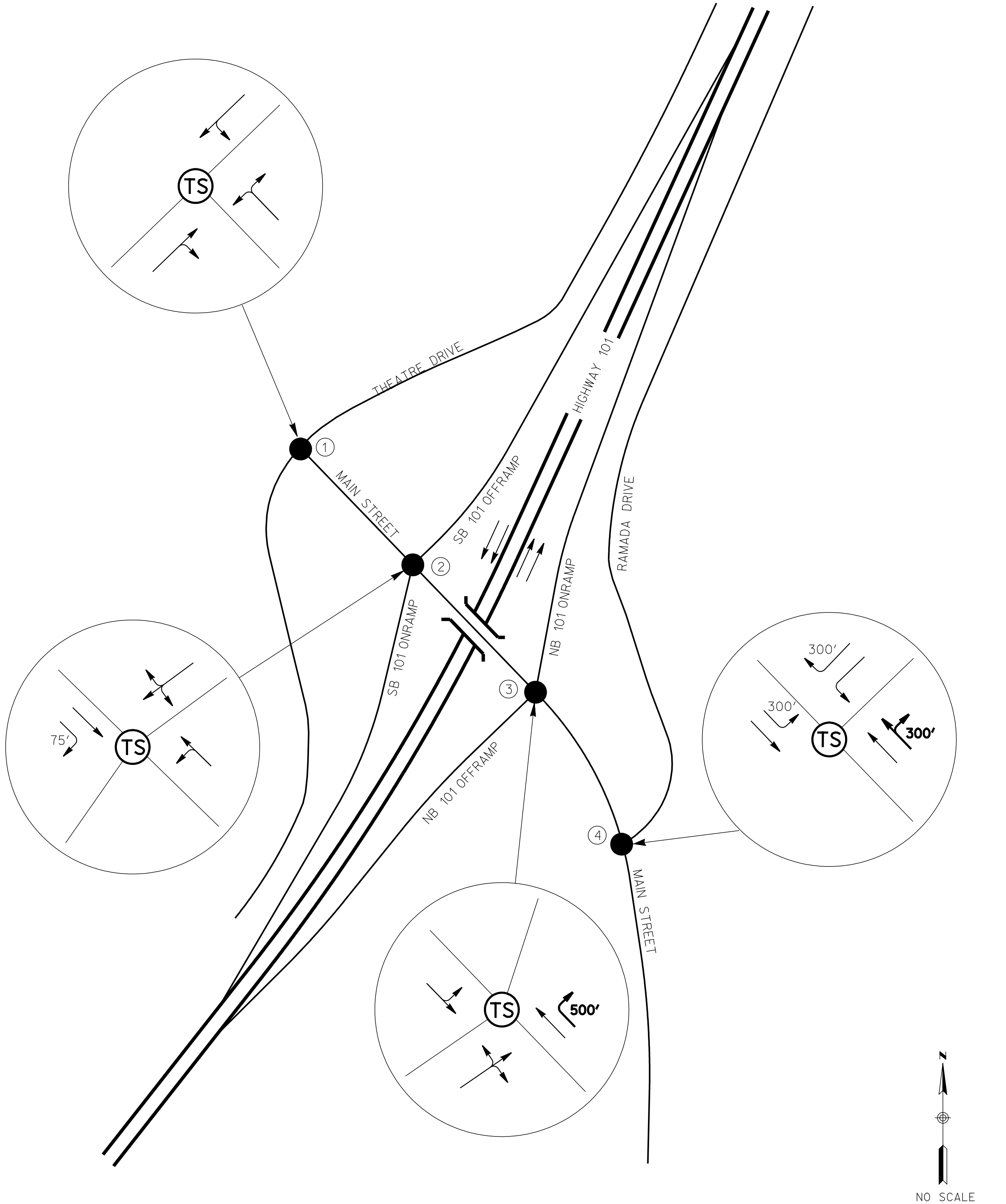
## 8.4 Alternative 4

This Alternative partially eliminates traffic operational difficulties at the US 101 / Main Street Interchange resulting from the closely spaced intersections. Relative to other alternatives, this alternative is anticipated to experience regulated traffic flow because of the separation between the study intersections.

