Transportation

February 20, 2013

Traffic Engineering and Analysis Report

Golden Gate Bridge Moveable Median Barrier Study

Prepared for the Golden Gate Bridge Highway and Transportation District

Federal Project Number STPL-6003(037)

Traffic Engineering and Analysis Report

Golden Gate Bridge Moveable Median Barrier Study

Prepared for the Golden Gate Bridge Highway and Transportation District

Federal Project Number STPL-6003(037)

February 20, 2013

Table of Contents

List of Tables

List of Figures

1.0 Introduction

This report presents the results of an extensive analysis of the traffic-related effects of the installation of a moveable median barrier (MMB) system on the Golden Gate Bridge (GGB) and on the north and south approaches to the GGB. The specific objectives presented in this report include the following evaluations:

Median Barrier Warrant Analysis
A median barrier warrant analysis was conducted in accordance with Chapter 7 of the *Caltrans Traffic Manual*, herein referred to as the *Traffic Manual*, to document that criteria is met for installation of a median barrier within the proposed project area.

Traffic Operations With and Without the Barrier
An in-depth assessment of traffic operations was conducted to determine the traffic operational effects of replacing the existing tubular pylon system with a moveable median barrier (MMB) system. The operations of existing and future conditions were evaluated, and the implementation of a MMB was added to the existing conditions. The results were used to determine the operational effects of the barrier installation.

Lane Width Alternatives
The width of the barrier would eliminate one foot of width on the existing roadway cross-section. The roadway cross section varies throughout the project limits and is already narrow, and has lanes that do not meet standard Caltrans width requirements. The intent is to install the barrier while minimizing effects of narrowing lanes. Therefore, an analysis of various lane width alternatives was conducted, and recommendations were given based on traffic engineering analyses.

Effects on Toll Collection
The implementation of a MMB system would require modifications to the Toll Plaza. The operational effects of the proposed improvements to the Toll Plaza and toll collection operations were evaluated.

Capacity Effects
Replacing the existing pylons with a moveable barrier is expected to have some effects on roadway capacity. Using standard manuals for highway capacity and driver behavior, the effects of the barrier were calculated.

Safety Effects
The main objective of this project is to reduce the likelihood of crossover accidents on the Golden Gate Bridge. Historical accident data were reviewed, and the effects of this project on safety were evaluated.

1.1 Analysis Elements

A description of the aforementioned analysis components is included as follows:

Existing Conditions – The Existing Conditions analysis includes an evaluation of the current traffic volumes and the corresponding characteristics, lane configurations, field observations, and a collision summary.

Design Year Conditions – The Design Year Conditions analysis includes an evaluation of the projected design year traffic volumes and the corresponding characteristics.

Roadway Capacity – The roadway capacity analysis consists of an assessment of the effects of the MMB on the roadway capacity. The roadway capacity is determined based on lane widths, lateral clearances, and the effect of the shy distance. This assessment includes the recommended lane widths based on several parameters.

Toll Plaza Capacity – The Toll Plaza capacity analysis consists of an evaluation of the existing and future demand, capacity, and operations of the tollbooths.

Traffic Operations – The traffic operations analysis consists of an evaluation of the existing and future freeway operations and conditions. The Sausalito Lateral Interchange (northern limit) and the Park Presidio Boulevard Interchange (southern limit) bound the evaluation area of the regional freeway analysis. This regional freeway analysis includes the development of a GGB and US 101 freeway simulation model.

Lane Configuration Schedule – The lane configuration schedule includes lane configuration modification schedules based on the time of day for weekdays and weekends.

Construction Conditions – The construction conditions analysis consists of an evaluation of roadway operations during construction of the northern approach and Toll Plaza improvements at the southern approach in the short-term condition.

The project area vicinity map is shown in **Figure 1**.

1.2 Analysis Methodology

The transportation network was evaluated by utilizing multiple analysis tools and methodologies. Each of the analysis procedures was developed to evaluate critical components of the transportation network.

The regional freeway analysis was developed to evaluate peak period conditions. Typically, on weekdays, vehicles in the vicinity of the US 101 freeway experience congested conditions between 7:00am and 9:00am in the morning peak period and between 4:00pm and 6:00pm in the evening peak period. Herein, these analysis periods are presented as the weekday AM peak period and weekday PM peak period, respectively.

The peak hour of congestion represents the most congested one hour within the corresponding peak period. Typically, on weekdays, vehicles in the vicinity of the US 101 freeway experience the peak hour conditions between 8:00am and 9:00am in the weekday AM peak period and between 5:00pm and 6:00pm in the weekday PM peak period. Herein, these analysis periods are presented as the weekday AM peak hour and weekday PM peak hour, respectively. Unless otherwise noted, the analysis and results presented herein are a representation of the peak hour of congestion.

The Toll Plaza capacity analysis was developed to evaluate weekday peak hour and weekend peak hour conditions. Similar to the regional freeway analysis, the greatest weekday demand at the Toll Plaza occurs between 8:00am and 9:00am in the weekday AM peak period and between 5:00pm and 6:00pm in the weekday PM peak period. The greatest weekend demand at the Toll Plaza occurs between 4:00pm and 5:00pm on Saturdays and between 5:00pm and 6:00pm on Sundays. Herein, these analysis periods are presented as the Saturday peak hour and Sunday peak hour, respectively.

The following analyses were conducted to evaluate the effects of the Proposed Project on the transportation network.

Traffic Volumes – Traffic volumes were collected on the mainline freeway and ramps within the evaluation area. This includes weekday and weekend hourly traffic volumes and toll collection type compositions. Specifically, the traffic volumes assessment includes the following detailed information:

- Weekday peak hour ramp volumes data collection summary;
- Weekday peak hour freeway traffic volumes diagrams;

Figure 1: Project Vicinity Map

- Weekday maximum hourly GGB traffic volumes summary;
- Weekday peak hour GGB heavy vehicle volumes summary;
- Hourly GGB traffic volumes diagrams; and
- Historical average daily traffic volumes summary.

Lane Configurations – An assessment of the existing geometry and operations of the Golden Gate Bridge and US 101 freeway were conducted. This assessment includes the schedule and procedure for reversing the direction of the lanes on the GGB and the existing lane widths. Given the existing geometry and operations, the theoretical lane capacities were determined based on the *1994 Highway Capacity Manual* (HCM).

Field Observations – Field observations were conducted on the GGB and US 101 freeway to determine locations that limit the operational capacity of the freeway facilities. These observations include operating conditions, sources of capacity limitations, and existing substandard features. This evaluation includes photographs of the existing freeway facilities.

Collision Summary – Collision data were obtained for freeway facilities along the GGB and US 101 freeway. The data were converted into accidents per million vehicle miles traveled and compared to statewide statistics for similar facilities.

Freeway Operations Analysis – A network simulation model was developed to evaluate the operations of the regional freeway facilities. The simulation model was developed in McTrans' TSIS-CORSIM 6.1 (Build 509) software program. The CORSIM software is a specialized corridor simulation tool that was originally developed by the Federal Highway Administration (FHWA). The CORSIM software was utilized for this analysis as the simulation model allows the user to evaluate the temporal build up of congested conditions and the recovery of the system at the end of the period. The peak period of congestion is complex and evaluating solutions under these conditions can only be accomplished using simulation tools such as CORSIM. CORSIM is an approved software for use in Caltrans facility analyses. The model development process was consistent with the methodology presented in the *Traffic Analysis Toolbox Volume IV: Guidelines for Applying CORSIM Microsimulation Modeling Software*.[i]

The northern and southern limits of the simulation model are as follows:

- Northern Limit – 4,800 feet north of the Sausalito Lateral Interchange; and
- Southern Limit – 3,800 feet south of the Park Presidio Boulevard Interchange.

The simulation model produced a visual representation of the results and quantitative Measures of Effectiveness (MOE). The following MOE were extracted from the model:

- Vehicle density;
- Vehicle speed; and
- Vehicle throughput.

The Level of Service (LOS) criteria is based on the vehicle density as reported in the *2000 Highway Capacity Manual*. Based on these criteria, a general Level of Service criteria was applied to all weaving, ramp junction, and mainline segments throughout the corridor. These criteria are included in **Table 1**.

Table 1: Freeway Level of Service Criteria

LOS	Description	Density
A	No traffic congestion	< 10.0
B	Little traffic congestion	> 10.0 and < 20.0
C	Average traffic congestion	> 20.0 and < 28.0
D	Heavy traffic congestion	> 28.0 and < 35.0
E	Very heavy traffic congestion	> 35.0 and < 43.0
F	Extreme traffic congestion	> 43.0

Source: *Highway Capacity Manual*, Transportation Research Board, 2000.
Notes:
- Density in terms of passenger cars per mile per lane.

The CORSIM model was calibrated based on traffic operations data collected in the evaluation area. A summary of the calibration methodology and results is included in this report and a supplementary *Model Calibration Analysis Memorandum* summarizing the calibration methodology and results is included in **Appendix A**.

Toll Plaza Capacity Analysis – The Toll Plaza capacity analysis was developed based on Toll Plaza demand data and Toll Plaza capacity data provided in the *Doyle Drive South Access to the Golden Gate Bridge Addendum to the Final Traffic and Transit Operations Report*.[ii] This document is herein referred to as the “Doyle Drive Report Addendum”. The Toll Plaza capacity analysis included the effects of traffic volumes on toll operations with respect to toll collection procedures. This analysis was conducted for weekday and weekend peak hours to determine the maximum number of tollbooths required to accommodate demand.

Page intentionally left blank

2.0 Project Description

The Golden Gate Bridge (GGB) serves as the main transportation link between Marin County (MRN) and the City and County of San Francisco (SF). The GGB is a six lane undivided highway administered by the Golden Gate Bridge, Highway and Transportation District (District) that spans nearly 9,000 feet from the northern San Francisco peninsula to the south-facing Marin County headlands, near the town of Sausalito. The corridor includes the GGB and the US 101 freeway at the south and north of the GGB. The Golden Gate Bridge is under the jurisdiction of the District and the US 101 freeway is under the jurisdiction of the California Department of Transportation (Caltrans), District 4. GGB is not on the State Highway System but is on the National Highway System.

2.1 Purpose and Need

The purpose of the Golden Gate Bridge Moveable Median Barrier Project is to enhance safety and minimize the potential for crossover collisions on the Golden Gate Bridge and its approaches by installing a physical barrier to replace the plastic-tubular pylons, which are currently used to delineate opposing directions of traffic on the GGB and to transition traffic from the approach traffic lanes to the six GGB traffic lanes.

Currently, the Golden Gate Bridge is a six lane undivided highway with average daily traffic volumes in excess of 110,000 vehicles. Opposing directions of traffic are now delineated by plastic-tubular pylons that do not prevent vehicles from inadvertently crossing over into oncoming traffic. Based on data from the Traffic Accident Surveillance and Analysis System (TASAS) for a five year period, from January 1, 2006 to December 31, 2010, eight crossover collisions occurred within this period, of which five collisions resulted in injuries. The types of collisions resulting from the crossover incursions into the opposite bound of traffic, including head-on collisions that would be eliminated are typically leading to substantial injuries, and at-times fatalities.

Caltrans established warrants, which describe under what conditions a median barrier is necessary on un-divided highway facilities. These warrants fall into two basic categories – traffic volumes/median width and crossover accident history. Analysis of these features on the Golden Gate Bridge found that the median barrier warrants were fully met in each category: (1) Traffic volumes are in excess of 20,000 vehicles per day combined with median width of zero feet; and, (2) The total Crossover Accident Rate for the study facility is 0.70 crossover collisions per mile per year (based on the five year traffic accident data referenced above), exceeding the accident criteria warrant of 0.50 total crossover collisions by 40 percent. A summary of the median barrier warrant analysis is provided in Section 2.2 and results are included as **Appendix B**.

In order to accommodate the daily variations in travel demand in each direction, the Golden Gate Bridge, Highway and Transportation District (District) alters the lane configurations on the GGB several times each day to provide an optimal number of lanes in each direction to serve the traffic demand. The three primary GGB configurations include the following:

- Four northbound lanes / Two southbound lanes;
- Three northbound lanes / Three southbound lanes; and
- Two northbound lanes / Four southbound lanes.

The configurations are set by District personnel by manually installing and removing plastic-tubular pylon delineators into and from holes in the roadway. As District personnel change the delineator configurations several times per day, they do so from trucks traveling on the undivided highway in a lane next to the lane carrying traffic in the opposite direction, which exposes them to hazardous conditions. The Proposed Project will eliminate the manual pylon installation and removal operations and the exposure of District personnel to the hazardous conditions. With the installation of the Moveable Median Barrier Project, the shift of the traffic bounds between lanes will occur using a barrier transfer machine wherein the machine operator will be protected from traffic. The barrier will eliminate the need to handle, store and maintain a supply of pylons; will eliminate the maintenance and repair of the pylon holes in the roadway; and, will eliminate the cost and hazards of retrieving pylons displaced from the roadway by errant drivers.

Installation of the MMB in the locations wherein plastic-tubular delineators are currently provided will enhance the safety of the traveling public and operating staff while not materially changing the operation and configuration of the facility.

The MMB system consists of a continuous chain of one meter long, steel-encased concrete barrier units that can be moved laterally across one lane of traffic at a time by a Barrier Transfer Machine (BTM). This system would provide a semi-rigid barrier between opposing traffic on the GGB while, at the same time, allowing the District the flexibility to reconfigure the lanes on the GGB to optimize traffic operations in each direction.

To implement the MMB system, improvements at each end of the GGB are required to safely terminate the barrier and transition back to the existing freeway geometry. In this installation, as typically occurs with these systems, the BTMs would be parked in the median of the freeway when not in use. The only periods when the BTMs would leave the freeway right-of-way would be for maintenance and / or fueling. At the north end of the GGB, a portion of the existing median barrier would be removed to provide enough space for the MMB to terminate safely and store the BTM when not in use. Due to the construction of the Presidio Parkway Project, two scenarios would be required; a short-term strategy to accommodate the BTM and MMB system before the Presidio Parkway Project is complete, and a long-term strategy that would extend the MMB system to the completed Presidio Parkway Project. In the short-term, the MMB system would terminate at a temporary MMB location just north of the Tollbooths (within District right-of-way). In the long-term, the MMB system would be extended approximately 750 feet south of the tollbooths (within District right-of-way). To accommodate the MMB system on the south end, the four easternmost toll booths would be removed and one toll lane would be re-constructed and modified to electronic toll collection only.

2.2 Median Barrier Warrant Analysis

For freeways, the median width, traffic volume (measured in Average Daily Traffic (ADT), and number / severity of cross-median type accidents are used to determine if the warrants for median barrier installation are met (see Topic 7-04 Median Barrier of the *Traffic Manual*). Each segment of the project was evaluated for barrier warrants. The results of the median barrier warrant analysis are included in **Appendix B**. The evaluated segments, from Post Mile to Post Mile, are as follows:

- US 101 and the GGB, from 04 SF 101 PM 009.400 to 04 SF 101 PM 011.181 (undivided);
- US 101 and the GGB, from 04 MRN 101 PM L000.000 to 04 MRN 101 PM L000.494 (undivided); and
- US 101, from 04 MRN 101 PM 000.000 to 04 MRN 101 PM 0.600 (divided).

It should be noted that PM 04 SF 101 PM 011.181 is equivalent to 04 MRN 101 PM L000.000, and 04 MRN 101 PM L000.494 is equivalent to 04 MRN 101 PM 000.000.

Figure 7-12 of the *Traffic Manual* indicates the need for a median barrier based on traffic volume (measured in ADT) and median width. According to Figure 7-12, the need for a median barrier should be considered on freeways when the volume and median width meet certain criteria. Analysis of these features on the Golden Gate Bridge found that the median barrier warrants were fully met in each category as traffic volumes are in excess of 20,000 vehicles per day and median width measures zero feet. In 1989, the ADT of the GGB was 120,000. In 2012, approximately 110,000 vehicles utilized the GGB daily and ADT is forecasted to increase to 138,000 by 2030. Median width varies between zero feet and eight feet on the three study segments. According to Figure 7-12, all three study segments meet warrant criteria under current and future year conditions. According to Section 7-04.04 of the *Traffic Manual*, the locations with cross-median accident rate data that exceed either 0.50 total crossover collisions per mile per year, or 0.12 fatal crossover collisions per mile per year, should be identified for improvements that would reduce the severity of collisions.

Traffic Accident Surveillance and Analysis System (TASAS) data were obtained for freeway facilities along the US 101 freeway. TASAS data was reviewed for a five year period, from January 1, 2006 to December 31, 2010. Of the 407 mainline accidents, 321 collisions occurred on the undivided highway segment, between 04 SF 101 PM 009.400 and 04 MRN 101 PM L00.494. Eight of the accidents were cross-median collisions, which resulted in five injuries and no fatalities. This freeway segment had a combined Total Crossover Accident Rate of 0.70 total crossover collisions per mile per year, which exceeds the accident criteria warrant of 0.50 total crossover collisions per mile per year by 40 percent. Based on the results of the traffic volume and median width and collision study warrants, the installation of a median barrier within the Proposed Project area is warranted.

The installation of the MMB is intended to improve the safety conditions on the Golden Gate Bridge while minimizing effects on traffic operations. The following section presents a more detailed description of the proposed improvements, a summary of the MMB operations, and the recommended roadway lane width alternative.

2.3 Project Improvements

The Proposed Project is intended to replace the existing plastic pylon with a Moveable Median Barrier. The MMB would be transitioned between lanes (maximum of 14 feet laterally) by a BTM. Since two of the lanes are reversible, two BTMs would be required. The MMB is constructed of interlocking units that are 39 inches long, 12 inches wide at the top, 24 inches wide at the base, and 32 inches high. Although the base of the MMB is 24 inches (12 inches wider than the top), the flare occurs near the bottom and vehicles can drive on the corresponding base flange. Additionally, since the flare occurs near the bottom, drivers are expected to perceive the MMB as being only 12 inches wide. Given the functionality of the base and the effect on driver perception, the MMB is evaluated as being 12 inches wide.

Upon completion, the 11,500 foot long MMB would extend from the concrete roadway divider 380 feet south of the Sausalito Road Undercrossing (northern limit) to just north of the Caltrans / District right-of-way (southern limit). The Proposed Project would result in the following significant modifications to the roadway network:

- The four-inch wide pylons that currently delineate the GGB would be replaced with a 12-inch wide median barrier. This would result in the narrowing of the lanes and the addition of a lateral feature.
- Tollbooth 8 would be removed, and a new wide toll lane 8 would be constructed, encompassing the area of existing toll lane 8, Tollbooth 8, and existing toll lane 9;
- Tollbooths 9 through 11 would be removed; and
- The MMB would terminate approximately 1,010 feet to the north of the existing southern terminus of the existing type 60C concrete barrier at the north approach.

Due to concurrent construction of the Presidio Parkway Project, the Proposed Project would likely be implemented in two stages – a short-term improvement and a long-term improvement. The following section provides a description of the Presidio Parkway Project and the staged MMB improvements.

Presidio Parkway Project

Doyle Drive is located in the Presidio of San Francisco between the southern approach to the Golden Gate Bridge and Richardson Avenue. Doyle Drive is classified as a multilane conventional highway with a posted speed of 45 miles per hour (mph) for its mainline section and 35 mph for the ramp and weaving sections. Doyle Drive is 1.5 miles long with six traffic lanes, which passes through the Presidio of San Francisco on an elevated concrete viaduct (low-viaduct) and transitions to a high steel truss viaduct (high-viaduct) as it approaches the Golden Gate Bridge Toll Plaza.

Currently, Doyle Drive has nonstandard design elements, including travel lanes from 9'-6" to 10'-0" feet in width, no fixed median barrier, no shoulders, exit ramps that have tight turning radii, and the roadway is structurally and seismically unsafe. At the west terminus, near the GGB, Doyle Drive operates as a free-flow roadway, while at the east terminus Doyle Drive operates as an arterial roadway meeting local streets. Within this segment, there are several ramps that carry substantial traffic, reversible lane configurations that are manually altered several times daily, and access to the local street network. The purpose of the Presidio Parkway Project is to replace Doyle Drive in order to improve the seismic, structural, and traffic safety of the roadway. Golden Gate Bridge traffic is directly affected by the Presidio Parkway Project, which began construction in late 2009 and will be completed in 2015.

The Presidio Parkway Project will replace the existing facility with a new six lane roadway and a southbound auxiliary lane, between the Park Presidio Interchange and the new Presidio access at Girard Road. The new facility will consist of improved lane, median, and shoulder widths. The northern limit of Presidio Parkway Project area will extend to the Lincoln Blvd. Undercrossing within the District right-of-way.

The Park Presidio Interchange will be reconfigured due to the realignment of Doyle Drive to the south. The Southbound US 101 Off-Ramp at Park Presidio Boulevard will be replaced with standard exit ramp geometry and widened to two lanes. The Northbound US 101 Off-Ramp at Park Presidio Boulevard will be improved to provide standard exit ramp geometry. The Northbound US 101 On-Ramp at Park Presidio Boulevard will be realigned to provide standard entrance ramp geometry. The Southbound US 101 On-Ramp at Park Presidio Boulevard will be reconstructed in a similar configuration as the existing directional ramp with improved sight lines, exit, and entrance geometry.

The construction of the Presidio Parkway Project will consist of the following traffic phases:[iii]

Presidio Parkway Project – Traffic Phase I: During the first traffic phase – between 2009 and 2012 – traffic will continue to utilize the existing Doyle Drive facilities. Construction will occur adjacent to the existing roadway. During Traffic Phase I, the facilities and the operations of Doyle Drive will perform similarly to the current conditions.

Presidio Parkway Project – Traffic Phase II: During the second traffic phase – between 2012 and 2015 – traffic will be redirected onto the new southbound structures and a temporary bypass, while construction occurs adjacent to the temporary bypass. In April 2012, traffic was shifted onto a five lane, divided bypass next to Doyle Drive. Once Traffic Phase II construction is complete, traffic will be redirected onto the final alignment. During Traffic Phase II, the facilities and the operations of Doyle Drive will perform similarly to the current conditions. Upon completion of Traffic Phase II, the facilities and the operations of Doyle Drive will differ from the current conditions as the roadway alignment and geometry will be modified.

Presidio Parkway Project – Final Alignment: In 2015, construction of the new Presidio Parkway alignment will be complete. Final project activities after this include removal of the temporary detour and landscaping.

Toll Plan

In April 2010, the District commissioned the *Strategic Plan for All Electronic Toll Collection on the Golden Gate Bridge* – herein referred to as the "Toll Plan".[iv] The Toll Plan is a preliminary study intended to investigate the feasibility of making all toll collection at the Toll Plaza electronic. The components of the Toll Plan will include toll collection conversion cost estimates, schedules, modifications, policy changes, and an evaluation of alternatives. The implementation of the Toll Plan would increase the rate in which tolls are collected at the Toll Plaza, reducing the number of tollbooths required.

On January 28, 2011, the District Board of Directors voted to proceed with the Toll Plan. All electronic toll collection is scheduled to begin functionality testing in January 2013 and planned to be fully implemented in March 2013.[v]

2.4 Project Analysis Scenarios

Due to the construction of the Presidio Parkway Project, the Proposed Project would likely be implemented in two stages – a short-term improvement and a long-term improvement. The short-term improvement and long-term improvement would only affect the improvements on the southern approach of the GGB. Improvements to the northern approach of the GGB would not be affected by the Presidio Parkway Project; therefore, the modifications to the northern approach of the GGB and the narrowing of the lanes on the GGB are applicable to the short-term improvement and long-term improvement scenarios. The northern approach Proposed Project improvements for the three potential roadway configurations are shown in **Figure 2**.

Additionally, modifications to the Toll Plaza would be incorporated in the short-term improvement and long-term improvement scenarios. The Toll Plaza modifications would consist of the following improvements:

- Tollbooth 1 through Tollbooth 7 would remain the same as the current configuration;
- The lane width through Tollbooth 8 would change from 9'-10" to 14'-0";
- Tollbooth 8 through 11 would be removed, including all at-grade toll collection equipment; and
- Toll collection equipment would be reconfigured to accommodate new widened toll lane 8 traffic.

Short-Term Improvements – Prior to the completion of the Presidio Parkway Project, the roadway to the south of the Toll Plaza would function similarly to the Existing Conditions. Northbound and southbound Doyle Drive would continue to be six lanes wide and delineated by pylons. To the north of the Toll Plaza, the MMB would be functional.

The MMB would terminate north of the Toll Plaza (within District right-of-way) during the short-term improvements, as the Presidio Parkway Project would not be completed. This location would provide storage for the BTM(s), when necessary. The southern Proposed Project short-term improvements for the three potential roadway configurations are shown in **Figure 3**.

During the short-term improvements, the BTM(s) would be stored near the tollbooths. Given the dimensions of the BTMs and the approach to the Toll Plaza, access to the tollbooths would be restricted with the presence of the BTM(s). Based on the lane configuration during the short-term improvements, the Toll Plaza would functions as follows:

- During periods where the lane configuration consists of two northbound lanes / four southbound lanes, a maximum of eight tollbooths could be active.
- During periods where the lane configuration consists of three northbound lanes / three southbound lanes, a maximum of eight tollbooths could be active.
- During periods where the lane configuration consists of four northbound lanes / two southbound lanes, a maximum of seven tollbooths could be active.

Figure 2: Northern Approach Project Improvements

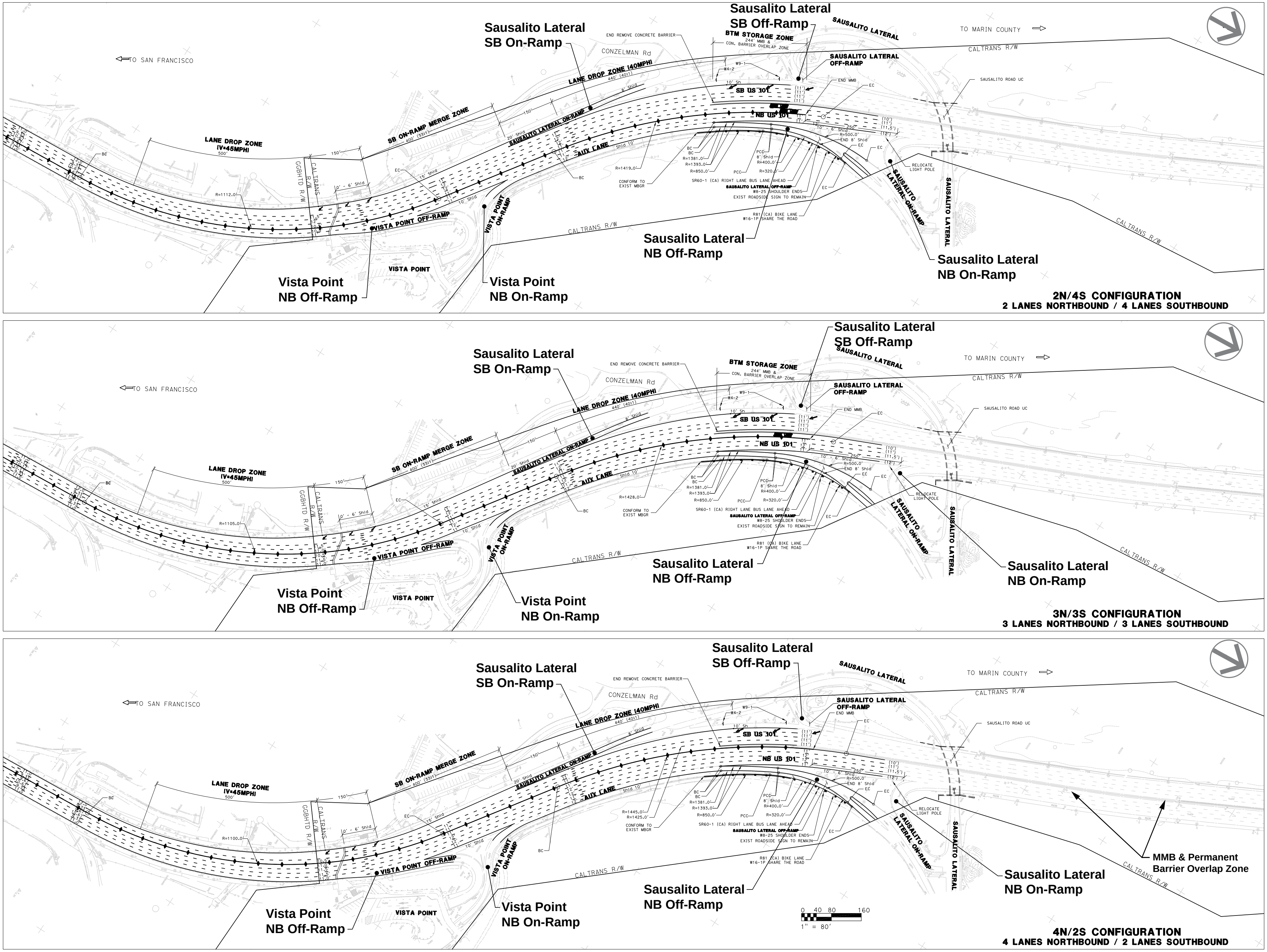
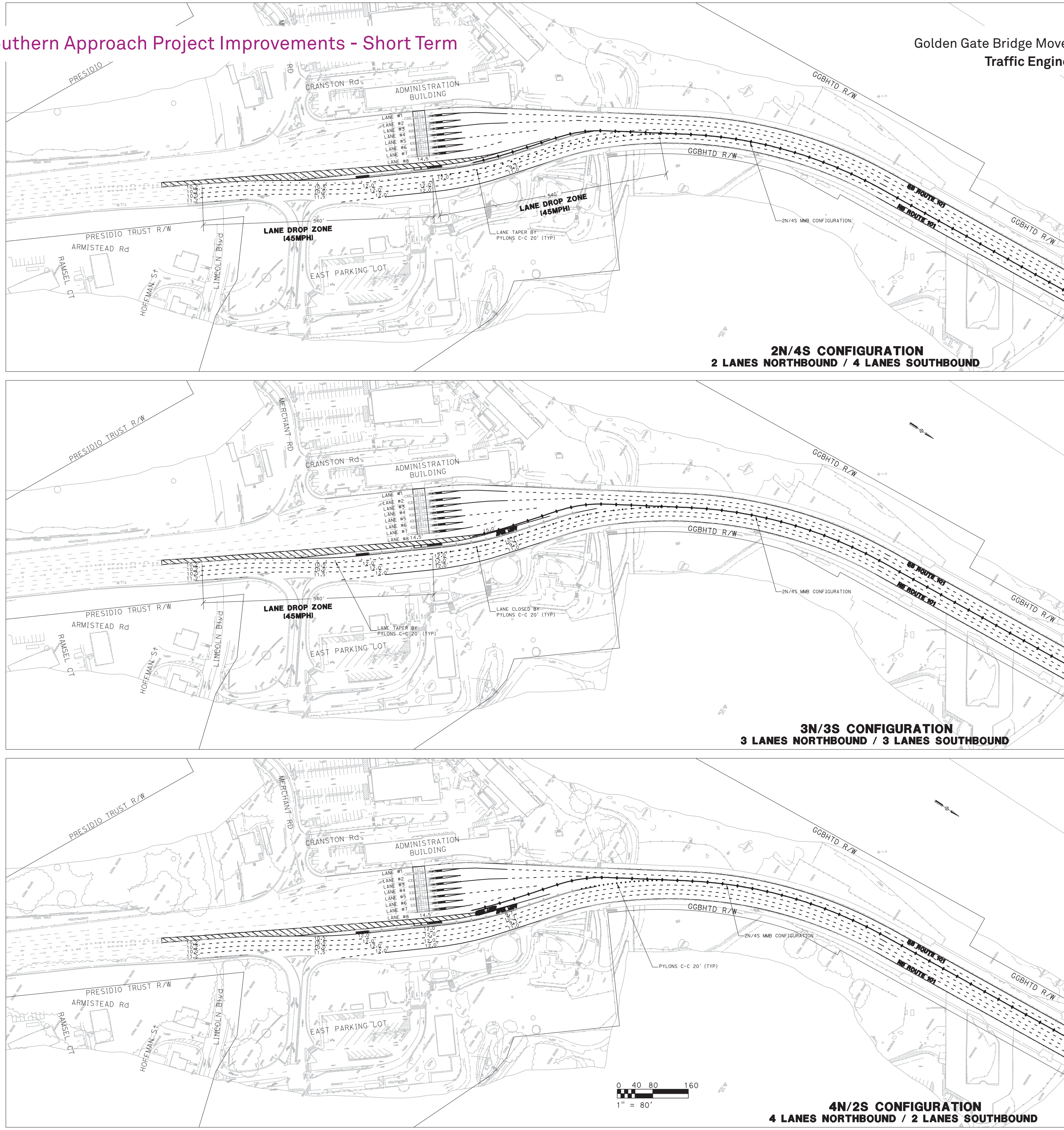


Figure 3: Southern Approach Project Improvements - Short Term



Long-Term Improvements – After the completion of the Presidio Parkway Project, the northbound and southbound Doyle Drive would be divided by a wide median and the lanes south of the District / Caltrans right-of-way would no longer be reversible. The MMB would terminate approximately 750 feet to the south of the Tollbooths within District right-of-way. The southern Proposed Project long-term improvements for the three potential roadway configurations are shown in **Figure 4**.

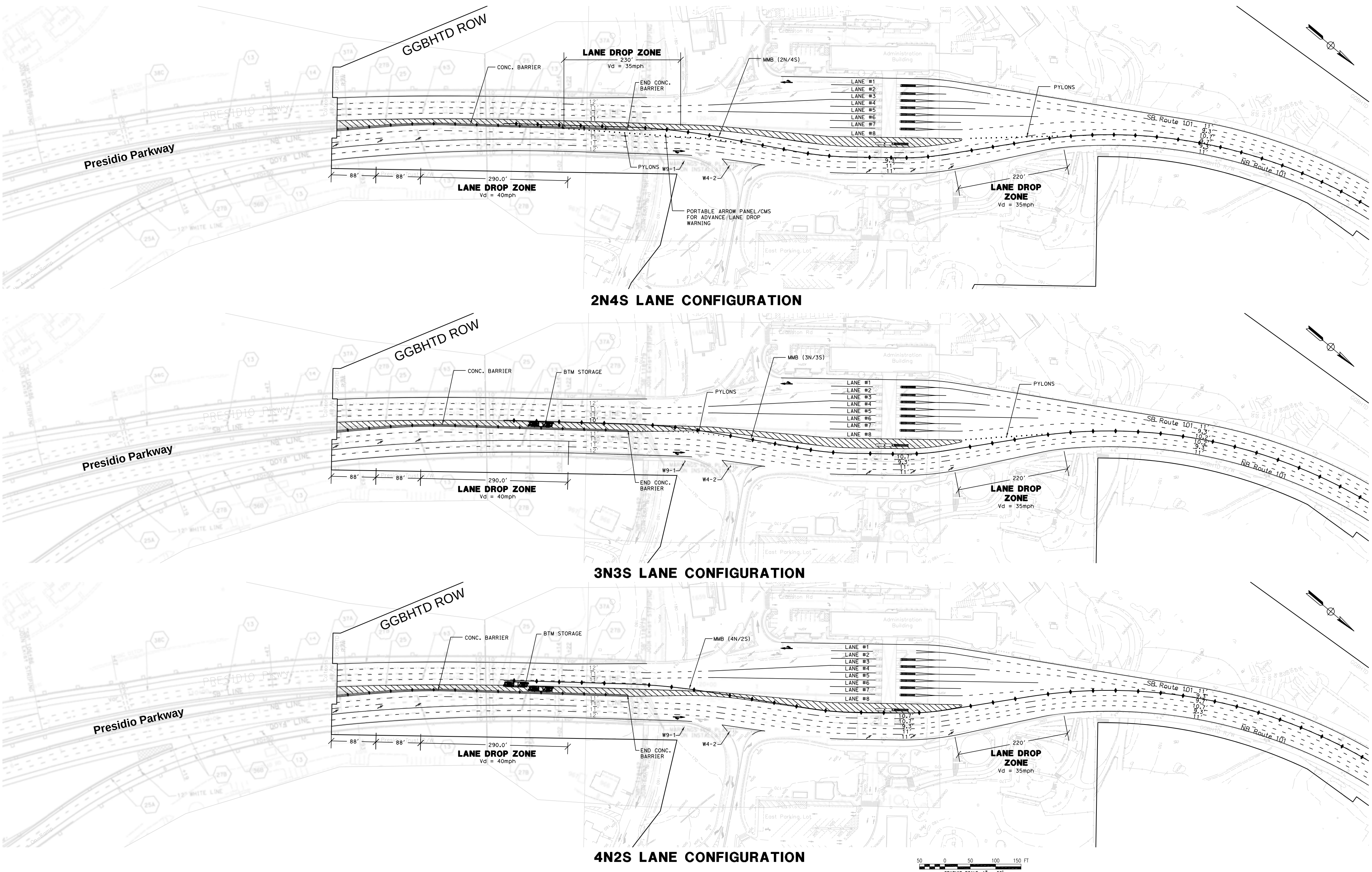
After the implementation of the long-term improvements, the BTM(s) would be stored 750 ft south of the Tollbooths within the District right-of-way. The MMB would provide the lane reductions south of the Tollbooths and the manual placement of pylons would no longer be necessary. All eight of the tollbooths could be functional regardless of the lane configuration on the GGB.

Project Analysis Scenarios – This analysis is intended to determine the effects of the Proposed Project on the Existing Conditions and the design year – typically 20 years beyond the opening year of the Proposed Project. The Existing Conditions and design year analyses are Caltrans requirements. Accordingly, this analysis consists of the following analysis scenarios:

- Existing (2012) Conditions;
- Existing (2012) plus Project Conditions;
- Design Year (2030) Conditions; and
- Design Year (2030) plus Project Conditions.

Since the Presidio Parkway Project is expected to be completed and functional in 2015, the geometry and operations within the vicinity of the Proposed Project are expected to vary between the Existing Conditions and the Design Year (2030) Conditions. Given the schedule of the Presidio Parkway Project, the Existing plus Project Conditions analysis assumes the completion of the short-term improvements, and the Design Year plus Project Conditions analysis assumes the completion of the long-term improvements. The Design Year Conditions analyses assume the Presidio Parkway Project improvements will be operational.

Figure 4: Southern Approach Project Improvements - Long Term



This page intentionally left blank

3.0 Model Calibration Analysis

The model calibration analysis consists of a comparative evaluation of the simulated freeway results to the observed freeway results. This comparison is conducted at several locations on the US 101 freeway between the Sausalito Lateral Interchange (north) and the Park Presidio Boulevard interchange (south). The project area vicinity map is shown in **Figure 1**.

The model calibration analysis includes an outline of the methodology utilized to develop, calibrate, and validate the traffic simulation model for use in this evaluation. A description of the modeling software and methodology is included in the following sections:

- Model Calibration Analysis Methodology;
- Model Development Procedure;
- Model Development Process;
- Model Calibration Process;
- Model Calibration Results; and
- Model Validation.

3.1 Model Calibration Analysis Methodology

A network simulation model was developed to evaluate the operations of the regional freeway facilities. The simulation model was developed in McTrans' TSIS-CORSIM 6.1 (Build 509) software program. The CORSIM software is a specialized corridor simulation tool that was originally developed by the Federal Highway Administration (FHWA). The model development process was consistent with the methodology presented in the *Traffic Analysis Toolbox Volume IV: Guidelines for Applying CORSIM Microsimulation Modeling Software*.[vi]

The CORSIM model was calibrated based on travel speed and traffic volumes. This calibration methodology is consistent with the procedures recommended by the FHWA.

Travel speed data on the US 101 freeway were utilized as a quantitative calibration parameter. The travel speeds simulated in the CORSIM model were calibrated to replicate observed travel speed within a 20 percent confidence level.[vii]

Traffic volumes were evaluated on a segment by segment basis and were utilized as a quantitative parameter. The traffic volumes were examined by utilizing the following conditional criteria included in **Table 2**.

It should be noted that the FHWA does not have a standard of acceptance for the number of calibration criteria that must be met to validate a model. The number of calibration criteria that should be met varies depends on factors such as the size of the model, resources available, purpose and objectives of the analysis, and types of alternatives analyzed. Although not required per the FHWA guidelines, a typical standard of acceptance is 85 percent. That is, at least 85 percent of the calibrated segments should meet the calibration targets.

The regional freeway analysis was developed to evaluate peak period conditions. Typically, on weekdays, vehicles in the vicinity of the US 101 freeway experience congested conditions between 7:00am and 9:00am in the morning peak period and between 4:00pm and 6:00pm in the evening peak period. Herein, these analysis periods are presented as the weekday AM peak period and weekday PM peak period, respectively.

Table 2: Freeway Traffic Volume Calibration Criteria

Simulated Segment Traffic Volume	Calibration Target[a]
$\leq$ 700 vph	$\leq$ 100 vph
> 700 vph and $\leq$ 2,700 vph	$\leq$ 15%
> 2,700 vph	$\leq$ 400 vph

Source: *Traffic Analysis Toolbox Volume IV.* Federal Highway Administration, 2007.
Notes:
- Calibration target in terms of absolute vehicles on the segment.
- vph = Vehicles Per Hour
[a] Calibration target based on comparison of simulated traffic volumes to observed traffic volumes.

The peak hour of congestion represents the most congested one hour within the corresponding peak period. Typically, on weekdays, vehicles in the vicinity of the US 101 freeway experience the peak hour conditions between 8:00am and 9:00am in the weekday AM peak period and between 5:00pm and 6:00pm in the weekday PM peak period. Herein, these analysis periods are presented as the weekday AM peak hour and weekday PM peak hour, respectively. Unless otherwise noted, the analysis and results presented herein are a representation of the peak hour of congestion.

