Prepared for :
City of Petaluma
Public Works
202 N. McDowell Blvd.
Petaluma, CA 94954

Submitted by :
AECOM
2020 L Street, Suite 300
Sacramento, CA 95811
November 19, 2021

Draft Technical Memorandum

for the

Caulfield Lane Bridge and Extension Project

City of Petaluma, Sonoma County

AECOM
2020 L Street, Suite 400
Sacramento, CA 95811
aecom.com

Project name:
Caulfield Bridge & Extension Project
City Project No. C16401824
Project ref:
AECOM Project Number 60580130

From:
Thomas R. Barnard, PE
Senior Project Manager

Date:
November 22, 2021

To: Mr. Ken Eichstaedt, PE
City of Petaluma, Public Works & Utilities Department
202 N. McDowell Blvd
Petaluma, CA 94954

Memorandum

Subject: Caulfield Bridge and Extension Project

Executive Summary

The City of Petaluma engaged AECOM in 2018 to study and report on the feasibility of extending Caulfield Lane past its current terminus at Hopper Street and over the Petaluma River via a new bridge, providing a "southern crossing" connecting to the north leg of the Petaluma Boulevard South/Crystal Lane roundabout.

The study was commissioned as part of the second phase of the City's project development, with the intent to confirm the alignment, ensure the necessary land is available to construct the crossing, perform sufficient engineering analysis to select the structure type and update the project development cost estimate in order to inform the project' s third and final phases, which would include; Final design, Environmental documentation, Funding analysis, Construction documents and Construction.

At the time of the study the physical project limits are defined by conforming to the existing roundabout on the south and to the Riverfront development at the Caulfield Lane/Bautista Way intersection. The available right of way is anticipated to be adequate for the construction of the proposed bridge with only minor adjustments to be determined during final design to accommodate the connections to existing vehicular roadway elements and the planned riverfront improvements. The river crossing will require negotiation of a new lease agreement with the California State Lands Commission. The process is anticipated to be straight forward; however, it should be initiated at the earliest opportunity once implementation of the final phase of the project is authorized.

The traffic impact study performed for the project (see Appendix B) highlighted the positive influence the "Southern connection" will have on pedestrian and bicycle movements. The study area includes nine intersections along Washington Street, East D Street, Lakeville Street and Caulfield Lane. These intersections were chosen because the new link on Caulfield Lane would be expected to pull traffic away from Washington Street and East D Street which also cross the Petaluma River, while adding this traffic to Lakeville Street which would connect to the new crossing. These intersections would operate at the same levels of service under Future with Bridge conditions as with Future conditions, except for the intersection of Caulfield Lane/Lakeville Street, which would degrade to LOS F. Improvements have been identified that to bring operations to LOS D or better (widening of Lakeville Street would be required) but are not recommended as it would conflict with the City of Petaluma's General Plan's multimodal objectives. The study indicates a slight reduction in regional VMT resulting from the implementation of the new river crossing.

Based on the geometric constraints, imposed by the anticipated roadway profile and the navigation clearances, the scope of the study was narrowed to movable bridges of the bascule type. Further refinement resulted in the selection of a rolling leaf bascule structure type because of the open channel hydraulics and navigation clearance issues associated with below deck counterweight foundations.

Early, proactive consultation with the US Coast Guard's (USCG) Bridge Section resulted in the agency issuing a "preliminary determination" on July 15, 2019 that was consistent with the navigation clearances presented in the preliminary layout. It is important to note that the "preliminary determination" does not constitute an approval or final agency determination; however, it was an important milestone in the development of the project, setting the basic geometry for the required navigation opening (see Appendix A for proposed structure configuration and USCG communications).

The bridge design hydrology/hydraulics study (Appendix C) showed that introduction of the new single span bridge within the waterway caused an increase in the water surface elevation of the base flood (100-year return period) of approximately 0.22 feet. Because the crossing is within the limits of a designated floodway, such an increase would require a Letter of Map Revision; a process that can be lengthy, expensive and may not result in approval for the encroachment and increased water surface. The study was peer reviewed by WEST Engineering, which found it to be reasonably conservative.

An alternative design was developed that uses a two (2) span bascule configuration that eliminates the obstructions to flow and was determined to have no material impact on the controlling water surface elevation. The change resulted in an increase to the bridge cost of approximately 25% - 30%; however, that cost may be reduced during final design and could benefit from additional hydraulic analyses and coordination with the agencies with jurisdiction over the floodway. The cost of the project, including escalation of 3%, is approximately $43 Million with a construction end date of 2026 (see Appendix I – Planning Cost Estimate)

Although the scope did not include a detailed analysis of the potential for sea level rise impacts within the project area, a review of the most recent guidance from "Our Coast Our Future" (OCOF) was performed (see Appendix H). Using the OCOF impact area viewer we looked at various sea level rise projections up to 200 cm (6.56 ft), coupled with a 100-year storm water event. The preliminary results indicate the proposed project would not be inundated; however, the rise in the ordinary high-water surface will reduce the available vertical navigation clearance in the structure closed position but does not impact operation of the draw.

Working with City staff we were able to shift some project resources to enable a limited geotechnical investigation (see Appendix D) that improved our understanding of the site-specific foundation requirements and increase the level of confidence in our opinion of probable construction costs (see Appendix I – Planning Cost Estimate).

The "Environmental Clearance Strategy" (see Appendix F) developed as part of this study anticipates that a joint NEPA/CEQA document may be the most efficient. The USCG is anticipated to be the lead federal agency, while the City of Petaluma will be the CEQA lead. It is anticipated that the appropriate level of NEPA documentation would be an Environmental Assessment (EA), as it is currently expected that all potentially significant environmental impacts due to the project could be reduced to a less than significant level with the incorporation of mitigation measures allowing the federal lead agency to issue a Finding of No Significant Impact (FONSI). On the CEQA side, it is anticipated that the appropriate level of documentation would be an Initial Study (IS), as the incorporation of mitigation measures would allowing the City to adopt a Mitigated Negative Declaration (MND). Final decision regarding type of document may depend on the funding source(s).

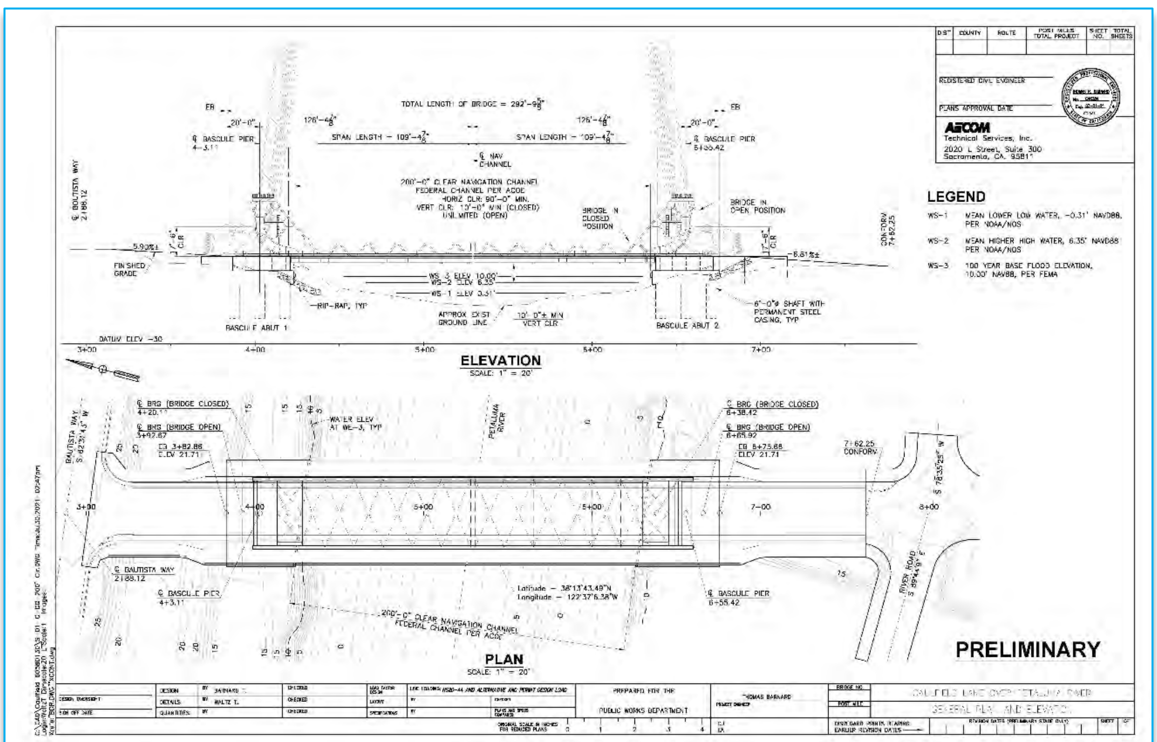
Our cultural resources database search (Appendix E) indicates there are no known historical resources, unique archaeological resources, or historic properties that have the potential to be affected/impacted by the project. The southern side of the project site is highly sensitive for buried/submerged. Depending on the depth and location of proposed subsurface project impacts, and the permitting nexus of the project (e.g., federal permitting requiring compliance with Section 106 of the National Historic Preservation Act [NHPA]), pre-construction subsurface investigations to identify potential buried resources may be warranted. No Native American consultation was undertaken as part of this preliminary investigation. Such consultation may be required by the lead agency under state (Assembly Bill 52) and/or federal (Section 106) law, prior to project implementation.

Recommended Structure Type

The recommended structure type for the Caulfield Lane Bridge and Extension project is a dual, rolling leaf bascule bridge with overhead counterweights, carrying two (2) -11' travel lanes with 6' wide bike lanes. A 6' clear pedestrian walkway would be attached to the outside of the truss on each side of the bridge; similar to that shown in Photo 1, which depicts a project in Chesapeake, Virginia with similar span arrangement.

Photo 1 - *A dual rolling leaf bascule bridge with overhead counterweight in Chesapeake Virginia of similar span length to that proposed for the Caulfield Lane Bridge over the Petaluma River.*

A "General Plan and Elevation" drawing of the recommended structure type is shown below and included in Appendix A. The overall length of the project between conform points at Bautista Way and the South Petaluma Blvd/Crystal Lane roundabout is approximately 500 feet, of which approximately 293 feet is the proposed bridge. The abutments are anticipated to be supported on cast-in-place (CIP) reinforced concrete abutments bearing on an array of large diameter cast-in-drilled-hole (CIDH) piling.



The recommended layout honors the horizontal and vertical clears cited in the USCG's preliminary determination and does not impact the FEMA base flood water surface elevation. The approach roadway components will be designed to meet City design criteria.

See the various attached appendices for detailed information.

APPENDIX A – Bridge General Plan/ USCG Preliminary Clear Determination

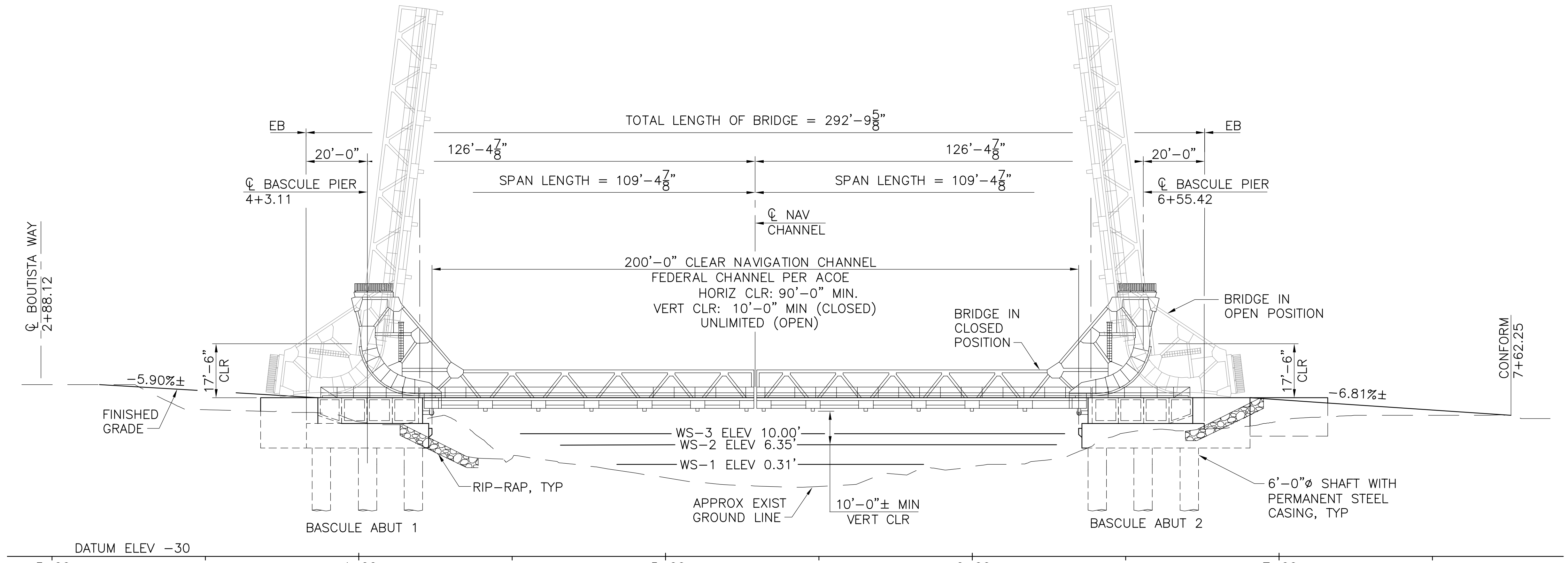
DIST	COUNTY	ROUTE	POST MILES TOTAL PROJECT	SHEET NO.	TOTAL SHEETS

REGISTERED CIVIL ENGINEER

PLANS APPROVAL DATE

REGISTERED PROFESSIONAL ENGINEER — THOMAS R. BARNARD — No. C46384 — Exp. 03-31-21 — CIVIL — STATE OF CALIFORNIA

AECOM
Technical Services, Inc.
2020 L Street, Suite 300
Sacramento, CA. 95811

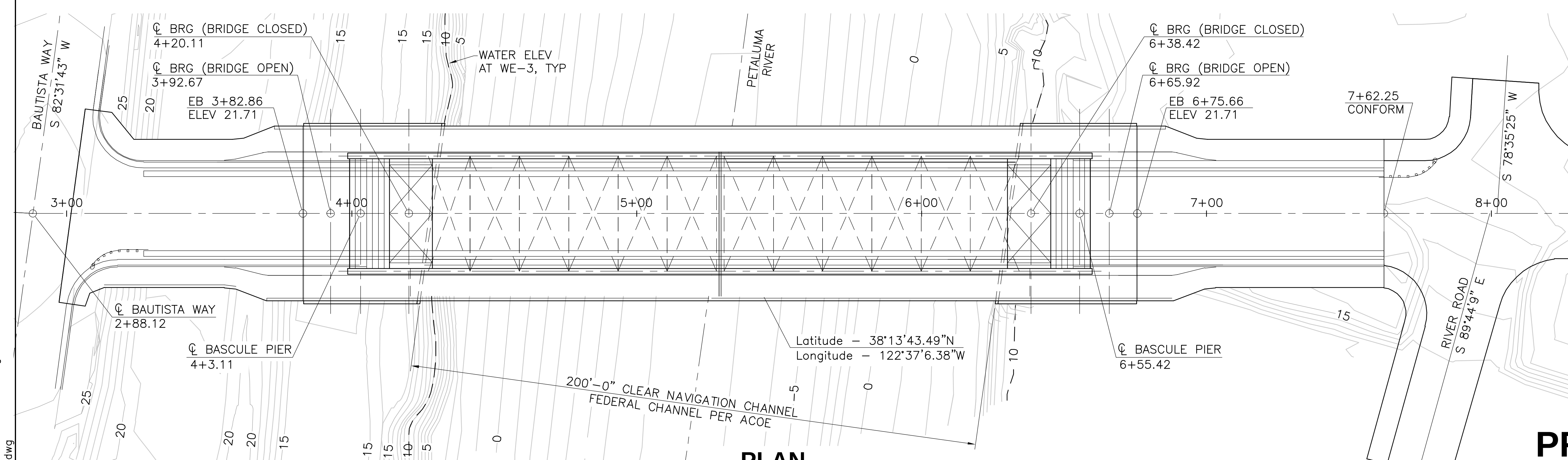


ELEVATION

SCALE: 1" = 20'

LEGEND

WS-1	MEAN LOWER LOW WATER, -0.31' NAVD88, PER NOAA/NOS
WS-2	MEAN HIGHER HIGH WATER, 6.35' NAVD88 PER NOAA/NOS
WS-3	100 YEAR BASE FLOOD ELEVATION, 10.00' NAV88, PER FEMA



PLAN

SCALE: 1" = 20'

PRELIMINARY

DESIGN OVERSIGHT	DESIGN	BY BARNARD T.	CHECKED	LOAD FACTOR DESIGN	LIVE LOADING: HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD		PREPARED FOR THE	THOMAS BARNARD — PROJECT ENGINEER
	DETAILS	BY WALTZ T.	CHECKED	LAYOUT	BY	CHECKED	PUBLIC WORKS DEPARTMENT	
SIGN OFF DATE	QUANTITIES	BY	CHECKED	SPECIFICATIONS	BY	PLANS AND SPECS COMPARED		

BRIDGE NO.

POST MILE

CAULFIELD LANE OVER PETALUMA RIVER

GENERAL PLAN AND ELEVATION

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3 4

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EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) | SHEET | OF

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U.S. Department of Homeland Security
United States Coast Guard

Commander
Eleventh Coast Guard District

Coast Guard Island, Bldg. 50-2
Alameda, CA 94501-5100
Staff Symbol: (dpw)
Phone: (510) 437-3515
Fax: (510) 437-5836
Email: Carl.T.Hausner@uscg.mil

16591
Petaluma River (12.7)
July 15, 2019

The City of Petaluma
Public Works
Attn: Mr. Jason Beatty
202 N. McDowell Blvd.
Petaluma, CA 94954

Dear Mr. Beatty:

We have reviewed the City of Petaluma's preliminary request for a navigational analysis of the proposed Caulfield Lane Bridge, mile 12.7, over the Petaluma River, at the City of Petaluma, Sonoma County, California.

The General Bridge Act of 1946, as amended, requires the location and plans for bridges over navigable waters of the United States be approved by the Commandant, U.S. Coast Guard prior to commencing construction. The Petaluma River is considered to be a navigable waterway of the United States for bridge administration purposes at the proposed bridge project site and a Coast Guard Bridge Permit will be required.

Coast Guard bridge permitting is considered to be a federal action and subject to the National Environmental Policy Act (NEPA).

Based upon the information currently available, we have made a preliminary determination that to provide for the current and prospective future reasonable needs of navigation on the Petaluma River, an application for the proposed Caulfield Lane Bridge should provide the following navigational clearances:

<u>Closed Position</u>	<u>Open Position</u>
Horizontal: 90 ft measured normal to the axis of the channel.	Horizontal: 90 ft measured normal to the axis of the channel.
Vertical: 10 ft above Mean High Water	Vertical: Unlimited

Please note that this preliminary determination does not constitute an approval or final agency determination, which we can only make, in accordance with regulation and after the City of Petaluma submits a completed bridge permit application.

To assist with the application for a Coast Guard Bridge Permit, please refer to the Coast Guard's Bridge Permit Application Guide (COMDTPUB P16591.3D, https://go.usa.gov/xRFk2).

You may contact Ms. Rachel Zamora, Project Manager by telephone at (510) 437-3515 or by email at Rachel.C.Zamora@uscg.mil, to discuss this project.

Sincerely,

C. T. HAUSNER
Chief, Bridge Section
Eleventh Coast Guard District
By direction of the District Commander

Copy: U.S. Army Corps of Engineers, San Francisco District, Regulatory Division
U.S. Coast Guard Sector San Francisco, Waterways Management
Thomas Barnard, P.E., AECOM

U.S. Department of Homeland Security
United States Coast Guard

Commander
Eleventh District

U.S. Coast Guard Island
Building 50-2
Alameda, CA 94501-5100
Staff Symbol: dpw
Phone: (510) 437-3516
Fax: (510) 437-5836

May 6, 2019

PRELIMINARY PUBLIC NOTICE (11-150)

The United States Coast Guard is soliciting public comments on a proposal by the City of Petaluma to construct a new bridge across the Petaluma River. The General Bridge Act of 1946 requires approval of the location and plans for bridges over navigable waters of the United States, prior to commencing construction. A Coast Guard Bridge Permit would be required for this project.

WATERWAY AND LOCATION: Petaluma River, mile 12.7, at the City of Petaluma, Sonoma County, California.
Position: 38.228802 -122.618358

CHARACTER OF PROPOSED WORK: The City of Petaluma is proposing to construct a drawbridge upstream of the US 101 Highway Bridge at mile 12.7 over the Petaluma River, at the City of Petaluma, Sonoma County, CA.

MINIMUM PROPOSED NAVIGATIONAL CLEARANCES:

	OPEN POSITION	CLOSED POSITION
Horizontal (normal to the axis of the channel)	90 feet	90 feet
Vertical Clearance Above Mean High Water (MHW)	Unlimited	10 feet

Datum: MHW elevation of 6.35 feet (NAVD 1988)

SOLICITATION OF COMMENTS:

Mariners are requested to comment on navigational safety issues, including the need for pier protection, clearance gauges and extent of nighttime navigation through the bridge. Interested parties are requested to express their views, in writing on the proposed bridge project, giving sufficient detail to establish a clear understanding of their reasons for support of or opposition to the proposed project. Comments will be received for the record at Commander (dpw), Eleventh Coast Guard District, Coast Guard Island, Bldg 50-2, Alameda, CA 94501-5100, through June 5, 2019.

Plans of the proposed project are included in this preliminary public notice.

//s//
CARL T. HAUSNER
Chief, Bridge Section
Eleventh Coast Guard District
By direction of the District Commander

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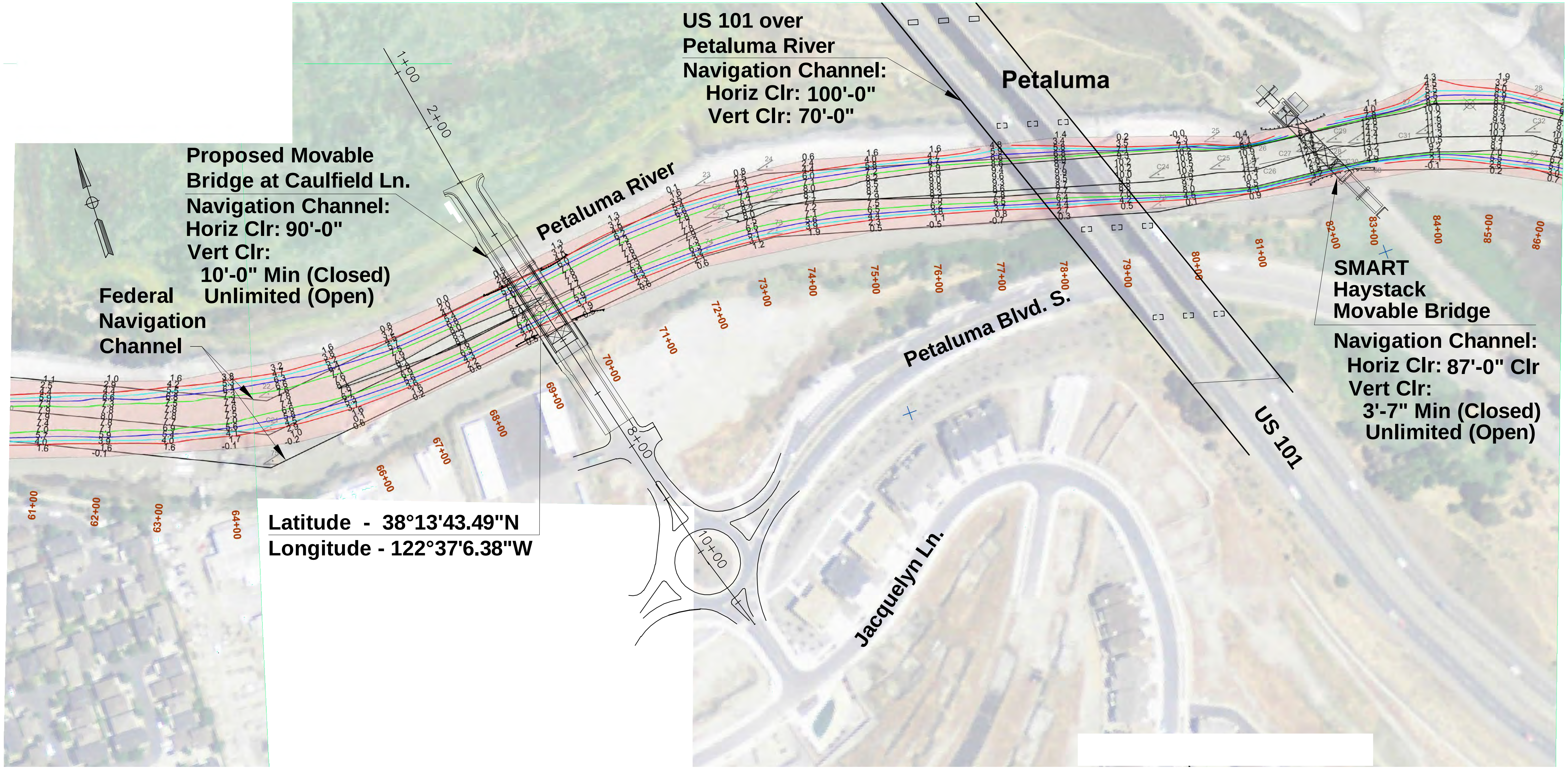

Notes:

1. *Background information taken from Army Corps of Engineers "Petaluma River Condition Survey" Dated December 2017.*

PLAN

SCALE: 1" = 80'

				Contours	
	Federal Navigation Channel		Beacon, General		-8
	Shoaling Area		Obstruction Point		-7
	Placement Area		Navigation Buoy		-6
	Anchorage Area		Navigation Buoy		-5
	Wreck Area		Shoalest Sounding*		-4
	Submerged Wreck				
	Angle Point				

DESIGN OVERSIGHT	DESIGN	BY BARNARD T.	CHECKED	LOAD FACTOR DESIGN	LIVE LOADING: *HS20-44 AND ALTERNATIVE AND PERMIT DESIGN LOAD*	
SIGN OFF DATE	DETAILS	BY WALTZ T.	CHECKED	LAYOUT	BY	CHECKED
	QUANTITIES	BY	CHECKED	SPECIFICATIONS	BY	PLANS AND SPECS COMPARED

PRELIMINARY

NOT FOR CONSTRUCTION

THOMAS BARNARD

PROJECT ENGINEER

BRIDGE NO.

POST MILE

CAULFIELD LANE OVER PETALUMA RIVER

CHANNEL ALIGNMENT AND NAVIGATION CLEARANCE

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3 4

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EA

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY)

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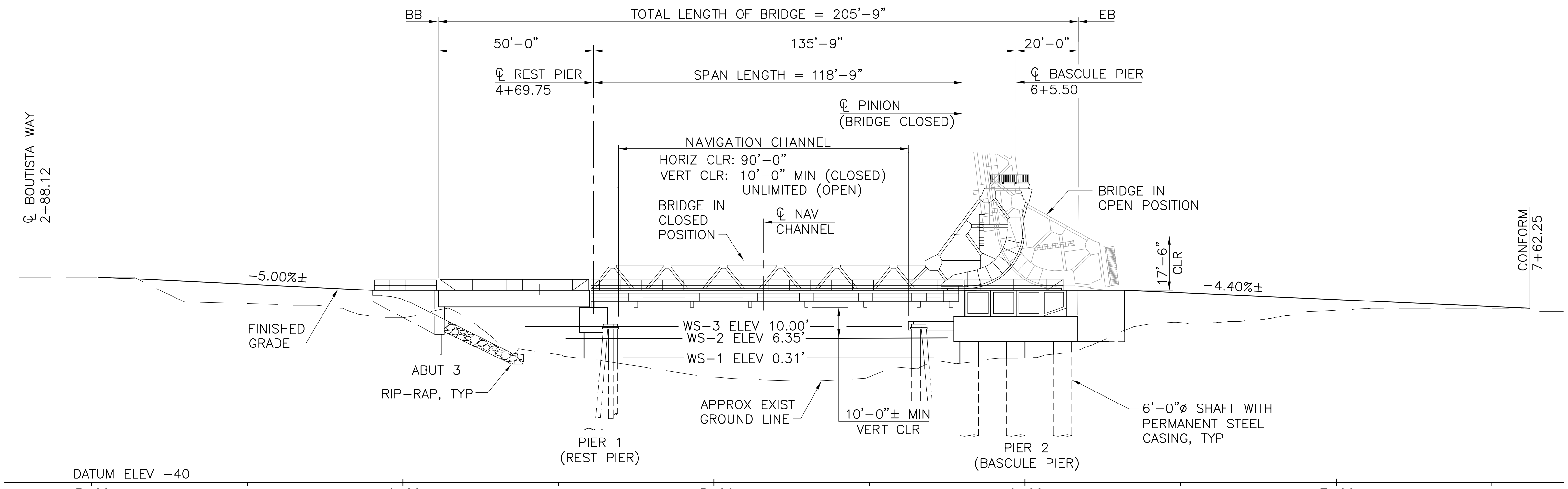

ELEVATION

SCALE: 1" = 20'

LEGEND

WS-1	MEAN LOWER LOW WATER, -0.31' NAVD88, PER NOAA/NOS
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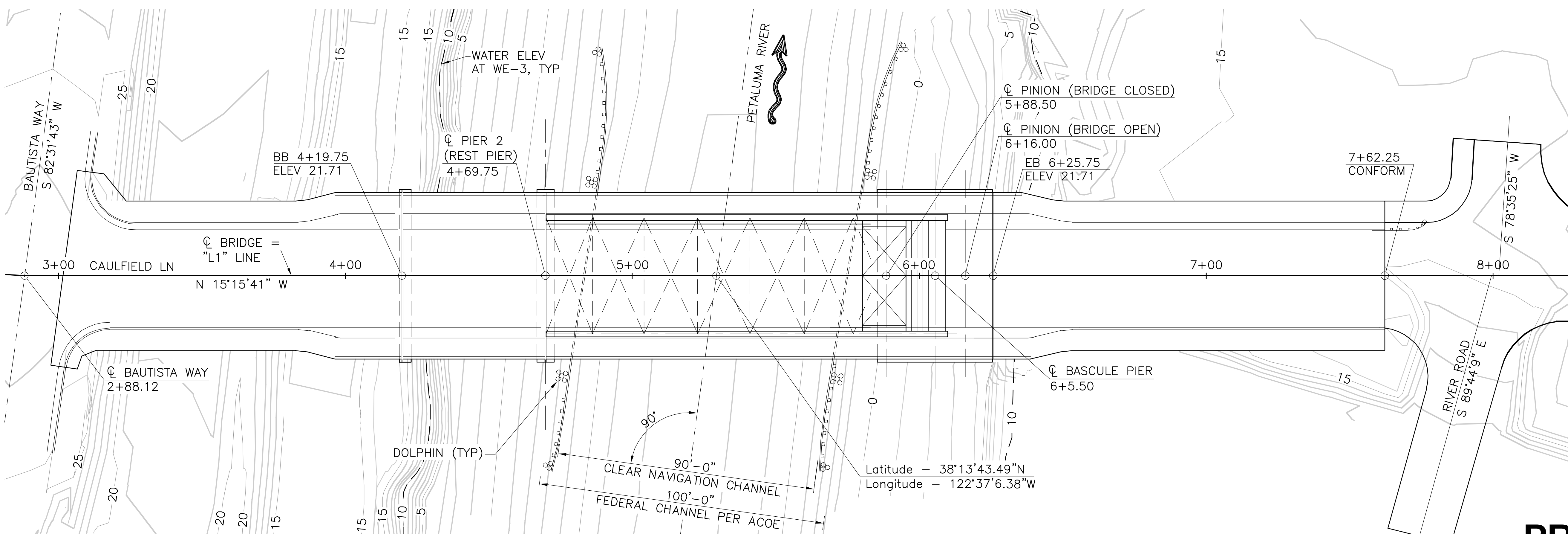
WS-3	100 YEAR BASE FLOOD ELEVATION, 10.00' NAV88, PER FEMA



PLAN

SCALE: 1" = 20'

PRELIMINARY

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REGISTERED PROFESSIONAL ENGINEER — THOMAS R. BARNARD — No. C46384 — Exp. 03-31-21 — CIVIL — STATE OF CALIFORNIA

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SIGN OFF DATE	QUANTITIES	BY	CHECKED	SPECIFICATIONS	BY	PLANS AND SPECS COMPARED	PUBLIC WORKS DEPARTMENT	

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3 4

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BRIDGE NO.
POST MILE

CAULFIELD LANE OVER PETALUMA RIVER

GENERAL PLAN AND ELEVATION

DISREGARD PRINTS BEARING EARLIER REVISION DATES

REVISION DATES (PRELIMINARY STAGE ONLY) | SHEET | OF

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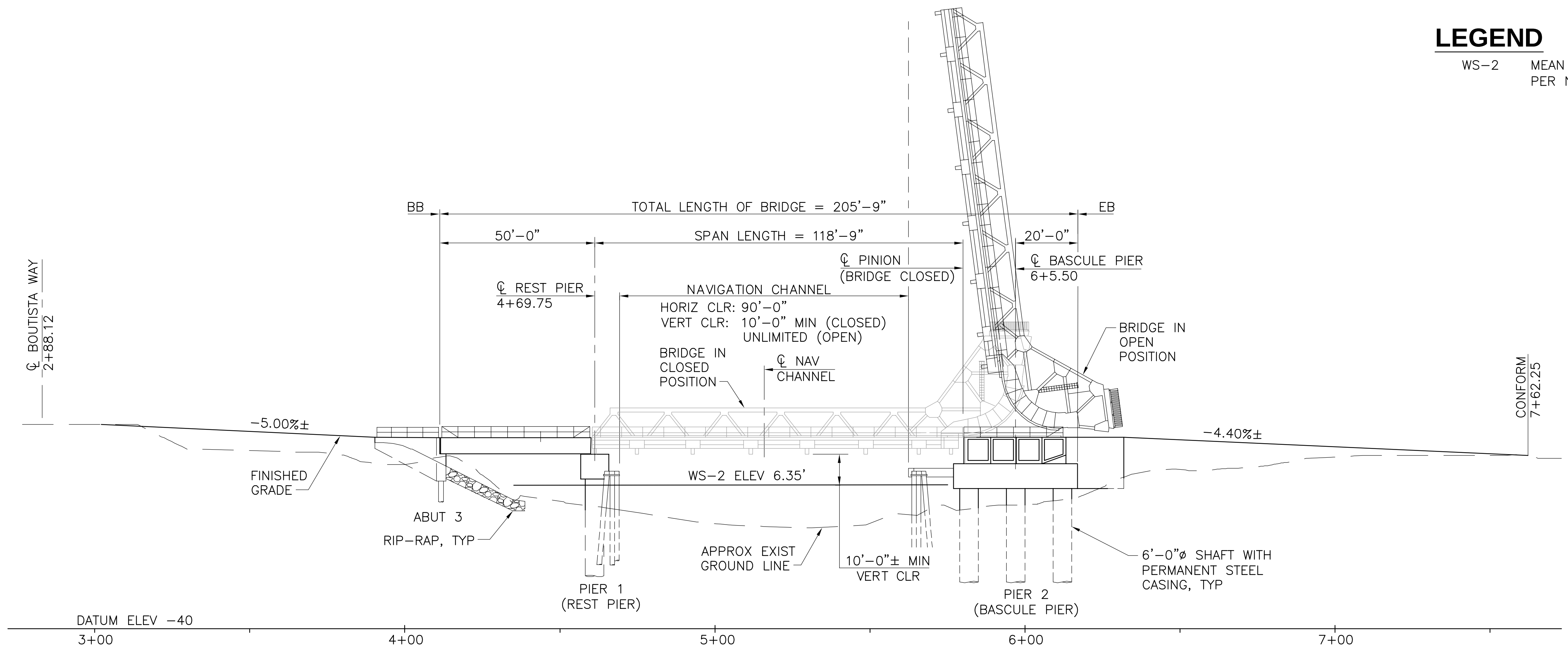
REGISTERED PROFESSIONAL ENGINEER THOMAS R. BARNARD No. C46384 Exp. 03-31-21 CIVIL STATE OF CALIFORNIA

AECOM
Technical Services, Inc.
2020 L Street, Suite 300
Sacramento, CA. 95811

LEGEND

WS-2 MEAN HIGHER HIGH WATER, 6.35' NAVD88 PER NOAA/NOS



ELEVATION

SCALE: 1" = 20'

PRELIMINARY

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SIGN OFF DATE	QUANTITIES	BY	CHECKED	SPECIFICATIONS	BY	PLANS AND SPECS COMPARED	PUBLIC WORKS DEPARTMENT

ORIGINAL SCALE IN INCHES FOR REDUCED PLANS 0 1 2 3 4

THOMAS BARNARD
PROJECT ENGINEER

CU
EA

BRIDGE NO.
POST MILE

CAULFIELD LANE OVER PETALUMA RIVER
VERTICAL CLEARANCE ELEVATION

DISREGARD PRINTS BEARING EARLIER REVISION DATES
REVISION DATES (PRELIMINARY STAGE ONLY)
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APPENDIX B – Traffic Impact Study

Traffic Impact Study for the Caulfield Bridge and Extension Project

Prepared for the City of Petaluma

Submitted by
W-Trans

January 11, 2021

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Executive Summary

The proposed Caulfield Bridge and Extension project in the City of Petaluma would extend Caulfield Lane from its current terminus at Hopper Street over the Petaluma River to connect to the north leg of the Petaluma Boulevard South/Crystal Lane roundabout. This project would not be expected to generate new trips in and of itself but would impact the surrounding roadway network by providing another access point across the Petaluma River thereby rerouting some local traffic.

The study area includes nine intersections along Washington Street, East D Street, Lakeville Street and Caulfield Lane. These intersections were chosen because the new link on Caulfield Lane would be expected to pull traffic away from Washington Street and East D Street which also cross the Petaluma River, while adding this traffic to Lakeville Street which would connect to the new crossing.

Due to the COVID-19 pandemic and the resulting changes in traffic patterns, historic traffic counts from 2015 to 2019 were preferred over collecting new counts. Historical traffic data was not available for two of the study intersections, so new counts were collected there as well as at Caulfield Lane/Lakeville Street, for which counts collected in 2018 were used as the "control". The 2018 and 2020 counts for Caulfield Lane/Lakeville Street were compared to determine probable volumes at the other two intersections in 2018. An annual growth rate derived from the Sonoma County Transportation Authority's (SCTA) gravity demand model was applied to all of these volumes to generate the Adjusted 2020 scenario, with all volumes grown to an estimated 2020 level had the pandemic not occurred. The nine study intersections would operate acceptably at Level of Service (LOS) D or better with the Adjusted 2020 volumes, except the intersection of East D Street/ Lakeville Highway, which would operate at LOS E.

Future volumes were developed for a horizon year of 2040 using data from the SCTA's gravity demand model. With the Future volumes applied, Washington Street/Petaluma Boulevard would operate at LOS E during the p.m. peak hour, East Washington Street/Lakeville Street would operate at LOS E during both peak hours, East Washington Street/Payran Street would operate at LOS E during the morning peak hour and LOS F during the evening peak hour, and East D Street/Lakeville Street would operate at LOS E during the a.m. peak hour. Operation would be acceptable at LOS D or better for the remaining time periods and study intersections.

The SCTA's model was run with an added link representing the Caulfield Bridge to generate Future with Bridge volumes. The difference between the Future with Bridge volumes and Future volumes was used to represent the change in travel that would occur with the bridge; these volumes were applied to the Adjusted 2020 volumes to generate the Adjusted 2020 with Bridge scenario. Under this scenario, East D Street/Lakeville Street would continue to operate at LOS E, which would not represent an adverse impact. The intersection of Caulfield Lane/Lakeville Street would degrade to LOS E, which could be mitigated to LOS D through reassigning several approach lanes on Caulfield Lane and applying a few changes to the signal phasing. The other intersections would operate acceptably under the Adjusted 2020 with Bridge conditions.

The intersections would operate at the same levels of service under Future with Bridge conditions as with Future conditions, except for the intersection of Caulfield Lane/Lakeville Street which would degrade to LOS F. The improvements indicated above would reduce the delay and raise the morning peak hour LOS to E, though the afternoon peak hour would continue to have LOS F operation. To improve operations to LOS D or better, widening would be required; this is not recommended as it would conflict with the City of Petaluma's *General Plan*'s multi-modal objectives.

Impacts to queue lengths were also assessed. In a few locations, a queue that would be contained within the turn pocket without the bridge would exceed capacity upon completion of the bridge, although the average queue in the adjacent through lane would extend beyond the entrance to the turn lane, preventing an adverse effect. Extending the 140-foot eastbound left-turn lane at Caulfield Lane/Lakeville Street by 75 feet through reducing the adjacent 145-foot westbound turn lane that serves a private driveway would address the queuing issue for this turn lane. Additionally, the northbound left-turn lane queue would grow to beyond the adjacent upstream intersection upon completion of the bridge, presenting an adverse effect.

The bridge would provide a critical link for pedestrians and bicyclists across the Petaluma River as the nearest crossing to the north is a mile away and there are no crossings further south. As such, the bridge should include sidewalks connecting to the existing network as well as bicycle lanes. It is recommended that the existing railroad at-grade crossing on Caulfield Lane be upgraded by adding striping denoting the continuation of the sidewalk across the tracks, a flashing-light signal on the northeast corner to raise visibility for pedestrians approaching this corner of the crossing, and flashing-light signals oriented east-west to raise visibility for pedestrians arriving from the east on the north side of Hopper Street.

Vehicle miles traveled (VMT) was assessed for the Caulfield Bridge as well as the proposed Rainier Avenue extension over US 101 and to Petaluma Boulevard North, either with or without ramps connecting to US 101. As compared to 2040 conditions without either bridge, the Caulfield Bridge would result in a slight reduction in regional VMT as would the Rainier Avenue bridge without US 101 ramps. The regional VMT would increase slightly with the Rainier Avenue bridge if US 101 ramps were to be included.

Introduction

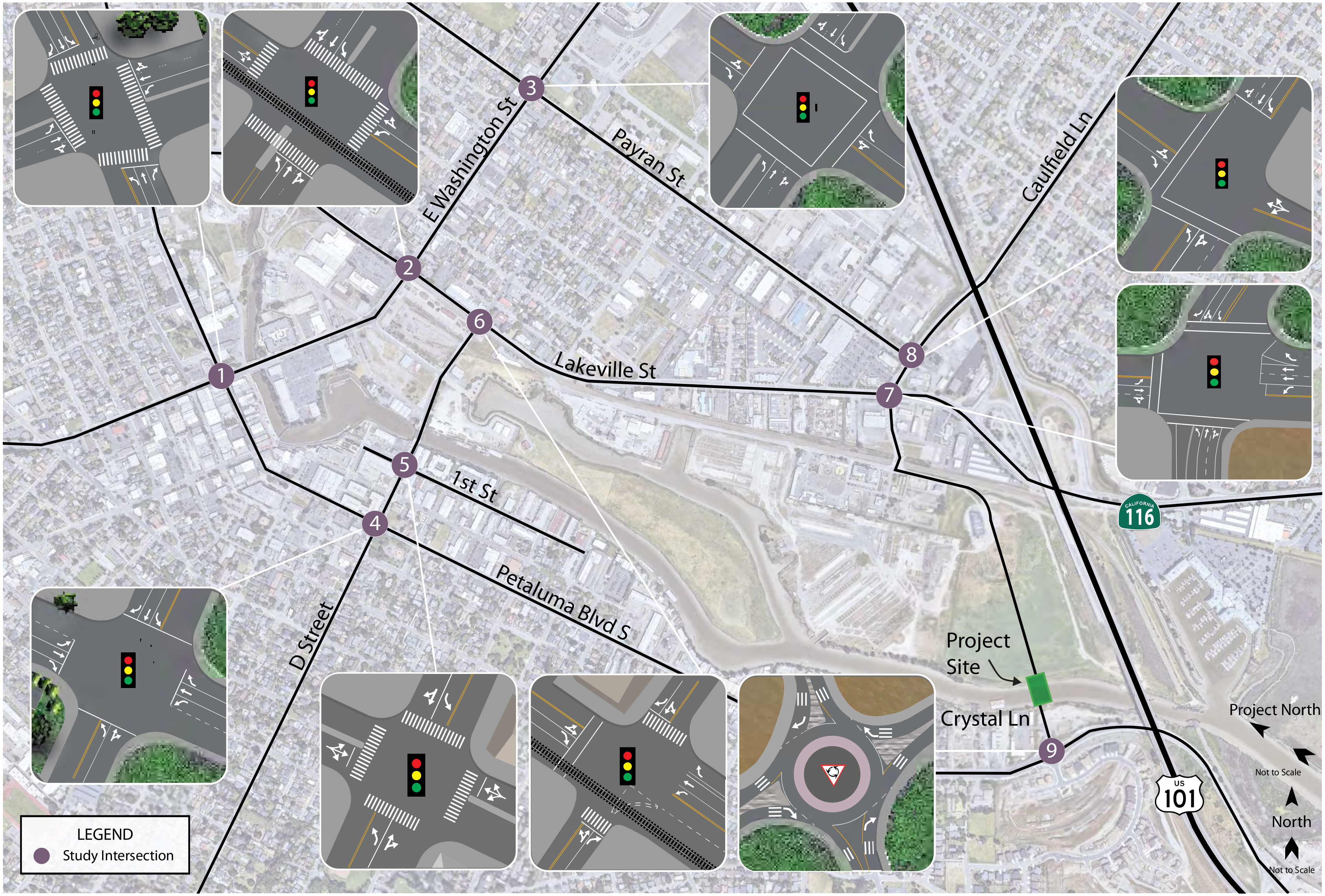
This report presents an analysis of the potential traffic impacts that would be associated with the extension of Caulfield Lane across the Petaluma River in the City of Petaluma. The traffic study was completed in accordance with the criteria established by the City of Petaluma and is consistent with standard traffic engineering techniques.

Prelude

The purpose of a traffic impact study is to provide City staff and policy makers with data they can use to make an informed decision regarding the potential traffic impacts of a proposed project, and any associated improvements that would be required to mitigate these impacts to a level of insignificance as defined by the City's *General Plan* or other policies. For this project vehicular traffic impacts were evaluated by determining how the project would affect traffic patterns, redistributing trips on the street system to make use of the new infrastructure, then analyzing the impact the new traffic patterns would be expected to have on critical intersections or roadway segments.

Project Profile

The proposed project would extend Caulfield Lane from the existing terminus on the north side of the Petaluma River south to the intersection of Petaluma Boulevard South/Crystal Lane. This would involve a new bridge across the Petaluma River that would enable drivers to cross the river without routing through downtown or via US 101, in addition to increasing bicycle and pedestrian connectivity in the area. The location of the project site is shown in Figure 1.



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Traffic Impact Study for the Caulfield Bridge and Extension Project

Figure 1 – Study Area and Existing Lane Configurations

Transportation Setting

Operational Analysis

Study Area and Periods

The study area consists of the following intersections:

1. Washington Street/Petaluma Boulevard
2. East Washington Street/Lakeville Street
3. East Washington Street/Payran Street
4. D Street/Petaluma Boulevard South
5. D Street/1st Street
6. East D Street/Lakeville Street
7. Caulfield Lane/Lakeville Street
8. Caulfield Lane/Payran Street
9. Petaluma Boulevard South/Crystal Lane

Operating conditions during the a.m. and p.m. peak periods were evaluated to capture the highest potential impacts for the proposed project as well as the highest volumes on the local transportation network. The morning peak hour occurs between 7:00 and 9:00 a.m. and reflects conditions during the home to work or school commute, while the p.m. peak hour occurs between 4:00 and 6:00 p.m. and typically reflects the highest level of congestion during the homeward bound commute.