Under the buildout conditions, the "static" LOS analysis results as presented earlier show that the study intersections would operate at acceptable overall LOS during both peak hours. However, some individual movements would experience delays within the LOS E range during the PM peak hour. Contrary to the overall acceptable conditions in "static" LOS analyses, the SimTraffic simulation analysis indicates that excessive queues would be experienced at some locations (listed in Section 6.2) resulting in moderate congestion on the southbound Ramada Drive, US 101 northbound off-ramp and eastbound Main Street. This would be primarily due to the downstream spillbacks and insufficient green times. The following improvement is recommended to release the capacity constraint and to serve the full peak hour demand:

1. Add a dedicated right-turn lane with 400 feet storage on the southbound Ramada Drive approach at Main Street.

**Exhibit 26** depicts the recommended lane configurations for this alternative. It should be noted that above recommendations were simply based on traffic operations perspective, and do not take account of any right of way and design related limitations.

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# EXHIBIT 25

BUILDOUT WITH ALTERNATIVE 3
RECOMMENDED LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- ● = STUDY INTERSECTION
- TS = TRAFFIC SIGNAL
- 40' = TRAFFIC LANE & STORAGE
- **40'** = REC. TRAFFIC LANE & STORAGE



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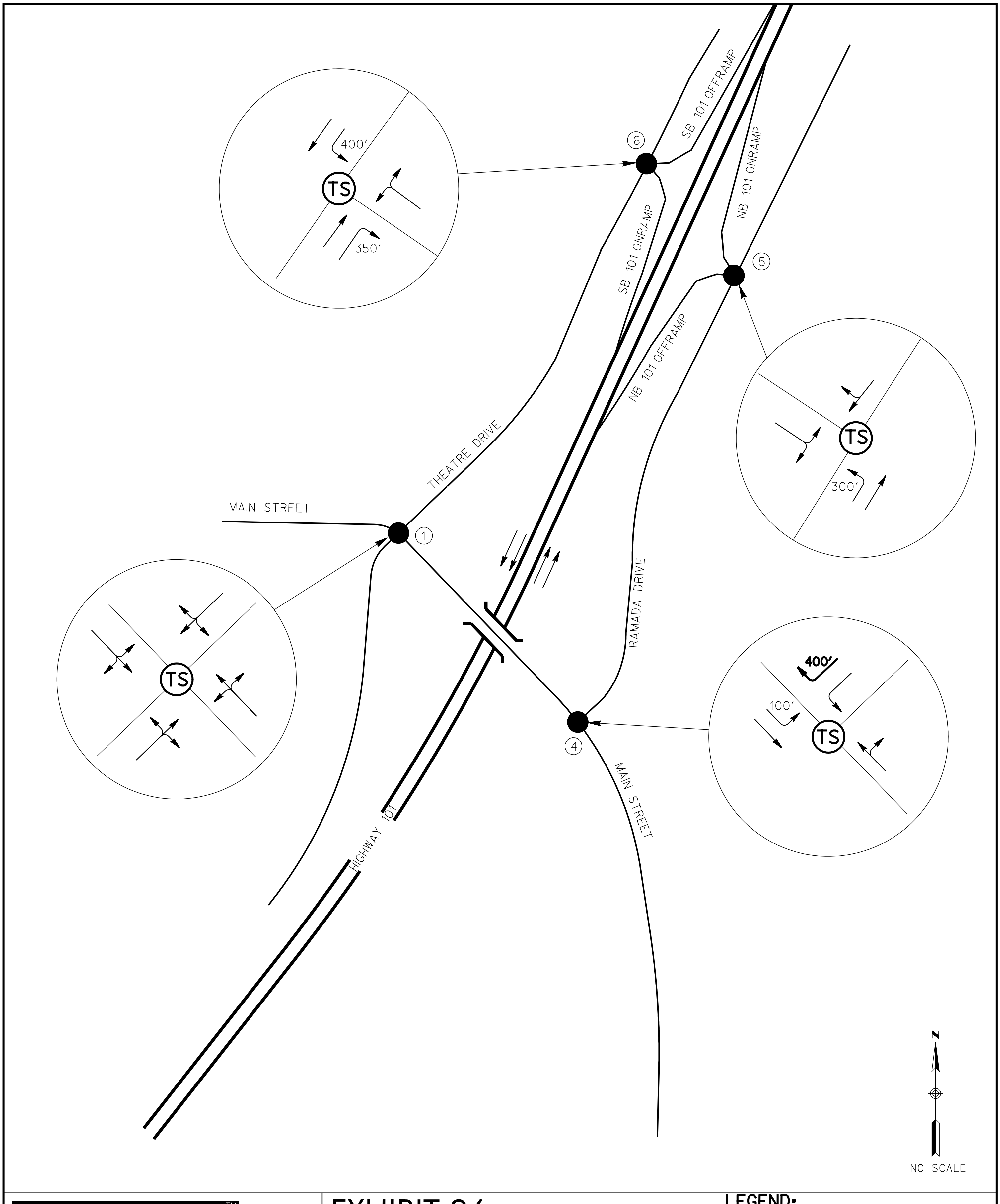