3.2 Model Development Procedure

Micro-simulation tools are typically preferable to more simplistic analyses, such as empirically developed *Highway Capacity Manual* (HCM) evaluations, for complex transportation networks. Micro-simulation tools are preferred because they are designed to model transportation networks temporally and account for interactions with other facilities. For example, micro-simulation tools can model the impacts of fluctuations in traffic characteristics over time and space. Likewise, these tools can model interactive geometric configurations, such as closely spaced intersections in relation to ramp metering. Ultimately, micro-simulation tools can be utilized to model unique conditions over time, whereas simplistic analysis tools are utilized to model an isolated facility for an independent moment.

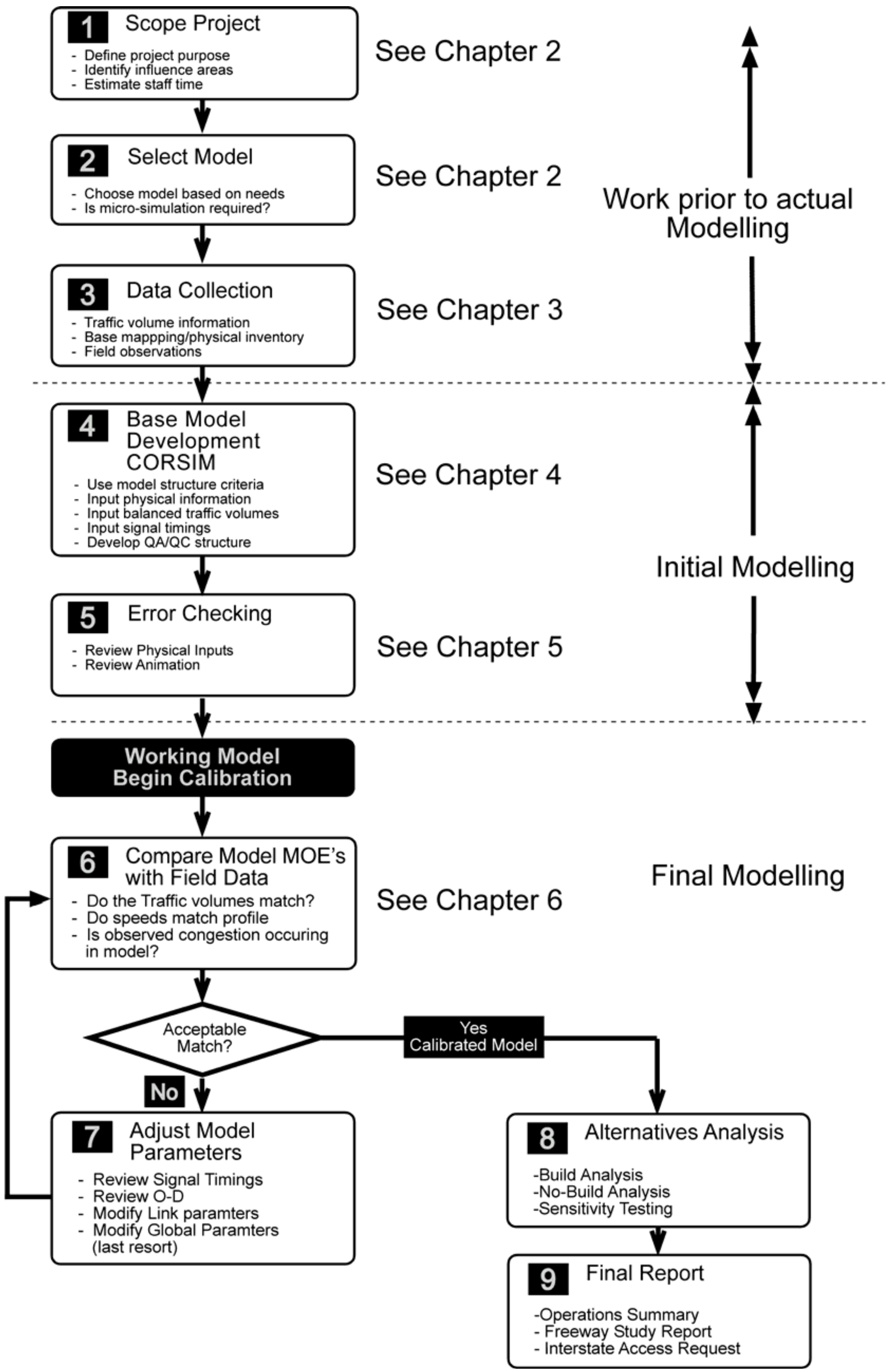
A flow diagram of the CORSIM modeling process is shown in **Figure 5**. This process was developed by the FHWA and is based on the best practices for simulation modeling. This diagram provides a specific outline of the model development process, the calibration procedure, and the stage at which the model is validated for alternatives analyses.

As shown in the CORSIM modeling process flow diagram, the complete development of a project alternative analysis is a nine step process. The nine step process consists of three sections:

1. Work prior to actual modeling (*Step 1* to *Step 3*);
2. Initial modeling (*Step 4* and *Step 5*); and
3. Final modeling (*Step 6* to *Step 9*).

The work prior to actual modeling section of the CORSIM modeling process flow diagram includes project scoping (*Step 1*), model selection (*Step 2*), and data collection (*Step 3*). The project scoping and model selection were conducted as part of the Moveable Median Barrier Project Scope of Work.

Figure 5: Simulation Model Process Flow Diagram



3.3 Model Development Process

The initial modeling section of the CORSIM modeling process flow diagram includes the base model development (*Step 4*) and error checking (*Step 5*). The development of the base CORSIM model consists of four fundamental components. These components include:

Component 1: Link-Node Diagram Development
The purpose of this component is to create a geometrically accurate representation of the freeway and arterial network. The physical roadway layout is discretized into a network of links and nodes for input into the CORSIM software.

Component 2: Lane Geometry Development
The purpose of this component is to input all of the freeway and arterial roadway lane geometry attributes. These lane geometry attributes include the number of lanes, length of the turn pockets, length of the acceleration and deceleration areas, etc. This information is collected using as-built drawings, aerial photographs, and field measurements. The network geometry is stored in a quality assurance (QA) / quality control (QC) database.

Component 3: Free-Flow Link Speeds Development
The purpose of this component is to input all of the freeway and arterial roadway free-flow link speeds. This information was collected based on posted speed limit signage and design speed throughout the study area. The free-flow link speeds are included in the QA / QC database.

Component 4: Traffic Volumes Database Development
The purpose of this component is to input all of the freeway volumes. The CORSIM model was utilized to simulate traffic conditions for the weekday AM peak period (7:00am to 9:00am) and the weekday PM peak period (4:00pm to 6:00pm).

Traffic volumes throughout the freeway and arterial network were divided into 15-minute time intervals for the two hour simulation periods. Where 15-minute interval traffic volume data were available (e.g., mainline freeway segments), the source data were utilized. Where 15-minute interval traffic volume data were unavailable (e.g., freeway ramps), the 60-minute interval traffic volume source data were disaggregated into 15-minute intervals assuming a 0.95 peak hour factor.

The regional mainline freeway traffic volumes were balanced utilizing one mainline control point and upstream / downstream ramps. Equilibrium was maintained throughout the freeway network.

Error checking (*Step 5*) was conducted by reviewing the physical model inputs and reviewing the animation. Once the inputs and animation were verified, the initial modeling section was completed.

3.4 Model Calibration Process

The final modeling section of the CORSIM modeling process flow diagram includes the model comparison to field data (*Step 6*), model parameter adjustments (*Step 7*), alternatives analysis (*Step 8*), and the final report (*Step 9*).

Mainline US 101 freeway travel speed surveys were conducted by “floating vehicles”, or moving observers, on a typical weekday.

The model comparison to field data (*Step 6*) and the model parameter adjustments (*Step 7*) are iterative. That is, the steps must be repeated until the conditional criteria are met. The model calibration methodology is an informed “guess and check” process. Typically, there are best practices for calibrating a model. For example, if observed congestion is caused by tight horizontal curvature, the desired free-flow speed or headway spacing may need to be adjusted. However, every transportation network is unique and the specific calibration parameters must be adjusted accordingly. The calibration process sequentially consists of the following four (4) conditions:

1. No modifications to the default parameters – No adjustments are made to the known global parameters (e.g., vehicle type distribution), local parameters (e.g., headway factors), or unknown global parameters (e.g., car following factors). If the model MOE meet the model calibration criteria, the model calibration is complete. If not, the known global parameters must be modified;
2. Modify the known global parameters – Adjustments are made to the global parameters that can be quantified. Known global parameters include vehicle headway distributions, vehicle type distribution, and lane biases (for heavy vehicles). If the model MOE meet the model calibration criteria, the model calibration is complete. If not, the local parameters must be modified;
3. Modify the local parameters – Adjustments are made to the local parameters that cannot be quantified but can be observed. Local parameters include warning sign locations for exit ramps / lane drops (for lane changing behavior) and free-flow speeds / headway factors (for geometric features). If the model MOE meet the model calibration criteria, the model calibration is complete. If not, the unknown global parameters must be modified; and
4. Modify the unknown global parameters – Adjustments are made to the global parameters that cannot be quantified or observed. Unknown global parameters include driver behavior, lane change parameters, and model parameters. Modifications to these parameters affect the model logic and should only be pursued if other calibration parameter adjustments are ineffective.

Upon completion of the model comparison to field data (*Step 6*) and model parameter adjustments (*Step 7*), the calibration of the model is complete. The model is only calibrated for the Existing Conditions. Calibration is not applicable to design year or project alternative analyses. The alternatives analysis (*Step 8*) and the final report (*Step 9*) components of the CORSIM modeling process flow diagram are included in this document.

The CORSIM model required several modifications to meet the calibration criteria. Unless otherwise noted, these calibration modifications are consistent for both the weekday AM and PM peak periods. The model calibration process included the following modifications:

Link Travel Speeds
The link travel speeds were adjusted for two freeway segments to represent the typical speed of the vehicles, rather than the posted freeway speed. The link travel speed was changed from 45 mph (posted) to 55 mph (field) at the following locations:

- Southbound US 101 between Sausalito Lateral and Vista Point; and
- Northbound US 101 between Vista Point and Sausalito Lateral.

Throughout the remainder of the US 101 corridor the posted speed was maintained.

Warning Sign Locations
The warning sign locations were adjusted for two ramps to accurately model the anticipatory lane changing behavior of vehicles on the mainline freeway. The warning sign locations were changed from 2,500 feet (default) to 1,000 feet (field) for the following off-ramps:

- Southbound US 101 Off-Ramp at Park Presidio Boulevard; and
- Northbound US 101 Off-Ramp at Merchant Road.

Car Following Factor
The car following factor was adjusted at several locations to represent the driver behavior at horizontal and vertical curves on the mainline freeway. For vertical curves, the car following factor was typically increased from 1.00 (default) to 1.10 (field). For the majority of the horizontal curves, the car following factor was typically increased from 1.00 (default) to 1.25 (field). The only exception was the abrupt horizontal curve at the northbound toll plaza, where the car following factor was increased from 1.00 (default) to 1.50 (field). The mainline freeway location that was calibrated to represent the vertical curve is:

- US 101 Overcrossing at Sausalito Lateral.

The mainline freeway locations that were calibrated to represent the horizontal curves include:

- US 101 between the Park Presidio Boulevard Ramps;
- US 101 between the Toll Plaza and the Golden Gate Bridge (south); and
- US 101 between the Golden Gate Bridge (north) and the Sausalito Lateral Ramps.

3.5 Model Calibration Results
An evaluation of the CORSIM simulation model was conducted prior to calibration (default model) and after calibration (calibrated model). Typically, the simulation model required approximately ten minutes to seed, or initialize, the network. For the purposes of this analysis, all results represent the average of five simulation runs. Each simulation run was conducted for the two hour peak period. The mainline traffic volumes data were collected between Saturday, May 19, 2012 and Friday, May 25, 2012. The mainline travel speed data and field observations were collected on Tuesday, September 25, 2012.

The CORSIM model was calibrated based mainline travel speed and traffic volumes. The results of the model calibration includes the following components:

Travel Speed
Travel speed data on the US 101 freeway were utilized as a quantitative calibration parameter. The travel speeds simulated in the CORSIM model were calibrated to replicate observed travel speeds within a 20 percent confidence level. Vehicle speed data were collected at the following seven locations in the southbound direction:

1. Southbound US 101 at the Sausalito Lateral On-Ramp;
2. Southbound US 101 at the Golden Gate Bridge Entrance (north);
3. Southbound US 101 at the Golden Gate Bridge Center Span;
4. Southbound US 101 at the Golden Gate Bridge Exit (south);
5. Southbound US 101 at the Golden Gate Bridge Toll Plaza;
6. Southbound US 101 at the Merchant Road Off-Ramp; and
7. Southbound US 101 at the Park Presidio Boulevard On-Ramp.

Vehicle speed data were collected at the following eight locations in the northbound direction:

1. Northbound US 101 at the Park Presidio Boulevard On-Ramp.
2. Northbound US 101 at the Merchant Road Off-Ramp;
3. Northbound US 101 at the Golden Gate Bridge Toll Plaza;
4. Northbound US 101 at the Golden Gate Bridge Entrance (south);
5. Northbound US 101 at the Golden Gate Bridge Center Span;

6. Northbound US 101 at the Golden Gate Bridge Exit (north);
7. Northbound US 101 at the Vista Point On-Ramp; and
8. Northbound US 101 at the Sausalito Lateral On-Ramp.

The calibrated CORSIM simulation travel speeds during the weekday AM peak hour (8:00am to 9:00am) and the weekday PM peak hour (5:00pm to 6:00pm) for the southbound and northbound directions are shown in **Figure 6**. The travel speeds from the CORSIM simulation model prior to the calibration are included (shown as "default model") to show the impacts of the calibration on the accuracy of the results.

In the weekday AM peak hour simulation, the model meets the travel speed calibration criteria at all seven locations in the southbound direction. The model meets the travel speed calibration criteria at all eight locations in the northbound direction. The travel speed calibration criteria are met at 100 percent of the locations in the weekday AM peak hour simulation.

In the weekday PM peak hour simulation, the model meets the travel speed calibration criteria at all seven locations in the southbound direction. The model meets the travel speed calibration criteria at all eight locations in the northbound direction. The travel speed calibration criteria are met at 100 percent of the locations in the weekday PM peak hour simulation.

Traffic Volumes
Traffic volumes were evaluated on a segment by segment basis and were utilized as a quantitative calibration parameter. A comparison of traffic demand to the traffic simulated in the model was conducted at the seven southbound locations and eight northbound locations on the mainline freeway where the travel speed was collected.

A comparison of calibrated CORSIM simulation volumes to the traffic demand during the weekday AM peak hour (8:00am to 9:00am) and the weekday PM peak hour (5:00pm to 6:00pm) for the US 101 freeway are included in **Table 3**.

In the weekday AM peak hour simulation, the model meets the traffic volume calibration criteria at all seven locations in the southbound direction. On average, the simulation is within one percent of the observed traffic demand. The model meets the traffic volume calibration criteria at all eight locations in the northbound direction. On average, the simulation is within two percent of the observed traffic demand. The traffic volumes calibration criteria is met at 100 percent of the locations in the weekday AM peak hour simulation.

In the weekday PM peak hour simulation, the model meets the traffic volume calibration criteria at all seven locations in the southbound direction. On average, the simulation is within one percent of the observed traffic demand. The model meets the traffic volume calibration criteria at all eight locations in the northbound direction. On average, the simulation is within two percent of the observed traffic demand. The traffic volumes calibration criteria is met at 100 percent of the locations in the weekday PM peak hour simulation.

3.6 Model Validation
The CORSIM simulation model was developed based on the procedure provided by the FHWA and is based on the best practices for simulation modeling. The process consisted of work prior to modeling, initial modeling, and final modeling.

The initial model development process consisted of the development of a link-node diagram, lane geometry, free-flow link speeds, a traffic volumes database, and traffic control devices. Error checking was conducted by reviewing the physical model inputs and reviewing the animation.

Figure 6: Simulation Model Freeway Travel Speeds

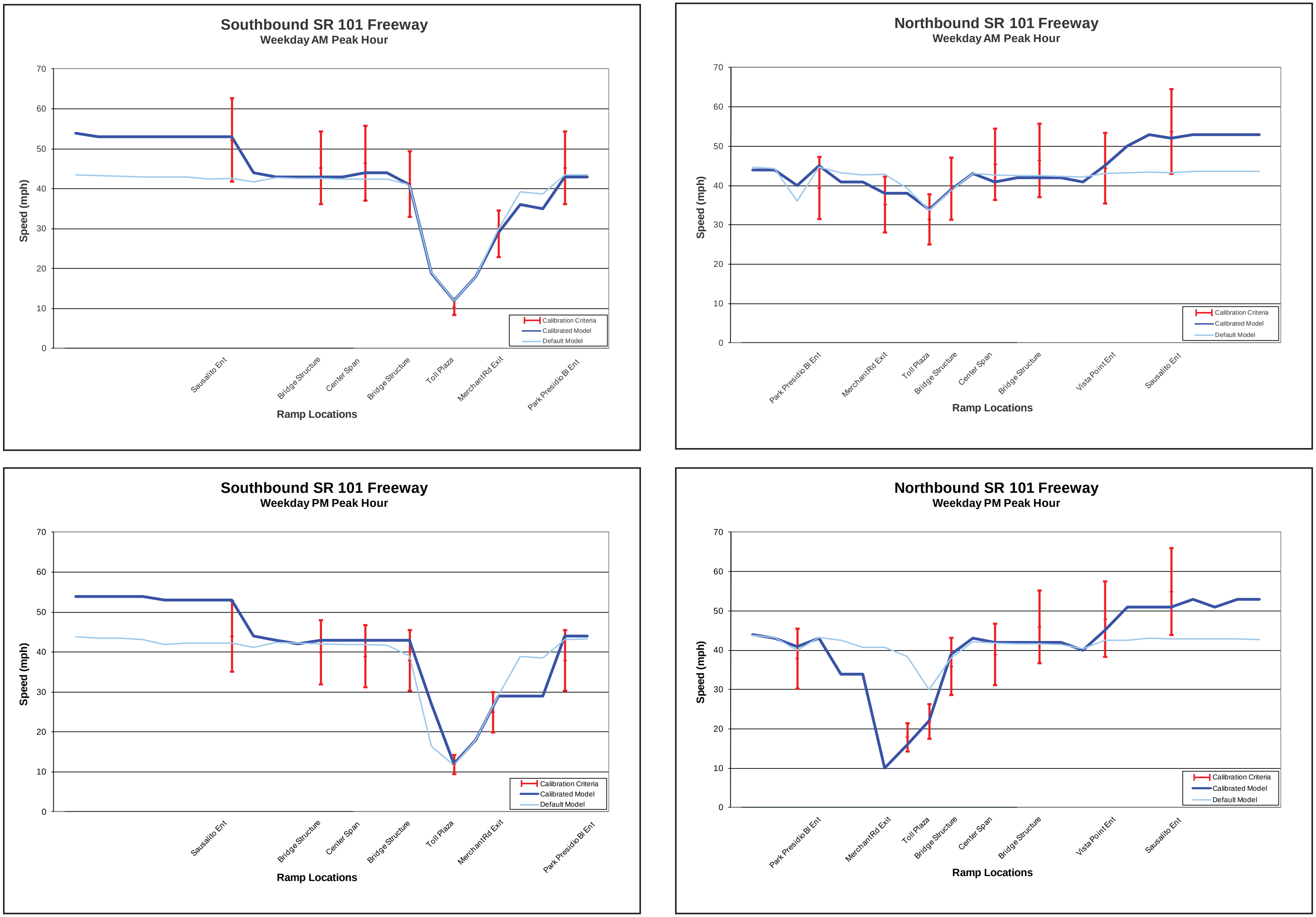


Table 3: Simulation Model Freeway Traffic Volumes

Freeway Section		Weekday AM Peak Hour			Weekday PM Peak Hour		
		Demand	Served	Difference	Demand	Served	Difference
Southbound US 101 Freeway							
1	Sausalito Lateral On-Ramp	4,900	4,910	10	3,150	3,155	5
2	Golden Gate Bridge Entrance (north)	5,182	5,189	7	3,524	3,530	6
3	Golden Gate Bridge Center Span	5,182	5,190	8	3,524	3,533	9
4	Golden Gate Bridge Exit (south)	5,182	5,198	17	3,524	3,535	11
5	Golden Gate Bridge Toll Plaza	5,182	5,197	15	3,524	3,539	15
6	Merchant Rd Off-Ramp	4,946	5,005	59	3,408	3,431	23
7	Park Presidio Bl On-Ramp	3,617	3,660	43	1,964	1,956	8
Northbound US 101 Freeway							
1	Park Presidio Bl On-Ramp	2,787	2,798	11	3,564	3,564	0
2	Merchant Rd Off-Ramp	2,787	2,797	10	3,564	3,567	3
3	Golden Gate Bridge Toll Plaza	3,023	2,939	84	5,171	5,082	89
4	Golden Gate Bridge Entrance (south)	3,023	2,942	81	5,171	5,079	92
5	Golden Gate Bridge Center Span	3,023	2,947	76	5,171	5,076	95
6	Golden Gate Bridge Exit (north)	3,023	2,953	70	5,171	5,074	97
7	Vista Point On-Ramp	3,023	2,951	72	5,171	5,071	100
8	Sausalito Lateral On-Ramp	2,896	2,831	65	5,044	4,947	97

Source: AECOM – February 2013

Notes:

- Volume in terms of vehicles per hour.
- Demand represents the peak hour mainline traffic volume demand.
- Served represents the peak hour mainline traffic demand volume that is served in the freeway simulation.
- Where the demand volume is greater than 2,700 vph, the difference represents the absolute peak hour mainline traffic volume difference between the demand traffic volume and the served traffic volume (400 vph criteria).
- Where the demand volume is less than or equal to 2,700 vph, the difference represents the percentage difference between the demand traffic volume and the served traffic volume (15 percent criteria).
- **Bold** denotes calibration target not met.

The model calibration process conditionally consisted of modifying the known global parameters, modifying the local parameters, and modifying the unknown global parameters. The model was calibrated by modifying the link travel speeds, the warning sign locations, and the car following factor.

Travel speeds were utilized as a quantitative calibration parameter for the CORSIM model. In the weekday AM peak hour simulation, the travel speed calibration criteria is met at 100 percent of the locations. In the weekday PM peak hour simulation, the travel speed calibration criteria is met at 100 percent of the locations.

Traffic volumes were utilized as a quantitative calibration parameter for the CORSIM model. In the weekday AM peak hour simulation, the traffic volumes calibration criteria is met at 100 percent of the locations. In the weekday PM peak hour simulation, the traffic volumes calibration criteria is met at 100 percent of the locations.

The development and calibration of the Existing Conditions CORSIM model is consistent with the methodology and procedure recommended by the FHWA. Based on the results of the regional freeway calibration analysis, the CORSIM model meets the recommended calibration criteria. The Existing Conditions CORSIM model is therefore valid for design year alternatives analyses and the final report.

4.0 Existing (2012) Conditions

The regional freeway Existing Conditions analysis included freeway facilities on US 101 between the Sausalito Lateral Interchange and the GGB and the GGB and the Park Presidio Boulevard Interchange. The GGB and US 101 freeway between the Sausalito Lateral Interchange and Park Presidio Boulevard is typically a six lane facility and is approximately 2.7 miles long. The northbound freeway section includes four on-ramps and four off-ramps. The southbound freeway section includes three on-ramps and three off-ramps.

4.1 Traffic Volumes

Existing Conditions traffic volumes were collected on the GGB and ramps within the evaluation area. This data includes peak hour traffic volumes, heavy vehicle volumes, hourly volumes, and historical average daily traffic volumes.

Weekday Peak Hour Freeway Traffic Volumes

Freeway traffic volumes on the Golden Gate Bridge and the north and south approaches to the GGB were obtained from the *Freeway Performance Measurement System* (PeMS).[viii] Traffic volumes were collected in the northbound and southbound directions between Saturday, May 19, 2012 and Friday, May 25, 2012. There are many PeMS traffic detectors located various distances to the north and south of the Golden Gate Bridge. In order to ensure the most reliable data was used for this Traffic Report, the northbound and southbound traffic volumes to the north of the Golden Gate Bridge were taken from two different detector locations. These locations were utilized as the "control points" for traffic volumes in each direction. The reasoning for using these two locations is as follows.

Detector health and data quality information was obtained from PeMS in order to ensure that the traffic volumes used in the analysis were accurate and valid. Detector health involves whether the detector actually measures what occurs on the ground and whether that data is sent to the controller and then on to the District Traffic Management Center. Each detector has diagnostics performed on it every day and a determination is made as to whether a detector is "good" or "bad". Detectors are determined to be "bad" if: 1) data for that detector was not received for a particular time period, or 2) data was received but it is believed that the data is bad as indicated by the diagnostic routines. In either case, imputation methods are applied to fill in the missing data. Imputation algorithms supplement the data with accurate estimates based on data from surrounding detectors in other lanes at the same location and from immediately upstream and downstream detectors.

For the detectors at CA PM MRN 0.30 = VDS 402553 (NB), the Caltrans PeMS information indicated all vehicle detectors were active and "good" detector health was reported across all traffic lanes. Therefore, data collected at VDS 402553 was determined to be valid and was utilized in the analysis.

For the detectors at CA PM MRN 0.30 = VDS 402554 (SB), the Caltrans PeMS information indicated that the lane 1 detector status was "Card Off," and the lane 4 detector status was "Intermittent." In situations like this, PeMS uses imputation methods to fill in the missing data. Therefore, even though the detector status at some lanes may not be registering as "Good", the data itself is automatically adjusted and may be used for performing a traffic analysis. While the data from this detector is considered valid and may be used for traffic evaluations, it was noted that another nearby detector, at CA PM MRN 0.80 = VDS 402556 (SB), showed all vehicle detectors as active and "good" detector health was reported across all traffic lanes. This detector is located close to the Golden Gate Bridge and just 0.5 miles north of detector VDS 402554. Rather than using data from a location where some of the data was determined through imputation, it was determined that data collected at VDS 402556 would be utilized in the analysis for SB traffic volumes since all of its lane detectors are shown to be active and in good health.

Ramp traffic volumes were obtained from Caltrans for all of the ramps between the Sausalito Lateral Interchange and Park Presidio Boulevard.[ix] The ramp traffic volumes were collected between March 2004 and October 2010 (most recent data available). Ramp traffic volumes were escalated to 2012 conditions. For the purposes of consistency, weekday peak period data collected on Wednesdays or Thursdays were utilized for this analysis.

The freeway and ramp volumes were adjusted based on seasonal traffic volume variation to represent average daily traffic volumes within the evaluation area. The seasonal traffic volume variation factors were developed based on statewide monthly vehicle miles of travel data. The ramp traffic volumes were escalated to 2012 conditions based on observed annual growth rates. The data collection date, source, and adjustment factor for the freeway ramps are included in **Table 4**.

Table 4: Ramp Data Collection Summary

Freeway Location		Count Date	Escalated To	Data Source	Seasonal Adjustment Factor[a]	Growth Adjustment Factor[b]
1	Sausalito Lateral SB Off-Ramp	Wednesday, October 6, 2010	2012	Caltrans	1.01	1.13
2	Sausalito Lateral SB On-Ramp	Wednesday, April 21, 2010	2012	Caltrans	1.00	1.03
3	Merchant Rd. SB Off-Ramp	Wednesday, March 10, 2004	2012	Caltrans	1.02	1.05
4	Merchant Rd. SB On-Ramp	Wednesday, March 10, 2004	2012	Caltrans	1.02	1.05
5	Park Presidio Bl. SB Off-Ramp	Wednesday, January 24, 2007	2012	Caltrans	1.11	1.03
6	Park Presidio Bl. SB On-Ramp	Wednesday, January 24, 2007	2012	Caltrans	1.11	1.03
7	Park Presidio Bl. NB Off-Ramp	Wednesday, April 7, 2004	2012	Caltrans	1.00	0.98
8	Park Presidio Bl. NB On-Ramp	Wednesday, March 17, 2004	2012	Caltrans	1.02	0.98
9	Merchant Rd. NB Off-Ramp	Thursday, September 20, 2007	2012	Caltrans	0.99	1.31
10	Merchant Rd. NB On-Ramp	Wednesday, April 14, 2004	2012	Caltrans	1.00	1.08
11	Vista Point NB Off-Ramp	Thursday, September 16, 2010	2012	Caltrans	0.99	1.02
12	Vista Point NB On-Ramp	Wednesday, September 15, 2010	2012	Caltrans	0.99	1.02
13	Sausalito Lateral NB Off-Ramp	Wednesday, February 3, 2010	2012	Caltrans	1.09	1.02
14	Sausalito Lateral NB On-Ramp	Wednesday, February 3, 2010	2012	Caltrans	1.09	1.10

Source: AECOM – February 2013

Notes:

[a] Seasonal adjustment factor calculated based on statewide monthly vehicle miles traveled data obtained from Caltrans.

[b] Growth adjustment factor calculated based on observed annual growth on the ramps extrapolated to 2012. Observed annual growth on the mainline between 2004 and 2012 is applied to the Park Presidio NB ramps. Observed annual growth on the mainline between 2009 and 2012 is applied to the Park Presidio SB ramps and Merchant Road SB ramps.

The Existing Conditions freeway traffic volumes at the northern and southern Project limits are shown in **Figure 7**.

Weekday Peak Hour GGB Heavy Vehicle Volumes

Detailed weekday peak hour heavy vehicle composition data by lane on the Golden Gate Bridge was collected as part of the *Golden Gate Bridge Moveable Median Barrier Feasibility Studies – Phase 2* (dated January 2002).[x] This document is herein referred to as the "Feasibility Study". The weekday peak hour heavy vehicle composition data was used to determine the heavy vehicle distribution by lane.

Figure 7a: Existing Conditions Freeway Traffic Volumes - Northern Approach

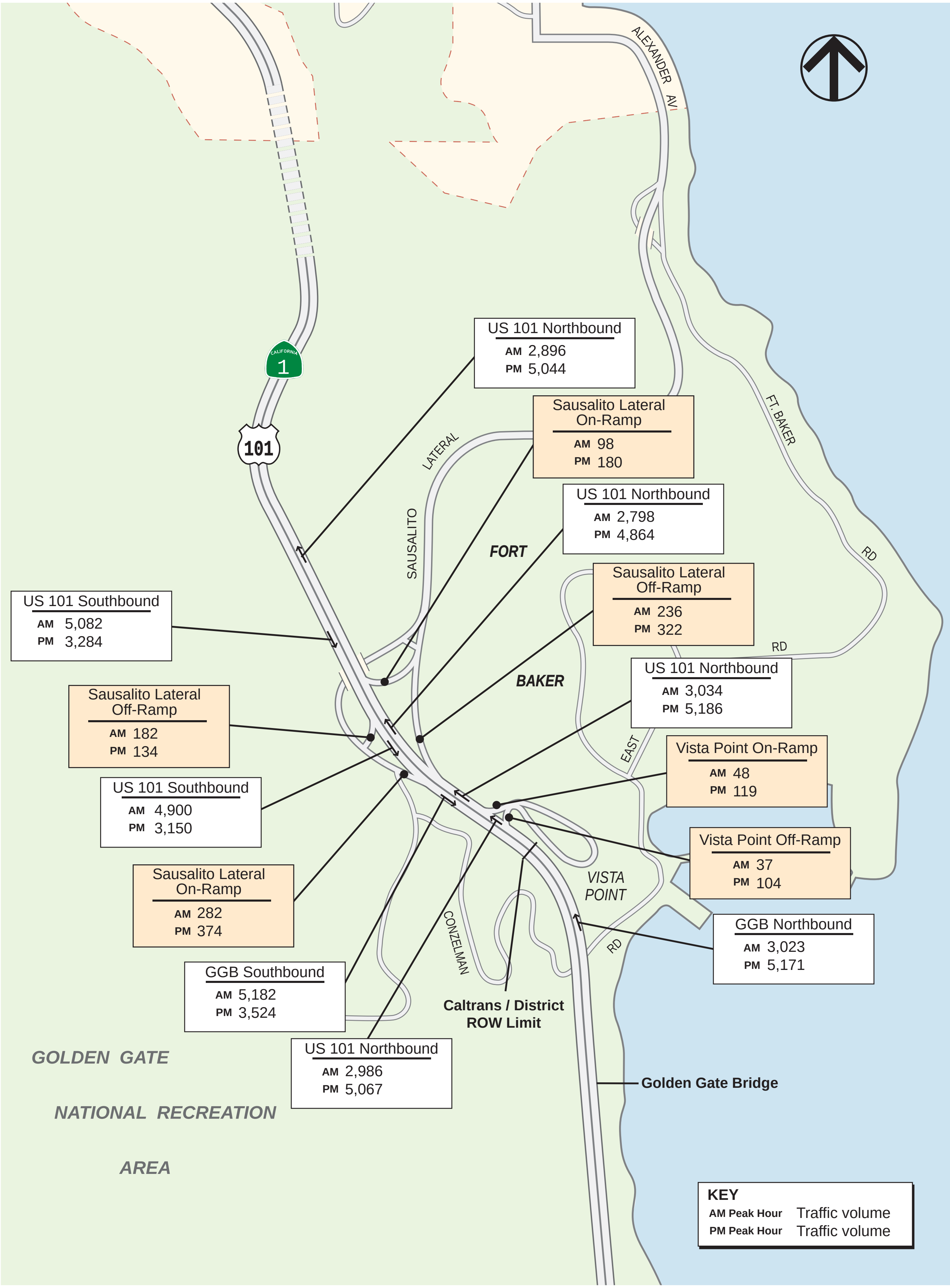
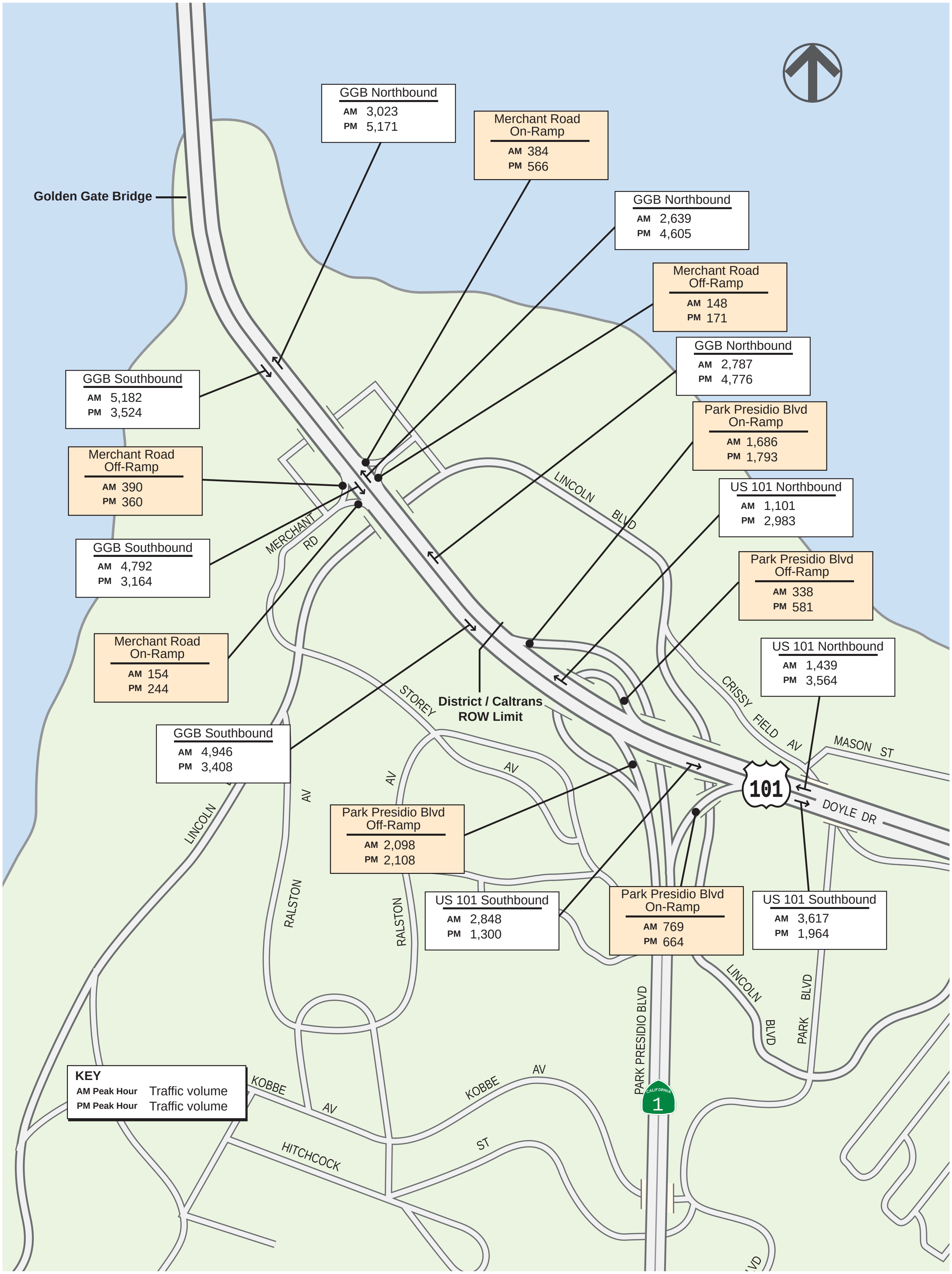


Figure 7b: Existing Conditions Freeway Traffic Volumes - Southern Approach



The GGB truck and bus traffic during the weekday peak hours varies from 1.9 percent to 2.6 percent of the total traffic. The heavy vehicle composition distributed by lane is included in **Table 5**.

Table 5: Heavy Vehicle Composition by Lane

Configuration	Lane Position	Heavy Vehicle Percentage	
		Southbound	Northbound
2 Lanes	Curb	4.0%	7.7%
	Inside	0.0%	0.1%
3 Lanes	Curb	6.6%	6.0%
	Middle	0.5%	0.0%
	Inside	0.0%	0.0%
4 Lanes	Curb	9.7%	9.0%
	Middle-R	0.4%	1.0%
	Middle-L	0.1%	0.0%
	Inside	0.0%	0.0%

Source: Feasibility Study, Parsons Brinckerhoff – January 2002.

The curb lanes accommodate the highest percentage of heavy vehicles in all three lane configurations. The highest observed percentage of heavy vehicles was 9.7 percent, which occurred in the southbound curb lane in the four lane configuration.

Weekday Maximum Hourly GGB Traffic Volumes
Observed weekday maximum hourly traffic volumes by lane on the Golden Gate Bridge was collected as part of the Feasibility Study.

The observed weekday maximum hourly traffic volumes data by lane varies from 1,020 vehicles per hour (vph) to 2,030 vph. It should be noted that weekday maximum hourly GGB traffic volumes were utilized to conduct a capacity analysis of the GGB. The weekday maximum hourly traffic volume data by lane was observed when the GGB was at capacity. The maximum observed traffic volumes per lane data was used to determine the lane capacity and shy distance capacity reductions and were not used in the operations analysis. The observed weekday maximum hourly traffic volumes by lane is included in **Table 6**.

Although data were collected in all three of the possible configurations in the northbound and southbound directions, the data were likely distorted based on upstream capacity constraints, vehicle positioning requirements, and heavy vehicle compositions. Based on these factors, the following data were determined to be biased:

- The southbound data are biased as southbound vehicles on the GGB are positioning themselves prior to approaching the Toll Plaza. Motorists typically utilize the lanes based on the toll collection type. For example, the vehicles destined for FasTrak-usage typically position themselves in the inside lane as the FasTrak lanes are located to the left of the Toll Plaza.
- The two lane configuration data are biased as the heavy vehicle composition between the two lanes varies significantly. For example, in the northbound two lane configuration the heavy vehicle percentages in the curb and inside lanes are 7.7 percent and 0.1 percent respectively. Lanes accommodating high volumes of heavy vehicles provide less capacity than lanes without heavy vehicles.

Table 6: Maximum Traffic Volume Data by Lane

Configuration	Lane Position	Maximum Hourly Traffic Volumes	
		Southbound	Northbound
2 Lanes	Curb	1,860	1,580
	Inside	1,520	1,870
	Total (Average)	**3,380 (1,690)**	**3,450 (1,725)**
3 Lanes	Curb	1,540	1,780
	Middle	1,020	1,760
	Inside	1,290	1,630
	Total (Average)	**3,850 (1,285)**	**5,170 (1,725)**
4 Lanes	Curb	1,540	1,720
	Middle-R	1,380	1,500
	Middle-L	1,540	1,600
	Inside	2,030	1,350
	Total (Average)	**6,490 (1,625)**	**6,170 (1,545)**

Source: Feasibility Study, Parsons Brinckerhoff – January 2002.

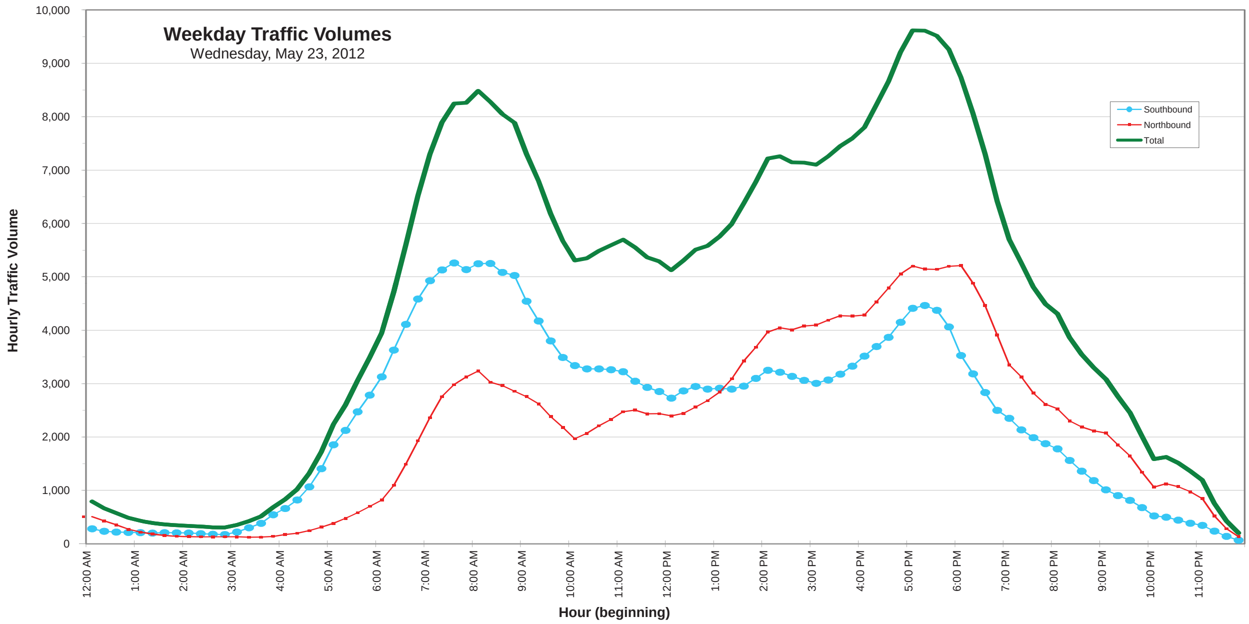
- The four lane configuration data are biased as the northbound traffic volumes are constrained by the upstream geometric conditions. For example, the maximum observed northbound traffic volumes per lane in the four lane configuration was 1,545 vph whereas the maximum observed northbound traffic volumes per lane in the three lane configuration was 1,725 vph.