For the purpose of this analysis, Washington Street, D Street, and Caulfield Lane are considered to be oriented east-west, while Payran Street and Lakeville Street are oriented north-south. Petaluma Boulevard South is considered to be oriented north-south at Washington Street and D Street, and east-west at Crystal Lane due to the curvature in its alignment as it parallels the Petaluma River.

Study Intersections

Washington Street/Petaluma Boulevard is a four-legged signalized intersection with protected left-turn phasing on all approaches and right-turn overlaps provided on the Petaluma Boulevard approaches. There are pedestrian signals and marked crosswalks on all legs of the intersection, and bicycle shared lane markings ("sharrows") on Petaluma Boulevard.

East Washington Street/Lakeville Street is a signalized, four-legged intersection with split phasing on the Lakeville Street approaches and protected left-turn phasing on the East Washington Street approaches. The SMART tracks run parallel to and along the west side of Lakeville Street and pass through the west East Washington Street leg of the intersection. There is railroad signal infrastructure, including crossing arms, for both East Washington Street legs. There are marked crosswalks and pedestrian signals on all legs of the intersection, and sharrows on Lakeville Street.

East Washington Street/Payran Street is a signalized intersection with protected left-turn phasing on all four approaches. There are pedestrian signals and marked crosswalks on all legs of the intersection and sharrows on Payran Street.

D Street/Petaluma Boulevard South is a four-legged signalized intersection. The D Street approaches have protected left-turn phasing and the Petaluma Boulevard South approaches have split phasing. There are marked crosswalks and pedestrian signals on all legs of the intersections, and sharrows on the Petaluma Boulevard South legs.

D Street/1st Street is a four-legged signalized intersection with protected left-turn phasing on the D Street approaches and permissive left-turn phasing on the 1st Street approaches. All legs of the intersection have marked crosswalks and pedestrian signals.

East D Street/Lakeville Street is a four-legged signalized intersection with protected left-turn phasing on the northbound Lakeville Street approach, permitted left-turn phasing on the southbound Lakeville Street approach and split phasing of the East D Street approaches, as well as a right-turn overlap on the eastbound East D Street approach. The SMART tracks run parallel to and along the west side of Lakeville Street and pass through the west leg of the intersection. Railroad signal infrastructure and crossing arms exist on both East D Street legs. There are marked crosswalks and pedestrian signals for all but the south Lakeville Street leg of the intersection. The south leg has bicycle lanes while the west and north legs have sharrows.

Caulfield Lane/Lakeville Street is a signalized intersection with four legs. The Caulfield Lane approaches have split phasing, while the Lakeville Street approaches have protected left-turn phasing. There are marked crosswalks and pedestrian signals on the east, north, and west legs of the intersection, and bicycle lanes on all legs.

Caulfield Lane/Payran Street is a four-legged signalized intersection with protected left-turn phasing on the Caulfield Lane approaches as well as a right-turn overlap on the eastbound Payran Street approach. There are marked crosswalks and pedestrian signals only on the west and north legs of the intersection, and bicycle lanes on Caulfield Lane.

Petaluma Boulevard South/Crystal Lane is a four-legged single-lane roundabout with yield controls on all legs of the intersection. Crosswalks and median crossing islands are on all four legs of the roundabout. There are bicycle lanes on Petaluma Boulevard South, with markings and curb ramps that enable bicyclists to traverse the roundabout either via bicycle or as a pedestrian.

The locations of the study intersections and the existing lane configurations and controls are shown in Figure 1.

Collision History

The collision history for the study area was reviewed to determine any trends or patterns that may indicate a safety issue. Collision rates were calculated based on records available from the California Highway Patrol as published in their Statewide Integrated Traffic Records System (SWITRS) reports. The most current five-year period available is January 1, 2015 through December 31, 2019.

As presented in Table 1, the calculated collision rates for the study intersections were compared to average collision rates for similar facilities statewide, as indicated in *2016 Collision Data on California State Highways*, California Department of Transportation (Caltrans). The calculated collision rates for most of the study intersections were higher than the statewide rate for similar facilities. The collision rate calculations are provided in Appendix A.

Table 1 – Collision Rates for the Study Intersections			
Study Intersection	**Number of Collisions (2015-2019)**	**Calculated Collision Rate (c/mve)**	**Statewide Average Collision Rate (c/mve)**
1. Washington St/Petaluma Blvd	31	**0.58**	0.24
2. E Washington St/Lakeville St	22*	**0.79**	0.24
3. E Washington St/Payran St	31	**0.67**	0.24
4. D St/Petaluma Blvd S	19	**0.52**	0.24
5. D St/1st St	7	0.20	0.24
6. E D St/Lakeville St	14*	**0.59**	0.24
7. Caulfield Ln/Lakeville St	18	**0.40**	0.24
8. Caulfield Ln/Payran St	6	0.17	0.24
9. Petaluma Blvd S/Crystal Ln	25	**1.41**	0.08

Note: c/mve = collisions per million vehicles entering; **bold** text indicates actual rates that are higher than the statewide average; * = these intersections were assessed for a three-year period from January 1, 2017 to December 31, 2019, due to significant geometric and/or operational changes in 2015 and 2016.

The most common primary collision factors for the 31 crashes reported for Washington Street/Petaluma Boulevard included improper turning (eight crashes), unsafe starting/backing (seven crashes), speeding (six crashes), and driving while under the influence (four crashes). Twelve collisions occurred between 1:00 p.m. and 3:00 p.m. when the signal operates under a midday phasing pattern, and 15 of the total collisions were rear-ends. Updates to the signal timing and coordination, particularly during the midday, may help alleviate these types of collisions. It is noted that the intersection had a below-average incidence of injuries, indicating that although there is an above-average number of crashes, there is not a demonstrated safety concern.

Of the 22 collisions reported for East Washington Street/Lakeville Street in the three-year period between 2017 and 2019, six were the result of northbound drivers turning left and colliding with a fixed object including two trucks with trailers and one pickup truck with a trailer. A review of the intersection geometry reveals that there is a small median island with a railroad crossing gate on the west leg of the intersection that affects the path of northbound left-turning vehicles by reducing the available turning radius. Given the constrained geometry, it is likely that some of these collisions were with this median island, including the three involving vehicles pulling trailers. Prohibiting trucks and other vehicles towing trailers from turning left on northbound Lakeville Street may reduce this collision type, though an engineering study should be conducted to determine the maximum length of vehicle that can be accommodated. This intersection had a below-average incidence of injuries, indicating that there is not a specific safety concern requiring mediation.

The most common type of collision for East Washington Street/Payran Street was rear-ends, accounting for 15 of the 31 total reported collisions. Improved signal timing and coordination may help alleviate this collision type. Additionally, 14 of the reported collisions included speeding as the primary collision factor, which may be alleviated by increased speeding enforcement.

The top three primary collision factors for the 19 collisions reported for the intersection of D Street/ Petaluma Boulevard South included speeding (five crashes), improper turning (four crashes), and red-light running (three crashes). Speed enforcement may reduce the incidence of speeding collisions and improving signal head visibility may reduce the incidence of red-light running. This intersection had a below-average injury rate for all crashes, so there is not a safety concern associated with the above-average crash rate.

The 14 collisions reported during the three-year study period for East D Street/Lakeville Street had a variety of primary collision factors, collision types, and other crash details. However, four similar crashes were the result of southbound left-turning drivers on Lakeville Highway failing to yield right-of-way to oncoming northbound through traffic. A flashing yellow arrow indication may increase awareness to southbound left-turning drivers of the need to yield to oncoming traffic. Additionally, due to the skew of the intersection it may be difficult for southbound drivers to observe oncoming northbound through vehicles if the adjacent northbound left-turn lane is saturated with queued vehicles. Providing a protected southbound left-turn phase would prevent this conflict, though it would need to be accommodated either through splitting the northbound and southbound phases, or reassigning the southbound lanes from a right-turn lane and shared through/left-turn lane to a shared through/right-turn lane and left-turn lane, effectively moving the through movement from the left lane to the right lane. As the injury rate for this location was slightly below the statewide average, there is not an immediate safety concern to be addressed, though the City may wish to evaluate this location for action at a future date.

The two most reported primary collision factors for Caulfield Lane/Lakeville Street were speeding and drunk driving, with four collisions each. Increased enforcement may reduce these types of collisions. Of the 18 reported collisions, nine were rear-ends which may be reduced through improved signal timing and coordination. This intersection also had a below-average injury rate, indicating that though the crash rate was higher than the statewide average, there is not a demonstrated safety concern requiring immediate remediation.

Of the 25 collisions reported for the Petaluma Boulevard South/Crystal Lane roundabout during the five-year study period, 23 were single-vehicle collisions with objects. There were 13 collisions with speeding listed as the primary collision factor as well as eight with driving under the influence. Increased enforcement may reduce the incidence of these types of collisions, especially between the hours of 9:00 p.m. and 7:30 a.m. when 14 of the collisions occurred and on Fridays and Saturdays when 15 of the collisions occurred. Of note, while the overall collision rate was several times higher than the statewide average for similar facilities, most of these crashes did not result in injury. Four of the 25 crashes resulted in injury leading to an injury rate of 16.1 percent, which is lower than the statewide average for similar facilities of 45.1 percent.

Capacity Analysis

Intersection Level of Service Methodologies

Level of Service (LOS) is used to rank traffic operation on various types of facilities based on traffic volumes and roadway capacity using a series of letter designations ranging from A to F. Generally, Level of Service A represents free flow conditions and Level of Service F represents forced flow or breakdown conditions. A unit of measure that indicates a level of delay generally accompanies the LOS designation.

The study intersections were analyzed using methodologies published in the *Highway Capacity Manual* (HCM), *Sixth Edition*, Transportation Research Board, 2017. This source contains methodologies for various types of intersection control, all of which are related to a measurement of delay in average number of seconds per vehicle.

The study intersections that are currently controlled by a traffic signal were evaluated using the signalized methodology from the HCM. This methodology is based on factors including traffic volumes, green time for each movement, phasing, whether the signals are coordinated or not, truck traffic, and pedestrian activity. Average stopped delay per vehicle in seconds is used as the basis for evaluation in this LOS methodology. For purposes of this study, delays were calculated using signal timing obtained from the City of Petaluma. Optimized signal timing was used for all but the Adjusted 2020 conditions scenario, which serves as the existing conditions scenario for this report as it approximates what traffic conditions would be like in 2020 had the COVID-19 pandemic not occurred. Therefore, the existing signal timings were assumed to be relevant for the Adjusted 2020 conditions scenario.

The intersection of Petaluma Boulevard South/Crystal Lane, which is currently controlled by a modern roundabout, was evaluated using the FHWA Roundabout Method, also contained within the Unsignalized Methodology of the HCM 6th Edition, Transportation Research Board, 2016. This methodology determines intersection operation using a gap acceptance method along with basic geometric and volume data to calculate entering and circulating flows. This information is then translated to average vehicle delays, with LOS break points at the same delays as used in the two-way stop-controlled methodology.

The ranges of delay associated with the various levels of service are indicated in Table 2.

Table 2 – Intersection Level of Service Criteria		
LOS	**Roundabout**	**Signalized**
A	Delay of 0 to 10 seconds.	Delay of 0 to 10 seconds. Most vehicles arrive during the green phase, so do not stop at all.
B	Delay of 10 to 15 seconds.	Delay of 10 to 20 seconds. More vehicles stop than with LOS A, but many drivers still do not have to stop.
C	Delay of 15 to 25 seconds.	Delay of 20 to 35 seconds. The number of vehicles stopping is significant, although many still pass through without stopping.
D	Delay of 25 to 35 seconds.	Delay of 35 to 55 seconds. The influence of congestion is noticeable, and most vehicles have to stop.
E	Delay of 35 to 50 seconds.	Delay of 55 to 80 seconds. Most, if not all, vehicles must stop and drivers consider the delay excessive.
F	Delay of more than 50 seconds.	Delay of more than 80 seconds. Vehicles may wait through more than one cycle to clear the intersection.

Reference: *Highway Capacity Manual*, Transportation Research Board, 2017

Traffic Operation Standards

The *City of Petaluma: General Plan 2025* has an adopted Level of Service (LOS) standard for streets that indicates the minimum acceptable operation is LOS D, with the following standard of significance for motor vehicle circulation:

> ***Policy 5-P-10:*** *Maintain an intersection level of service (LOS) standard for motor vehicle circulation that ensures efficient traffic flow and supports multi-modal mobility goals. LOS should be maintained at Level D or better for motor vehicles due to traffic from any development project.*

With the current General Plan, the City is shifting toward a multimodal emphasis and LOS standard. "A multimodal analysis that, in addition to motor vehicles, takes into consideration the overall mobility and conditions for non-auto road users (e.g., bicycles and pedestrians) is highly encouraged." The Community Character Element of the General Plan also contains circulation-related objectives and policies. This element directs that pedestrian and bicycle circulation be integrated into street designs and improvements. It also states that the amount of paving and the apparent width of streets should be reduced where possible.

Per the General Plan, the project would have an adverse effect if it causes the average delay at an intersection already operating or expected to operate at LOS D or E to deteriorate to the next lower level of service. Additionally, the project would have an adverse effect if it adds additional vehicle trips to an intersection already operating or expected to operate at LOS F.

The *Petaluma General Plan 2025 Draft Environmental Impact Report*, 2006, included cumulative analysis of major intersections in Petaluma. On pages 3.2-34 and 3.2-35 of the DEIR, it is acknowledged that buildout of the General Plan would result in unacceptable operations at the intersection of Lakeville Street/East D Street. The DEIR found these impacts to be significant and unavoidable, citing that the addition of new lanes and/or expanded capacity would be in conflict with the Plan's policies relating to improving multi-modal circulation.

The *General Plan* does not prescribe thresholds of significance regarding queue lengths. However, an increase in queue length due to project traffic was considered a potential impact if the increase would cause the queue to extend out of a dedicated turn lane into a through traffic lane, or the back of queue into a visually restricted area, such as a blind corner.

Adjusted 2020 Conditions

Derivation of Traffic Volumes

Typically, an "Existing Conditions" scenario is assessed using recently-collected traffic volume data. However, due to the COVID-19 pandemic, reliable traffic volumes could not be collected as a result of sheltering-in-place, working-from-home and other shifts in travel patterns. Instead, traffic counts collected in 2015 through 2019 were used for seven of the study intersections. These volumes were factored up to 2020 using an annual growth rate of 0.77 percent per year derived from comparing Sonoma County Transportation Authority's (SCTA) gravity demand model volumes for the 2010 and 2040 model years.

Historical traffic data was not available for the intersections of Caulfield Lane/Payran Street and Petaluma Boulevard South/Crystal Lane. New traffic volumes were therefore collected at these two locations. Additionally, new volumes were collected at Caulfield Lane/Lakeville Street to provide a point of comparison using the data collected at this location in 2018. It was determined that the 2018 volumes for Caulfield Lane/Lakeville Street were 25.2 percent higher than the 2020 volumes, so this growth percentage was applied to the 2020 data collected for Caulfield Lane/Payran Street and Petaluma Boulevard South/Crystal Lane to "grow" the volumes to 2018 baseline levels. These estimated 2018 volumes were then adjusted to 2020 using the same annual growth rate of 0.77 percent as applied to the other intersections. One last adjustment was made to balance volumes between closely-spaced study intersections with limited options for adding or attracting trips, such as East Washington Street/Lakeville Street and East D Street/Lakeville Street.

With these growth factors and adjustments applied, the Adjusted 2020 traffic volumes were estimated for the a.m. and p.m. peak periods to reflect what likely would have been normal 2020 conditions without changes in travel patterns as a result of the pandemic.

Intersection Levels of Service

Under Adjusted 2020 conditions, all study intersections would operate acceptably at LOS D or better, except for East D Street/Lakeville Street, which would operate at LOS E during the a.m. peak hour. However, no mitigation is possible as the *Petaluma General Plan 2025* DEIR found that new lanes or additional vehicular capacity would conflict with the multi-modal policies of the *General Plan*.

The existing traffic volumes are shown in Figure 2. A summary of the intersection LOS calculations is contained in Table 3, and copies of the calculations are provided in Appendix B.

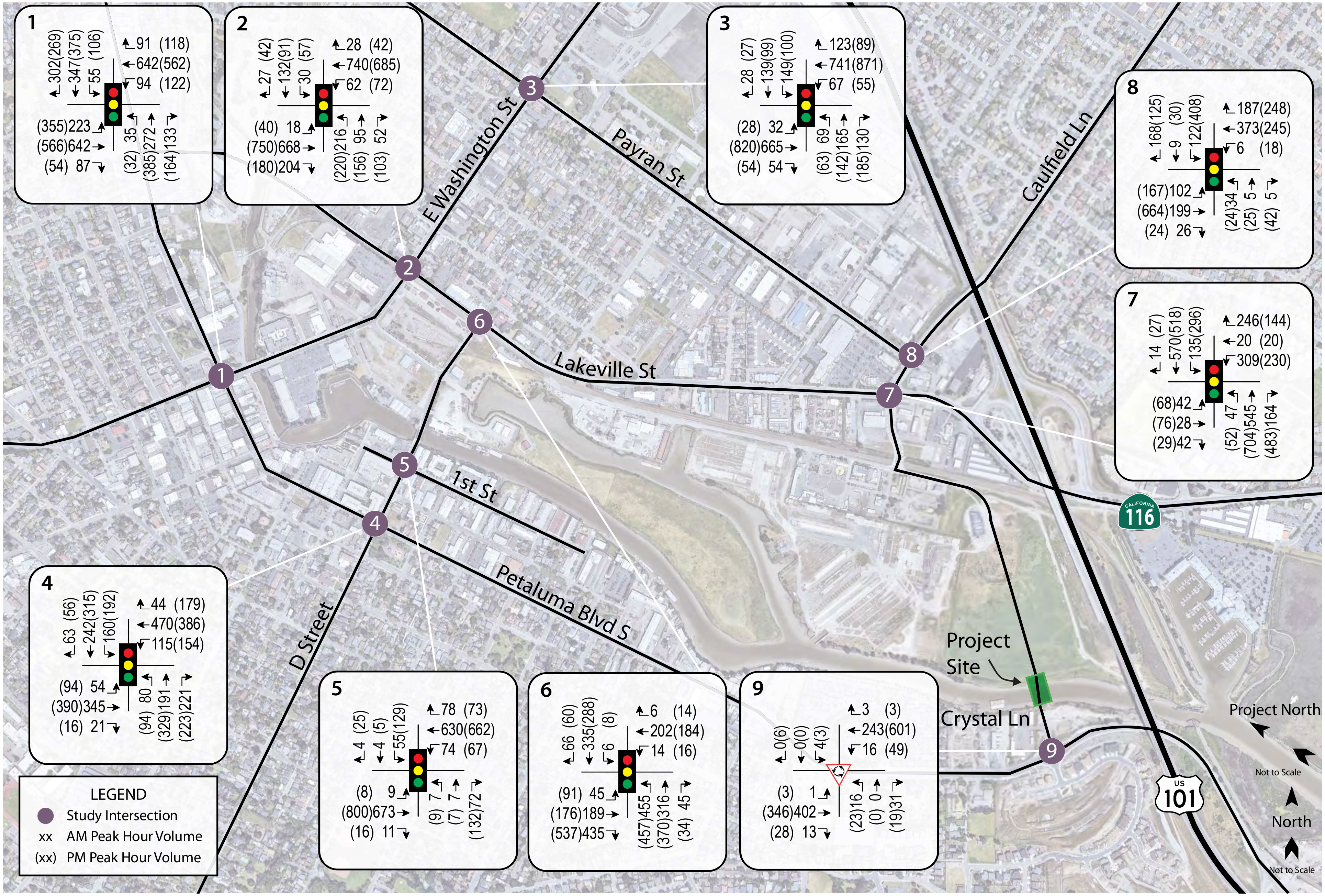

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Figure 2 – Adjusted 2020 No Bridge Traffic Volumes

Table 3 – Adjusted 2020 Peak Hour Intersection Levels of Service				
Study Intersection	**AM Peak**		**PM Peak**	
	Delay	**LOS**	**Delay**	**LOS**
1. Washington St/Petaluma Blvd	48.3	D	46.4	D
2. E Washington St/Lakeville St	48.1	D	47.8	D
3. E Washington St/Payran St	41.9	D	31.4	C
4. D St/Petaluma Blvd S	46.6	D	48.3	D
5. D St/1st St	20.8	C	27.3	C
6. E D St/Lakeville St	**58.6**	**E**	44.0	D
7. Caulfield Ln/Lakeville St	24.6	C	34.6	C
8. Caulfield Ln/Payran St	14.1	B	38.7	D
9. Petaluma Blvd S/Crystal Ln	5.3	A	7.3	A

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service; **Bold** text = deficient operation

Future Conditions

Segment volumes for the horizon year of 2040 were obtained from the SCTA's gravity demand model and translated to turning movement volumes at each of the study intersections using the "Furness" method. The Furness method is an iterative process that employs existing turn movement data, existing link volumes and future link volumes to project likely turning future movement volumes at intersections. This 2040 model reflects the existing roadway network so does not include a link representing the Caufield Bridge.

Under the anticipated Future volumes, and without the Caulfield bridge, the intersection of East D Street/ Lakeville Street would continue to operate at an unacceptable LOS E during the a.m. peak hour. Additionally, the intersections of East Washington Street/Lakeville Street and East Washington Street/Payran Street would operate at unacceptable LOS E or LOS F during both peak hours, while the intersection of Washington Street/Petaluma Boulevard would operate at LOS E during the p.m. peak hour. The other study intersections would operate acceptably at LOS D or better. Future volumes are shown in Figure 3 and operating conditions are summarized in Table 4.

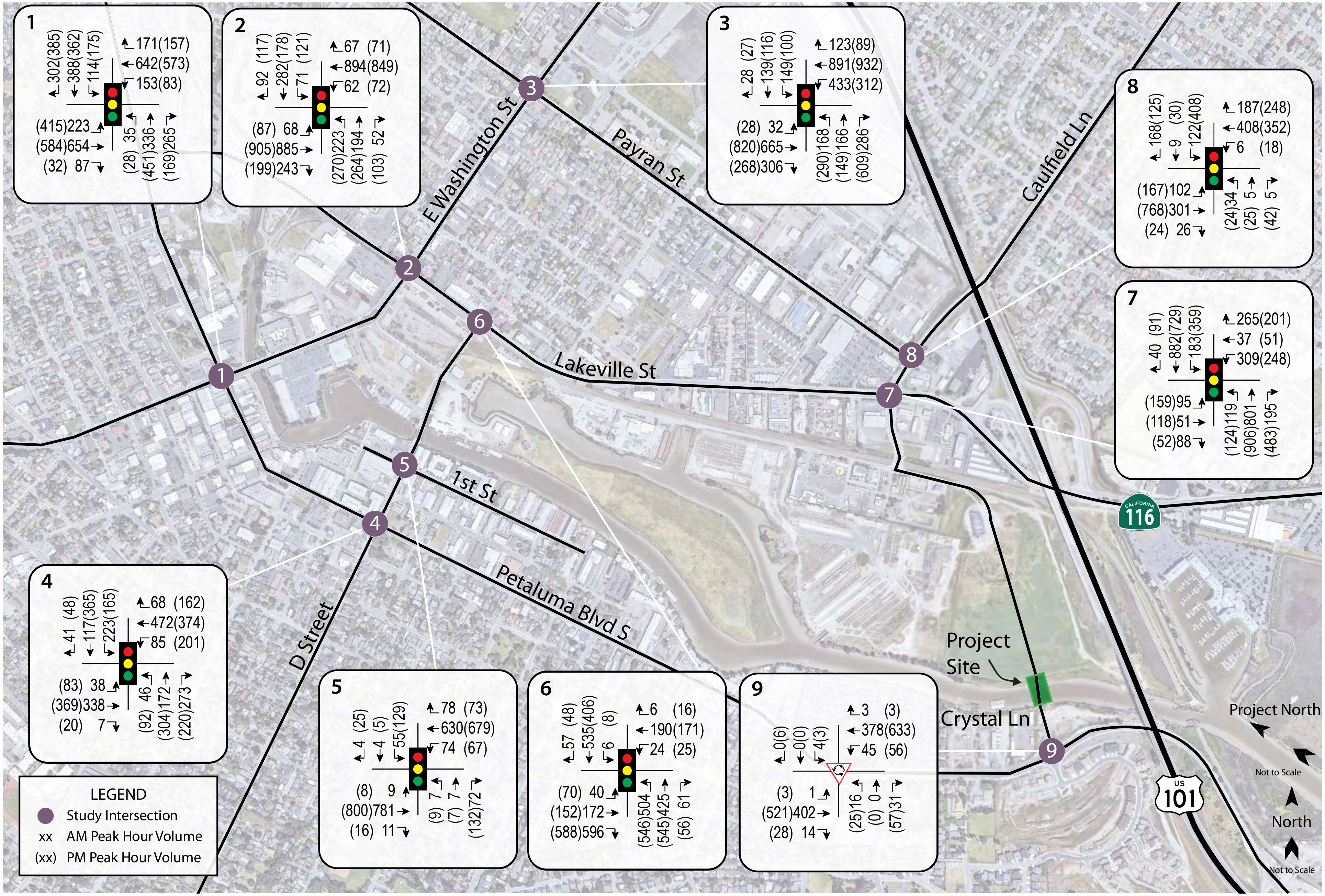

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Figure 3 – Future Traffic Volumes No Bridge

Table 4 – Future Peak Hour Intersection Levels of Service				
Study Intersection	**AM Peak**		**PM Peak**	
	Delay	**LOS**	**Delay**	**LOS**
1. Washington St/Petaluma Blvd	44.5	D	**55.7**	**E**
2. E Washington St/Lakeville St	**78.0**	**E**	**79.1**	**E**
3. E Washington St/Payran St	**66.5**	**E**	**135.2**	**F**
4. D St/Petaluma Blvd S	44.6	D	43.5	D
5. D St/1st St	21.7	C	21.7	C
6. E D St/Lakeville St	**73.9**	**E**	41.2	D
7. Caulfield Ln/Lakeville St	32.0	C	51.2	D
8. Caulfield Ln/Payran St	12.1	B	36.5	D
9. Petaluma Blvd S/Crystal Ln	5.9	A	8.2	A

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service; **Bold** text = deficient operation

Project Description

The project consists of a new bridge connecting Caulfield Lane across the Petaluma River to the north leg of the roundabout at Petaluma Boulevard South/Crystal Lane. This would provide a link between the north and south sides of the Petaluma River to allow drivers to traverse the river as an alternative to routing through the downtown or via US 101. It would provide more direct access between the southwest and southeast quadrants of Petaluma, including between the neighborhoods off Petaluma Boulevard South and those off Caulfield Lane and Lakeville Highway.

Volume Development

This project would not generate trips in the traditional way associated with land use developments such as a new office or apartment complex. Instead, trips that would otherwise route through downtown Petaluma or along US 101 would instead use the Caulfield Bridge to cross the Petaluma River. This would increase the traffic volumes of the intersections near the bridge and decrease the traffic volumes of intersections on alternative routes, such as D Street and Washington Street. Additionally, there may be some induced demand in the form of new trips resulting from shortened travel times across the river.

To assess the impact of the bridge on regional travel, the SCTA's gravity demand model was edited to assess 2040 conditions with the additional link across the Petaluma River. Similar to the Future Conditions scenario, the resulting segment volumes were then run through the Furness method to convert these volumes to turning movements at the study intersections. The results were adjusted to balance volumes between nearby intersections, similar to the development of the Adjusted 2020 volumes.

The result of this process was a projected 1.1 percent decrease in total volumes on the Washington Street corridor, a 5.5 percent decrease in total volumes along the D Street corridor, and an increase in total volumes of 26.1 percent along the Caulfield Lane corridor, including the intersection of Petaluma Boulevard South/Crystal Lane. The increase in volumes would be primarily expected between Petaluma Boulevard South to the west of Crystal Lane, the new segment of Caulfield Lane, and Lakeville Street to the east of Caulfield Lane. There would likewise be an expected increase in volumes along Caulfield Lane to the east of Lakeville Street as well. A decrease in traffic traveling between Lakeville Street, D Street, and Petaluma Boulevard South would be expected to occur as some of these trips would instead route across the Caulfield Bridge.

These model-generated volumes were used as the basis for the Future with Bridge scenario. The difference in turning movements between the Future and Future with Bridge volumes was then applied to the Adjusted 2020 volumes to generate Adjusted 2020 with Bridge volumes. It is noted that in some cases the volume adjustments would have resulted in negative volumes if applied directly; in such cases a volume of zero vehicles was used as a floor.

Intersection Operation

Adjusted 2020 with Bridge Conditions

With the changes to traffic patterns expected upon opening of the Caulfield Bridge, the study intersections would be expected to operate at the same levels of service as without the bridge, except the Petaluma Boulevard South/Crystal Lane roundabout would degrade from LOS A to LOS B during the p.m. peak hour, and the intersection of Caulfield Lane/Lakeville Street would degrade from LOS C to an unacceptable LOS E during both peak hours. These results are summarized in Table 5. Adjusted 2020 with Bridge volumes are shown in Figure 4.

Table 5 – Adjusted 2020 and Adjusted 2020 with Bridge Peak Hour Intersection Levels of Service

Study Intersection *Mitigation*	Adjusted 2020				Adjusted 2020 with Bridge			
	AM Peak		PM Peak		AM Peak		PM Peak	
	Delay	LOS	Delay	LOS	Delay	LOS	Delay	LOS
1. Washington St/Petaluma Blvd	48.3	D	46.4	D	47.1	D	47.0	D
2. E Washington St/Lakeville St	48.1	D	47.8	D	47.6	D	48.7	D
3. E Washington St/Payran St	41.9	D	31.4	C	42.0	D	31.9	C
4. D St/Petaluma Blvd S	46.6	D	48.3	D	53.7	D	49.3	D
5. D St/1st St	20.8	C	27.3	C	22.3	C	26.1	C
6. E D St/Lakeville St	**58.6**	**E**	44.0	D	**64.5**	**E**	54.3	D
7. Caulfield Ln/Lakeville St	24.6	C	34.6	C	**72.9**	**E**	**68.1**	**E**
Lane Reassignment, Phase Changes					41.7	D	41.3	D
8. Caulfield Ln/Payran St	14.1	B	38.7	D	13.4	B	37.8	D
9. Petaluma Blvd S/Crystal Ln	5.3	A	7.3	A	8.6	A	12.3	B

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service; **Bold** text = deficient operation; Shaded cells = conditions with recommended improvements

With the Caulfield Bridge, the delay would increase for some intersections and decrease for others. The *General Plan* does not consider a change in delay to be adverse if the resulting operation is LOS D or better, which applies to most of the study intersections. East D Street/Lakeville Street, which operates at LOS E during the a.m. peak hour under Adjusted 2020 conditions, would continue to operate at LOS E under Adjusted 2020 with Bridge conditions. The *General Plan* specifies that a change in delay is not adverse if the intersection would operate at LOS E without the project and would continue to operate at LOS E with the project, precluding an adverse impact in this instance.

The construction of the bridge and ensuing shifts in travel patterns would result in an increase in delay at Caulfield Lane/Lakeville Street, deteriorating operation from LOS C to LOS E. This would represent an adverse effect as it would bring the delay from within the standard of LOS D or better, to worse than the standard.

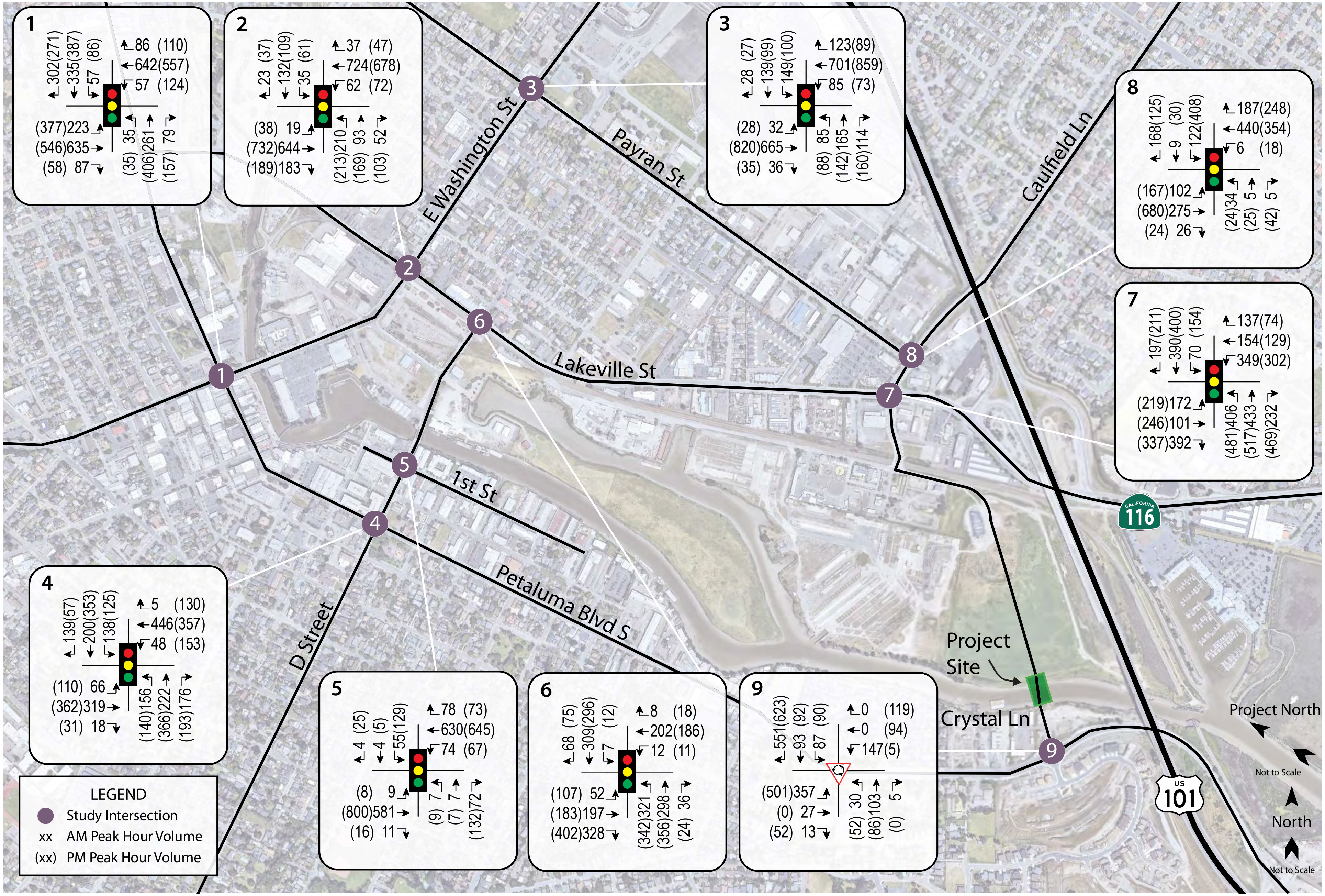

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Figure 4 – Adjusted 2020 With Bridge Traffic Volumes

There are a few modifications that together could mitigate this adverse effect. The westbound direction is currently striped with a left-turn lane, a through/left-turn shared lane, a bicycle lane, and a right-turn lane. Restriping this as a left-turn lane, a second left-turn lane, and a through/right-turn lane, essentially moving the through movement from the middle lane to the rightmost lane, would better serve the anticipated Adjusted 2020 with Bridge volumes. The bicycle lane would need to be moved to the shoulder, similar to the existing design for the eastbound direction. In the eastbound direction, the approach is currently striped as a left-turn lane, a through lane, a through/right-turn shared lane, and a bicycle lane. By restriping the through/right-turn shared lane as just a right-turn lane, the anticipated increase in eastbound right-turn movements could be better accommodated. The eastbound bicycle lane would need to be restriped to be between the new right-turn lane and through lane. With these changes to the lane assignments the eastbound-westbound split phasing could be removed and the two Caulfield Lane approaches could operate simultaneously with protected left-turn phasing. The last change proposed is to add an eastbound right-turn overlap phase to enable drivers to continue without stopping while the northbound left-turn phase is active. With these modifications combined, operation for the intersection of Caulfield Lane/Lakeville Street would be expected to improve to an acceptable LOS D.

Finding – The study intersections are expected to operate acceptably under either Adjusted 2020 or Adjusted 2020 with Bridge conditions, except for two locations. East D Street/Lakeville Street would operate at an unacceptable LOS E during the a.m. peak hour with or without the Caulfield Bridge, precluding an adverse impact. Caulfield Lane/Lakeville Street would degrade from an acceptable LOS C under Adjusted 2020 conditions to an unacceptable LOS E under Adjusted 2020 with Bridge conditions. This adverse effect could be mitigated with changes to the lane assignments and signal phasing.

Recommendation – To achieve acceptable operation at Caulfield Lane/Lakeville Street it is recommended that the westbound through movement be reassigned from the middle to the rightmost lane, the eastbound through/right-turn shared lane be reassigned to right-turn movements only, the eastbound and westbound split phasing be converted to protected left-turn phasing, and an overlap phase be added for the eastbound right-turn movement simultaneous with the northbound left-turn phase. With implementation of these changes acceptable LOS D operation would be expected.

Future with Bridge Conditions

Most of the study intersections would operate at the same levels of service under either Future conditions or Future with Bridge conditions, although the Petaluma Boulevard South/Crystal Lane roundabout would degrade from LOS A without the bridge to LOS B under Future with Bridge conditions, and Caulfield Lane/ Lakeville Street would decline from an acceptable LOS C under Future conditions to an unacceptable LOS F under Future with Bridge conditions. The Future with Bridge operating conditions are summarized in Table 6, and volume are presented in Figure 5.

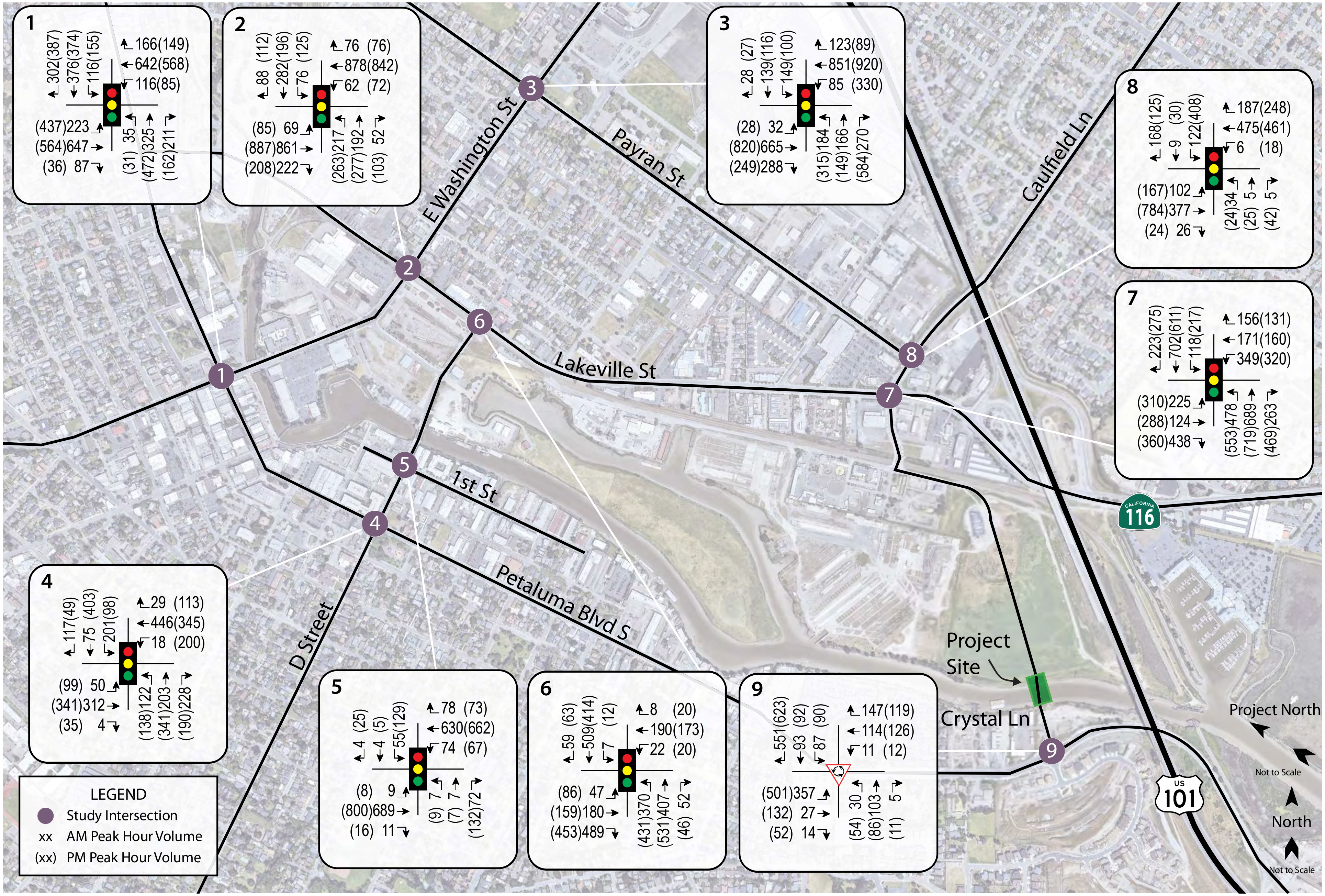

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Figure 5 – Future Traffic Volumes With Bridge

Table 6 – Future and Future with Bridge Peak Hour Intersection Levels of Service								
Study Intersection	**Future Conditions**				**Future with Bridge**			
Mitigation	**AM Peak**		**PM Peak**		**AM Peak**		**PM Peak**	
	Delay	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**	**Delay**	**LOS**
1. Washington St/Petaluma Blvd	44.5	D	**55.7**	**E**	44.5	D	**64.8**	**E**
2. E Washington St/Lakeville St	**78.0**	**E**	**79.1**	**E**	**74.6**	**E**	**70.3**	**E**
3. E Washington St/Payran St	**66.5**	**E**	**135.2**	**F**	**68.1**	**E**	**122.8**	**F**
4. D St/Petaluma Blvd S	44.6	D	43.5	D	41.9	D	46.4	D
5. D St/1st St	21.7	C	21.7	C	20.8	C	21.9	C
6. E D St/Lakeville St	**73.9**	**E**	41.2	D	**57.3**	**E**	49.8	D
7. Caulfield Ln/Lakeville St	32.0	C	51.2	D	**125.5**	**F**	**107.4**	**F**
Lane Reassignment, Phase Changes					**75.3**	**E**	**86.4**	**F**
Above with Roadway Widening					44.1	D	54.9	D
8. Caulfield Ln/Payran St	12.1	B	36.5	D	12.0	B	36.2	D
9. Petaluma Blvd S/Crystal Ln	5.9	A	8.2	A	10.7	B	14.5	B

Notes: Delay is measured in average seconds per vehicle; LOS = Level of Service; **Bold** text = deficient operation; Shaded cells = conditions with recommended improvements

The *General Plan* does not consider a change in delay to be an adverse effect if the level of service would be LOS E with and without the project, which is the case for Washington Street/Petaluma Boulevard during the p.m. peak hour, East Washington Street/Lakeville Street during both peak hours, East Washington Street/Payran Street during the a.m. peak hour, and East D Street/Lakeville Street during the a.m. peak hour. The intersection of East Washington Street/Payran Street would operate at LOS F without or with the Caulfield Bridge. This would not present an adverse effect however as the *General Plan* only considers an adverse effect to occur at a facility operating at LOS F if the project would increase the traffic volumes, and construction of the bridge would result in an estimated 13 fewer peak hour vehicles entering the intersection of East Washington Street/Payran Street.

There is potential for an adverse effect to occur at Caulfield Lane/Lakeville Street, which would degrade from LOS C or D under Future conditions to LOS F under Future with Bridge conditions. By incorporating the modifications described under the Adjusted 2020 with Bridge conditions section, the a.m. peak hour operations could be improved to LOS E while the p.m. peak hour operations would remain at LOS F, though with a 21.0 second reduction in delay.

To improve operations at Caulfield Lane/Lakeville Street to LOS D, further modifications would be required. In addition to the lane reassignment and phasing changes previously described, the westbound approach would need to be widened to provide a second left-turn lane, for two left-turn lanes, one through lane and one right-turn lane. The southbound approach would need to be expanded with a separate right-turn lane, for one left-turn lane, two through lanes, and one right-turn lane. Lastly, all approaches would need to be upgraded with right-turn overlap phasing. With these capacity enhancements, LOS D operations could be achieved, avoiding an adverse impact.

In the *General Plan* DEIR, capacity expansion improvements such as widening to provide additional lanes were not supported as they would conflict with the Plan's multi-modal objectives. As this situation is similar, the City may prefer to focus on the lane reassignment and phasing remediations, and forgo the roadway widening to avoid negative impacts to multi-modal circulation. As these are long-term adverse effects under the Future with Bridge scenario, the City may wish to instead perform a corridor operations study along Lakeville Street and Lakeville

Highway to determine means to improve corridor operations in a way that best balances the *General Plan*'s operational and multi-modal objectives.

Finding – With the exception of Caulfield Lane/Lakeville Street, the study intersections would either operate acceptably under both Future and Future with Bridge conditions, operate at LOS E under both scenarios, or operate at LOS F under both scenarios with a reduction in vehicles expected for Future with Bridge conditions. Operation is projected to degrade from acceptable levels of service to an unacceptable LOS F at Caulfield Lane/Lakeville Street. Non-widening mitigations such as lane reassignment and signal phasing adjustments have the potential to improve operations somewhat, though not to acceptable levels. Widening the approaches to the intersection would enable acceptable operations, though it may conflict with the multi-modal objectives of the *General Plan*.

Recommendation – To improve operations at Caulfield Lane/Lakeville Street under Future with Bridge conditions, improvements beyond those identified for Adjusted 2020 plus Project conditions would be required. In lieu of roadway widening to achieve acceptable LOS D or better operations, it is recommended that a corridor study be conducted for the Lakeville Street and Lakeville Highway corridor to determine how operations along the corridor might be improved through timing modifications and/or adjustments to phasing without conflicting with the *General Plan*'s multi-modal objectives.

Queuing

Under each scenario, the projected maximum queues in left-turn and right-turn pockets at the study intersections were determined using the 95th percentile queue length methodology from the HCM. Summarized in Table 7 are the predicted queue lengths for all turn lanes at the study intersections. Copies of the queue length calculations are contained in Appendix C.