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# EXHIBIT 26

BUILDOUT WITH ALTERNATIVE 4
RECOMMENDED LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- = STUDY INTERSECTION
- TS = TRAFFIC SIGNAL
- 40' = TRAFFIC LANE & STORAGE
- **40'** = REC. TRAFFIC LANE & STORAGE



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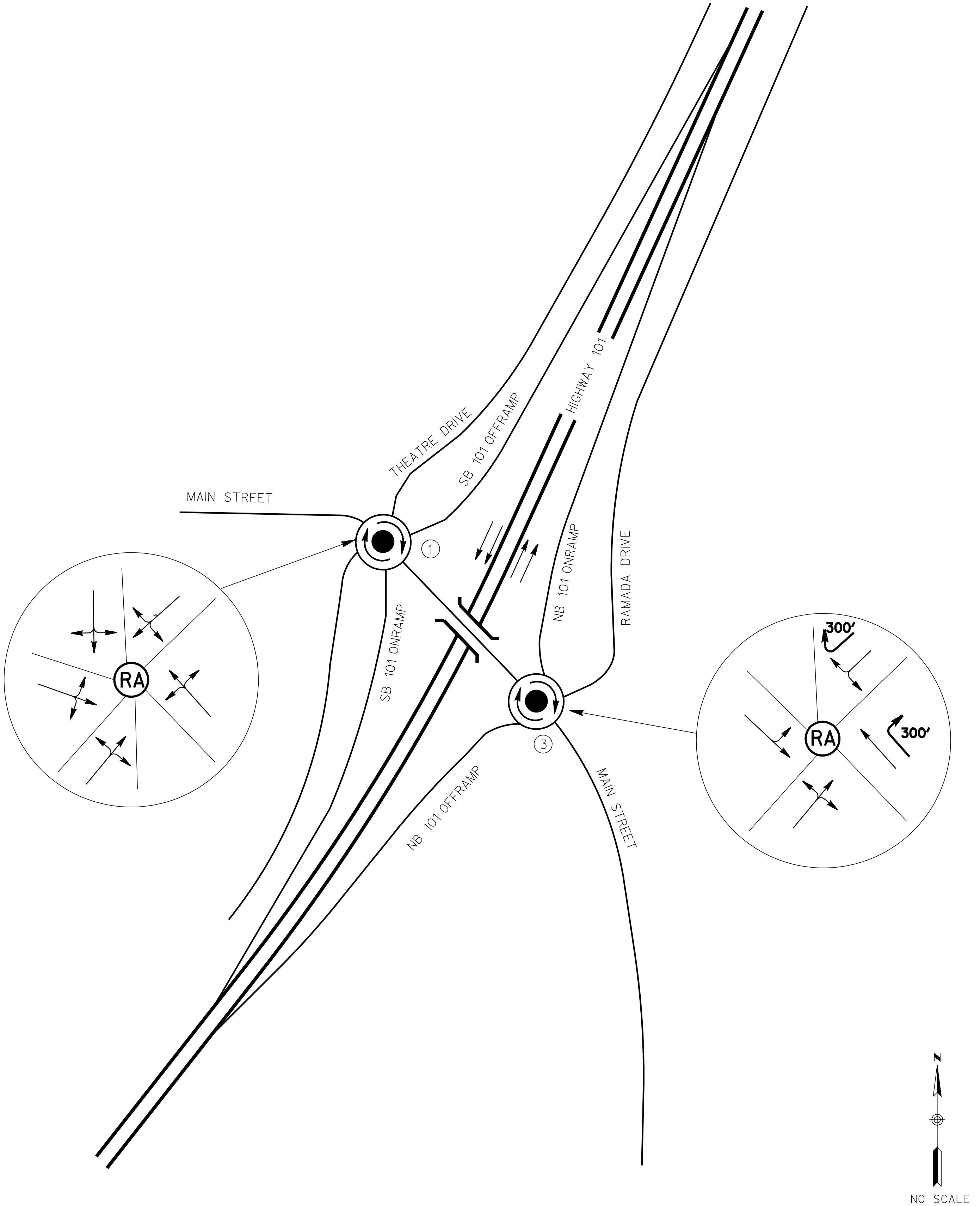
## 8.5 Alternative 5