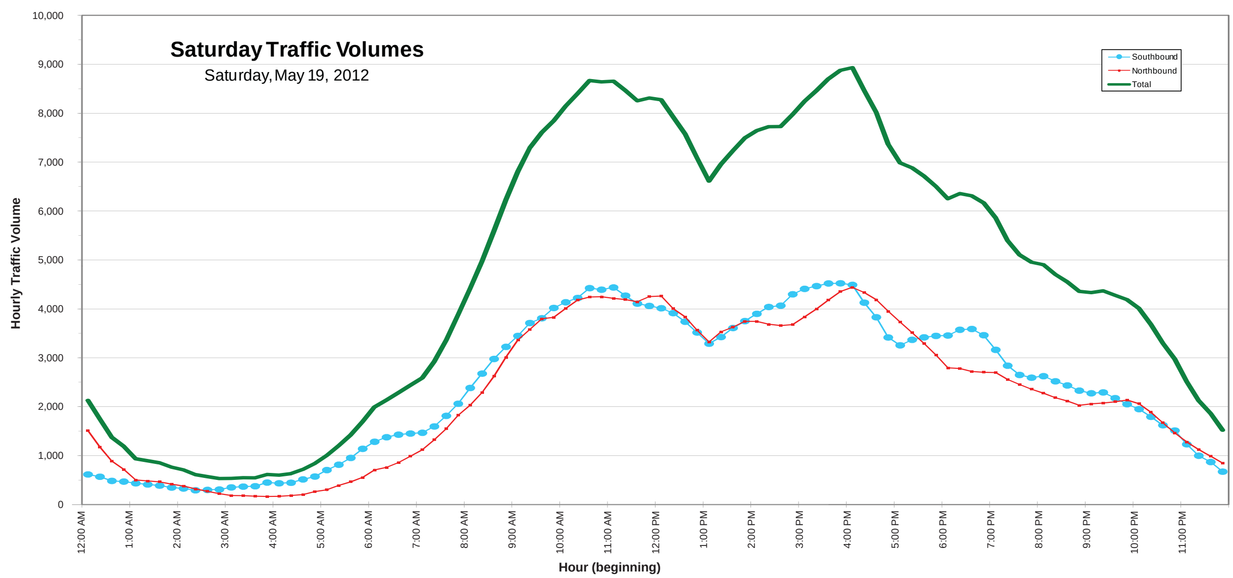
Based on this evaluation, the maximum observed traffic volumes per lane in the northbound three lane configuration are representative of the GGB capacity. From the maximum observed traffic volumes per lane in the northbound three lane configuration data, the following assumptions are derived:

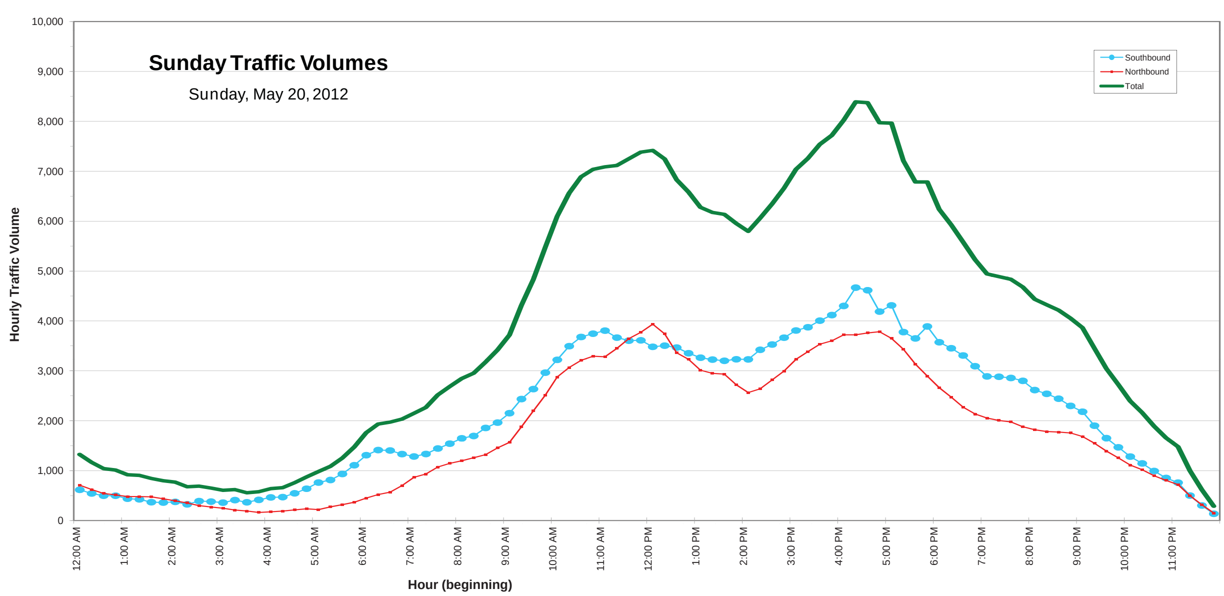
- Given the existing lane widths, heavy vehicle percentages, and the absence of a physical barrier, the maximum GGB capacity per lane in the three lane configuration is 1,725 vph – or a total of approximately 5,170 vph in the northbound direction.
- Fewer vehicles utilize the inside lane (1,630 vph) than the middle lane (1,760 vph) due to the effect of shy distance. The heavy vehicle composition in both of these lanes is zero percent. Shy distance effects the capacity of the GGB as vehicles are less likely to utilize the inside lane due to the perception that traffic traveling in the opposite direction could potentially cross the pylon median. This causes motorists to drive more cautiously or use a different lane, which results in a lower lane capacity. The absence of a physical median barrier on the GGB results in a shy distance capacity reduction of 130 vph. This factor is applicable to both directions in all three configurations.

Hourly GGB Traffic Volumes
Hourly freeway traffic volumes on the Golden Gate Bridge were obtained for weekday, Saturday, and Sunday from PeMS. The traffic volumes were collected in the northbound and southbound directions between Saturday, May 19, 2012 and Friday, May 25, 2012. During the weekday AM peak hour (8:00am to 9:00am), the southbound traffic volume is approximately 5,200 vph and northbound traffic volume is approximately 3,000 vph. The total traffic volume is approximately 8,200 vph. The hourly traffic volume data represents a 24-hour data collection period and are shown in **Figure 8**.

Figure 8: Existing Conditions Freeway Traffic Volumes







During the Saturday peak hour (4:00pm to 5:00pm), the southbound traffic volume is approximately 3,600 vph and northbound traffic volume is approximately 4,400 vph. The total traffic volume is approximately 8,000 vph. This is the maximum hourly traffic volume on the Golden Gate Bridge on a Saturday.

During the Sunday peak hour (4:00pm to 5:00pm), the southbound traffic volume is approximately 4,000 vph and northbound traffic volume is approximately 3,700 vph. The total traffic volume is approximately 7,700 vph. This is the maximum hourly traffic volume on the Golden Gate Bridge on a Sunday.

Historical Average Daily Traffic Volumes
Annual average daily traffic volumes on the Golden Gate Bridge were obtained from the District. The average annual daily traffic volumes on the Golden Gate Bridge from fiscal year 1980 to 2011 are shown in **Figure 9**.

The highest average daily traffic volume on the Golden Gate Bridge was 120,276 daily vehicles, which occurred in 1989. Between 1980 and 1989 the average daily traffic volume on the GGB increased from 97,080 daily vehicles to 120,276 daily vehicles – an annual increase of 2.7 percent. Since 1989, the average daily traffic on the GGB has decreased from 120,276 daily vehicles to 110,113 daily vehicles – an annual decrease of 0.4 percent.

4.2 Lane Configurations

An assessment of the existing geometry and operations of the GGB was conducted. This assessment includes the schedule and procedure for reversing the direction of the lanes on the GGB and the existing lane widths.

Reversible Lane Operations
The directional flow of traffic on the Golden Gate Bridge varies throughout the day. Typically, traffic volumes in the southbound direction are highest during the weekday AM peak hour and traffic volumes in the northbound direction are highest during the weekday PM peak hour. During the weekday AM peak hour traffic volumes are approximately 3,000 vehicles and approximately 5,200 vehicles in the northbound and southbound directions, respectively. During the weekday PM peak hour traffic volumes are approximately 5,200 vehicles and approximately 3,500 vehicles in the northbound and southbound directions, respectively.

To maximize the utilization of the GGB, given the dynamic direction flow of traffic, the center two lanes of traffic are reversible. The direction of traffic flow is delineated by plastic pylons that must be manually removed and reinstalled to reverse the direction of a lane. These lanes are also reversed on Doyle Drive between the Park Presidio Boulevard Interchange and Marina Boulevard (0.9 mile). For a crew to reverse the direction of a lane between the Sausalito Lateral interchange and Marina Boulevard, approximately 30 minutes is required – this time may increase in heavy traffic conditions. During this time period one lane is effectively closed.

Golden Gate Bridge lane configurations are established based on time-of-day traffic demand trends that are constantly being monitored and evaluated by the District. Lane configurations can also be changed to accommodate extenuating circumstances such as accidents or large special events south of the GGB. This is a delicate balancing exercise that becomes very difficult when demand is heavy in both directions at once, or when stalls, accidents, or other incidents on the roadway interfere with the normal flow of traffic in either direction.

On typical weekdays the lane configuration on the GGB is two northbound lanes / four southbound lanes during the weekday AM peak hour and either three northbound lanes / three southbound lanes or four northbound lanes / two southbound lanes during the weekday PM peak hour. The lane configuration schedule for typical weekday and weekends is shown in **Figure 10**.[xi]

Figure 9: Historical Average Daily Traffic Volumes

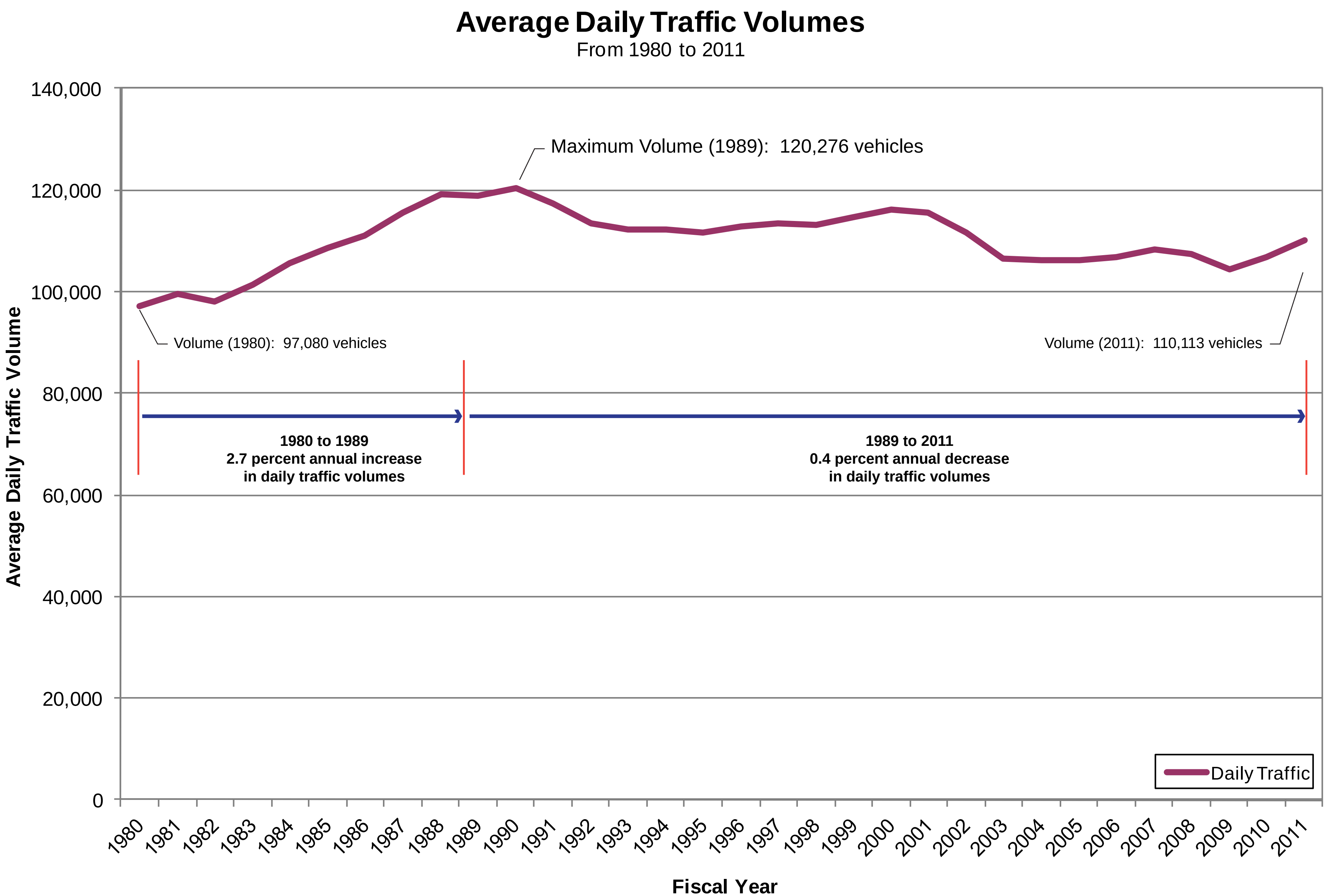
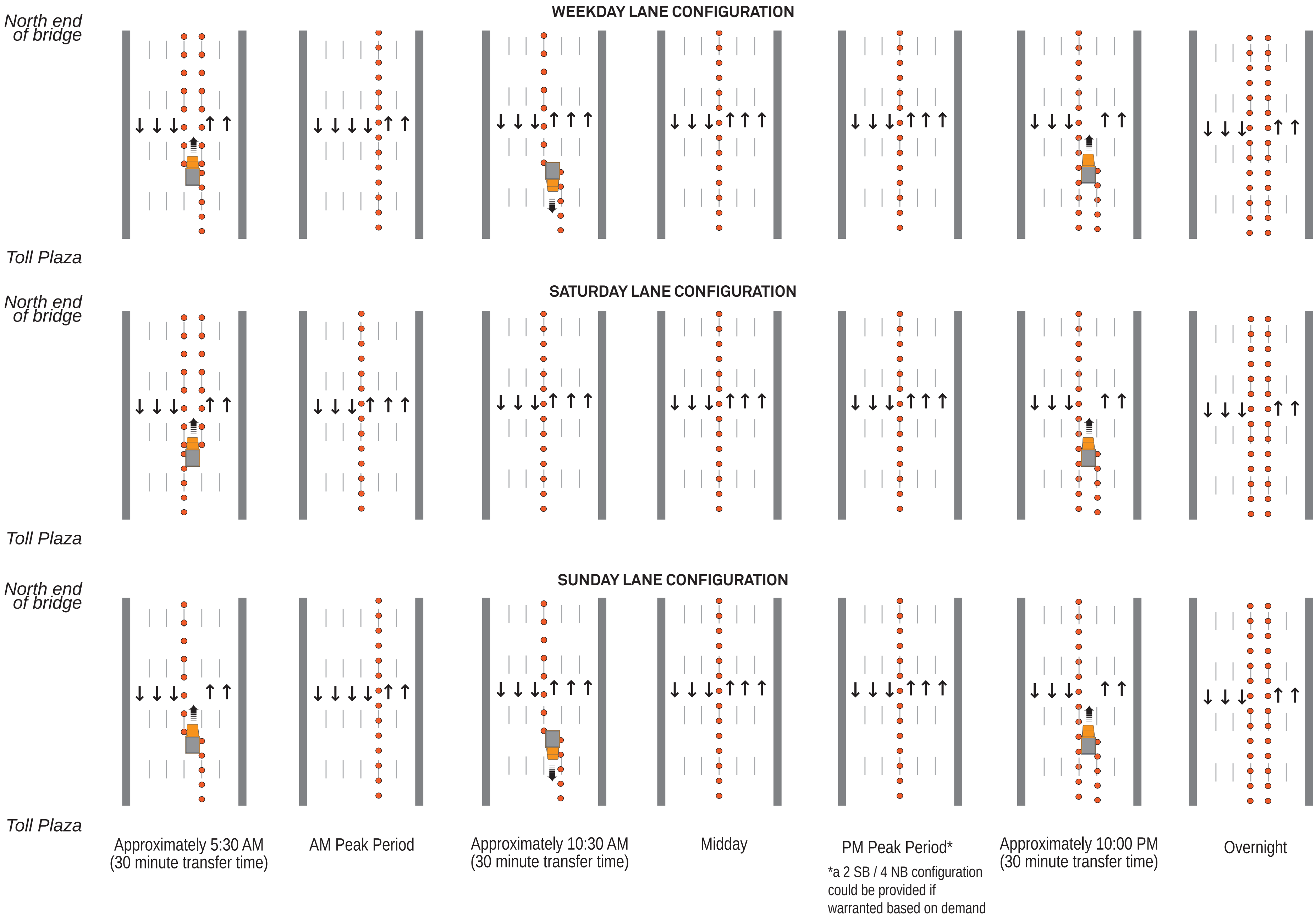


Figure 10: Existing Conditions Roadway Configurations - Golden Gate Bridge



The four northbound lanes / two southbound lanes configuration is utilized during the weekday PM peak period and as needed due to the occurrence of accidents or large special events to the south of the GGB on weekdays and weekends. Additionally, this configuration was the standard afternoon configuration prior to 2002 and briefly between 2009 and 2010.

Lane Widths
The Golden Gate Bridge mid-span roadway is 62 feet wide and accommodates private vehicles, trucks, and buses. A Toll Plaza is located on the southern side of the GGB and includes 11 tollbooths that serve southbound traffic. To accommodate the larger, heavy vehicles, the curb lanes on the GGB are 11’-0” wide (the four interior lanes are 10’-0” wide). The lane widths for each of the three lane configurations are shown in **Figure 11**.

Lane Capacities
The capacity of individual lanes is affected by several factors, including: lane width, closeness of barriers to the travel way, whether or not there is a barrier between the directions of travel, and on the percentage of heavy vehicles. A detailed estimate of the GGB lane capacities was developed utilizing the *1994 HCM* based on traffic volume data specific to the GGB – specifically, the maximum observed traffic volumes in the northbound three lane configuration. The estimated traffic capacities of the individual lanes on the GGB in the Existing Conditions are included in **Table 7**.

Table 7: Estimated Traffic Capacity – Existing Conditions

Configuration	Lane Position	Existing Conditions	
		Width	Capacity
2 Lanes	Curb	11’-0”	1,680
	Inside	10’-0”	1,590
	Total	**21’-0”**	**3,270**
3 Lanes	Curb	11’-0”	1,780
	Middle	10’-0”	1,760
	Inside	10’-0”	1,630
	Total	**31’-0”**	**5,170**
4 Lanes	Curb	11’-0”	1,790
	Middle-R	10’-0”	1,770
	Middle-L	10’-0”	1,770
	Inside	10’-0”	1,640
	Total	**41’-0”**	**6,970**

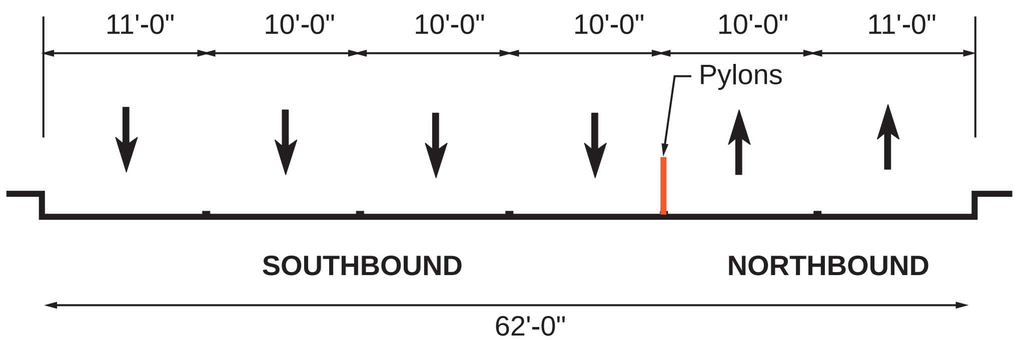
Source: AECOM – February 2013
Notes:
- Capacity in terms of vehicles per hour.

The Existing Conditions lane capacities were calibrated based on observed weekday maximum hourly traffic volumes data by lane on the Golden Gate Bridge in the northbound three lane configuration.

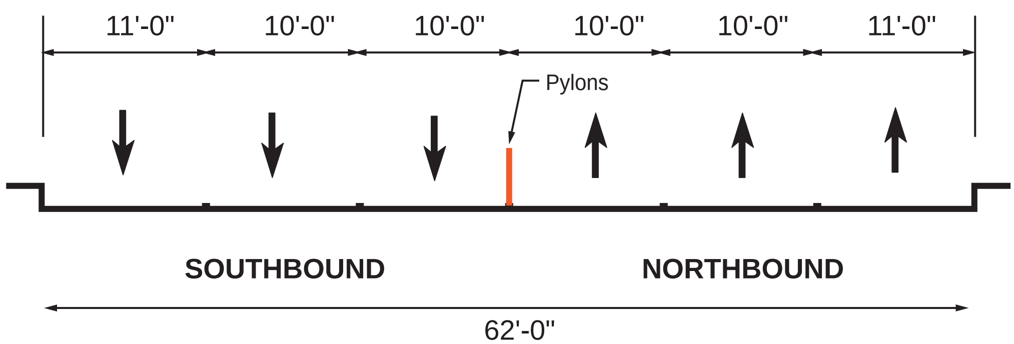
The detailed lane capacity calculations are included in **Appendix C**.

Figure 11: Existing Conditions Roadway Cross Sections

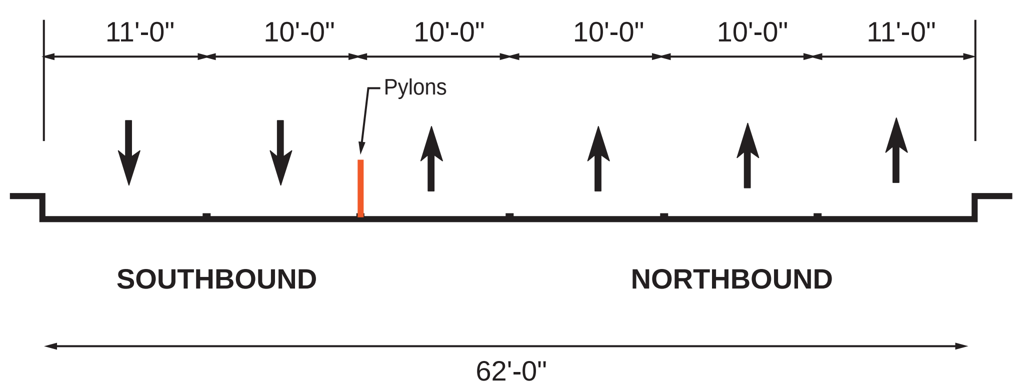
4 Southbound / 2 Northbound Lane Configuration



3 Southbound / 3 Northbound Lane Configuration



2 Southbound / 4 Northbound Lane Configuration



NO SCALE

4.3 Field Observations

The GGB and portions of US 101 freeway between the Sausalito Lateral Interchange and the Park Presidio Boulevard Interchange includes many features that reduce the roadway capacity. The locations, types, and characteristics of these features are shown in **Figure 12**. A photo log of the existing freeway network is shown in **Figure 13**.

Roadway Grade

A steep roadway grade – known as the Waldo Grade – exists to the north of the Golden Gate Bridge between the Vista Point Ramps and the Waldo Tunnel. The grade is approximately six percent and extends for 0.8 mile. This section of roadway is eight lanes wide (four lanes in each direction) and the speed limit is 55 mph. Despite the grade being significant, the effects on traffic operations are small as northbound traffic entering the Waldo Grade from the GGB typically increase travel speeds (posted speed limit increases) and increase roadway capacity (additional lane(s)). In the southbound direction the effect of the downgrade is minimal as upstream travel speeds decrease (posted speed limit decreases) and adequate sight lines are provided on the roadway segment.

Roadway grades elsewhere in the evaluation area are less than two percent and do not significantly affect traffic operations.

Horizontal Curvature

Sharp horizontal curvature on the GGB and mainline US 101 freeway limits the sight distance and speeds of motorists and results in increased traffic congestion and reduced roadway capacity. The locations of sharp horizontal curvature, and the corresponding effects, include:

- The freeway between the Park Presidio On- and Off-Ramps includes a substantial horizontal curve where vehicles frequently slow. In the northbound direction this segment of the freeway frequently becomes congested as vehicles slow and change lanes in anticipation of the upstream merges prior to the GGB. In the southbound direction, this segment of the freeway frequently becomes congested as vehicles slow and change lanes in anticipation of the upstream Park Presidio Boulevard Ramps. Additionally, frequent lane maneuvers occur on this roadway segment as vehicles depart from the Toll Plaza (up to 11 lanes) and approach the Park Presidio Boulevard Ramps.
- The freeway between the Toll Plaza and the southern approach to the GGB includes a horizontal curve. In the northbound direction, this segment of the freeway frequently becomes congested as vehicles approach the GGB. In addition to the horizontal curve, the narrow lanes (approximately 10 feet wide) further increase congestion as motorists are cautious to remain within the lane. In the southbound direction, this segment of the freeway becomes congested as vehicles slow and change lanes in anticipation of the upstream Toll Plaza. Motorists frequently change lanes to enter the tollbooths with the shortest vehicle queues or the tollbooths with the appropriate toll collection type.
- The freeway between the northern approach to the GGB and the Sausalito Lateral Ramps includes a horizontal curve. Despite the presence of the horizontal curve, the effects on the northbound traffic operations within this segment of the roadway are negligible as the travel speed increases (posted speed limit increases) and roadway capacity typically increases (additional lane(s)). In the southbound direction the effects of the horizontal curve on traffic operations is minimal as the roadway segment is on a steep downgrade and the upstream lane capacity typically decreases (reduced lane(s)).

Toll Plaza

The Toll Plaza is located to the south of Golden Gate Bridge south abutment and serves southbound vehicles. The current configuration includes a total of 11 tollbooths. The tollbooths are numbered from Tollbooth 1 to Tollbooth 11, where Tollbooth 1 is the rightmost tollbooth for approaching southbound traffic. Generally, the nine rightmost tollbooths are active to serve southbound vehicles regardless of the lane configuration. Given the vehicle demand on the GGB and the FasTrak patronage, no more than 10 tollbooths are usually active.

Figure 12a: Existing Conditions Freeway Operations - Geometric Features Northern Approach

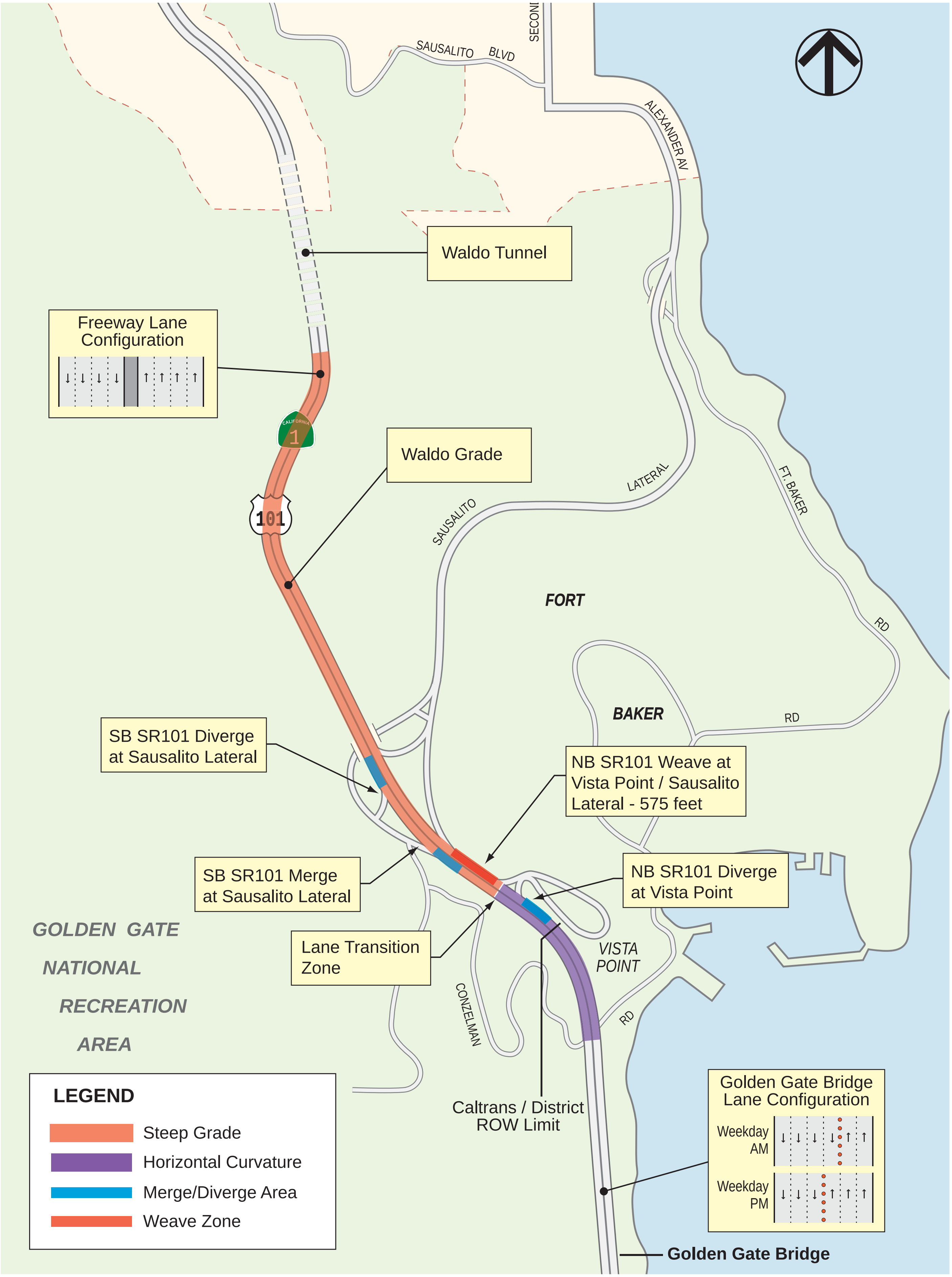


Figure 12b: Existing Conditions Freeway Operations - Geometric Features Southern Approach

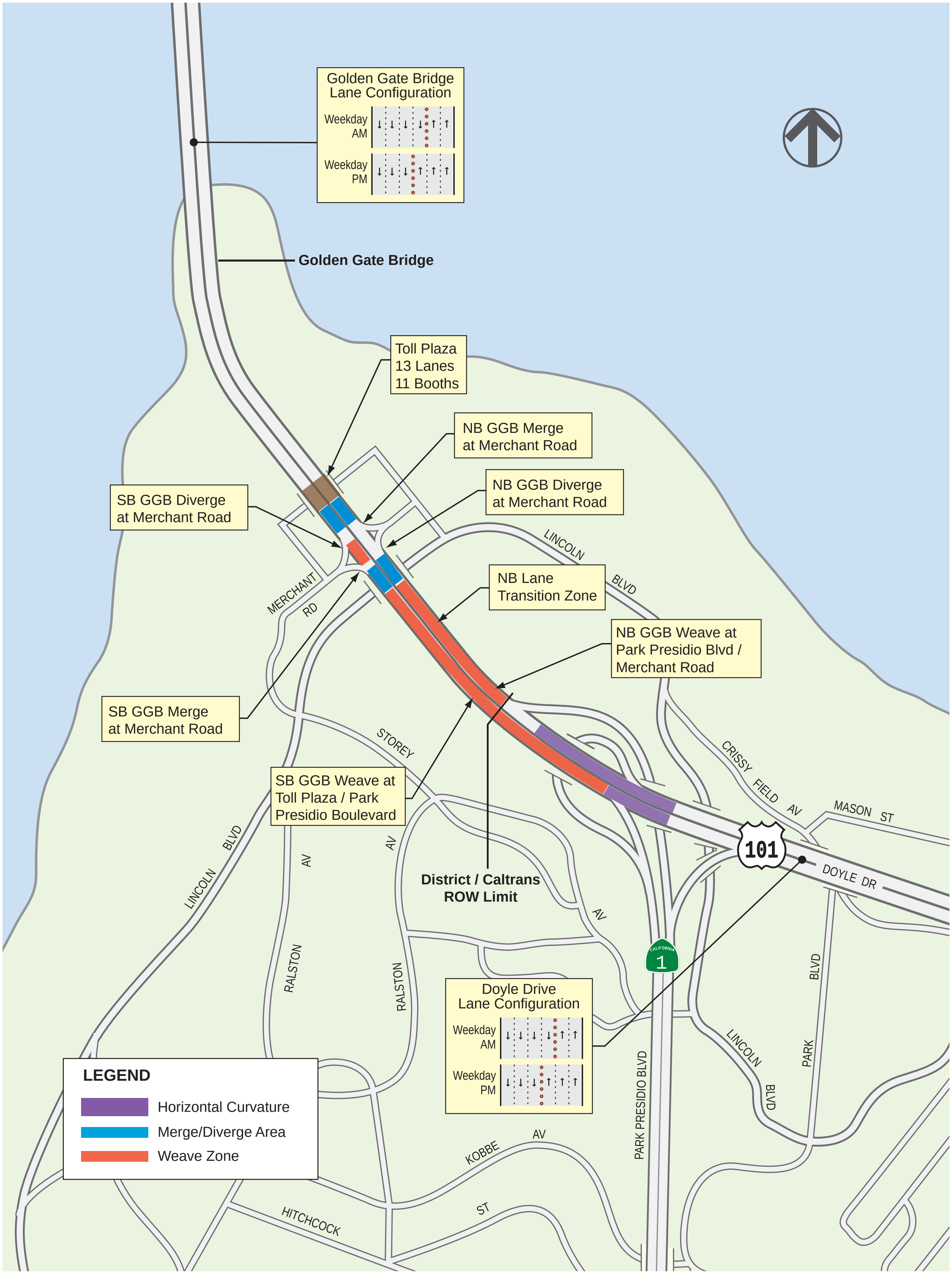


Figure 13a: Existing Conditions Photograph Log - Weekday AM Peak Hour

Figure 13b: Existing Conditions Photograph Log - Weekday AM Peak Hour

Figure 13c: Existing Conditions Photograph Log - Weekday AM Peak Hour

Figure 13d: Existing Conditions Photograph Log - Weekday PM Peak Hour

Figure 13e: Existing Conditions Photograph Log - Weekday PM Peak Hour

Figure 13f: Existing Conditions Photograph Log - Weekday PM Peak Hour

The toll collection type varies; however, lanes dedicated for FasTrak users are always the leftmost of the active tollbooths. Based on the lane configuration, the Toll Plaza functions as follows:

- During periods where the lane configuration consists of two northbound lanes / four southbound lanes, 10 tollbooths are typically active and one is inactive. No traffic is permitted to use the easternmost tollbooth in this configuration.
- During periods where the lane configuration consists of three northbound lanes / three southbound lanes, nine tollbooths are typically active and two are inactive. The traffic flow through the easternmost two tollbooths is reversed to accommodate northbound traffic.
- During periods where the lane configuration consists of four northbound lanes / two southbound lanes, nine tollbooths are typically active and two are inactive. The traffic flow through the easternmost two tollbooths is reversed to accommodate northbound traffic.

The Toll Plaza directly affects southbound roadway operations as vehicles slow (FasTrak users) or stop (cash and carpool users) at the tollbooths. During the weekday AM and PM peak hours, average traffic speeds were observed to slow to between 10 mph and 15 mph approaching the Toll Plaza; however, the queue lengths at the tollbooths were not observed to exceed three vehicles at any point in time and Toll Plaza operations did not limit the freeway capacity in the southbound direction.

The Toll Plaza indirectly affects northbound roadway operations as vehicles slow to navigate through and around the tollbooths. The Toll Plaza has the following effects on northbound vehicle operations:

- During the weekday AM peak period the GGB is in the two northbound lanes / four southbound lanes configuration and vehicles circumvent the Toll Plaza as no lanes are reversed. This approach does have minor effects on freeway operations as vehicles slow as the lane curves (abrupt with a small radius) around the Toll Plaza. This results in a minor reduction in roadway capacity and an increase in delay.
- During the weekday PM peak hour the GGB is in a three northbound lanes / three southbound lanes configuration and northbound vehicles in the leftmost lane drive through the reversed tollbooth. Due to the narrow width of this lane (nine feet) vehicles slow as they approach the tollbooth. Coupled with the slowing of the two rightmost lanes, which circumvent the Toll Plaza, this three lane approach has moderate effects on freeway operations. This results in a moderate reduction in roadway capacity and an increase in delay.

Weaving Sections

The weaving sections on the freeway increase vehicle density and reduce traffic speeds. These characteristics result in increased traffic congestion and delay. The locations of the weaving sections, and the corresponding impacts, include:

- The section of roadway between the Northbound Route 1 On-Ramp at Park Presidio Boulevard and the Northbound Off-Ramp at Merchant Road is approximately 750 feet long. This weave is particularly challenging for motorists as the northbound mainline vehicles must traverse two lanes within the weaving section to exit the freeway. During the weekday AM peak hour 40 percent of the vehicles in this weaving section enter from Northbound US 101 and 60 percent of the vehicles enter from the Northbound Route 1 On-Ramp at Park Presidio Boulevard. Of the approximately 2,900 vph entering this section, approximately 150 vph exit at Merchant Road. During the weekday PM peak hour 60 percent of the vehicles in this weaving section enter from Northbound US 101 and 40 percent of the vehicles in enter from the Northbound Route 1 On-Ramp at Park Presidio Boulevard. Of the approximately 4,600 vph entering this section, approximately 170 vph exit at Merchant Road. In addition to vehicle weaving, this section is congested as vehicles merge as they approach the GGB. This section of roadway operates as LOS B and LOS D during the weekday AM and PM peak hours, respectively.

- The auxiliary lane within the Northbound US 101 weaving section between the Vista Point On-Ramp and the Sausalito Lateral Off-Ramp is approximately 575 feet long. The traffic volumes utilizing the Northbound US 101 On-Ramp at the Vista Point are approximately 50 vph and approximately 120 vph during the weekday AM and PM peak hours, respectively. The traffic volumes utilizing the Northbound US 101 Off-Ramp at the Sausalito Lateral are approximately 240 vph and approximately 320 vph during the weekday AM and PM peak hours, respectively. This section of roadway operates as LOS B and LOS C during the weekday AM and PM peak hours, respectively.
- The section of roadway between the Golden Gate Bridge Toll Plaza and the Southbound US 101 Off-Ramp at Park Presidio Boulevard is approximately 1,800 feet long. This weave is particularly challenging for motorists as the vehicles on US 101 must traverse up to eight lanes within the weaving section to exit the freeway. Of the approximately 5,200 vph exiting the Toll Plaza during the weekday AM peak hour, approximately 2,100 vph exit at the Southbound US 101 Off-Ramp at Park Presidio Boulevard. Of the approximately 3,500 vph exiting the Toll Plaza during the weekday PM peak hour, approximately 2,100 vph exit at the Southbound US 101 Off-Ramp at Park Presidio Boulevard. In addition to vehicle weaving, this section is congested as vehicles merge as they exit the Toll Plaza.

Merge Sections

There are several merge sections on the freeway that reduce the capacity of the freeway and result in vehicle delays. The locations of these merge sections, and the corresponding effects on freeway operations, include:

- The Southbound US 101 On-Ramp at Sausalito Lateral serves approximately 280 vph and approximately 370 vph during the weekday AM and PM peak hours, respectively. This merge section is frequently congested as many vehicles are entering the freeway and others are preparing to merge as motorists are anticipating the downstream lane drops approaching the GGB. The lane drops occur approximately 300 feet downstream of the ramp.
- The Southbound On-Ramp at Merchant Road serves approximately 150 vph and 240 vph during the weekday AM and PM peak hours, respectively. This merge section is approximately 300 feet downstream of the Toll Plaza and is within the Southbound US 101 Off-Ramp at Park Presidio Boulevard weaving section. Vehicles utilizing the substandard hook ramp enter the freeway at approximately 25 mph.
- The Northbound GGB On-Ramp at Merchant Road serves approximately 380 vph and approximately 570 vph during the weekday AM and PM peak hours, respectively. This merge section is approximately 250 feet upstream of the Toll Plaza and is within the Northbound On-Ramp at Park Presidio Boulevard weaving section. This merge section is frequently congested as many vehicles are merging at the lane drops approaching the GGB. The lane drops occur within the vicinity of the Northbound GGB Ramps at Merchant Road. Vehicles utilizing the substandard hook ramp enter the freeway at approximately 25 mph.