Table 7 – Maximum Turn Lane Queues Exceeding Available Storage

Study Intersection Approach	Available Storage	Maximum Queues							
		AM Peak Hour				PM Peak Hour			
		A	A+B	F	F+B	A	A+B	F	F+B
1. Washington St/Petaluma Blvd									
Westbound Left-Turn	365	109	80	138	106	169	172	78	84
Northbound Left-Turn	115	67	67	88	88	61	66	61	69
Northbound Right-Turn	115	46	30	*128*	*117*	78	79	89	89
Southbound Left-Turn	155	94	97	*199*	*200*	*183*	136	*332*	*315*
Southbound Right-Turn	155	66	66	151	151	70	76	151	150
2. E Washington St/Lakeville St									
Eastbound Left-Turn	45	33	35	*103*	*113*	*68*	*65*	*114*	*121*
Westbound Left-Turn	100	100	100	69	69	89	93	90	90
Northbound Left-Turn	200	*262*	*249*	*363*	*349*	*263*	*249*	*367*	*341*
3. E Washington St/Payran St									
Eastbound Left-Turn	125	56	56	24	26	28	27	21	23
Westbound Left-Turn	130	97	117	*726*	*749*	89	110	*521*	*573*
Northbound Right-Turn	150	48	34	*175*	*174*	63	58	*591*	*622*
Southbound Right-Turn	150	0	0	0	0	0	0	0	0

Table 7 – Maximum Turn Lane Queues Exceeding Available Storage									
Study Intersection Approach	**Available Storage**	**Maximum Queues**							
		AM Peak Hour				**PM Peak Hour**			
		A	**A+B**	**F**	**F+B**	**A**	**A+B**	**F**	**F+B**
4. D St/Petaluma Blvd S									
Eastbound Left-Turn	100	72	89	60	77	*134*	*168*	*118*	*136*
Westbound Right-Turn	40	0	0	8	1	*78*	17	13	7
Northbound Left-Turn	150	114	***245***	68	145	129	***206***	134	***191***
Southbound Left-Turn	175	*241*	*204*	*309*	*277*	*277*	*189*	*224*	133
Southbound Right-Turn	275	0	34	0	35	0	12	0	0
5. D St/1st St									
Eastbound Left-Turn	85	8	9	10	13	12	13	9	11
Westbound Left-Turn	110	86	86	110	102	82	108	86	86
6. E D St/Lakeville St									
Eastbound Left-Turn	225	59	53	*318*	190	*285*	52	*324*	160
Westbound Left-Turn	90	30	27	51	46	32	26	52	46
Northbound Right-Turn	350	15	10	16	17	9	2	16	16
Southbound Right-Turn	215	38	42	0	1	27	48	1	5
7. Caulfield Ln/Lakeville St									
Eastbound Left-Turn	140	63	***270***	112	***371***	98	***344***	*203*	*534*
Westbound Right-Turn	80	32	***365***	*178*	*411*	*162*	*373*	*157*	*420*
Northbound Left-Turn	250	68	***549***	135	***684***	81	***691***	141	***866***
Northbound Right-Turn	130	81	66	91	116	*307*	*234*	*263*	*306*
Southbound Left-Turn	130	*150*	126	*225*	*166*	*428*	*232*	*491*	*340*
8. Caulfield Ln/Payran St									
Eastbound Left-Turn	100	102	102	68	80	173	173	173	173
Westbound Left-Turn	80	10	10	8	9	25	25	25	25

Notes: Maximum Queue based on the HCM calculation for the 95th percentile queue length; all distances are measured in feet; A = Adjusted 2020 conditions; A+B = Adjusted 2020 with Bridge conditions; F = Future conditions; F+B = Future with Bridge conditions; *Italic* text = queue length exceeds available storage; **Bold** text = queue length is within available storage without the project and exceeds available storage with project

The anticipated queue lengths for several locations are expected to exceed the provided storage length. The change in traffic patterns as a result of the Caulfield Bridge would be expected to increase the lengths of some queues and reduce the lengths of others. For several locations where the queue would be contained within the turn pocket without the bridge it would extend outside the turn pocket and into the adjacent through lane with the bridge.

One such location is the northbound left-turn pocket at D Street/Petaluma Boulevard South, where the queue would be shorter than the 150-foot stacking capacity under Adjusted 2020 and Future conditions for both the a.m. and p.m. peak hours. This queue would exceed 150 feet under Adjusted 2020 with Bridge conditions during the a.m. peak hour and Future with Bridge conditions during both peak hours. It is noted that the adjacent through lane would be anticipated to have an average queue during these three peak hours that exceeds 150 feet, which would be achieved before a 95th percentile left-turn queue, thereby blocking access to the left-turn lane. As a result, the excess queue in the turn lane would not affect traffic operation and therefore not represent an adverse effect.

At Caulfield Lane/Lakeville Street, the eastbound left-turn queue would be contained within the 140-foot turn pocket under Adjusted 2020 conditions for both peak hours and Future conditions during the a.m. peak hour, but would extend beyond the available stacking length under Adjusted 2020 with Bridge a.m. and p.m. peak hour volumes as well as during the a.m. peak hour under Future with Bridge volumes. The average queue for the adjacent through lane would extend beyond 140 feet under Adjusted 2020 with Bridge conditions for the p.m. peak hour and under Future 2020 with Bridge conditions for the a.m. peak hour, which would likely occur before the 95th percentile left-turn queue and therefore block access to the turn lane and preclude an adverse effect. For the Adjusted 2020 with Bridge a.m. peak hour, construction of the lane reassignment and phasing alteration recommended to achieve acceptable operation under Adjusted 2020 with Bridge volumes would reduce the eastbound left-turn queue to 212 feet. It is therefore recommended that, in addition to implementing these changes, the turn lane should be extended by 75 feet to 215 feet by modifying the median and adjacent 145-foot westbound left-turn lane, which serves a private driveway and could therefore likely be reduced to 70 feet to accommodate the longer eastbound left-turn lane at Caulfield Lane/Lakeville Street.

The westbound right-turn queue at Caulfield Lane/Lakeville Street, which could be contained in the 80-foot turn lane under Adjusted 2020 a.m. peak hour volumes, would extend well beyond this length under Adjusted 2020 with Bridge volumes. Construction of the previously recommended improvements would include moving the westbound through movement to the right-turn lane, which would eliminate this conflict. Alternatively, construction of the roadway widening identified to address operational deficiencies under Future plus Bridge volumes would reduce the queue to 38 feet. Alleviation of the unacceptable traffic operations at Caulfield Lane/Lakeville Street would also eliminate the queue overage anticipated for the westbound right-turn lane, avoiding an adverse effect.

The northbound left-turn lane at Caulfield Lane/Lakeville Street provides 250 feet of stacking capacity, which would contain the anticipated queue lengths under Adjusted 2020 and Future conditions without the bridge. Upon construction of the bridge and creation of a new link between Lakeville Highway and southwest Petaluma, the anticipated northbound left-turn volume would grow substantially for all study scenarios. As there is approximately 520 feet between the stop bars for the intersections of Caulfield Lane/Lakeville Street and US 101 South Ramps/Lakeville Street which is the next intersection to the south, extending the northbound left-turn lane at Caulfield Lane/Lakeville Street to contain the entire queue length is not achievable. Widening Lakeville Street with additional turn lanes may contain the queue but would conflict with the *General Plan*'s multi-modal objectives. Therefore, the increase in queue length in the northbound left-turn lane would present an adverse effect.

Finding – The queue lengths at several locations would exceed the available capacity with changes to travel patterns anticipated as a result of the Caulfield Bridge. The queue lengths would be contained within the turn pocket without the bridge but exceed capacity upon completion of the bridge for the northbound left-turn lane at D Street/Petaluma Boulevard South, although the adjacent through lane would have a longer average queue, preventing an adverse effect. For the eastbound left-turn lane at Caulfield Lane/Lakeville Street, the lane reassignment and phase change modifications recommended to achieve acceptable operation under Adjusted 2020 with Bridge conditions would reduce the maximum relevant queue to 212 feet, which is greater than the 140-foot storage length but could be accommodated with modifications to the median. The queue overage anticipated for the westbound right-turn lane at Caulfield Lane/Lakeville Street would be eliminated with adoption of the recommended intersection modifications detailed for Adjusted 2020 with Bridge or Future with Bridge operations. The queue in the northbound left-turn lane at Caulfield Lane/Lakeville Street would substantially increase upon completion of the bridge, presenting an adverse effect.

Recommendation – It is recommended that the eastbound left-turn lane at Caulfield Lane/Lakeville Street be expanded by 75 feet from 140 feet to 215 feet. This could be accomplished by modifying the median to take pavement surface area from the adjacent westbound left-turn lane into the private driveway west of the intersection, to reduce it from 145 feet to 70 feet. Additionally, it is recommended that the improvements identified to address operational deficiencies under Adjusted 2020 with Bridge volumes be adopted for the intersection of Caulfield Lane/Lakeville Street to prevent an adverse impact for the westbound right-turn lane.

Alternative Modes

Pedestrian Facilities

The bridge would provide a critical link for pedestrians across the Petaluma River, as the nearest alternative crossing is the East D Street bridge one mile to the north. There is no river crossing accessible to pedestrians to the south.

Recommendation – It is recommended that the bridge include pedestrian facilities such as sidewalks that connect to the existing network, such as the sidewalks on Caulfield Lane and Petaluma Boulevard South.

Bicycle Facilities

Caulfield Lane is included in the *Countywide Bicycle and Pedestrian Master Plan, 2014 Update*, SCTA, 2014, as a Class II bicycle lane facility, including between Lakeville Street and Petaluma Boulevard South which includes the proposed bridge. Similar to with pedestrian facilities, the nearest bicycle-assessable crossing of the Petaluma River is East D Street one mile to the north, and there are no facilities available to bicyclists to the south.

Recommendation – It is recommended that the bridge include Class II bicycle lanes per the *Master Plan*.

Railroad Facilities

Construction of the bridge would increase pedestrian and bicycle activity along the Caulfield Lane corridor, including across the Sonoma-Marin Area Rail Transit (SMART) at-grade crossing on Caulfield Lane. The crossing includes tactile domes and "watch for trains" sidewalk markings for pedestrians. Other SMART at-grade crossings include striping to mark the continuation of the sidewalk across the tracks, which could clarify the preferred crossing area for the Caulfield Lane crossing. Other SMART at-grade crossings also typically include flashing-light signals on all four quadrants although the Caulfield Lane crossing does not have a signal on the northeast corner. Adding a flashing-light signal on the northeast corner would improve visibility to southbound pedestrians approaching the crossing from the east side of Caulfield Lane. Currently, the flashing-light signals are oriented north-south, which may be difficult to observe for a pedestrian arriving from the east on the sidewalk on the north side of Hopper Street.

Recommendation – It is recommended that the SMART at-grade crossing on Caulfield Lane be upgraded with striping to denote the continuation of the sidewalks across the tracks, installation of a flashing-light signal on the northeast corner, and installation of flashing-light signals oriented east-west to raise visibility for pedestrians arriving from the east.

Vehicle Miles Traveled

To determine the relative vehicle miles traveled (VMT) associated with constructing the Caulfield Bridge, the SCTA gravity demand model was consulted. In addition to the Caulfield Bridge and Extension project, the City's long-range plans include extension of Rainier Avenue over US 101 and the Petaluma River to Petaluma Boulevard North. This project could include ramps connecting Rainier Avenue to US 101 or to just cross over the freeway without ramps. Together, four models were assessed: 2040 No Bridges, 2040 with Caulfield Bridge, 2040 with Rainier Bridge, and 2040 with Rainier Bridge and Ramps. It is important to note that the Caulfield Bridge model does not include the Rainier Bridge, and neither of the Rainier Bridge models includes the Caulfield Bridge.

As these proposed bridges would impact regional travel by providing alternative routes or additional access to US 101, the VMT for the entire model (consisting of the entire County of Sonoma) was sampled. The construction of the Caulfield Bridge would reduce the trip lengths of existing trips in the area crossing the Petaluma River through either downtown Petaluma or via US 101 but might induce new trips by providing additional vehicular capacity. The addition of the Caulfield Bridge would be expected to reduce VMT by 3,654 miles, or 0.022 percent.

The construction of the Rainier Bridge would decrease the distance traveled for trips between the east and west side of US 101 that would need to divert to Corona Road or East Washington Street. Without ramps, the Rainier Bridge would result in an estimated decrease of 7,158 miles or 0.042 percent compared to conditions without either bridge. With ramps, the additional freeway access would be expected to induce demand and result in an increase in the VMT of 11,133 miles or 0.066 percent. These results are summarized in Table 8.

Table 8 – VMT Estimates

Scenario	Total VMT	Difference
2040 (No Bridges)	16,854.427	n/a
2040 with Caulfield Bridge	16,850,773	-3.654 (-0.022%)
2040 with Rainier Bridge (no ramps)	16,847,269	-7,158 (-0.042%)
2040 with Rainier Bridge and Ramps	16,865,560	+11,133 (+0.066%)

Note: Changes to VMT associated with each of the three "build" scenarios reflects difference with the "no bridges" result.

Conclusions and Recommendations

Conclusions

- Under Adjusted 2020 volumes that were developed to reflect conditions without the pandemic, eight of the study intersections would be expected to operate acceptably during both peak hours, while the intersection of East D Street/Lakeville Street would operate at an unacceptable LOS E during the a.m. peak hour. However, the *General Plan* DEIR states that capacity expansion for this location is not possible without conflicting with the *General Plan*'s multi-modal objectives, so this operation is considered acceptable.

- Under Future volumes, in addition to LOS E operations at East D Street/Lakeville Street, other intersections would degrade to an unacceptable level including LOS E operation at Washington Street/Petaluma Boulevard for the p.m. peak hour, LOS E at East Washington Street/Lakeville Street for both peak hours, LOS E at East Washington Street/Payran Street during the a.m. peak hour, and LOS F at East Washington Street/Payran Street during the p.m. peak hour.

- Under Adjusted 2020 with Bridge volumes, seven of the study intersections would continue to operate acceptably while the intersection of East D Street/Lakeville Street would continue to operate at LOS E during the a.m. peak hour. Under the *General Plan* policy the LOS E operation at East D Street/Lakeville Street would be acceptable as the intersection would continue to operate at LOS E. For Caulfield Lane/Lakeville Street, there is the potential for an adverse effect as the LOS C operation during both peak hours under both Adjusted 2020 peak volumes would degrade to LOS E under Adjusted 2020 with Bridge volumes. Operation at an acceptable LOS D could be achieved through reassigning several lanes on Caulfield Lane and altering the signal phasing, including adding an eastbound right-turn overlap phase.

- Under Future with Bridge conditions, each location that would operate at LOS E under Future conditions would continue to operate at LOS E. East Washington Street/Payran Street would operate at LOS F during the p.m. peak hour under Future conditions without or with the bridge. As there would be a slight decrease in traffic under Future with Bridge conditions, the change in traffic volumes due to the project would be acceptable. For Caulfield Lane/Lakeville Street, the LOS C and LOS D operation under Future conditions would degrade to LOS F under Future with Bridge conditions. The recommended improvements for the Adjusted 2020 with Bridge scenario would improve delay, though the a.m. operation would be LOS E and the p.m. operation would still be LOS F. Adding lanes would run counter to the multi-modal objectives of the *General Plan* but would enable LOS D operation during both peak hours.

- Queuing was also assessed, and two locations that would remain within the available storage length without the project would exceed the available storage without the project. The northbound left-turn lane queue at D Street/Petaluma Boulevard South would increase to exceed capacity, but the turn lane would be blocked by the average through movement queue which would extend past the entrance to the turn lane before the left-turn queue would. The 140-foot eastbound left-turn lane at Caulfield Lane/Lakeville Street would endure a maximum queue length of 212 feet under Adjusted 2020 a.m. peak hour conditions with lane assignment and phasing improvements implemented. Extending this turn lane by 75 feet would provide adequate capacity at the expense of capacity in the adjacent westbound left-turn lane. The westbound right-turn queue would extend beyond the turn lane with construction of the bridge, but the conflict would be eliminated with the recommended lane assignment and phasing improvements. The northbound left-turn queue would grow beyond the next upstream intersection and therefore present an adverse effect.

- Pedestrian and bicycle facilities were assessed, and it was determined that this bridge would provide a key link across the Petaluma River for pedestrians and bicyclists. The SMART at-grade crossing on Caulfield Lane includes flashing-light signals, crossing arms, tactile domes, and warning signs for pedestrians, but could be upgraded with sidewalk continuation striping, a flashing-light signal on the northeast corner, and a flashing-light signal oriented east-west for pedestrians arriving from the east.

- VMT was also assessed, and the addition of the Caulfield Bridge or Rainier Bridge (without ramps) would result in a minor decrease to VMT, whereas the Rainier Bridge with ramps to/from US 101 would result in a slight increase to VMT.

Recommendations

- To alleviate the unacceptable operations and a queue overage in the westbound right-turn lane at Caulfield Lane/Lakeville Street under the Adjusted 2020 with Bridge scenario, it is recommended that the westbound through movement be reassigned from the middle lane to the rightmost lane and the eastbound through/right-turn shared lane be reassigned to just right-turn movements. It is further recommended that the split phasing on Caulfield Lane be replaced with simultaneous through movement and protected left-turn phasing. Finally, it is also recommended that a right-turn overlap phase be added for the eastbound right-turn movement concurrent with the northbound left-turn phase.

- The above recommendation would improve, but not fully abate the unacceptable operations at Caulfield Lane/Lakeville Street under Future with Bridge conditions. In lieu of widening the approaches to achieve LOS D or better operation, it is recommended that a corridor study be conducted for the Lakeville Street and Lakeville Highway corridor to assess how operations might be improved without conflicting with the *General Plan*'s multi-modal objectives.

- It is recommended that the 140-foot eastbound left-turn lane at Caulfield Lane/Lakeville Street be extended by 75 feet to 215 feet in total length. This could be accommodated through modifying the median to take 75 feet from the adjacent 145-foot westbound left-turn lane which serves a driveway.

- It is recommended that the bridge include sidewalks that connect to the existing sidewalks on Caulfield Lane and Petaluma Boulevard South, and Class II bicycle lanes.

- It is recommended that the SMART at-grade crossing on Caulfield Lane be upgraded with striping to mark the continuation of the sidewalks across the tracks, a flashing-light signal on the northeast corner, and a flashing-light signal oriented east-west.

Study Participants and References

Study Participants

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PET230

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Appendix A

Collision Rate Calculations

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Intersection Collision Rate Calculations

PET230

Intersection # 1: Washington Street & Petaluma Boulevard
Date of Count: Thursday, August 29, 2019

Number of Collisions: 31
Number of Injuries: 12
Number of Fatalities: 0
ADT: 29400
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{31 \times 1{,}000{,}000}{29{,}400 \times 365 \times 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.58 c/mve**	**0.0%**	**38.7%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection # 2: E Washington Street & Lakeville Street
Date of Count: Wednesday, May 16, 2018

Number of Collisions: 22
Number of Injuries: 4
Number of Fatalities: 0
ADT: 25500
Start Date: January 1, 2017
End Date: December 31, 2019
Number of Years: 3

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{22 \times 1{,}000{,}000}{25{,}500 \times 365 \times 3}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.79 c/mve**	**0.0%**	**18.2%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection Collision Rate Calculaions

PET230

Intersection # 3: E Washington Street & Payran Street

Date of Count: Wednesday, January 23, 2019

Number of Collisions: 31
Number of Injuries: 16
Number of Fatalities: 0
ADT: 25200
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{31 \times 1{,}000{,}000}{25{,}200 \times 365 \times 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.67 c/mve**	**0.0%**	**51.6%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection # 4: D Street & Petaluma Boulevard South

Date of Count: Wednesday, May 29, 2019

Number of Collisions: 19
Number of Injuries: 7
Number of Fatalities: 0
ADT: 20100
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{19 \times 1{,}000{,}000}{20{,}100 \times 365 \times 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.52 c/mve**	**0.0%**	**36.8%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection Collision Rate Calculaions

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Intersection # 5: D Street & 1st Street

Date of Count: Tuesday, September 29, 2015

Number of Collisions: 7
Number of Injuries: 3
Number of Fatalities: 0
ADT: 19600
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{7 \quad x \quad 1{,}000{,}000}{19{,}600 \quad x \quad 365 \quad x \quad 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.20 c/mve**	**0.0%**	**42.9%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection # 6: D St & Lakeville Street

Date of Count: Saturday, May 26, 2018

Number of Collisions: 14
Number of Injuries: 5
Number of Fatalities: 0
ADT: 21500
Start Date: January 1, 2017
End Date: December 31, 2019
Number of Years: 3

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{14 \quad x \quad 1{,}000{,}000}{21{,}500 \quad x \quad 365 \quad x \quad 3}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.59 c/mve**	**0.0%**	**35.7%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection Collision Rate Calculaions

PET230

Intersection # 7: Caulfield Lane & Lakeville Street

Date of Count: Wednesday, May 16, 2018

Number of Collisions: 18
Number of Injuries: 6
Number of Fatalities: 0
ADT: 24500
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{18 \quad x \quad 1{,}000{,}000}{24{,}500 \quad x \quad 365 \quad x \quad 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.40 c/mve**	**0.0%**	**33.3%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection # 8: Caulfield Lane & Payran Street

Date of Count: Adjusted 2018 from 8/4/2020 Counts

Number of Collisions: 6
Number of Injuries: 4
Number of Fatalities: 0
ADT: 19800
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Four-Legged
Control Type: Signals
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{6 \quad x \quad 1{,}000{,}000}{19{,}800 \quad x \quad 365 \quad x \quad 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**0.17 c/mve**	**0.0%**	**66.7%**
Statewide Average*	**0.24 c/mve**	**0.5%**	**44.6%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

Intersection Collision Rate Calculaions

PET230

Intersection # 9: Petaluma Boulevard S & Crystal Lane
Date of Count: Adjusted 2018 from 8/4/2020 Counts

Number of Collisions: 25
Number of Injuries: 4
Number of Fatalities: 0
ADT: 9700
Start Date: January 1, 2015
End Date: December 31, 2019
Number of Years: 5

Intersection Type: Other
Control Type: Stop & Yield Controls
Area: Urban

$$\text{collision rate} = \frac{\text{Number of Collisions x 1 Million}}{\text{ADT x 365 Days per Year x Number of Years}}$$

$$\text{collision rate} = \frac{25 \times 1{,}000{,}000}{9{,}700 \times 365 \times 5}$$

	Collision Rate	Fatality Rate	Injury Rate
Study Intersection	**1.41 c/mve**	**0.0%**	**16.0%**
Statewide Average*	**0.08 c/mve**	**1.0%**	**45.1%**

ADT = average daily total vehicles entering intersection
c/mve = collisions per million vehicles entering intersection
* 2016 Collision Data on California State Highways, Caltrans

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Appendix B

Intersection Level of Service Calculations

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Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	223	642	87	94	642	91	35	272	133	55	347	302
Future Volume (veh/h)	223	642	87	94	642	91	35	272	133	55	347	302
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	253	730	99	107	730	103	40	309	151	62	394	343
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	614	1607	218	132	733	103	51	426	479	80	456	933
Arrive On Green	0.34	0.51	0.51	0.07	0.23	0.23	0.03	0.23	0.23	0.04	0.24	0.24
Sat Flow, veh/h	1781	3144	426	1781	3127	441	1781	1870	1585	1781	1870	1585
Grp Volume(v), veh/h	253	412	417	107	415	418	40	309	151	62	394	343
Grp Sat Flow(s),veh/h/ln	1781	1777	1794	1781	1777	1791	1781	1870	1585	1781	1870	1585
Q Serve(g_s), s	13.6	18.5	18.5	7.4	29.1	29.2	2.8	19.1	9.2	4.3	25.2	2.1
Cycle Q Clear(g_c), s	13.6	18.5	18.5	7.4	29.1	29.2	2.8	19.1	9.2	4.3	25.2	2.1
Prop In Lane	1.00		0.24	1.00		0.25	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	614	908	916	132	416	420	51	426	479	80	456	933
V/C Ratio(X)	0.41	0.45	0.45	0.81	1.00	1.00	0.78	0.72	0.32	0.78	0.86	0.37
Avail Cap(c_a), veh/h	614	908	916	242	416	420	157	521	558	157	527	993
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	31.3	19.5	19.5	57.0	47.8	47.8	60.3	44.6	33.7	59.1	45.3	6.2
Incr Delay (d2), s/veh	0.2	1.6	1.6	4.5	43.0	43.1	9.1	4.7	0.5	6.0	13.4	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.9	8.1	8.2	3.5	17.9	18.1	1.4	9.5	3.7	2.1	13.5	3.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	31.4	21.1	21.1	61.5	90.8	90.9	69.4	49.3	34.2	65.1	58.6	6.5
LnGrp LOS	C	C	C	E	F	F	E	D	C	E	E	A
Approach Vol, veh/h		1082			940			500			799	
Approach Delay, s/veh		23.5			87.5			46.3			36.8	
Approach LOS		C			F			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	13.2	68.5	7.6	35.7	47.7	34.0	9.6	33.7
Change Period (Y+Rc), s	4.0	4.6	4.0	* 5.2	* 4.6	* 4.7	4.0	* 5.2
Max Green Setting (Gmax), s	17.0	44.4	11.0	* 35	* 32	* 29	11.0	* 35
Max Q Clear Time (g_c+I1), s	9.4	20.5	4.8	27.2	15.6	31.2	6.3	21.1
Green Ext Time (p_c), s	0.1	8.3	0.0	3.3	0.3	0.0	0.0	2.9

Intersection Summary	
HCM 6th Ctrl Delay	48.3
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	18	668	204	62	740	28	216	95	52	30	132	27
Future Volume (veh/h)	18	668	204	62	740	28	216	95	52	30	132	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	1.00		0.97	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	19	718	215	67	796	27	232	102	51	32	142	29
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	455	1178	353	129	918	31	271	177	89	41	182	37
Arrive On Green	0.26	0.46	0.46	0.07	0.28	0.28	0.05	0.05	0.05	0.15	0.15	0.15
Sat Flow, veh/h	1781	2543	761	1781	3335	113	1697	1109	554	270	1198	245
Grp Volume(v), veh/h	19	478	455	67	404	419	232	0	153	203	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1612	1781	1692	1756	1697	0	1663	1713	0	0
Q Serve(g_s), s	1.0	26.4	26.4	4.5	28.4	28.4	17.0	0.0	11.2	14.3	0.0	0.0
Cycle Q Clear(g_c), s	1.0	26.4	26.4	4.5	28.4	28.4	17.0	0.0	11.2	14.3	0.0	0.0
Prop In Lane	1.00		0.47	1.00		0.06	1.00		0.33	0.16		0.14
Lane Grp Cap(c), veh/h	455	784	747	129	466	483	271	0	266	260	0	0
V/C Ratio(X)	0.04	0.61	0.61	0.52	0.87	0.87	0.86	0.00	0.58	0.78	0.00	0.00
Avail Cap(c_a), veh/h	455	784	747	157	466	483	394	0	386	407	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.95	0.00	0.95	1.00	0.00	0.00
Uniform Delay (d), s/veh	35.0	25.1	25.1	55.9	43.1	43.1	57.8	0.0	55.1	51.0	0.0	0.0
Incr Delay (d2), s/veh	0.0	3.5	3.7	1.2	19.1	18.6	8.1	0.0	0.7	1.9	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.4	11.4	10.9	2.1	14.3	14.8	8.4	0.0	5.1	6.3	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	35.0	28.6	28.8	57.1	62.2	61.7	65.9	0.0	55.8	52.9	0.0	0.0
LnGrp LOS	D	C	C	E	E	E	E	A	E	D	A	A
Approach Vol, veh/h		952			890			385			203	
Approach Delay, s/veh		28.8			61.6			61.9			52.9	
Approach LOS		C			E			E			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	13.0	62.7		24.3	36.7	39.0		25.0
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0
Max Green Setting (Gmax), s	11.0	* 36		* 30	13.0	* 34		29.0
Max Q Clear Time (g_c+I1), s	6.5	28.4		16.3	3.0	30.4		19.0
Green Ext Time (p_c), s	0.0	2.9		0.6	0.0	1.4		0.7

Intersection Summary	
HCM 6th Ctrl Delay	48.1
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

HCM 6th Signalized Intersection Summary
3: Payran St & E Washington St

10/02/2020

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	665	54	67	741	123	69	165	130	149	139	28
Future Volume (veh/h)	32	665	54	67	741	123	69	165	130	149	139	28
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.99	1.00		0.99	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	34	715	58	72	797	132	74	177	140	160	149	30
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	93	930	75	358	1336	221	89	212	255	178	166	293
Arrive On Green	0.05	0.28	0.28	0.20	0.44	0.44	0.16	0.16	0.16	0.19	0.19	0.19
Sat Flow, veh/h	1781	3327	270	1781	3046	504	543	1300	1568	944	879	1549
Grp Volume(v), veh/h	34	382	391	72	465	464	251	0	140	309	0	30
Grp Sat Flow(s),veh/h/ln	1781	1777	1820	1781	1777	1773	1843	0	1568	1823	0	1549
Q Serve(g_s), s	2.1	22.1	22.1	3.8	22.3	22.3	14.8	0.0	9.2	18.5	0.0	1.8
Cycle Q Clear(g_c), s	2.1	22.1	22.1	3.8	22.3	22.3	14.8	0.0	9.2	18.5	0.0	1.8
Prop In Lane	1.00		0.15	1.00		0.28	0.29		1.00	0.52		1.00
Lane Grp Cap(c), veh/h	93	497	509	358	779	778	300	0	255	344	0	293
V/C Ratio(X)	0.36	0.77	0.77	0.20	0.60	0.60	0.84	0.00	0.55	0.90	0.00	0.10
Avail Cap(c_a), veh/h	183	497	509	358	779	778	451	0	383	396	0	336
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	51.3	37.0	37.0	37.3	23.9	23.9	45.4	0.0	43.1	44.4	0.0	37.6
Incr Delay (d2), s/veh	0.9	10.9	10.7	0.1	3.4	3.4	5.2	0.0	0.7	19.2	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.9	11.0	11.2	1.7	9.9	9.9	7.2	0.0	3.6	10.2	0.0	0.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	52.1	47.9	47.7	37.4	27.3	27.3	50.7	0.0	43.8	63.6	0.0	37.6
LnGrp LOS	D	D	D	D	C	C	D	A	D	E	A	D
Approach Vol, veh/h		807			1001			391			339	
Approach Delay, s/veh		48.0			28.0			48.2			61.3	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	27.3	36.0		25.9	9.4	53.9		22.9
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6
Max Green Setting (Gmax), s	* 12	* 31		* 24	11.5	* 31		27.4
Max Q Clear Time (g_c+I1), s	5.8	24.1		20.5	4.1	24.3		16.8
Green Ext Time (p_c), s	0.0	1.4		0.3	0.0	1.7		0.6

Intersection Summary		
HCM 6th Ctrl Delay		41.9
HCM 6th LOS		D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	54	345	21	115	470	44	80	191	221	160	242	63
Future Volume (veh/h)	54	345	21	115	470	44	80	191	221	160	242	63
Initial Q (Qb), veh	5	0	0	1	0	0	0	0	0	0	3	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	64	406	21	135	553	26	94	225	89	188	285	40
Peak Hour Factor	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	121	462	24	320	710	585	129	328	270	173	381	320
Arrive On Green	0.07	0.29	0.29	0.20	0.43	0.43	0.08	0.19	0.19	0.10	0.22	0.22
Sat Flow, veh/h	1603	1582	82	1603	1683	1387	1603	1683	1386	1603	1683	1388
Grp Volume(v), veh/h	64	0	427	135	553	26	94	225	89	188	285	40
Grp Sat Flow(s),veh/h/ln	1603	0	1664	1603	1683	1387	1603	1683	1386	1603	1683	1388
Q Serve(g_s), s	3.5	0.0	22.0	6.6	25.0	0.6	5.2	11.2	5.0	9.0	14.3	2.1
Cycle Q Clear(g_c), s	3.5	0.0	22.0	6.6	25.0	0.6	5.2	11.2	5.0	9.0	14.3	2.1
Prop In Lane	1.00		0.05	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	121	0	486	320	710	585	129	328	270	173	381	320
V/C Ratio(X)	0.53	0.00	0.88	0.42	0.78	0.04	0.73	0.69	0.33	1.08	0.75	0.12
Avail Cap(c_a), veh/h	143	0	486	325	727	599	143	473	390	160	488	403
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	40.6	0.0	30.3	31.6	22.5	6.1	40.4	33.7	31.2	39.9	32.7	27.5
Incr Delay (d2), s/veh	1.3	0.0	19.6	0.3	5.1	0.0	12.7	3.6	1.0	92.5	5.7	0.2
Initial Q Delay(d3),s/veh	26.1	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	1.8	0.0
%ile BackOfQ(50%),veh/ln	2.7	0.0	11.3	2.7	10.7	0.3	2.5	4.8	1.7	8.0	6.8	0.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	68.0	0.0	50.0	32.0	27.6	6.1	53.2	37.3	32.2	132.3	40.2	27.7
LnGrp LOS	E	A	D	C	C	A	D	D	C	F	D	C
Approach Vol, veh/h		491			714			408			513	
Approach Delay, s/veh		52.3			27.6			39.8			73.0	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	22.9	31.0	11.2	24.9	10.4	43.5	13.9	22.2
Change Period (Y+Rc), s	* 4.6	* 4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7
Max Green Setting (Gmax), s	* 12	* 26	8.0	* 26	8.0	30.4	* 9	* 25
Max Q Clear Time (g_c+I1), s	8.6	24.0	7.2	16.3	5.5	27.0	11.0	13.2
Green Ext Time (p_c), s	0.1	0.5	0.0	1.7	0.0	1.1	0.0	1.7

Intersection Summary	
HCM 6th Ctrl Delay	46.6
HCM 6th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	9	673	11	74	630	78	7	7	72	55	4	4
Future Volume (veh/h)	9	673	11	74	630	78	7	7	72	55	4	4
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	10	740	12	81	692	86	8	8	79	60	4	4
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	474	1264	20	137	805	100	50	18	117	181	12	7
Arrive On Green	0.27	0.69	0.69	0.08	0.49	0.49	0.09	0.09	0.09	0.09	0.09	0.09
Sat Flow, veh/h	1781	1835	30	1781	1631	203	70	201	1342	1208	140	84
Grp Volume(v), veh/h	10	0	752	81	0	778	95	0	0	68	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1865	1781	0	1834	1613	0	0	1432	0	0
Q Serve(g_s), s	0.4	0.0	18.9	4.0	0.0	33.6	1.1	0.0	0.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	0.4	0.0	18.9	4.0	0.0	33.6	5.1	0.0	0.0	3.9	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.11	0.08		0.83	0.88		0.06
Lane Grp Cap(c), veh/h	474	0	1285	137	0	905	184	0	0	200	0	0
V/C Ratio(X)	0.02	0.00	0.59	0.59	0.00	0.86	0.52	0.00	0.00	0.34	0.00	0.00
Avail Cap(c_a), veh/h	474	0	1285	198	0	905	423	0	0	418	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.47	0.00	0.47	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	24.4	0.0	7.3	40.1	0.0	20.1	39.8	0.0	0.0	39.2	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	2.0	1.9	0.0	5.3	2.2	0.0	0.0	1.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	7.2	1.8	0.0	14.8	2.1	0.0	0.0	1.5	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	24.4	0.0	9.3	42.0	0.0	25.4	42.0	0.0	0.0	40.2	0.0	0.0
LnGrp LOS	C	A	A	D	A	C	D	A	A	D	A	A
Approach Vol, veh/h		762			859			95			68	
Approach Delay, s/veh		9.5			26.9			42.0			40.2	
Approach LOS		A			C			D			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	10.9	66.6		12.5	28.5	49.0		12.5
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6
Max Green Setting (Gmax), s	10.0	44.4		22.4	10.0	* 44		21.4
Max Q Clear Time (g_c+I1), s	6.0	20.9		5.9	2.4	35.6		7.1
Green Ext Time (p_c), s	0.1	6.1		0.3	0.0	3.8		0.4

Intersection Summary			
HCM 6th Ctrl Delay			20.8
HCM 6th LOS			C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	45	189	435	14	202	6	455	316	45	6	335	66
Future Volume (veh/h)	45	189	435	14	202	6	455	316	45	6	335	66
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.95	1.00		0.98	1.00		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	50	210	483	16	224	3	506	351	49	7	372	63
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	39	164	745	280	289	4	645	1064	958	32	370	329
Arrive On Green	0.11	0.11	0.11	0.16	0.16	0.16	0.36	0.62	0.62	0.22	0.22	0.22
Sat Flow, veh/h	356	1496	1560	1781	1840	25	1781	1722	1550	11	1701	1511
Grp Volume(v), veh/h	260	0	483	16	0	227	506	351	49	379	0	63
Grp Sat Flow(s),veh/h/ln	1853	0	1560	1781	0	1864	1781	1722	1550	1712	0	1511
Q Serve(g_s), s	13.7	0.0	0.0	1.0	0.0	14.6	31.6	12.2	1.6	9.4	0.0	4.3
Cycle Q Clear(g_c), s	13.7	0.0	0.0	1.0	0.0	14.6	31.6	12.2	1.6	27.2	0.0	4.3
Prop In Lane	0.19		1.00	1.00		0.01	1.00		1.00	0.02		1.00
Lane Grp Cap(c), veh/h	203	0	745	280	0	293	645	1064	958	402	0	329
V/C Ratio(X)	1.28	0.00	0.65	0.06	0.00	0.77	0.78	0.33	0.05	0.94	0.00	0.19
Avail Cap(c_a), veh/h	203	0	745	395	0	413	645	1064	958	402	0	329
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.79	0.00	0.79	1.00	0.00	1.00	1.00	1.00	1.00	0.66	0.00	0.66
Uniform Delay (d), s/veh	55.7	0.0	25.1	44.8	0.0	50.5	35.5	11.5	9.4	49.1	0.0	39.9
Incr Delay (d2), s/veh	152.7	0.0	1.8	0.1	0.0	4.9	5.8	0.8	0.1	24.8	0.0	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	15.1	0.0	11.1	0.4	0.0	7.3	14.7	4.8	0.6	14.6	0.0	1.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	208.3	0.0	26.9	44.9	0.0	55.4	41.3	12.3	9.5	73.9	0.0	40.8
LnGrp LOS	F	A	C	D	A	E	D	B	A	E	A	D
Approach Vol, veh/h		743			243			906			442	
Approach Delay, s/veh		90.4			54.8			28.4			69.2	
Approach LOS		F			D			C			E	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		19.0	50.1	32.0		23.9		82.1				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 14	* 38	* 27		27.7		* 69				
Max Q Clear Time (g_c+I1), s		15.7	33.6	29.2		16.6		14.2				
Green Ext Time (p_c), s		0.0	0.3	0.0		0.8		1.6				
Intersection Summary												
HCM 6th Ctrl Delay			58.6									
HCM 6th LOS			E									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	42	28	42	309	20	246	47	545	164	135	570	14
Future Volume (veh/h)	42	28	42	309	20	246	47	545	164	135	570	14
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	44	29	25	337	0	211	49	568	171	141	594	0
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	247	266	203	613	0	526	182	892	431	290	1092	0
Arrive On Green	0.14	0.14	0.14	0.17	0.00	0.17	0.10	0.27	0.27	0.16	0.33	0.00
Sat Flow, veh/h	1781	1921	1463	3563	0	1557	1781	3272	1580	1781	3358	0
Grp Volume(v), veh/h	44	27	27	337	0	211	49	568	171	141	594	0
Grp Sat Flow(s),veh/h/ln	1781	1777	1607	1781	0	1557	1781	1636	1580	1781	1636	0
Q Serve(g_s), s	1.7	1.0	1.1	6.5	0.0	7.9	1.9	11.6	6.7	5.4	11.2	0.0
Cycle Q Clear(g_c), s	1.7	1.0	1.1	6.5	0.0	7.9	1.9	11.6	6.7	5.4	11.2	0.0
Prop In Lane	1.00		0.91	1.00		1.00	1.00		1.00	1.00		0.00
Lane Grp Cap(c), veh/h	247	246	222	613	0	526	182	892	431	290	1092	0
V/C Ratio(X)	0.18	0.11	0.12	0.55	0.00	0.40	0.27	0.64	0.40	0.49	0.54	0.00
Avail Cap(c_a), veh/h	824	822	744	1413	0	876	353	1947	940	471	1947	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00
Uniform Delay (d), s/veh	28.8	28.5	28.6	28.6	0.0	19.3	31.4	24.2	22.4	28.8	20.5	0.0
Incr Delay (d2), s/veh	0.3	0.1	0.2	0.6	0.0	0.4	0.3	1.1	0.8	0.5	0.6	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.7	0.4	0.4	2.7	0.0	2.8	0.8	4.2	2.4	2.3	4.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	29.0	28.6	28.7	29.2	0.0	19.7	31.7	25.3	23.3	29.2	21.1	0.0
LnGrp LOS	C	C	C	C	A	B	C	C	C	C	C	A
Approach Vol, veh/h		98			548			788			735	
Approach Delay, s/veh		28.8			25.5			25.2			22.7	
Approach LOS		C			C			C			C	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	11.7	30.5		18.1	16.3	25.9		15.3				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	15.0	* 45		* 30	20.0	45.0		35.0				
Max Q Clear Time (g_c+I1), s	3.9	13.2		9.9	7.4	13.6		3.7				
Green Ext Time (p_c), s	0.0	6.2		1.4	0.1	6.7		0.3				
Intersection Summary												
HCM 6th Ctrl Delay			24.6									
HCM 6th LOS			C									

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	102	199	26	6	373	187	34	5	5	122	9	168
Future Volume (veh/h)	102	199	26	6	373	187	34	5	5	122	9	168
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	110	214	19	6	401	164	37	5	0	131	10	179
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	141	1918	169	20	1258	507	221	22	0	368	22	378
Arrive On Green	0.08	0.58	0.58	0.01	0.51	0.51	0.16	0.16	0.00	0.16	0.16	0.16
Sat Flow, veh/h	1781	3304	291	1781	2454	990	598	137	0	1482	134	1550
Grp Volume(v), veh/h	110	114	119	6	289	276	42	0	0	141	0	179
Grp Sat Flow(s),veh/h/ln	1781	1777	1818	1781	1777	1667	735	0	0	1616	0	1550
Q Serve(g_s), s	3.3	1.6	1.6	0.2	5.2	5.3	1.3	0.0	0.0	0.0	0.0	5.4
Cycle Q Clear(g_c), s	3.3	1.6	1.6	0.2	5.2	5.3	5.4	0.0	0.0	4.1	0.0	5.4
Prop In Lane	1.00		0.16	1.00		0.59	0.88		0.00	0.93		1.00
Lane Grp Cap(c), veh/h	141	1031	1055	20	911	854	243	0	0	390	0	378
V/C Ratio(X)	0.78	0.11	0.11	0.30	0.32	0.32	0.17	0.00	0.00	0.36	0.00	0.47
Avail Cap(c_a), veh/h	162	1031	1055	227	911	854	300	0	0	453	0	446
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.95	0.95	0.95	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	24.9	5.2	5.2	27.0	7.8	7.8	22.4	0.0	0.0	21.0	0.0	17.9
Incr Delay (d2), s/veh	15.5	0.2	0.2	3.1	0.9	1.0	0.1	0.0	0.0	0.2	0.0	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.9	0.5	0.5	0.1	1.6	1.6	0.5	0.0	0.0	1.5	0.0	1.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	40.4	5.4	5.4	30.1	8.7	8.8	22.5	0.0	0.0	21.2	0.0	18.2
LnGrp LOS	D	A	A	C	A	A	C	A	A	C	A	B
Approach Vol, veh/h		343			571			42			320	
Approach Delay, s/veh		16.6			9.0			22.5			19.5	
Approach LOS		B			A			C			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	4.6	36.8		13.6	8.3	33.1		13.6				
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6				
Max Green Setting (Gmax), s	7.0	23.1		11.4	5.0	25.1		* 12				
Max Q Clear Time (g_c+I1), s	2.2	3.6		7.4	5.3	7.3		7.4				
Green Ext Time (p_c), s	0.0	1.5		0.3	0.0	3.7		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			14.1
HCM 6th LOS			B

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	5.3
Intersection LOS	A

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	452	284	51	4
Demand Flow Rate, veh/h	461	289	52	4
Vehicles Circulating, veh/h	21	18	451	303
Vehicles Exiting, veh/h	286	485	31	4
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	5.8	4.5	4.8	3.6
Approach LOS	A	A	A	A

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	461	289	52	4
Cap Entry Lane, veh/h	1351	1355	871	1013
Entry HV Adj Factor	0.981	0.982	0.981	1.000
Flow Entry, veh/h	452	284	51	4
Cap Entry, veh/h	1325	1330	854	1013
V/C Ratio	0.341	0.213	0.060	0.004
Control Delay, s/veh	5.8	4.5	4.8	3.6
LOS	A	A	A	A
95th %tile Queue, veh	2	1	0	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	355	566	54	122	562	118	32	385	164	106	375	269
Future Volume (veh/h)	355	566	54	122	562	118	32	385	164	106	375	269
Initial Q (Qb), veh	0	0	0	0	0	0	0	4	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.98	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	382	609	54	131	604	113	34	414	148	114	403	270
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	407	1361	120	157	811	151	43	521	577	138	621	884
Arrive On Green	0.23	0.41	0.41	0.09	0.27	0.27	0.02	0.28	0.28	0.08	0.33	0.33
Sat Flow, veh/h	1781	3297	292	1781	2980	556	1781	1870	1573	1781	1870	1571
Grp Volume(v), veh/h	382	328	335	131	360	357	34	414	148	114	403	270
Grp Sat Flow(s),veh/h/ln	1781	1777	1812	1781	1777	1759	1781	1870	1573	1781	1870	1571
Q Serve(g_s), s	26.3	16.6	16.7	9.0	23.1	23.2	2.4	25.6	8.2	7.9	22.9	11.4
Cycle Q Clear(g_c), s	26.3	16.6	16.7	9.0	23.1	23.2	2.4	25.6	8.2	7.9	22.9	11.4
Prop In Lane	1.00		0.16	1.00		0.32	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	407	733	748	157	483	479	43	521	577	138	621	884
V/C Ratio(X)	0.94	0.45	0.45	0.84	0.74	0.75	0.79	0.80	0.26	0.82	0.65	0.31
Avail Cap(c_a), veh/h	456	733	748	228	483	479	157	521	577	157	621	884
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	47.3	26.4	26.5	56.1	41.5	41.6	60.7	42.3	27.7	56.8	35.6	14.6
Incr Delay (d2), s/veh	24.8	2.0	1.9	11.0	10.0	10.2	11.0	11.9	1.1	23.4	5.2	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.1	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	14.5	7.5	7.7	4.6	11.5	11.5	1.2	14.6	3.3	4.5	11.5	4.3
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	72.1	28.4	28.4	67.1	51.5	51.8	71.6	56.2	28.8	80.2	40.8	15.5
LnGrp LOS	E	C	C	E	D	D	E	E	C	F	D	B
Approach Vol, veh/h		1045			848			596			787	
Approach Delay, s/veh		44.4			54.0			50.3			37.8	
Approach LOS		D			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	15.0	56.3	7.0	46.7	32.6	38.7	13.7	40.0
Change Period (Y+Rc), s	4.0	* 4.7	4.0	* 5.2	4.0	* 4.7	4.0	* 5.2
Max Green Setting (Gmax), s	16.0	* 44	11.0	* 35	32.0	* 29	11.0	* 35
Max Q Clear Time (g_c+I1), s	11.0	18.7	4.4	24.9	28.3	25.2	9.9	27.6
Green Ext Time (p_c), s	0.1	6.5	0.0	3.6	0.3	2.1	0.0	2.5