Under the buildout conditions, the proposed roundabout at the US 101 Northbound Ramps/Main Street/Ramada Drive intersection would experience delays in the LOS E-F range during both peak hours. The higher circulating traffic on the north side of the roundabout causes this intersection to fail. To mitigate, higher volume conflicting movements should be isolated. The following improvement is anticipated to resolve delay and queuing issues at this location:

1. Add second entry lane with 300 feet storage on the southbound Ramada Drive approach designated only for the traffic accessing the US 101 northbound on-ramp. This lane will act as a yielding bypass lane.
2. Add second entry lane with 300 feet storage on the westbound Main Street approach designated only for the traffic heading northbound on Ramada Drive. This lane will act as a yielding bypass lane.

The above listed mitigations may require modification to the circulating path. However, the roundabout would still be classified as a single lane roundabout. **Exhibit 27** depicts the recommended lane configurations for this alternative. It should be noted that above recommendations were simply based on traffic operations perspective, and do not take account of any right of way and design related limitations.

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# EXHIBIT 27

BUILDOUT WITH ALTERNATIVE 5
RECOMMENDED LANE CONFIGURATION

US 101/MAIN STREET TRAFFIC STUDY

**LEGEND:**

- ● = STUDY INTERSECTION
- RA = ROUNDABOUT
- 40' = TRAFFIC LANE & STORAGE
- 40' = REC. TRAFFIC LANE & STORAGE



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## 9.0 ADDITIONAL INTERCHANGE ALTERNATIVES

This section identifies additional interchange alternatives that should be considered based on the findings of five alternatives studied. It should be noted that these additional alternatives are based on the traffic operations perspective and do not include design and/or right-of-way considerations.

Alternative 3A

Since the realignment of Theatre Drive under Alternative 3 on the west side of the interchange is very similar to that shown in Alternative 2, it is recommended to relocate Theater Drive so that it is just west of the Caltrans Maintenance Yard (between the Maintenance Yard and the residence). Then, the proposed intersection of Theater Drive and Main Street can move westerly, and the US 101 southbound ramps can intersect Main Street at about the same location as existing Theater Drive/Main Street intersection. This will allow for additional storage for queues on the bridge as well as between the US 101 southbound ramps and Theatre Drive on Main Street.

Alternative 6

This alternative would construct a roundabout for the US 101 southbound ramps/Main Street/Theatre Drive intersection, as proposed in the Alternative 5. The existing US 101 northbound ramps at Main Street would be removed and would connect to Ramada Drive with the Type L-6 ramps, as proposed in the Alternative 4. This “hybrid” alternative is anticipated to overcome traffic operational deficiencies of Alternative 4 and 5.

Alternative 7A, 7B and 7C

This alternative would construct a roundabout for the US 101 southbound ramps/Main Street/Theatre Drive intersection, as proposed in the Alternative 5. The US 101 northbound off-ramp would be relocated to intersect Main Street across from Ramada Drive, which will essentially form a standard four-legged intersection. The following three options can be considered for the US 101 northbound on-ramp configuration:

A. Remove existing diagonal on-ramp and construct a loop ramp which will be teed up with the Ramada Drive/Main Street intersection and will accommodate all US 101 northbound on-ramp traffic. This configuration would create the Type L-7 configuration for the US 101 northbound ramps.
B. Retain existing diagonal on-ramp for the westbound Main Street traffic. Construct a loop for the eastbound Main Street traffic. Both movements would be freely flowing.
C. Remove existing diagonal on-ramp and construct a hook on-ramp that would connect Ramada Drive with the Type L-6 ramp (similar to northbound on-ramp under Alternative 5).