Diverge Sections

There are several diverge sections on the freeway that reduce the capacity of the freeway and result in vehicle delays. The locations of these diverge sections, and the corresponding effects on freeway operations, include:

- The Southbound US 101 Off-Ramp at Sausalito Lateral serves approximately 180 vph and approximately 130 vph during the weekday AM and PM peak hours, respectively. Vehicles utilizing the off-ramp begin to decelerate prior to exiting the freeway as the substandard hook ramp is only 175 feet long and stop controlled at the terminus. This diverge section is frequently congested as some vehicles are slowing to exit the freeway and others are preparing to merge as motorists are anticipating the downstream lane drops approaching the GGB. The lane drops occur approximately 1,200 feet downstream of the ramp.
- The Southbound GGB Off-Ramp at Merchant Road serves approximately 390 vph and approximately 360 vph during the weekday AM and PM peak hours, respectively. This diverge section is approximately 200 feet downstream of the Toll Plaza. Vehicles utilizing the substandard hook ramp exit the freeway at approximately 25 mph.

- The Northbound GGB Off-Ramp at Merchant Road serves approximately 150 vph and 170 vph during the weekday AM and PM peak hours, respectively. This diverge section is approximately 350 feet upstream of the Toll Plaza and is within the Northbound GGB On-Ramp at Park Presidio Boulevard weaving section. This diverge section is frequently congested as some vehicles are slowing to exit the freeway and others are merging at the lane drops approaching the GGB. The lane drops occur within the vicinity of the Northbound Ramps at Merchant Road. Since the ramp provides access to popular viewing area with limited parking capacity (fewer than 100 parking spaces) congestion occasionally propagates onto the off-ramp and freeway. This condition was observed on the weekend and also occurs on weekdays. Vehicles utilizing the substandard hook ramp exit the freeway at approximately 25 mph.
- The Northbound US 101 Off-Ramp at Vista Point serves approximately 40 vph and approximately 100 vph during the weekday AM and PM peak hours, respectively. This diverge section is located within the vicinity of the lane additions as vehicles exit the GGB. Vehicles utilizing the off-ramp begin to decelerate prior to exiting the freeway as the substandard hook ramp is extremely short. Since the ramp provides access to popular viewing area with limited parking capacity (fewer than 150 parking spaces) congestion occasionally propagates onto the off-ramp and freeway. This condition was observed on the weekend and also occurs on weekdays. Vehicles utilizing the substandard hook ramp exit the freeway at approximately 25 mph.

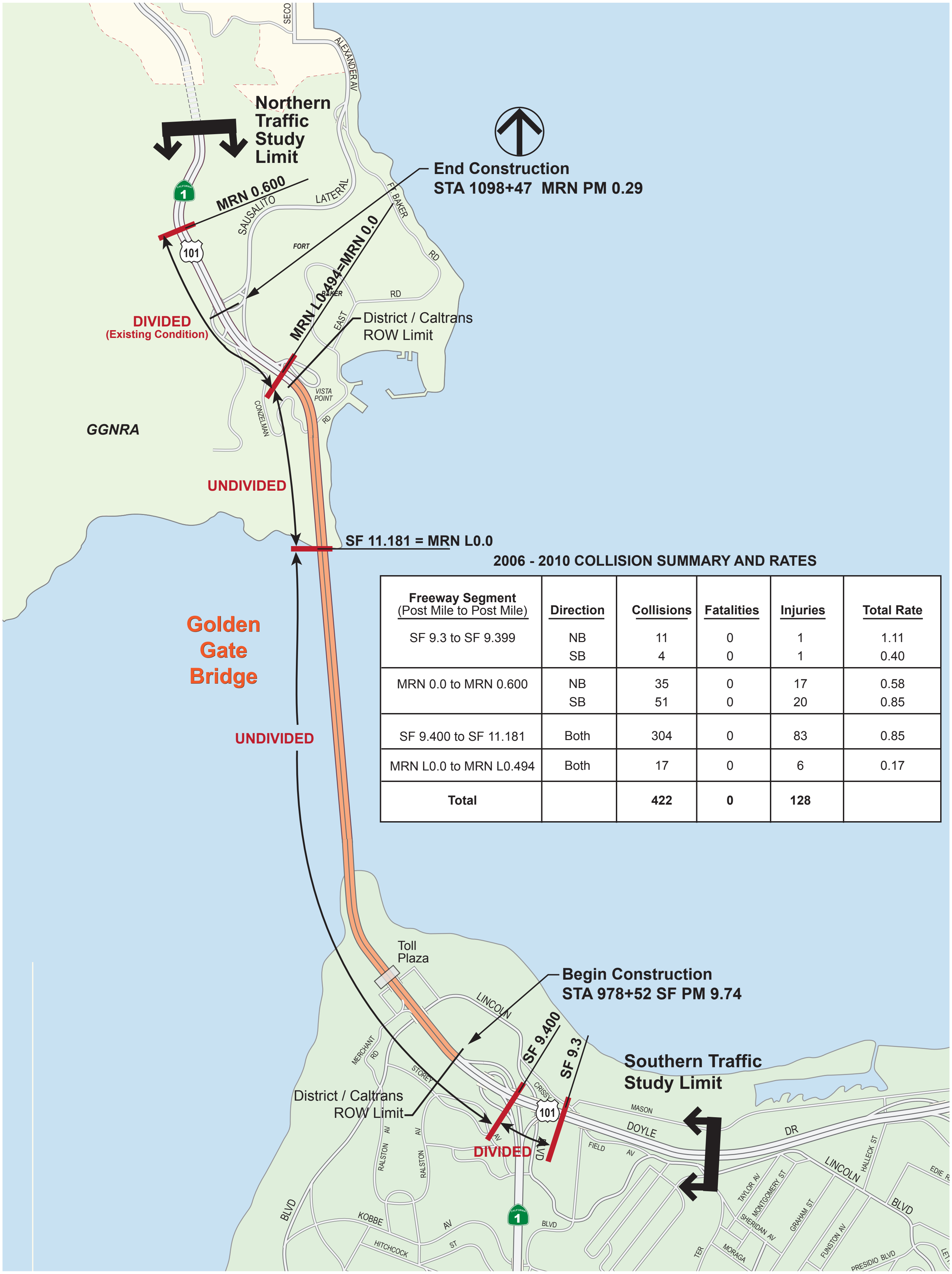
Collision Summary
Traffic Accident Surveillance and Analysis System (TASAS) data were obtained for freeway facilities along the GGB and US 101 freeway. TASAS data was reviewed for a five-year period, from January 1, 2006 to December 31, 2010. The data were converted into accidents per million vehicle miles traveled and compared to statewide statistics for similar facilities. Within the Project limits, for both northern and southern approaches to the GGB, the freeway includes divided and undivided segments. TASAS data on the undivided segments are not separated for each direction (northbound and southbound). Collision rates on the study roadway segments, and corresponding post miles, are included in **Table 8** and shown in **Figure 14**. Collision rates on the study ramps, and corresponding post miles, are included in **Table 9**.

Table 8: Freeway Collision Rate Summary – January 1, 2006 to December 31, 2010

Freeway Segment (Post Mile to Post Mile)	Number of Accidents			Collision Rate					
				Actual			Average		
	Total	Fatal	Fatal + Injury	Fatal	Fatal + Injury	Total	Fatal	Fatal + Injury	Total
US 101 Northbound - Divided									
MRN 000.000 to MRN 000.600	35	0	17	0.00	0.28	0.58	0.01	0.21	0.68
US 101 Southbound - Divided									
MRN 000.000 to MRN 000.600	51	0	20	0.00	**0.33**	**0.85**	0.01	0.21	0.68
NB & SB US 101 and GGB - Undivided									
SF 009.400 to SF 011.181	304	0	83	0.00	0.23	0.85	0.01	0.37	1.15
MRN L000.000 to MRN L000.494[(a)]	17	0	6	0.00	0.06	0.17	0.01	0.15	0.35
TOTAL	**407**	**0**	**126**						

Source: Caltrans – 2012. TASAS – 2012.
Notes:
- Collision rate in terms of collisions per million vehicle miles traveled.
- **Bold** indicates the actual collision rate exceeds the statewide average collision rate for a similar facility.
(a) MRN L000.494 = MRN 000.000

Figure 14: Freeway Collision Summary



2006 - 2010 COLLISION SUMMARY AND RATES

Freeway Segment (Post Mile to Post Mile)	Direction	Collisions	Fatalities	Injuries	Total Rate
SF 9.3 to SF 9.399	NB	11	0	1	1.11
	SB	4	0	1	0.40
MRN 0.0 to MRN 0.600	NB	35	0	17	0.58
	SB	51	0	20	0.85
SF 9.400 to SF 11.181	Both	304	0	83	0.85
MRN L0.0 to MRN L0.494	Both	17	0	6	0.17
Total		**422**	**0**	**128**	

Table 9: Ramp Collision Rate Summary – January 1, 2006 to December 31, 2010

Ramp (Post Mile)	Number of Accidents			Collision Rate					
				Actual			Average		
	Total	Fatal	Fatal + Injury	Fatal	Fatal + Injury	Total	Fatal	Fatal + Injury	Total
US 101 Northbound									
Park Presidio Bl. Off-Ramp (SF 9.421)	5	0	3	0	0.19	0.32	0.005	0.20	0.60
Park Presidio Bl. On-Ramp (SF 9.611)	14	0	6	0	0.13	0.31	0.004	0.15	0.45
Merchant Rd. Off-Ramp (SF 9.801)	1	0	0	0	0.00	0.23	0.004	0.28	0.95
Merchant Rd. On-Ramp (SF 9.811)	13	0	4	0	**0.43**	**1.40**	0.002	0.16	0.55
Vista Point Off-Ramp (MRN 0.065)	6	0	3	0	**0.94**	**1.89**	0.003	0.05	0.60
Vista Point On-Ramp (MRN 0.084)	2	0	2	0	**0.61**	**0.61**	0.001	0.02	0.25
Sausalito Lateral Off-Ramp (MRN 0.264)	9	0	3	0	**0.44**	**1.31**	0.004	0.26	0.85
Sausalito Lateral On-Ramp (MRN 0.312)	3	0	2	0	**0.76**	**1.14**	0.002	0.14	0.45
US 101 Southbound									
Sausalito Lateral Off-Ramp (MRN 0.253)	4	0	1	0	0.25	**1.01**	0.004	0.28	0.95
Sausalito Lateral On-Ramp (MRN 0.170)	1	0	0	0	0	0.13	0.003	0.20	0.65
Merchant Rd. Off-Ramp (SF 9.812)	0	0	0	0	0	0	0.004	0.28	0.95
Merchant Rd. On-Ramp (SF 9.802)	12	0	5	0	**1.03**	**2.48**	0.002	0.16	0.55
Park Presidio Bl. Off-Ramp (SF 9.462)	2	0	1	0	0.03	0.05	0.005	0.15	0.45
Park Presidio Bl. On-Ramp (SF 9.350)	3	0	1	0	0.07	0.20	0.003	0.11	0.35

Source: Caltrans – 2012. TASAS – 2012.
Notes:
- Collision rate in terms of collisions per million vehicles.
- **Bold** indicates the actual collision rate exceeds the statewide average collision rate for a similar facility.

A total of 407 collisions were reported to occur on the study freeway segment between January 1, 2006 and December 31, 2010. These collisions resulted in 126 injuries and no fatalities. A total of 321 of the reported collisions occurred in the undivided section of the highway (SF PM 009.400 to MRN PM L000.494) and 86 of the reported collisions occurred in the divided section of the highway (MRN PM 000.000 to MRN PM 000.600). Eight of the reported collisions were cross-median collisions, which resulted in five injuries and no fatalities.

As shown in **Table 8**, the southbound freeway segment north of the GGB (MRN PM 000.000 to MRN PM 000.600) experienced collision rates (Fatal + Injury, and Total) higher than the statewide average for similar facilities. At this location, 65 percent of the collisions were attributed to speeding and 67 percent of the collisions were rear end collisions. This divided section of highway is located north of the GGB where there are horizontal reverse curves with a short intermediate tangent. Additionally, the approach profile grade to the GGB is approximately six percent and the number of lanes transitions from eight-lanes, near the Sausalito Road Undercrossing to six-lanes at the GGB. These factors, combined with congestion, could be contributing factors to the high accident rate at this location. The Project proposes to replace the left-side lane drops at this location, on the northern approach in the southbound direction, with right-side lane drops to improve decision sight distance. The Proposed Project would be expected to improve safety conditions on this segment.

As shown in **Table 9**, seven of the 14 study ramps experienced five year collision rates that exceeded the statewide average for similar facilities. The Project proposes to conform to the existing ramps at the gore area and maintain the existing lengths and geometric configurations with the exception of the southbound and northbound Sausalito Lateral On- and Off-Ramps. These ramps will be slightly modified up to the gore area to accommodate the improvements along the mainline. Based on the minimal modifications to the existing ramps within the Project area, it is anticipated that the collision rates along the ramps will not increase as a result of the Project.

The Proposed Project would replace the plastic pylons used to delineate and transition the lanes with a physical barrier. These proposed improvements are expected to reduce the likelihood of cross-median collisions. Additionally, the right side lane drops may reduce the likelihood of sideswipe collisions in the southbound direction.

This page intentionally left blank

5.0 Design Year (2030) Conditions

The regional freeway Design Year (2030) Conditions analysis included freeway facilities on the Golden Gate Bridge and US 101 between the Sausalito Lateral Interchange and the Park Presidio Boulevard Interchange. The freeway between the Sausalito Lateral Interchange and Park Presidio Boulevard is typically a six lane facility in the Design Year Conditions and is approximately 2.7 miles long. The northbound freeway includes four on-ramps and four off-ramps. The southbound freeway includes three on-ramps and three off-ramps.

5.1 Traffic Volumes

Design Year Conditions traffic volumes were forecasted on the GGB and ramps within the evaluation area. Concurrence with the forecasting methodology was received from the Caltrans' Forecast Unit on September 22, 2012. This data consists of peak hour traffic volumes.

Weekday Peak Hour Freeway Traffic Volumes

The design year traffic volumes were developed based on the forecasted traffic volumes provided in the Doyle Drive Report Addendum. The corresponding forecasting methodology is provided in the *Doyle Drive South Access to the Golden Gate Bridge Final Traffic and Transit Operations Report*.[xii] This document is herein referred to as the "Doyle Drive Report". The Year 2030 volumes from the Doyle Drive report were directly applied to this analysis to develop the Design Year Conditions traffic volumes.

As discussed in the Doyle Drive Report, at the time the forecasts were prepared, the horizon year for the San Francisco Travel Demand Model (SF-TDM) was 2020. However, to satisfy the analysis needs, a 2030 horizon year was necessary. This horizon year exceeded the latest available regional land use and travel demand forecasts. Utilizing 2025 travel demand and land use forecasts prepared by the Metropolitan Transportation Commission (MTC) and the Association of Bay Area Governments (ABAG), Year 2030 forecasts were developed. The Year 2025 MTC trip table was expanded to the Design Year horizon using a purpose-specific methodology developed with MTC and Caltrans. A projection to Year 2030 conditions was developed by extending the growth from Year 2020 to Year 2025.

The SF-TDM incorporated the *Projections 2002* series of demographic assumptions for the region, including out-of-county trips produced by MTC for the *Draft 2001 Regional Transportation Plan*, and the *Draft Presidio Trust Management Plan* (PTMP) proposed improvements and projects, and include all likely major transportation projects in the region. Additional refinements to the San Francisco networks, such as Muni service changes, were also incorporated into future year networks. These assumptions provide a cumulative analysis that incorporates land use growth and local transportation projects. At the time the forecasts were prepared, there was no additional information on changes to the regional transportation system beyond 2025. Accordingly, the input highway and transit networks used in 2030 forecasts were the 2025 networks. The SF-TDM predicted the 2030 behavior of San Francisco residents, and then integrated this demand with the regional 2030 demand, which was extrapolated from the MTC 2025 regional trip tables.

Because the SF-TDM estimates weekday traveler behavior using a three-hour peak period, the peak-period volumes from the model were multiplied by a peak-period to peak-hour ratio. To provide consistent comparisons, a single overall ratio was developed for each peak period by calculating the percentage of the peak period traffic that occurs during the peak hour. Additionally, project-specific weekend travel demand model was developed to analyze existing and future conditions. Travel behavior during a peak weekend period was developed by adjusting weekday demand by trip purpose to reflect weekend conditions.

No major changes in population or employment were projected by any government agency; therefore, design year forecasts and methodology used in the Doyle Drive Report were assumed to be appropriate for use in this analysis. Further details about the forecasting methodology are provided in the Doyle Drive Report. Further details about the SF-TDM can be found in the *San Francisco Travel Model Development – Model Validation Report,* issued in 2001.

Design Year Conditions traffic volumes at the Park Presidio Boulevard Ramps and US 101 were provided in the Doyle Drive Report Addendum and were directly utilized as the Park Presidio Ramp volumes and the mainline freeway "control point" for this analysis.

The Merchant Road and Vista Point land uses are not expected to change significantly between the Existing Conditions and the Design Year Conditions. Additionally, the Vista Point parking facilities operate near capacity in the Existing Conditions and additional vehicle trips could not be accommodated regardless of increases in demand. As such, a nominal five percent growth rate was applied to account for changes in traffic volumes between the Existing Conditions and Design Year Conditions at these locations.

To account for the expected traffic volume growth in the Design Year Conditions for the ramps at the Sausalito Lateral Interchange, a traffic volume increase of one percent per year was applied to the Existing Conditions traffic volumes.

The weekday peak hour Design Year Conditions freeway traffic volumes on the GGB and US 101 freeway are shown in **Figure 15**.

Figure 15a: Design Year Conditions Traffic Volumes - Northern Approach

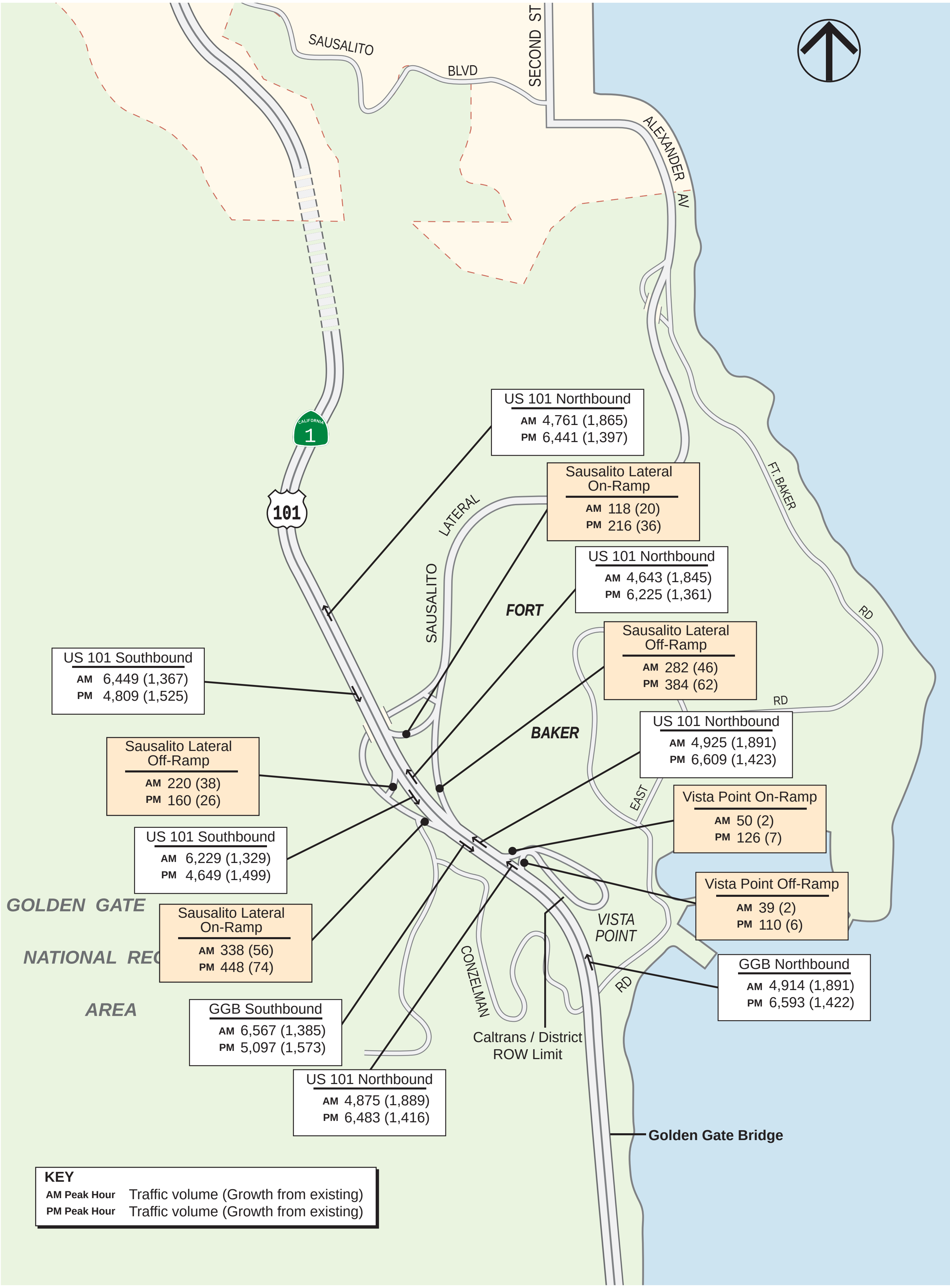
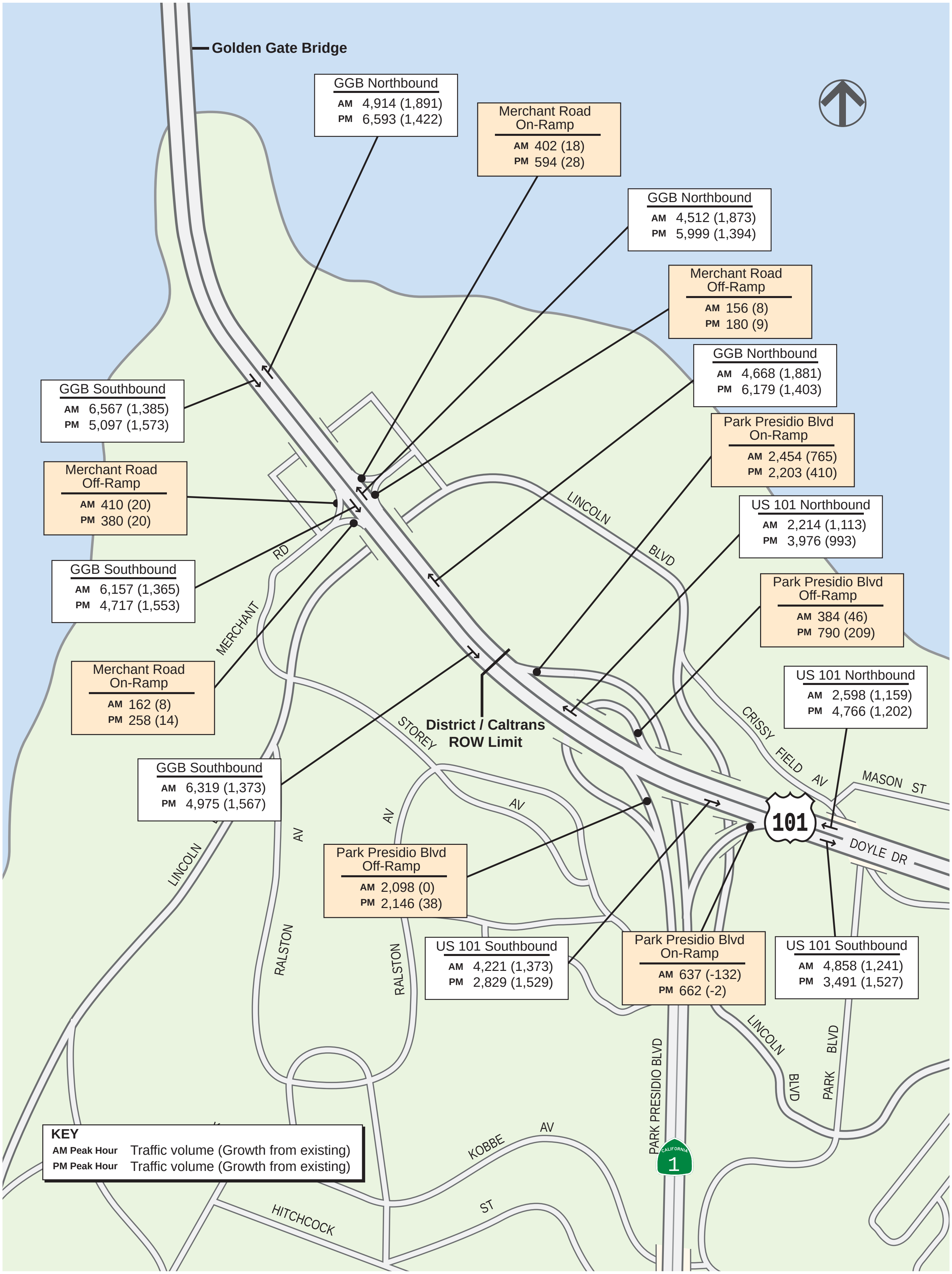


Figure 15b: Design Year Conditions Traffic Volumes - Southern Approach



6.0 Roadway Capacity

This section addresses the effects of the Proposed Project on roadway capacity and provides lane width recommendations. This evaluation includes a summary of other MMB applications and operational procedures.

6.1 Moveable Median Barrier Operations

The following sections address the projected effects of the MMB based on worldwide applications and the operational procedure of the barrier transfer machines.

Worldwide Applications

The first MMB was installed on the Auckland Harbour Bridge in New Zealand in 1990. Currently, MMBs are being utilized on other bridges throughout the world – including the United States. Locations within the United States include: the Coronado Bridge in California, the Tappan Zee Bridge in New York, the Theodore Roosevelt Bridge in Washington D.C, the Benjamin Franklin Bridge, the Walt Whitman Bridge, the Commodore Barry Bridge, and the Betsy Ross Bridge in Philadelphia. Examples of moveable median barriers on several of these bridges are shown in **Figure 16**.

Auckland Harbour Bridge, New Zealand – The Auckland Harbour Bridge has many similarities to the Golden Gate Bridge, including: lane widths, traffic speeds, heavy winds, the absence of shoulders, and alignment. With a two foot wide MMB, the four middle lanes average 9'-11" wide. Individually, these lanes range from 9'-4" to 11'-6" wide. In each lane configuration, the two narrow lanes are located next to a wider lane consisting of at least 10'-6" in width. During the morning peak period, five of the eight lanes are allocated for southbound traffic. During the afternoon peak period, five lanes are allocated to northbound traffic. At all other times of the day, four lanes are allocated for each direction of traffic. The barrier transfer machine makes the transfer in approximately 20 minutes. The lane widths for the three lane configurations of the Auckland Harbour Bridge are shown in **Figure 17**.

As discussed in the Feasibility Study, the Auckland Harbour Bridge has a posted speed limit of 50 mph and an estimated capacity of 180,000 vehicles per day. The average speed on the Auckland Harbour Bridge is 46 mph to 50 mph. The hourly average speed ranges from 37 mph to 55 mph with the heaviest 15-minute period at 34 mph.

Observations have shown that motorists utilizing the lanes adjacent to the barrier tend to shy away, and in some cases induce motorists in the adjacent lane to also shy away. When the barrier was deployed motorists complained about the narrow lanes; however, motorists eventually became accustomed to the narrow lanes and there was a significant reduction in collisions. Prior to the installation of the barrier in 1990 there was an average of three crossover-related fatalities per year. No crossover collisions have occurred since the installation. Vehicles have collided and moved the barrier a distance into the opposing lane an average of 1.3 times per year, but no secondary accidents have resulted.

Tappan Zee Bridge, New York – The 3.0 mile long, 90-foot wide Tappan Zee Bridge is a major link in the New York State Thruway system, spanning the Hudson River 13.0 miles north of New York City to connect Rockland and Westchester counties. It was built as a six lane bridge in the early 1950s with an estimated capacity of 100,000 vehicles per day. The bridge was reconfigured for seven lanes in the mid-1980s when the 10'-0" wide median was replaced with a fixed concrete median barrier to separate four southbound and three northbound lanes. In 1992, the concrete barrier was replaced with a two foot wide MMB to maximize roadway efficiency. Twice a day transfer machines move the 3.0 mile chain of barriers 12-feet across the roadway to change the direction of travel of the center lane. The barrier transfer machine makes the transfer in approximately 60 minutes.

Figure 16a: Moveable Median Barrier Applications

Auckland Harbour Bridge
Moveable Median Barrier,
Auckland, New Zealand

Auckland Harbour Bridge
Moveable Median Barrier,
Auckland, New Zealand

Auckland Harbour Bridge
Barrier Transfer Machine

Figure 16b: Moveable Median Barrier Applications

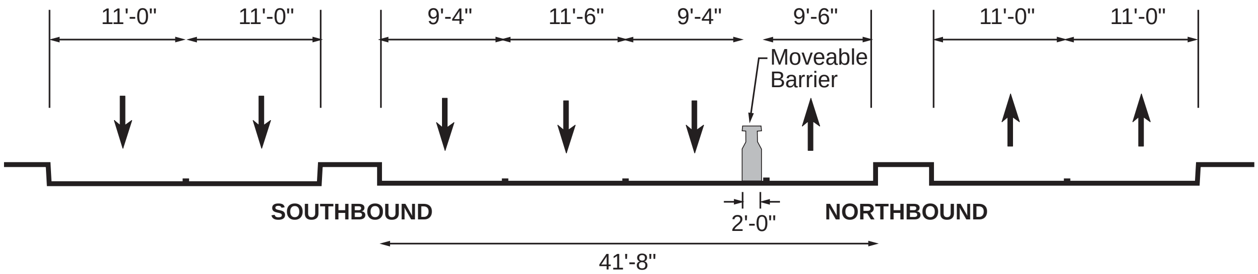
Tappan Zee Bridge Moveable Median Barrier, Tarrytown, NY

Tappan Zee Bridge Moveable Median Barrier, Tarrytown, NY

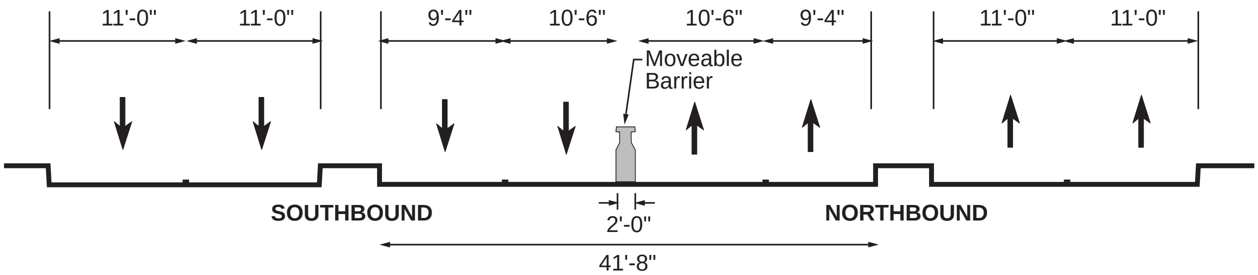
Coronado Bridge Moveable Median Barrier, San Diego, CA

Figure 17: Auckland Harbour Bridge Roadway Cross Sections

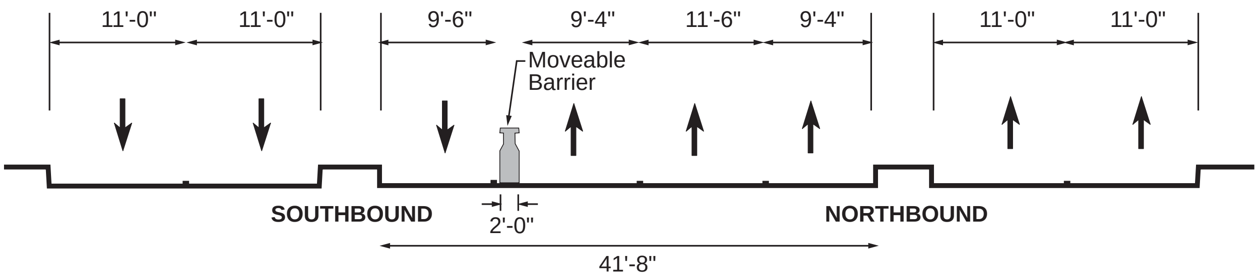
5 Southbound / 3 Northbound Lane Configuration



4 Southbound / 4 Northbound Lane Configuration



3 Southbound / 5 Northbound Lane Configuration



NO SCALE

Coronado Bridge, San Diego, CA – The 2.1-mile long, 62-foot wide Coronado Bridge was constructed to carry a maximum of five traffic lanes with two 12’-0” wide lanes on each side and a 14’-0” wide middle lane. A 2’-0” wide MMB was installed in 1993 as a safety measure to reduce the chance of head-on collisions and as part of the overall strategy to manage traffic on the bridge. The lane configuration is changed twice a day to accommodate peak hour traffic with three travel lanes in the direction of peak flow.

Operational Procedure
Two BTMs would be required for the Golden Gate Bridge. Each BTM is able to transfer the barrier one lane width at a time, similar to the current pylon transfer crews. When in operation, a BTM transfers the barrier from the left to the right, with respect to the BTM’s direction of travel – the same as the current pylon transfer movement. This ensures that the BTM is protected from adjacent traffic in both directions. While the BTM is in operation, only five of the six lanes would be open to traffic.

During the short-term improvements, the BTM(s) would be stored near the tollbooths. After the implementation of the long-term improvements, the BTM(s) would be stored approximately 750 ft south of the tollbooths within the District right-of-way. In a four northbound lanes / two southbound lanes configuration, the BTMs would be stored on the southern side of the GGB. In a three northbound lanes / three southbound lanes configuration, the BTMs would be stored on opposite sides of the GGB. In a two northbound lanes / four southbound lanes configuration, the BTMs would be stored on the northern side of the GGB.

In operation, the BTM moves at approximately seven miles per hour and would take an estimated 20 minutes to complete a one lane transfer. This includes the maneuvering of the BTM on and off the barrier ends upon completion of moving the barrier.[xiii] The wheels at both ends of the BTM are steerable, requiring two operators at each end of the BTM when maneuvering the machine on and off the barrier ends.

6.2 Roadway Lane Widths

The MMB will effectively decrease the width of the Golden Gate Bridge by 12 inches – from 62’-0” to 61’-0”. This is significant as the existing lane widths are narrow given the traffic volumes and types of vehicles utilizing the GGB. The following section discusses the lane width alternatives, effects on capacity. For reference purposes, the existing striping along the GGB is shown in **Figure 18**.

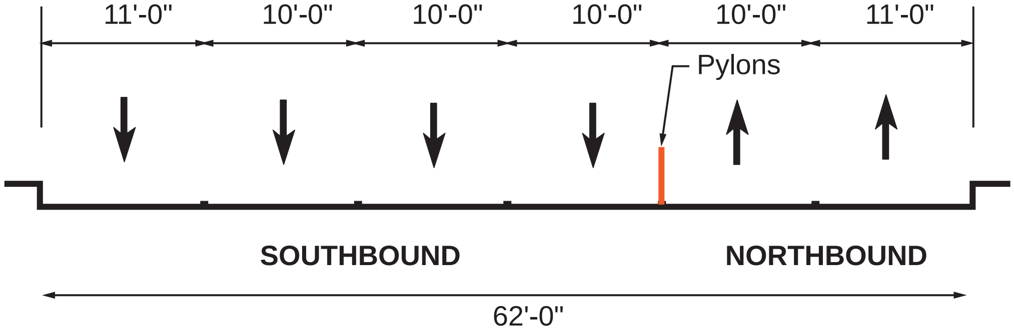
The Golden Gate Bridge is a unique facility that accommodates the conflicting needs of vehicles, trucks, and buses. In ideal conditions, no lane would be less than 10’-0” wide, while to accommodate buses and large trucks, the curb lane would be at least 11’-0” wide. However, adding the 12-inch wide median barrier to the existing 62’-0” curb-to-curb distance would require compromise.

Guidelines for Determining Lane Width Alternatives
The primary lane width constraint on the GGB is the width of the Golden Gate Transit buses. These buses are approximately 8’-6” wide and occupy approximately 9’-0” of width on the 1,000-foot radius curves on the GGB. Additionally, each of the side-view mirrors protrude approximately 12 inches on either side of the bus. Therefore, on a curve a Golden Gate Transit bus occupies approximately 11’-0” of width. Therefore, since the buses utilize the curb lanes, the minimum curb lane width must be at least 11’-0”.

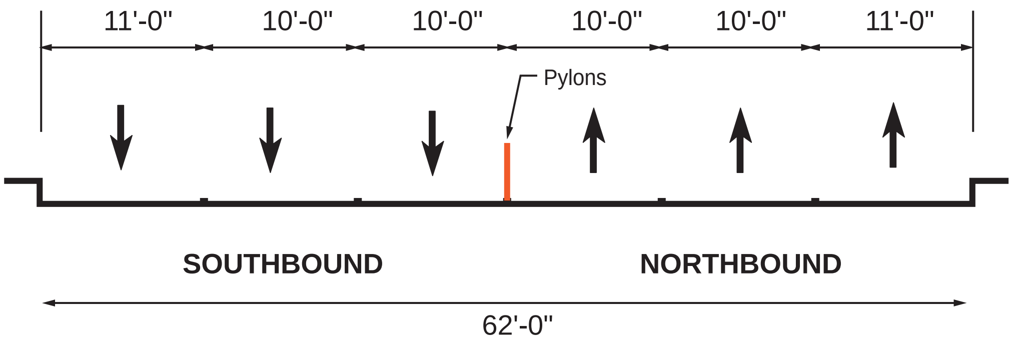
As presented in the Feasibility Study, the narrowest lanes currently utilized on a bridge with a MMB system are on the Auckland Harbour Bridge. These lanes are 9’-4” wide and are always adjacent to a lane that is at least 10’-6” wide. This 19’-10” section represents the minimum width allocated to a two lane section on the Auckland Harbour Bridge. Since this is the minimum functional configuration observed, it is recommended that the minimum lane width be at least 9’-4” and the minimum two lane section width be at least 19’-10”.

Figure 18: Existing Conditions Roadway Cross Sections

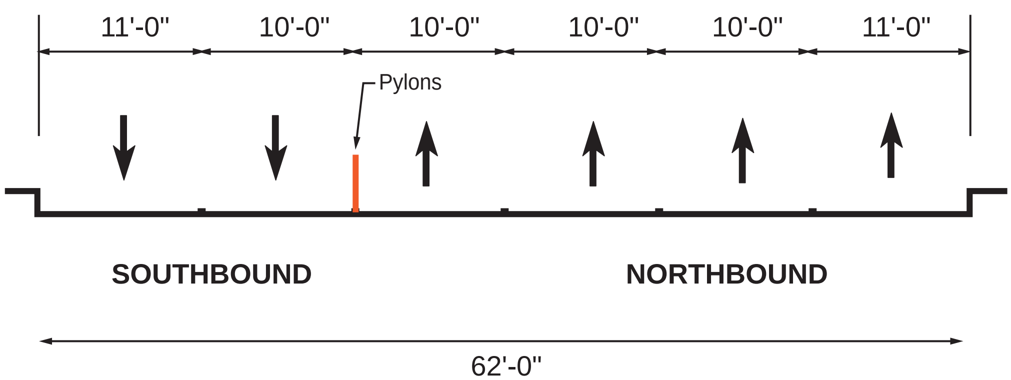
4 Southbound / 2 Northbound Lane Configuration



3 Southbound / 3 Northbound Lane Configuration



2 Southbound / 4 Northbound Lane Configuration



NO SCALE

Since vehicles occupying narrow lanes would likely encroach into adjacent lanes, the width of adjacent lanes – regardless of the configuration – should be maximized. Additionally, maximizing the width of the narrowest lane in each configuration is desirable.

To summarize, the following lane width guidelines are recommended:

1. The curb lanes should be at least 11’-0” wide;
2. All lanes should be at least 9’-4” wide;
3. Two lane sections should be at least 19’-10” wide;
4. The total width of any two adjacent lanes should be maximized; and
5. The width of the narrowest lane should be maximized.

Using these guidelines, the following two lane width alternatives were developed.

Alternative 1: Unequal Lane Width Alternative
Based on these guidelines, the following lane width characteristics would be maintained for all three of the lane configurations with this alternative:

1. The curb lanes would be 11’-0” wide;
2. All lanes would be at least 9’-4” wide;
3. Two lane sections would be 20’-4” wide;
4. The total width of any two adjacent lanes would be at least 19’-6” wide; and
5. The width of the narrowest lane would be 9’-4” wide.

In this alternative, the minimum total width of any two adjacent lanes would be at least 19’-6” wide in the three lane direction. In the two lane and four lane directions, the total width of any two adjacent lanes would be 20’-4” wide. The MMB would be moved 10’-2” by the BTM to change configurations. This lane width alternative would meet or exceed all of the recommended lane width guidelines. The lane widths for each of the three lane configurations in the unequal lane width alternative are shown in **Figure 19**.

Alternative 2: Distributed Lane Width Alternative
In addition to the unequal lane width alternative, a second lane width alternative has been evaluated. This alternative was developed based on the concept of maintaining the existing, equally distributed lane width striping and centering the MMB on the lane striping. This distributed lane width alternative was assessed based on the recommended lane width guidelines. The following lane width characteristics would be maintained for all three of the lane configurations with this alternative:

1. The curb lanes would be 11’-0” wide;
2. All lanes would be at least 9’-6” wide;
3. Two lane sections would be 20’-6” wide;
4. The total width of any two adjacent lanes would be at least 19’-6” wide; and
5. The width of the narrowest lane would be 9’-6” wide.

The minimum total width of the two adjacent lanes would be 20’-6” wide in the two lane direction. The minimum total width of any two adjacent lanes would be at least 19’-6” wide in the three lane and four lane directions. The MMB would be moved 10’-0” by the BTM to change configurations. The lane widths for each of the three lane configurations in the distributed lane width alternative are shown in **Figure 20**.