Intersection Summary	
HCM 6th Ctrl Delay	46.4
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	40	750	180	72	685	42	220	156	103	57	91	42
Future Volume (veh/h)	40	750	180	72	685	42	220	156	103	57	91	42
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	1.00		0.95	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	41	773	177	74	706	40	227	161	102	59	94	42
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	385	1150	263	132	901	51	321	189	120	80	128	57
Arrive On Green	0.22	0.42	0.42	0.07	0.28	0.28	0.06	0.06	0.06	0.16	0.16	0.16
Sat Flow, veh/h	1781	2713	621	1781	3247	184	1697	998	633	506	806	360
Grp Volume(v), veh/h	41	482	468	74	368	378	227	0	263	195	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1641	1781	1692	1739	1697	0	1631	1671	0	0
Q Serve(g_s), s	2.3	28.5	28.5	5.0	24.9	24.9	16.3	0.0	19.8	13.8	0.0	0.0
Cycle Q Clear(g_c), s	2.3	28.5	28.5	5.0	24.9	24.9	16.3	0.0	19.8	13.8	0.0	0.0
Prop In Lane	1.00		0.38	1.00		0.11	1.00		0.39	0.30		0.22
Lane Grp Cap(c), veh/h	385	718	696	132	469	482	321	0	309	265	0	0
V/C Ratio(X)	0.11	0.67	0.67	0.56	0.78	0.78	0.71	0.00	0.85	0.74	0.00	0.00
Avail Cap(c_a), veh/h	385	718	696	158	469	482	369	0	355	414	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.00	0.91	1.00	0.00	0.00
Uniform Delay (d), s/veh	39.0	28.8	28.8	55.4	41.4	41.4	54.7	0.0	56.4	49.7	0.0	0.0
Incr Delay (d2), s/veh	0.0	5.0	5.1	1.4	12.3	12.1	3.5	0.0	13.3	1.5	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	12.6	12.2	2.3	11.9	12.3	7.8	0.0	9.8	5.9	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	39.0	33.7	33.9	56.8	53.7	53.4	58.2	0.0	69.7	51.2	0.0	0.0
LnGrp LOS	D	C	C	E	D	D	E	A	E	D	A	A
Approach Vol, veh/h		991			820			490			195	
Approach Delay, s/veh		34.0			53.8			64.4			51.2	
Approach LOS		C			D			E			D	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	13.2	57.4		24.9	31.6	39.0		28.5				
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0				
Max Green Setting (Gmax), s	11.0	* 36		* 31	13.0	* 34		27.0				
Max Q Clear Time (g_c+I1), s	7.0	30.5		15.8	4.3	26.9		21.8				
Green Ext Time (p_c), s	0.0	2.3		0.6	0.0	2.0		0.7				
Intersection Summary												
HCM 6th Ctrl Delay			47.8									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	28	820	54	55	871	89	63	142	185	100	99	27
Future Volume (veh/h)	28	820	54	55	871	89	63	142	185	100	99	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		1.00	1.00		0.97	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	29	845	56	57	898	92	65	146	191	103	102	28
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	82	1693	112	111	1686	173	91	204	246	123	122	210
Arrive On Green	0.05	0.50	0.50	0.06	0.52	0.52	0.16	0.16	0.16	0.13	0.13	0.13
Sat Flow, veh/h	1781	3377	224	1781	3254	333	567	1275	1535	917	908	1567
Grp Volume(v), veh/h	29	445	456	57	490	500	211	0	191	205	0	28
Grp Sat Flow(s),veh/h/ln	1781	1777	1823	1781	1777	1810	1842	0	1535	1825	0	1567
Q Serve(g_s), s	2.0	20.6	20.6	3.8	22.8	22.8	13.5	0.0	14.8	13.6	0.0	2.0
Cycle Q Clear(g_c), s	2.0	20.6	20.6	3.8	22.8	22.8	13.5	0.0	14.8	13.6	0.0	2.0
Prop In Lane	1.00		0.12	1.00		0.18	0.31		1.00	0.50		1.00
Lane Grp Cap(c), veh/h	82	891	915	111	921	938	295	0	246	245	0	210
V/C Ratio(X)	0.36	0.50	0.50	0.51	0.53	0.53	0.72	0.00	0.78	0.84	0.00	0.13
Avail Cap(c_a), veh/h	165	891	915	151	921	938	407	0	339	358	0	307
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	57.4	20.5	20.5	56.3	19.9	19.9	49.4	0.0	50.0	52.4	0.0	47.3
Incr Delay (d2), s/veh	1.0	2.0	1.9	1.4	2.2	2.2	1.7	0.0	4.8	7.4	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.9	9.0	9.2	1.8	9.9	10.0	6.4	0.0	6.1	6.8	0.0	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	58.3	22.5	22.5	57.7	22.1	22.1	51.1	0.0	54.7	59.8	0.0	47.4
LnGrp LOS	E	C	C	E	C	C	D	A	D	E	A	D
Approach Vol, veh/h		930			1047			402			233	
Approach Delay, s/veh		23.6			24.0			52.8			58.3	
Approach LOS		C			C			D			E	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	11.2	67.0		21.3	9.2	69.0		24.5				
Change Period (Y+Rc), s	3.5	* 4.8		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	10.5	* 44		* 24	11.5	* 43		27.4				
Max Q Clear Time (g_c+I1), s	5.8	22.6		15.6	4.0	24.8		16.8				
Green Ext Time (p_c), s	0.0	2.4		0.4	0.0	2.6		0.6				

Intersection Summary			
HCM 6th Ctrl Delay			31.4
HCM 6th LOS			C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	94	390	16	154	386	179	94	329	223	192	315	56
Future Volume (veh/h)	94	390	16	154	386	179	94	329	223	192	315	56
Initial Q (Qb), veh	4	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	96	398	16	157	394	160	96	336	121	196	321	36
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	128	619	25	181	705	580	117	365	299	219	482	401
Arrive On Green	0.07	0.39	0.39	0.11	0.43	0.43	0.07	0.22	0.22	0.14	0.29	0.29
Sat Flow, veh/h	1603	1604	64	1603	1683	1385	1603	1683	1377	1603	1683	1399
Grp Volume(v), veh/h	96	0	414	157	394	160	96	336	121	196	321	36
Grp Sat Flow(s),veh/h/ln	1603	0	1668	1603	1683	1385	1603	1683	1377	1603	1683	1399
Q Serve(g_s), s	7.3	0.0	25.1	11.9	21.7	5.8	7.3	24.2	7.2	14.9	20.9	2.3
Cycle Q Clear(g_c), s	7.3	0.0	25.1	11.9	21.7	5.8	7.3	24.2	7.2	14.9	20.9	2.3
Prop In Lane	1.00		0.04	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	128	0	644	181	705	580	117	365	299	219	482	401
V/C Ratio(X)	0.75	0.00	0.64	0.87	0.56	0.28	0.82	0.92	0.41	0.89	0.67	0.09
Avail Cap(c_a), veh/h	168	0	644	284	717	590	233	384	314	246	482	401
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	56.4	0.0	31.1	54.1	27.3	9.4	56.7	47.5	24.5	52.6	39.0	32.4
Incr Delay (d2), s/veh	8.3	0.0	4.9	9.8	3.2	1.2	5.3	26.9	1.3	27.6	3.9	0.1
Initial Q Delay(d3),s/veh	28.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.8	0.0	11.1	5.3	9.5	3.1	3.1	12.8	2.5	7.7	9.2	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	92.9	0.0	36.0	63.9	30.5	10.6	62.0	74.4	25.7	80.3	42.9	32.5
LnGrp LOS	F	A	D	E	C	B	E	E	C	F	D	C
Approach Vol, veh/h		510			711			553			553	
Approach Delay, s/veh		46.7			33.4			61.6			55.5	
Approach LOS		D			C			E			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	18.0	52.5	13.0	40.4	13.0	57.5	21.9	31.6
Change Period (Y+Rc), s	4.0	* 4.7	4.0	* 4.9	4.0	* 4.7	* 4.9	* 4.7
Max Green Setting (Gmax), s	22.0	* 37	18.0	* 29	13.0	* 46	* 19	* 28
Max Q Clear Time (g_c+I1), s	13.9	27.1	9.3	22.9	9.3	23.7	16.9	26.2
Green Ext Time (p_c), s	0.1	1.6	0.1	1.4	0.0	2.5	0.1	0.7

Intersection Summary	
HCM 6th Ctrl Delay	48.3
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	800	16	67	662	73	9	7	132	129	5	25
Future Volume (veh/h)	8	800	16	67	662	73	9	7	132	129	5	25
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	9	851	17	71	704	78	10	7	140	137	5	27
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	289	1191	24	105	899	100	40	24	275	206	9	31
Arrive On Green	0.16	0.65	0.65	0.06	0.54	0.54	0.18	0.18	0.18	0.18	0.18	0.18
Sat Flow, veh/h	1781	1827	37	1781	1654	183	50	132	1502	837	52	169
Grp Volume(v), veh/h	9	0	868	71	0	782	157	0	0	169	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1864	1781	0	1837	1685	0	0	1058	0	0
Q Serve(g_s), s	0.5	0.0	37.6	4.8	0.0	41.9	0.0	0.0	0.0	9.1	0.0	0.0
Cycle Q Clear(g_c), s	0.5	0.0	37.6	4.8	0.0	41.9	10.9	0.0	0.0	20.0	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.10	0.06		0.89	0.81		0.16
Lane Grp Cap(c), veh/h	289	0	1215	105	0	999	339	0	0	246	0	0
V/C Ratio(X)	0.03	0.00	0.71	0.68	0.00	0.78	0.46	0.00	0.00	0.69	0.00	0.00
Avail Cap(c_a), veh/h	289	0	1215	158	0	999	463	0	0	348	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.39	0.00	0.39	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	43.7	0.0	14.1	57.2	0.0	22.5	45.9	0.0	0.0	50.7	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	3.6	3.0	0.0	2.5	1.0	0.0	0.0	3.4	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	16.4	2.3	0.0	18.5	4.5	0.0	0.0	5.4	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	43.8	0.0	17.7	60.1	0.0	25.0	46.8	0.0	0.0	54.1	0.0	0.0
LnGrp LOS	D	A	B	E	A	C	D	A	A	D	A	A
Approach Vol, veh/h		877			853			157			169	
Approach Delay, s/veh		18.0			27.9			46.8			54.1	
Approach LOS		B			C			D			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	11.3	85.4		27.3	24.7	72.0		27.3
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6
Max Green Setting (Gmax), s	11.0	67.4		32.4	11.0	* 67		32.4
Max Q Clear Time (g_c+I1), s	6.8	39.6		22.0	2.5	43.9		12.9
Green Ext Time (p_c), s	0.0	8.1		0.7	0.0	6.6		0.9

Intersection Summary	
HCM 6th Ctrl Delay	27.3
HCM 6th LOS	C

Notes
* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	91	176	537	16	184	14	457	370	34	8	288	60
Future Volume (veh/h)	91	176	537	16	184	14	457	370	34	8	288	60
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	97	187	571	17	196	14	486	394	36	9	306	58
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	107	207	830	252	244	17	630	984	903	32	267	278
Arrive On Green	0.29	0.29	0.29	0.14	0.14	0.14	0.35	0.57	0.57	0.18	0.18	0.18
Sat Flow, veh/h	628	1211	1574	1781	1722	123	1781	1722	1580	14	1494	1554
Grp Volume(v), veh/h	284	0	571	17	0	210	486	394	36	315	0	58
Grp Sat Flow(s),veh/h/ln	1839	0	1574	1781	0	1845	1781	1722	1580	1508	0	1554
Q Serve(g_s), s	18.4	0.0	0.0	1.0	0.0	13.7	30.1	15.8	1.2	6.4	0.0	3.9
Cycle Q Clear(g_c), s	18.4	0.0	0.0	1.0	0.0	13.7	30.1	15.8	1.2	22.2	0.0	3.9
Prop In Lane	0.34		1.00	1.00		0.07	1.00		1.00	0.03		1.00
Lane Grp Cap(c), veh/h	314	0	830	252	0	261	630	984	903	300	0	278
V/C Ratio(X)	0.90	0.00	0.69	0.07	0.00	0.80	0.77	0.40	0.04	1.05	0.00	0.21
Avail Cap(c_a), veh/h	322	0	836	398	0	412	630	984	903	300	0	278
HCM Platoon Ratio	1.67	1.67	1.67	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.60	0.00	0.60	1.00	0.00	1.00	1.00	1.00	1.00	0.67	0.00	0.67
Uniform Delay (d), s/veh	43.3	0.0	17.5	46.1	0.0	51.6	35.6	14.8	11.6	51.5	0.0	43.4
Incr Delay (d2), s/veh	18.9	0.0	1.6	0.1	0.0	4.7	5.3	1.2	0.1	56.5	0.0	1.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	9.4	0.0	10.0	0.5	0.0	6.7	13.9	6.4	0.5	14.2	0.0	1.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	62.3	0.0	19.1	46.2	0.0	56.3	40.9	16.0	11.7	108.0	0.0	44.5
LnGrp LOS	E	A	B	D	A	E	D	B	B	F	A	D
Approach Vol, veh/h		855			227			916			373	
Approach Delay, s/veh		33.5			55.5			29.0			98.2	
Approach LOS		C			E			C			F	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		26.5	48.7	27.0		21.9		75.7				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 22	* 34	* 22		27.7		* 60				
Max Q Clear Time (g_c+I1), s		20.4	32.1	24.2		15.7		17.8				
Green Ext Time (p_c), s		0.7	0.1	0.0		0.8		1.7				
Intersection Summary												
HCM 6th Ctrl Delay			44.0									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	68	76	29	230	20	144	52	704	483	296	518	27
Future Volume (veh/h)	68	76	29	230	20	144	52	704	483	296	518	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	72	80	17	257	0	108	55	741	505	312	545	3
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	204	337	69	459	0	502	164	1244	601	340	1600	9
Arrive On Green	0.11	0.11	0.11	0.13	0.00	0.13	0.09	0.38	0.38	0.19	0.48	0.48
Sat Flow, veh/h	1781	2933	605	3563	0	1548	1781	3272	1581	1781	3337	18
Grp Volume(v), veh/h	72	48	49	257	0	108	55	741	505	312	267	281
Grp Sat Flow(s),veh/h/ln	1781	1777	1761	1781	0	1548	1781	1636	1581	1781	1636	1719
Q Serve(g_s), s	3.9	2.5	2.7	7.0	0.0	5.3	3.0	18.8	30.2	17.8	10.5	10.5
Cycle Q Clear(g_c), s	3.9	2.5	2.7	7.0	0.0	5.3	3.0	18.8	30.2	17.8	10.5	10.5
Prop In Lane	1.00		0.34	1.00		1.00	1.00		1.00	1.00		0.01
Lane Grp Cap(c), veh/h	204	204	202	459	0	502	164	1244	601	340	784	824
V/C Ratio(X)	0.35	0.23	0.24	0.56	0.00	0.21	0.34	0.60	0.84	0.92	0.34	0.34
Avail Cap(c_a), veh/h	601	599	594	1030	0	750	257	1419	686	343	784	824
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	42.4	41.8	41.8	42.4	0.0	25.8	44.1	25.8	29.3	41.2	16.8	16.8
Incr Delay (d2), s/veh	0.8	0.4	0.5	0.8	0.0	0.2	0.4	0.7	8.9	27.8	0.4	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.8	1.1	1.2	3.1	0.0	2.0	1.3	7.1	12.3	10.3	3.9	4.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	43.1	42.2	42.3	43.2	0.0	26.0	44.6	26.5	38.1	68.9	17.2	17.2
LnGrp LOS	D	D	D	D	A	C	D	C	D	E	B	B
Approach Vol, veh/h		169			365			1301			860	
Approach Delay, s/veh		42.6			38.1			31.8			35.9	
Approach LOS		D			D			C			D	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	13.5	55.1		18.5	23.8	44.8		16.7				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	15.0	* 45		* 30	20.0	45.0		35.0				
Max Q Clear Time (g_c+I1), s	5.0	12.5		9.0	19.8	32.2		5.9				
Green Ext Time (p_c), s	0.0	5.1		0.9	0.0	7.3		0.6				

Intersection Summary		
HCM 6th Ctrl Delay		34.6
HCM 6th LOS		C

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	167	664	24	18	245	248	24	25	42	408	30	125
Future Volume (veh/h)	167	664	24	18	245	248	24	25	42	408	30	125
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.97	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	169	671	17	18	247	204	24	25	12	412	30	118
Peak Hour Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	204	1549	39	60	667	526	72	64	16	377	20	772
Arrive On Green	0.11	0.44	0.44	0.03	0.36	0.36	0.38	0.38	0.38	0.38	0.38	0.38
Sat Flow, veh/h	1781	3539	90	1781	1866	1471	0	168	41	733	53	1562
Grp Volume(v), veh/h	169	337	351	18	235	216	61	0	0	442	0	118
Grp Sat Flow(s),veh/h/ln	1781	1777	1852	1781	1777	1561	209	0	0	786	0	1562
Q Serve(g_s), s	6.5	9.2	9.2	0.7	6.8	7.2	0.0	0.0	0.0	0.0	0.0	2.9
Cycle Q Clear(g_c), s	6.5	9.2	9.2	0.7	6.8	7.2	26.5	0.0	0.0	26.5	0.0	2.9
Prop In Lane	1.00		0.05	1.00		0.94	0.39		0.20	0.93		1.00
Lane Grp Cap(c), veh/h	204	778	811	60	635	557	151	0	0	397	0	772
V/C Ratio(X)	0.83	0.43	0.43	0.30	0.37	0.39	0.40	0.00	0.00	1.11	0.00	0.15
Avail Cap(c_a), veh/h	204	778	811	204	635	557	151	0	0	397	0	772
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.61	0.61	0.61	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	30.3	13.7	13.7	33.0	16.7	16.8	17.9	0.0	0.0	25.1	0.0	9.7
Incr Delay (d2), s/veh	15.1	1.1	1.0	1.0	1.7	2.0	0.6	0.0	0.0	79.5	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.6	3.7	3.8	0.3	2.8	2.6	0.6	0.0	0.0	15.3	0.0	0.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	45.5	14.7	14.7	34.0	18.3	18.8	18.6	0.0	0.0	104.6	0.0	9.8
LnGrp LOS	D	B	B	C	B	B	B	A	A	F	A	A
Approach Vol, veh/h		857			469			61			560	
Approach Delay, s/veh		20.8			19.2			18.6			84.6	
Approach LOS		C			B			B			F	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	6.4	35.6		31.1	12.0	30.0		31.1				
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6				
Max Green Setting (Gmax), s	8.0	22.1		26.4	8.0	22.1		* 27				
Max Q Clear Time (g_c+I1), s	2.7	11.2		28.5	8.5	9.2		28.5				
Green Ext Time (p_c), s	0.0	3.8		0.0	0.0	2.5		0.0				
Intersection Summary												
HCM 6th Ctrl Delay			38.7									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	7.3
Intersection LOS	A

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	401	694	44	9
Demand Flow Rate, veh/h	409	708	44	9
Vehicles Circulating, veh/h	56	27	381	729
Vehicles Exiting, veh/h	682	398	84	6
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	5.7	8.4	4.3	5.6
Approach LOS	A	A	A	A

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	409	708	44	9
Cap Entry Lane, veh/h	1303	1342	936	656
Entry HV Adj Factor	0.980	0.981	1.000	1.000
Flow Entry, veh/h	401	694	44	9
Cap Entry, veh/h	1277	1316	936	656
V/C Ratio	0.314	0.527	0.047	0.014
Control Delay, s/veh	5.7	8.4	4.3	5.6
LOS	A	A	A	A
95th %tile Queue, veh	1	3	0	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	223	654	87	153	642	171	35	336	265	114	388	302
Future Volume (veh/h)	223	654	87	153	642	171	35	336	265	114	388	302
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	253	743	99	174	730	194	40	382	301	130	441	343
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	376	1339	178	200	891	237	51	436	547	154	562	810
Arrive On Green	0.21	0.42	0.42	0.11	0.32	0.32	0.03	0.23	0.23	0.09	0.30	0.30
Sat Flow, veh/h	1781	3152	420	1781	2777	738	1781	1870	1585	1781	1870	1585
Grp Volume(v), veh/h	253	419	423	174	467	457	40	382	301	130	441	343
Grp Sat Flow(s),veh/h/ln	1781	1777	1795	1781	1777	1738	1781	1870	1585	1781	1870	1585
Q Serve(g_s), s	17.0	23.0	23.1	12.5	31.5	31.5	2.9	25.6	14.0	9.3	28.1	2.1
Cycle Q Clear(g_c), s	17.0	23.0	23.1	12.5	31.5	31.5	2.9	25.6	14.0	9.3	28.1	2.1
Prop In Lane	1.00		0.23	1.00		0.42	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	376	755	763	200	570	557	51	436	547	154	562	810
V/C Ratio(X)	0.67	0.55	0.55	0.87	0.82	0.82	0.78	0.88	0.55	0.84	0.79	0.42
Avail Cap(c_a), veh/h	376	755	763	274	570	557	69	495	597	178	616	856
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	47.2	28.1	28.1	56.8	40.7	40.7	62.7	48.0	18.2	58.5	41.6	10.9
Incr Delay (d2), s/veh	3.8	2.9	2.9	15.7	12.4	12.7	23.5	15.6	1.3	23.5	6.7	0.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	8.0	10.5	10.6	6.5	15.8	15.5	1.7	13.9	5.4	5.3	14.1	4.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	51.0	31.0	31.0	72.5	53.1	53.4	86.2	63.6	19.5	82.0	48.3	11.4
LnGrp LOS	D	C	C	E	D	D	F	E	B	F	D	B
Approach Vol, veh/h		1095			1098			723			914	
Approach Delay, s/veh		35.7			56.3			46.5			39.2	
Approach LOS		D			E			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	18.6	59.8	7.7	43.8	32.0	46.4	16.1	35.5
Change Period (Y+Rc), s	4.0	4.6	4.0	* 4.8	* 4.6	* 4.7	* 4.8	* 5.2
Max Green Setting (Gmax), s	20.0	44.8	5.0	* 43	* 23	* 42	* 13	* 34
Max Q Clear Time (g_c+I1), s	14.5	25.1	4.9	30.1	19.0	33.5	11.3	27.6
Green Ext Time (p_c), s	0.1	7.7	0.0	4.8	0.2	4.7	0.0	2.7

Intersection Summary	
HCM 6th Ctrl Delay	44.5
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	68	885	243	62	894	67	223	194	52	71	282	92
Future Volume (veh/h)	68	885	243	62	894	67	223	194	52	71	282	92
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	73	952	257	67	961	69	240	209	51	76	303	99
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	153	970	261	125	1120	80	248	201	49	71	285	93
Arrive On Green	0.09	0.37	0.37	0.07	0.35	0.35	0.05	0.05	0.05	0.27	0.27	0.27
Sat Flow, veh/h	1781	2611	703	1781	3194	229	1697	1374	335	269	1074	351
Grp Volume(v), veh/h	73	616	593	67	509	521	240	0	260	478	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1622	1781	1692	1731	1697	0	1709	1694	0	0
Q Serve(g_s), s	5.1	46.7	47.1	4.7	36.3	36.3	18.4	0.0	19.0	34.5	0.0	0.0
Cycle Q Clear(g_c), s	5.1	46.7	47.1	4.7	36.3	36.3	18.4	0.0	19.0	34.5	0.0	0.0
Prop In Lane	1.00		0.43	1.00		0.13	1.00		0.20	0.16		0.21
Lane Grp Cap(c), veh/h	153	629	602	125	594	607	248	0	250	450	0	0
V/C Ratio(X)	0.48	0.98	0.98	0.54	0.86	0.86	0.97	0.00	1.04	1.06	0.00	0.00
Avail Cap(c_a), veh/h	164	629	602	137	594	607	248	0	250	450	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.00	0.91	1.00	0.00	0.00
Uniform Delay (d), s/veh	56.6	40.4	40.5	58.4	39.2	39.2	61.6	0.0	61.9	47.8	0.0	0.0
Incr Delay (d2), s/veh	0.8	31.2	33.1	1.3	14.8	14.6	45.3	0.0	65.3	60.3	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.3	24.9	24.3	2.2	17.4	17.8	11.5	0.0	13.3	22.0	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	57.5	71.6	73.6	59.7	54.0	53.8	106.9	0.0	127.2	108.0	0.0	0.0
LnGrp LOS	E	E	E	E	D	D	F	A	F	F	A	A
Approach Vol, veh/h		1282			1097			500			478	
Approach Delay, s/veh		71.7			54.2			117.4			108.0	
Approach LOS		E			D			F			F	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	13.1	53.1		39.8	16.0	50.2		24.0				
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0				
Max Green Setting (Gmax), s	10.0	* 47		* 35	12.0	* 46		19.0				
Max Q Clear Time (g_c+I1), s	6.7	49.1		36.5	7.1	38.3		21.0				
Green Ext Time (p_c), s	0.0	0.0		0.0	0.0	2.8		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			78.0
HCM 6th LOS			E

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	665	306	433	891	123	168	166	286	149	139	28
Future Volume (veh/h)	32	665	306	433	891	123	168	166	286	149	139	28
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.99	1.00		0.99	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	34	715	329	466	958	132	181	178	308	160	149	30
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	87	648	298	697	1962	270	191	188	326	173	161	283
Arrive On Green	0.05	0.27	0.27	0.39	0.63	0.63	0.21	0.21	0.21	0.18	0.18	0.18
Sat Flow, veh/h	1781	2360	1085	1781	3134	432	920	905	1571	944	879	1549
Grp Volume(v), veh/h	34	538	506	466	543	547	359	0	308	309	0	30
Grp Sat Flow(s),veh/h/ln	1781	1777	1668	1781	1777	1789	1824	0	1571	1823	0	1549
Q Serve(g_s), s	2.4	35.7	35.7	28.0	21.4	21.4	25.2	0.0	25.1	21.7	0.0	2.1
Cycle Q Clear(g_c), s	2.4	35.7	35.7	28.0	21.4	21.4	25.2	0.0	25.1	21.7	0.0	2.1
Prop In Lane	1.00		0.65	1.00		0.24	0.50		1.00	0.52		1.00
Lane Grp Cap(c), veh/h	87	488	458	697	1113	1120	379	0	326	333	0	283
V/C Ratio(X)	0.39	1.10	1.10	0.67	0.49	0.49	0.95	0.00	0.94	0.93	0.00	0.11
Avail Cap(c_a), veh/h	123	488	458	697	1113	1120	379	0	326	341	0	290
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	59.9	47.2	47.2	32.6	13.1	13.1	50.8	0.0	50.8	52.3	0.0	44.3
Incr Delay (d2), s/veh	1.1	71.9	73.4	2.0	1.5	1.5	32.6	0.0	34.9	29.8	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.1	25.4	24.0	12.4	8.8	8.8	15.0	0.0	13.1	12.7	0.0	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	61.0	119.1	120.5	34.6	14.6	14.6	83.4	0.0	85.6	82.0	0.0	44.3
LnGrp LOS	E	F	F	C	B	B	F	A	F	F	A	D
Approach Vol, veh/h		1078			1556			667			339	
Approach Delay, s/veh		117.9			20.6			84.4			78.7	
Approach LOS		F			C			F			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	56.4	40.4		28.5	9.9	87.0		31.6				
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	* 26	* 36		* 24	9.0	* 52		27.0				
Max Q Clear Time (g_c+I1), s	30.0	37.7		23.7	4.4	23.4		27.2				
Green Ext Time (p_c), s	0.0	0.0		0.1	0.0	3.2		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			66.5
HCM 6th LOS			E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	38	338	7	85	472	68	46	172	273	223	117	41
Future Volume (veh/h)	38	338	7	85	472	68	46	172	273	223	117	41
Initial Q (Qb), veh	5	0	0	1	0	0	0	0	0	0	3	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	45	398	4	100	555	54	54	202	150	262	138	14
Peak Hour Factor	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	100	525	5	243	695	572	100	305	251	264	483	406
Arrive On Green	0.06	0.32	0.32	0.15	0.42	0.42	0.06	0.18	0.18	0.16	0.29	0.29
Sat Flow, veh/h	1603	1663	17	1603	1683	1386	1603	1683	1383	1603	1683	1397
Grp Volume(v), veh/h	45	0	402	100	555	54	54	202	150	262	138	14
Grp Sat Flow(s),veh/h/ln	1603	0	1679	1603	1683	1386	1603	1683	1383	1603	1683	1397
Q Serve(g_s), s	2.7	0.0	21.5	5.6	28.5	1.3	3.3	11.2	10.0	16.0	6.4	0.7
Cycle Q Clear(g_c), s	2.7	0.0	21.5	5.6	28.5	1.3	3.3	11.2	10.0	16.0	6.4	0.7
Prop In Lane	1.00		0.01	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	100	0	531	243	695	572	100	305	251	264	483	406
V/C Ratio(X)	0.45	0.00	0.76	0.41	0.80	0.09	0.54	0.66	0.60	0.99	0.29	0.03
Avail Cap(c_a), veh/h	128	0	531	247	707	582	144	438	360	257	552	458
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	45.8	0.0	30.8	38.5	25.7	5.6	45.5	38.1	37.6	41.6	27.9	25.4
Incr Delay (d2), s/veh	1.2	0.0	9.7	0.4	6.2	0.1	1.7	3.5	3.2	53.8	0.5	0.0
Initial Q Delay(d3),s/veh	32.4	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.4	0.0
%ile BackOfQ(50%),veh/ln	2.5	0.0	10.1	2.4	12.5	0.7	1.3	4.9	3.6	10.2	3.0	0.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	79.4	0.0	40.5	39.1	31.9	5.6	47.2	41.6	40.8	95.4	28.8	25.5
LnGrp LOS	E	A	D	D	C	A	D	D	D	F	C	C
Approach Vol, veh/h		447			709			406			414	
Approach Delay, s/veh		44.4			30.9			42.1			70.8	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	20.0	36.3	10.2	33.5	9.7	46.6	20.9	22.8
Change Period (Y+Rc), s	* 4.6	* 4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7
Max Green Setting (Gmax), s	* 9	* 32	9.0	* 33	8.0	32.7	* 16	* 26
Max Q Clear Time (g_c+I1), s	7.6	23.5	5.3	8.4	4.7	30.5	18.0	13.2
Green Ext Time (p_c), s	0.0	1.3	0.0	1.1	0.0	0.8	0.0	1.9

Intersection Summary	
HCM 6th Ctrl Delay	44.6
HCM 6th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	9	781	11	74	630	78	7	7	72	55	4	4
Future Volume (veh/h)	9	781	11	74	630	78	7	7	72	55	4	4
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	10	858	12	81	692	86	8	8	79	60	4	4
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	344	1078	15	361	972	121	45	16	108	159	11	6
Arrive On Green	0.19	0.59	0.59	0.20	0.60	0.60	0.08	0.08	0.08	0.08	0.08	0.08
Sat Flow, veh/h	1781	1840	26	1781	1631	203	75	202	1368	1149	135	80
Grp Volume(v), veh/h	10	0	870	81	0	778	95	0	0	68	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1866	1781	0	1834	1645	0	0	1364	0	0
Q Serve(g_s), s	0.5	0.0	36.2	3.8	0.0	29.8	0.8	0.0	0.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	0.5	0.0	36.2	3.8	0.0	29.8	5.6	0.0	0.0	4.8	0.0	0.0
Prop In Lane	1.00		0.01	1.00		0.11	0.08		0.83	0.88		0.06
Lane Grp Cap(c), veh/h	344	0	1093	361	0	1093	169	0	0	176	0	0
V/C Ratio(X)	0.03	0.00	0.80	0.22	0.00	0.71	0.56	0.00	0.00	0.39	0.00	0.00
Avail Cap(c_a), veh/h	344	0	1093	361	0	1093	364	0	0	341	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.09	0.00	0.09	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	32.8	0.0	16.1	33.3	0.0	14.2	45.0	0.0	0.0	44.6	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	6.0	0.0	0.0	0.4	2.9	0.0	0.0	1.4	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	16.2	1.7	0.0	11.7	2.4	0.0	0.0	1.7	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	32.8	0.0	22.1	33.3	0.0	14.5	47.9	0.0	0.0	46.0	0.0	0.0
LnGrp LOS	C	A	C	C	A	B	D	A	A	D	A	A
Approach Vol, veh/h		880			859			95			68	
Approach Delay, s/veh		22.2			16.3			47.9			46.0	
Approach LOS		C			B			D			D	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	24.3	63.2		12.5	23.3	64.2		12.5				
Change Period (Y+Rc), s	4.0	4.6		4.6	4.0	4.6		4.6				
Max Green Setting (Gmax), s	8.0	58.6		20.2	7.0	59.6		20.2				
Max Q Clear Time (g_c+I1), s	5.8	38.2		6.8	2.5	31.8		7.6				
Green Ext Time (p_c), s	0.0	7.2		0.2	0.0	6.9		0.3				

Intersection Summary			
HCM 6th Ctrl Delay			21.7
HCM 6th LOS			C

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	40	172	596	24	190	6	504	425	61	6	535	57
Future Volume (veh/h)	40	172	596	24	190	6	504	425	61	6	535	57
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.92	1.00		0.98	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	44	191	662	27	211	3	560	472	67	7	594	53
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	43	187	644	199	205	3	507	1126	1014	30	578	516
Arrive On Green	0.12	0.12	0.12	0.11	0.11	0.11	0.28	0.65	0.65	0.34	0.34	0.34
Sat Flow, veh/h	347	1506	1560	1781	1837	26	1781	1722	1550	6	1708	1526
Grp Volume(v), veh/h	235	0	662	27	0	214	560	472	67	601	0	53
Grp Sat Flow(s),veh/h/ln	1853	0	1560	1781	0	1863	1781	1722	1550	1715	0	1526
Q Serve(g_s), s	16.1	0.0	16.1	1.8	0.0	14.5	37.0	17.0	2.0	14.3	0.0	3.1
Cycle Q Clear(g_c), s	16.1	0.0	16.1	1.8	0.0	14.5	37.0	17.0	2.0	44.0	0.0	3.1
Prop In Lane	0.19		1.00	1.00		0.01	1.00		1.00	0.01		1.00
Lane Grp Cap(c), veh/h	229	0	644	199	0	208	507	1126	1014	608	0	516
V/C Ratio(X)	1.02	0.00	1.03	0.14	0.00	1.03	1.10	0.42	0.07	0.99	0.00	0.10
Avail Cap(c_a), veh/h	229	0	644	199	0	208	507	1126	1014	608	0	516
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.73	0.00	0.73	1.00	0.00	1.00	1.00	1.00	1.00	0.17	0.00	0.17
Uniform Delay (d), s/veh	57.0	0.0	38.5	52.1	0.0	57.8	46.5	10.7	8.1	43.7	0.0	29.5
Incr Delay (d2), s/veh	57.1	0.0	37.5	0.2	0.0	70.4	71.7	1.1	0.1	12.5	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	11.2	0.0	26.8	0.8	0.0	10.9	26.3	6.6	0.7	21.5	0.0	1.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	114.0	0.0	76.0	52.3	0.0	128.2	118.2	11.9	8.3	56.2	0.0	29.5
LnGrp LOS	F	A	F	D	A	F	F	B	A	E	A	C
Approach Vol, veh/h		897			241			1099			654	
Approach Delay, s/veh		86.0			119.7			65.8			54.0	
Approach LOS		F			F			E			D	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		21.4	41.0	48.8		18.8		89.8				
Change Period (Y+Rc), s		* 5.3	4.0	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 16	37.0	* 44		14.5		* 85				
Max Q Clear Time (g_c+I1), s		18.1	39.0	46.0		16.5		19.0				
Green Ext Time (p_c), s		0.0	0.0	0.0		0.0		2.2				
Intersection Summary												
HCM 6th Ctrl Delay			73.9									
HCM 6th LOS			E									

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	95	51	88	309	37	265	119	801	195	183	882	40
Future Volume (veh/h)	95	51	88	309	37	265	119	801	195	183	882	40
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	99	53	73	350	0	231	124	834	203	191	919	27
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	244	243	217	595	0	494	233	1074	519	263	1120	33
Arrive On Green	0.14	0.14	0.14	0.17	0.00	0.17	0.13	0.33	0.33	0.15	0.35	0.35
Sat Flow, veh/h	1781	1777	1585	3563	0	1557	1781	3272	1581	1781	3246	95
Grp Volume(v), veh/h	99	53	73	350	0	231	124	834	203	191	463	483
Grp Sat Flow(s),veh/h/ln	1781	1777	1585	1781	0	1557	1781	1636	1581	1781	1636	1705
Q Serve(g_s), s	4.4	2.3	3.6	7.9	0.0	10.4	5.7	20.0	8.6	8.9	22.6	22.6
Cycle Q Clear(g_c), s	4.4	2.3	3.6	7.9	0.0	10.4	5.7	20.0	8.6	8.9	22.6	22.6
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.06
Lane Grp Cap(c), veh/h	244	243	217	595	0	494	233	1074	519	263	565	588
V/C Ratio(X)	0.41	0.22	0.34	0.59	0.00	0.47	0.53	0.78	0.39	0.73	0.82	0.82
Avail Cap(c_a), veh/h	245	244	218	899	0	627	245	1230	594	286	658	686
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	34.4	33.5	34.1	33.6	0.0	24.1	35.4	26.4	22.6	35.5	26.1	26.1
Incr Delay (d2), s/veh	0.8	0.3	0.7	0.7	0.0	0.5	0.8	3.1	0.7	6.7	7.8	7.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.0	1.0	1.4	3.3	0.0	3.8	2.4	7.7	3.1	4.2	9.5	9.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	35.2	33.8	34.7	34.3	0.0	24.6	36.2	29.5	23.3	42.2	33.9	33.6
LnGrp LOS	D	C	C	C	A	C	D	C	C	D	C	C
Approach Vol, veh/h		225			581			1161			1137	
Approach Delay, s/veh		34.7			30.4			29.1			35.2	
Approach LOS		C			C			C			D	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	15.4	35.4		19.7	16.9	33.9		16.7				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	12.0	* 35		* 22	14.0	32.8		12.0				
Max Q Clear Time (g_c+I1), s	7.7	24.6		12.4	10.9	22.0		6.4				
Green Ext Time (p_c), s	0.1	5.5		1.2	0.1	5.8		0.4				

Intersection Summary			
HCM 6th Ctrl Delay			32.0
HCM 6th LOS			C

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	102	301	26	6	408	187	34	5	5	122	9	168
Future Volume (veh/h)	102	301	26	6	408	187	34	5	5	122	9	168
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	110	324	19	6	439	164	37	5	0	131	10	179
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	157	1468	86	20	891	329	339	36	0	490	31	483
Arrive On Green	0.09	0.43	0.43	0.01	0.35	0.35	0.22	0.22	0.00	0.22	0.22	0.22
Sat Flow, veh/h	1781	3412	199	1781	2522	932	766	161	0	1432	142	1555
Grp Volume(v), veh/h	110	168	175	6	308	295	42	0	0	141	0	179
Grp Sat Flow(s),veh/h/ln	1781	1777	1834	1781	1777	1678	927	0	0	1574	0	1555
Q Serve(g_s), s	2.4	2.4	2.4	0.1	5.4	5.5	0.8	0.0	0.0	0.0	0.0	3.6
Cycle Q Clear(g_c), s	2.4	2.4	2.4	0.1	5.4	5.5	3.4	0.0	0.0	2.7	0.0	3.6
Prop In Lane	1.00		0.11	1.00		0.56	0.88		0.00	0.93		1.00
Lane Grp Cap(c), veh/h	157	764	789	20	628	593	374	0	0	521	0	483
V/C Ratio(X)	0.70	0.22	0.22	0.30	0.49	0.50	0.11	0.00	0.00	0.27	0.00	0.37
Avail Cap(c_a), veh/h	223	764	789	312	628	593	382	0	0	527	0	490
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.89	0.89	0.89	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	17.7	7.2	7.2	19.6	10.1	10.1	13.7	0.0	0.0	13.2	0.0	10.8
Incr Delay (d2), s/veh	1.9	0.6	0.6	3.0	2.7	3.0	0.0	0.0	0.0	0.1	0.0	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	0.8	0.8	0.1	1.9	1.9	0.3	0.0	0.0	0.9	0.0	1.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	19.6	7.8	7.8	22.6	12.8	13.1	13.8	0.0	0.0	13.3	0.0	11.0
LnGrp LOS	B	A	A	C	B	B	B	A	A	B	A	B
Approach Vol, veh/h		453			609			42			320	
Approach Delay, s/veh		10.6			13.1			13.8			12.0	
Approach LOS		B			B			B			B	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	4.5	22.1		13.4	7.5	19.0		13.4				
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6				
Max Green Setting (Gmax), s	7.0	10.5		9.0	5.0	12.5		* 9.1				
Max Q Clear Time (g_c+I1), s	2.1	4.4		5.6	4.4	7.5		5.4				
Green Ext Time (p_c), s	0.0	1.2		0.2	0.0	1.8		0.0				

Intersection Summary		
HCM 6th Ctrl Delay		12.1
HCM 6th LOS		B

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	5.9
Intersection LOS	A

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	453	463	51	4
Demand Flow Rate, veh/h	462	472	52	4
Vehicles Circulating, veh/h	54	18	451	486
Vehicles Exiting, veh/h	436	485	65	4
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	6.1	5.9	4.8	4.3
Approach LOS	A	A	A	A

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	462	472	52	4
Cap Entry Lane, veh/h	1306	1355	871	841
Entry HV Adj Factor	0.981	0.980	0.981	1.000
Flow Entry, veh/h	453	463	51	4
Cap Entry, veh/h	1281	1328	854	841
V/C Ratio	0.354	0.348	0.060	0.005
Control Delay, s/veh	6.1	5.9	4.8	4.3
LOS	A	A	A	A
95th %tile Queue, veh	2	2	0	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	415	584	32	83	573	157	28	451	169	175	362	385
Future Volume (veh/h)	415	584	32	83	573	157	28	451	169	175	362	385
Initial Q (Qb), veh	0	0	0	0	0	0	0	4	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.98	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	446	628	30	89	616	155	30	485	154	188	389	395
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	544	1703	81	111	687	172	38	499	518	185	653	1032
Arrive On Green	0.31	0.49	0.49	0.06	0.25	0.25	0.02	0.27	0.27	0.10	0.35	0.35
Sat Flow, veh/h	1781	3450	165	1781	2802	703	1781	1870	1573	1781	1870	1571
Grp Volume(v), veh/h	446	323	335	89	390	381	30	485	154	188	389	395
Grp Sat Flow(s),veh/h/ln	1781	1777	1838	1781	1777	1728	1781	1870	1573	1781	1870	1571
Q Serve(g_s), s	31.3	15.2	15.2	6.7	28.7	28.8	2.3	34.7	9.8	14.0	23.1	2.3
Cycle Q Clear(g_c), s	31.3	15.2	15.2	6.7	28.7	28.8	2.3	34.7	9.8	14.0	23.1	2.3
Prop In Lane	1.00		0.09	1.00		0.41	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	544	877	907	111	436	424	38	499	518	185	653	1032
V/C Ratio(X)	0.82	0.37	0.37	0.80	0.90	0.90	0.79	0.97	0.30	1.02	0.60	0.38
Avail Cap(c_a), veh/h	544	877	907	172	436	424	79	499	518	185	653	1032
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	43.5	21.2	21.2	62.5	49.3	49.3	65.8	49.5	33.7	60.5	36.1	7.3
Incr Delay (d2), s/veh	9.1	1.2	1.2	7.1	23.6	24.5	12.6	34.0	1.5	70.9	4.0	1.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	16.5	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	15.3	6.7	7.0	3.3	15.7	15.4	1.2	23.9	4.0	10.0	11.4	4.4
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	52.5	22.4	22.3	69.6	72.9	73.8	78.3	100.0	35.2	131.4	40.1	8.4
LnGrp LOS	D	C	C	E	E	E	E	F	D	F	D	A
Approach Vol, veh/h		1104			860			669			972	
Approach Delay, s/veh		34.5			73.0			84.1			44.9	
Approach LOS		C			E			F			D	
Timer - Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	12.4	71.2	6.9	52.3	45.8	37.8	18.0	41.2				
Change Period (Y+Rc), s	4.0	4.6	4.0	* 5.2	* 4.6	* 4.7	4.0	* 5.2				
Max Green Setting (Gmax), s	13.0	54.2	6.0	* 44	* 34	* 33	14.0	* 36				
Max Q Clear Time (g_c+I1), s	8.7	17.2	4.3	25.1	33.3	30.8	16.0	36.7				
Green Ext Time (p_c), s	0.0	7.1	0.0	5.8	0.1	1.4	0.0	0.0				
Intersection Summary												
HCM 6th Ctrl Delay			55.7									
HCM 6th LOS			E									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	87	905	199	72	849	71	270	264	103	121	178	117
Future Volume (veh/h)	87	905	199	72	849	71	270	264	103	121	178	117
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	1.00		0.96	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	90	933	196	74	875	70	278	272	102	125	184	120
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	153	978	205	124	1069	85	339	243	91	113	167	109
Arrive On Green	0.09	0.35	0.35	0.07	0.34	0.34	0.20	0.20	0.20	0.23	0.23	0.23
Sat Flow, veh/h	1781	2761	579	1781	3163	253	1697	1217	457	482	710	463
Grp Volume(v), veh/h	90	571	558	74	468	477	278	0	374	429	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1648	1781	1692	1724	1697	0	1674	1655	0	0
Q Serve(g_s), s	6.6	44.4	44.6	5.4	34.2	34.2	21.2	0.0	27.0	31.7	0.0	0.0
Cycle Q Clear(g_c), s	6.6	44.4	44.6	5.4	34.2	34.2	21.2	0.0	27.0	31.7	0.0	0.0
Prop In Lane	1.00		0.35	1.00		0.15	1.00		0.27	0.29		0.28
Lane Grp Cap(c), veh/h	153	600	584	124	572	582	339	0	335	389	0	0
V/C Ratio(X)	0.59	0.95	0.95	0.60	0.82	0.82	0.82	0.00	1.12	1.10	0.00	0.00
Avail Cap(c_a), veh/h	158	600	584	132	572	582	339	0	335	389	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.83	0.00	0.83	1.00	0.00	0.00
Uniform Delay (d), s/veh	59.4	42.5	42.5	61.0	40.9	40.9	51.7	0.0	54.0	51.7	0.0	0.0
Incr Delay (d2), s/veh	3.4	26.9	27.7	4.1	12.4	12.2	11.7	0.0	80.5	76.8	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.1	23.1	22.6	2.6	16.2	16.4	10.1	0.0	18.8	21.4	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	62.8	69.4	70.2	65.0	53.3	53.1	63.3	0.0	134.5	128.5	0.0	0.0
LnGrp LOS	E	E	E	E	D	D	E	A	F	F	A	A
Approach Vol, veh/h		1219			1019			652			429	
Approach Delay, s/veh		69.3			54.0			104.2			128.5	
Approach LOS		E			D			F			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	13.4	52.6		37.0	15.6	50.4		32.0
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.0	* 4.8		5.0
Max Green Setting (Gmax), s	10.0	* 47		* 32	12.0	* 45		27.0
Max Q Clear Time (g_c+I1), s	7.4	46.6		33.7	8.6	36.2		29.0
Green Ext Time (p_c), s	0.0	0.4		0.0	0.0	3.0		0.0

Intersection Summary			
HCM 6th Ctrl Delay			79.1
HCM 6th LOS			E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	28	820	268	312	932	89	290	149	609	100	116	27
Future Volume (veh/h)	28	820	268	312	932	89	290	149	609	100	116	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		1.00	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	29	845	276	322	961	92	299	154	628	103	120	28
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	79	799	261	337	1507	144	269	139	349	118	138	220
Arrive On Green	0.04	0.31	0.31	0.19	0.46	0.46	0.23	0.23	0.23	0.14	0.14	0.14
Sat Flow, veh/h	1781	2613	852	1781	3277	314	1195	616	1549	844	984	1568
Grp Volume(v), veh/h	29	574	547	322	521	532	453	0	628	223	0	28
Grp Sat Flow(s),veh/h/ln	1781	1777	1688	1781	1777	1813	1811	0	1549	1828	0	1568
Q Serve(g_s), s	2.1	41.3	41.3	24.1	30.3	30.3	30.4	0.0	30.4	16.1	0.0	2.1
Cycle Q Clear(g_c), s	2.1	41.3	41.3	24.1	30.3	30.3	30.4	0.0	30.4	16.1	0.0	2.1
Prop In Lane	1.00		0.50	1.00		0.17	0.66		1.00	0.46		1.00
Lane Grp Cap(c), veh/h	79	544	517	337	817	834	408	0	349	257	0	220
V/C Ratio(X)	0.37	1.06	1.06	0.95	0.64	0.64	1.11	0.00	1.80	0.87	0.00	0.13
Avail Cap(c_a), veh/h	119	544	517	337	817	834	408	0	349	329	0	282
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	62.7	46.9	46.9	54.2	27.9	27.9	52.3	0.0	52.3	56.8	0.0	50.8
Incr Delay (d2), s/veh	1.1	54.3	56.0	36.8	3.8	3.7	78.2	0.0	371.4	15.1	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	26.3	25.3	14.3	13.7	13.9	22.7	0.0	47.9	8.6	0.0	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	63.7	101.2	102.9	91.0	31.6	31.6	130.5	0.0	423.7	71.9	0.0	50.9
LnGrp LOS	E	F	F	F	C	C	F	A	F	E	A	D
Approach Vol, veh/h		1150			1375			1081			251	
Approach Delay, s/veh		101.0			45.5			300.8			69.6	
Approach LOS		F			D			F			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	30.4	46.0		23.6	9.5	66.9		35.0				
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	* 22	* 41		* 24	9.0	* 54		30.4				
Max Q Clear Time (g_c+I1), s	26.1	43.3		18.1	4.1	32.3		32.4				
Green Ext Time (p_c), s	0.0	0.0		0.3	0.0	2.9		0.0				

Intersection Summary		
HCM 6th Ctrl Delay		135.2
HCM 6th LOS		F

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	83	369	20	201	374	162	92	304	220	165	365	48
Future Volume (veh/h)	83	369	20	201	374	162	92	304	220	165	365	48
Initial Q (Qb), veh	4	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.95	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	85	377	20	205	382	142	94	310	117	168	372	28
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	196	459	24	280	579	476	124	381	312	186	458	380
Arrive On Green	0.12	0.29	0.29	0.17	0.34	0.34	0.08	0.23	0.23	0.12	0.27	0.27
Sat Flow, veh/h	1603	1580	84	1603	1683	1382	1603	1683	1378	1603	1683	1398
Grp Volume(v), veh/h	85	0	397	205	382	142	94	310	117	168	372	28
Grp Sat Flow(s),veh/h/ln	1603	0	1663	1603	1683	1382	1603	1683	1378	1603	1683	1398
Q Serve(g_s), s	4.7	0.0	21.1	11.5	18.3	7.1	5.5	16.6	6.8	9.8	19.6	1.0
Cycle Q Clear(g_c), s	4.7	0.0	21.1	11.5	18.3	7.1	5.5	16.6	6.8	9.8	19.6	1.0
Prop In Lane	1.00		0.05	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	196	0	483	280	579	476	124	381	312	186	458	380
V/C Ratio(X)	0.43	0.00	0.82	0.73	0.66	0.30	0.76	0.81	0.38	0.91	0.81	0.07
Avail Cap(c_a), veh/h	196	0	483	280	579	476	135	461	377	186	510	424
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	39.1	0.0	31.4	37.1	26.4	22.8	43.0	34.9	31.1	41.5	32.3	12.7
Incr Delay (d2), s/veh	0.6	0.0	14.5	8.3	5.8	1.6	17.5	10.1	1.1	39.6	9.5	0.1
Initial Q Delay(d3),s/veh	5.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.6	0.0	10.3	5.1	8.2	2.5	2.7	7.7	2.3	5.9	9.1	0.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	44.9	0.0	45.9	45.3	32.2	24.4	60.5	45.0	32.2	81.1	41.8	12.8
LnGrp LOS	D	A	D	D	C	C	E	D	C	F	D	B
Approach Vol, veh/h		482			729			521			568	
Approach Delay, s/veh		45.8			34.4			44.9			52.0	
Approach LOS		D			C			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	20.6	32.3	11.3	30.8	15.6	37.3	15.9	26.2
Change Period (Y+Rc), s	* 4	4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7
Max Green Setting (Gmax), s	* 13	27.6	8.0	* 29	8.0	32.7	* 11	* 26
Max Q Clear Time (g_c+I1), s	13.5	23.1	7.5	21.6	6.7	20.3	11.8	18.6
Green Ext Time (p_c), s	0.0	0.9	0.0	1.8	0.0	2.0	0.0	1.8

Intersection Summary	
HCM 6th Ctrl Delay	43.5
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	800	16	67	679	73	9	7	132	129	5	25
Future Volume (veh/h)	8	800	16	67	679	73	9	7	132	129	5	25
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	9	851	17	71	722	78	10	7	140	137	5	27
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	201	1128	23	127	945	102	49	23	258	223	12	31
Arrive On Green	0.11	0.62	0.62	0.07	0.57	0.57	0.17	0.17	0.17	0.17	0.17	0.17
Sat Flow, veh/h	1781	1827	37	1781	1659	179	49	133	1497	894	68	183
Grp Volume(v), veh/h	9	0	868	71	0	800	157	0	0	169	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1864	1781	0	1838	1679	0	0	1144	0	0
Q Serve(g_s), s	0.4	0.0	31.7	3.7	0.0	31.5	0.0	0.0	0.0	5.5	0.0	0.0
Cycle Q Clear(g_c), s	0.4	0.0	31.7	3.7	0.0	31.5	8.5	0.0	0.0	14.0	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.10	0.06		0.89	0.81		0.16
Lane Grp Cap(c), veh/h	201	0	1151	127	0	1047	330	0	0	266	0	0
V/C Ratio(X)	0.04	0.00	0.75	0.56	0.00	0.76	0.48	0.00	0.00	0.64	0.00	0.00
Avail Cap(c_a), veh/h	201	0	1151	150	0	1047	402	0	0	325	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.20	0.00	0.20	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	37.6	0.0	13.0	42.7	0.0	15.6	36.1	0.0	0.0	38.7	0.0	0.0
Incr Delay (d2), s/veh	0.1	0.0	4.6	0.8	0.0	1.1	1.1	0.0	0.0	2.9	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	13.6	1.6	0.0	12.6	3.4	0.0	0.0	4.0	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	37.7	0.0	17.6	43.4	0.0	16.7	37.1	0.0	0.0	41.6	0.0	0.0
LnGrp LOS	D	A	B	D	A	B	D	A	A	D	A	A
Approach Vol, veh/h		877			871			157			169	
Approach Delay, s/veh		17.8			18.9			37.1			41.6	
Approach LOS		B			B			D			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	10.8	63.3		21.0	15.3	58.7		21.0
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6
Max Green Setting (Gmax), s	8.0	53.1		20.7	7.0	* 54		20.7
Max Q Clear Time (g_c+I1), s	5.7	33.7		16.0	2.4	33.5		10.5
Green Ext Time (p_c), s	0.0	7.0		0.4	0.0	6.4		0.6

Intersection Summary	
HCM 6th Ctrl Delay	21.7
HCM 6th LOS	C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	70	152	588	25	171	16	546	545	56	8	406	48
Future Volume (veh/h)	70	152	588	25	171	16	546	545	56	8	406	48
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.97	1.00		1.00	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	74	162	626	27	182	16	581	580	60	9	432	45
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	80	175	830	202	191	17	688	1232	1130	31	496	459
Arrive On Green	0.14	0.14	0.14	0.11	0.11	0.11	0.39	0.72	0.72	0.29	0.29	0.29
Sat Flow, veh/h	577	1264	1571	1781	1690	149	1781	1722	1581	13	1691	1566
Grp Volume(v), veh/h	236	0	626	27	0	198	581	580	60	441	0	45
Grp Sat Flow(s),veh/h/ln	1841	0	1571	1781	0	1838	1781	1722	1581	1704	0	1566
Q Serve(g_s), s	17.1	0.0	0.0	1.8	0.0	14.5	40.1	19.5	1.5	8.7	0.0	2.8
Cycle Q Clear(g_c), s	17.1	0.0	0.0	1.8	0.0	14.5	40.1	19.5	1.5	33.2	0.0	2.8
Prop In Lane	0.31		1.00	1.00		0.08	1.00		1.00	0.02		1.00
Lane Grp Cap(c), veh/h	255	0	830	202	0	208	688	1232	1130	527	0	459
V/C Ratio(X)	0.93	0.00	0.75	0.13	0.00	0.95	0.84	0.47	0.05	0.84	0.00	0.10
Avail Cap(c_a), veh/h	255	0	830	202	0	208	688	1232	1130	527	0	459
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.56	0.00	0.56	1.00	0.00	1.00	1.00	1.00	1.00	0.15	0.00	0.15
Uniform Delay (d), s/veh	57.5	0.0	25.2	53.9	0.0	59.5	37.7	8.3	5.7	45.3	0.0	34.7
Incr Delay (d2), s/veh	25.1	0.0	2.4	0.2	0.0	48.1	9.0	1.3	0.1	2.5	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	9.8	0.0	16.0	0.9	0.0	9.6	19.1	7.3	0.5	14.4	0.0	1.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	82.6	0.0	27.6	54.1	0.0	107.6	46.7	9.5	5.8	47.9	0.0	34.8
LnGrp LOS	F	A	C	D	A	F	D	A	A	D	A	C
Approach Vol, veh/h		862			225			1221			486	
Approach Delay, s/veh		42.6			101.2			27.0			46.7	
Approach LOS		D			F			C			D	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		24.0	57.0	44.4		19.6		101.4				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 19	* 43	* 40		15.3		* 87				
Max Q Clear Time (g_c+I1), s		19.1	42.1	35.2		16.5		21.5				
Green Ext Time (p_c), s		0.0	0.1	0.9		0.0		2.8				
Intersection Summary												
HCM 6th Ctrl Delay			41.2									
HCM 6th LOS			D									

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	159	118	52	248	51	201	124	906	483	359	729	91
Future Volume (veh/h)	159	118	52	248	51	201	124	906	483	359	729	91
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	167	124	41	300	0	168	131	954	505	378	767	71
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	235	350	111	518	0	487	227	1133	547	294	1163	108
Arrive On Green	0.13	0.13	0.13	0.15	0.00	0.15	0.13	0.35	0.35	0.17	0.38	0.38
Sat Flow, veh/h	1781	2652	844	3563	0	1552	1781	3272	1581	1781	3028	280
Grp Volume(v), veh/h	167	82	83	300	0	168	131	954	505	378	414	424
Grp Sat Flow(s),veh/h/ln	1781	1777	1718	1781	0	1552	1781	1636	1581	1781	1636	1672
Q Serve(g_s), s	8.2	3.8	4.0	7.1	0.0	7.6	6.3	24.4	27.9	15.0	19.0	19.0
Cycle Q Clear(g_c), s	8.2	3.8	4.0	7.1	0.0	7.6	6.3	24.4	27.9	15.0	19.0	19.0
Prop In Lane	1.00		0.49	1.00		1.00	1.00		1.00	1.00		0.17
Lane Grp Cap(c), veh/h	235	235	227	518	0	487	227	1133	547	294	628	642
V/C Ratio(X)	0.71	0.35	0.37	0.58	0.00	0.34	0.58	0.84	0.92	1.29	0.66	0.66
Avail Cap(c_a), veh/h	235	235	227	863	0	638	235	1145	553	294	632	646
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	37.8	35.9	36.0	36.2	0.0	24.2	37.4	27.4	28.5	37.9	23.1	23.1
Incr Delay (d2), s/veh	9.0	0.7	0.7	0.8	0.0	0.3	2.0	6.0	21.4	151.8	2.8	2.8
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.1	1.7	1.7	3.0	0.0	2.8	2.8	9.8	13.0	18.7	7.4	7.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	46.8	36.5	36.7	37.0	0.0	24.5	39.3	33.4	49.9	189.7	25.9	25.9
LnGrp LOS	D	D	D	D	A	C	D	C	D	F	C	C
Approach Vol, veh/h		332			468			1590			1216	
Approach Delay, s/veh		41.7			32.5			39.1			76.8	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	15.6	40.2		18.3	19.0	36.8		16.8				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	12.0	* 35		* 22	15.0	31.8		12.0				
Max Q Clear Time (g_c+I1), s	8.3	21.0		9.6	17.0	29.9		10.2				
Green Ext Time (p_c), s	0.1	6.0		1.0	0.0	1.6		0.2				

Intersection Summary			
HCM 6th Ctrl Delay			51.2
HCM 6th LOS			D

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	167	768	24	18	352	248	24	25	42	408	30	125
Future Volume (veh/h)	167	768	24	18	352	248	24	25	42	408	30	125
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.97	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	169	776	17	18	356	204	24	25	12	412	30	118
Peak Hour Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	204	1555	34	60	776	435	72	64	16	377	20	772
Arrive On Green	0.11	0.44	0.44	0.03	0.36	0.36	0.38	0.38	0.38	0.38	0.38	0.38
Sat Flow, veh/h	1781	3553	78	1781	2171	1219	0	168	41	733	53	1562
Grp Volume(v), veh/h	169	388	405	18	290	270	61	0	0	442	0	118
Grp Sat Flow(s),veh/h/ln	1781	1777	1854	1781	1777	1614	209	0	0	786	0	1562
Q Serve(g_s), s	6.5	11.0	11.0	0.7	8.8	9.0	0.0	0.0	0.0	0.0	0.0	2.9
Cycle Q Clear(g_c), s	6.5	11.0	11.0	0.7	8.8	9.0	26.5	0.0	0.0	26.5	0.0	2.9
Prop In Lane	1.00		0.04	1.00		0.76	0.39		0.20	0.93		1.00
Lane Grp Cap(c), veh/h	204	778	812	60	635	576	151	0	0	397	0	772
V/C Ratio(X)	0.83	0.50	0.50	0.30	0.46	0.47	0.40	0.00	0.00	1.11	0.00	0.15
Avail Cap(c_a), veh/h	204	778	812	204	635	576	151	0	0	397	0	772
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.37	0.37	0.37	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	30.3	14.2	14.2	33.0	17.3	17.4	17.9	0.0	0.0	25.1	0.0	9.7
Incr Delay (d2), s/veh	9.8	0.8	0.8	1.0	2.4	2.7	0.6	0.0	0.0	79.5	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.3	4.3	4.5	0.3	3.6	3.4	0.6	0.0	0.0	15.3	0.0	0.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	40.1	15.0	15.0	34.0	19.6	20.1	18.6	0.0	0.0	104.6	0.0	9.8
LnGrp LOS	D	B	B	C	B	C	B	A	A	F	A	A
Approach Vol, veh/h		962			578			61			560	
Approach Delay, s/veh		19.4			20.3			18.6			84.6	
Approach LOS		B			C			B			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	6.4	35.6		31.1	12.0	30.0		31.1
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6
Max Green Setting (Gmax), s	8.0	22.1		26.4	8.0	22.1		* 27
Max Q Clear Time (g_c+I1), s	2.7	13.0		28.5	8.5	11.0		28.5
Green Ext Time (p_c), s	0.0	4.0		0.0	0.0	2.9		0.0

Intersection Summary	
HCM 6th Ctrl Delay	36.5
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	8.2
Intersection LOS	A

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	587	736	88	9
Demand Flow Rate, veh/h	599	750	90	9
Vehicles Circulating, veh/h	64	31	571	775
Vehicles Exiting, veh/h	720	630	92	6
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	7.6	9.0	6.0	5.9
Approach LOS	A	A	A	A

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	599	750	90	9
Cap Entry Lane, veh/h	1293	1337	771	626
Entry HV Adj Factor	0.980	0.981	0.978	1.000
Flow Entry, veh/h	587	736	88	9
Cap Entry, veh/h	1267	1311	754	626
V/C Ratio	0.463	0.561	0.117	0.014
Control Delay, s/veh	7.6	9.0	6.0	5.9
LOS	A	A	A	A
95th %tile Queue, veh	3	4	0	0

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	223	635	87	57	642	86	35	261	79	57	335	302
Future Volume (veh/h)	223	635	87	57	642	86	35	261	79	57	335	302
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	253	722	99	65	730	98	40	297	90	65	381	343
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	624	1706	234	84	738	99	51	412	424	83	446	933
Arrive On Green	0.35	0.54	0.54	0.05	0.23	0.23	0.03	0.22	0.22	0.05	0.24	0.24
Sat Flow, veh/h	1781	3140	430	1781	3149	422	1781	1870	1585	1781	1870	1585
Grp Volume(v), veh/h	253	408	413	65	412	416	40	297	90	65	381	343
Grp Sat Flow(s),veh/h/ln	1781	1777	1793	1781	1777	1794	1781	1870	1585	1781	1870	1585
Q Serve(g_s), s	13.4	17.0	17.1	4.5	28.9	28.9	2.8	18.4	5.5	4.5	24.4	2.1
Cycle Q Clear(g_c), s	13.4	17.0	17.1	4.5	28.9	28.9	2.8	18.4	5.5	4.5	24.4	2.1
Prop In Lane	1.00		0.24	1.00		0.24	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	624	966	974	84	416	421	51	412	424	83	446	933
V/C Ratio(X)	0.41	0.42	0.42	0.78	0.99	0.99	0.78	0.72	0.21	0.78	0.85	0.37
Avail Cap(c_a), veh/h	624	966	974	242	416	421	157	521	516	157	527	1002
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	30.7	16.9	16.9	58.9	47.7	47.7	60.3	45.2	35.6	58.9	45.5	6.2
Incr Delay (d2), s/veh	0.2	1.4	1.3	5.7	41.4	41.4	9.1	4.4	0.4	5.8	12.3	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.9	7.4	7.4	2.2	17.6	17.8	1.4	9.1	2.2	2.2	12.9	3.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	30.9	18.3	18.3	64.6	89.1	89.1	69.4	49.6	35.9	64.7	57.9	6.5
LnGrp LOS	C	B	B	E	F	F	E	D	D	E	E	A
Approach Vol, veh/h		1074			893			427			789	
Approach Delay, s/veh		21.2			87.3			48.5			36.1	
Approach LOS		C			F			D			D	
Timer - Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	9.9	72.5	7.6	35.0	48.4	34.0	9.8	32.7				
Change Period (Y+Rc), s	4.0	4.6	4.0	* 5.2	* 4.6	* 4.7	4.0	* 5.2				
Max Green Setting (Gmax), s	17.0	44.4	11.0	* 35	* 32	* 29	11.0	* 35				
Max Q Clear Time (g_c+I1), s	6.5	19.1	4.8	26.4	15.4	30.9	6.5	20.4				
Green Ext Time (p_c), s	0.0	8.4	0.0	3.4	0.3	0.0	0.0	2.5				
Intersection Summary												
HCM 6th Ctrl Delay			47.1									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	19	644	183	62	724	37	210	93	52	35	132	23
Future Volume (veh/h)	19	644	183	62	724	37	210	93	52	35	132	23
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		0.96	1.00		0.97	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	20	692	193	67	778	37	226	100	51	38	142	25
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	460	1207	337	129	903	43	266	172	88	49	182	32
Arrive On Green	0.26	0.47	0.47	0.07	0.28	0.28	0.05	0.05	0.05	0.15	0.15	0.15
Sat Flow, veh/h	1781	2591	722	1781	3283	156	1697	1100	561	319	1190	210
Grp Volume(v), veh/h	20	452	433	67	401	414	226	0	151	205	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1621	1781	1692	1746	1697	0	1662	1719	0	0
Q Serve(g_s), s	1.1	24.3	24.3	4.5	28.1	28.2	16.5	0.0	11.1	14.3	0.0	0.0
Cycle Q Clear(g_c), s	1.1	24.3	24.3	4.5	28.1	28.2	16.5	0.0	11.1	14.3	0.0	0.0
Prop In Lane	1.00		0.45	1.00		0.09	1.00		0.34	0.19		0.12
Lane Grp Cap(c), veh/h	460	789	755	129	466	481	266	0	260	262	0	0
V/C Ratio(X)	0.04	0.57	0.57	0.52	0.86	0.86	0.85	0.00	0.58	0.78	0.00	0.00
Avail Cap(c_a), veh/h	460	789	755	157	466	481	394	0	386	408	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.96	0.00	0.96	1.00	0.00	0.00
Uniform Delay (d), s/veh	34.8	24.3	24.3	55.9	43.0	43.0	57.8	0.0	55.3	51.0	0.0	0.0
Incr Delay (d2), s/veh	0.0	3.0	3.2	1.2	18.5	18.1	7.3	0.0	0.7	2.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.5	10.5	10.0	2.1	14.1	14.5	8.1	0.0	5.0	6.3	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	34.8	27.3	27.5	57.1	61.5	61.1	65.1	0.0	56.0	53.0	0.0	0.0
LnGrp LOS	C	C	C	E	E	E	E	A	E	D	A	A
Approach Vol, veh/h		905			882			377			205	
Approach Delay, s/veh		27.6			61.0			61.5			53.0	
Approach LOS		C			E			E			D	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	13.0	63.0		24.4	37.1	39.0		24.6				
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0				
Max Green Setting (Gmax), s	11.0	* 36		* 30	13.0	* 34		29.0				
Max Q Clear Time (g_c+I1), s	6.5	26.3		16.3	3.1	30.2		18.5				
Green Ext Time (p_c), s	0.0	3.1		0.6	0.0	1.5		0.7				
Intersection Summary												
HCM 6th Ctrl Delay			47.6									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	665	36	85	701	123	85	165	114	149	139	28
Future Volume (veh/h)	32	665	36	85	701	123	85	165	114	149	139	28
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.99	1.00		0.99	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	34	715	39	91	754	132	91	177	123	160	149	30
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	93	957	52	342	1297	227	107	208	269	178	166	293
Arrive On Green	0.05	0.28	0.28	0.19	0.43	0.43	0.17	0.17	0.17	0.19	0.19	0.19
Sat Flow, veh/h	1781	3426	187	1781	3017	528	624	1215	1568	944	879	1549
Grp Volume(v), veh/h	34	371	383	91	444	442	268	0	123	309	0	30
Grp Sat Flow(s),veh/h/ln	1781	1777	1836	1781	1777	1769	1839	0	1568	1823	0	1549
Q Serve(g_s), s	2.1	21.3	21.3	4.9	21.3	21.3	15.8	0.0	7.9	18.5	0.0	1.8
Cycle Q Clear(g_c), s	2.1	21.3	21.3	4.9	21.3	21.3	15.8	0.0	7.9	18.5	0.0	1.8
Prop In Lane	1.00		0.10	1.00		0.30	0.34		1.00	0.52		1.00
Lane Grp Cap(c), veh/h	93	497	513	342	764	760	315	0	269	344	0	293
V/C Ratio(X)	0.36	0.75	0.75	0.27	0.58	0.58	0.85	0.00	0.46	0.90	0.00	0.10
Avail Cap(c_a), veh/h	183	497	513	342	764	760	450	0	384	396	0	336
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	51.3	36.7	36.7	38.5	24.3	24.3	45.0	0.0	41.7	44.4	0.0	37.6
Incr Delay (d2), s/veh	0.9	9.8	9.6	0.2	3.2	3.2	7.4	0.0	0.5	19.2	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.9	10.5	10.8	2.1	9.5	9.4	7.9	0.0	3.1	10.2	0.0	0.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	52.1	46.6	46.3	38.7	27.5	27.5	52.4	0.0	42.2	63.6	0.0	37.6
LnGrp LOS	D	D	D	D	C	C	D	A	D	E	A	D
Approach Vol, veh/h		788			977			391			339	
Approach Delay, s/veh		46.7			28.5			49.2			61.3	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	26.3	36.0		25.9	9.4	53.0		23.8				
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	* 12	* 31		* 24	11.5	* 31		27.4				
Max Q Clear Time (g_c+I1), s	6.9	23.3		20.5	4.1	23.3		17.8				
Green Ext Time (p_c), s	0.0	1.4		0.3	0.0	1.7		0.6				

Intersection Summary			
HCM 6th Ctrl Delay			42.0
HCM 6th LOS			D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	66	319	18	48	446	5	156	222	176	138	200	139
Future Volume (veh/h)	66	319	18	48	446	5	156	222	176	138	200	139
Initial Q (Qb), veh	5	0	0	1	0	0	0	0	0	0	3	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		1.00	1.00		0.97	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	78	375	17	56	525	-20	184	261	36	162	235	130
Peak Hour Factor	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	129	466	21	305	782	662	143	352	290	194	389	347
Arrive On Green	0.08	0.29	0.29	0.19	0.41	0.00	0.09	0.21	0.21	0.10	0.23	0.23
Sat Flow, veh/h	1603	1594	72	1603	1683	1427	1603	1683	1389	1603	1683	1389
Grp Volume(v), veh/h	78	0	392	56	525	-20	184	261	36	162	235	130
Grp Sat Flow(s),veh/h/ln	1603	0	1666	1603	1683	1427	1603	1683	1389	1603	1683	1389
Q Serve(g_s), s	4.3	0.0	19.6	2.6	24.0	0.0	8.0	13.1	1.9	9.0	11.3	7.2
Cycle Q Clear(g_c), s	4.3	0.0	19.6	2.6	24.0	0.0	8.0	13.1	1.9	9.0	11.3	7.2
Prop In Lane	1.00		0.04	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	129	0	487	305	782	662	143	352	290	194	389	347
V/C Ratio(X)	0.61	0.00	0.80	0.18	0.67	-0.03	1.29	0.74	0.12	0.83	0.60	0.37
Avail Cap(c_a), veh/h	143	0	487	303	694	588	143	473	390	160	488	403
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	40.6	0.0	29.5	30.7	19.1	0.0	41.0	33.3	28.9	38.7	31.2	28.0
Incr Delay (d2), s/veh	3.5	0.0	13.3	0.1	2.7	0.0	173.4	5.3	0.3	32.4	2.2	1.0
Initial Q Delay(d3),s/veh	27.7	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	1.1	0.0
%ile BackOfQ(50%),veh/ln	3.2	0.0	9.5	1.1	8.8	0.0	10.0	5.7	0.6	5.2	5.2	2.4
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	71.8	0.0	42.7	30.9	21.8	0.0	214.4	38.6	29.2	71.1	34.4	29.0
LnGrp LOS	E	A	D	C	C	A	F	D	C	E	C	C
Approach Vol, veh/h		470			561			481			527	
Approach Delay, s/veh		47.6			23.5			105.2			44.4	
Approach LOS		D			C			F			D	
Timer - Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	21.6	31.0	12.0	25.4	10.9	41.7	13.9	23.5				
Change Period (Y+Rc), s	* 4.6	* 4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7				
Max Green Setting (Gmax), s	* 12	* 26	8.0	* 26	8.0	30.4	* 9	* 25				
Max Q Clear Time (g_c+I1), s	4.6	21.6	10.0	13.3	6.3	26.0	11.0	15.1				
Green Ext Time (p_c), s	0.0	0.9	0.0	2.1	0.0	1.2	0.0	1.5				
Intersection Summary												
HCM 6th Ctrl Delay			53.7									
HCM 6th LOS			D									

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	9	581	11	74	630	78	7	7	72	55	4	4
Future Volume (veh/h)	9	581	11	74	630	78	7	7	72	55	4	4
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	10	638	12	81	692	86	8	8	79	60	4	4
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	474	1260	24	137	805	100	50	18	117	181	12	7
Arrive On Green	0.27	0.69	0.69	0.08	0.49	0.49	0.09	0.09	0.09	0.09	0.09	0.09
Sat Flow, veh/h	1781	1830	34	1781	1631	203	70	201	1342	1208	140	84
Grp Volume(v), veh/h	10	0	650	81	0	778	95	0	0	68	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1864	1781	0	1834	1613	0	0	1432	0	0
Q Serve(g_s), s	0.4	0.0	15.0	4.0	0.0	33.6	1.1	0.0	0.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	0.4	0.0	15.0	4.0	0.0	33.6	5.1	0.0	0.0	3.9	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.11	0.08		0.83	0.88		0.06
Lane Grp Cap(c), veh/h	474	0	1284	137	0	905	184	0	0	200	0	0
V/C Ratio(X)	0.02	0.00	0.51	0.59	0.00	0.86	0.52	0.00	0.00	0.34	0.00	0.00
Avail Cap(c_a), veh/h	474	0	1284	198	0	905	423	0	0	418	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.72	0.00	0.72	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	24.4	0.0	6.7	40.1	0.0	20.1	39.8	0.0	0.0	39.2	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	1.4	2.9	0.0	7.8	2.2	0.0	0.0	1.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	5.6	1.8	0.0	15.5	2.1	0.0	0.0	1.5	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	24.4	0.0	8.1	43.0	0.0	27.9	42.0	0.0	0.0	40.2	0.0	0.0
LnGrp LOS	C	A	A	D	A	C	D	A	A	D	A	A
Approach Vol, veh/h		660			859			95			68	
Approach Delay, s/veh		8.4			29.3			42.0			40.2	
Approach LOS		A			C			D			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	10.9	66.6		12.5	28.5	49.0		12.5
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6
Max Green Setting (Gmax), s	10.0	44.4		22.4	10.0	* 44		21.4
Max Q Clear Time (g_c+I1), s	6.0	17.0		5.9	2.4	35.6		7.1
Green Ext Time (p_c), s	0.1	5.2		0.3	0.0	3.8		0.4

Intersection Summary	
HCM 6th Ctrl Delay	22.3
HCM 6th LOS	C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	52	197	328	12	202	8	321	298	36	7	309	68
Future Volume (veh/h)	52	197	328	12	202	8	321	298	36	7	309	68
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.95	1.00		0.98	1.00		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	58	219	364	13	224	5	357	331	39	8	343	66
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	42	160	743	282	288	6	643	1063	956	33	369	329
Arrive On Green	0.11	0.11	0.11	0.16	0.16	0.16	0.36	0.62	0.62	0.22	0.22	0.22
Sat Flow, veh/h	388	1463	1560	1781	1820	41	1781	1722	1550	14	1696	1511
Grp Volume(v), veh/h	277	0	364	13	0	229	357	331	39	351	0	66
Grp Sat Flow(s),veh/h/ln	1851	0	1560	1781	0	1860	1781	1722	1550	1710	0	1511
Q Serve(g_s), s	13.7	0.0	0.0	0.8	0.0	14.8	20.0	11.4	1.2	8.1	0.0	4.5
Cycle Q Clear(g_c), s	13.7	0.0	0.0	0.8	0.0	14.8	20.0	11.4	1.2	25.2	0.0	4.5
Prop In Lane	0.21		1.00	1.00		0.02	1.00		1.00	0.02		1.00
Lane Grp Cap(c), veh/h	203	0	743	282	0	294	643	1063	956	402	0	329
V/C Ratio(X)	1.37	0.00	0.49	0.05	0.00	0.78	0.56	0.31	0.04	0.87	0.00	0.20
Avail Cap(c_a), veh/h	203	0	743	395	0	412	643	1063	956	402	0	329
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.85	0.00	0.85	1.00	0.00	1.00	1.00	1.00	1.00	0.66	0.00	0.66
Uniform Delay (d), s/veh	55.7	0.0	22.7	44.6	0.0	50.5	31.9	11.3	9.4	48.1	0.0	40.0
Incr Delay (d2), s/veh	189.0	0.0	0.6	0.0	0.0	5.2	0.6	0.8	0.1	16.0	0.0	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	17.1	0.0	7.4	0.4	0.0	7.4	8.7	4.5	0.4	12.5	0.0	1.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	244.6	0.0	23.3	44.7	0.0	55.7	32.5	12.1	9.5	64.1	0.0	40.9
LnGrp LOS	F	A	C	D	A	E	C	B	A	E	A	D
Approach Vol, veh/h		641			242			727			417	
Approach Delay, s/veh		119.0			55.1			22.0			60.4	
Approach LOS		F			E			C			E	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		19.0	49.9	32.0		24.1		81.9				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 14	* 38	* 27		27.7		* 69				
Max Q Clear Time (g_c+I1), s		15.7	22.0	27.2		16.8		13.4				
Green Ext Time (p_c), s		0.0	0.3	0.0		0.8		1.4				
Intersection Summary												
HCM 6th Ctrl Delay			64.5									
HCM 6th LOS			E									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	172	101	392	349	154	137	406	433	232	70	390	197
Future Volume (veh/h)	172	101	392	349	154	137	406	433	232	70	390	197
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	179	105	389	262	303	98	423	451	242	73	406	190
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	311	310	277	333	350	447	451	1252	605	174	493	228
Arrive On Green	0.17	0.17	0.17	0.19	0.19	0.19	0.25	0.38	0.38	0.10	0.23	0.23
Sat Flow, veh/h	1781	1777	1585	1781	1870	1560	1781	3272	1581	1781	2173	1005
Grp Volume(v), veh/h	179	105	389	262	303	98	423	451	242	73	305	291
Grp Sat Flow(s),veh/h/ln	1781	1777	1585	1781	1870	1560	1781	1636	1581	1781	1636	1541
Q Serve(g_s), s	11.2	6.3	21.2	17.0	19.1	5.8	28.2	12.0	13.6	4.7	21.5	21.9
Cycle Q Clear(g_c), s	11.2	6.3	21.2	17.0	19.1	5.8	28.2	12.0	13.6	4.7	21.5	21.9
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.65
Lane Grp Cap(c), veh/h	311	310	277	333	350	447	451	1252	605	174	371	350
V/C Ratio(X)	0.58	0.34	1.41	0.79	0.87	0.22	0.94	0.36	0.40	0.42	0.82	0.83
Avail Cap(c_a), veh/h	311	310	277	453	476	552	645	1769	855	191	471	444
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	46.0	44.0	50.1	47.1	47.9	33.1	44.4	26.9	27.3	51.5	44.6	44.8
Incr Delay (d2), s/veh	2.2	0.5	203.2	5.5	11.0	0.2	14.4	0.2	0.6	0.6	10.0	11.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	5.2	2.8	24.0	7.9	9.8	2.3	13.9	4.6	5.1	2.1	9.6	9.4
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	48.3	44.5	253.3	52.6	58.9	33.3	58.8	27.1	28.0	52.1	54.6	56.4
LnGrp LOS	D	D	F	D	E	C	E	C	C	D	D	E
Approach Vol, veh/h		673			663			1116			669	
Approach Delay, s/veh		166.2			52.6			39.3			55.1	
Approach LOS		F			D			D			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	34.8	32.9		27.8	15.9	51.8		26.0				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	44.0	* 35		* 31	13.0	65.7		21.2				
Max Q Clear Time (g_c+I1), s	30.2	23.9		21.1	6.7	15.6		23.2				
Green Ext Time (p_c), s	0.5	3.7		1.6	0.0	6.2		0.0				

Intersection Summary		
HCM 6th Ctrl Delay		72.9
HCM 6th LOS		E

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	102	275	26	6	440	187	34	5	5	122	9	168
Future Volume (veh/h)	102	275	26	6	440	187	34	5	5	122	9	168
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	110	296	19	6	473	164	37	5	0	131	10	179
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	141	1969	126	20	1322	455	221	22	0	368	22	378
Arrive On Green	0.08	0.58	0.58	0.01	0.51	0.51	0.16	0.16	0.00	0.16	0.16	0.16
Sat Flow, veh/h	1781	3392	217	1781	2578	887	598	137	0	1482	134	1550
Grp Volume(v), veh/h	110	154	161	6	325	312	42	0	0	141	0	179
Grp Sat Flow(s),veh/h/ln	1781	1777	1831	1781	1777	1688	735	0	0	1616	0	1550
Q Serve(g_s), s	3.3	2.2	2.2	0.2	6.0	6.1	1.3	0.0	0.0	0.0	0.0	5.4
Cycle Q Clear(g_c), s	3.3	2.2	2.2	0.2	6.0	6.1	5.4	0.0	0.0	4.1	0.0	5.4
Prop In Lane	1.00		0.12	1.00		0.53	0.88		0.00	0.93		1.00
Lane Grp Cap(c), veh/h	141	1031	1063	20	911	865	243	0	0	390	0	378
V/C Ratio(X)	0.78	0.15	0.15	0.30	0.36	0.36	0.17	0.00	0.00	0.36	0.00	0.47
Avail Cap(c_a), veh/h	162	1031	1063	227	911	865	300	0	0	453	0	446
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.84	0.84	0.84	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	24.9	5.3	5.3	27.0	8.0	8.0	22.4	0.0	0.0	21.0	0.0	17.9
Incr Delay (d2), s/veh	13.9	0.3	0.3	3.1	1.1	1.2	0.1	0.0	0.0	0.2	0.0	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.9	0.7	0.7	0.1	1.9	1.9	0.5	0.0	0.0	1.5	0.0	1.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	38.8	5.6	5.6	30.1	9.1	9.2	22.5	0.0	0.0	21.2	0.0	18.2
LnGrp LOS	D	A	A	C	A	A	C	A	A	C	A	B
Approach Vol, veh/h		425			643			42			320	
Approach Delay, s/veh		14.2			9.3			22.5			19.5	
Approach LOS		B			A			C			B	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	4.6	36.8		13.6	8.3	33.1		13.6
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6
Max Green Setting (Gmax), s	7.0	23.1		11.4	5.0	25.1		* 12
Max Q Clear Time (g_c+I1), s	2.2	4.2		7.4	5.3	8.1		7.4
Green Ext Time (p_c), s	0.0	2.1		0.3	0.0	4.1		0.0

Intersection Summary	
HCM 6th Ctrl Delay	13.4
HCM 6th LOS	B

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	8.6
Intersection LOS	A

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	431	160	150	795
Demand Flow Rate, veh/h	440	163	153	811
Vehicles Circulating, veh/h	200	544	523	34
Vehicles Exiting, veh/h	645	132	117	673
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	7.3	6.9	6.5	10.0
Approach LOS	A	A	A	A

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	440	163	153	811
Cap Entry Lane, veh/h	1125	792	809	1333
Entry HV Adj Factor	0.980	0.982	0.979	0.980
Flow Entry, veh/h	431	160	150	795
Cap Entry, veh/h	1103	778	792	1306
V/C Ratio	0.391	0.206	0.189	0.608
Control Delay, s/veh	7.3	6.9	6.5	10.0
LOS	A	A	A	A
95th %tile Queue, veh	2	1	1	4

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	172	101	392	349	154	137	406	433	232	70	390	197
Future Volume (veh/h)	172	101	392	349	154	137	406	433	232	70	390	197
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	179	105	314	364	160	143	423	451	121	73	406	205
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	213	355	705	445	180	161	455	1242	600	198	497	248
Arrive On Green	0.12	0.19	0.19	0.13	0.20	0.20	0.26	0.38	0.38	0.11	0.24	0.24
Sat Flow, veh/h	1781	1870	1585	3456	903	807	1781	3272	1581	1781	2114	1055
Grp Volume(v), veh/h	179	105	314	364	0	303	423	451	121	73	313	298
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	0	1710	1781	1636	1581	1781	1636	1532
Q Serve(g_s), s	10.0	4.9	14.0	10.5	0.0	17.6	23.7	10.1	5.3	3.9	18.5	18.9
Cycle Q Clear(g_c), s	10.0	4.9	14.0	10.5	0.0	17.6	23.7	10.1	5.3	3.9	18.5	18.9
Prop In Lane	1.00		1.00	1.00		0.47	1.00		1.00	1.00		0.69
Lane Grp Cap(c), veh/h	213	355	705	445	0	340	455	1242	600	198	385	361
V/C Ratio(X)	0.84	0.30	0.45	0.82	0.00	0.89	0.93	0.36	0.20	0.37	0.81	0.83
Avail Cap(c_a), veh/h	384	425	765	744	0	383	558	1524	737	227	463	433
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	44.0	35.5	19.6	43.3	0.0	39.9	37.2	22.8	21.3	42.1	36.9	37.1
Incr Delay (d2), s/veh	6.5	0.3	0.3	2.8	0.0	19.9	18.4	0.3	0.2	0.4	10.1	11.7
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	4.8	2.3	5.2	4.5	0.0	9.0	12.2	3.8	2.0	1.7	8.3	8.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	50.5	35.9	20.0	46.1	0.0	59.8	55.6	23.1	21.5	42.5	47.0	48.8
LnGrp LOS	D	D	B	D	A	E	E	C	C	D	D	D
Approach Vol, veh/h		598			667			995			684	
Approach Delay, s/veh		31.9			52.3			36.7			47.3	
Approach LOS		C			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	30.1	29.3	17.3	25.4	15.4	44.1	18.3	24.5
Change Period (Y+Rc), s	4.0	* 5.3	* 5.1	* 5.1	4.0	5.3	* 5.1	* 5.1
Max Green Setting (Gmax), s	32.0	* 29	* 22	* 23	13.0	47.6	* 22	* 23
Max Q Clear Time (g_c+I1), s	25.7	20.9	12.0	19.6	5.9	12.1	12.5	6.9
Green Ext Time (p_c), s	0.4	3.0	0.3	0.4	0.0	5.1	0.7	0.1

Intersection Summary	
HCM 6th Ctrl Delay	41.7
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	377	546	58	124	557	110	35	406	157	86	387	271
Future Volume (veh/h)	377	546	58	124	557	110	35	406	157	86	387	271
Initial Q (Qb), veh	0	0	0	0	0	0	0	4	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.98	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	405	587	58	133	599	104	38	437	141	92	416	272
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	429	1386	137	159	825	143	49	521	579	115	590	877
Arrive On Green	0.24	0.42	0.42	0.09	0.27	0.27	0.03	0.28	0.28	0.06	0.32	0.32
Sat Flow, veh/h	1781	3261	322	1781	3020	523	1781	1870	1573	1781	1870	1570
Grp Volume(v), veh/h	405	319	326	133	352	351	38	437	141	92	416	272
Grp Sat Flow(s),veh/h/ln	1781	1777	1806	1781	1777	1766	1781	1870	1573	1781	1870	1570
Q Serve(g_s), s	27.9	15.7	15.8	9.2	22.4	22.6	2.7	27.5	7.8	6.4	24.5	11.6
Cycle Q Clear(g_c), s	27.9	15.7	15.8	9.2	22.4	22.6	2.7	27.5	7.8	6.4	24.5	11.6
Prop In Lane	1.00		0.18	1.00		0.30	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	429	755	767	159	485	482	49	521	579	115	590	877
V/C Ratio(X)	0.94	0.42	0.42	0.84	0.72	0.73	0.78	0.84	0.24	0.80	0.71	0.31
Avail Cap(c_a), veh/h	456	755	767	228	485	482	157	521	579	157	590	877
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	46.6	25.2	25.2	56.0	41.2	41.2	60.4	43.0	27.5	57.7	37.7	14.9
Incr Delay (d2), s/veh	27.1	1.7	1.7	11.7	9.1	9.3	9.6	15.0	1.0	13.3	6.9	0.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.6	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	15.6	7.1	7.2	4.7	11.1	11.1	1.3	16.0	3.1	3.3	12.4	4.4
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	73.7	26.9	26.9	67.8	50.3	50.5	70.0	60.6	28.5	71.0	44.6	15.8
LnGrp LOS	E	C	C	E	D	D	E	E	C	E	D	B
Approach Vol, veh/h		1050			836			616			780	
Approach Delay, s/veh		45.0			53.2			53.8			37.7	
Approach LOS		D			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	15.1	57.8	7.4	44.6	34.1	38.8	12.0	40.0
Change Period (Y+Rc), s	4.0	* 4.7	4.0	* 5.2	4.0	* 4.7	4.0	* 5.2
Max Green Setting (Gmax), s	16.0	* 44	11.0	* 35	32.0	* 29	11.0	* 35
Max Q Clear Time (g_c+I1), s	11.2	17.8	4.7	26.5	29.9	24.6	8.4	29.5
Green Ext Time (p_c), s	0.1	6.4	0.0	3.3	0.2	2.4	0.0	2.0

Intersection Summary	
HCM 6th Ctrl Delay	47.0
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	38	732	189	72	678	47	213	169	103	61	109	37
Future Volume (veh/h)	38	732	189	72	678	47	213	169	103	61	109	37
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	1.00		0.95	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	39	755	186	74	699	45	220	174	102	63	112	37
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	364	1100	271	132	893	57	330	201	118	83	147	49
Arrive On Green	0.20	0.41	0.41	0.07	0.28	0.28	0.06	0.06	0.06	0.17	0.17	0.17
Sat Flow, veh/h	1781	2668	657	1781	3218	207	1697	1033	605	501	892	295
Grp Volume(v), veh/h	39	479	462	74	367	377	220	0	276	212	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1633	1781	1692	1733	1697	0	1638	1688	0	0
Q Serve(g_s), s	2.2	28.8	28.8	5.0	24.8	24.9	15.7	0.0	20.7	14.9	0.0	0.0
Cycle Q Clear(g_c), s	2.2	28.8	28.8	5.0	24.8	24.9	15.7	0.0	20.7	14.9	0.0	0.0
Prop In Lane	1.00		0.40	1.00		0.12	1.00		0.37	0.30		0.17
Lane Grp Cap(c), veh/h	364	698	673	132	469	481	330	0	318	279	0	0
V/C Ratio(X)	0.11	0.69	0.69	0.56	0.78	0.78	0.67	0.00	0.87	0.76	0.00	0.00
Avail Cap(c_a), veh/h	364	698	673	158	469	481	369	0	357	418	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.92	0.00	0.92	1.00	0.00	0.00
Uniform Delay (d), s/veh	40.1	29.9	29.9	55.4	41.3	41.4	54.1	0.0	56.4	49.4	0.0	0.0
Incr Delay (d2), s/veh	0.0	5.4	5.6	1.4	12.2	12.1	2.5	0.0	15.7	1.9	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.0	12.8	12.4	2.3	11.9	12.2	7.4	0.0	10.5	6.4	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	40.2	35.3	35.5	56.8	53.6	53.4	56.7	0.0	72.2	51.3	0.0	0.0
LnGrp LOS	D	D	D	E	D	D	E	A	E	D	A	A
Approach Vol, veh/h		980			818			496			212	
Approach Delay, s/veh		35.6			53.8			65.3			51.3	
Approach LOS		D			D			E			D	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	13.2	55.9		25.8	30.1	39.0		29.1				
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0				
Max Green Setting (Gmax), s	11.0	* 36		* 31	13.0	* 34		27.0				
Max Q Clear Time (g_c+I1), s	7.0	30.8		16.9	4.2	26.9		22.7				
Green Ext Time (p_c), s	0.0	2.2		0.6	0.0	2.0		0.7				
Intersection Summary												
HCM 6th Ctrl Delay			48.7									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	28	820	35	73	859	89	88	142	160	100	99	27
Future Volume (veh/h)	28	820	35	73	859	89	88	142	160	100	99	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		1.00	1.00		0.97	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	29	845	36	75	886	92	91	146	165	103	102	28
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	82	1711	73	120	1671	174	115	185	251	123	122	210
Arrive On Green	0.05	0.49	0.49	0.07	0.51	0.51	0.16	0.16	0.16	0.13	0.13	0.13
Sat Flow, veh/h	1781	3468	148	1781	3249	337	705	1131	1536	917	908	1567
Grp Volume(v), veh/h	29	433	448	75	485	493	237	0	165	205	0	28
Grp Sat Flow(s),veh/h/ln	1781	1777	1839	1781	1777	1809	1835	0	1536	1825	0	1567
Q Serve(g_s), s	2.0	20.2	20.2	5.1	22.6	22.6	15.4	0.0	12.5	13.6	0.0	2.0
Cycle Q Clear(g_c), s	2.0	20.2	20.2	5.1	22.6	22.6	15.4	0.0	12.5	13.6	0.0	2.0
Prop In Lane	1.00		0.08	1.00		0.19	0.38		1.00	0.50		1.00
Lane Grp Cap(c), veh/h	82	876	907	120	914	931	300	0	251	245	0	210
V/C Ratio(X)	0.36	0.49	0.49	0.63	0.53	0.53	0.79	0.00	0.66	0.84	0.00	0.13
Avail Cap(c_a), veh/h	165	876	907	151	914	931	406	0	339	358	0	307
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	57.4	21.0	21.0	56.3	20.1	20.1	49.8	0.0	48.6	52.4	0.0	47.3
Incr Delay (d2), s/veh	1.0	2.0	1.9	2.0	2.2	2.2	5.0	0.0	1.1	7.4	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.9	8.8	9.1	2.4	9.8	10.0	7.5	0.0	4.9	6.8	0.0	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	58.3	23.0	23.0	58.3	22.3	22.3	54.8	0.0	49.7	59.8	0.0	47.4
LnGrp LOS	E	C	C	E	C	C	D	A	D	E	A	D
Approach Vol, veh/h		910			1053			402			233	
Approach Delay, s/veh		24.1			24.8			52.7			58.3	
Approach LOS		C			C			D			E	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	11.8	66.0		21.3	9.2	68.6		24.9				
Change Period (Y+Rc), s	3.5	* 4.8		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	10.5	* 44		* 24	11.5	* 43		27.4				
Max Q Clear Time (g_c+I1), s	7.1	22.2		15.6	4.0	24.6		17.4				
Green Ext Time (p_c), s	0.0	2.3		0.4	0.0	2.6		0.6				
Intersection Summary												
HCM 6th Ctrl Delay			31.9									
HCM 6th LOS			C									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	110	362	31	153	357	130	140	366	193	125	353	57
Future Volume (veh/h)	110	362	31	153	357	130	140	366	193	125	353	57
Initial Q (Qb), veh	4	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	112	369	32	156	364	110	143	373	90	128	360	37
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	217	506	44	260	607	499	168	415	340	152	409	339
Arrive On Green	0.14	0.33	0.33	0.16	0.36	0.36	0.10	0.25	0.25	0.09	0.24	0.24
Sat Flow, veh/h	1603	1521	132	1603	1683	1383	1603	1683	1380	1603	1683	1394
Grp Volume(v), veh/h	112	0	401	156	364	110	143	373	90	128	360	37
Grp Sat Flow(s),veh/h/ln	1603	0	1652	1603	1683	1383	1603	1683	1380	1603	1683	1394
Q Serve(g_s), s	7.3	0.0	23.9	10.1	19.8	6.2	9.8	24.0	5.9	8.8	23.1	1.7
Cycle Q Clear(g_c), s	7.3	0.0	23.9	10.1	19.8	6.2	9.8	24.0	5.9	8.8	23.1	1.7
Prop In Lane	1.00		0.08	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	217	0	550	260	607	499	168	415	340	152	409	339
V/C Ratio(X)	0.52	0.00	0.73	0.60	0.60	0.22	0.85	0.90	0.26	0.84	0.88	0.11
Avail Cap(c_a), veh/h	217	0	550	260	607	499	186	470	386	172	452	375
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	45.5	0.0	32.9	43.5	29.2	24.9	49.3	40.8	34.0	49.9	40.8	16.9
Incr Delay (d2), s/veh	1.0	0.0	8.2	2.7	4.3	1.0	25.8	19.3	0.6	24.9	17.5	0.2
Initial Q Delay(d3),s/veh	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.8	0.0	10.8	4.2	8.7	0.1	5.1	12.1	2.0	4.6	11.5	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	51.5	0.0	41.1	46.2	33.5	25.9	75.1	60.2	34.6	74.8	58.3	17.1
LnGrp LOS	D	A	D	D	C	C	E	E	C	E	E	B
Approach Vol, veh/h		513			630			606			525	
Approach Delay, s/veh		43.4			35.3			59.9			59.4	
Approach LOS		D			D			E			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	22.2	42.0	15.7	32.1	19.2	45.0	15.5	32.3
Change Period (Y+Rc), s	* 4	4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7
Max Green Setting (Gmax), s	* 14	37.3	13.0	* 30	11.0	40.4	* 12	* 31
Max Q Clear Time (g_c+I1), s	12.1	25.9	11.8	25.1	9.3	21.8	10.8	26.0
Green Ext Time (p_c), s	0.0	1.7	0.0	1.4	0.0	2.1	0.0	1.6