Figure 19: Preferred Alternative Roadway Cross Sections

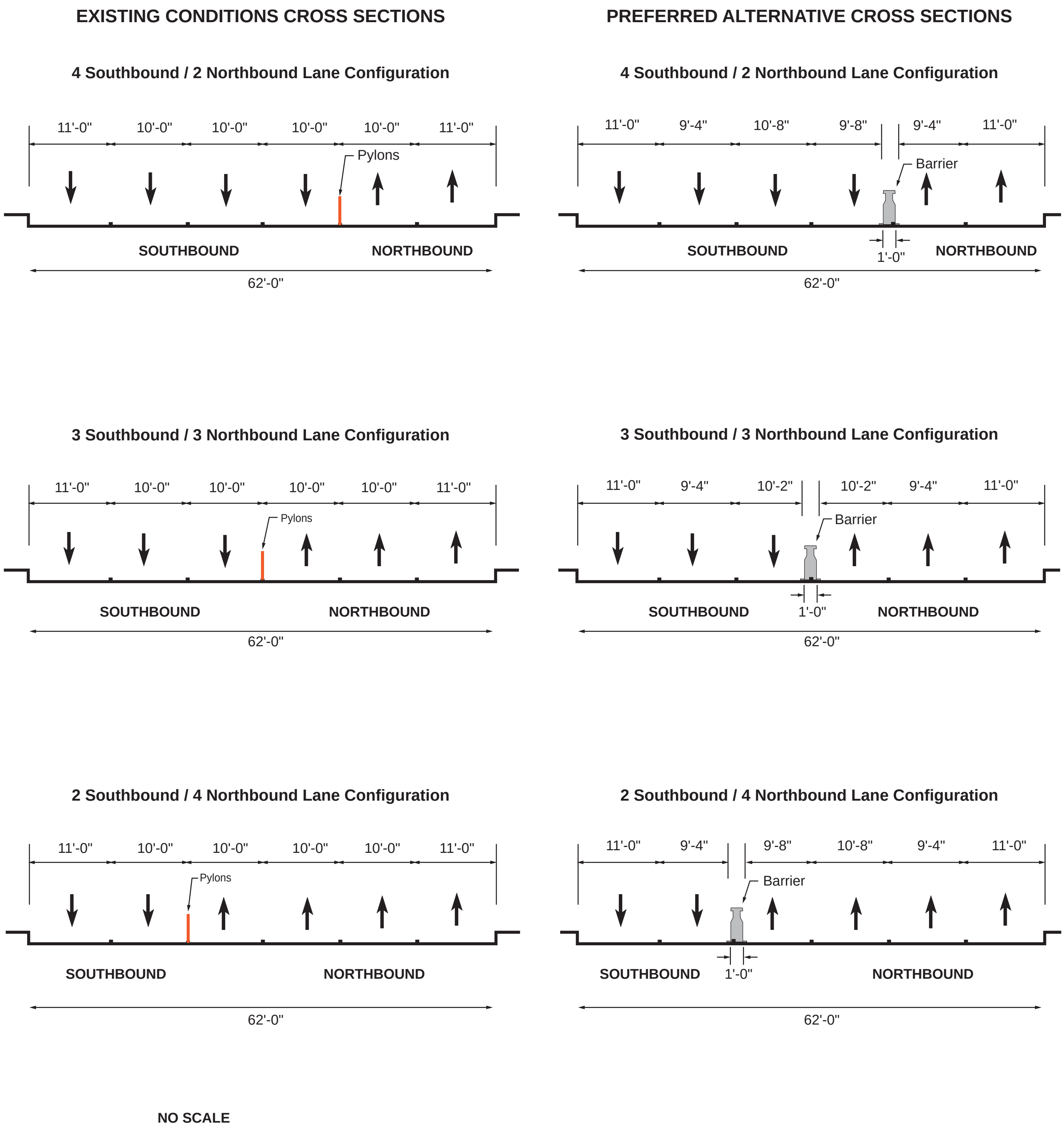
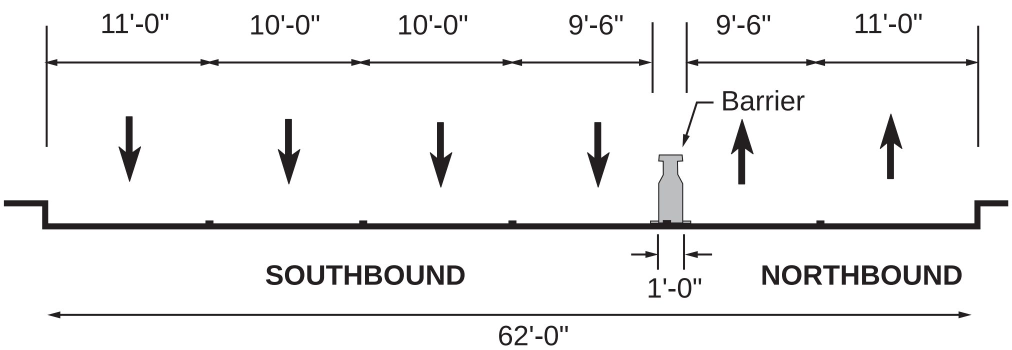
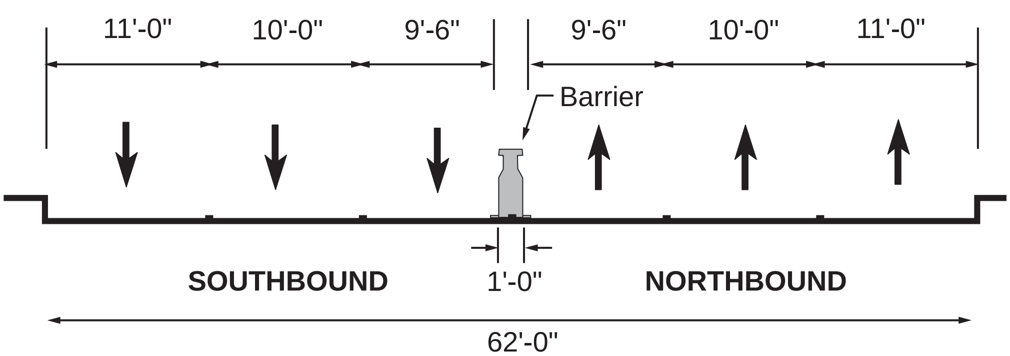


Figure 20: Distributed Lane Width Alternative Cross Sections

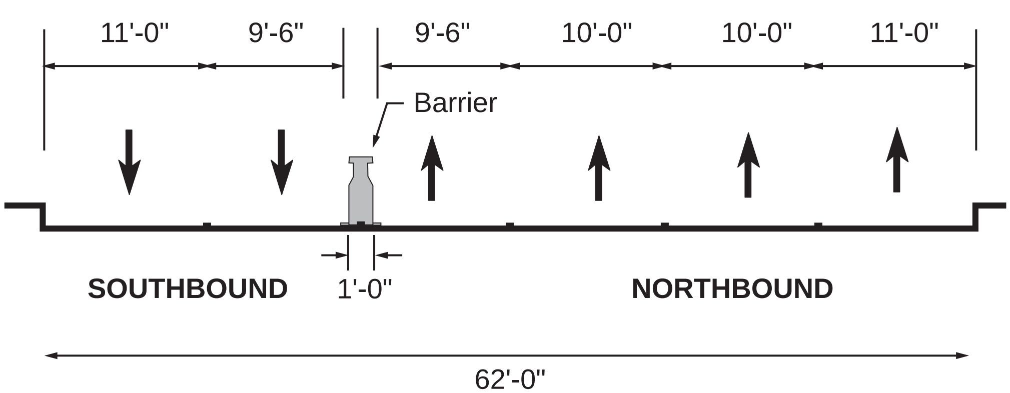
4 Southbound / 2 Northbound Lane Configuration



3 Southbound / 3 Northbound Lane Configuration



2 Southbound / 4 Northbound Lane Configuration



NO SCALE

Lane Widths Alternatives Comparison

The primary considerations addressed in the unequal lane width alternative and the distributed lane width alternative are included in **Table 10**.

Table 10: Lane Width Alternatives Comparison

Lane Width Guideline	**Lane Width Alternative**	
	Unequal Lane Width **Alternative**	**Distributed Lane Width Alternative**
Curb lane width	11'-0"	11'-0"
Two adjacent lanes section width		
Two lane direction	20'-4"	20'-6"
Three lane direction	19'-6"	19'-6"
Four lane direction	20'-4"	19'-6"
Average lane width		
Two lane direction	10'-2"	10'-3"
Three lane direction	10'-2"	10'-2"
Four lane direction	10'-2"	10'-1.5"
Narrowest lane width	9'-4"	9'-6"
BTM barrier movement distance	10'-2"	10-0"

Source: AECOM – February 2013
- The two adjacent lanes section width represents the minimum width of any two adjacent lanes.
- The BTM barrier movement distance is consistent for all three transitions.

Both of the proposed lane width alternatives are feasible and each present advantages and disadvantages. The unequal lane width alternative features consistent characteristics for all three roadway configurations. For example, the average lane width is 10'-2" for each roadway configuration. The distributed lane width alternative is inconsistent but allows for more favorable characteristics in some configurations and less favorable characteristics in others in comparison to the unequal lane width alternative. For example, the average lane width is 10'-3" in the two lane direction but only 10'-1.5" in the four lane direction.

From a capacity and operations perspective, both alternatives would be expected to perform similarly and any differences would be negligible. Given the similarities of the two alternatives and for the purposes of this analysis, the unequal lane width alternative will be utilized in the project analyses. Although both alternatives are viable, the unequal lane width alternative is herein presented as the Proposed Project.

A detailed estimate of the Proposed Project lane capacities on the GGB are included in **Table 11**.

Table 11: Estimated Traffic Capacity – Proposed Project Conditions

Configuration	Lane Position	Existing Conditions		Proposed Project Conditions	
		Width	Capacity	Width	Capacity
2 Lanes	Curb	11'-0"	1,680	11'-0"	1,680
	Inside	10'-0"	1,590	9'-4"	1,600
	Total	**21'-0"**	**3,270**	**20'-4"**	**3,280**
3 Lanes	Curb	11'-0"	1,780	11'-0"	1,780
	Middle	10'-0"	1,760	9'-4"	1,600
	Inside	10'-0"	1,630	10'-2"	1,810
	Total	**31'-0"**	**5,170**	**30'-6"**	**5,190**
4 Lanes	Curb	11'-0"	1,790	11'-0"	1,790
	Middle-R	10'-0"	1,770	9'-4"	1,630
	Middle-L	10'-0"	1,770	10'-8"	1,840
	Inside	10'-0"	1,640	9'-8"	1,770
	Total	**41'-0"**	**6,970**	**40'-8"**	**7,030**

Source: AECOM – February 2013
Notes:
- Capacity in terms of vehicles per hour.

The modifications to the roadway characteristics and the reduction in shy distance would be expected to increase the overall capacity of the roadway in all three of the roadway configurations. The net change in roadway capacity with the implementation of the Proposed Project was estimated to be the following:

- Two lane direction – 10 vph increase;
- Three lane direction – 20 vph increase; and
- Four lane direction – 60 vph increase.

These net impact estimates are relative, since the differences are within the margin of error of the estimation procedure. The actual effect on traffic capacity of a MMB would have to be determined by actual experience. The net traffic capacity impacts need to be balanced against lane width and safety requirements.

Capacity Impacts

The Proposed Project would require the overall narrowing of the travel lanes with the implementation of the MMB. Generally, the narrowing of the lanes would reduce the capacity of the roadway. However, the addition of the MMB would reduce the effect of shy distance on the inside lane and marginally increase the capacity of the roadway. The following factors would influence the capacity of the Golden Gate Bridge:

1. **Roadway Characteristics –** Lane widths, closeness of barriers to the travel way, whether or not there is a barrier between the directions of travel, and the percentage of heavy vehicles affects the capacity of individual lanes. The effect of the Proposed Project on roadway capacity would vary based on the corresponding roadway characteristics.
2. **Shy Distance –** Shy distance is defined as the distance from the edge of the traveled way beyond which a roadside object might not be perceived by a typical driver as an immediate feature to be avoided to the extent that the driver will

change the vehicle's placement or speed. Shy distance increases for vehicles traveling on the inside lane next to the pylons, causing either a reduction of speed or a change in vehicle placement, which results in a reduction of lane capacity by an estimated 130 vph. With implementation of the MMB the effect of shy distance on the inside lanes of the GGB would be reduced. The presence of the MMB would change driver behavior such that they would increase their speed or travel closer to the barrier compared to the pylons (and opposing traffic). This is due to the fact that opposing traffic (and potential crossovers) would not be perceived as such a threat. While it is true that some drivers would still choose not to utilize the inside lane, it is expected that there would be a comparative increase in the number of drivers that would be more comfortable utilizing the inside lane. As indicated in Section 5.0, historic observations of traffic levels in each of the Bridge's travel lanes show that many vehicles avoid the existing lanes adjacent the pylons. Therefore, it was determined that the Proposed Project would result in a maximum lane capacity increase of approximately 60 vph on the inside lanes in the four lane direction.

7.0 Toll Plaza Capacity

This section addresses the effects of the Proposed Project on the capacity of the Toll Plaza based on the demand and the corresponding collection type.

7.1 Existing (2012) Conditions

Southbound traffic exiting the Golden Gate Bridge passes through a Toll Plaza consisting of a maximum of 11 tollbooths prior to Merchant Road. Given the adjustable configuration of the lanes on the GGB, the function of the tollbooths can be modified to accommodate the variable traffic volumes and toll payment types. Currently, vehicles passing through the tollbooths can pay the toll via FasTrak or make a cash transaction.

High Occupancy Vehicles (HOV) with three or more passengers, motorcycles, and buses are required to pay a discounted toll of $3.00 on weekdays between 5:00am and 9:00am and between 4:00pm and 6:00pm, with the exception of holidays.[xiv] Vehicles that qualified for the HOV toll discount must have a FasTrak transponder, and must utilize a cash transaction lane where the tollbooth operator may validate these vehicles and allow them to pass with a discounted toll.

The island width for each tollbooth varies from 3'-6" to 4'-6". The southbound curb lane is 12'-0" wide and the second lane from the curb is 11'-0" wide. The curb-to-curb lane widths at the other nine tollbooths range from 9'-0" to 11'-3". The average lane width of the 11 tollbooths at the Toll Plaza in the southbound direction is 10'-2". The speed limit at the tollbooths is five mph.

Toll Plaza Capacity

The maximum vehicle demand at the Toll Plaza was evaluated for the weekday AM and PM peak hours and the Saturday and Sunday peak hours. The following Toll Plaza alignments were observed for the peak hours of vehicle demand ("general" vehicle lanes consist of FasTrak, cash, and carpool toll collection types):

- Weekday AM peak hour (8:00am to 9:00am) – Six "general" vehicle lanes, four FasTrak-only lanes, one inactive lane;
- Weekday PM peak hour (5:00pm to 6:00pm) – Seven "general" vehicle lanes and two FasTrak-only lanes;
- Saturday peak hour (4:00pm to 5:00pm) – Eight "general" vehicle lanes and two FasTrak-only lanes; and
- Sunday peak hour (5:00pm to 6:00pm) – Eight "general" vehicle lanes and two FasTrak-only lanes.

Based on empirical data provided in the Doyle Drive Report Addendum, FasTrak tollbooths can accommodate a demand of at least 1,100 vph. Tollbooths that exclusively serve cash and carpool transactions can accommodate a demand of approximately 380 vph. Tollbooths that accommodate FasTrak, cash, and carpool transactions have been observed to accommodate up to 400 vph.

Toll Plaza Demand

FasTrak data was obtained from the District.[xv] A summary of the traffic demand and the transaction types at the Golden Gate Bridge Toll Plaza is included in **Table 12**.

Table 12: Toll Collection Data Summary – Existing Conditions

Peak Hour	Volume (vehicles)	Toll Collection Type			
		FasTrak	Cash	Carpool[d]	Other[d]
Weekday AM[a]	5,182	75.0%	20.4%	4.1%	0.5%
Weekday PM[b]	3,524	66.0%	29.4%	4.1%	0.5%
Saturday[c]	3,589	48.0%	51.5%	0.0%	0.5%
Sunday[c]	3,987	48.0%	51.5%	0.0%	0.5%

Source: District – September 2012; AECOM – February 2013
(a) Weekday AM peak hour FasTrak usage assumed to be same as weekday AM peak period.
(b) Weekday PM peak hour FasTrak usage assumed to be same as weekday PM peak period.
(c) Weekend peak hour FasTrak usage assumed to be same as weekend average.
(d) Carpool and Other toll collection types assumed to be weekly average. Other vehicle toll collection is assumed to be similar to cash transactions.

Toll Plaza Analysis
Based on the empirical data provided in the Doyle Drive Report Addendum and the Existing Conditions peak hour traffic volumes, the minimum number of tollbooths that are currently required are included in **Table 13**.

Table 13: Toll Plaza Analysis – Existing Conditions

Peak Hour	Tollbooths Available[a]	Minimum Tollbooths Required		
		FasTrak	General	Total
Weekday AM	11	4	3	7
Weekday PM	9	3	3	6
Saturday	10	2	5	7
Sunday	10	2	6	8

Source: AECOM – February 2013
Notes:
- FasTrak-only tollbooths only accommodate FasTrak transactions.
- "General" tollbooths accommodate FasTrak, cash, and carpool transactions.
(a) All tollbooths can function as FasTrak-only or "general" tollbooths in the Existing Conditions. The number of tollbooths available includes inactive tollbooths but not tollbooths that are serving northbound traffic.

The number of tollbooths available exceeds the minimum number or tollbooths required for all of the peak hours. Typically, no more than eight tollbooths are required in the Existing Conditions. The Toll Plaza analysis calculations are included in **Appendix D**.

7.2 Existing (2012) Plus Project Conditions

The following section assesses the effects of the Proposed Project on the operations of the Toll Plaza in the Existing Conditions. This includes an evaluation of the proposed capacity, projected demand, and the corresponding analysis.

Toll Plaza Capacity
In the Existing plus Project Conditions (short-term project), the barrier transfer machines would be stored near the tollbooths. Given the dimensions of the BTMs and the approach to the Toll Plaza, the number of available tollbooths would be reduced to seven during a four northbound lanes / two southbound lanes configuration with the presence of the BTM(s). Based on the lane configuration during the short-term improvements, the Toll Plaza would function with a maximum of

eight active tollbooths. As discussed in Section 2.4, the Toll Plaza modifications would consist of the following improvements:

- Tollbooth 1 through Tollbooth 7 would remain the same as the current configuration;
- The lane width through Tollbooth 8 would change from 9'-10" to 14'-0";
- Tollbooth 8 through 11 would be removed, including all at-grade toll collection equipment; and
- Toll collection equipment would be reconfigured to accommodate new widened toll lane 8 traffic.

In a four northbound lanes / two southbound lanes configuration, the estimated maximum capacity on the GGB would be approximately 3,280 vph in the southbound direction. With implementation of the Toll Plan (see Section 2.3), 3,280 vph would require a minimum of three tollbooths. Therefore, in this configuration, southbound vehicle queuing and delay would be caused by the capacity of the GGB rather than the capacity of the Toll Plaza.

Toll Plaza Demand
With the Proposed Project, the traffic volumes approaching the Toll Plaza would not be expected to differ from the Existing Conditions traffic volumes. However, implementation of the Toll Plan is assumed under Existing plus Project Conditions. Therefore, the transaction composition and toll collection types would be different compared to Existing Conditions.

Toll Plaza Analysis
Based on the empirical data provided in the Doyle Drive Report Addendum and the Existing Conditions peak hour traffic volumes, the number of tollbooths available and the demand by collection type for Existing plus Project Conditions are included in **Table 14**.

Table 14: Toll Plaza Analysis – Existing plus Project Conditions

Peak Hour	Tollbooths Available[a]	FasTrak			General		
		Booths[b]	Demand	Unserved Demand	Booths	Demand	Unserved Demand
Weekday AM	8	5	4,944	-	1	238	-
Weekday PM	8	4	3,362	-	1	162	-
Saturday	8	4	3,571	-	1	18	-
Sunday	8	4	3,968	-	1	20	-

Source: AECOM – February 2013
Notes:
- FasTrak-only tollbooths only accommodate FasTrak transactions.
- "General" tollbooths accommodate FasTrak, transit, and carpool transactions.
- Demand measured in vehicles per hour.
[a] The number of tollbooths available includes inactive tollbooths but not tollbooths that are serving northbound traffic.
[b] Existing plus Project Conditions analysis assumes full implementation of the Toll Plan.

Existing plus Project Conditions analysis assumes implementation of the Toll Plan. With the ensuing increase in FasTrak usage, the peak hour demand could be accommodated.

A detailed evaluation of the FasTrak usage relative to the traffic volume increases is addressed in the Design Year Conditions analysis.

If the Toll Plan is incorporated and the rate that toll collection transactions are completed is similar to the rate FasTrak transactions are completed (each lane can accommodate 1,100 vph), the following Toll Plaza requirements could be expected in the Existing plus Project Conditions (assume one lane is dedicated for carpool and transit usage only):

- Weekday AM peak hour (8:00am to 9:00am) – Six lanes would be required if the Toll Plan is incorporated;
- Weekday PM peak hour (5:00pm to 6:00pm) – Five lanes would be required if the Toll Plan is incorporated;
- Saturday peak hour (4:00pm to 5:00pm) – Five lanes would be required if the Toll Plan is incorporated; and
- Sunday peak hour (4:00pm to 5:00pm) – Five lanes would be required if the Toll Plan is incorporated.

Typically, no more than six lanes would be required at the Toll Plaza in the Existing plus Project Conditions with the incorporation of the Toll Plan.

Summary
With the implementation of the Toll Plan, no more than six lanes would be required at the Toll Plaza in the Existing plus Project Conditions. No queuing or congestion would be expected at the Toll Plaza.

7.3 Design Year (2030) Conditions

The following section assesses the projected operations of the Toll Plaza in the Design Year Conditions. This includes an evaluation of the proposed capacity, projected demand, and the corresponding analysis.

Toll Plaza Capacity
In the Design Year Conditions eight tollbooths would be available and all of the tollbooths would be FasTrak-only regardless of the lane configuration on the GGB. Assuming the Toll Plan is fully implemented, the rate at which toll collection transactions are completed would be expected to be similar to the rate FasTrak transactions are completed (each lane can accommodate 1,100 vph). Implementation of the Toll Plan would likely increase the toll collection rate at the Toll Plaza in the Design Year Conditions and reduce the number of tollbooths required.

Toll Plaza Demand
A summary of the traffic demand and the transaction types at the Golden Gate Bridge Toll Plaza is included in **Table 15**.

Table 15: Toll Collection Data Summary – Design Year Conditions

Peak Hour	Volume (vehicles)		Design Year Conditions Toll Collection Type[a]			
	Existing	Design Year	FasTrak	Cash	Carpool	Other
Weekday AM	5,182	6,567	95.4%	0.0%	4.1%	0.5%
Weekday PM	3,524	5,097	95.4%	0.0%	4.1%	0.5%
Saturday	3,589	5,403	99.5%	0.0%	0.0%	0.5%
Sunday	3,987	5,188	99.5%	0.0%	0.0%	0.5%

Source: AECOM – February 2013
[a] Design Year Conditions analysis assumes full implementation of the Toll Plan. "Carpool" and "Other" toll collection types assumed to be weekly average. Other vehicle toll collection is assumed to be similar to cash transactions.

Toll Plaza Analysis
Based on the empirical data provided in the Doyle Drive Report Addendum and the Design Year Conditions peak hour traffic volumes, the minimum number of tollbooths that are expected to be required in the Design Year Conditions are included in **Table 16**.

Table 16: Toll Plaza Analysis – Design Year Conditions

Peak Hour	Tollbooths Available[(a)]	Minimum Tollbooths Required		
		FasTrak[(b)]	General[(c)]	Total
Weekday AM	8	6	1	7
Weekday PM	8	5	1	6
Saturday	8	5	1	6
Sunday	8	5	1	6

Source: AECOM – February 2013
Notes:
- "General" tollbooths accommodate FasTrak, transit and carpool transactions.
(a) The number of tollbooths available includes inactive tollbooths but not tollbooths that are serving northbound traffic.
(b) Design Year Conditions analysis assumes full implementation of the Toll Plan.
(c) A minimum of one tollbooth would be provided to accommodate transit and carpool demand.

Under Design Year Conditions, the number of tollbooths available would be expected to meet or exceed the number of tollbooths required during the weekday AM and PM and Saturday and Sunday peak hours.

Typically, no more than seven lanes would be required at the Toll Plaza in the Design Year Conditions with the incorporation of the Toll Plan. No queuing or congestion would be expected at the Toll Plaza.

7.4 Design Year (2030) Plus Project Conditions
The following section assesses the effects of the Proposed Project on the operations of the Toll Plaza in the Design Year Conditions. This includes an evaluation of the proposed capacity, projected demand, and the corresponding analysis.

Toll Plaza Capacity
In the Design Year plus Project Conditions, the barrier transfer machines would be stored approximately 750 feet to the south of the Toll Plaza with the District right-of-way. All eight of the tollbooths could be functional regardless of the lane configuration on the GGB.

Toll Plaza Demand
With the Proposed Project, the traffic volumes approaching the Toll Plaza would not be expected to differ from the Design Year Conditions traffic volumes. Implementation of the Toll Plan is also assumed under Design Year plus Project Conditions. Therefore, the transaction composition and toll collection types would be similar to Design Year Conditions.

Toll Plaza Analysis
Based on the empirical data provided in the Doyle Drive Report Addendum and the Design Year Conditions peak hour traffic volumes, the number of tollbooths available and the demand by collection type for Design Year plus Project Conditions are included in **Table 17**.

Table 17: Toll Plaza Analysis – Design Year Plus Project Conditions

Peak Hour	Tollbooths Available[a]	FasTrak			General		
		Booths[b]	Demand	Unserved Demand	Booths[c]	Demand	Unserved Demand
Weekday AM	8	6	6,265	-	1	302	-
Weekday PM	8	5	4,863	-	1	234	-
Saturday	8	5	5,376	-	1	27	-
Sunday	8	5	5,162	-	1	26	-

Source: AECOM – February 2013
Notes:
- "General" tollbooths accommodate FasTrak, transit, and carpool transactions.
- Demand measured in vehicles per hour.
(a) The number of tollbooths available includes inactive tollbooths but not tollbooths that are serving northbound traffic.
(b) Design Year Conditions analysis assumes full implementation of the Toll Plan.
(c) A minimum of one tollbooth would be provided to accommodate transit and carpool demand.

Under Design Year plus Project Conditions, the number of tollbooths available would be expected to meet or exceed the number of tollbooths required during the weekday AM and PM and Saturday and Sunday peak hours.

Typically, no more than seven lanes would be required at the Toll Plaza in the Design Year plus Project Conditions with the incorporation of the Toll Plan.

Summary
An eight tollbooth configuration could adequately accommodate the project demand under Design Year plus Project Conditions during the weekday AM and PM and Saturday and Sunday peak hours.

With the implementation of the Toll Plan, no more than seven lanes would be required at the Toll Plaza in the Design Year plus Project Conditions. No queuing or congestion would be expected at the Toll Plaza.

8.0 Traffic Operations

This section addresses the effects of the Proposed Project on the traffic operations. This assessment addresses traffic speeds, volumes, and measures of effectiveness.

8.1 Existing (2012) Conditions

The freeway network was simulated using the CORSIM model for the weekday AM and PM peak periods. The lane schematic diagrams, which contain the results of the simulation, are shown in **Figure 21**. The simulated peak hour freeway speeds are shown in **Figure 22**.

The lane schematic diagrams include MOE that were extracted from the CORSIM model that represent the peak hour conditions. The operations for each of the mainline freeway ramp junctions are included for the following locations:

- SB US 101 diverge section at Sausalito Lateral;
- SB US 101 merge section at Sausalito Lateral;
- SB diverge section at Merchant Road;
- SB merge section at Merchant Road;
- SB weaving section between the Toll Plaza and Park Presidio Boulevard;
- NB weaving section between Park Presidio Boulevard and the Toll Plaza;
- NB diverge section at Merchant Road;
- NB merge section at Merchant Road;
- NB US 101 diverge section at Vista Point; and
- NB US 101 weaving section between Vista Point and Sausalito Lateral.

The vehicle density and speed at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM models and is included in **Table 18.**

The simulated traffic volumes on the GGB and US 101 mainline freeway were compared to vehicle demand volumes. The traffic volumes at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM model and are included in **Table 19**.

A workbook containing the initial CORSIM modeling data is included in **Appendix E**. The CORSIM modeling workbook includes the following information:

- Link-Node Diagram;
- Quality Assurance / Quality Control Database;
- Traffic Volumes Database; and
- Simulation Output.

Figure 21a: Existing Conditions Freeway Operations - Measures of Effectiveness - Weekday AM Peak Hour

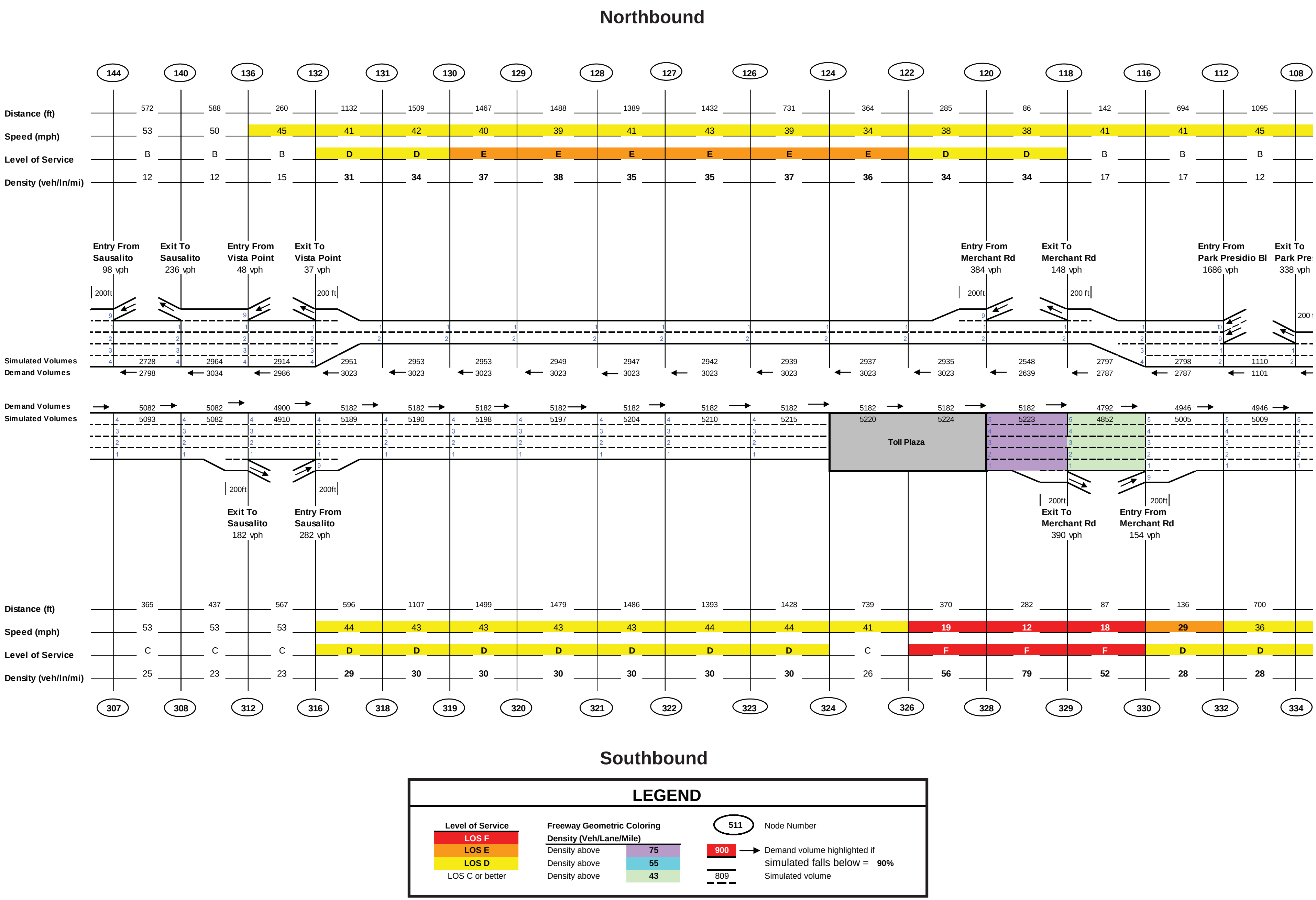


Figure 21b: Existing Conditions Freeway Operations - Measures of Effectiveness - Weekday PM Peak Hour

Northbound

Segment (nodes)	144–140	140–136	136–132	132–131	131–130	130–129	129–128	128–127	127–126	126–124	124–122	122–120	120–118	118–116	116–112	112–108
Distance (ft)	572	588	260	1132	1509	1467	1488	1389	1432	731	364	285	86	142	694	1095
Speed (mph)	51	51	45	40	42	42	42	42	43	39	22	16	10	34	34	43
Level of Service	C	C	D	E	E	E	E	E	E	F	F	F	F	D	D	C
Density (veh/ln/mi)	22	20	28	38	40	40	40	40	39	43	75	70	69	32	32	22
Simulated Volumes	4766	5086	4965	5071	5074	5074	5076	5076	5076	5079	5079	5082	4517	4761	4767	2984
Demand Volumes	4864	5186	5067	5171	5171	5171	5171	5171	5171	5171	5171	5171	4605	4776	4776	2983

Ramps (Northbound):
- Entry From Sausalito 180 vph (node 144), 100ft
- Exit To Sausalito 322 vph (node 140)
- Entry From Vista Point 119 vph (node 136)
- Exit To Vista Point 104 vph (node 132), 100ft
- Entry From Merchant Rd 566 vph (node 120), 100ft
- Exit To Merchant Rd 171 vph (node 118), 100ft
- Entry From Park Presidio Bl 1793 vph (node 112)
- Exit To Park Presi 581 vph (node 108), 100ft

Southbound

Segment (nodes)	307–308	308–312	312–316	316–318	318–319	319–320	320–321	321–322	322–323	323–324	324–326	326–328	328–329	329–330	330–332	332–334
Demand Volumes	3284	3284	3150	3524	3524	3524	3524	3524	3524	3524	3524	3524	3524	3164	3408	3408
Simulated Volumes	3288	3284	3155	3530	3533	3535	3539	3542	3540	3541	3541	3541	3543	3186	3432	3431
Distance (ft)	365	437	567	596	1107	1499	1479	1486	1393	1428	739	370	282	87	136	700
Speed (mph)	53	53	53	44	43	42	43	43	43	43	43	27	12	18	29	29
Level of Service	B	B	C	C	C	C	C	C	C	C	C	D	F	F	C	C
Density (veh/ln/mi)	17	19	20	25	27	27	27	27	27	27	27	32	60	43	26	26

Toll Plaza (between nodes 324 and 328)

Ramps (Southbound):
- Exit To Sausalito 134 vph (node 312), 100ft
- Entry From Sausalito 374 vph (node 316), 100ft
- Exit To Merchant Rd 360 vph (node 329), 100ft
- Entry From Merchant Rd 244 vph (node 330), 100ft

LEGEND

Level of Service
- LOS F
- LOS E
- LOS D
- LOS C or better

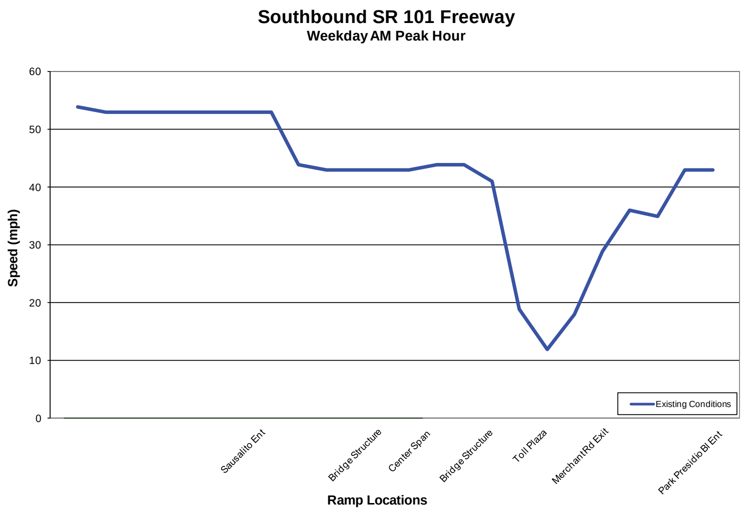
Freeway Geometric Coloring
Density (Veh/Lane/Mile)
- Density above 75
- Density above 55
- Density above 43

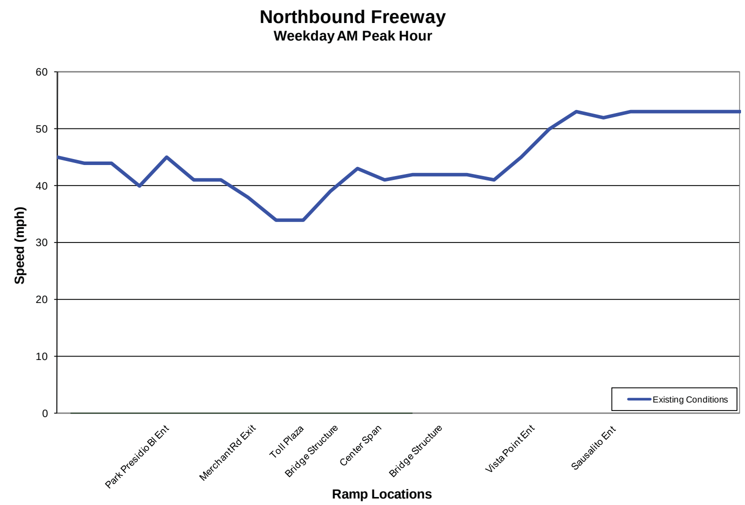
511 — Node Number

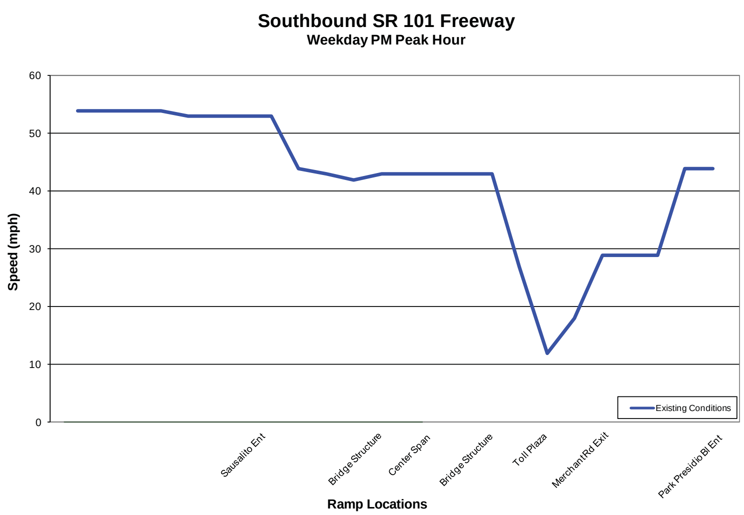
900 → Demand volume highlighted if simulated falls below = 90%

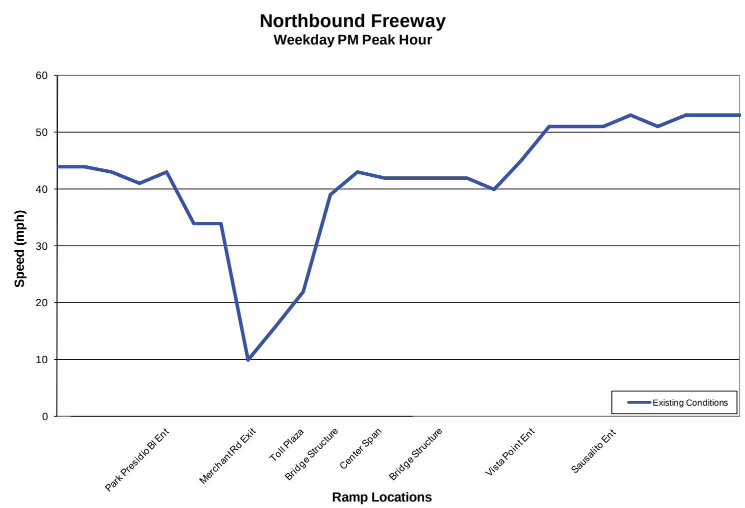
809 — Simulated volume

Figure 22: Existing Conditions Freeway Operations – Travel Speeds









Existing Travel Speeds rev3.indd

Table 18: Freeway Measures of Effectiveness – Existing Conditions

Freeway Section	Weekday AM Peak Hour			Weekday PM Peak Hour		
	Density	LOS	Speed	Density	LOS	Speed
Southbound GGB and US 101 Freeway						
Diverge at Sausalito Lateral	23	C	53	19	B	53
Merge at Sausalito Lateral	29	D	44	25	C	44
Diverge at Merchant Rd.	79	F	12	60	F	12
Merge at Merchant Rd.	28	D	29	26	C	29
Weave between Toll Plaza and Park Presidio Bl.	28	D	36	26	C	29
Northbound GGB and US 101 Freeway						
Weave between Park Presidio Bl. and Toll Plaza	17	B	41	32	D	34
Diverge at Merchant Rd.	17	B	41	32	D	34
Merge at Merchant Rd.	34	D	38	70	F	16
Diverge at Vista Point	31	D	41	38	E	40
Weave between Vista Point and Sausalito Lateral	12	B	50	20	C	51
Merge at Sausalito Lateral	14	B	52	23	C	51

Source: AECOM – February 2013
Notes:
- Density in terms of passenger cars / lane / mile.
- Speed in terms of miles per hour.