Intersection Summary	
HCM 6th Ctrl Delay	49.3
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	800	16	67	645	73	9	7	132	129	5	25
Future Volume (veh/h)	8	800	16	67	645	73	9	7	132	129	5	25
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	9	851	17	71	686	78	10	7	140	137	5	27
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	281	1167	23	113	889	101	43	24	270	212	11	31
Arrive On Green	0.16	0.64	0.64	0.06	0.54	0.54	0.18	0.18	0.18	0.18	0.18	0.18
Sat Flow, veh/h	1781	1827	37	1781	1649	188	49	133	1498	857	59	174
Grp Volume(v), veh/h	9	0	868	71	0	764	157	0	0	169	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1864	1781	0	1837	1680	0	0	1091	0	0
Q Serve(g_s), s	0.5	0.0	35.3	4.4	0.0	36.8	0.0	0.0	0.0	7.5	0.0	0.0
Cycle Q Clear(g_c), s	0.5	0.0	35.3	4.4	0.0	36.8	9.9	0.0	0.0	17.4	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.10	0.06		0.89	0.81		0.16
Lane Grp Cap(c), veh/h	281	0	1190	113	0	990	336	0	0	254	0	0
V/C Ratio(X)	0.03	0.00	0.73	0.63	0.00	0.77	0.47	0.00	0.00	0.66	0.00	0.00
Avail Cap(c_a), veh/h	281	0	1190	127	0	990	481	0	0	373	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.67	0.00	0.67	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	39.9	0.0	13.7	51.1	0.0	20.4	41.7	0.0	0.0	45.6	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	3.9	5.3	0.0	4.0	1.0	0.0	0.0	3.0	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	15.2	2.1	0.0	16.3	4.1	0.0	0.0	4.8	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	40.0	0.0	17.6	56.5	0.0	24.3	42.7	0.0	0.0	48.5	0.0	0.0
LnGrp LOS	D	A	B	E	A	C	D	A	A	D	A	A
Approach Vol, veh/h		877			835			157			169	
Approach Delay, s/veh		17.9			27.0			42.7			48.5	
Approach LOS		B			C			D			D	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	11.1	76.1		24.8	22.2	65.0		24.8				
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6				
Max Green Setting (Gmax), s	8.0	60.4		30.4	8.0	* 60		30.4				
Max Q Clear Time (g_c+I1), s	6.4	37.3		19.4	2.5	38.8		11.9				
Green Ext Time (p_c), s	0.0	7.6		0.7	0.0	6.2		0.9				

Intersection Summary			
HCM 6th Ctrl Delay			26.1
HCM 6th LOS			C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	107	183	402	11	186	18	342	356	24	12	296	75
Future Volume (veh/h)	107	183	402	11	186	18	342	356	24	12	296	75
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	114	195	428	12	198	18	364	379	26	13	315	74
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	119	203	824	258	244	22	617	971	891	34	249	278
Arrive On Green	0.17	0.17	0.17	0.14	0.14	0.14	0.35	0.56	0.56	0.18	0.18	0.18
Sat Flow, veh/h	678	1159	1574	1781	1685	153	1781	1722	1579	21	1393	1554
Grp Volume(v), veh/h	309	0	428	12	0	216	364	379	26	328	0	74
Grp Sat Flow(s),veh/h/ln	1836	0	1574	1781	0	1838	1781	1722	1579	1414	0	1554
Q Serve(g_s), s	20.7	0.0	0.0	0.7	0.0	14.1	20.8	15.3	0.9	6.9	0.0	5.1
Cycle Q Clear(g_c), s	20.7	0.0	0.0	0.7	0.0	14.1	20.8	15.3	0.9	22.2	0.0	5.1
Prop In Lane	0.37		1.00	1.00		0.08	1.00		1.00	0.04		1.00
Lane Grp Cap(c), veh/h	321	0	824	258	0	266	617	971	891	283	0	278
V/C Ratio(X)	0.96	0.00	0.52	0.05	0.00	0.81	0.59	0.39	0.03	1.16	0.00	0.27
Avail Cap(c_a), veh/h	321	0	824	398	0	411	617	971	891	283	0	278
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.59	0.00	0.59	1.00	0.00	1.00	1.00	1.00	1.00	0.64	0.00	0.64
Uniform Delay (d), s/veh	50.7	0.0	19.5	45.7	0.0	51.4	33.3	15.1	12.0	51.6	0.0	43.9
Incr Delay (d2), s/veh	29.1	0.0	0.5	0.1	0.0	5.6	1.0	1.2	0.1	93.6	0.0	1.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	12.2	0.0	8.1	0.3	0.0	7.0	9.2	6.2	0.3	16.3	0.0	2.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	79.9	0.0	19.9	45.7	0.0	57.0	34.3	16.3	12.0	145.2	0.0	45.4
LnGrp LOS	E	A	B	D	A	E	C	B	B	F	A	D
Approach Vol, veh/h		737			228			769			402	
Approach Delay, s/veh		45.1			56.4			24.7			126.8	
Approach LOS		D			E			C			F	
Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		27.0	47.7	27.0		22.3		74.7				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 22	* 34	* 22		27.7		* 60				
Max Q Clear Time (g_c+I1), s		22.7	22.8	24.2		16.1		17.3				
Green Ext Time (p_c), s		0.0	0.3	0.0		0.8		1.6				
Intersection Summary												
HCM 6th Ctrl Delay			54.3									
HCM 6th LOS			D									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	219	246	337	302	129	74	481	517	469	154	400	211
Future Volume (veh/h)	219	246	337	302	129	74	481	517	469	154	400	211
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	231	259	341	227	263	34	506	544	491	162	421	197
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	326	325	290	280	294	410	529	1361	658	187	485	225
Arrive On Green	0.18	0.18	0.18	0.16	0.16	0.16	0.30	0.42	0.42	0.10	0.22	0.22
Sat Flow, veh/h	1781	1777	1585	1781	1870	1555	1781	3272	1582	1781	2172	1006
Grp Volume(v), veh/h	231	259	341	227	263	34	506	544	491	162	316	302
Grp Sat Flow(s),veh/h/ln	1781	1777	1585	1781	1870	1555	1781	1636	1582	1781	1636	1541
Q Serve(g_s), s	16.8	19.2	25.2	17.0	19.0	2.3	38.4	16.0	36.2	12.3	25.6	26.1
Cycle Q Clear(g_c), s	16.8	19.2	25.2	17.0	19.0	2.3	38.4	16.0	36.2	12.3	25.6	26.1
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.65
Lane Grp Cap(c), veh/h	326	325	290	280	294	410	529	1361	658	187	366	345
V/C Ratio(X)	0.71	0.80	1.18	0.81	0.90	0.08	0.96	0.40	0.75	0.87	0.86	0.88
Avail Cap(c_a), veh/h	326	325	290	309	324	436	608	1394	674	297	416	391
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	52.8	53.8	56.3	56.1	57.0	38.4	47.5	28.2	34.1	60.7	51.5	51.7
Incr Delay (d2), s/veh	6.6	12.6	109.4	13.3	23.7	0.1	23.4	0.3	4.8	8.9	16.5	18.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	8.2	9.8	19.0	8.6	10.8	0.9	20.1	6.3	14.4	6.0	12.1	11.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	59.4	66.4	165.7	69.4	80.7	38.4	71.0	28.5	38.9	69.7	68.0	70.5
LnGrp LOS	E	E	F	E	F	D	E	C	D	E	E	E
Approach Vol, veh/h		831			524			1541			780	
Approach Delay, s/veh		105.2			73.0			45.8			69.3	
Approach LOS		F			E			D			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	44.9	36.1		26.7	18.4	62.6		30.0				
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8				
Max Green Setting (Gmax), s	47.0	* 35		* 24	23.0	58.7		25.2				
Max Q Clear Time (g_c+I1), s	40.4	28.1		21.0	14.3	38.2		27.2				
Green Ext Time (p_c), s	0.5	2.7		0.6	0.1	7.8		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			68.1
HCM 6th LOS			E

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	167	680	24	18	354	248	24	25	42	408	30	125
Future Volume (veh/h)	167	680	24	18	354	248	24	25	42	408	30	125
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.97	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	169	687	17	18	358	204	24	25	12	412	30	118
Peak Hour Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	204	1550	38	60	777	434	72	64	16	377	20	772
Arrive On Green	0.11	0.44	0.44	0.03	0.36	0.36	0.38	0.38	0.38	0.38	0.38	0.38
Sat Flow, veh/h	1781	3542	88	1781	2176	1215	0	168	41	733	53	1562
Grp Volume(v), veh/h	169	345	359	18	291	271	61	0	0	442	0	118
Grp Sat Flow(s),veh/h/ln	1781	1777	1852	1781	1777	1615	209	0	0	786	0	1562
Q Serve(g_s), s	6.5	9.5	9.5	0.7	8.8	9.1	0.0	0.0	0.0	0.0	0.0	2.9
Cycle Q Clear(g_c), s	6.5	9.5	9.5	0.7	8.8	9.1	26.5	0.0	0.0	26.5	0.0	2.9
Prop In Lane	1.00		0.05	1.00		0.75	0.39		0.20	0.93		1.00
Lane Grp Cap(c), veh/h	204	778	811	60	635	577	151	0	0	397	0	772
V/C Ratio(X)	0.83	0.44	0.44	0.30	0.46	0.47	0.40	0.00	0.00	1.11	0.00	0.15
Avail Cap(c_a), veh/h	204	778	811	204	635	577	151	0	0	397	0	772
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.59	0.59	0.59	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	30.3	13.7	13.7	33.0	17.3	17.4	17.9	0.0	0.0	25.1	0.0	9.7
Incr Delay (d2), s/veh	14.7	1.1	1.0	1.0	2.4	2.7	0.6	0.0	0.0	79.5	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.5	3.8	3.9	0.3	3.6	3.4	0.6	0.0	0.0	15.3	0.0	0.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	45.0	14.8	14.8	34.0	19.7	20.1	18.6	0.0	0.0	104.6	0.0	9.8
LnGrp LOS	D	B	B	C	B	C	B	A	A	F	A	A
Approach Vol, veh/h		873			580			61			560	
Approach Delay, s/veh		20.6			20.3			18.6			84.6	
Approach LOS		C			C			B			F	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	6.4	35.6		31.1	12.0	30.0		31.1				
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6				
Max Green Setting (Gmax), s	8.0	22.1		26.4	8.0	22.1		* 27				
Max Q Clear Time (g_c+I1), s	2.7	11.5		28.5	8.5	11.1		28.5				
Green Ext Time (p_c), s	0.0	3.9		0.0	0.0	2.9		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			37.8
HCM 6th LOS			D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	12.3
Intersection LOS	B

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	588	232	146	857
Demand Flow Rate, veh/h	600	237	149	874
Vehicles Circulating, veh/h	203	693	642	163
Vehicles Exiting, veh/h	834	98	161	767
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	9.6	10.0	7.5	15.5
Approach LOS	A	B	A	C

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	600	237	149	874
Cap Entry Lane, veh/h	1122	681	717	1169
Entry HV Adj Factor	0.980	0.979	0.981	0.981
Flow Entry, veh/h	588	232	146	857
Cap Entry, veh/h	1099	666	703	1146
V/C Ratio	0.535	0.348	0.208	0.748
Control Delay, s/veh	9.6	10.0	7.5	15.5
LOS	A	B	A	C
95th %tile Queue, veh	3	2	1	7

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	219	246	337	302	129	74	481	517	469	154	400	211
Future Volume (veh/h)	219	246	337	302	129	74	481	517	469	154	400	211
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	231	259	252	318	136	57	506	544	248	162	421	146
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	265	324	749	403	175	73	534	1293	625	223	528	181
Arrive On Green	0.15	0.17	0.17	0.12	0.14	0.14	0.30	0.40	0.40	0.13	0.22	0.22
Sat Flow, veh/h	1781	1870	1585	3456	1243	521	1781	3272	1581	1781	2390	820
Grp Volume(v), veh/h	231	259	252	318	0	193	506	544	248	162	287	280
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	0	1763	1781	1636	1581	1781	1636	1574
Q Serve(g_s), s	13.0	13.7	10.2	9.2	0.0	10.9	28.6	12.4	11.6	9.0	17.0	17.3
Cycle Q Clear(g_c), s	13.0	13.7	10.2	9.2	0.0	10.9	28.6	12.4	11.6	9.0	17.0	17.3
Prop In Lane	1.00		1.00	1.00		0.30	1.00		1.00	1.00		0.52
Lane Grp Cap(c), veh/h	265	324	749	403	0	248	534	1293	625	223	361	347
V/C Ratio(X)	0.87	0.80	0.34	0.79	0.00	0.78	0.95	0.42	0.40	0.73	0.79	0.81
Avail Cap(c_a), veh/h	381	406	819	740	0	377	572	1321	639	347	458	441
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	42.8	40.8	17.0	44.2	0.0	42.6	35.2	22.5	22.3	43.3	37.8	38.0
Incr Delay (d2), s/veh	12.7	8.0	0.2	2.6	0.0	4.3	23.8	0.3	0.6	1.7	8.5	9.6
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	6.7	7.0	3.8	4.0	0.0	4.9	15.3	4.6	4.4	4.0	7.5	7.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	55.5	48.8	17.2	46.8	0.0	46.9	59.0	22.9	22.9	45.0	46.3	47.6
LnGrp LOS	E	D	B	D	A	D	E	C	C	D	D	D
Approach Vol, veh/h		742			511			1298			729	
Approach Delay, s/veh		40.2			46.8			37.0			46.5	
Approach LOS		D			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	34.8	28.0	20.4	19.6	16.9	45.9	17.1	22.9
Change Period (Y+Rc), s	4.0	* 5.3	* 5.1	* 5.1	4.0	5.3	* 5.1	* 5.1
Max Green Setting (Gmax), s	33.0	* 29	* 22	* 22	20.0	41.5	* 22	* 22
Max Q Clear Time (g_c+I1), s	30.6	19.3	15.0	12.9	11.0	14.4	11.2	15.7
Green Ext Time (p_c), s	0.3	3.1	0.3	0.5	0.1	6.7	0.6	0.4

Intersection Summary	
HCM 6th Ctrl Delay	41.3
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	223	647	87	116	642	166	35	325	211	116	376	302
Future Volume (veh/h)	223	647	87	116	642	166	35	325	211	116	376	302
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	253	735	99	132	730	189	40	369	240	132	427	343
Peak Hour Factor	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88	0.88
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	397	1451	195	157	896	232	51	423	498	156	533	805
Arrive On Green	0.22	0.46	0.46	0.09	0.32	0.32	0.03	0.23	0.23	0.09	0.29	0.29
Sat Flow, veh/h	1781	3147	424	1781	2794	723	1781	1870	1585	1781	1870	1585
Grp Volume(v), veh/h	253	415	419	132	464	455	40	369	240	132	427	343
Grp Sat Flow(s),veh/h/ln	1781	1777	1794	1781	1777	1740	1781	1870	1585	1781	1870	1585
Q Serve(g_s), s	16.7	21.3	21.4	9.5	31.2	31.2	2.9	24.7	15.9	9.5	27.5	2.1
Cycle Q Clear(g_c), s	16.7	21.3	21.4	9.5	31.2	31.2	2.9	24.7	15.9	9.5	27.5	2.1
Prop In Lane	1.00		0.24	1.00		0.42	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	397	819	827	157	570	558	51	423	498	156	533	805
V/C Ratio(X)	0.64	0.51	0.51	0.84	0.81	0.81	0.78	0.87	0.48	0.84	0.80	0.43
Avail Cap(c_a), veh/h	397	819	827	233	570	558	69	495	559	178	616	875
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	45.7	24.6	24.7	58.4	40.6	40.6	62.7	48.5	36.0	58.4	43.0	10.9
Incr Delay (d2), s/veh	2.6	2.2	2.2	10.6	12.1	12.3	23.5	15.0	1.0	24.2	7.2	0.5
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	7.8	9.6	9.7	4.8	15.7	15.4	1.7	13.4	6.4	5.4	13.9	4.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	48.3	26.9	26.9	69.0	52.7	52.9	86.2	63.4	37.0	82.6	50.3	11.4
LnGrp LOS	D	C	C	E	D	D	F	E	D	F	D	B
Approach Vol, veh/h		1087			1051			649			902	
Approach Delay, s/veh		31.9			54.9			55.1			40.2	
Approach LOS		C			D			E			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	15.5	64.5	7.7	42.3	33.6	46.4	15.4	34.6
Change Period (Y+Rc), s	4.0	4.6	4.0	* 5.2	* 4.6	* 4.7	4.0	* 5.2
Max Green Setting (Gmax), s	17.0	47.8	5.0	* 43	* 23	* 42	13.0	* 34
Max Q Clear Time (g_c+I1), s	11.5	23.4	4.9	29.5	18.7	33.2	11.5	26.7
Green Ext Time (p_c), s	0.1	8.4	0.0	4.8	0.2	4.8	0.0	2.7

Intersection Summary	
HCM 6th Ctrl Delay	44.5
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	69	861	222	62	878	76	217	192	52	76	282	88
Future Volume (veh/h)	69	861	222	62	878	76	217	192	52	76	282	88
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	74	926	235	67	944	79	233	206	51	82	303	95
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	153	976	247	125	1096	92	248	200	50	78	287	90
Arrive On Green	0.09	0.37	0.37	0.07	0.35	0.35	0.05	0.05	0.05	0.27	0.27	0.27
Sat Flow, veh/h	1781	2650	671	1781	3152	264	1697	1370	339	290	1071	336
Grp Volume(v), veh/h	74	591	570	67	507	516	233	0	257	480	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1629	1781	1692	1723	1697	0	1709	1696	0	0
Q Serve(g_s), s	5.1	44.0	44.2	4.7	36.3	36.3	17.8	0.0	19.0	34.9	0.0	0.0
Cycle Q Clear(g_c), s	5.1	44.0	44.2	4.7	36.3	36.3	17.8	0.0	19.0	34.9	0.0	0.0
Prop In Lane	1.00		0.41	1.00		0.15	1.00		0.20	0.17		0.20
Lane Grp Cap(c), veh/h	153	623	600	125	588	599	248	0	250	455	0	0
V/C Ratio(X)	0.48	0.95	0.95	0.54	0.86	0.86	0.94	0.00	1.03	1.05	0.00	0.00
Avail Cap(c_a), veh/h	164	623	600	137	588	599	248	0	250	455	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.33	0.33	0.33	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.91	0.00	0.91	1.00	0.00	0.00
Uniform Delay (d), s/veh	56.6	39.8	39.9	58.4	39.5	39.5	61.3	0.0	61.9	47.6	0.0	0.0
Incr Delay (d2), s/veh	0.9	25.2	26.4	1.3	15.3	15.0	38.1	0.0	62.1	57.2	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.4	22.7	22.1	2.2	17.4	17.7	10.8	0.0	13.1	21.9	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	57.5	65.0	66.3	59.7	54.8	54.5	99.4	0.0	123.9	104.7	0.0	0.0
LnGrp LOS	E	E	E	E	D	D	F	A	F	F	A	A
Approach Vol, veh/h		1235			1090			490			480	
Approach Delay, s/veh		65.2			55.0			112.2			104.7	
Approach LOS		E			D			F			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	13.1	52.7		40.2	16.0	49.8		24.0
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0
Max Green Setting (Gmax), s	10.0	* 47		* 35	12.0	* 45		19.0
Max Q Clear Time (g_c+I1), s	6.7	46.2		36.9	7.1	38.3		21.0
Green Ext Time (p_c), s	0.0	0.5		0.0	0.0	2.7		0.0

Intersection Summary	
HCM 6th Ctrl Delay	74.6
HCM 6th LOS	E

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	32	665	288	451	851	123	184	166	270	149	139	28
Future Volume (veh/h)	32	665	288	451	851	123	184	166	270	149	139	28
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.99	1.00		0.99	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	34	715	310	485	915	132	198	178	290	160	149	30
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	87	643	279	711	1949	281	199	179	326	173	161	283
Arrive On Green	0.05	0.27	0.27	0.40	0.63	0.63	0.21	0.21	0.21	0.18	0.18	0.18
Sat Flow, veh/h	1781	2408	1044	1781	3113	449	960	863	1571	944	879	1549
Grp Volume(v), veh/h	34	527	498	485	522	525	376	0	290	309	0	30
Grp Sat Flow(s),veh/h/ln	1781	1777	1675	1781	1777	1786	1822	0	1571	1823	0	1549
Q Serve(g_s), s	2.4	34.7	34.7	29.2	20.2	20.2	26.8	0.0	23.3	21.7	0.0	2.1
Cycle Q Clear(g_c), s	2.4	34.7	34.7	29.2	20.2	20.2	26.8	0.0	23.3	21.7	0.0	2.1
Prop In Lane	1.00		0.62	1.00		0.25	0.53		1.00	0.52		1.00
Lane Grp Cap(c), veh/h	87	474	447	711	1113	1118	378	0	326	333	0	283
V/C Ratio(X)	0.39	1.11	1.11	0.68	0.47	0.47	0.99	0.00	0.89	0.93	0.00	0.11
Avail Cap(c_a), veh/h	123	474	447	711	1113	1118	378	0	326	341	0	290
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	59.9	47.6	47.7	32.3	12.9	12.9	51.4	0.0	50.0	52.3	0.0	44.3
Incr Delay (d2), s/veh	1.1	75.6	76.8	2.2	1.4	1.4	44.4	0.0	23.7	29.8	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.1	25.2	23.9	13.0	8.3	8.3	16.9	0.0	11.4	12.7	0.0	0.8
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	61.0	123.2	124.5	34.5	14.3	14.3	95.8	0.0	73.8	82.0	0.0	44.3
LnGrp LOS	E	F	F	C	B	B	F	A	E	F	A	D
Approach Vol, veh/h		1059			1532			666			339	
Approach Delay, s/veh		121.8			20.7			86.2			78.7	
Approach LOS		F			C			F			E	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	57.4	39.4		28.5	9.9	87.0		31.6				
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	* 27	* 35		* 24	9.0	* 52		27.0				
Max Q Clear Time (g_c+I1), s	31.2	36.7		23.7	4.4	22.2		28.8				
Green Ext Time (p_c), s	0.0	0.0		0.1	0.0	3.0		0.0				

Intersection Summary	
HCM 6th Ctrl Delay	68.1
HCM 6th LOS	E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	50	312	4	18	448	29	122	203	228	201	75	117
Future Volume (veh/h)	50	312	4	18	448	29	122	203	228	201	75	117
Initial Q (Qb), veh	5	0	0	1	0	0	0	0	0	0	3	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.97	1.00		0.97	1.00		0.95
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	59	367	1	21	527	8	144	239	97	236	88	104
Peak Hour Factor	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85	0.85
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	107	585	2	203	703	579	329	322	265	260	229	200
Arrive On Green	0.06	0.35	0.35	0.13	0.42	0.42	0.22	0.19	0.19	0.16	0.13	0.13
Sat Flow, veh/h	1603	1678	5	1603	1683	1386	1603	1683	1386	1603	1683	1362
Grp Volume(v), veh/h	59	0	368	21	527	8	144	239	97	236	88	104
Grp Sat Flow(s),veh/h/ln	1603	0	1682	1603	1683	1386	1603	1683	1386	1603	1683	1362
Q Serve(g_s), s	3.8	0.0	19.2	1.2	27.8	0.2	8.1	14.0	6.4	15.2	5.0	6.3
Cycle Q Clear(g_c), s	3.8	0.0	19.2	1.2	27.8	0.2	8.1	14.0	6.4	15.2	5.0	6.3
Prop In Lane	1.00		0.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	107	0	586	203	703	579	329	322	265	260	229	200
V/C Ratio(X)	0.55	0.00	0.63	0.10	0.75	0.01	0.44	0.74	0.37	0.91	0.38	0.52
Avail Cap(c_a), veh/h	122	0	586	203	706	581	352	417	343	260	414	335
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	48.1	0.0	28.5	40.7	25.9	6.1	36.5	40.0	36.9	43.2	41.6	28.3
Incr Delay (d2), s/veh	1.6	0.0	5.0	0.1	4.2	0.0	0.3	6.2	1.2	32.2	1.5	3.0
Initial Q Delay(d3),s/veh	34.8	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	2.0	0.0
%ile BackOfQ(50%),veh/ln	3.1	0.0	8.5	0.6	11.8	0.1	3.2	6.3	2.2	8.3	2.6	2.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	84.5	0.0	33.5	41.0	30.2	6.1	36.9	46.2	38.1	75.4	45.1	31.3
LnGrp LOS	F	A	C	D	C	A	D	D	D	E	D	C
Approach Vol, veh/h		427			556			480			428	
Approach Delay, s/veh		40.6			30.2			41.8			58.5	
Approach LOS		D			C			D			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	17.9	41.3	27.0	18.8	10.6	48.6	21.0	24.8
Change Period (Y+Rc), s	* 4.6	* 4.7	* 4	4.9	4.0	4.6	* 4	4.7
Max Green Setting (Gmax), s	* 8	* 37	* 17	25.8	8.0	36.7	* 17	26.0
Max Q Clear Time (g_c+I1), s	3.2	21.2	10.1	8.3	5.8	29.8	17.2	16.0
Green Ext Time (p_c), s	0.0	1.7	0.1	1.1	0.0	1.7	0.0	1.6

Intersection Summary	
HCM 6th Ctrl Delay	41.9
HCM 6th LOS	D

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	9	689	11	74	630	78	7	7	72	55	4	4
Future Volume (veh/h)	9	689	11	74	630	78	7	7	72	55	4	4
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	10	757	12	81	692	86	8	8	79	60	4	4
Peak Hour Factor	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91	0.91
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	328	1074	17	379	1000	124	43	16	106	151	10	6
Arrive On Green	0.18	0.58	0.58	0.21	0.61	0.61	0.08	0.08	0.08	0.08	0.08	0.08
Sat Flow, veh/h	1781	1836	29	1781	1631	203	76	203	1379	1120	132	78
Grp Volume(v), veh/h	10	0	769	81	0	778	95	0	0	68	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1865	1781	0	1834	1658	0	0	1330	0	0
Q Serve(g_s), s	0.5	0.0	30.6	3.9	0.0	29.9	0.6	0.0	0.0	0.0	0.0	0.0
Cycle Q Clear(g_c), s	0.5	0.0	30.6	3.9	0.0	29.9	5.8	0.0	0.0	5.2	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.11	0.08		0.83	0.88		0.06
Lane Grp Cap(c), veh/h	328	0	1091	379	0	1125	165	0	0	167	0	0
V/C Ratio(X)	0.03	0.00	0.71	0.21	0.00	0.69	0.58	0.00	0.00	0.41	0.00	0.00
Avail Cap(c_a), veh/h	328	0	1091	379	0	1125	351	0	0	325	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.30	0.00	0.30	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	35.1	0.0	15.4	34.1	0.0	13.6	47.5	0.0	0.0	47.1	0.0	0.0
Incr Delay (d2), s/veh	0.0	0.0	3.8	0.1	0.0	1.1	3.2	0.0	0.0	1.6	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	13.5	1.7	0.0	12.0	2.6	0.0	0.0	1.8	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	35.2	0.0	19.2	34.2	0.0	14.7	50.6	0.0	0.0	48.7	0.0	0.0
LnGrp LOS	D	A	B	C	A	B	D	A	A	D	A	A
Approach Vol, veh/h		779			859			95			68	
Approach Delay, s/veh		19.4			16.5			50.6			48.7	
Approach LOS		B			B			D			D	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	26.3	66.0		12.7	23.3	69.0		12.7				
Change Period (Y+Rc), s	4.0	4.6		4.6	4.0	4.6		4.6				
Max Green Setting (Gmax), s	10.0	61.4		20.4	7.0	64.4		20.4				
Max Q Clear Time (g_c+I1), s	5.9	32.6		7.2	2.5	31.9		7.8				
Green Ext Time (p_c), s	0.1	6.8		0.2	0.0	7.2		0.3				
Intersection Summary												
HCM 6th Ctrl Delay			20.8									
HCM 6th LOS			C									

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	47	180	489	22	190	8	370	407	52	7	509	59
Future Volume (veh/h)	47	180	489	22	190	8	370	407	52	7	509	59
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.93	1.00		0.98	1.00		0.96
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	52	200	543	24	211	5	411	452	57	8	566	56
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90	0.90
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	55	211	590	221	225	5	411	1070	963	31	614	550
Arrive On Green	0.14	0.14	0.14	0.12	0.12	0.12	0.23	0.62	0.62	0.36	0.36	0.36
Sat Flow, veh/h	382	1469	1561	1781	1816	43	1781	1722	1550	8	1705	1527
Grp Volume(v), veh/h	252	0	543	24	0	216	411	452	57	574	0	56
Grp Sat Flow(s),veh/h/ln	1851	0	1561	1781	0	1859	1781	1722	1550	1713	0	1527
Q Serve(g_s), s	17.5	0.0	18.7	1.6	0.0	15.0	30.0	17.5	1.9	11.3	0.0	3.2
Cycle Q Clear(g_c), s	17.5	0.0	18.7	1.6	0.0	15.0	30.0	17.5	1.9	41.9	0.0	3.2
Prop In Lane	0.21		1.00	1.00		0.02	1.00		1.00	0.01		1.00
Lane Grp Cap(c), veh/h	266	0	590	221	0	230	411	1070	963	645	0	550
V/C Ratio(X)	0.95	0.00	0.92	0.11	0.00	0.94	1.00	0.42	0.06	0.89	0.00	0.10
Avail Cap(c_a), veh/h	266	0	590	221	0	230	411	1070	963	645	0	550
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.81	0.00	0.81	1.00	0.00	1.00	1.00	1.00	1.00	0.19	0.00	0.19
Uniform Delay (d), s/veh	55.2	0.0	38.9	50.6	0.0	56.5	50.0	12.6	9.7	40.0	0.0	27.6
Incr Delay (d2), s/veh	35.9	0.0	17.1	0.2	0.0	42.2	44.3	1.2	0.1	4.0	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	10.9	0.0	19.1	0.7	0.0	9.8	18.4	7.0	0.7	18.1	0.0	1.2
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	91.1	0.0	56.0	50.7	0.0	98.7	94.3	13.8	9.8	43.9	0.0	27.7
LnGrp LOS	F	A	E	D	A	F	F	B	A	D	A	C
Approach Vol, veh/h		795			240			920			630	
Approach Delay, s/veh		67.1			93.9			49.6			42.5	
Approach LOS		E			F			D			D	

Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		24.0	34.0	51.6		20.4		85.6				
Change Period (Y+Rc), s		* 5.3	4.0	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 19	30.0	* 47		16.1		* 81				
Max Q Clear Time (g_c+I1), s		20.7	32.0	43.9		17.0		19.5				
Green Ext Time (p_c), s		0.0	0.0	0.9		0.0		2.1				

Intersection Summary			
HCM 6th Ctrl Delay			57.3
HCM 6th LOS			E

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Future Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	234	129	437	271	308	117	498	718	274	123	731	217
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	235	235	209	302	317	420	465	1480	716	176	721	214
Arrive On Green	0.13	0.13	0.13	0.17	0.17	0.17	0.26	0.45	0.45	0.10	0.29	0.29
Sat Flow, veh/h	1781	1777	1585	1781	1870	1557	1781	3272	1582	1781	2487	738
Grp Volume(v), veh/h	234	129	437	271	308	117	498	718	274	123	481	467
Grp Sat Flow(s),veh/h/ln	1781	1777	1585	1781	1870	1557	1781	1636	1582	1781	1636	1589
Q Serve(g_s), s	17.1	8.9	17.2	19.4	21.3	7.7	34.0	20.1	14.9	8.7	37.8	37.8
Cycle Q Clear(g_c), s	17.1	8.9	17.2	19.4	21.3	7.7	34.0	20.1	14.9	8.7	37.8	37.8
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.46
Lane Grp Cap(c), veh/h	235	235	209	302	317	420	465	1480	716	176	475	461
V/C Ratio(X)	1.00	0.55	2.09	0.90	0.97	0.28	1.07	0.49	0.38	0.70	1.01	1.01
Avail Cap(c_a), veh/h	235	235	209	302	317	420	465	1480	716	219	475	461
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	56.5	52.9	56.6	53.0	53.8	37.7	48.2	25.0	23.6	56.9	46.2	46.3
Incr Delay (d2), s/veh	57.2	2.3	505.9	27.1	42.5	0.3	62.2	0.4	0.5	4.5	44.6	45.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	11.5	4.2	36.3	10.8	13.6	3.0	22.7	7.7	5.6	4.1	21.0	20.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	113.7	55.2	562.4	80.1	96.2	38.0	110.3	25.4	24.1	61.4	90.9	91.5
LnGrp LOS	F	E	F	F	F	D	F	C	C	E	F	F
Approach Vol, veh/h		800			696			1490			1071	
Approach Delay, s/veh		349.4			80.1			53.5			87.8	
Approach LOS		F			F			D			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	38.0	43.1		27.2	16.8	64.3		22.0
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8
Max Green Setting (Gmax), s	34.0	* 38		* 22	16.0	55.5		17.2
Max Q Clear Time (g_c+I1), s	36.0	39.8		23.3	10.7	22.1		19.2
Green Ext Time (p_c), s	0.0	0.0		0.0	0.1	9.6		0.0

Intersection Summary	
HCM 6th Ctrl Delay	125.5
HCM 6th LOS	F

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	102	377	26	6	475	187	34	5	5	122	9	168
Future Volume (veh/h)	102	377	26	6	475	187	34	5	5	122	9	168
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	110	405	19	6	511	164	37	5	0	131	10	179
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	148	1697	79	20	1104	352	292	30	0	442	28	439
Arrive On Green	0.08	0.49	0.49	0.01	0.42	0.42	0.20	0.20	0.00	0.20	0.20	0.20
Sat Flow, veh/h	1781	3456	162	1781	2634	840	716	154	0	1454	143	1553
Grp Volume(v), veh/h	110	208	216	6	344	331	42	0	0	141	0	179
Grp Sat Flow(s),veh/h/ln	1781	1777	1841	1781	1777	1697	870	0	0	1597	0	1553
Q Serve(g_s), s	2.7	3.0	3.0	0.2	6.3	6.3	0.9	0.0	0.0	0.0	0.0	4.2
Cycle Q Clear(g_c), s	2.7	3.0	3.0	0.2	6.3	6.3	4.0	0.0	0.0	3.1	0.0	4.2
Prop In Lane	1.00		0.09	1.00		0.50	0.88		0.00	0.93		1.00
Lane Grp Cap(c), veh/h	148	872	904	20	745	711	323	0	0	470	0	439
V/C Ratio(X)	0.74	0.24	0.24	0.30	0.46	0.47	0.13	0.00	0.00	0.30	0.00	0.41
Avail Cap(c_a), veh/h	198	872	904	277	745	711	367	0	0	518	0	491
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.73	0.73	0.73	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	20.2	6.6	6.6	22.1	9.4	9.4	16.6	0.0	0.0	15.7	0.0	13.2
Incr Delay (d2), s/veh	4.5	0.5	0.5	3.1	2.1	2.2	0.1	0.0	0.0	0.1	0.0	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.2	1.0	1.0	0.1	2.1	2.1	0.4	0.0	0.0	1.1	0.0	1.3
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	24.7	7.1	7.1	25.1	11.5	11.6	16.6	0.0	0.0	15.8	0.0	13.4
LnGrp LOS	C	A	A	C	B	B	B	A	A	B	A	B
Approach Vol, veh/h		534			681			42			320	
Approach Delay, s/veh		10.7			11.7			16.6			14.5	
Approach LOS		B			B			B			B	

Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	4.5	27.0		13.5	7.7	23.8		13.5				
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6				
Max Green Setting (Gmax), s	7.0	14.1		10.4	5.0	16.1		* 11				
Max Q Clear Time (g_c+I1), s	2.2	5.0		6.2	4.7	8.3		6.0				
Green Ext Time (p_c), s	0.0	2.0		0.3	0.0	2.8		0.0				

Intersection Summary			
HCM 6th Ctrl Delay			12.0
HCM 6th LOS			B

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	10.7
Intersection LOS	B

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	432	296	150	795
Demand Flow Rate, veh/h	441	301	153	811
Vehicles Circulating, veh/h	212	544	523	172
Vehicles Exiting, veh/h	771	132	130	673
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	7.4	9.3	6.5	13.7
Approach LOS	A	A	A	B

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	441	301	153	811
Cap Entry Lane, veh/h	1112	792	809	1158
Entry HV Adj Factor	0.981	0.982	0.979	0.980
Flow Entry, veh/h	432	296	150	795
Cap Entry, veh/h	1090	778	792	1135
V/C Ratio	0.397	0.380	0.189	0.700
Control Delay, s/veh	7.4	9.3	6.5	13.7
LOS	A	A	A	B
95th %tile Queue, veh	2	2	1	6

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Future Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	234	129	394	364	178	162	498	718	137	123	731	217
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	230	341	703	423	157	143	465	1470	711	176	714	212
Arrive On Green	0.13	0.18	0.18	0.12	0.18	0.18	0.26	0.45	0.45	0.10	0.29	0.29
Sat Flow, veh/h	1781	1870	1585	3456	894	813	1781	3272	1582	1781	2487	738
Grp Volume(v), veh/h	234	129	394	364	0	340	498	718	137	123	481	467
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	0	1707	1781	1636	1582	1781	1636	1589
Q Serve(g_s), s	16.8	7.9	23.8	13.5	0.0	22.9	34.0	20.2	6.8	8.7	37.4	37.4
Cycle Q Clear(g_c), s	16.8	7.9	23.8	13.5	0.0	22.9	34.0	20.2	6.8	8.7	37.4	37.4
Prop In Lane	1.00		1.00	1.00		0.48	1.00		1.00	1.00		0.46
Lane Grp Cap(c), veh/h	230	341	703	423	0	300	465	1470	711	176	470	456
V/C Ratio(X)	1.02	0.38	0.56	0.86	0.00	1.13	1.07	0.49	0.19	0.70	1.02	1.02
Avail Cap(c_a), veh/h	230	341	703	552	0	300	465	1470	711	219	470	456
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	56.7	46.8	26.9	56.1	0.0	53.7	48.1	25.3	21.6	56.9	46.4	46.5
Incr Delay (d2), s/veh	64.3	0.5	0.9	9.7	0.0	92.9	62.2	0.4	0.2	4.5	47.8	48.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	11.7	3.8	9.3	6.4	0.0	17.4	22.7	7.7	2.6	4.1	21.2	20.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	121.1	47.3	27.7	65.8	0.0	146.6	110.3	25.7	21.8	61.4	94.2	94.8
LnGrp LOS	F	D	C	E	A	F	F	C	C	E	F	F
Approach Vol, veh/h		757			704			1353			1071	
Approach Delay, s/veh		59.9			104.8			56.4			90.7	
Approach LOS		E			F			E			F	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	38.0	42.7	21.6	28.0	16.8	63.9	20.7	28.9
Change Period (Y+Rc), s	4.0	* 5.3	* 4.8	* 5.1	4.0	5.3	* 4.8	* 5.1
Max Green Setting (Gmax), s	34.0	* 37	* 17	* 23	16.0	55.1	* 21	* 19
Max Q Clear Time (g_c+I1), s	36.0	39.4	18.8	24.9	10.7	22.2	15.5	9.9
Green Ext Time (p_c), s	0.0	0.0	0.0	0.0	0.1	8.5	0.5	0.1

Intersection Summary	
HCM 6th Ctrl Delay	75.3
HCM 6th LOS	E

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Future Volume (veh/h)	225	124	438	349	171	156	478	689	263	118	702	223
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1870
Adj Flow Rate, veh/h	234	129	333	364	178	84	498	718	137	123	731	116
Peak Hour Factor	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	2
Cap, veh/h	241	267	648	444	254	414	474	1322	843	229	871	636
Arrive On Green	0.14	0.14	0.14	0.13	0.14	0.14	0.27	0.40	0.40	0.13	0.27	0.27
Sat Flow, veh/h	1781	1870	1585	3456	1870	1550	1781	3272	1582	1781	3272	1585
Grp Volume(v), veh/h	234	129	333	364	178	84	498	718	137	123	731	116
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	1870	1550	1781	1636	1582	1781	1636	1585
Q Serve(g_s), s	12.8	6.2	13.9	10.0	8.9	4.1	26.0	16.4	4.3	6.3	20.6	4.6
Cycle Q Clear(g_c), s	12.8	6.2	13.9	10.0	8.9	4.1	26.0	16.4	4.3	6.3	20.6	4.6
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	241	267	648	444	254	414	474	1322	843	229	871	636
V/C Ratio(X)	0.97	0.48	0.51	0.82	0.70	0.20	1.05	0.54	0.16	0.54	0.84	0.18
Avail Cap(c_a), veh/h	241	331	703	644	421	553	474	1360	861	274	1002	699
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	42.1	38.6	21.6	41.5	40.3	27.9	35.8	22.2	11.7	39.9	33.9	18.9
Incr Delay (d2), s/veh	49.9	1.0	0.5	4.7	2.6	0.2	55.2	0.6	0.1	0.7	6.2	0.2
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	8.9	2.9	5.7	4.4	4.1	1.6	18.0	6.0	1.5	2.8	8.7	1.7
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	92.0	39.6	22.1	46.2	42.9	28.1	91.0	22.8	11.8	40.6	40.1	19.1
LnGrp LOS	F	D	C	D	D	C	F	C	B	D	D	B
Approach Vol, veh/h		696			626			1353			970	
Approach Delay, s/veh		48.8			42.8			46.8			37.6	
Approach LOS		D			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	30.0	31.3	18.0	18.4	16.5	44.8	17.4	19.0
Change Period (Y+Rc), s	4.0	* 5.3	* 4.8	* 5.1	4.0	5.3	* 4.8	* 5.1
Max Green Setting (Gmax), s	26.0	* 30	* 13	* 22	15.0	40.6	* 18	* 17
Max Q Clear Time (g_c+I1), s	28.0	22.6	14.8	10.9	8.3	18.4	12.0	8.2
Green Ext Time (p_c), s	0.0	3.3	0.0	0.7	0.1	7.4	0.5	0.1

Intersection Summary			
HCM 6th Ctrl Delay			44.1
HCM 6th LOS			D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	437	564	36	85	568	149	31	472	162	155	374	387
Future Volume (veh/h)	437	564	36	85	568	149	31	472	162	155	374	387
Initial Q (Qb), veh	0	0	0	0	0	0	0	4	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.98	1.00		0.99	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	470	606	35	91	611	146	33	508	146	167	402	397
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	479	1355	78	176	655	156	86	524	597	172	619	946
Arrive On Green	0.27	0.40	0.40	0.10	0.23	0.23	0.05	0.28	0.28	0.10	0.33	0.33
Sat Flow, veh/h	1781	3411	197	1781	2834	676	1781	1870	1573	1781	1870	1571
Grp Volume(v), veh/h	470	315	326	91	383	374	33	508	146	167	402	397
Grp Sat Flow(s),veh/h/ln	1781	1777	1831	1781	1777	1733	1781	1870	1573	1781	1870	1571
Q Serve(g_s), s	38.0	18.9	18.9	7.0	30.6	30.7	2.6	38.9	0.0	13.6	26.6	12.9
Cycle Q Clear(g_c), s	38.0	18.9	18.9	7.0	30.6	30.7	2.6	38.9	0.0	13.6	26.6	12.9
Prop In Lane	1.00		0.11	1.00		0.39	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	479	706	727	176	411	400	86	524	597	172	619	946
V/C Ratio(X)	0.98	0.45	0.45	0.52	0.93	0.94	0.38	0.97	0.24	0.97	0.65	0.42
Avail Cap(c_a), veh/h	479	706	727	184	411	400	86	524	597	172	619	946
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	52.6	32.0	32.0	62.1	54.6	54.7	66.9	52.2	30.8	65.3	41.3	7.3
Incr Delay (d2), s/veh	36.0	2.0	2.0	0.9	30.2	31.3	1.0	32.6	1.0	59.6	5.2	1.4
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	14.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	21.9	8.7	9.0	3.3	17.2	17.0	1.2	25.9	3.8	9.1	13.3	4.6
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	88.6	34.1	34.0	63.0	84.8	86.0	68.0	98.8	31.8	124.9	46.5	8.7
LnGrp LOS	F	C	C	E	F	F	E	F	C	F	D	A
Approach Vol, veh/h		1111			848			687			966	
Approach Delay, s/veh		57.1			83.0			83.1			44.5	
Approach LOS		E			F			F			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	19.0	62.2	11.0	52.8	43.0	38.2	18.0	45.8
Change Period (Y+Rc), s	4.7	* 4.6	* 4	4.8	4.0	* 4.7	* 4	5.2
Max Green Setting (Gmax), s	15.0	* 58	* 7	48.0	39.0	* 34	* 14	40.6
Max Q Clear Time (g_c+I1), s	9.0	20.9	4.6	28.6	40.0	32.7	15.6	40.9
Green Ext Time (p_c), s	0.0	6.9	0.0	5.9	0.0	0.5	0.0	0.0

Intersection Summary	
HCM 6th Ctrl Delay	64.8
HCM 6th LOS	E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	85	887	208	72	842	76	263	277	103	125	196	112
Future Volume (veh/h)	85	887	208	72	842	76	263	277	103	125	196	112
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.96	1.00		0.96	1.00		0.96	1.00		0.97
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1781	1781	1870	1781	1781	1781	1781	1781	1781	1781	1781
Adj Flow Rate, veh/h	88	914	205	74	868	75	271	286	102	129	202	114
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	8	8	2	8	8	8	8	8	8	8	8
Cap, veh/h	327	1224	274	117	1027	89	363	265	94	119	186	105
Arrive On Green	0.18	0.45	0.45	0.07	0.33	0.33	0.14	0.14	0.14	0.25	0.25	0.25
Sat Flow, veh/h	1781	2726	611	1781	3140	271	1697	1237	441	482	755	426
Grp Volume(v), veh/h	88	567	552	74	468	475	271	0	388	445	0	0
Grp Sat Flow(s),veh/h/ln	1781	1692	1644	1781	1692	1719	1697	0	1679	1663	0	0
Q Serve(g_s), s	6.2	40.3	40.4	5.9	37.3	37.3	22.2	0.0	31.0	35.7	0.0	0.0
Cycle Q Clear(g_c), s	6.2	40.3	40.4	5.9	37.3	37.3	22.2	0.0	31.0	35.7	0.0	0.0
Prop In Lane	1.00		0.37	1.00		0.16	1.00		0.26	0.29		0.26
Lane Grp Cap(c), veh/h	327	760	739	117	553	562	363	0	359	410	0	0
V/C Ratio(X)	0.27	0.75	0.75	0.63	0.85	0.85	0.75	0.00	1.08	1.09	0.00	0.00
Avail Cap(c_a), veh/h	327	760	739	123	553	562	363	0	359	410	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	0.67	0.67	0.67	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	0.83	0.00	0.83	1.00	0.00	0.00
Uniform Delay (d), s/veh	50.8	33.1	33.1	66.1	45.4	45.4	58.4	0.0	62.1	54.7	0.0	0.0
Incr Delay (d2), s/veh	0.2	6.6	6.8	6.9	14.7	14.5	6.2	0.0	66.8	69.8	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	2.8	18.0	17.6	2.9	17.9	18.2	10.5	0.0	20.3	22.9	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	51.0	39.7	39.9	73.0	60.1	59.9	64.5	0.0	128.9	124.4	0.0	0.0
LnGrp LOS	D	D	D	E	E	E	E	A	F	F	A	A
Approach Vol, veh/h		1207			1017			659			445	
Approach Delay, s/veh		40.6			60.9			102.4			124.4	
Approach LOS		D			E			F			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	13.5	70.4		41.0	31.9	52.0		36.0
Change Period (Y+Rc), s	4.0	* 4.8		* 5.3	4.8	* 4.6		5.0
Max Green Setting (Gmax), s	10.0	* 49		* 36	12.0	* 47		31.0
Max Q Clear Time (g_c+I1), s	7.9	42.4		37.7	8.2	39.3		33.0
Green Ext Time (p_c), s	0.0	3.2		0.0	0.0	2.8		0.0

Intersection Summary	
HCM 6th Ctrl Delay	70.3
HCM 6th LOS	E

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	28	820	249	330	920	89	315	149	584	100	116	27
Future Volume (veh/h)	28	820	249	330	920	89	315	149	584	100	116	27
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.97	1.00		1.00	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	29	845	257	340	948	92	325	154	602	103	120	28
Peak Hour Factor	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97	0.97
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	76	796	242	337	1484	144	300	142	379	117	136	217
Arrive On Green	0.04	0.30	0.30	0.19	0.45	0.45	0.24	0.24	0.24	0.14	0.14	0.14
Sat Flow, veh/h	1781	2665	810	1781	3272	318	1227	582	1552	844	984	1568
Grp Volume(v), veh/h	29	563	539	340	515	525	479	0	602	223	0	28
Grp Sat Flow(s),veh/h/ln	1781	1777	1697	1781	1777	1813	1809	0	1552	1828	0	1568
Q Serve(g_s), s	2.3	43.3	43.3	27.4	32.3	32.3	35.4	0.0	35.4	17.4	0.0	2.3
Cycle Q Clear(g_c), s	2.3	43.3	43.3	27.4	32.3	32.3	35.4	0.0	35.4	17.4	0.0	2.3
Prop In Lane	1.00		0.48	1.00		0.18	0.68		1.00	0.46		1.00
Lane Grp Cap(c), veh/h	76	531	507	337	806	822	442	0	379	253	0	217
V/C Ratio(X)	0.38	1.06	1.06	1.01	0.64	0.64	1.08	0.00	1.59	0.88	0.00	0.13
Avail Cap(c_a), veh/h	111	531	507	337	806	822	442	0	379	306	0	263
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	0.00	1.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	67.5	50.9	50.9	58.8	30.5	30.5	54.8	0.0	54.8	61.3	0.0	54.8
Incr Delay (d2), s/veh	1.2	56.4	57.8	51.1	3.9	3.8	67.5	0.0	277.2	19.4	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.1	27.5	26.5	17.2	14.7	15.0	24.5	0.0	43.1	9.5	0.0	0.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	68.7	107.2	108.6	109.9	34.4	34.3	122.3	0.0	332.0	80.8	0.0	54.9
LnGrp LOS	E	F	F	F	C	C	F	A	F	F	A	D
Approach Vol, veh/h		1131			1380			1081			251	
Approach Delay, s/veh		106.9			52.9			239.1			77.9	
Approach LOS		F			D			F			E	
Timer - Assigned Phs	1	2		4	5	6		8				
Phs Duration (G+Y+Rc), s	32.2	48.0		24.8	9.7	70.5		40.0				
Change Period (Y+Rc), s	* 4.8	* 4.7		* 4.7	3.5	* 4.8		4.6				
Max Green Setting (Gmax), s	* 25	* 43		* 24	9.0	* 59		35.4				
Max Q Clear Time (g_c+I1), s	29.4	45.3		19.4	4.3	34.3		37.4				
Green Ext Time (p_c), s	0.0	0.0		0.3	0.0	2.9		0.0				
Intersection Summary												
HCM 6th Ctrl Delay			122.8									
HCM 6th LOS			F									

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	99	341	35	200	345	113	138	341	190	98	403	49
Future Volume (veh/h)	99	341	35	200	345	113	138	341	190	98	403	49
Initial Q (Qb), veh	4	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.95	1.00		0.97	1.00		0.97	1.00		0.98
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683	1683
Adj Flow Rate, veh/h	101	348	36	204	352	92	141	348	87	100	411	29
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	169	429	44	254	574	471	168	410	336	189	445	369
Arrive On Green	0.11	0.29	0.29	0.16	0.34	0.34	0.10	0.24	0.24	0.12	0.26	0.26
Sat Flow, veh/h	1603	1492	154	1603	1683	1382	1603	1683	1379	1603	1683	1397
Grp Volume(v), veh/h	101	0	384	204	352	92	141	348	87	100	411	29
Grp Sat Flow(s),veh/h/ln	1603	0	1646	1603	1683	1382	1603	1683	1379	1603	1683	1397
Q Serve(g_s), s	5.7	0.0	20.6	11.7	16.6	4.5	8.2	18.7	4.8	5.6	22.6	1.1
Cycle Q Clear(g_c), s	5.7	0.0	20.6	11.7	16.6	4.5	8.2	18.7	4.8	5.6	22.6	1.1
Prop In Lane	1.00		0.09	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	169	0	473	254	574	471	168	410	336	189	445	369
V/C Ratio(X)	0.60	0.00	0.81	0.80	0.61	0.20	0.84	0.85	0.26	0.53	0.92	0.08
Avail Cap(c_a), veh/h	169	0	473	254	574	471	169	484	396	189	462	384
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	41.0	0.0	31.5	38.6	26.1	22.1	41.7	34.3	29.0	39.4	34.0	14.0
Incr Delay (d2), s/veh	4.0	0.0	14.1	15.8	4.8	0.9	28.1	12.7	0.6	1.4	24.3	0.1
Initial Q Delay(d3),s/veh	10.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.3	0.0	9.9	5.7	7.3	1.5	4.5	8.9	1.6	2.3	12.1	0.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	54.9	0.0	45.5	54.4	30.9	23.0	69.9	46.9	29.6	40.8	58.3	14.1
LnGrp LOS	D	A	D	D	C	C	E	D	C	D	E	B
Approach Vol, veh/h		485			648			576			540	
Approach Delay, s/veh		47.5			37.2			49.9			52.7	
Approach LOS		D			D			D			D	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	19.0	32.0	14.0	30.0	14.0	37.0	16.1	27.8
Change Period (Y+Rc), s	* 4	4.7	4.0	* 4.9	4.0	4.6	* 4.9	* 4.7
Max Green Setting (Gmax), s	* 14	27.3	10.0	* 26	9.0	32.4	* 9	* 27
Max Q Clear Time (g_c+I1), s	13.7	22.6	10.2	24.6	7.7	18.6	7.6	20.7
Green Ext Time (p_c), s	0.0	0.9	0.0	0.5	0.0	1.8	0.0	1.7

Intersection Summary		
HCM 6th Ctrl Delay		46.4
HCM 6th LOS		D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

HCM 6th Signalized Intersection Summary
5: 1st St & D St

10/02/2020

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	8	800	16	67	662	73	9	7	132	129	5	25
Future Volume (veh/h)	8	800	16	67	662	73	9	7	132	129	5	25
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	9	851	17	71	704	78	10	7	140	137	5	27
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	201	1128	23	127	942	104	49	23	258	223	12	31
Arrive On Green	0.11	0.62	0.62	0.07	0.57	0.57	0.17	0.17	0.17	0.17	0.17	0.17
Sat Flow, veh/h	1781	1827	37	1781	1654	183	49	133	1497	894	68	183
Grp Volume(v), veh/h	9	0	868	71	0	782	157	0	0	169	0	0
Grp Sat Flow(s),veh/h/ln	1781	0	1864	1781	0	1837	1679	0	0	1144	0	0
Q Serve(g_s), s	0.4	0.0	31.7	3.7	0.0	30.3	0.0	0.0	0.0	5.5	0.0	0.0
Cycle Q Clear(g_c), s	0.4	0.0	31.7	3.7	0.0	30.3	8.5	0.0	0.0	14.0	0.0	0.0
Prop In Lane	1.00		0.02	1.00		0.10	0.06		0.89	0.81		0.16
Lane Grp Cap(c), veh/h	201	0	1151	127	0	1046	330	0	0	266	0	0
V/C Ratio(X)	0.04	0.00	0.75	0.56	0.00	0.75	0.48	0.00	0.00	0.64	0.00	0.00
Avail Cap(c_a), veh/h	201	0	1151	150	0	1046	402	0	0	325	0	0
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	0.00	1.00	0.36	0.00	0.36	1.00	0.00	0.00	1.00	0.00	0.00
Uniform Delay (d), s/veh	37.6	0.0	13.0	42.7	0.0	15.3	36.1	0.0	0.0	38.7	0.0	0.0
Incr Delay (d2), s/veh	0.1	0.0	4.6	1.4	0.0	1.8	1.1	0.0	0.0	2.9	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	0.2	0.0	13.6	1.7	0.0	12.3	3.4	0.0	0.0	4.0	0.0	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	37.7	0.0	17.6	44.1	0.0	17.1	37.1	0.0	0.0	41.6	0.0	0.0
LnGrp LOS	D	A	B	D	A	B	D	A	A	D	A	A
Approach Vol, veh/h		877			853			157			169	
Approach Delay, s/veh		17.8			19.4			37.1			41.6	
Approach LOS		B			B			D			D	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	10.8	63.3		21.0	15.3	58.7		21.0
Change Period (Y+Rc), s	4.0	4.6		4.6	4.6	* 4.6		4.6
Max Green Setting (Gmax), s	8.0	53.1		20.7	7.0	* 54		20.7
Max Q Clear Time (g_c+I1), s	5.7	33.7		16.0	2.4	32.3		10.5
Green Ext Time (p_c), s	0.0	7.0		0.4	0.0	6.4		0.6

Intersection Summary			
HCM 6th Ctrl Delay			21.9
HCM 6th LOS			C

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	86	159	453	20	173	20	431	531	46	12	414	63
Future Volume (veh/h)	86	159	453	20	173	20	431	531	46	12	414	63
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.99	1.00		0.97	1.00		1.00	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1722	1722	1870
Adj Flow Rate, veh/h	91	169	482	21	184	20	459	565	49	13	440	61
Peak Hour Factor	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94	0.94
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	12	12	2
Cap, veh/h	100	185	683	217	202	22	493	1095	1004	33	536	510
Arrive On Green	0.15	0.15	0.15	0.12	0.12	0.12	0.28	0.64	0.64	0.33	0.33	0.33
Sat Flow, veh/h	643	1195	1573	1781	1652	180	1781	1722	1580	22	1647	1568
Grp Volume(v), veh/h	260	0	482	21	0	204	459	565	49	453	0	61
Grp Sat Flow(s),veh/h/ln	1838	0	1573	1781	0	1832	1781	1722	1580	1668	0	1568
Q Serve(g_s), s	20.2	0.0	0.0	1.5	0.0	16.0	36.4	25.8	1.7	10.6	0.0	4.0
Cycle Q Clear(g_c), s	20.2	0.0	0.0	1.5	0.0	16.0	36.4	25.8	1.7	36.4	0.0	4.0
Prop In Lane	0.35		1.00	1.00		0.10	1.00		1.00	0.03		1.00
Lane Grp Cap(c), veh/h	285	0	683	217	0	224	493	1095	1004	569	0	510
V/C Ratio(X)	0.91	0.00	0.71	0.10	0.00	0.91	0.93	0.52	0.05	0.80	0.00	0.12
Avail Cap(c_a), veh/h	288	0	685	217	0	224	493	1095	1004	569	0	510
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.56	0.00	0.56	1.00	0.00	1.00	1.00	1.00	1.00	0.14	0.00	0.14
Uniform Delay (d), s/veh	60.3	0.0	33.6	56.5	0.0	62.9	51.1	14.3	9.9	45.0	0.0	34.3
Incr Delay (d2), s/veh	21.0	0.0	2.1	0.1	0.0	36.9	24.1	1.7	0.1	1.7	0.0	0.1
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	11.2	0.0	14.2	0.7	0.0	9.8	19.5	10.4	0.6	15.3	0.0	1.5
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	81.3	0.0	35.7	56.7	0.0	99.8	75.1	16.1	10.0	46.7	0.0	34.4
LnGrp LOS	F	A	D	E	A	F	E	B	B	D	A	C
Approach Vol, veh/h		742			225			1073			514	
Approach Delay, s/veh		51.7			95.8			41.1			45.2	
Approach LOS		D			F			D			D	

Timer - Assigned Phs		2	3	4		6		8				
Phs Duration (G+Y+Rc), s		27.8	45.0	52.0		22.0		97.0				
Change Period (Y+Rc), s		* 5.3	* 4.8	* 4.8		4.3		* 4.8				
Max Green Setting (Gmax), s		* 23	* 39	* 47		17.7		* 90				
Max Q Clear Time (g_c+I1), s		22.2	38.4	38.4		18.0		27.8				
Green Ext Time (p_c), s		0.3	0.0	1.4		0.0		2.7				

Intersection Summary			
HCM 6th Ctrl Delay			49.8
HCM 6th LOS			D

Notes

User approved pedestrian interval to be less than phase max green.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Future Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	326	303	365	252	286	94	582	757	491	228	643	264
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	295	294	262	279	293	467	495	1309	633	251	595	244
Arrive On Green	0.17	0.17	0.17	0.16	0.16	0.16	0.28	0.40	0.40	0.14	0.26	0.26
Sat Flow, veh/h	1781	1777	1585	1781	1870	1555	1781	3272	1581	1781	2262	929
Grp Volume(v), veh/h	326	303	365	252	286	94	582	757	491	228	465	442
Grp Sat Flow(s),veh/h/ln	1781	1777	1585	1781	1870	1555	1781	1636	1581	1781	1636	1555
Q Serve(g_s), s	23.2	23.2	23.2	19.5	21.4	6.3	39.0	25.3	37.9	17.7	36.9	36.9
Cycle Q Clear(g_c), s	23.2	23.2	23.2	19.5	21.4	6.3	39.0	25.3	37.9	17.7	36.9	36.9
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		0.60
Lane Grp Cap(c), veh/h	295	294	262	279	293	467	495	1309	633	251	430	409
V/C Ratio(X)	1.11	1.03	1.39	0.90	0.98	0.20	1.18	0.58	0.78	0.91	1.08	1.08
Avail Cap(c_a), veh/h	295	294	262	279	293	467	495	1309	633	279	430	409
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	58.5	58.5	58.6	58.1	58.9	36.8	50.6	32.8	36.6	59.4	51.7	51.7
Incr Delay (d2), s/veh	84.2	60.8	198.4	29.7	45.5	0.2	98.5	0.8	6.4	28.0	66.7	67.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	17.4	15.5	23.8	11.0	13.7	2.5	30.5	10.0	15.4	9.9	22.9	21.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	142.7	119.4	256.9	87.8	104.4	37.0	149.1	33.6	43.0	87.4	118.4	119.6
LnGrp LOS	F	F	F	F	F	D	F	C	D	F	F	F
Approach Vol, veh/h		994			632			1830			1135	
Approach Delay, s/veh		177.5			87.7			72.9			112.7	
Approach LOS		F			F			E			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	43.0	42.2		27.1	23.8	61.4		28.0
Change Period (Y+Rc), s	4.0	* 5.3		* 5.1	4.0	5.3		4.8
Max Green Setting (Gmax), s	39.0	* 37		* 22	22.0	53.6		23.2
Max Q Clear Time (g_c+I1), s	41.0	38.9		23.4	19.7	39.9		25.2
Green Ext Time (p_c), s	0.0	0.0		0.0	0.1	7.7		0.0

Intersection Summary			
HCM 6th Ctrl Delay			107.4
HCM 6th LOS			F

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	167	784	24	18	461	248	24	25	42	408	30	125
Future Volume (veh/h)	167	784	24	18	461	248	24	25	42	408	30	125
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		0.98	1.00		0.97	1.00		0.98	1.00		0.99
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	169	792	17	18	466	204	24	25	12	412	30	118
Peak Hour Factor	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99	0.99
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	204	1556	33	60	854	371	72	64	16	377	20	772
Arrive On Green	0.11	0.44	0.44	0.03	0.36	0.36	0.38	0.38	0.38	0.38	0.38	0.38
Sat Flow, veh/h	1781	3555	76	1781	2391	1038	0	168	41	733	53	1562
Grp Volume(v), veh/h	169	396	413	18	345	325	61	0	0	442	0	118
Grp Sat Flow(s),veh/h/ln	1781	1777	1855	1781	1777	1652	209	0	0	786	0	1562
Q Serve(g_s), s	6.5	11.3	11.3	0.7	10.9	11.0	0.0	0.0	0.0	0.0	0.0	2.9
Cycle Q Clear(g_c), s	6.5	11.3	11.3	0.7	10.9	11.0	26.5	0.0	0.0	26.5	0.0	2.9
Prop In Lane	1.00		0.04	1.00		0.63	0.39		0.20	0.93		1.00
Lane Grp Cap(c), veh/h	204	778	812	60	635	590	151	0	0	397	0	772
V/C Ratio(X)	0.83	0.51	0.51	0.30	0.54	0.55	0.40	0.00	0.00	1.11	0.00	0.15
Avail Cap(c_a), veh/h	204	778	812	204	635	590	151	0	0	397	0	772
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	0.43	0.43	0.43	1.00	1.00	1.00	1.00	0.00	0.00	1.00	0.00	1.00
Uniform Delay (d), s/veh	30.3	14.2	14.2	33.0	17.9	18.0	17.9	0.0	0.0	25.1	0.0	9.7
Incr Delay (d2), s/veh	11.2	1.0	1.0	1.0	3.3	3.7	0.6	0.0	0.0	79.5	0.0	0.0
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	3.3	4.4	4.6	0.3	4.5	4.3	0.6	0.0	0.0	15.3	0.0	0.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	41.5	15.3	15.2	34.0	21.3	21.7	18.6	0.0	0.0	104.6	0.0	9.8
LnGrp LOS	D	B	B	C	C	C	B	A	A	F	A	A
Approach Vol, veh/h		978			688			61			560	
Approach Delay, s/veh		19.8			21.8			18.6			84.6	
Approach LOS		B			C			B			F	

Timer - Assigned Phs	1	2		4	5	6		8
Phs Duration (G+Y+Rc), s	6.4	35.6		31.1	12.0	30.0		31.1
Change Period (Y+Rc), s	4.0	4.9		4.6	4.0	4.9		* 4.6
Max Green Setting (Gmax), s	8.0	22.1		26.4	8.0	22.1		* 27
Max Q Clear Time (g_c+I1), s	2.7	13.3		28.5	8.5	13.0		28.5
Green Ext Time (p_c), s	0.0	3.9		0.0	0.0	3.1		0.0

Intersection Summary	
HCM 6th Ctrl Delay	36.2
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Intersection	
Intersection Delay, s/veh	14.5
Intersection LOS	B

Approach	EB	WB	NB	SB
Entry Lanes	1	1	1	1
Conflicting Circle Lanes	1	1	1	1
Adj Approach Flow, veh/h	728	274	160	857
Demand Flow Rate, veh/h	743	280	163	874
Vehicles Circulating, veh/h	211	695	785	208
Vehicles Exiting, veh/h	871	253	169	767
Ped Vol Crossing Leg, #/h	0	0	0	0
Ped Cap Adj	1.000	1.000	1.000	1.000
Approach Delay, s/veh	13.0	11.2	9.3	17.9
Approach LOS	B	B	A	C

Lane	Left	Left	Left	Left
Designated Moves	LTR	LTR	LTR	LTR
Assumed Moves	LTR	LTR	LTR	LTR
RT Channelized				
Lane Util	1.000	1.000	1.000	1.000
Follow-Up Headway, s	2.609	2.609	2.609	2.609
Critical Headway, s	4.976	4.976	4.976	4.976
Entry Flow, veh/h	743	280	163	874
Cap Entry Lane, veh/h	1113	679	620	1116
Entry HV Adj Factor	0.980	0.980	0.983	0.981
Flow Entry, veh/h	728	274	160	857
Cap Entry, veh/h	1091	665	609	1094
V/C Ratio	0.668	0.412	0.263	0.783
Control Delay, s/veh	13.0	11.2	9.3	17.9
LOS	B	B	A	C
95th %tile Queue, veh	5	2	1	8

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations	↰	↑	↱	↰↰	↱		↰	↑↑	↱	↰	↑↱	
Traffic Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Future Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.98	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1722
Adj Flow Rate, veh/h	326	303	321	337	168	94	582	757	279	228	643	264
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	12
Cap, veh/h	295	394	775	385	175	98	495	1307	631	252	595	244
Arrive On Green	0.17	0.21	0.21	0.11	0.16	0.16	0.28	0.40	0.40	0.14	0.26	0.26
Sat Flow, veh/h	1781	1870	1585	3456	1118	625	1781	3272	1581	1781	2262	929
Grp Volume(v), veh/h	326	303	321	337	0	262	582	757	279	228	465	442
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	0	1743	1781	1636	1581	1781	1636	1555
Q Serve(g_s), s	23.2	21.4	18.2	13.5	0.0	20.9	39.0	25.4	18.1	17.7	36.9	36.9
Cycle Q Clear(g_c), s	23.2	21.4	18.2	13.5	0.0	20.9	39.0	25.4	18.1	17.7	36.9	36.9
Prop In Lane	1.00		1.00	1.00		0.36	1.00		1.00	1.00		0.60
Lane Grp Cap(c), veh/h	295	394	775	385	0	273	495	1307	631	252	430	409
V/C Ratio(X)	1.11	0.77	0.41	0.87	0.00	0.96	1.18	0.58	0.44	0.90	1.08	1.08
Avail Cap(c_a), veh/h	295	394	775	421	0	273	495	1307	631	355	430	409
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	0.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	58.5	52.2	23.0	61.4	0.0	58.7	50.6	32.9	30.7	59.3	51.7	51.7
Incr Delay (d2), s/veh	84.2	8.6	0.3	16.7	0.0	42.9	98.5	0.8	0.7	16.5	66.7	67.9
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	17.4	11.1	7.0	6.7	0.0	12.4	30.5	10.0	7.1	9.1	22.9	21.9
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	142.7	60.7	23.3	78.1	0.0	101.6	149.1	33.7	31.4	75.7	118.4	119.6
LnGrp LOS	F	E	C	E	A	F	F	C	C	E	F	F
Approach Vol, veh/h		950			599			1618			1135	
Approach Delay, s/veh		76.2			88.4			74.8			110.3	
Approach LOS		E			F			E			F	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	43.0	42.2	28.0	27.1	23.9	61.3	20.4	34.7
Change Period (Y+Rc), s	4.0	* 5.3	* 4.8	* 5.1	4.0	5.3	* 4.8	* 5.1
Max Green Setting (Gmax), s	39.0	* 37	* 23	* 22	28.0	47.6	* 17	* 28
Max Q Clear Time (g_c+I1), s	41.0	38.9	25.2	22.9	19.7	27.4	15.5	23.4
Green Ext Time (p_c), s	0.0	0.0	0.0	0.0	0.2	8.3	0.2	0.4

Intersection Summary	
HCM 6th Ctrl Delay	86.4
HCM 6th LOS	F

Notes

User approved volume balancing among the lanes for turning movement.

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Future Volume (veh/h)	310	288	360	320	160	131	553	719	469	217	611	275
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		0.97	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1722	1870	1870	1722	1870
Adj Flow Rate, veh/h	326	303	321	337	168	70	582	757	271	228	643	145
Peak Hour Factor	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95	0.95
Percent Heavy Veh, %	2	2	2	2	2	2	2	12	2	2	12	2
Cap, veh/h	341	364	843	390	217	405	600	1328	821	254	692	638
Arrive On Green	0.19	0.19	0.19	0.11	0.12	0.12	0.34	0.41	0.41	0.14	0.21	0.21
Sat Flow, veh/h	1781	1870	1585	3456	1870	1544	1781	3272	1582	1781	3272	1585
Grp Volume(v), veh/h	326	303	321	337	168	70	582	757	271	228	643	145
Grp Sat Flow(s),veh/h/ln	1781	1870	1585	1728	1870	1544	1781	1636	1582	1781	1636	1585
Q Serve(g_s), s	24.1	20.7	15.8	12.8	11.6	4.7	42.9	23.8	13.3	16.8	25.7	8.0
Cycle Q Clear(g_c), s	24.1	20.7	15.8	12.8	11.6	4.7	42.9	23.8	13.3	16.8	25.7	8.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Lane Grp Cap(c), veh/h	341	364	843	390	217	405	600	1328	821	254	692	638
V/C Ratio(X)	0.96	0.83	0.38	0.86	0.77	0.17	0.97	0.57	0.33	0.90	0.93	0.23
Avail Cap(c_a), veh/h	341	427	896	451	309	481	602	1328	821	374	702	644
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Uniform Delay (d), s/veh	53.3	51.5	18.3	58.1	57.2	38.3	43.5	30.6	18.6	56.2	51.6	26.2
Incr Delay (d2), s/veh	37.1	10.9	0.2	13.8	6.2	0.1	28.9	0.7	0.3	13.6	19.0	0.3
Initial Q Delay(d3),s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	14.4	10.9	6.0	6.3	5.8	1.8	23.1	9.3	5.0	8.4	12.3	3.1
Unsig. Movement Delay, s/veh												
LnGrp Delay(d),s/veh	90.4	62.5	18.5	71.9	63.4	38.5	72.4	31.3	19.0	69.8	70.6	26.4
LnGrp LOS	F	E	B	E	E	D	E	C	B	E	E	C
Approach Vol, veh/h		950			575			1610			1016	
Approach Delay, s/veh		57.2			65.4			44.1			64.1	
Approach LOS		E			E			D			E	

Timer - Assigned Phs	1	2	3	4	5	6	7	8
Phs Duration (G+Y+Rc), s	48.9	33.5	30.3	20.6	23.0	59.4	19.8	31.1
Change Period (Y+Rc), s	4.0	* 5.3	* 4.8	* 5.1	4.0	5.3	* 4.8	* 5.1
Max Green Setting (Gmax), s	45.0	* 29	* 26	* 22	28.0	45.3	* 17	* 30
Max Q Clear Time (g_c+I1), s	44.9	27.7	26.1	13.6	18.8	25.8	14.8	22.7
Green Ext Time (p_c), s	0.0	0.5	0.0	0.5	0.2	8.1	0.3	0.5

Intersection Summary	
HCM 6th Ctrl Delay	54.9
HCM 6th LOS	D

Notes

* HCM 6th computational engine requires equal clearance times for the phases crossing the barrier.

Appendix C

Queuing Calculations

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Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	253	829	107	833	40	309	151	63	394	343
v/c Ratio	0.56	0.51	0.64	0.78	0.39	0.69	0.22	0.53	0.84	0.37
Control Delay	45.9	26.7	60.6	32.4	66.8	51.3	5.9	71.3	60.2	5.2
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	45.9	26.7	60.6	32.4	66.8	51.3	5.9	71.3	60.2	5.2
Queue Length 50th (ft)	178	248	66	196	32	226	11	50	298	43
Queue Length 95th (ft)	260	352	109	#461	67	305	46	94	395	66
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	453	1634	240	1073	155	518	736	155	530	918
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.56	0.51	0.45	0.78	0.26	0.60	0.21	0.41	0.74	0.37

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	19	937	67	826	232	158	203
v/c Ratio	0.11	0.63	0.45	0.49	0.80	0.51	0.77
Control Delay	42.1	22.7	64.3	26.0	68.8	46.4	68.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	42.1	22.7	64.3	26.0	68.8	46.4	68.3
Queue Length 50th (ft)	15	340	53	197	186	107	155
Queue Length 95th (ft)	m33	#525	100	407	m262	m163	227
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	184	1491	159	1675	390	410	416
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.10	0.63	0.42	0.49	0.59	0.39	0.49

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	34	773	72	929	251	140	309	30
v/c Ratio	0.24	0.54	0.42	0.60	0.81	0.37	0.89	0.07
Control Delay	52.8	29.8	55.0	28.3	63.9	8.6	70.8	0.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	52.8	29.8	55.0	28.3	63.9	8.6	70.8	0.4
Queue Length 50th (ft)	23	234	49	286	176	0	214	0
Queue Length 95th (ft)	56	335	97	406	249	48	#344	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	181	1424	181	1545	448	487	394	447
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.19	0.54	0.40	0.60	0.56	0.29	0.78	0.07

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	64	431	135	553	52	94	225	260	188	285	74
v/c Ratio	0.45	0.86	0.67	0.90	0.08	0.67	0.64	0.53	0.70	0.54	0.14
Control Delay	49.9	49.3	45.4	41.8	0.4	63.7	40.4	7.8	53.9	31.2	0.5
Queue Delay	0.0	3.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	49.9	52.7	45.4	41.8	0.4	63.7	40.4	7.8	53.9	31.2	0.5
Queue Length 50th (ft)	35	232	73	308	0	53	117	0	102	138	0
Queue Length 95th (ft)	72	#367	#134	#475	m0	#114	164	46	#241	206	0
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	141	499	212	614	618	141	471	572	270	527	547
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	27	0	0	0	0	0	4	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.45	0.91	0.64	0.90	0.08	0.67	0.48	0.46	0.70	0.54	0.14

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	10	752	81	778	95	68
v/c Ratio	0.07	0.58	0.42	0.54	0.38	0.51
Control Delay	27.7	5.2	43.6	7.9	15.8	48.6
Queue Delay	0.0	0.4	0.0	0.0	0.0	0.0
Total Delay	27.7	5.6	43.6	7.9	15.8	48.6
Queue Length 50th (ft)	5	48	44	119	8	36
Queue Length 95th (ft)	m8	m252	86	436	50	75
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	196	1291	211	1449	459	290
Starvation Cap Reductn	0	174	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.05	0.67	0.38	0.54	0.21	0.23

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	260	483	16	231	506	351	50	379	73
v/c Ratio	1.29	0.51	0.06	0.77	0.94	0.34	0.05	0.81	0.15
Control Delay	205.2	3.8	41.7	66.1	69.5	13.8	2.7	57.5	11.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	205.2	3.8	41.7	66.1	69.5	13.8	2.7	57.5	11.6
Queue Length 50th (ft)	~267	0	11	180	399	131	0	241	6
Queue Length 95th (ft)	#439	59	30	254	#615	219	15	#526	m38
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	202	947	392	410	538	1039	966	466	487
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.29	0.51	0.04	0.56	0.94	0.34	0.05	0.81	0.15

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	44	73	171	172	256	49	568	171	141	609
v/c Ratio	0.16	0.14	0.54	0.53	0.34	0.18	0.61	0.32	0.44	0.45
Control Delay	38.8	20.0	40.3	40.2	2.8	38.9	29.1	12.1	39.6	21.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	38.8	20.0	40.3	40.2	2.8	38.9	29.1	12.1	39.6	21.6
Queue Length 50th (ft)	20	6	86	87	0	23	132	24	68	139
Queue Length 95th (ft)	63	32	181	182	32	68	221	81	150	213
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	798	1477	649	655	859	342	1869	949	456	2086
Starvation Cap Reductn	0	0	11	12	7	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.06	0.05	0.27	0.27	0.30	0.14	0.30	0.18	0.31	0.29

Intersection Summary

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	110	242	6	602	47	141	181
v/c Ratio	0.60	0.10	0.03	0.33	0.20	0.59	0.33
Control Delay	41.4	5.3	21.5	6.5	19.2	30.9	4.2
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	41.4	5.3	21.5	6.5	19.2	30.9	4.2
Queue Length 50th (ft)	35	11	2	40	12	43	0
Queue Length 95th (ft)	#102	40	10	68	34	88	31
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	184	2343	225	1819	266	272	546
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.60	0.10	0.03	0.33	0.18	0.52	0.33

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	382	667	131	731	34	414	176	114	403	289
v/c Ratio	0.92	0.49	0.72	0.83	0.35	0.78	0.26	0.78	0.63	0.29
Control Delay	73.6	30.6	75.1	52.5	66.0	53.4	10.0	89.3	41.2	4.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	73.6	30.6	75.1	52.5	66.0	53.4	10.0	89.3	41.2	4.3
Queue Length 50th (ft)	294	211	104	296	27	311	34	91	284	27
Queue Length 95th (ft)	#460	278	169	#405	61	#464	78	#183	415	70
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	453	1352	226	884	155	528	718	155	640	1025
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.84	0.49	0.58	0.83	0.22	0.78	0.25	0.74	0.63	0.28

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	41	959	74	749	227	267	196
v/c Ratio	0.23	0.65	0.49	0.51	0.74	0.82	0.76
Control Delay	54.3	31.6	67.4	11.9	66.4	68.0	66.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	54.3	31.6	67.4	11.9	66.4	68.0	66.3
Queue Length 50th (ft)	31	309	43	66	186	182	145
Queue Length 95th (ft)	68	#548	89	172	m263	m262	215
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	185	1473	161	1464	375	392	433
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.22	0.65	0.46	0.51	0.61	0.68	0.45

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	29	901	57	990	211	191	205	28
v/c Ratio	0.23	0.50	0.43	0.53	0.80	0.50	0.80	0.10
Control Delay	54.0	23.4	64.7	22.5	72.2	10.6	73.0	0.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	54.0	23.4	64.7	22.5	72.2	10.6	73.0	0.6
Queue Length 50th (ft)	17	346	45	280	166	0	161	0
Queue Length 95th (ft)	m28	463	89	426	238	63	233	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	164	1812	151	1883	405	486	355	374
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.18	0.50	0.38	0.53	0.52	0.39	0.58	0.07

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	96	414	157	394	183	96	336	228	196	321	57
v/c Ratio	0.68	0.68	0.75	0.58	0.29	0.62	0.91	0.47	0.87	0.73	0.13
Control Delay	77.4	41.8	87.0	36.1	14.6	70.3	76.0	8.5	86.4	52.6	0.6
Queue Delay	0.0	5.7	0.0	0.5	0.0	0.0	0.0	0.1	0.0	0.0	0.0
Total Delay	77.4	47.5	87.0	36.6	14.6	70.3	76.0	8.6	86.4	52.6	0.6
Queue Length 50th (ft)	75	287	134	241	43	75	262	0	154	228	0
Queue Length 95th (ft)	134	#474	m204	262	m78	129	#428	68	#277	#381	0
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	167	610	282	685	636	231	382	489	244	442	447
Starvation Cap Reductn	0	0	0	67	0	0	0	0	0	0	0
Spillback Cap Reductn	0	141	0	0	0	0	0	19	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.57	0.88	0.56	0.64	0.29	0.42	0.88	0.49	0.80	0.73	0.13

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	9	868	71	782	157	169
v/c Ratio	0.08	0.73	0.50	0.60	0.37	0.91
Control Delay	51.6	19.5	71.2	10.2	10.7	92.7
Queue Delay	0.0	1.2	0.0	0.0	0.0	0.0
Total Delay	51.6	20.7	71.2	10.2	10.7	92.7
Queue Length 50th (ft)	7	653	60	191	11	128
Queue Length 95th (ft)	m12	#873	m82	m249	64	#216
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	157	1185	159	1300	522	247
Starvation Cap Reductn	0	136	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.06	0.83	0.45	0.60	0.30	0.68

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	284	571	17	211	486	394	36	315	64
v/c Ratio	0.89	0.55	0.06	0.75	1.00	0.42	0.04	0.75	0.14
Control Delay	84.6	12.2	42.6	65.6	86.6	18.5	1.7	60.3	11.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	84.6	12.2	42.6	65.6	86.6	18.5	1.7	60.3	11.4
Queue Length 50th (ft)	217	144	12	162	~392	175	0	198	2
Queue Length 95th (ft)	m#393	285	32	233	#618	284	9	#424	m27
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	320	1031	395	412	485	945	880	420	455
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.89	0.55	0.04	0.51	1.00	0.42	0.04	0.75	0.14

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	72	111	131	132	152	55	741	508	312	573
v/c Ratio	0.34	0.25	0.56	0.56	0.24	0.26	0.65	0.73	0.89	0.38
Control Delay	49.2	33.4	52.5	52.4	3.5	48.2	30.7	22.3	69.8	19.2
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	49.2	33.4	52.5	52.4	3.5	48.2	30.7	22.3	69.8	19.2
Queue Length 50th (ft)	43	24	84	85	0	33	205	157	196	126
Queue Length 95th (ft)	98	57	162	163	31	81	293	307	#428	189
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	613	1195	498	504	642	262	1435	818	350	1630
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.12	0.09	0.26	0.26	0.24	0.21	0.52	0.62	0.89	0.35

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	169	695	18	498	91	442	126
v/c Ratio	0.83	0.42	0.09	0.40	0.16	0.92	0.15
Control Delay	64.5	14.5	29.0	10.0	9.7	49.2	2.1
Queue Delay	0.0	0.4	0.0	0.0	0.0	0.0	0.0
Total Delay	64.5	14.9	29.0	10.0	9.7	49.2	2.1
Queue Length 50th (ft)	73	93	7	41	13	172	0
Queue Length 95th (ft)	#173	185	25	77	41	#338	20
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	204	1661	202	1242	589	504	820
Starvation Cap Reductn	0	467	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.83	0.58	0.09	0.40	0.15	0.88	0.15

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	253	842	174	924	40	382	301	130	441	343
v/c Ratio	0.81	0.60	0.78	0.75	0.60	0.86	0.46	0.78	0.76	0.41
Control Delay	71.7	33.8	56.3	18.1	95.4	66.0	14.4	87.8	49.7	12.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	71.7	33.8	56.3	18.1	95.4	66.0	14.4	87.8	49.7	12.3
Queue Length 50th (ft)	207	305	118	402	34	302	85	107	326	99
Queue Length 95th (ft)	#328	379	m138	m120	#88	408	128	#199	438	151
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	313	1412	272	1225	68	492	692	179	613	833
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.81	0.60	0.64	0.75	0.59	0.78	0.43	0.73	0.72	0.41

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	73	1213	67	1033	240	265	478
v/c Ratio	0.45	0.97	0.49	0.83	0.98	1.03	1.03
Control Delay	55.5	49.0	68.5	23.5	100.6	107.4	95.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	55.5	49.0	68.5	23.5	100.6	107.4	95.5
Queue Length 50th (ft)	59	~573	46	442	206	~233	~424
Queue Length 95th (ft)	m103	#704	m69	m452	m#363	m#391	#641
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	163	1255	136	1247	244	258	464
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.45	0.97	0.49	0.83	0.98	1.03	1.03

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	34	1044	466	1090	359	308	309	30
v/c Ratio	0.28	1.05	1.34	0.70	0.97	0.64	0.94	0.08
Control Delay	58.2	52.6	212.8	32.7	91.0	21.6	90.1	0.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	58.2	52.6	212.8	32.7	91.0	21.6	90.1	0.4
Queue Length 50th (ft)	24	~516	~512	412	301	72	258	0
Queue Length 95th (ft)	m24	m#543	#726	500	#494	175	#429	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	122	993	347	1562	377	488	339	389
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.28	1.05	1.34	0.70	0.95	0.63	0.91	0.08

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	45	406	100	555	80	54	202	321	262	138	48
v/c Ratio	0.35	0.72	0.70	0.84	0.13	0.41	0.63	0.62	0.79	0.24	0.09
Control Delay	51.8	38.8	62.6	38.2	1.2	52.8	45.3	9.5	56.6	25.8	0.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	51.8	38.8	62.6	38.2	1.2	52.8	45.3	9.5	56.6	25.8	0.3
Queue Length 50th (ft)	28	232	63	~360	1	33	120	2	155	63	0
Queue Length 95th (ft)	60	321	#131	#531	8	68	166	54	#309	105	0
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	127	560	143	660	638	143	435	590	333	571	567
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.35	0.72	0.70	0.84	0.13	0.38	0.46	0.54	0.79	0.24	0.08

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	10	870	81	778	95	68
v/c Ratio	0.08	0.63	0.57	0.52	0.39	0.60
Control Delay	42.2	3.9	61.2	6.6	17.5	61.2
Queue Delay	0.0	0.2	0.0	0.0	0.0	0.0
Total Delay	42.2	4.2	61.2	6.6	17.5	61.2
Queue Length 50th (ft)	7	60	51	123	9	40
Queue Length 95th (ft)	m10	138	#110	388	54	83
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	123	1377	141	1493	388	216
Starvation Cap Reductn	0	101	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.08	0.68	0.57	0.52	0.24	0.31

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	235	662	27	218	560	472	68	601	63
v/c Ratio	1.03	0.81	0.14	1.05	1.11	0.43	0.07	1.05	0.11
Control Delay	122.9	22.4	54.1	131.7	117.9	12.2	2.0	66.7	2.0
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	122.9	22.4	54.1	131.7	117.9	12.2	2.0	66.7	2.0
Queue Length 50th (ft)	~211	197	21	~200	~539	177	0	~542	0
Queue Length 95th (ft)	#379	318	51	#365	#765	248	16	m#576	m0
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	228	819	197	207	503	1108	1031	571	569
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.03	0.81	0.14	1.05	1.11	0.43	0.07	1.05	0.11

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	99	145	180	181	276	124	834	203	191	961
v/c Ratio	0.42	0.29	0.64	0.64	0.46	0.53	0.76	0.33	0.72	0.83
Control Delay	44.5	17.7	46.8	46.5	9.8	47.8	32.7	11.3	55.4	34.3
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	44.5	17.7	46.8	46.5	9.8	47.8	32.7	11.3	55.4	34.3
Queue Length 50th (ft)	54	14	106	106	42	69	221	31	108	256
Queue Length 95th (ft)	112	45	178	178	84	135	324	91	#225	#374
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	234	503	407	412	605	234	1165	643	273	1245
Starvation Cap Reductn	0	0	0	0	5	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.42	0.29	0.44	0.44	0.46	0.53	0.72	0.32	0.70	0.77

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	110	352	6	640	47	141	181
v/c Ratio	0.50	0.17	0.02	0.40	0.16	0.48	0.29
Control Delay	26.0	7.0	13.8	8.3	13.2	19.7	3.1
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	26.0	7.0	13.8	8.3	13.2	19.7	3.1
Queue Length 50th (ft)	24	16	1	40	8	28	1
Queue Length 95th (ft)	#68	57	8	74	26	65	22
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	221	2020	309	1613	288	296	614
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.50	0.17	0.02	0.40	0.16	0.48	0.29

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	446	662	89	785	30	485	182	188	389	414
v/c Ratio	1.00	0.45	0.64	0.92	0.41	0.98	0.29	1.03	0.58	0.41
Control Delay	93.4	29.4	76.6	37.8	79.5	84.3	11.5	132.1	40.5	8.1
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	93.4	29.4	76.6	37.8	79.5	84.3	11.5	132.1	40.5	8.1
Queue Length 50th (ft)	~394	215	66	250	26	424	37	~175	289	98
Queue Length 95th (ft)	#619	278	m78	m#438	61	#649	89	#332	403	151
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	445	1471	170	850	78	496	652	183	667	1000
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.00	0.45	0.52	0.92	0.38	0.98	0.28	1.03	0.58	0.41

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	90	1138	74	948	278	378	430
v/c Ratio	0.57	0.94	0.56	0.85	0.83	1.09	1.05
Control Delay	63.4	45.0	89.4	29.4	59.5	108.5	104.9
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	63.4	45.0	89.4	29.4	59.5	108.5	104.9
Queue Length 50th (ft)	71	456	66	424	240	~217	~399
Queue Length 95th (ft)	m114	m#638	m90	m460	m#367	m#551	#611
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	157	1207	131	1115	334	348	411
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.57	0.94	0.56	0.85	0.83	1.09	1.05

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	29	1121	322	1053	453	628	223	28
v/c Ratio	0.25	1.06	1.15	0.69	0.96	0.98	0.85	0.09
Control Delay	36.9	69.7	148.7	34.5	82.9	53.3	83.6	0.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	36.9	69.7	148.7	34.5	82.9	53.3	83.6	0.5
Queue Length 50th (ft)	21	~565	~330	411	397	304	192	0
Queue Length 95th (ft)	m21	m#606	#521	497	#670	#591	277	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	118	1058	281	1521	470	644	327	375
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.25	1.06	1.15	0.69	0.96	0.98	0.68	0.07

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	85	397	205	382	165	94	310	224	168	372	49
v/c Ratio	0.63	0.76	0.94	0.58	0.26	0.70	0.77	0.45	0.84	0.73	0.09
Control Delay	64.3	41.3	77.0	20.3	2.2	70.5	46.7	6.9	75.6	39.3	0.4
Queue Delay	0.0	0.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	64.3	41.7	77.0	20.3	2.2	70.5	46.7	6.9	75.6	39.3	0.4
Queue Length 50th (ft)	51	221	124	194	9	56	169	0	100	200	0
Queue Length 95th (ft)	#118	#376	m#267	m225	m13	#134	260	54	#224	#308	0
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	134	523	217	658	642	134	458	540	200	515	533
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	13	0	0	0	0	0	1	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.63	0.78	0.94	0.58	0.26	0.70	0.68	0.42	0.84	0.72	0.09