Table 19: Freeway Traffic Volumes – Existing Conditions

Freeway Section	Weekday AM Peak Hour		Weekday PM Peak Hour	
	Demand	Simulated	Demand	Simulated
Southbound GGB and US 101 Freeway				
Diverge at Sausalito Lateral	5,082	5,082	3,284	3,284
Merge at Sausalito Lateral	5,182	5,189	3,524	3,530
Diverge at Merchant Rd.	5,182	5,223	3,524	3,543
Merge at Merchant Rd.	4,946	5,005	3,408	3,432
Weave between Toll Plaza and Park Presidio Bl.	4,946	5,009	3,408	3,431
Northbound GGB and US 101 Freeway				
Weave between Park Presidio Bl. And Toll Plaza	2,787	2,798	4,776	4,767
Diverge at Merchant Rd.	2,787	2,797	4,776	4,761
Merge at Merchant Rd.	3,023	2,935	5,171	5,082
Diverge at Vista Point	3,023	2,951	5,171	5,071
Weave between Vista Point and Sausalito Lateral	3,034	2,964	5,186	5,086
Merge at Sausalito Lateral	2,896	2,825	5,044	4,947

Source: AECOM – February 2013
Notes:
- Volume in terms of vehicles per hour.
- Demand represents the peak hour mainline traffic volume demand.
- Simulated represents the peak hour mainline traffic demand volume that is served in the freeway simulation.

8.2 Existing (2012) Plus Project Conditions

The Proposed Project would not be expected to affect traffic volumes. The traffic volumes presented in the Existing plus Project Conditions analysis are assumed to be consistent with the Existing Conditions traffic volumes.

The freeway network was simulated using the CORSIM model for the weekday AM and PM peak periods. The lane schematic diagrams, which contain the results of the simulation, are shown in **Figure 23**. The simulated peak hour freeway speeds are shown in **Figure 24**.

The vehicle density and speed at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM models and is included in **Table 20.**

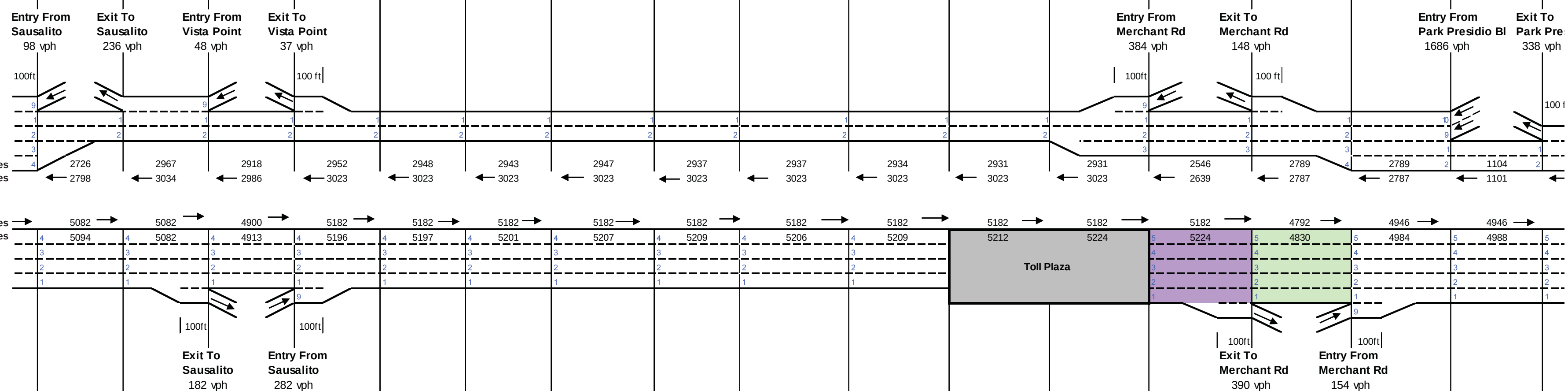
The Proposed Project would have minimal affects on the freeway operations in the Existing Conditions. With the removal of the reverse direction tollbooths, northbound traffic speeds would increase slightly near the Toll Plaza. The density would increase slightly at locations where the roadway width was reduced. This would occur at the northbound US 101 diverge section at Vista Point and at the weaving section between Vista Point and Sausalito Lateral.

The simulated traffic volumes on the mainline freeway were compared to vehicle demand volumes. The traffic volumes at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM model and are included in **Table 21**.

Figure 23a: Existing Plus Project Conditions Freeway Operations - Measures of Effectiveness - Weekday AM Peak Hour

Northbound

Segment (nodes)	144–140	140–136	136–132	132–131	131–130	130–129	129–128	128–127	127–126	126–124	124–122	122–120	120–118	118–116	116–112	112–108
Distance (ft)	572	473	368	1132	1509	1467	1488	1389	1432	731	364	285	86	142	694	1095
Speed (mph)	52	47	43	43	39	39	39	38	40	39	37	37	40	40	40	45
Level of Service	C	C	D	D	E	E	E	E	E	E	D	D	D	B	B	B
Density (veh/ln/mi)	20	27	33	33	38	38	38	38	36	38	32	32	32	17	17	12
Simulated Volumes	2726	2967	2918	2952	2948	2943	2947	2937	2937	2934	2931	2931	2546	2789	2789	1104
Demand Volumes	2798	3034	2986	3023	3023	3023	3023	3023	3023	3023	3023	3023	2639	2787	2787	1101



Segment (nodes)	307–308	308–312	312–316	316–318	318–319	319–320	320–321	321–322	322–323	323–324	324–326	326–328	328–329	329–330	330–332	332–334
Demand Volumes	5082	5082	4900	5182	5182	5182	5182	5182	5182	5182	5182	5182	5182	4792	4946	4946
Simulated Volumes	5094	5082	4913	5196	5197	5201	5207	5209	5206	5209	5212	5224	5224	4830	4984	4988
Distance (ft)	373	455	567	596	1107	1499	1479	1486	1393	1428	739	370	282	87	136	700
Speed (mph)	53	50	43	34	39	40	40	40	40	40	40	17	12	18	31	33
Level of Service	C	C	C	D	D	D	D	D	D	D	D	F	F	F	D	D
Density (veh/ln/mi)	25	24	23	31	33	32	32	32	32	32	28	59	78	52	28	29

Southbound

LEGEND

Level of Service
- LOS F
- LOS E
- LOS D
- LOS C or better

Freeway Geometric Coloring
Density (Veh/Lane/Mile)
- Density above 75
- Density above 55
- Density above 43

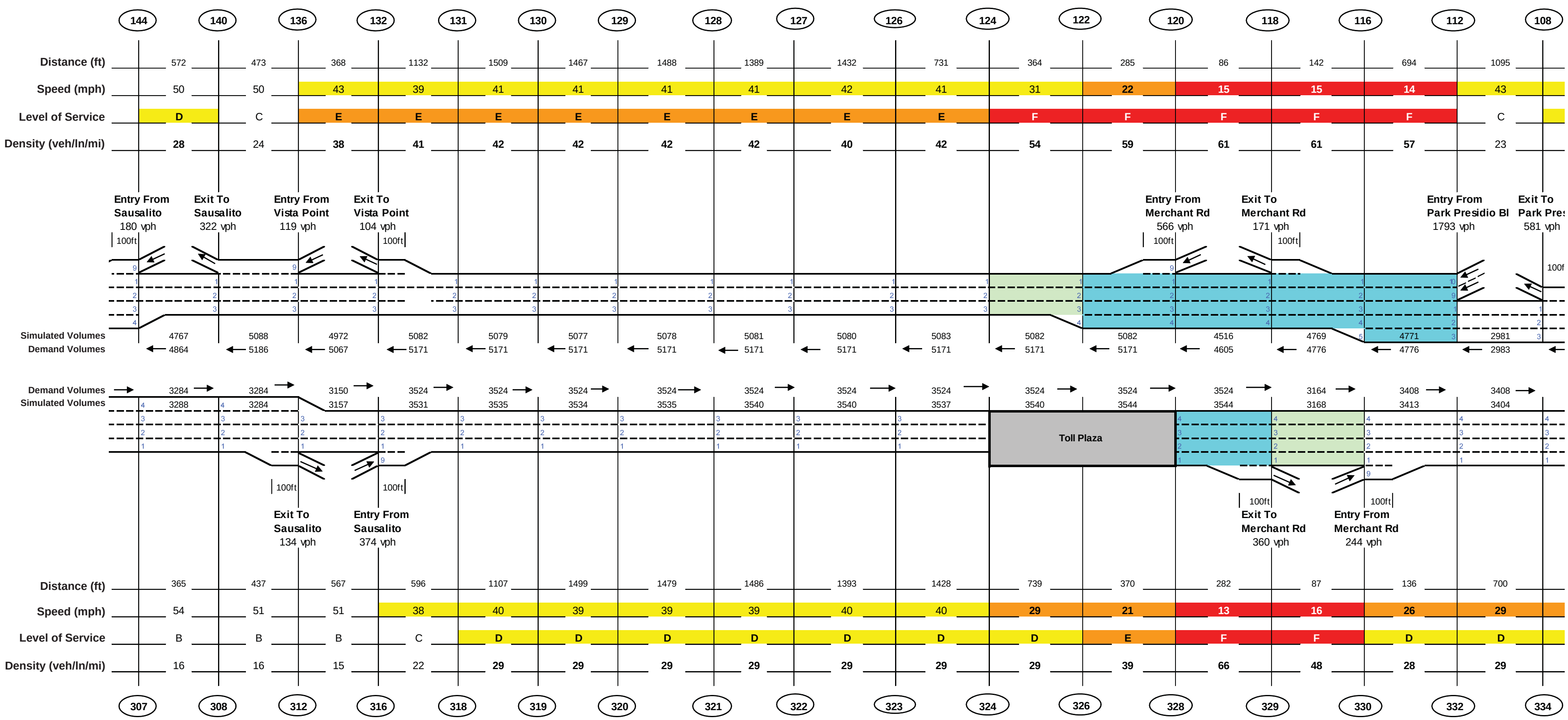
511 Node Number

900 → Demand volume highlighted if simulated falls below = **90%**

809 Simulated volume

Figure 23b: Existing Plus Project Conditions Freeway Operations - Measures of Effectiveness - Weekday PM Peak Hour

Northbound



Southbound

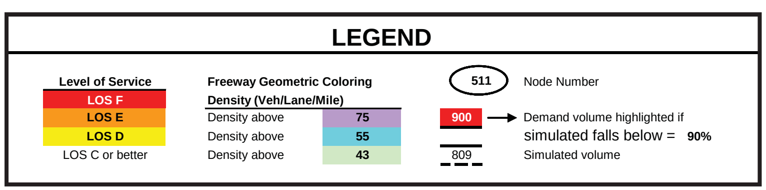
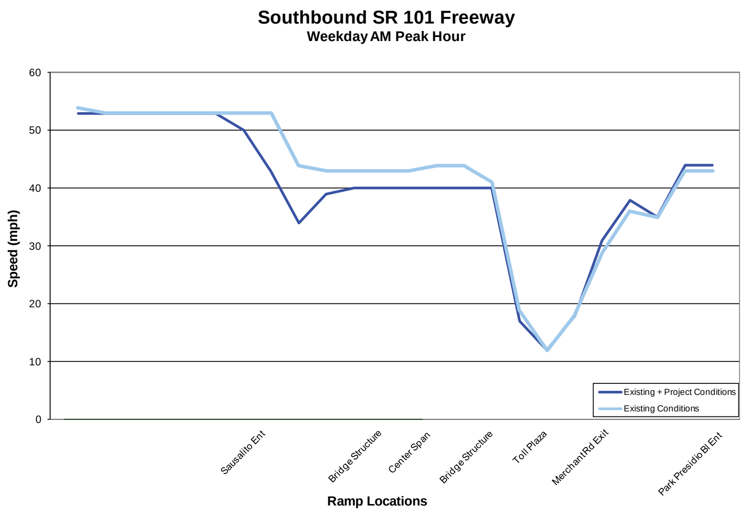
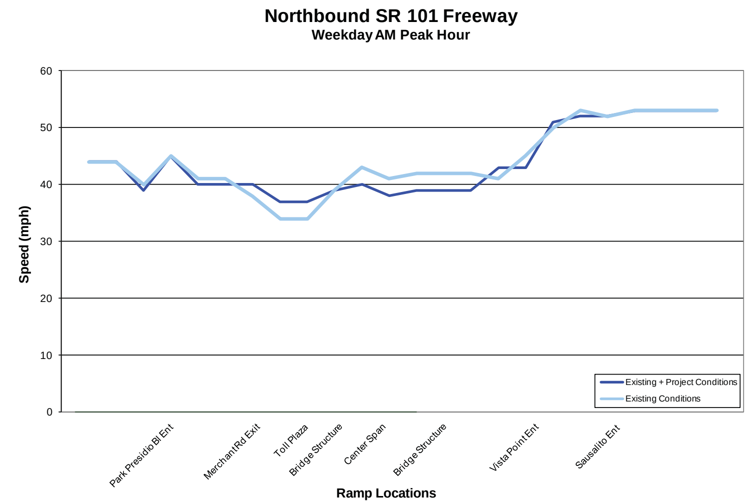
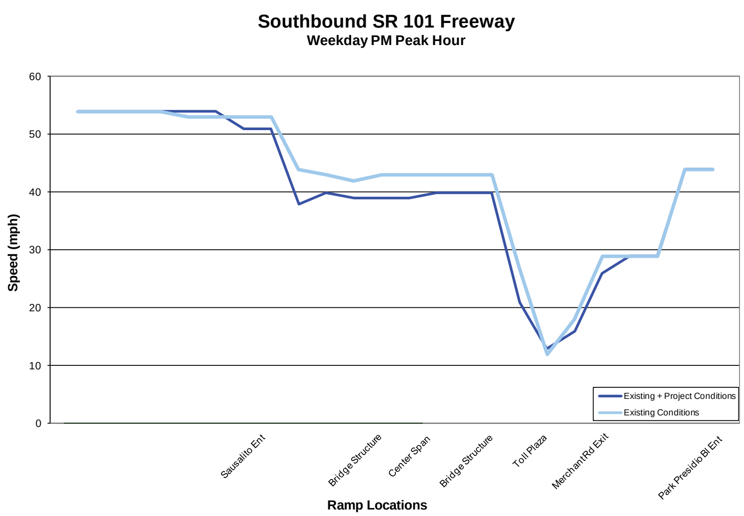


Figure 24: Existing Plus Project Conditions Freeway Operations – Travel Speeds







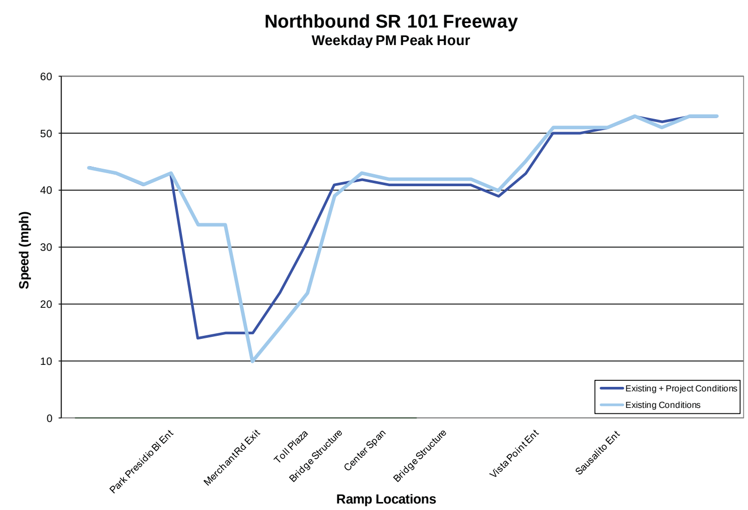


Table 20: Freeway Measures of Effectiveness – Existing Plus Project Conditions

Freeway Section	Peak Hour	Existing Conditions			Existing + Project Conditions		
		Density	LOS	Speed	Density	LOS	Speed
Southbound GGB and US 101 Freeway							
Diverge at Sausalito Lateral	AM	23	C	53	24	C	50
	PM	19	B	53	16	B	51
Merge at Sausalito Lateral	AM	29	D	44	31	D	34
	PM	25	C	44	22	C	38
Diverge at Merchant Rd.	AM	79	F	12	78	F	12
	PM	60	F	12	66	F	13
Merge at Merchant Rd.	AM	28	D	29	31	D	28
	PM	26	C	29	28	D	26
Weave between Toll Plaza and Park Presidio Bl.	AM	28	D	36	29	D	29
	PM	26	C	29	29	D	29
Northbound GGB and US 101 Freeway							
Weave between Park Presidio Bl. and Toll Plaza	AM	17	B	41	17	B	40
	PM	69	F	10	57	F	14
Diverge at Merchant Rd.	AM	17	B	41	17	B	40
	PM	70	F	16	61	F	15
Merge at Merchant Rd.	AM	34	D	38	32	D	37
	PM	75	F	22	59	F	22
Diverge at Vista Point	AM	31	D	41	33	D	43
	PM	38	E	40	41	E	39
Weave between Vista Point and Sausalito Lateral	AM	12	B	50	27	C	47
	PM	20	C	51	24	C	50
Merge at Sausalito Lateral	AM	14	B	52	14	B	52
	PM	23	C	51	23	C	51

Source: AECOM – February 2013
Notes:
- Density in terms of passenger cars / lane / mile.
- Speed in terms of miles per hour.

Table 21: Freeway Traffic Volumes – Existing Plus Project Conditions

Freeway Section	Peak Hour	Existing Conditions		Existing + Project Conditions	
		Demand	Simulated	Demand	Simulated
Southbound GGB and US 101 Freeway					
Diverge at Sausalito Lateral	AM	5,082	5,082	5,082	5,082
	PM	3,284	3,284	3,284	3,284
Merge at Sausalito Lateral	AM	5,182	5,189	5,182	5,196
	PM	3,524	3,530	3,524	3,531
Diverge at Merchant Rd.	AM	5,182	5,223	5,182	5,244
	PM	3,524	3,543	3,524	3,544
Merge at Merchant Rd.	AM	4,946	5,005	4,946	4,984
	PM	3,408	3,432	3,408	3,413
Weave between Toll Plaza and Park Presidio Bl.	AM	4,946	5,009	4,946	4,988
	PM	3,408	3,431	3,408	3,404
Northbound GGB and US 101 Freeway					
Weave between Park Presidio Bl. and Toll Plaza	AM	2,787	2,798	2,787	2,789
	PM	4,776	4,767	4,776	4,771
Diverge at Merchant Rd.	AM	2,787	2,797	2,787	2,804
	PM	4,776	4,761	4,776	4,769
Merge at Merchant Rd.	AM	3,023	2,935	3,023	2,931
	PM	5,171	5,082	5,171	5,082
Diverge at Vista Point	AM	3,023	2,951	3,023	2,952
	PM	5,171	5,071	5,171	5,082
Weave between Vista Point and Sausalito Lateral	AM	3,034	2,964	3,034	2,967
	PM	5,186	5,086	5,186	5,088
Merge at Sausalito Lateral	AM	2,896	2,825	2,896	2,767
	PM	5,044	4,947	5,044	4,948

Source: AECOM – February 2013
Notes:
- Volume in terms of vehicles per hour.
- Demand represents the peak hour mainline traffic volume demand.
- Simulated represents the peak hour mainline traffic demand volume that is served in the freeway simulation.

During the weekday AM and PM peak hours of the Existing plus Project Conditions, the roadway capacity could adequately serve the traffic demand in the southbound direction.

During the weekday AM peak hour of the Existing plus Project Conditions, the roadway capacity could adequately serve the traffic demand in the northbound direction.

During the weekday PM peak hour of the Existing plus Project Conditions, the traffic demand would be constrained by the roadway capacity in the northbound direction. With the implementation of the Proposed Project, the roadway capacity in the three lane direction would be expected to essentially remain the same (increase from 5,170 vph to 5,190 vph).

The CORSIM output is included in **Appendix F**.

8.3 Design Year (2030) Conditions

The freeway network was simulated using the CORSIM model for the weekday AM and PM peak periods. The lane schematic diagrams, which contain the results of the simulation, are shown in **Figure 25**. The simulated peak hour freeway speeds are shown in **Figure 26**.

The vehicle density and speed at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM models and is included in **Table 22**.

During the weekday AM and PM peak hours of the Design Year Conditions, the southbound ramp junctions near the Toll Plaza would operate at LOS F with low travel speeds. Elsewhere, the southbound ramp junctions would operate at LOS E or better and travel speeds would generally exceed 30 mph.

During the weekday AM and PM peak hours of the Design Year Conditions, the northbound ramp junctions upstream of the GGB would operate at LOS F with low travel speeds. The northbound ramp junctions downstream of the GGB would operate at LOS E or better and travel speeds would generally exceed 40 mph.

The simulated traffic volumes on the GGB and US 101 mainline freeway were compared to vehicle demand volumes. The traffic volumes at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM model and are included in **Table 23**.

Figure 25a: Design Year Conditions Freeway Operations - Measures of Effectiveness - Weekday AM Peak Hour

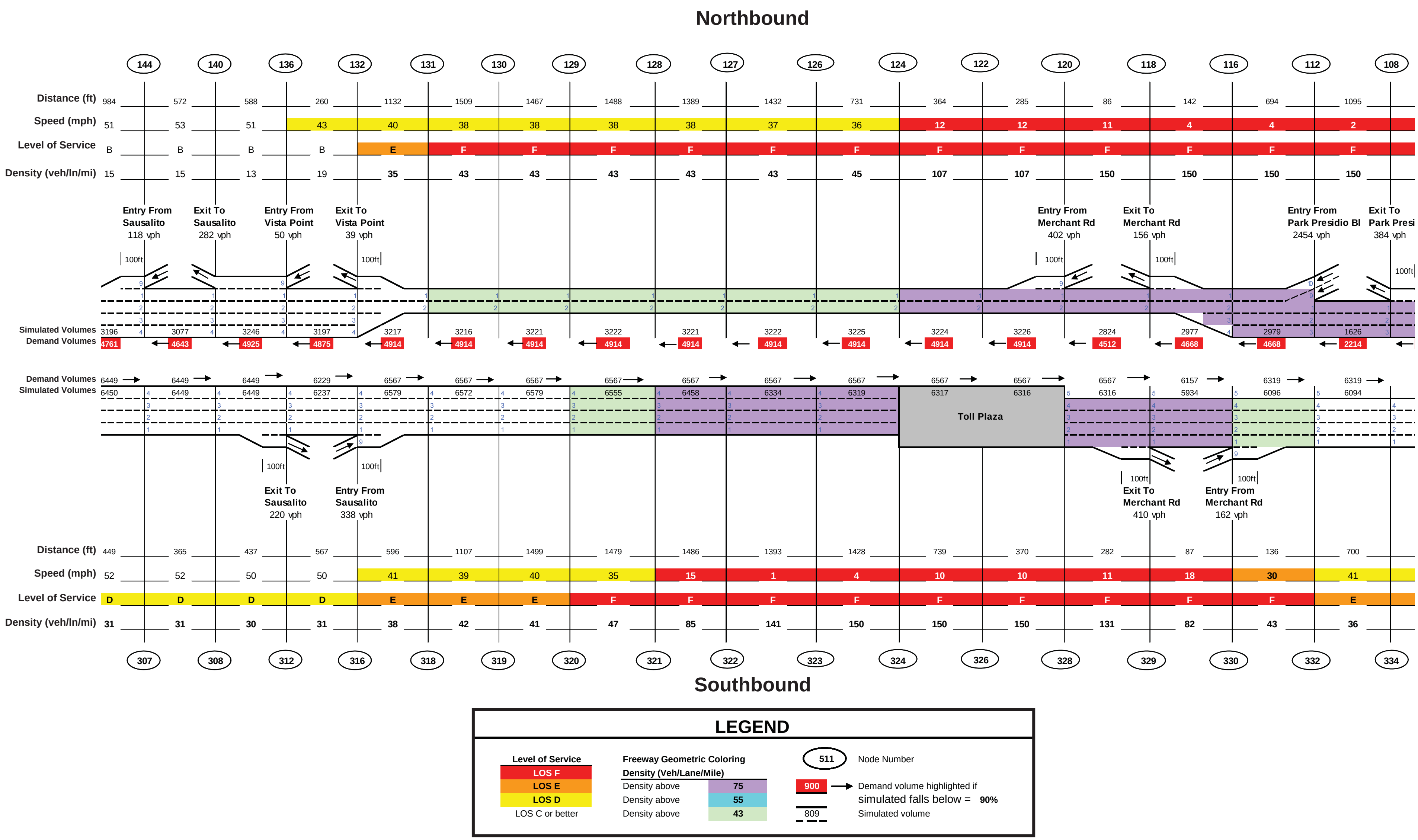


Figure 25b: Design Year Conditions Freeway Operations - Measures of Effectiveness - Weekday PM Peak Hour

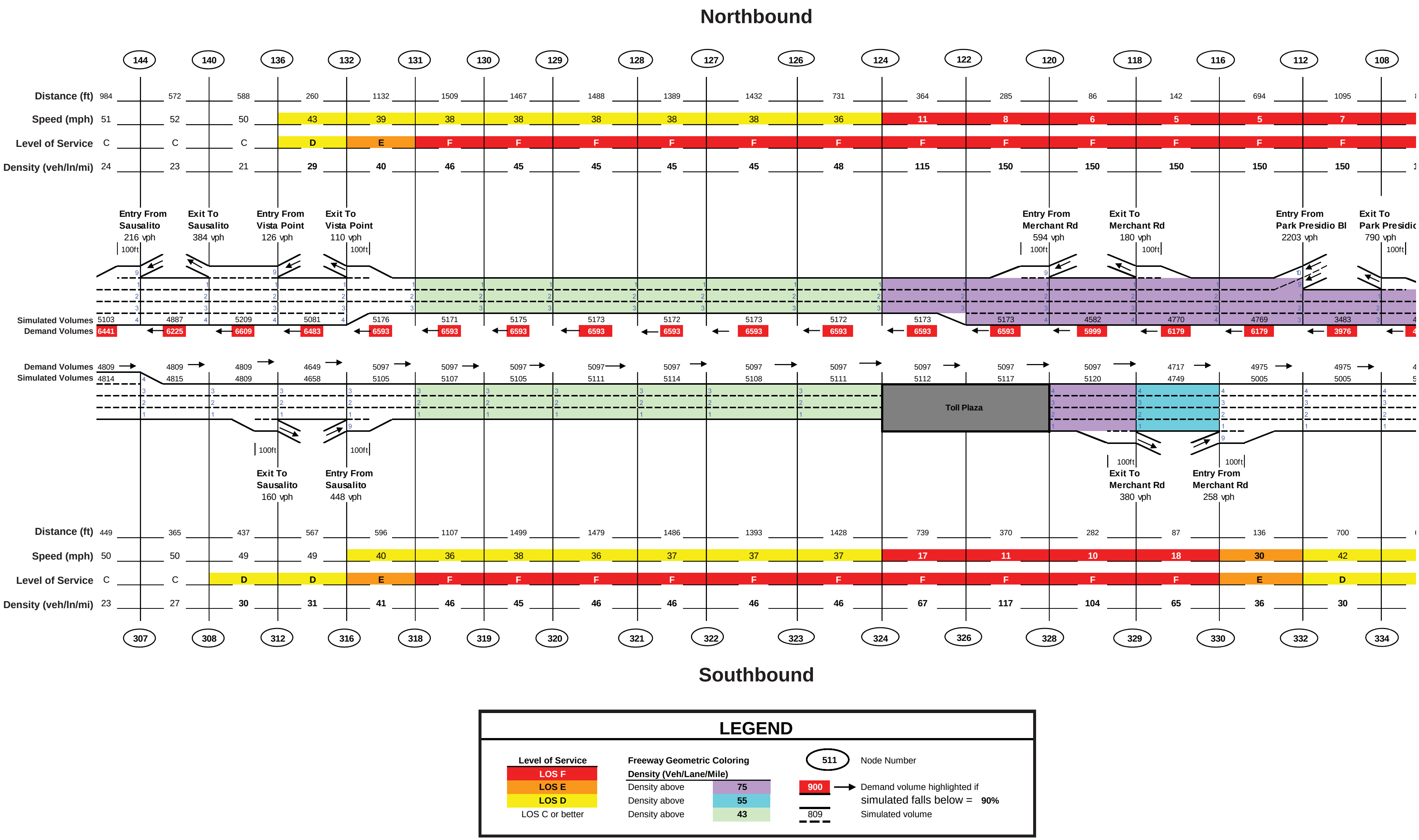
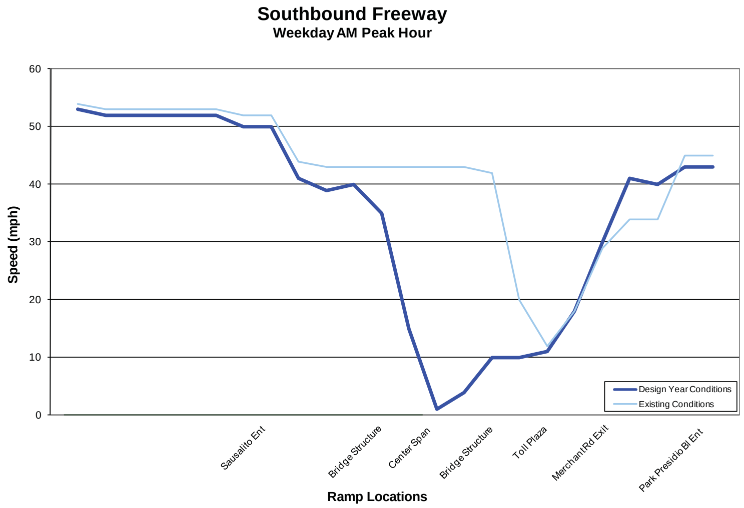
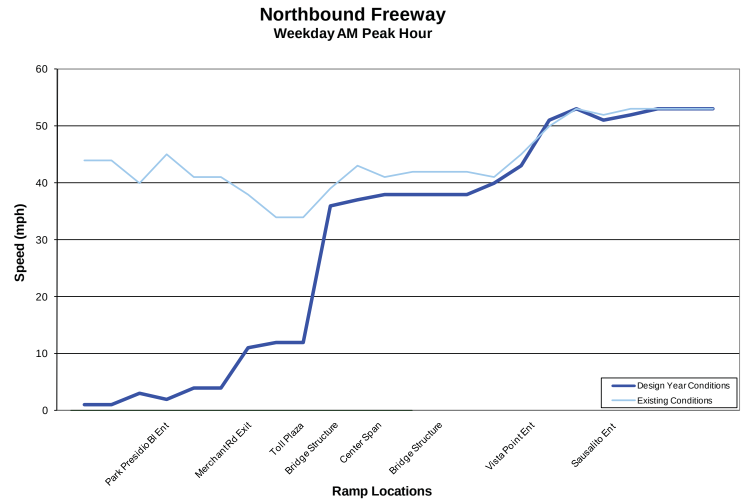
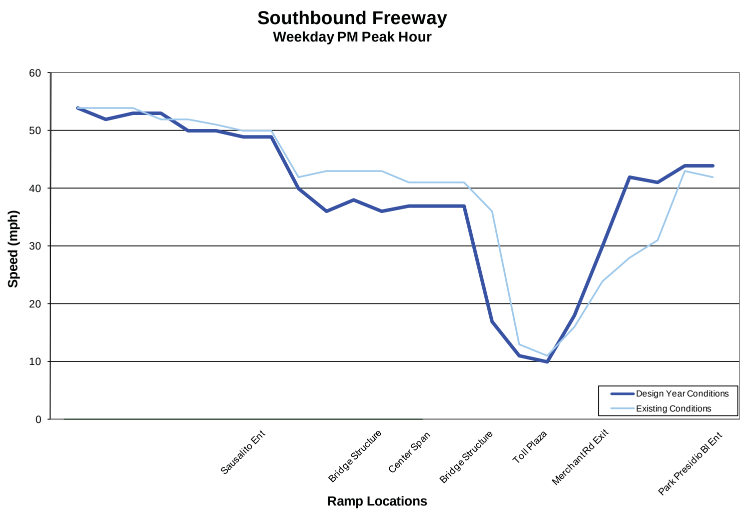
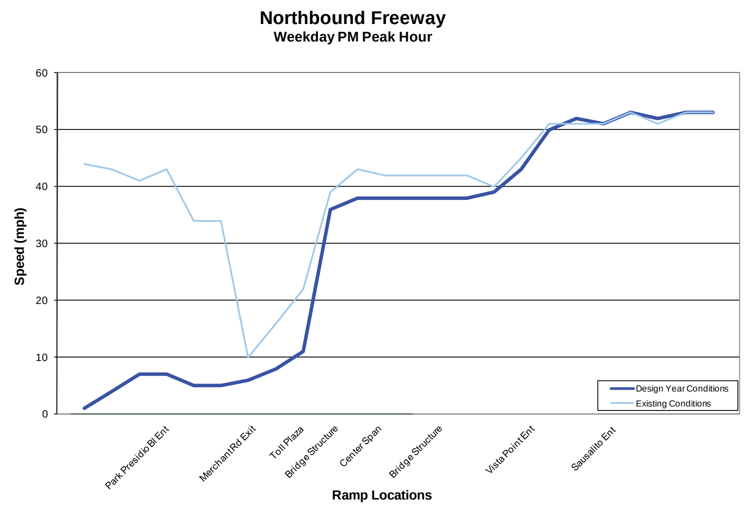


Figure 26: Design Year Conditions Freeway Operations – Travel Speeds









Design Year Travel Speeds rev1.indd

Table 22: Freeway Measures of Effectiveness – Design Year Conditions

Freeway Section	Peak Hour	Existing Conditions			Design Year Conditions		
		Density	LOS	Speed	Density	LOS	Speed
Southbound GGB and US 101 Freeway							
Diverge at Sausalito Lateral	AM	23	C	53	31	D	50
	PM	19	B	53	31	D	49
Merge at Sausalito Lateral	AM	29	D	44	38	E	41
	PM	25	C	44	41	E	40
Diverge at Merchant Rd.	AM	79	F	12	131	F	11
	PM	60	F	12	104	F	10
Merge at Merchant Rd.	AM	28	D	29	43	F	30
	PM	26	C	29	36	E	30
Weave between Toll Plaza and Park Presidio Bl.	AM	29	D	36	38	E	40
	PM	26	C	29	32	D	41
Northbound GGB and US 101 Freeway							
Weave between Park Presidio Bl. and Toll Plaza	AM	17	B	41	150	F	4
	PM	69	F	10	150	F	5
Diverge at Merchant Rd.	AM	17	B	41	150	F	11
	PM	70	F	16	150	F	8
Merge at Merchant Rd.	AM	34	D	38	107	F	12
	PM	75	F	22	115	F	11
Diverge at Vista Point	AM	31	D	41	35	E	30
	PM	38	E	40	40	E	39
Weave between Vista Point and Sausalito Lateral	AM	12	B	50	13	B	51
	PM	20	C	51	21	C	50
Merge at Sausalito Lateral	AM	14	B	52	15	B	51
	PM	23	C	51	24	C	51

Source: AECOM – February 2013
Notes:
- Density in terms of passenger cars / lane / mile.
- Speed in terms of miles per hour.

Table 23: Freeway Traffic Volumes – Design Year Conditions

Freeway Section	Peak Hour	Existing Conditions		Design Year Conditions	
		Demand	Simulated	Demand	Simulated
Southbound GGB and US 101 Freeway					
Diverge at Sausalito Lateral	AM	5,082	5,082	6,449	6,449
	PM	3,284	3,284	4,809	4,811
Merge at Sausalito Lateral	AM	5,182	5,189	6,567	6,579
	PM	3,524	3,530	5,097	5,105
Diverge at Merchant Rd.	AM	5,182	5,223	6,567	6,316
	PM	3,524	3,543	5,097	5,120
Merge at Merchant Rd.	AM	4,946	5,005	6,319	6,096
	PM	3,408	3,432	4,975	5,005
Weave between Toll Plaza and Park Presidio Bl.	AM	4,946	5,009	6,319	6,094
	PM	3,408	3,431	4,975	5,004
Northbound GGB and US 101 Freeway					
Weave between Park Presidio Bl. and Toll Plaza	AM	2,787	2,798	4,668	2,979
	PM	4,776	4,767	6,179	4,769
Diverge at Merchant Rd.	AM	2,787	2,797	4,668	2,979
	PM	4,776	4,761	6,179	4,770
Merge at Merchant Rd.	AM	3,023	2,935	4,914	3,226
	PM	5,171	5,082	6,593	5,173
Diverge at Vista Point	AM	3,023	2,951	4,914	3,217
	PM	5,171	5,071	6,593	5,176
Weave between Vista Point and Sausalito Lateral	AM	3,034	2,964	4,925	3,246
	PM	5,186	5,086	6,609	5,209
Merge at Sausalito Lateral	AM	2,896	2,825	4,761	3,196
	PM	5,044	4,947	6,441	5,103

Source: AECOM – February 2013
Notes:
- Volume in terms of vehicles per hour.
- Demand represents the peak hour mainline traffic volume demand.
- Simulated represents the peak hour mainline traffic demand volume that is served in the freeway simulation.

Queuing Analysis

During the weekday AM peak hour of the Design Year Conditions, the northbound traffic volume demand would be constrained by the limited capacity of the GGB, which would be in a two northbound lanes / four southbound lanes configuration. The demand would generally exceed the capacity of the GGB by approximately 2,000 vph. These vehicles would experience significant delay and queue onto Doyle Drive and Park Presidio Boulevard. An assessment of vehicle queue lengths was conducted by determining the proportion of vehicles originating from each freeway facility and assuming an average vehicle length of 25 feet. Based on this analysis, the queue would extend back from the Toll Plaza approximately 1.9 miles south on Park Presidio Boulevard and approximately 1.7 miles east on Doyle Drive. Vehicle queues on Doyle Drive may extend beyond Marina Boulevard and Richardson Avenue and cause congestion on local roadway facilities.

The lanes on the GGB could be modified to utilize the three northbound lanes / three southbound lanes configuration during the weekday AM peak hour to improve the northbound capacity. One additional lane in the northbound direction could potentially increase the northbound capacity by 1,900 vph, which would accommodate the excess demand; however, this would negatively impact the southbound traffic operations as the capacity of the GGB would be reduced. With implementation of the project, the GGB capacity in the two lane configuration would increase by approximately 10 vph. Therefore, the queue length would be reduced by approximately 250 feet under Design Year plus Project Conditions and the project would be expected to reduce the duration of congestion.

During the weekday PM peak hour of the Design Year Conditions, the northbound traffic volume demand would be constrained by the limited capacity of the GGB, which would be in a three northbound lanes / three southbound lanes configuration. The demand would generally exceed the capacity of the GGB by approximately 1,500 vph. These vehicles would experience significant delay and queue onto Doyle Drive and Park Presidio Boulevard. Based on the queuing analysis, the queue would extend back from the Toll Plaza approximately 1.0 miles south on Park Presidio Boulevard and approximately 1.5 miles east on Doyle Drive. Vehicle queues on Doyle Drive may extend beyond Marina Boulevard and Richardson Avenue and cause congestion on local roadway facilities.

The lanes on the GGB could be modified to utilize the four northbound lanes / two southbound lanes configuration during the weekday PM peak hour to improve the northbound capacity. One additional lane in the northbound direction could potentially increase the northbound capacity by 1,800 vph, which would accommodate the excess demand; however, this would negatively impact the southbound traffic operations as the capacity of the GGB would be reduced. With implementation of the project, the GGB capacity in the three lane configuration would increase by approximately 20 vph. Therefore, the queue length would be reduced by approximately 500 feet under Design Year plus Project Conditions than and the project would be expected to reduce the duration of congestion.

8.4 Design Year (2030) Plus Project Conditions

The Proposed Project would not be expected to affect traffic volumes. The traffic volumes presented in the Design Year plus Project Conditions analysis are assumed to be consistent with the Design Year Conditions traffic volumes.

The freeway network was simulated using the CORSIM model for the weekday AM and PM peak periods. The lane schematic diagrams, which contain the results of the simulation, are shown in **Figure 27**. The simulated peak hour freeway speeds are shown in **Figure 28**.

The vehicle density and speed at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM models and is included in **Table 24**.

The Proposed Project would have minimal affects on the freeway operations in the Design Year plus Project Conditions. With the implementation of the Proposed Project, the Level of Service at most of the ramp junctions would remain the same or improve. One of the factors in the improved operations is the increase in roadway capacity. The modifications to the roadway characteristics would be expected to slightly increase the overall capacity of the roadway in all three of the roadway configurations.

Detailed lane capacity calculations are included in **Appendix C**.

Figure 27a: Design Year Plus Project Conditions Freeway Operations - Measures of Effectiveness - Weekday AM Peak Hour

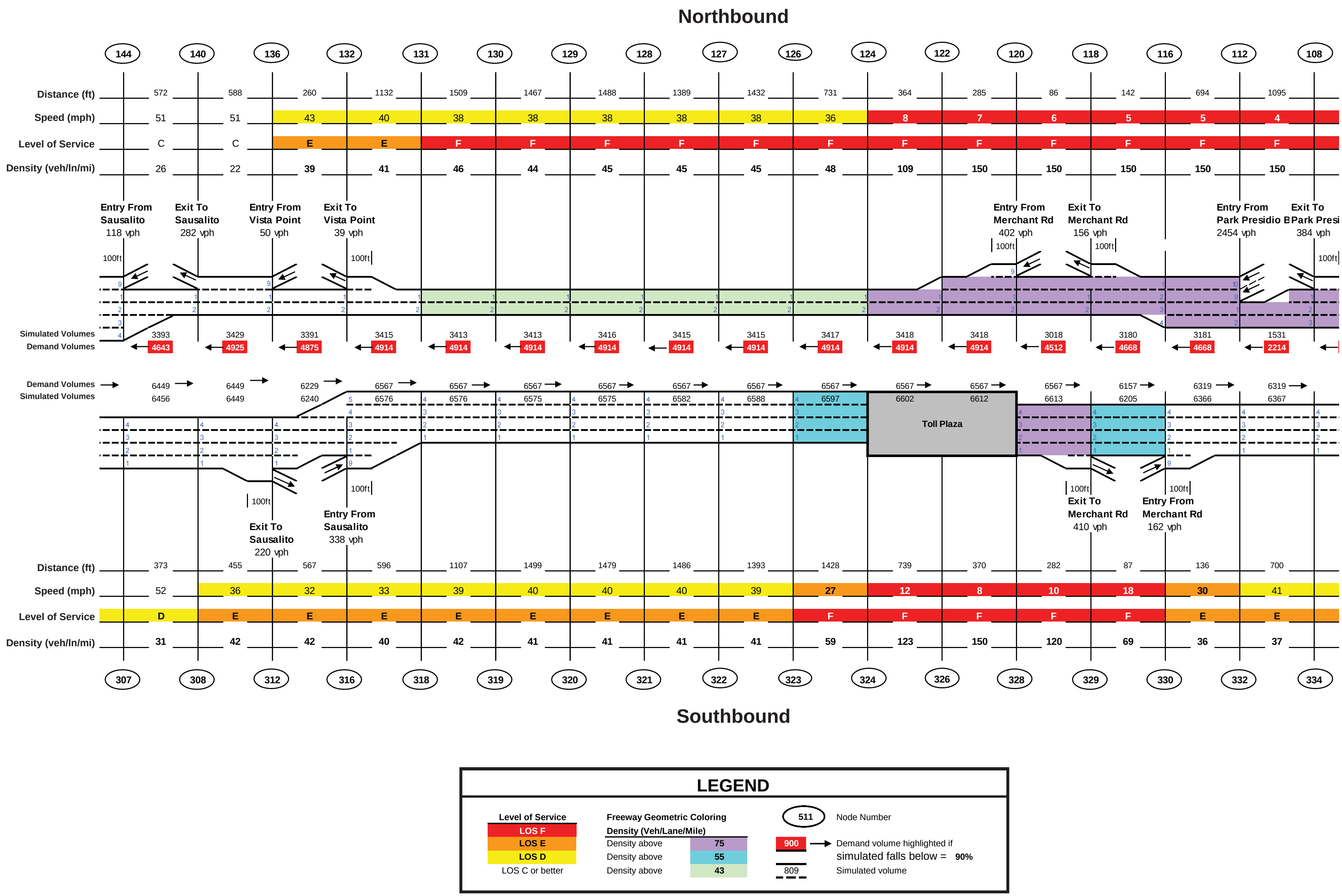


Figure 27b: Design Year Plus Project Conditions Freeway Operations - Measures of Effectiveness - Weekday PM Peak Hour

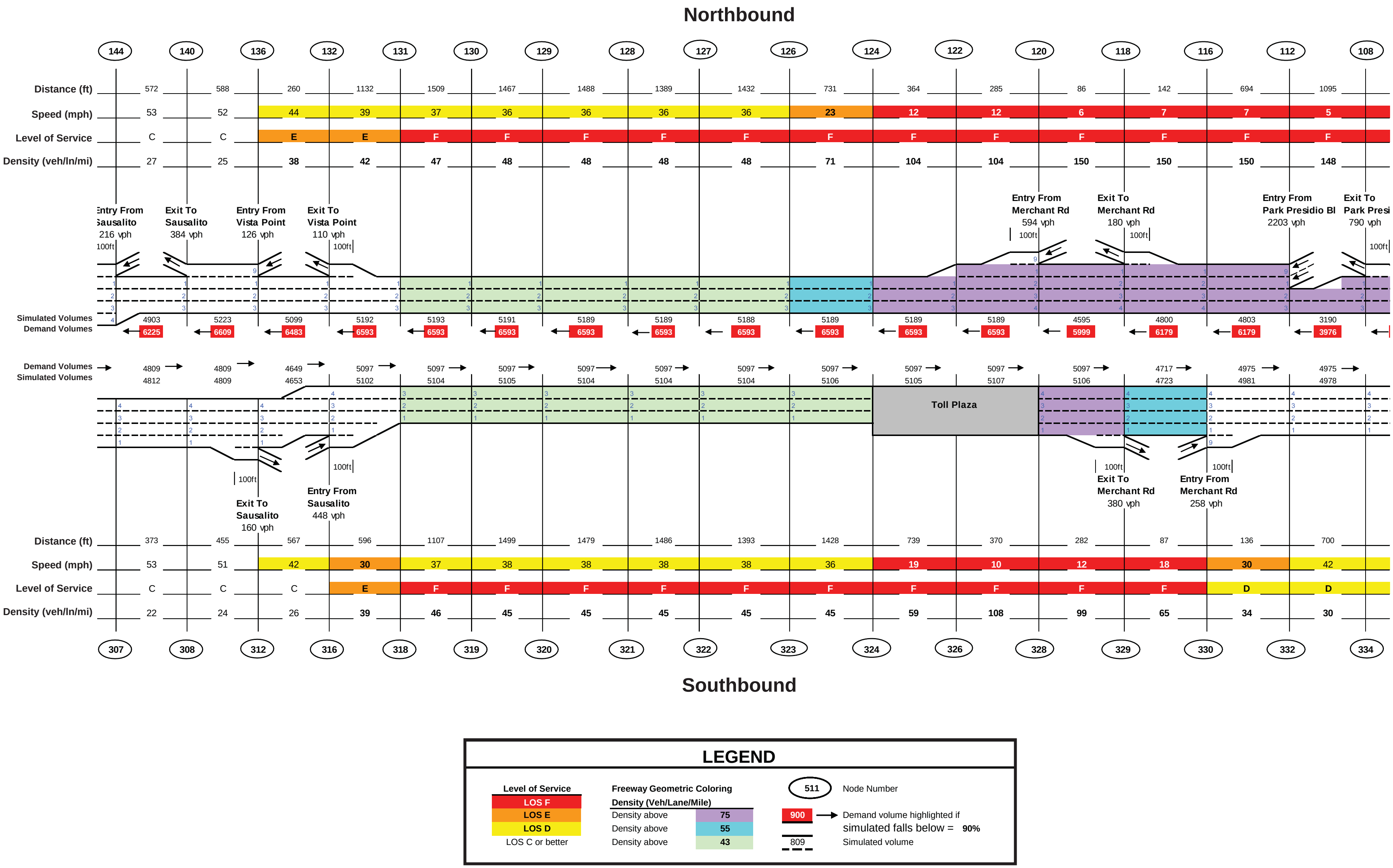
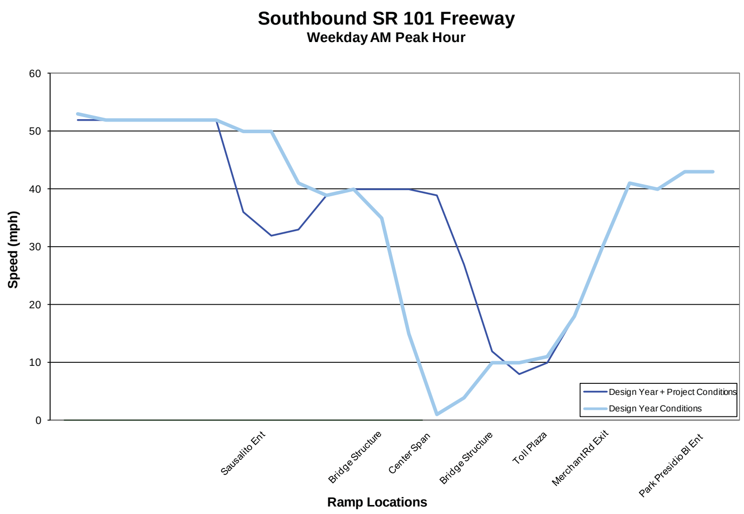
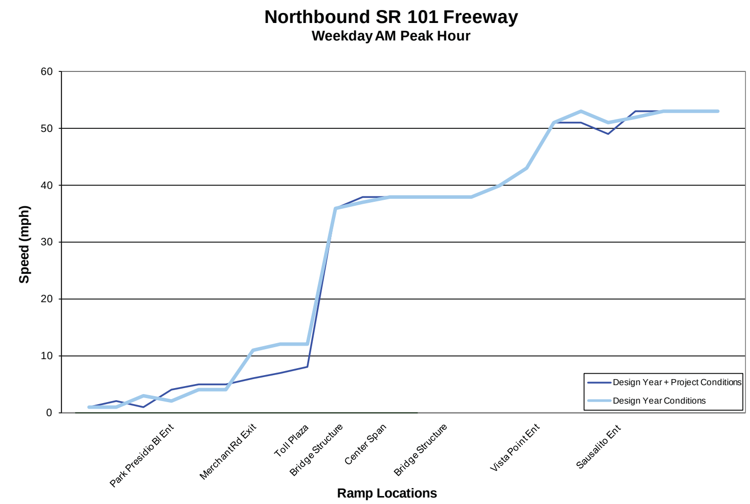
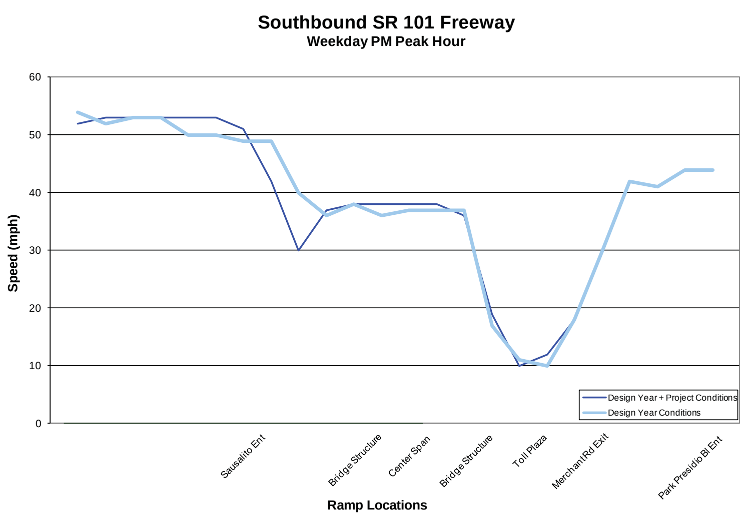
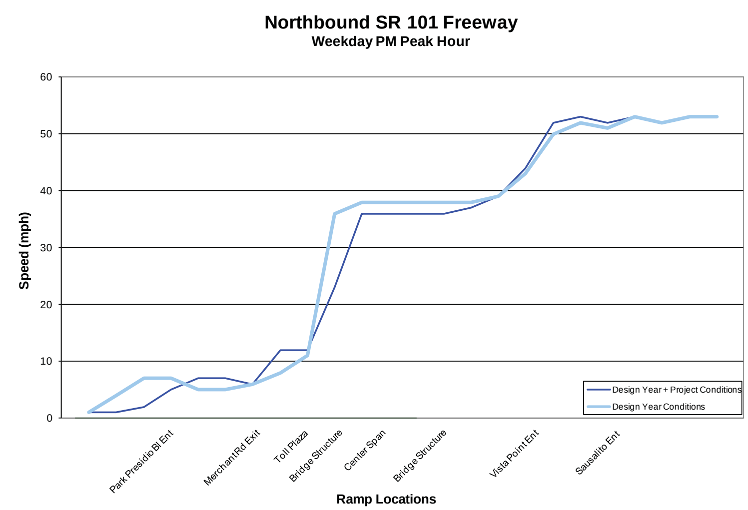


Figure 28: Design Year Plus Project Conditions Freeway Operations – Travel Speeds









Design Year + Project Travel Speeds rev3.indd

Table 24: Freeway Measures of Effectiveness – Design Year Plus Project Conditions

Freeway Section	Peak Hour	Design Year Conditions			Design Year + Project Conditions		
		Density	LOS	Speed	Density	LOS	Speed
Southbound GGB and US 101 Freeway							
Diverge at Sausalito Lateral	AM	31	D	50	42	E	36
	PM	31	D	49	24	C	51
Merge at Sausalito Lateral	AM	38	E	41	40	E	33
	PM	41	E	40	39	E	30
Diverge at Merchant Rd.	AM	131	F	11	120	F	10
	PM	104	F	10	99	F	12
Merge at Merchant Rd.	AM	43	F	30	36	E	30
	PM	36	E	30	34	D	30
Weave between Toll Plaza and Park Presidio Bl.	AM	38	E	40	37	E	41
	PM	32	D	14	30	D	42
Northbound GGB and US 101 Freeway							
Weave between Park Presidio Bl. and Toll Plaza	AM	150	F	4	150	F	5
	PM	150	F	5	150	F	7
Diverge at Merchant Rd.	AM	150	F	11	150	F	5
	PM	150	F	8	150	F	7
Merge at Merchant Rd.	AM	107	F	12	109	F	8
	PM	115	F	11	104	F	12
Diverge at Vista Point	AM	35	E	30	41	E	40
	PM	40	E	39	42	E	39
Weave between Vista Point and Sausalito Lateral	AM	13	B	51	22	C	51
	PM	21	C	50	25	C	52
Merge at Sausalito Lateral	AM	15	B	51	19	B	49
	PM	24	C	51	25	C	52

Source: AECOM – February 2013
Notes:
- Density in terms of passenger cars / lane / mile.
- Speed in terms of miles per hour.

The simulated traffic volumes on the mainline freeway were compared to vehicle demand volumes. The traffic volumes at each of the freeway ramp junctions for the weekday AM and PM peak hours were extracted from the CORSIM model and are included in **Table 25**.

Table 25: Freeway Traffic Volumes – Design Year Plus Project Conditions

Freeway Section	Peak Hour	Design Year Conditions		Design Year + Project Conditions	
		Demand	Simulated	Demand	Simulated
Southbound GGB and US 101 Freeway					
Diverge at Sausalito Lateral	AM	6,449	6,449	6,449	6,449
	PM	4,809	4,811	4,809	4,809
Merge at Sausalito Lateral	AM	6,567	6,579	6,567	6,576
	PM	5,097	5,105	5,097	5,102
Diverge at Merchant Rd.	AM	6,567	6,316	6,567	6,613
	PM	5,097	5,120	5,097	5,106
Merge at Merchant Rd.	AM	6,319	6,096	6,319	6,366
	PM	4,975	5,005	4,975	4,891
Weave between Toll Plaza and Park Presidio Bl.	AM	6,319	6,094	6,319	6,367
	PM	4,975	5,004	4,975	4,978
Northbound GGB and US 101 Freeway					
Weave between Park Presidio Bl. and Toll Plaza	AM	4,668	2,979	4,668	3,181
	PM	6,179	4,769	6,179	4,803
Diverge at Merchant Rd.	AM	4,668	2,979	4,668	3,180
	PM	6,179	4,770	6,179	4,800
Merge at Merchant Rd.	AM	4,914	3,226	4,914	3,418
	PM	6,593	5,173	6,593	5,189
Diverge at Vista Point	AM	4,914	3,217	4,914	3,415
	PM	6,593	5,176	6,593	5,192
Weave between Vista Point and Sausalito Lateral	AM	4,925	3,246	4,925	3,429
	PM	6,609	5,209	6,609	5,223
Merge at Sausalito Lateral	AM	4,761	3,196	4,761	3,674
	PM	6,441	5,103	6,441	5,118

Source: AECOM – February 2013

Notes:

- Volume in terms of vehicles per hour.
- Demand represents the peak hour mainline traffic volume demand.
- Simulated represents the peak hour mainline traffic demand volume that is served in the freeway simulation.

During the weekday PM peak hour of the Design Year plus Project Conditions, the traffic demand would be constrained by the GGB capacity in the northbound direction. With the implementation of the Proposed Project, the roadway capacity in the three lane direction would be expected to increase from 5,170 vph to 5,190 vph.

The modifications to the roadway characteristics and the reduction in shy distance would be expected to slightly increase the overall capacity of the roadway in all three of the roadway configurations. Detailed lane capacity calculations are included in **Appendix C**.

During the weekday AM and PM peak hours of the Design Year plus Project Conditions, the roadway capacity could adequately serve the traffic demand in the southbound direction.

During the weekday AM peak hour of the Design Year plus Project Conditions, the traffic demand would be constrained by the GGB capacity in the northbound direction. With the implementation of the Proposed Project, the roadway capacity in the two lane direction would be expected to increase from 3,270 vph to 3,280 vph.

9.0 Lane Configuration Schedule

This section addresses the effects of the Proposed Project on the lane configuration schedule. A revised lane configuration schedule was developed to accommodate the implementation of the MMB system.

9.1 Lane Configuration Modifications

The implementation of the MMB on the Golden Gate Bridge would likely have several effects on the lane configuration schedule, including the following:

- The BTM would transfer the MMB in approximately 20 minutes whereas the current operation requires approximately 30 minutes to complete. This would reduce the duration in which one lane on the GGB is effectively closed.
- Typically a buffer lane is added between the northbound and southbound directions during the night and when the weather conditions are poor. With the installation of the MMB, a buffer lane on the GGB would no longer be necessary or possible.
- The implementation of the MMB and the modifications to the lane widths would increase the roadway capacity by 10 vph, 20 vph, and 60 vph in the two lane, three lane, and four lane directions, respectively. The increased capacity could potentially reduce the need to move the barrier as frequently as both directions of travel could accommodate more vehicles.

9.2 Lane Configuration Recommendations

Based on the current lane configuration operations on the GGB, the roadway geometry, the BTM operations, and the traffic characteristics, the following considerations were addressed in developing a lane configuration schedule:

- The BTM requires approximately 20 minutes to transition the MMB one lane. Any modifications to the lane configuration should commence at least 20 minutes prior to the point at which a modification to the lane configuration is warranted.
- A modification to the lane configuration is warranted once traffic volumes exceed 90 percent of the roadway capacity. Roadways typically approach LOS E or worse once they exceed a volume-to-capacity ratio of 90 percent.
- The lane configuration should not be modified if the overall throughput of the roadway would be reduced. The effective closure of one lane for 20 minutes could reduce the capacity of the GGB by approximately 600 vehicles. Regardless of the excess demand, if both directions of the roadway are near capacity, the lane configuration should not be modified. Excess demand should be accounted for in the lane configuration schedule.
- During time periods when traffic volumes are less than 90 percent of capacity in both directions, the GGB should be in a three northbound lanes / three southbound lanes configuration.

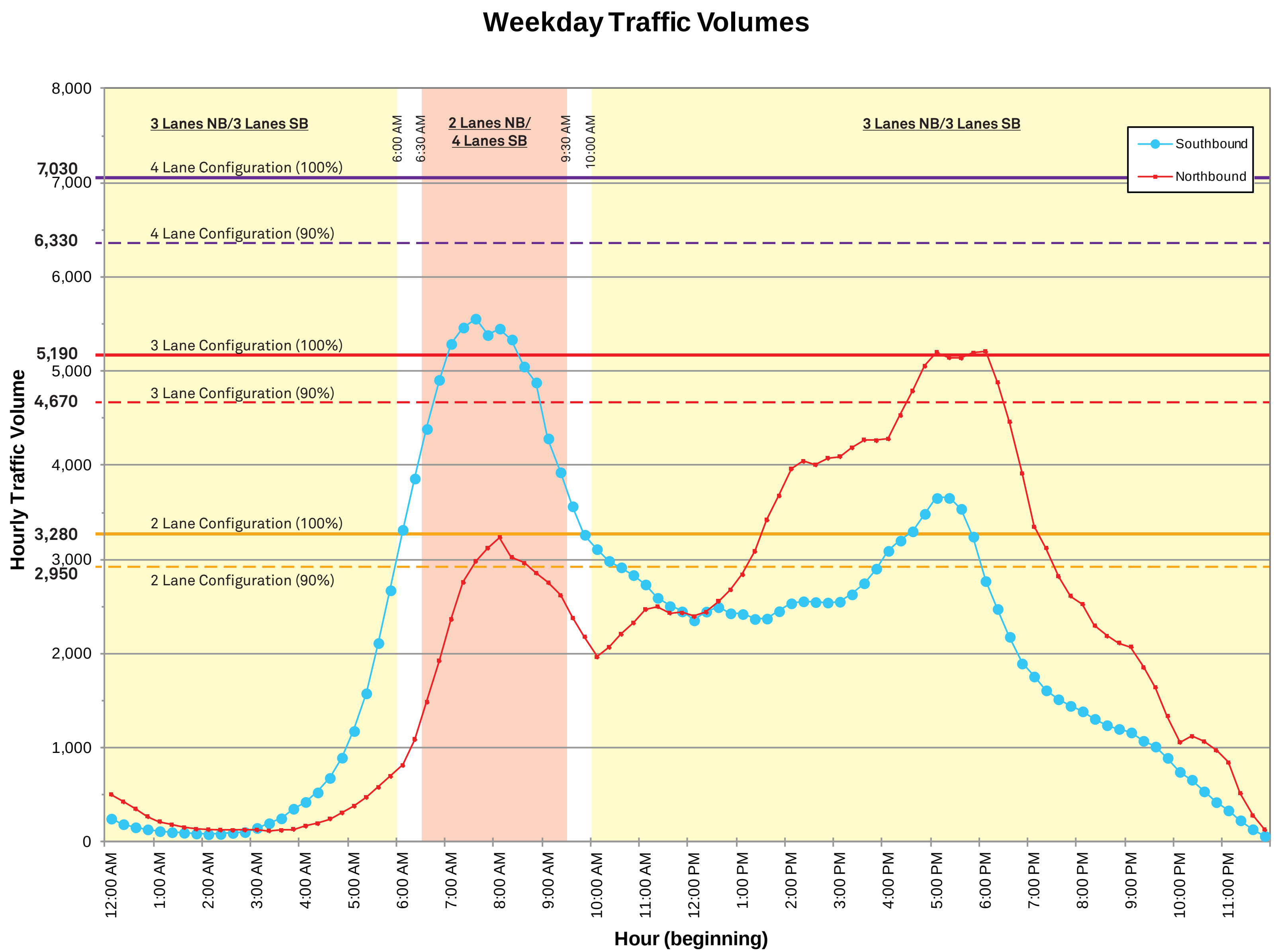
The hourly traffic volumes on the Golden Gate GGB and the corresponding lane capacities are shown in **Figure 29**.

9.3 Lane Configuration Schedule

A lane configuration schedule was developed based on the hourly traffic volumes and the lane configuration recommendations. The proposed roadway configuration volumes are shown in **Figure 30**.

As discussed previously, lane configurations may need to be changed to accommodate extenuating circumstances such as accidents or large special events south of the GGB. This schedule is intended to serve as a recommendation that is subject to modification as necessary.

Figure 29a: Proposed Roadway Configuration Traffic Volumes - Weekday



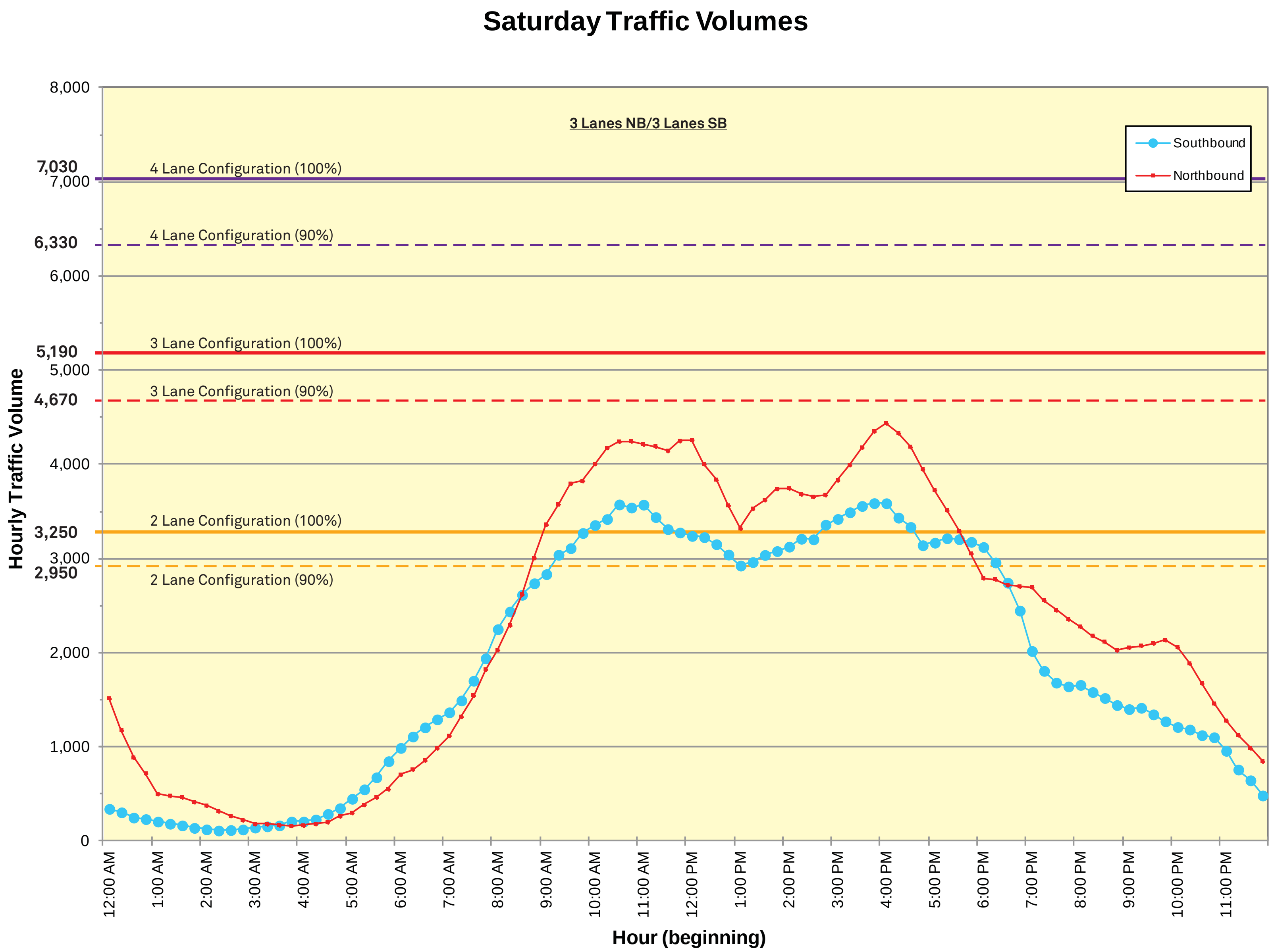

Saturday Traffic Volumes
3 Lanes NB/3 Lanes SB
Southbound
Northbound
4 Lane Configuration (100%)
4 Lane Configuration (90%)
3 Lane Configuration (100%)
3 Lane Configuration (90%)
2 Lane Configuration (100%)
2 Lane Configuration (90%)
7,030
6,330
5,190
4,670
3,250
2,950
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
Hourly Traffic Volume
12:00 AM
1:00 AM
2:00 AM
3:00 AM
4:00 AM
5:00 AM
6:00 AM
7:00 AM
8:00 AM
9:00 AM
10:00 AM
11:00 AM
12:00 PM
1:00 PM
2:00 PM
3:00 PM
4:00 PM
5:00 PM
6:00 PM
7:00 PM
8:00 PM
9:00 PM
10:00 PM
11:00 PM
Hour (beginning)

Figure 29c: Proposed Roadway Configuration Traffic Volumes - Sunday

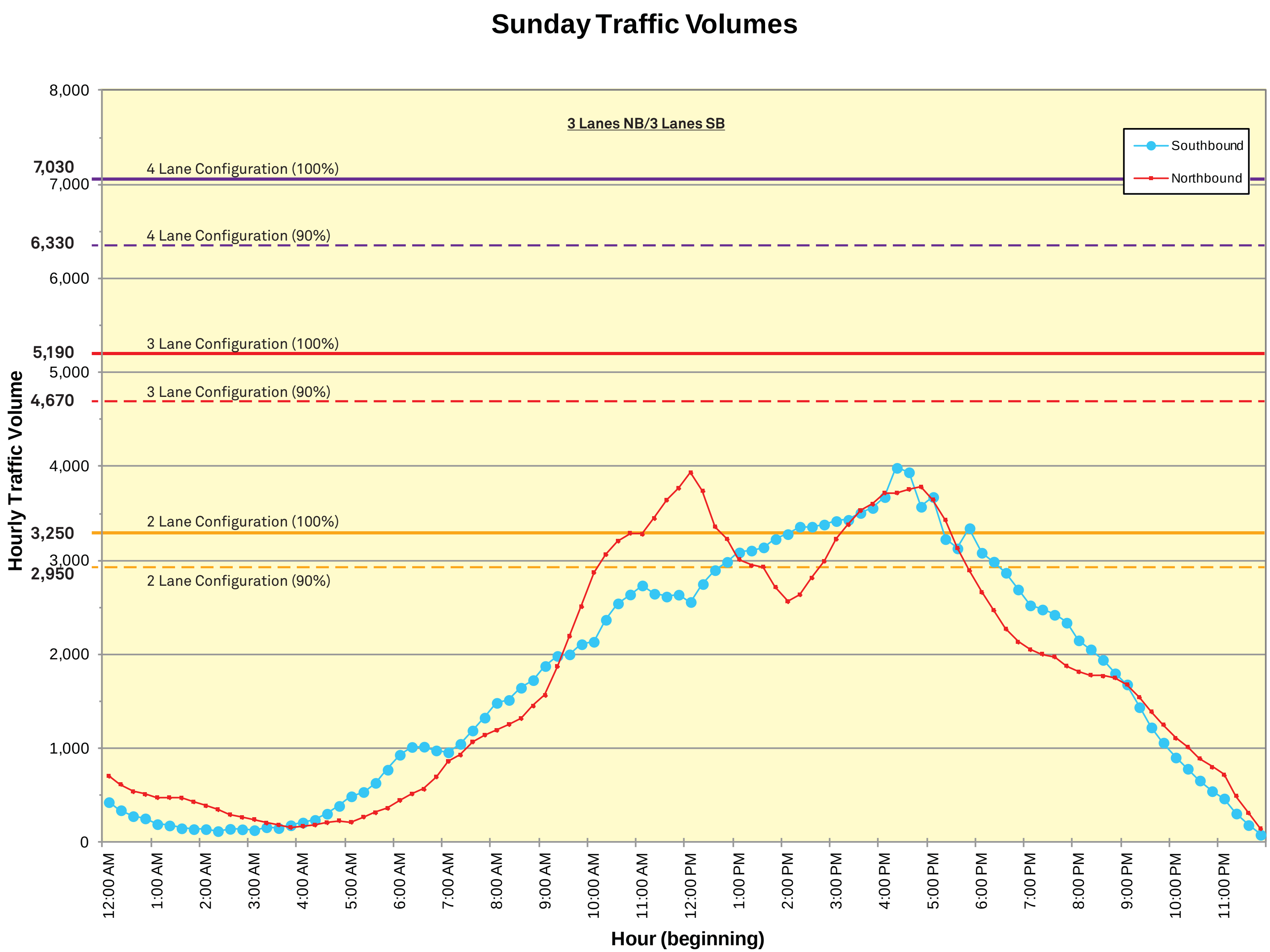
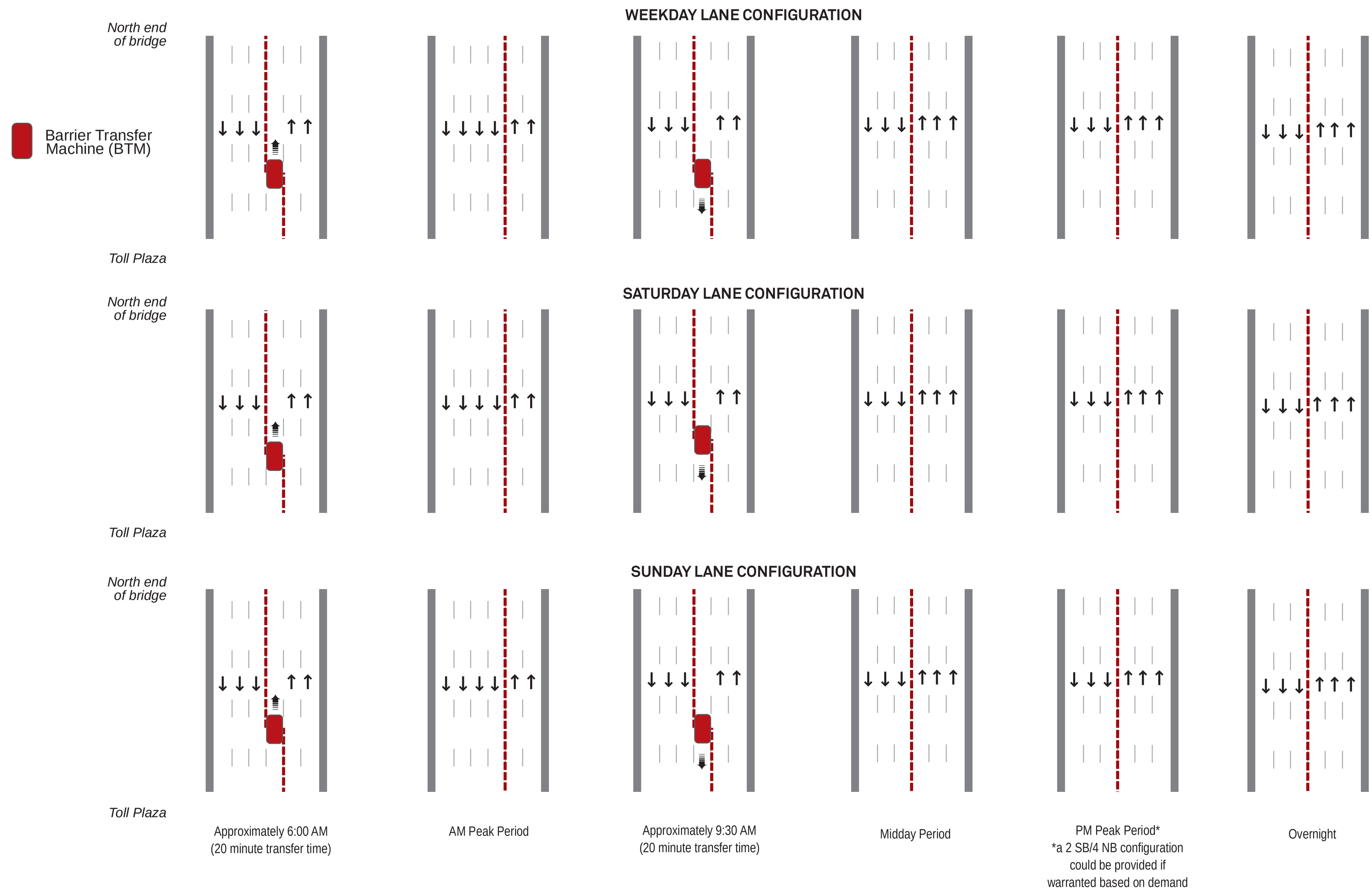


Figure 30: Proposed Roadway Configuration Schedule



A lane configuration schedule was not developed for the Design Year Conditions as hourly traffic volumes are difficult to predict. The traffic volumes on the GGB should be monitored on a regular basis and the schedule should be adjusted as necessary.

10.0Construction Conditions

Construction of the Proposed Project would occur in several stages and improvements at each end of the GGB would be required. A continuous route for bicycles would be provided during all phases and stages of construction. At the northern approach, a portion of the existing median barrier would be removed to provide enough space for the MMB to terminate safely and store the BTM when not in use. Once completed, lane striping would be applied to the roadway. Any construction that would reduce the number of lanes on the GGB would be conducted at night. Potential stage construction drawings for the Northern Approach Improvements are shown in **Figure 31**.

To accommodate the MMB system at the southern approach, the easternmost tollbooths would be demolished, reducing the Toll Plaza to six or seven functioning tollbooths. Once demolished, the new tollbooth facilities and equipment would be constructed. Lane striping would be applied to the roadway once the Toll Plaza construction was completed. Construction on the southern approach may require the Toll Plaza to be reduced to six or seven tollbooths for a minimum of one week. Additionally, the Toll Plaza may need to be reduced at night to five lanes occasionally, for less than eight hours per occurrence. Potential stage construction drawings for the Southern Approach Improvements are shown in **Figure 32**.

Based on the expected construction staging requirements, the following conditions have been analyzed:

- The Northern Construction Condition;
- The Southern Construction – Weekly Closure Condition; and
- The Southern Construction – Hourly Closure Condition.

This analysis was conducted using Existing Conditions traffic volumes and all tollbooths were assumed to have the ability to function as FasTrak-only lanes or "general" lanes, as needed.

10.1 Northern Construction Condition

Under all phases of the Northern Construction Conditions, a minimum of two lanes will be open in each direction at all times and a continuous bicycle route will be provided. This will occur during demolition of the existing median barrier and the striping of the roadway. The Northern Construction Conditions analysis is intended to determine the work windows in which there is sufficient capacity to accommodate the expected demand when the minimum number of lanes are open. The capacity of the Northern Construction Condition is based on construction zone methodology presented in the *Highway Capacity Manual Special Report 209.* [xvi] This methodology takes into account the impacts of construction work zone lane closures on roadway capacity. Based on HCM construction zone methodology the capacity of a two-lane freeway in a construction zone is 2,960 vph. The northern Construction Conditions traffic volumes are shown in **Figure 33**.

The GGB could be reduced to two lanes in each direction without interfering with traffic operations on the northern approach between the following hours:

- Weekdays from 8:00pm to 6:00am – 10 hour work window; and
- Friday from 8:00pm to Saturday at 9:00am – 13 hour work window.

Figure 31a: Stage Construction - Northern Approach Improvements

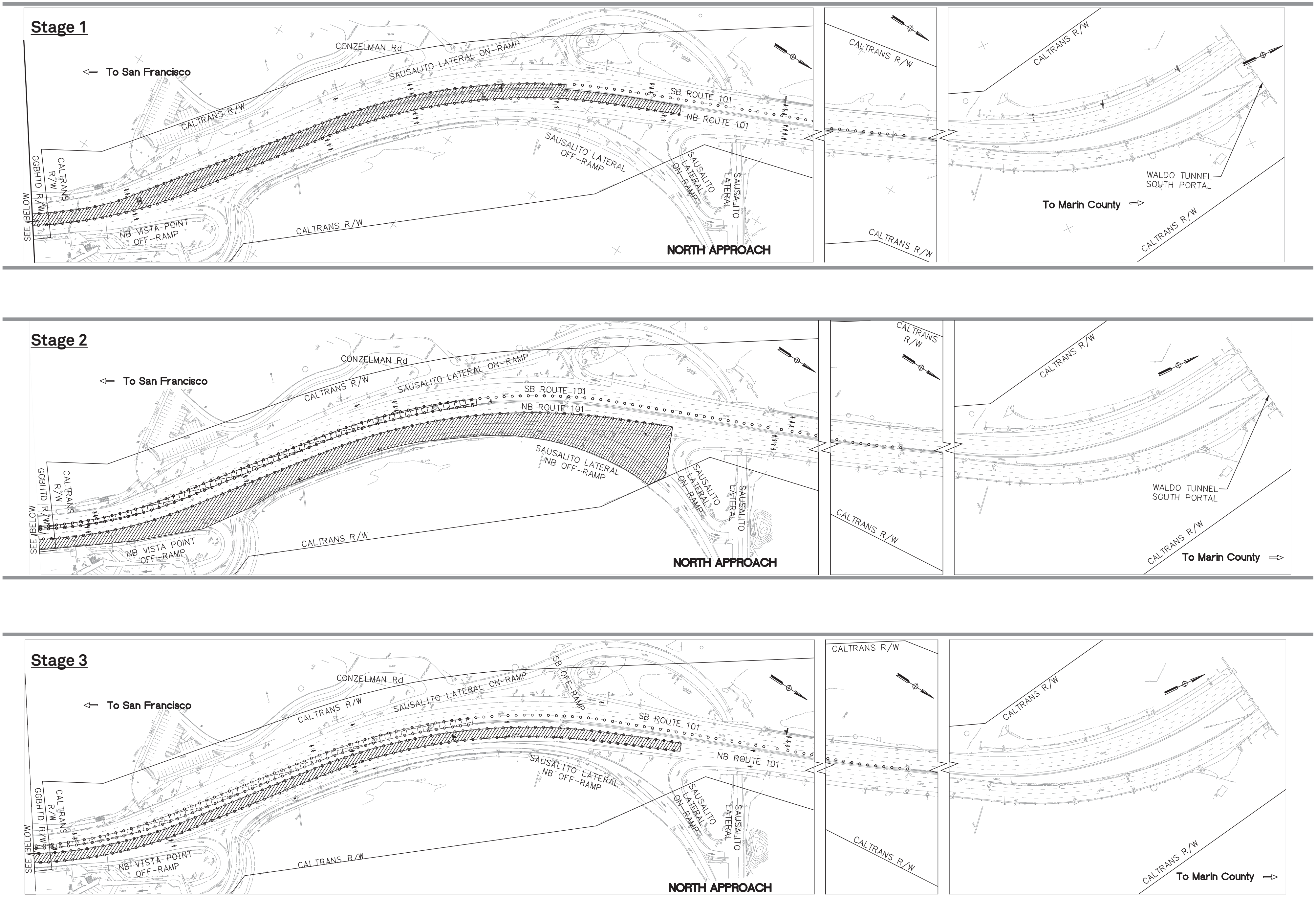
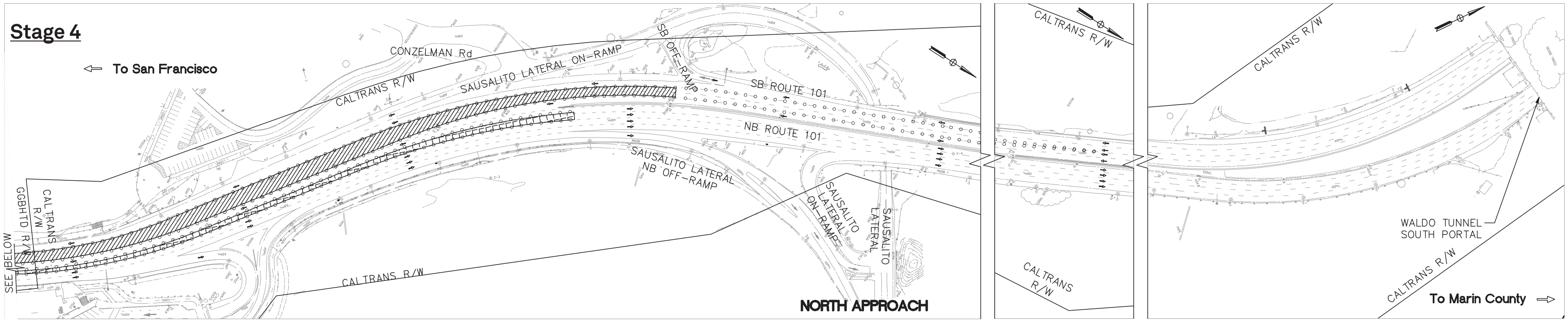


Figure 31b: Stage Construction - Northern Approach Improvements



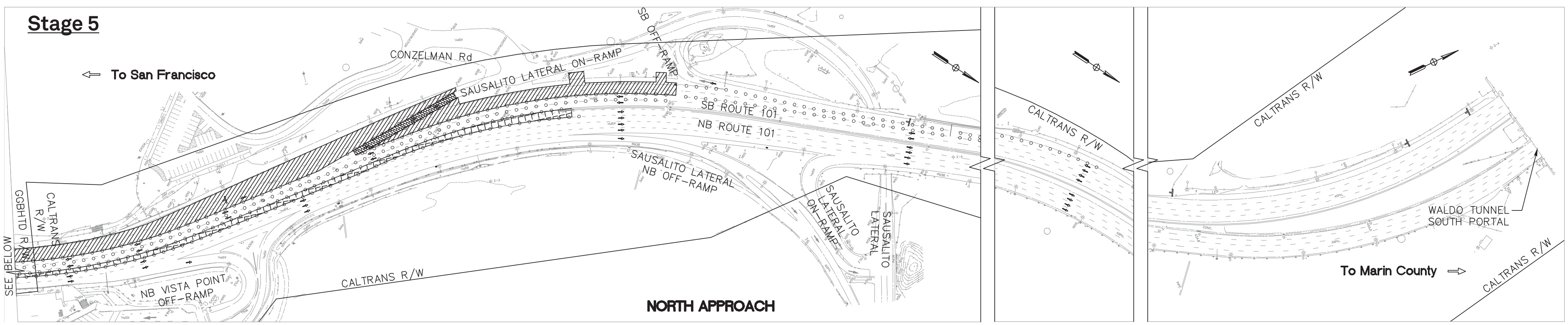


Figure 31b: Stage Construction - Northern Approach Improvements

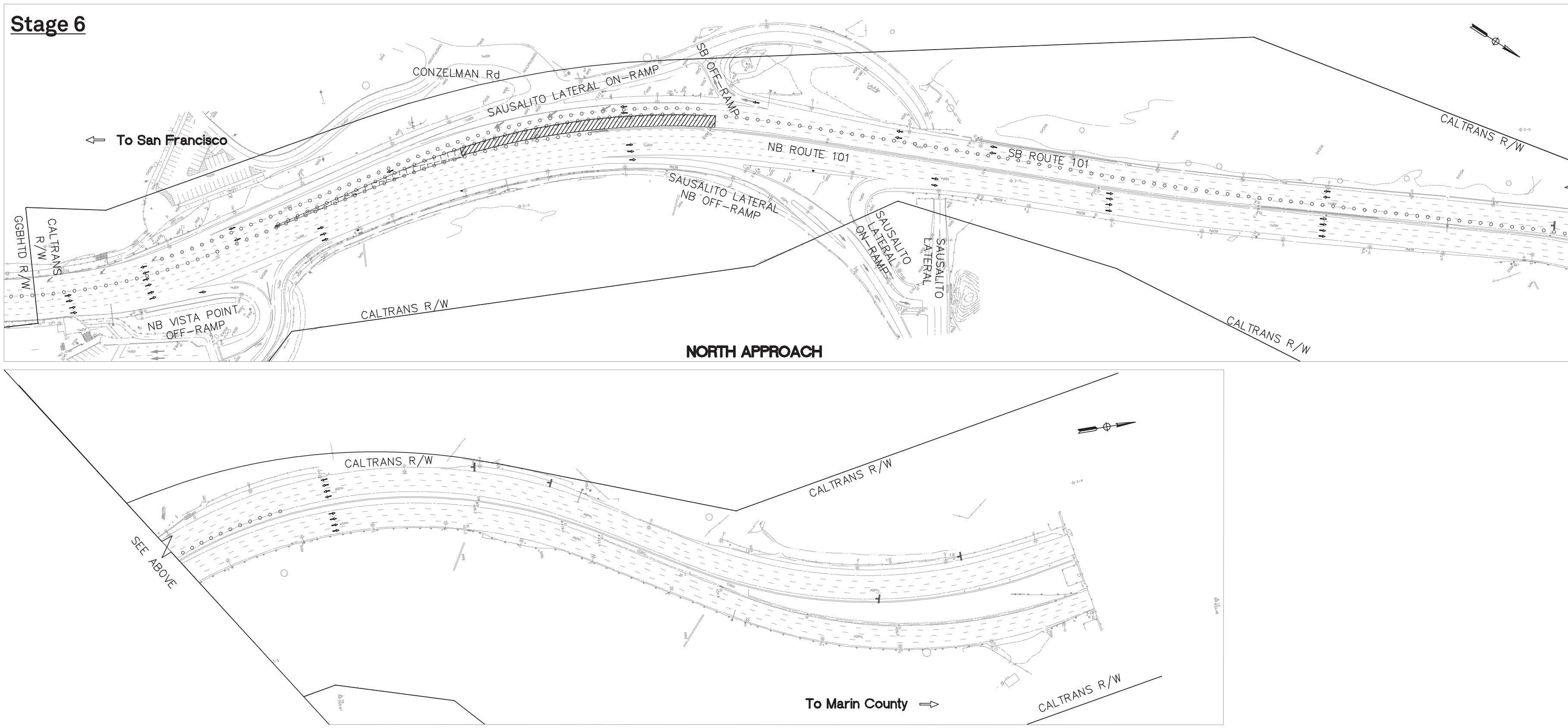
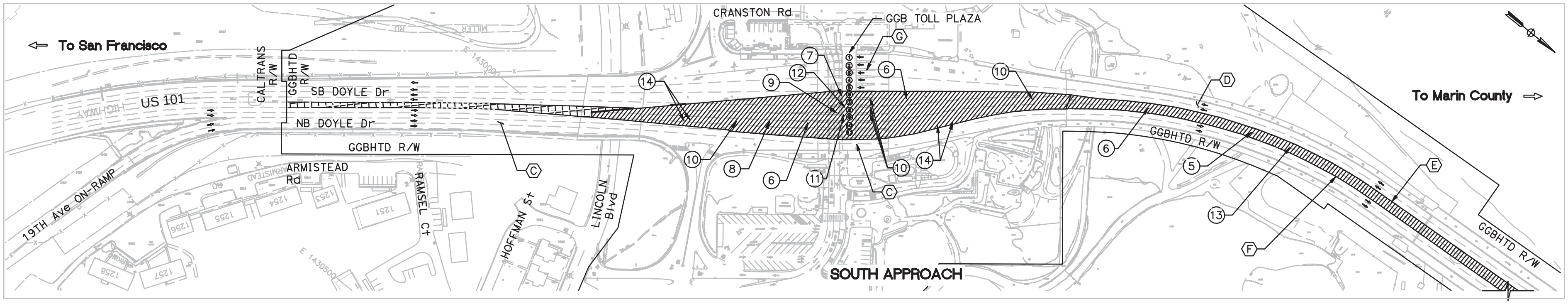
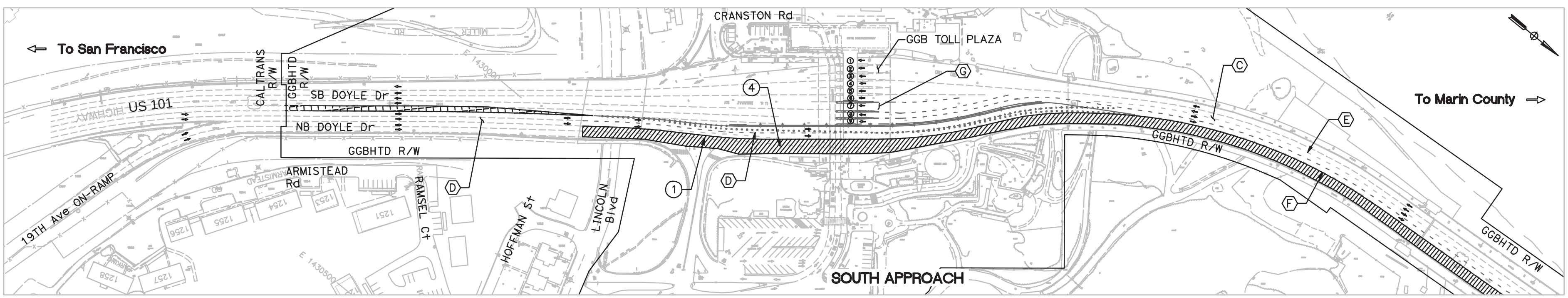


Figure 32a: Stage Construction - South End Improvements-Toll Plaza

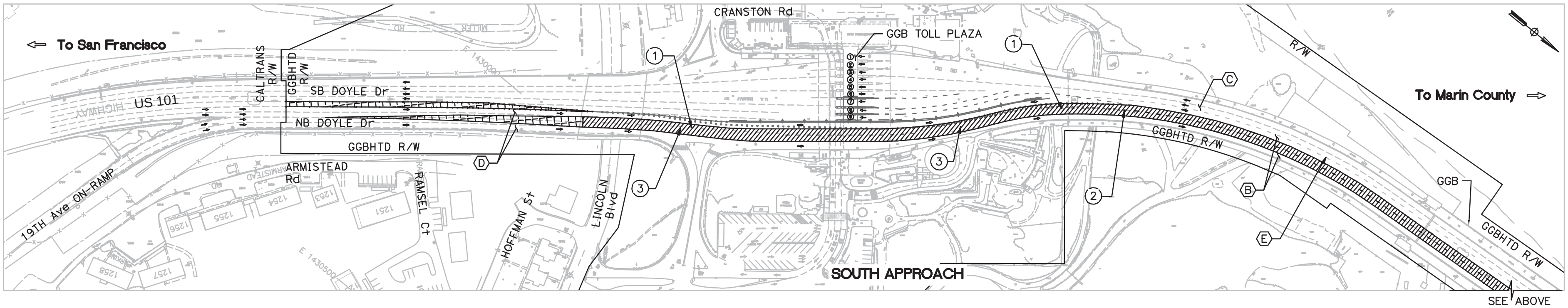


Stage 1

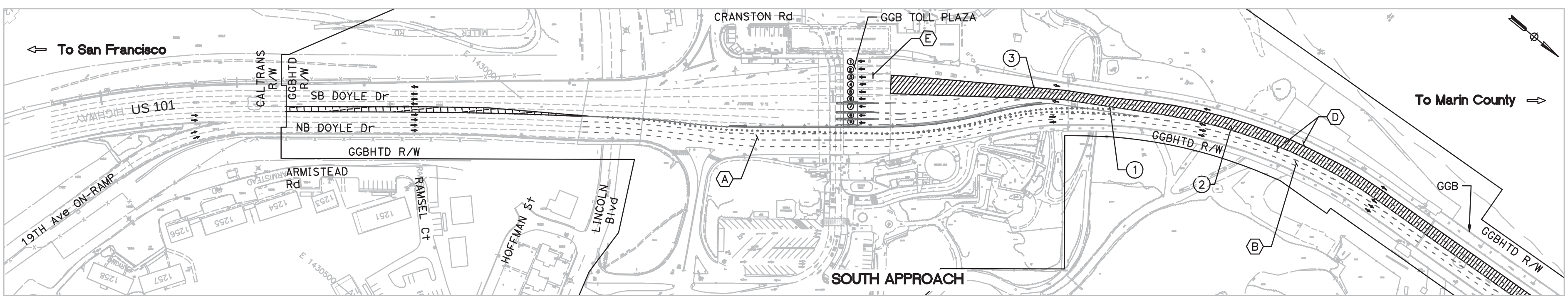


Stage 2

Figure 32b: Stage Construction - South End Improvements-Toll Plaza

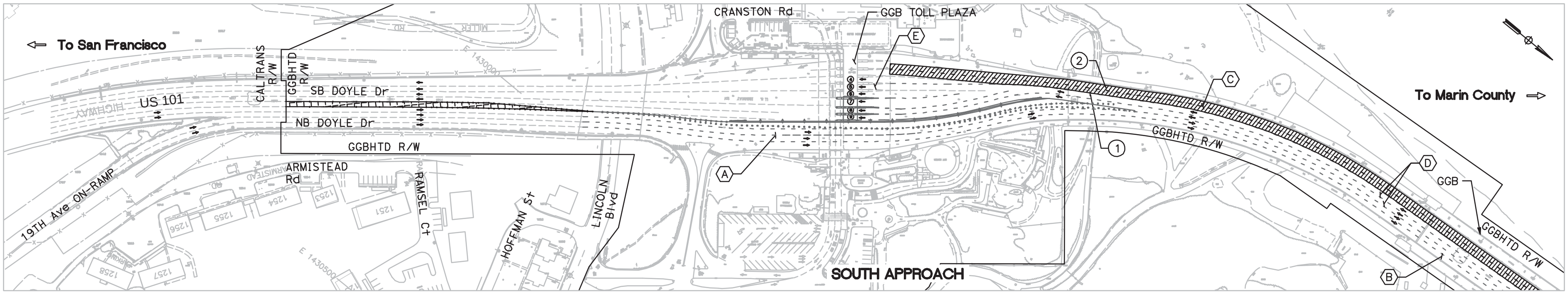


Stage 3



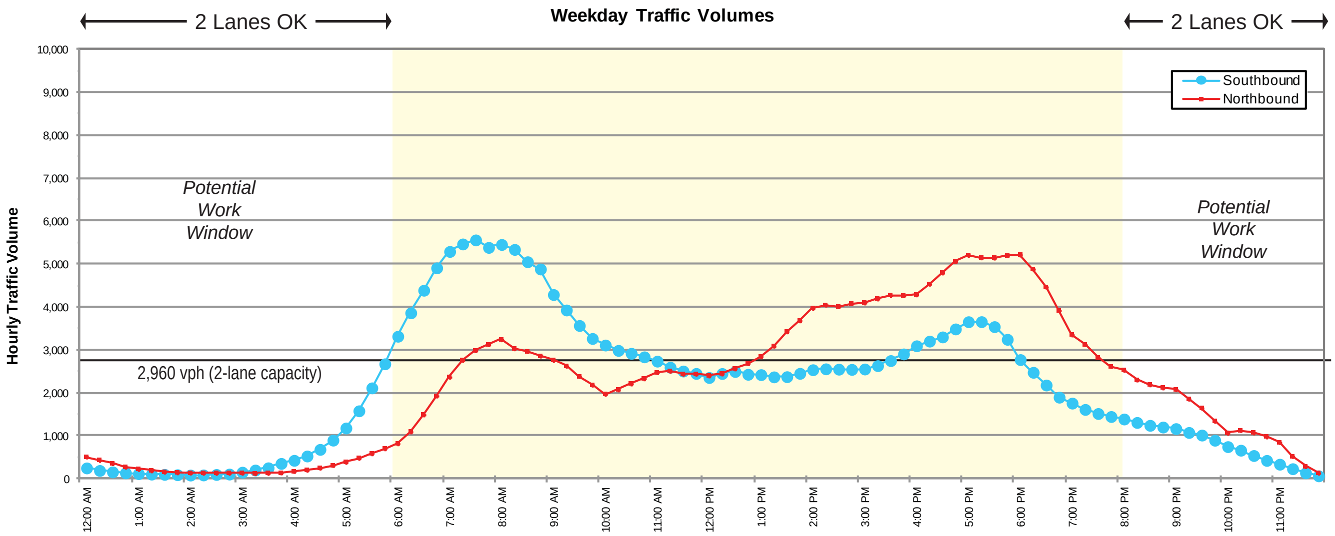
Stage 4

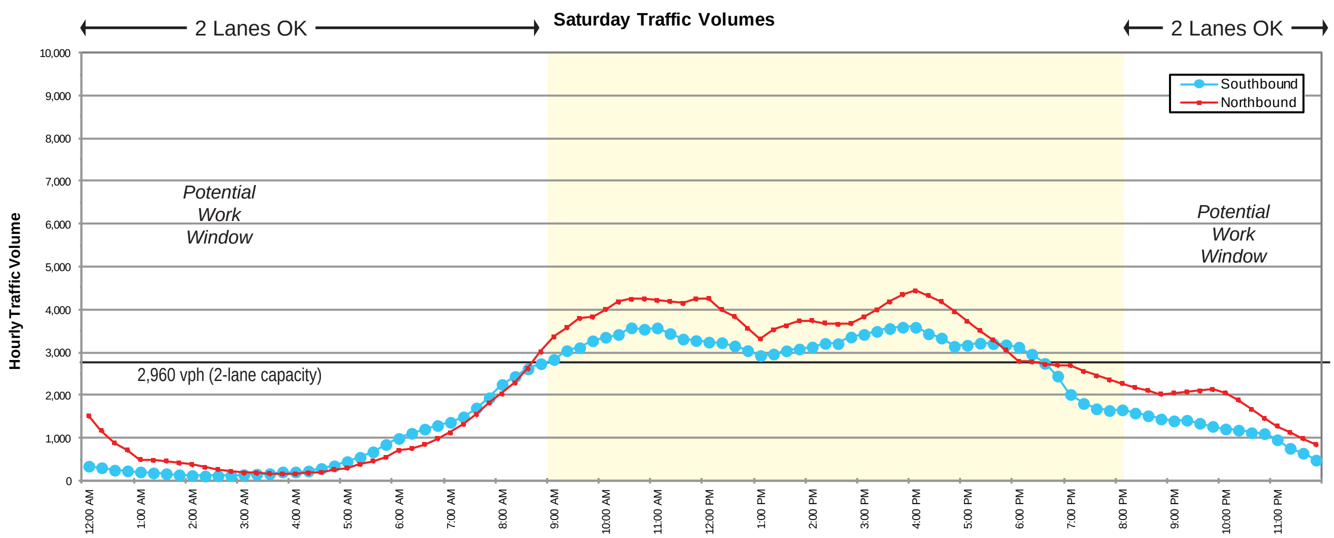
Figure 32c: Stage Construction - South End Improvements-Toll Plaza

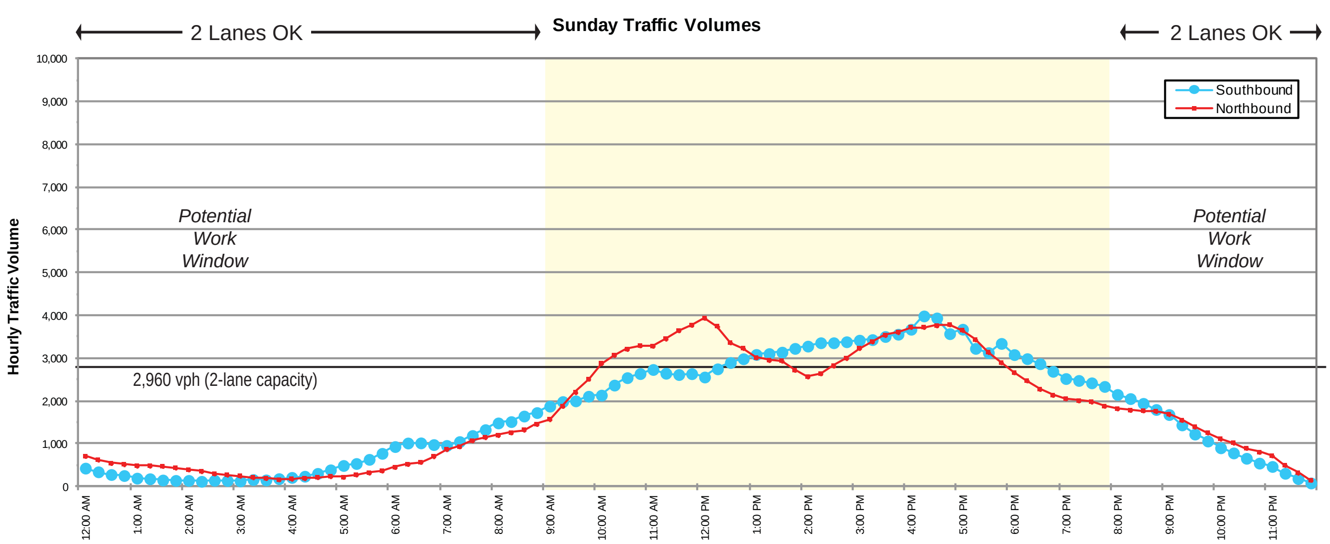


Stage 5

Figure 33: Northern Construction Condition Traffic Volumes







10.2 Southern Construction – Weekly Closure Condition

Under Southern Construction Condition – Weekly Closure Conditions, work will occur continuously for a minimum of one week. This will occur during the demolition and replacement of the tollbooths and the striping of the roadway. All work in the south approach would be within District right-of-way. The Southern Construction – Weekly Closure Condition analysis is intended to determine the effect of a six or seven tollbooth configuration on traffic demand. Analysis is conducted using the same methodology and assumes the same traffic demand and transaction shares as identified in Section 6.

To serve the traffic with a six or seven tollbooth configuration, the following three alternatives were analyzed:

- Alternative 1 – Optimize toll plaza configuration to minimize queuing and delay when demand exceeds capacity;
- Alternative 2 – Allow vehicles to pass toll-free during periods when demand would exceed capacity; and
- Alternative 3 – Conduct work on the Toll Plaza once the AET system is operational.

Alternative 1

The expected delay caused by reducing the Toll Plaza to six or seven tollbooths on weekdays is included in **Table 26**. In a six tollbooth configuration, demand would exceed capacity from 7:00am to 10:00am on weekdays, and the resulting queue would not dissipate until 11:00am. In a seven tollbooth configuration, demand would not exceed capacity.

The expected effects of reducing the Toll Plaza to six or seven tollbooths on Saturdays is included in **Table 27**. In a six tollbooth configuration, demand would exceed capacity from 10:00am to 12:00pm, and from 3:00pm to 5:00pm on Saturday, and the resulting queue would not dissipate until 1:00pm or 6:00pm, respectively. In a seven tollbooth configuration, demand would not exceed capacity.

The expected effects of reducing the Toll Plaza to six or seven tollbooths on Sundays is included in **Table 28**. In a six tollbooth configuration, demand would exceed capacity from 2:00pm to 5:00pm on Sunday, and the resulting queue would not dissipate until 6:00pm. In a seven tollbooth configuration, demand would exceed capacity from 4:00pm to 5:00pm, and the resulting queue would not dissipate until after 5:00pm.

Demand would exceed capacity every day of the week in a six tollbooth configuration, and would exceed capacity on the Sundays for approximately one hour in a seven tollbooth configuration. A minimum of eight tollbooths would be required to serve demand without significant queuing.

Table 26: Estimated Vehicle Delay – Southern Construction – Weekly Closure Condition – Weekday

Hour	6-Tollbooth Configuration					7-Tollbooth Configuration				
	Tollbooths		Unserved Demand		Delay (min)	Tollbooths		Unserved Demand		Delay (min)
	FasTrak	General	Hourly	Cml.		FasTrak	General	Hourly	Cml.	
12-1am	2	4	-	-	-	2	5	-	-	-
1-2am	2	4	-	-	-	2	5	-	-	-
2-3am	2	4	-	-	-	2	5	-	-	-
3-4am	2	4	-	-	-	2	5	-	-	-
4-5am	2	4	-	-	-	2	5	-	-	-
5-6am	2	4	-	-	-	2	5	-	-	-
6-7am	2	4	-	-	-	2	5	-	-	-
7-8am	3	3	506	506	11	3	4	126	126	3
8-9am	3	3	361	867	20	3	4	-	-	-
9-10am	2	4	202	1,069	24	2	5	-	-	-
10-11am	2	4	-	336	8	2	5	-	-	-
11-12pm	2	4	-	-	-	2	5	-	-	-
12-1pm	2	4	-	-	-	2	5	-	-	-
1-2pm	2	4	-	-	-	2	5	-	-	-
2-3pm	2	4	-	-	-	2	5	-	-	-
3-4pm	2	4	-	-	-	2	5	-	-	-
4-5pm	2	4	-	-	-	2	5	-	-	-
5-6pm	2	4	-	-	-	2	5	-	-	-
6-7pm	2	4	-	-	-	2	5	-	-	-
7-8pm	2	4	-	-	-	2	5	-	-	-
8-9pm	2	4	-	-	-	2	5	-	-	-
9-10pm	2	4	-	-	-	2	5	-	-	-
10-11pm	2	4	-	-	-	2	5	-	-	-
11-12am	2	4	-	-	-	2	5	-	-	-

Source: AECOM – February 2013

Notes:

- Cml. = Cumulative
- Demand in terms of vehicles per hour.
- Delay in terms of minutes of delay per vehicle.

Table 27: Estimated Vehicle Delay – Southern Construction – Weekly Closure Condition – Saturday

Hour	6-Tollbooth Configuration					7-Tollbooth Configuration				
	Tollbooths		Unserved Demand		Delay (min)	Tollbooths		Unserved Demand		Delay (min)
	FasTrak	General	Hourly	Cml.		FasTrak	General	Hourly	Cml.	
12-1am	1	5	-	-	-	1	6	-	-	-
1-2am	1	5	-	-	-	1	6	-	-	-
2-3am	1	5	-	-	-	1	6	-	-	-
3-4am	1	5	-	-	-	1	6	-	-	-
4-5am	1	5	-	-	-	1	6	-	-	-
5-6am	1	5	-	-	-	1	6	-	-	-
6-7am	1	5	-	-	-	1	6	-	-	-
7-8am	1	5	-	-	-	1	6	-	-	-
8-9am	1	5	-	-	-	1	6	-	-	-
9-10am	1	5	-	-	-	1	6	-	-	-
10-11am	1	5	69	69	2	1	6	-	-	-
11-12pm	1	5	83	152	4	1	6	-	-	-
12-1pm	1	5	-	88	3	1	6	-	-	-
1-2pm	1	5	-	-	-	1	6	-	-	-
2-3pm	1	5	-	-	-	1	6	-	-	-
3-4pm	1	5	118	118	3	1	6	-	-	-
4-5pm	1	5	79	197	5	1	6	-	-	-
5-6pm	1	5	-	125	3	1	6	-	-	-
6-7pm	1	5	-	-	-	1	6	-	-	-
7-8pm	1	5	-	-	-	1	6	-	-	-
8-9pm	1	5	-	-	-	1	6	-	-	-
9-10pm	1	5	-	-	-	1	6	-	-	-
10-11pm	1	5	-	-	-	1	6	-	-	-
11-12am	1	5	-	-	-	1	6	-	-	-

Source: AECOM – February 2013
Notes:
- Cml. = Cumulative
- Demand in terms of vehicles per hour.
- Delay in terms of minutes of delay per vehicle.

Table 28: Estimated Vehicle Delay – Southern Construction – Weekly Closure Condition – Sunday

Hour	6-Tollbooth Configuration					7-Tollbooth Configuration				
	Tollbooths		Unserved Demand		Delay (min)	Tollbooths		Unserved Demand		Delay (min)
	FasTrak	General	Hourly	Cml.		FasTrak	General	Hourly	Cml.	
12-1am	1	5	-	-	-	1	6	-	-	-
1-2am	1	5	-	-	-	1	6	-	-	-
2-3am	1	5	-	-	-	1	6	-	-	-
3-4am	1	5	-	-	-	1	6	-	-	-
4-5am	1	5	-	-	-	1	6	-	-	-
5-6am	1	5	-	-	-	1	6	-	-	-
6-7am	1	5	-	-	-	1	6	-	-	-
7-8am	1	5	-	-	-	1	6	-	-	-
8-9am	1	5	-	-	-	1	6	-	-	-
9-10am	1	5	-	-	-	1	6	-	-	-
10-11am	1	5	-	-	-	1	6	-	-	-
11-12pm	1	5	-	-	-	1	6	-	-	-
12-1pm	1	5	-	-	-	1	6	-	-	-
1-2pm	1	5	-	-	-	1	6	-	-	-
2-3pm	1	5	28	28	1	1	6	-	-	-
3-4pm	1	5	78	106	3	2	5	-	-	-
4-5pm	1	5	458	564	13	2	5	78	78	2
5-6pm	1	5	-	500	11	2	5	-	-	-
6-7pm	1	5	-	-	-	2	5	-	-	-
7-8pm	1	5	-	-	-	2	5	-	-	-
8-9pm	1	5	-	-	-	1	6	-	-	-
9-10pm	1	5	-	-	-	1	6	-	-	-
10-11pm	1	5	-	-	-	1	6	-	-	-
11-12am	1	5	-	-	-	1	6	-	-	-

Source: AECOM – February 2013

Notes:

- Cml. = Cumulative
- Demand in terms of vehicles per hour.
- Delay in terms of minutes of delay per vehicle.

Alternative 2
During periods where demand would exceed capacity, vehicles could be permitted to pass through the Toll Plaza toll-free in order to maintain acceptable operations at the Toll Plaza. The rate the toll-free vehicles would pass through the tollbooths is assumed to be similar to the rate FasTrak vehicles are processed. Each lane would be expected to accommodate 1,100 vph. Therefore, the capacity of the six toll-free tollbooths would be 6,600 vph, and the capacity of the seven toll-free tollbooths would be 7,700 vph.

As discussed for Alternative 1, with a six tollbooth configuration, demand would exceed capacity during the following time periods:

- Weekdays from 7:00am to 10:00am – toll-free GGB operations for 3 hours;
- Saturdays from 10:00am to 12:00pm and 3:00pm to 5:00pm – toll-free GGB operations for 4 hours; and
- Sundays from 2:00pm to 5:00pm – toll-free GGB operations for 3 hours.

As discussed for Alternative 1, with a seven tollbooth configuration, demand would exceed capacity during the following time periods:

- Sundays from 4:00pm to 5:00pm – toll-free GGB operations for 1 hour.

The expected daily and weekly lost toll revenue assuming implementation of toll-free GGB operations with a six or seven tollbooth configuration is included in **Table 29**. This analysis does not account for lost truck toll revenue.

With a six tollbooth configuration, the expected weekly lost toll revenue would be approximately $516,000. With a seven tollbooth configuration, the expected weekly lost toll revenue would be approximately $22,000. This is not a feasible alternative for the District.

Alternative 3
Alternatively, work could be conducted after implementation of the Toll Plan when the AET system is operational. Implementation of the Toll Plan would increase the rate in which tolls are collected at the Toll Plaza, thereby reducing the number of tollbooths required. If the Toll Plan is incorporated and the rate that toll collection transactions are completed is expected to be similar to the rate FasTrak transactions are completed (each lane can accommodate 1,100 vph).

Assuming installation of the Toll Plan when the AET system is operational, work could be conducted at any time when demand is less than 6,600 vph. Since demand never exceeds 6,600 vph, the Toll Plaza could be reduced to six or seven tollbooths.

Table 29: Estimated Lost Toll Revenue – Southern Construction – Weekly Construction Condition

Time Period	Description	6-Tollbooth Configuration			7-Tollbooth Configuration		
		FasTrak	Cash	Carpool	FasTrak	Cash	Carpool
Weekday	Current Toll	$5.00	$6.00	$3.00	$5.00	$6.00	$3.00
	Toll-Free Vehicles	9,533	4,423	597	-	-	-
	Lost Toll Revenue	$47,655	$26,538	$1,791	$0	$0	$0
	Total		**$75,994**			**$0**	
Saturday	Current Toll	$5.00	$6.00	-	$5.00	$6.00	-
	Toll-Free Vehicles	6,620	7,172	-	-	-	-
	Lost Toll Revenue	$33,100	$43,032	-	$0	$0	-
	Total		**$76,132**			**$0**	
Sunday	Current Toll	$5.00	$6.00	-	$5.00	$6.00	-
	Toll-Free Vehicles	5,175	5,607	-	1,914	2,073	-
	Lost Toll Revenue	$25,875	$33,642	-	$9,570	$2,073	-
	Total		**$59,517**			**$22,008**	
Weekly Total			**$515,619**			**$22,008**	

Source: AECOM – February 2013
Notes:
- Golden Gate Bridge Tolls current as of January 11, 2013.

10.3 Southern Construction – Hourly Construction Condition

The Toll Plaza will be reduced to five tollbooths during short periods of time (less than eight hours per occurrence) to supplement the work being conducted on the Toll Plaza during the Southern Construction – Weekly Construction Condition. This would be required to supplement the demolition and construction of the Toll Plaza. All work in the south approach would be within District right-of-way. This analysis is intended to determine the work windows in which there is sufficient capacity to accommodate the expected demand when only five tollbooths are available. This analysis also accounts for the dissipation of any vehicle queuing and delay resulting from the Southern Construction – Weekly Construction Condition.

Alternative 1

Assuming Alternative 1 for the Southern Construction – Weekly Construction Condition, the work window is dependent on the dissipation of the queue. Therefore, with a six tollbooth configuration, the five tollbooth work windows would be defined as follows:

- Work can be conducted on weekdays from 6:00pm to 5:00am – 11 hour work window;
- Work can be conducted on Friday from 6:00pm to Saturday at 8:00am – 14 hour work window;
- Work can be conducted on Saturday from 5:00pm to Sunday at 10:00am – 17 hour work window; and
- Work can be conducted on Sunday from 5:00pm to Monday at 5:00am – 12 hour work window.

With a seven tollbooth configuration, the five tollbooth work windows would be defined as follows:

- Work can be conducted on weekdays from 6:00pm to 5:00am – 11 hour work window;
- Work can be conducted on Friday from 6:00pm to Saturday at 8:00am – 14 hour work window;
- Work can be conducted on Saturday from 5:00pm to Sunday at 10:00am – 17 hour work window; and
- Work can be conducted on Sunday from 5:00pm to Monday at 5:00am – 12 hour work window.

Alternative 2
Assuming Alternative 2 for the Southern Construction – Weekly Construction Condition, no queues would be expected at the Toll Plaza, therefore the work windows would be similar for a six or seven tollbooth configuration and would be dependent on the capacity of a five tollbooth configuration. The work windows would be defined as follows:

- Work can be conducted on weekdays from 6:00pm to 5:00am – 11 hour work window;
- Work can be conducted on Friday from 6:00pm to Saturday at 8:00am – 14 hour work window;
- Work can be conducted on Saturday from 5:00pm to Sunday at 10:00am – 17 hour work window; and
- Work can be conducted on Sunday from 5:00pm to Monday at 5:00am – 12 hour work window.

This is not a feasible alternative for the District.

Alternative 3
Assuming implementation of the Toll Plan and installation of the AET system, work could be conducted at any time when demand is less than 5,500 vph. Since demand never exceeds 5,500 vph, the Toll Plaza could be reduced to five lanes.

11.0 Conclusions

The Proposed Project would reduce the overall usable width of the Golden Gate Bridge from 62'-0" to 61'-0" and modify the configuration of the Toll Plaza. These modifications would affect the roadway capacity, Toll Plaza capacity, traffic operations, and the lane configuration schedule. These elements were evaluated based on the Existing Conditions and Design Year Conditions.

11.1 Roadway Capacity

Two lane width alternatives were proposed in this analysis: the unequal lane width alternative and the distributed lane width alternative. Both alternatives would result in an overall 12 inch reduction in roadway width on the GGB; however, the individual lane widths would differ. From a capacity and operations perspective, both alternatives would be expected to perform similarly and any differences would be negligible.

11.2 Toll Plaza Capacity

In the Existing Conditions, the capacity of the Toll Plaza would be constrained by the lane configuration of the GGB with the implementation of the Proposed Project. The barrier transfer machines would be stored near the Toll Plaza and restrict vehicles from entering up to two of the tollbooths depending on the lane configuration. With implementation of the Toll Plan in the Existing plus Project Conditions, queuing and congestion would not be expected in an eight tollbooth configuration.

In the Design Year Conditions, the Toll Plaza is expected to consist of eight tollbooths with the Proposed Project and would not be affected by the lane configuration of the GGB. During the Weekday AM, Saturday and Sunday peak hours, the Toll Plaza would provide insufficient capacity for vehicles making cash transactions. With implementation of the Toll Plan in the Design Year plus Project Conditions, queuing and congestion would not be expected in an eight tollbooth configuration.

11.3 Traffic Operations

The Proposed Project would have minimal affects on the freeway operations in the Existing and Design Year Conditions. With the implementation of the Proposed Project, the Level of Service at all of the ramp junctions would remain essentially the same. The Proposed Project would not significantly impact the roadway operations.

11.4 Lane Configuration Schedule

A lane configuration schedule was developed based on the hourly traffic volumes and the lane configuration recommendations. The traffic volumes on the GGB should be monitored on a regular basis and the schedule should be adjusted as necessary.

The lane configurations may need to be changed to accommodate extenuating circumstances such as accidents or large special events such as baseball games at AT&T Park. The schedule is intended to serve as a recommendation that is subject to modification as necessary.

11.5 Construction Conditions

Based on the preliminary stage construction drawings, analyses were conducted for construction activities on the northern and southern approaches.

Construction on the northern approach would likely require that the GGB be reduced to two lanes in each direction. A continuous bicycle route will be provided during all stages and phases of construction on the northern approach. To maintain enough capacity to serve demand, the northern approach construction could be conducted within a 9 hour work window on weeknights (from 8:00pm to 5:00am).

Construction on the southern approach would likely require that the GGB be reduced to six or seven lanes at the Toll Plaza for more than a one week period. A continuous bicycle route will be provided during all stages and phases of construction on the southern approach. All work in the south approach would be within District right-of-way. Reducing the Toll Plaza to six or seven lanes would result in significant vehicle congestion and queues. To reduce delays at the Toll Plaza during construction, one of the following alternatives could be implemented:

- Implement toll-free GGB operations during periods of congestion. The estimated weekly lost toll revenue with the six or seven tollbooth configurations is estimated to be $980,000 or $325,000, respectively. This is not a feasible alternative for the District.
- Commence construction once the Toll Plan is implemented. All electronic tolling is expected to be activated in March 2013.

Construction on the southern approach would likely require that the GGB be reduced to five lanes at the Toll Plaza for short periods of time (less than eight hours per occurrence). To maintain enough capacity to serve demand, the southern approach construction could be conducted within a minimum of an 11 hour work window on weeknights (from 6:00pm to 5:00am).

End Notes

(i) Traffic Analysis Toolbox Volume IV: Guidelines for Applying CORSIM Microsimulation Modeling Software. Section 5.0 – Calibration. U.S. Department of Transportation. Federal Highway Administration, 400 Seventh Street S.W. Room 4410 Washington, DC 20590, Publication No. FHWA-HOP-07-079, January 2007.

(ii) Doyle Drive South Access to the Golden Gate Bridge Addendum to the Final Traffic and Transit Operations Report. October 2006. Prepared for the San Francisco County Transportation Authority by DKS Associates.

(iii) Presidio Parkway. Phases During Construction. http://www.presidioparkway.org/traffic_info/. Page accessed on April 15, 2010.

(iv) Development of Strategic Plan for All Electronic Toll Collection on Golden Gate Bridge. http://goldengate.org/tolls/index.php. Golden Gate Bridge, Highway and Transportation District. Accessed September 19, 2012.

(v) Golden Gate Bridge Current Projects: All Electronic Tolling. http://goldengate.org/tolls/index.php. Golden Gate Bridge, Highway and Transportation District. Accessed January 3, 2013.

(vi) Traffic Analysis Toolbox Volume IV: Guidelines for Applying CORSIM Microsimulation Modeling Software. Section 5.0 – Calibration. U.S. Department of Transportation. Federal Highway Administration, 400 Seventh Street S.W. Room 4410 Washington, DC 20590, Publication No. FHWA-HOP-07-079, January 2007.

(vii) Advanced CORSIM Training Manual. Minnesota Department of Transportation. SEH No. A-MNDOT0318.00. January 31, 2008.

(viii) Freeway Performance Measurement System (PeMS). The Department of Electrical Engineering and Computer Sciences at the University of California, Berkeley, California Department of Transportation (Caltrans), California Partners for Advanced Transit and Highways (PATH), and Berkeley Transportation Systems (BTS).

(ix) The ramp traffic volumes data were distributed via email on February 23, 2012 and September 24, 2012 by Evelyn Gestuvo and Jordan Chan of Caltrans Highway and Operations Division.

(x) Golden Gate Bridge Moveable Median Barrier Feasibility Studies – Phase 2. January 2002. Prepared for the Golden Gate Bridge, Highway and Transportation District by Parsons Brinckerhoff.

(xi) Northbound Afternoon Commute Lane Configuration Challenge at Golden Gate Bridge Current Afternoon Configuration is 3 Lanes South and 3 Lanes North. Golden Gate Bridge, Highway and Transportation District. http://goldengate.org/news/bridge/newlaneconfig.php. Accessed April 14, 2010.

(xii) Doyle Drive South Access to the Golden Gate Bridge Final Traffic and Transit Operations Report. December 2004. Prepared for the San Francisco County Transportation Authority by DKS Associates.

(xiii) Golden Gate Bridge Moveable Median Barrier Project – Conceptual Engineering Design Studies. February 2001. Prepared for the Golden Gate Bridge, Highway and Transportation District by Parsons Brinckerhoff.

(xiv) Golden Gate Bridge Tolls and Traffic Operations – Golden Gate Bridge Toll Rates and Carpools. http://goldengatebridge.org/tolls_traffic/toll_rates.php#policy. Page accessed September 19, 2012.

(xv) Golden Gate Bridge Tolls and Traffic Operations – Golden Gate Bridge FasTrak Usage 2001 to 2011. http://goldengatebridge.org/tolls_traffic/updatesmarketsummary.php. Page accessed September 24, 2012.

(xvi) Highway Capacity Manual Special Report 209. 1994. Transportation Research Board.