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	9	868	71	800	157	169
v/c Ratio	0.07	0.76	0.48	0.63	0.38	0.87
Control Delay	27.9	9.6	52.7	12.5	10.0	72.7
Queue Delay	0.0	0.1	0.0	0.0	0.0	0.0
Total Delay	27.9	9.7	52.7	12.5	10.0	72.7
Queue Length 50th (ft)	5	104	42	232	8	91
Queue Length 95th (ft)	m9	m#186	86	500	59	#199
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	130	1135	149	1267	458	222
Starvation Cap Reductn	0	10	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.07	0.77	0.48	0.63	0.34	0.76

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	236	626	27	199	581	580	60	441	51
v/c Ratio	0.93	0.69	0.14	0.95	1.03	0.53	0.06	0.90	0.10
Control Delay	98.1	15.5	55.8	108.0	91.3	15.4	2.2	42.9	2.8
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	98.1	15.5	55.8	108.0	91.3	15.4	2.2	42.9	2.8
Queue Length 50th (ft)	207	188	22	174	~544	260	0	328	0
Queue Length 95th (ft)	#368	324	52	#331	#774	357	16	m367	m1
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	254	911	200	210	563	1087	1004	491	509
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.93	0.69	0.14	0.95	1.03	0.53	0.06	0.90	0.10

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	167	179	157	158	212	131	954	508	378	863
v/c Ratio	0.73	0.37	0.60	0.60	0.37	0.57	0.86	0.71	1.32	0.70
Control Delay	58.8	28.1	46.5	46.1	10.1	49.1	38.1	18.8	198.1	28.0
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	58.8	28.1	46.5	46.1	10.1	49.1	38.1	18.8	198.1	28.0
Queue Length 50th (ft)	94	34	91	91	36	72	264	116	~282	212
Queue Length 95th (ft)	#203	70	157	158	71	#141	#415	263	#491	311
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	230	486	400	408	566	230	1109	712	287	1226
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.73	0.37	0.39	0.39	0.37	0.57	0.86	0.71	1.32	0.70

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	169	800	18	607	91	442	126
v/c Ratio	0.83	0.48	0.09	0.48	0.16	0.92	0.15
Control Delay	64.5	15.2	29.0	12.3	9.7	49.2	2.1
Queue Delay	0.0	0.6	0.0	0.0	0.0	0.0	0.0
Total Delay	64.5	15.8	29.0	12.3	9.7	49.2	2.1
Queue Length 50th (ft)	73	112	7	61	13	172	0
Queue Length 95th (ft)	#173	218	25	105	41	#338	20
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	204	1661	202	1261	589	504	820
Starvation Cap Reductn	0	450	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.83	0.66	0.09	0.48	0.15	0.88	0.15

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	253	821	65	828	40	297	90	65	381	343
v/c Ratio	0.56	0.47	0.51	0.76	0.39	0.68	0.15	0.53	0.83	0.38
Control Delay	45.9	23.8	64.6	30.9	66.8	51.4	5.1	71.6	59.9	5.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	45.9	23.8	64.6	30.9	66.8	51.4	5.1	71.6	59.9	5.4
Queue Length 50th (ft)	178	231	40	192	32	219	0	52	289	44
Queue Length 95th (ft)	260	328	m80	#371	67	293	30	97	381	66
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	453	1734	240	1092	155	518	704	155	527	910
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.56	0.47	0.27	0.76	0.26	0.57	0.13	0.42	0.72	0.38

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	20	889	67	818	226	156	205
v/c Ratio	0.11	0.59	0.45	0.49	0.80	0.51	0.77
Control Delay	43.9	22.6	64.3	25.5	68.9	46.3	68.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	43.9	22.6	64.3	25.5	68.9	46.3	68.4
Queue Length 50th (ft)	16	320	53	193	178	103	158
Queue Length 95th (ft)	m35	#468	100	396	m249	m152	229
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	184	1503	159	1683	387	406	416
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.11	0.59	0.42	0.49	0.58	0.38	0.49

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	34	754	91	886	268	123	309	30
v/c Ratio	0.24	0.54	0.53	0.59	0.82	0.31	0.89	0.07
Control Delay	52.8	30.5	59.3	28.6	63.5	6.1	70.8	0.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	52.8	30.5	59.3	28.6	63.5	6.1	70.8	0.4
Queue Length 50th (ft)	23	231	63	272	187	0	214	0
Queue Length 95th (ft)	56	331	117	389	262	34	#344	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	181	1395	181	1513	447	487	394	447
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.19	0.54	0.50	0.59	0.60	0.25	0.78	0.07

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	78	396	56	525	6	184	261	207	162	235	164
v/c Ratio	0.55	0.70	0.28	0.83	0.01	1.05	0.70	0.44	0.71	0.55	0.34
Control Delay	55.2	35.9	30.0	33.8	0.0	123.6	41.6	7.1	57.7	33.4	5.8
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	55.2	35.9	30.0	33.8	0.0	123.6	41.6	7.1	57.7	33.4	5.8
Queue Length 50th (ft)	43	207	29	285	0	~135	136	0	88	110	0
Queue Length 95th (ft)	#89	#322	m51	#439	m0	#245	191	42	#204	168	34
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	141	566	212	632	630	176	471	533	229	486	518
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.55	0.70	0.26	0.83	0.01	1.05	0.55	0.39	0.71	0.48	0.32

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	10	650	81	778	95	68
v/c Ratio	0.07	0.50	0.42	0.54	0.38	0.51
Control Delay	26.8	3.9	43.6	7.9	15.8	48.6
Queue Delay	0.0	0.3	0.0	0.0	0.0	0.0
Total Delay	26.8	4.2	43.6	7.9	15.8	48.6
Queue Length 50th (ft)	6	37	44	119	8	36
Queue Length 95th (ft)	m9	95	86	436	50	75
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	196	1289	211	1449	459	290
Starvation Cap Reductn	0	191	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.05	0.59	0.38	0.54	0.21	0.23

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	277	364	13	233	357	331	40	351	76
v/c Ratio	1.37	0.41	0.04	0.77	0.66	0.32	0.04	0.76	0.16
Control Delay	236.6	3.5	41.3	66.2	45.0	13.6	1.7	53.4	11.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	236.6	3.5	41.3	66.2	45.0	13.6	1.7	53.4	11.5
Queue Length 50th (ft)	~296	0	9	182	252	122	0	221	7
Queue Length 95th (ft)	#472	53	27	256	362	206	10	#470	m42
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	202	878	392	410	538	1037	965	464	485
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.37	0.41	0.03	0.57	0.66	0.32	0.04	0.76	0.16

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	179	513	258	266	143	423	451	242	73	611
v/c Ratio	0.70	0.64	0.80	0.80	0.25	0.87	0.35	0.32	0.38	0.79
Control Delay	71.0	16.5	70.6	70.0	4.5	64.3	27.4	5.7	67.5	51.5
Queue Delay	0.0	0.0	2.4	2.8	0.1	0.0	0.0	0.0	0.0	0.0
Total Delay	71.0	16.5	73.1	72.8	4.7	64.3	27.4	5.7	67.5	51.5
Queue Length 50th (ft)	151	43	232	238	0	357	141	12	62	245
Queue Length 95th (ft)	#270	111	#365	371	36	#549	193	66	126	347
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	311	884	430	444	577	645	1756	947	190	955
Starvation Cap Reductn	0	0	82	92	74	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.58	0.58	0.74	0.76	0.28	0.66	0.26	0.26	0.38	0.64

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	110	324	6	674	47	141	181
v/c Ratio	0.60	0.14	0.03	0.37	0.20	0.59	0.33
Control Delay	41.4	5.6	21.5	7.7	19.2	30.9	4.2
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	41.4	5.6	21.5	7.7	19.2	30.9	4.2
Queue Length 50th (ft)	35	15	2	53	12	43	0
Queue Length 95th (ft)	#102	53	10	86	34	88	31
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	184	2350	225	1808	266	272	546
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.60	0.14	0.03	0.37	0.18	0.52	0.33

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	179	105	408	364	303	423	451	242	73	611
v/c Ratio	0.69	0.30	0.49	0.70	0.87	0.90	0.34	0.31	0.35	0.80
Control Delay	61.3	44.6	17.5	54.7	66.8	65.1	25.7	4.6	55.7	46.4
Queue Delay	0.0	0.0	0.0	0.1	7.2	0.0	0.0	0.0	0.0	0.0
Total Delay	61.3	44.6	17.5	54.8	74.0	65.1	25.7	4.6	55.7	46.4
Queue Length 50th (ft)	134	70	156	139	206	314	126	2	53	207
Queue Length 95th (ft)	212	129	263	193	#385	#535	182	55	106	291
Internal Link Dist (ft)		1054			226		1224			1898
Turn Bay Length (ft)	110			60		250		100	130	
Base Capacity (vph)	350	388	864	679	376	509	1379	803	207	869
Starvation Cap Reductn	0	0	0	24	44	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.51	0.27	0.47	0.56	0.91	0.83	0.33	0.30	0.35	0.70

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	179	105	408	364	160	143	423	451	242	73	406	205
v/c Ratio	0.57	0.33	0.59	0.47	0.51	0.23	1.19	0.57	0.28	0.25	0.60	0.25
Control Delay	39.2	33.8	16.2	32.3	38.2	4.5	142.8	30.2	2.5	35.3	33.0	2.8
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	39.2	33.8	16.2	32.3	38.2	4.5	142.8	30.2	2.5	35.3	33.0	2.8
Queue Length 50th (ft)	79	45	88	82	71	0	~237	98	0	30	91	0
Queue Length 95th (ft)	166	104	216	151	152	38	#544	173	35	84	162	34
Internal Link Dist (ft)		1054			226			1224			1898	
Turn Bay Length (ft)	110			60		75	250		100	130		150
Base Capacity (vph)	489	521	695	1016	515	632	355	1356	965	288	1247	964
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.37	0.20	0.59	0.36	0.31	0.23	1.19	0.33	0.25	0.25	0.33	0.21

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	405	649	133	717	38	437	169	92	416	291
v/c Ratio	0.94	0.48	0.72	0.84	0.38	0.81	0.25	0.68	0.69	0.30
Control Delay	76.7	30.4	75.4	53.9	66.5	54.9	10.5	79.9	44.7	4.8
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	76.7	30.4	75.4	53.9	66.5	54.9	10.5	79.9	44.7	4.8
Queue Length 50th (ft)	317	204	105	290	30	334	35	73	297	31
Queue Length 95th (ft)	#502	270	172	#392	66	#506	79	#136	433	76
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	453	1348	226	857	155	538	721	155	607	998
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.89	0.48	0.59	0.84	0.25	0.81	0.23	0.59	0.69	0.29

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	39	950	74	747	220	280	213
v/c Ratio	0.22	0.67	0.50	0.50	0.69	0.83	0.78
Control Delay	54.3	33.1	68.5	13.1	61.9	67.6	67.0
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	54.3	33.1	68.5	13.1	61.9	67.6	67.0
Queue Length 50th (ft)	29	316	44	110	176	185	160
Queue Length 95th (ft)	65	#541	m93	202	m249	m261	232
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	185	1420	157	1495	378	394	433
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.21	0.67	0.47	0.50	0.58	0.71	0.49

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	29	881	75	978	237	165	205	28
v/c Ratio	0.23	0.50	0.54	0.53	0.82	0.43	0.80	0.10
Control Delay	52.8	24.2	69.7	23.8	71.6	9.8	73.0	0.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	52.8	24.2	69.7	23.8	71.6	9.8	73.0	0.6
Queue Length 50th (ft)	17	344	59	285	186	0	161	0
Queue Length 95th (ft)	m27	455	110	433	261	58	233	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	164	1759	151	1833	403	466	355	374
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.18	0.50	0.50	0.53	0.59	0.35	0.58	0.07

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	112	401	156	364	133	143	373	197	128	360	58
v/c Ratio	0.76	0.66	0.83	0.55	0.21	0.83	0.86	0.39	0.80	0.86	0.14
Control Delay	81.0	37.2	68.6	21.2	2.6	83.7	59.3	6.9	82.6	61.0	2.7
Queue Delay	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	2.3	0.0	0.0
Total Delay	81.0	37.5	68.6	21.2	2.6	83.7	59.3	6.9	84.9	61.0	2.7
Queue Length 50th (ft)	80	250	111	216	20	101	249	0	91	242	0
Queue Length 95th (ft)	#168	369	m#225	201	m17	#206	#396	55	#189	#387	12
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	156	605	199	659	626	184	468	527	170	450	434
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	20	0	0	0	0	0	5	8	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.72	0.69	0.78	0.55	0.21	0.78	0.80	0.38	0.79	0.80	0.13

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	9	868	71	764	157	169
v/c Ratio	0.08	0.75	0.49	0.59	0.37	0.88
Control Delay	38.0	12.8	61.4	13.4	10.0	80.2
Queue Delay	0.0	0.2	0.0	0.0	0.0	0.0
Total Delay	38.0	13.0	61.4	13.4	10.0	80.2
Queue Length 50th (ft)	6	148	49	228	10	114
Queue Length 95th (ft)	m13	#535	#108	579	60	182
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	126	1164	144	1288	537	264
Starvation Cap Reductn	0	36	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.07	0.77	0.49	0.59	0.29	0.64

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	309	428	12	217	364	379	26	328	80
v/c Ratio	0.97	0.45	0.04	0.76	0.75	0.40	0.03	0.80	0.18
Control Delay	93.4	3.2	41.7	65.4	52.2	18.5	0.4	64.6	16.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	93.4	3.2	41.7	65.4	52.2	18.5	0.4	64.6	16.4
Queue Length 50th (ft)	249	0	8	166	268	168	0	219	5
Queue Length 95th (ft)	#432	52	26	238	384	274	2	#469	m48
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	320	954	395	412	485	939	875	411	451
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.97	0.45	0.03	0.53	0.75	0.40	0.03	0.80	0.18

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	231	614	223	231	78	506	544	494	162	643
v/c Ratio	0.79	0.87	0.85	0.85	0.15	0.93	0.42	0.59	0.71	0.84
Control Delay	76.3	52.5	86.5	86.0	4.5	71.9	31.2	12.9	77.4	57.6
Queue Delay	0.0	0.0	7.6	9.5	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	76.3	52.5	94.1	95.5	4.5	71.9	31.2	12.9	77.4	57.6
Queue Length 50th (ft)	219	215	225	233	0	477	192	104	156	290
Queue Length 95th (ft)	#344	#312	#373	#384	24	#691	258	234	232	366
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	330	766	297	307	575	616	1431	881	301	861
Starvation Cap Reductn	0	0	44	51	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.70	0.80	0.88	0.90	0.14	0.82	0.38	0.56	0.54	0.75

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	169	711	18	609	91	442	126
v/c Ratio	0.83	0.43	0.09	0.48	0.16	0.92	0.15
Control Delay	64.5	14.6	29.0	12.4	9.7	49.2	2.1
Queue Delay	0.0	0.4	0.0	0.0	0.0	0.0	0.0
Total Delay	64.5	15.0	29.0	12.4	9.7	49.2	2.1
Queue Length 50th (ft)	73	96	7	61	13	172	0
Queue Length 95th (ft)	#173	190	25	106	41	#338	20
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	204	1661	202	1261	589	504	820
Starvation Cap Reductn	0	464	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.83	0.59	0.09	0.48	0.15	0.88	0.15

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	253	834	132	919	40	369	240	132	427	343
v/c Ratio	0.81	0.55	0.72	0.74	0.60	0.85	0.36	0.80	0.75	0.42
Control Delay	71.7	30.3	71.9	11.3	95.4	65.8	14.2	90.7	49.9	12.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	71.7	30.3	71.9	11.3	95.4	65.8	14.2	90.7	49.9	12.6
Queue Length 50th (ft)	207	277	92	108	34	295	66	110	323	105
Queue Length 95th (ft)	#328	361	m106	m133	#88	392	117	#200	421	151
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	313	1513	231	1248	68	492	711	177	613	822
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.81	0.55	0.57	0.74	0.59	0.75	0.34	0.75	0.70	0.42

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	74	1165	67	1026	233	262	480
v/c Ratio	0.45	0.93	0.49	0.83	0.95	1.02	1.02
Control Delay	57.3	45.4	66.2	25.3	93.2	104.0	93.5
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	57.3	45.4	66.2	25.3	93.2	104.0	93.5
Queue Length 50th (ft)	62	516	45	443	200	~227	~424
Queue Length 95th (ft)	m113	#662	m69	m521	m#349	m#388	#642
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	163	1246	136	1236	244	258	469
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.45	0.93	0.49	0.83	0.95	1.02	1.02

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	34	1025	485	1047	376	290	309	30
v/c Ratio	0.28	1.09	1.35	0.68	0.97	0.61	0.94	0.08
Control Delay	59.5	70.7	213.6	32.6	91.0	22.2	90.1	0.4
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	59.5	70.7	213.6	32.6	91.0	22.2	90.1	0.4
Queue Length 50th (ft)	24	~510	~533	388	320	74	258	0
Queue Length 95th (ft)	m26	m#563	#749	473	#528	174	#429	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	122	937	360	1530	386	478	339	389
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.28	1.09	1.35	0.68	0.97	0.61	0.91	0.08

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	59	372	21	527	34	144	239	268	236	88	138
v/c Ratio	0.49	0.49	0.17	0.78	0.05	0.37	0.71	0.55	0.86	0.40	0.45
Control Delay	60.7	26.0	41.7	33.5	0.4	37.1	50.1	8.6	70.4	45.2	9.7
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	60.7	26.0	41.7	33.5	0.4	37.1	50.1	8.6	70.4	45.2	9.7
Queue Length 50th (ft)	39	167	13	333	0	75	149	0	150	56	0
Queue Length 95th (ft)	77	286	m26	#484	m1	145	208	51	#277	85	35
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	121	754	121	680	620	385	415	539	281	411	446
Starvation Cap Reductn	0	0	0	1	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.49	0.49	0.17	0.78	0.05	0.37	0.58	0.50	0.84	0.21	0.31

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	10	769	81	778	95	68
v/c Ratio	0.08	0.56	0.52	0.52	0.40	0.63
Control Delay	47.8	4.3	57.8	6.5	18.1	67.1
Queue Delay	0.0	0.2	0.0	0.0	0.0	0.0
Total Delay	47.8	4.5	57.8	6.5	18.1	67.1
Queue Length 50th (ft)	7	85	52	126	10	43
Queue Length 95th (ft)	m13	m124	102	391	56	86
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	118	1371	168	1503	377	200
Starvation Cap Reductn	0	112	0	0	0	0
Spillback Cap Reductn	0	0	0	42	1	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.08	0.61	0.48	0.53	0.25	0.34

Intersection Summary

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	252	543	24	220	411	452	58	574	66
v/c Ratio	0.95	0.68	0.11	0.96	1.01	0.43	0.06	0.95	0.11
Control Delay	99.2	12.7	52.1	106.4	96.1	14.3	2.5	38.6	2.0
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	99.2	12.7	52.1	106.4	96.1	14.3	2.5	38.6	2.0
Queue Length 50th (ft)	214	99	18	186	~353	185	0	363	0
Queue Length 95th (ft)	#381	190	46	#349	#565	259	17	m412	m1
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	265	800	219	229	408	1054	979	607	600
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.95	0.68	0.11	0.96	1.01	0.43	0.06	0.95	0.11

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	234	585	266	276	163	498	718	274	123	963
v/c Ratio	1.00	0.77	0.94	0.95	0.31	1.08	0.50	0.35	0.64	1.02
Control Delay	114.4	24.9	94.4	94.1	8.8	108.7	27.8	9.9	70.5	77.1
Queue Delay	0.0	0.0	24.9	34.7	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	114.4	24.9	119.3	128.8	8.8	108.7	27.8	9.9	70.5	77.1
Queue Length 50th (ft)	200	80	235	244	22	~467	222	48	102	~440
Queue Length 95th (ft)	#371	150	#411	#424	60	#684	291	116	166	#576
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	234	758	286	296	546	463	1423	786	218	948
Starvation Cap Reductn	0	0	30	38	21	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.00	0.77	1.04	1.07	0.31	1.08	0.50	0.35	0.56	1.02

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	110	433	6	712	47	141	181
v/c Ratio	0.51	0.20	0.02	0.41	0.17	0.51	0.30
Control Delay	29.9	6.8	16.5	9.3	14.7	22.5	3.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	29.9	6.8	16.5	9.3	14.7	22.5	3.6
Queue Length 50th (ft)	27	21	1	57	9	33	2
Queue Length 95th (ft)	#80	72	9	96	28	72	26
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	216	2143	275	1716	297	304	594
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.51	0.20	0.02	0.41	0.16	0.46	0.30

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	470	645	91	771	33	508	174	167	402	416
v/c Ratio	0.99	0.46	0.50	0.96	0.42	0.98	0.26	0.98	0.62	0.41
Control Delay	90.5	33.4	46.1	47.0	83.1	85.1	11.5	128.7	45.4	7.8
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	90.5	33.4	46.1	47.0	83.1	85.1	11.5	128.7	45.4	7.8
Queue Length 50th (ft)	445	233	68	231	31	477	37	160	324	98
Queue Length 95th (ft)	#676	290	m84	m#467	69	#712	89	#315	443	150
Internal Link Dist (ft)		1070		460		1083			1273	
Turn Bay Length (ft)			365		100		100	155		155
Base Capacity (vph)	476	1390	183	801	85	521	676	170	644	1009
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.99	0.46	0.50	0.96	0.39	0.98	0.26	0.98	0.62	0.41

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBL	NBT	SBT
Lane Group Flow (vph)	88	1128	74	946	271	392	446
v/c Ratio	0.60	1.02	0.61	0.87	0.76	1.06	1.04
Control Delay	63.9	61.5	76.0	34.4	55.0	101.6	105.1
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	63.9	61.5	76.0	34.4	55.0	101.6	105.1
Queue Length 50th (ft)	67	~337	61	458	249	~230	~445
Queue Length 95th (ft)	m121	m#686	m90	m525	m341	m#588	#666
Internal Link Dist (ft)		1123		1236		595	1259
Turn Bay Length (ft)	45		90		200		
Base Capacity (vph)	146	1106	122	1084	357	370	429
Starvation Cap Reductn	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.60	1.02	0.61	0.87	0.76	1.06	1.04

Intersection Summary

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Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	29	1102	340	1040	479	602	223	28
v/c Ratio	0.27	1.07	1.14	0.68	0.99	0.97	0.86	0.09
Control Delay	52.7	65.2	146.9	35.9	89.8	56.4	90.3	0.6
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	52.7	65.2	146.9	35.9	89.8	56.4	90.3	0.6
Queue Length 50th (ft)	23	~603	~373	431	~465	343	207	0
Queue Length 95th (ft)	m23	m#575	#573	516	#729	#622	#311	0
Internal Link Dist (ft)		365		1548	1101		1333	
Turn Bay Length (ft)	125		130			150		150
Base Capacity (vph)	109	1031	299	1535	486	621	305	351
Starvation Cap Reductn	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.27	1.07	1.14	0.68	0.99	0.97	0.73	0.08

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Group Flow (vph)	101	384	204	352	115	141	348	194	100	411	50
v/c Ratio	0.69	0.77	0.88	0.60	0.19	0.87	0.70	0.36	0.62	0.92	0.10
Control Delay	66.3	42.8	64.5	21.6	1.4	85.8	39.3	6.0	59.2	61.3	0.4
Queue Delay	0.0	0.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	66.3	43.3	64.5	21.6	1.4	85.8	39.3	6.0	59.2	61.3	0.4
Queue Length 50th (ft)	60	211	122	173	4	85	189	0	59	238	0
Queue Length 95th (ft)	#136	#360	m#256	m194	m7	#191	#292	50	#133	#409	0
Internal Link Dist (ft)		1011		183			280			723	
Turn Bay Length (ft)	80		90		40	150			175		170
Base Capacity (vph)	150	496	234	591	594	167	510	555	164	460	494
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	12	0	0	0	0	0	1	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.67	0.79	0.87	0.60	0.19	0.84	0.68	0.35	0.61	0.89	0.10

Intersection Summary

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT
Lane Group Flow (vph)	9	868	71	782	157	169
v/c Ratio	0.07	0.76	0.48	0.62	0.38	0.87
Control Delay	29.8	11.5	52.7	12.2	10.0	72.7
Queue Delay	0.0	0.1	0.0	0.0	0.0	0.0
Total Delay	29.8	11.5	52.7	12.2	10.0	72.7
Queue Length 50th (ft)	4	142	42	223	8	91
Queue Length 95th (ft)	m11	#238	86	482	59	#199
Internal Link Dist (ft)		177		1231	922	276
Turn Bay Length (ft)	85		100			
Base Capacity (vph)	130	1135	149	1267	458	222
Starvation Cap Reductn	0	9	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0
Reduced v/c Ratio	0.07	0.77	0.48	0.62	0.34	0.76

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBT	EBR	WBL	WBT	NBL	NBT	NBR	SBT	SBR
Lane Group Flow (vph)	260	482	21	205	459	565	49	453	67
v/c Ratio	0.91	0.54	0.10	0.92	0.96	0.53	0.05	0.83	0.12
Control Delay	94.1	8.6	58.0	105.2	85.3	17.7	2.7	34.1	4.9
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay	94.1	8.6	58.0	105.2	85.3	17.7	2.7	34.1	4.9
Queue Length 50th (ft)	245	63	18	192	430	287	0	344	0
Queue Length 95th (ft)	#411	160	46	#346	#654	389	16	m359	m5
Internal Link Dist (ft)	1231			1159		1400		595	
Turn Bay Length (ft)		225					350		215
Base Capacity (vph)	286	890	216	225	476	1058	973	544	553
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.91	0.54	0.10	0.91	0.96	0.53	0.05	0.83	0.12

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

m Volume for 95th percentile queue is metered by upstream signal.

Lane Group	EBL	EBT	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT
Lane Group Flow (vph)	326	682	249	256	138	582	757	494	228	932
v/c Ratio	1.11	0.97	0.95	0.94	0.26	1.18	0.59	0.64	0.89	1.07
Control Delay	138.0	69.1	102.4	100.3	9.0	143.8	36.2	20.0	91.4	97.1
Queue Delay	0.0	0.0	26.8	36.2	0.1	0.0	0.0	0.0	0.0	0.0
Total Delay	138.0	69.1	129.2	136.5	9.0	143.8	36.2	20.0	91.4	97.1
Queue Length 50th (ft)	~339	244	238	246	21	~634	291	176	204	~476
Queue Length 95th (ft)	#534	#375	#420	#424	54	#866	360	306	#340	#613
Internal Link Dist (ft)		1054		226			1224			1898
Turn Bay Length (ft)	110		60			250		100	130	
Base Capacity (vph)	293	701	264	273	559	494	1273	772	278	870
Starvation Cap Reductn	0	0	27	35	35	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	1.11	0.97	1.05	1.08	0.26	1.18	0.59	0.64	0.82	1.07

Intersection Summary

~ Volume exceeds capacity, queue is theoretically infinite.
Queue shown is maximum after two cycles.

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Lane Group	EBL	EBT	WBL	WBT	NBT	SBT	SBR
Lane Group Flow (vph)	169	816	18	717	91	442	126
v/c Ratio	0.83	0.49	0.09	0.59	0.16	0.92	0.15
Control Delay	64.5	15.4	29.0	17.9	9.7	49.2	2.1
Queue Delay	0.0	0.6	0.0	0.0	0.0	0.0	0.0
Total Delay	64.5	16.0	29.0	17.9	9.7	49.2	2.1
Queue Length 50th (ft)	73	115	7	106	13	172	0
Queue Length 95th (ft)	#173	224	25	161	41	#338	20
Internal Link Dist (ft)		226		1054	197	1082	
Turn Bay Length (ft)	75		60				
Base Capacity (vph)	204	1662	202	1206	589	504	820
Starvation Cap Reductn	0	447	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0
Reduced v/c Ratio	0.83	0.67	0.09	0.59	0.15	0.88	0.15

Intersection Summary

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

APPENDIX C –
Draft Design Hydrology/Hydraulic Report

Caulfield Lane Over Petaluma River Moveable Bridge Project
City of Petaluma, California

Draft Bridge Design Hydraulic Study Report

Prepared for:

PETALUMA
SONOMA COUNTY
CALIFORNIA-USA

Prepared by:

WRECO

April 2021

Caulfield Lane Over Petaluma River Moveable Bridge Project
City of Petaluma, California

Draft Bridge Design Hydraulic Study Report

Submitted to:
City of Petaluma

This report has been prepared by or under the supervision of the following Registered Engineer. The Registered Civil Engineer attests to the technical information contained herein and has judged the qualifications of any technical specialists providing engineering data upon which recommendations, conclusions, and decisions are based.

____

Han-Bin Liang, Ph.D., P.E.
Registered Civil Engineer

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Date

April 2021

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Executive Summary

The City of Petaluma proposes to construct a moveable double bascule-style bridge across the Petaluma River, approximately 700 feet upstream (west) of the United States Highway 101 (US 101) Bridge. The proposed bridge would extend Caulfield Lane South across the river to connect with Petaluma Boulevard South at Crystal Lane. The Caulfield Lane over Petaluma River Moveable Bridge Project (Project) is located approximately 1 mile east of downtown Petaluma, near the southern limits of the City of Petaluma and in southwest part of Sonoma County.

The purpose of the Project is to construct a new cross-town connector that will extend Caulfield Lane south over the Petaluma River to connect with Petaluma Boulevard South at Crystal Lane. The Petaluma River at the proposed bridge site is a navigable waterway under the jurisdiction of the US Coast Guard (USCG) under Section 9 of the Rivers and Harbors Act.

The purpose of this *Bridge Design Hydraulic Stud*y is to present the design flow characteristics for the existing and the proposed conditions. This report provides the calculated scour potential and recommendations on the need for scour countermeasures for the proposed bridge.

The bridge freeboard requirements applicable to the Project are the criteria of Federal Highway Administration (FHWA), California Department of Transportation (Caltrans), USCG, and Sonoma County. To summarize, both the FHWA and Caltrans criteria requires the proposed bridge profile provide adequate freeboard to pass anticipated drift for the 50-year design flood, to pass the 100-year base flood without freeboard, or the flood of record without freeboard, whichever is greater. Two (2) feet (ft) of freeboard for the 50-year storm is commonly used in bridge designs. Per the preliminary clearance determination from the USCG, the required clearance for the proposed bridge is 90 ft of horizontal clearance in both open and closed bridge position, 10 ft of vertical clearance above mean high water elevation (MHW) in the closed position. Per Sonoma County design criterion for bridges, bridges are required to have a minimum of 1 ft of freeboard above the 100-year water surface elevations (WSE).

The Project is located within Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) number 06097C1001G, effective October 2, 2015. The FIRM indicates that the Project site is located in an area classified by FEMA as Special Flood Hazard Area Zone AE, which represents areas subject to flooding by the 100-year flood event determined by detailed methods where base flood elevations are shown. The Project site is also within a regulatory floodway. According to Title 44, Section 60.3(d)(3) of the CFR, a community shall "prohibit encroachments, including fill, new construction, substantial improvements, and other development within the adopted regulatory floodway unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed encroachment would not result in any increase in flood levels within the community during the occurrence of the base flood discharge."

The effective FEMA Flood Insurance Study (FIS) for Sonoma County, California unincorporated areas (Community Number 060376), which was effective on March 7, 2017. The FIS provides detailed hydrologic and hydraulic information for Petaluma River from the confluence of the river with San Pablo Bay to approximately 12.8 miles upstream of the confluence. The Project site is located within the detailed study segments along the Petaluma River, and a flood profile for the Petaluma River at the Project site is included in the FIS. The FIS includes the peak flows for the Petaluma River at seven locations. The Project site is located between two locations (see table below), but is much closer to the location at US 101 Bridge. The peak discharge at US 101 from FEMA FIS were selected as design peak discharge for the Project.

Peak Discharges for the Petaluma River from Sonoma County FIS

Location/Distance from Project Location	Drainage Area (square miles)	Peak Discharges (cubic feet per second)		
		50-year	100-year	500-year
At US 101 Bridge (700 ft Downstream)	50.8	9,149	10,494	13,694
Confluence with Washington Creek (11,260 ft Upstream)	44.55	8,459	9,757	13,056

The hydraulics of the existing and proposed conditions were analyzed using the US Army Corps of Engineers' (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) hydraulic modeling software (Version 5.0.7). The results of the hydraulic modeling indicated the proposed condition would not result in an increase in WSE relative to the existing condition with the studied reach of the Petaluma River during the 50-, 100-, and 500-year storm evens with different downstream boundary condition such as stillwater elevations from FEMA FIS and tidal elevations of Petaluma River recorded in the Project vicinity.

The proposed bridge would have a minimum soffit elevation of 16.35 ft NAVD 88 to meet the USCG's 10-ft vertical clearance criteria when bridge is closed. The available bridge freeboard during the 50-, 100- and 500-year storm events with stillwater elevations as downstream boundary conditions are summarized in the following table, which are representative of the minimum soffit elevations and WSEs at the upstream cross section of the proposed bridge. Based on the results of the analysis, the proposed bridge would meet the freeboard criteria of the FHWA, Caltrans, USCG, and Sonoma County.

50-, 100-, and 500-Year Water Surface Elevations and Available Freeboard

Storm Event	Downstream Control WSE* (ft NAVD 88**)	WSE at Proposed Bridge* (ft NAVD 88)	Available Freeboard* (ft)
50-Year	9.5	9.8	6.6
100-Year	10.0	10.4	6.0
500-Year	11.0	11.6	4.8

Notes: *WSEs were rounded to nearest 0.1 ft.
**NAVD 88= North American vertical Datum of 1988

Based on the particle size distribution of the bed sample collected at the railroad bridge over Petaluma River, located approximately 1,100 ft downstream of the Project site, the existing channel bed varied from fine sand to coarse gravel (URS, 2010). The median particle size (D_{50}) from the distribution curve is 3.3 millimeters (mm), which is larger than 2 mm and can be considered as cohesionless bed material. The scour calculations were performed based on the FHWA's Hydraulic Engineering Circular No. 18 (HEC-18), *Evaluating Scour at Bridges* (2012). The design capacity flow WSEs and channel flow velocities from the hydraulic analyses were used to assess the local and contraction scour depths. Total scour is the sum of local scour, contraction scour, and long-term bed elevation change. The long-term bed elevation changes were not considered for the proposed bridge because the proposed bridge is located at a location where the river channel is navigable and the main channel will be dredged periodically to maintain channel depth for navigation.

The calculated scour depths and elevations for the proposed bridge are summarized in the following two tables. The scour elevations at the abutments were calculated by subtracting the total scour depth from finished grade elevations because rock slope protection (RSP) will be incorporated at both abutments in the design.

Total Scour Depth for Abutment 1 (Northwest)

Storm Event	Downstream Boundary Condition	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Calculated Scour Elevation* (ft NAVD 88)
100-Year	Stillwater Elevation	0.0	8.7	8.7	-2.9
	Mean Higher-High Water (MHHW) Elevation	0.4	5.4	5.8	0.4
	Mean Low Water (MLW) Elevation	0.6	-**	0.6	5.8
500-Year	Stillwater Elevation	0.0	11.4	11.4	-5.6
	MHHW Elevation	0.0	6.8	6.8	-1.0
	MLW Elevation	0.2	-**	0.2	5.8

Note: *The finished grade elevation at Abutment 1 is approximately 5.8 ft NAVD 88.
**The ground elevation at the abutment is above the WSE.

Total Scour Depth for Abutment 2 (Southeast)

Storm Event	Downstream Boundary Condition	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Calculated Scour Elevation* (ft NAVD 88)
100-Year	Stillwater Elevation	0.0	5.2	5.2	4.4
	MHHW Elevation	0.4	-**	0.4	9.6
	MLW Elevation	0.6	-**	0.6	9.6
500-Year	Stillwater Elevation	0.0	6.1	6.1	3.5
	MHHW Elevation	0.0	2.4	2.4	7.2
	MLW Elevation	0.2	-**	0.2	9.6

Note: *The finished grade elevation at Abutment 2 is approximately 9.6 ft NAVD 88.
**The ground elevation at the abutment is above the WSE.

According to the Caltrans *Bridge Memo to Designers*, bridge footings supported on soil or degradable rock should be embedded below the maximum computed scour depth (2003). If the footing is supported by massive, competent rock formations resistant to scour, the footing should be placed directly on the cleaned rock surface.

If there is no bedrock existence at the Project site, the foundations should be designed to withstand the conditions of scour. Caltrans' *Memo to Designers 16-1* (2017) provides additional guidance on foundation placement:

> The top of a spread footing must be placed at or below the anticipated total scour (Degradation + Contraction + Local) elevation (*LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-1*) unless founded on competent, scour-resistant bedrock.
>
> The top of a pile cap footing must be placed at or below the estimated degradation plus contraction scour depth (LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-2). The bottom of a pile cap footing should be placed at or below the anticipated Total Scour elevation.

RSP sizing calculations were performed for the proposed bridge to estimate a minimum recommended rock class to protect the banks and embankment slopes from erosion. Two procedures were considered to determine the RSP design for the proposed bridge: FHWA's Hydraulic Engineering Circular No. 23 (HEC-23), *Bridge Scour and Stream Instability Countermeasures* (2009) and the Caltrans' *Highway Design Manual* (2020).

A minimum size of Class IV RSP is recommended to protect the embankment slopes at the proposed bridge based on the engineering judgement. RSP is typically placed at the embankment slopes at the abutments. Class IV RSP has a median particle weight of 300 lb and a median particle diameter of 15 inches. The minimum layer thickness of the Class IV (300 lb) RSP is 3 ft. The RSP should extend from 2 ft above the 100-year WSE and from the face of the abutments to the toe of slope. The RSP should extend horizontally 25 ft from upstream and downstream faces of the bridge. The RSP should be placed using

Method B, which involves dumping rock near its planned location, and working the rock to its final position with machinery. A Class 8 RSP geotextile filter fabric should be placed on the bank as the initial filter separator material between the layer of RSP and the channel bank. If the Class 8 RSP geotextile filter fabric is not feasible at the Project location, a layer of gravel filter composed of well compacted gravels with thickness of approximately 6 to 8 inches should be placed on the bank as a separate material between the RSP and the existing bed material.

Acronyms

Caltrans	California Department of Transportation
cfs	cubic feet per second
ESRI	Environmental Systems Research Institute
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
ft	foot or feet
HDM	Highway Design Manual
HEC-18	Hydraulic Engineering Circular No. 18
HEC-RAS	Hydrologic Engineering Center's River Analysis System
HU	Hydrologic Unit
MHHW	Mean Higher-High Water
MHW	Mean High Water
mi	miles
MLLW	Mean Lower-Low Water
MLW	Mean Low Water
mm	millimeters
NAVD 88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
Project	Caulfield Lane over Petaluma River Moveable Bridge Project
RSP	rock slope protection
sq mi	square miles
US 101	United States Highway 101
USACE	United States Army Corps of Engineers
USCG	United States Coast Guard
USGS	United States Geological Survey
WSE	Water Surface Elevation

1 GENERAL DESCRIPTION

1.1 Project Description

The City of Petaluma proposes to construct a moveable bascule-style bridge across the Petaluma River, approximately 700 feet (ft) upstream (west) of the United States Highway 101 (US 101) Bridge. The proposed bridge would extend Caulfield Lane South across the river to connect with Petaluma Boulevard South at Crystal Lane. The Caulfield Lane over Petaluma River Moveable Bridge Project (Project) is located approximately 1 mile east of downtown Petaluma, near the southern limits of the City of Petaluma and in southwest part of Sonoma County. See Figure 1 for the Project location map, Figure 2Figure 1. Project Location Map for the Project vicinity map, and Figure 3 for the Project aerial map.

The Project site encompasses the southern bank of the river to approximately 300 ft south to the crosswalk of the roundabout on Petaluma Boulevard South, with a width of approximately 85 ft. On the north side of the river, the Project site extends from the river approximately 300 ft north, with the same width. There appears to be no historic-age resources in the vicinity of the Project site that would warrant consideration of indirect Project effects (e.g., visual or vibration impacts) (AECOM, 2020).

The Petaluma River at the proposed bridge site is a navigable waterway under the jurisdiction of the US Coast Guard (USCG) under Section 9 of the Rivers and Harbors Act.

1.2 Purpose of Project

The purpose of the Project is to construct a new cross-town connector that will extend Caulfield Lane south over the Petaluma River to connect with Petaluma Boulevard South at Crystal Lane.

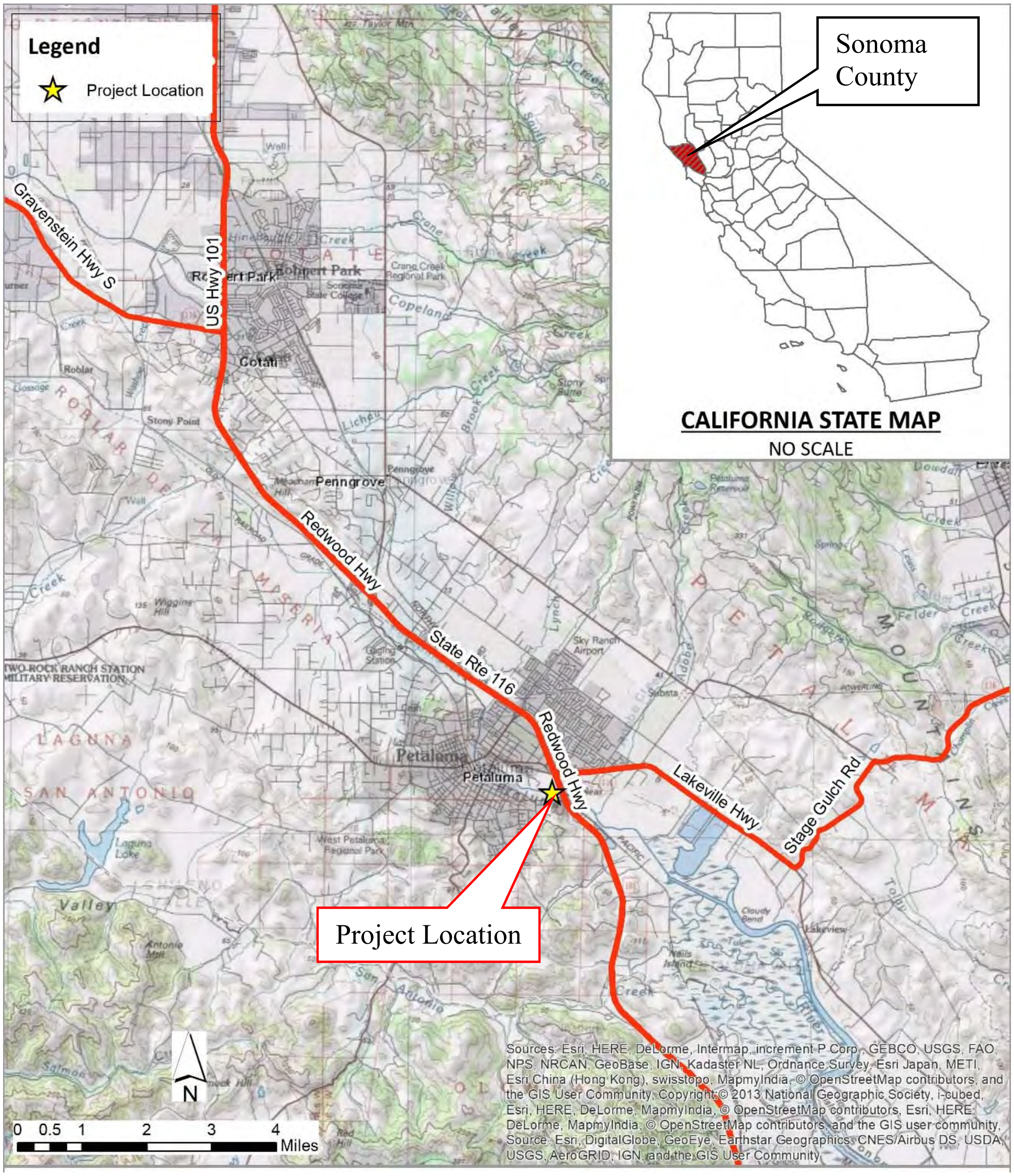

Figure 1. Project Location Map

Source: US Geological Survey (USGS, 2001)

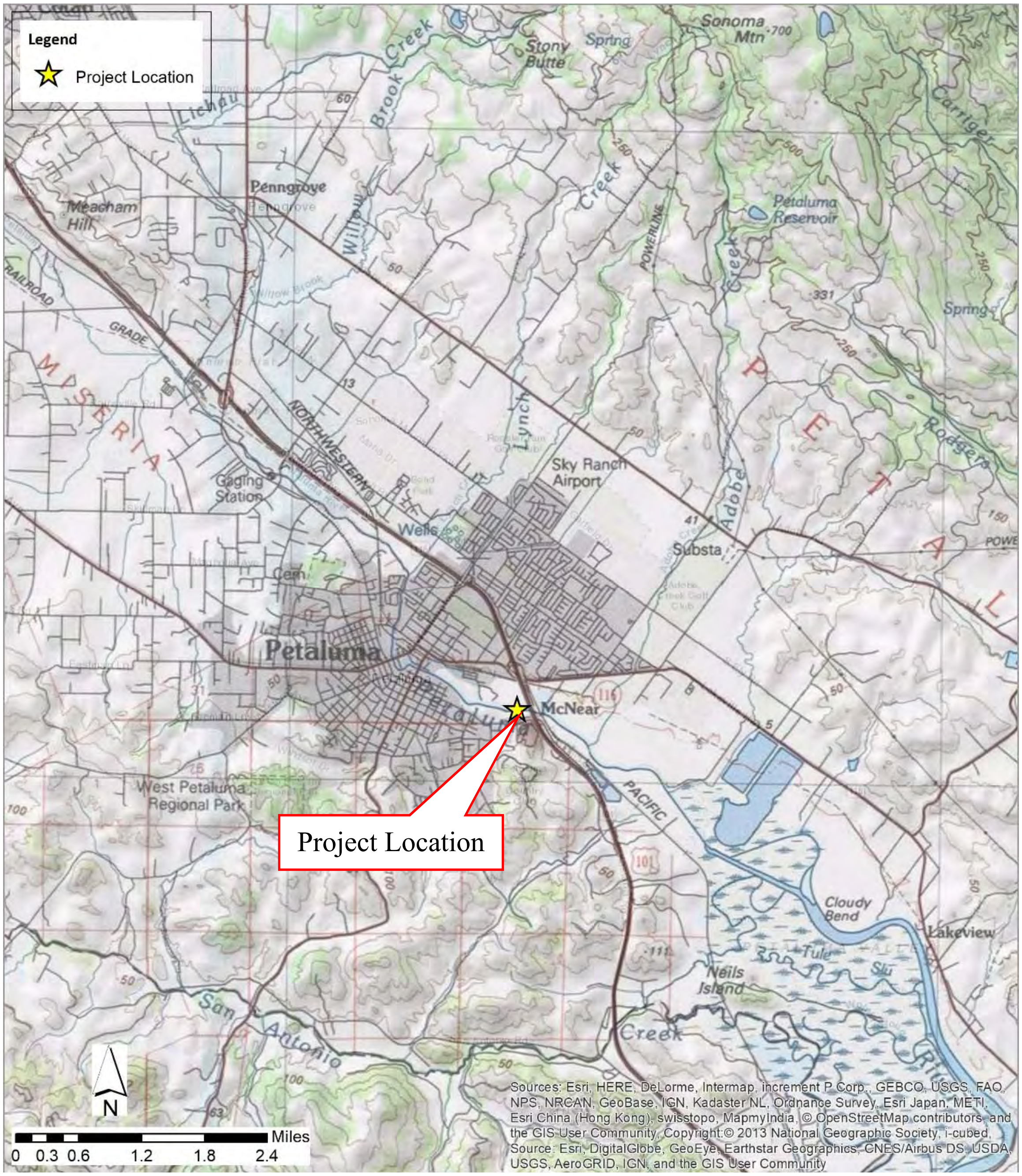

Figure 2. Project Vicinity Map

Source: USGS (2001)

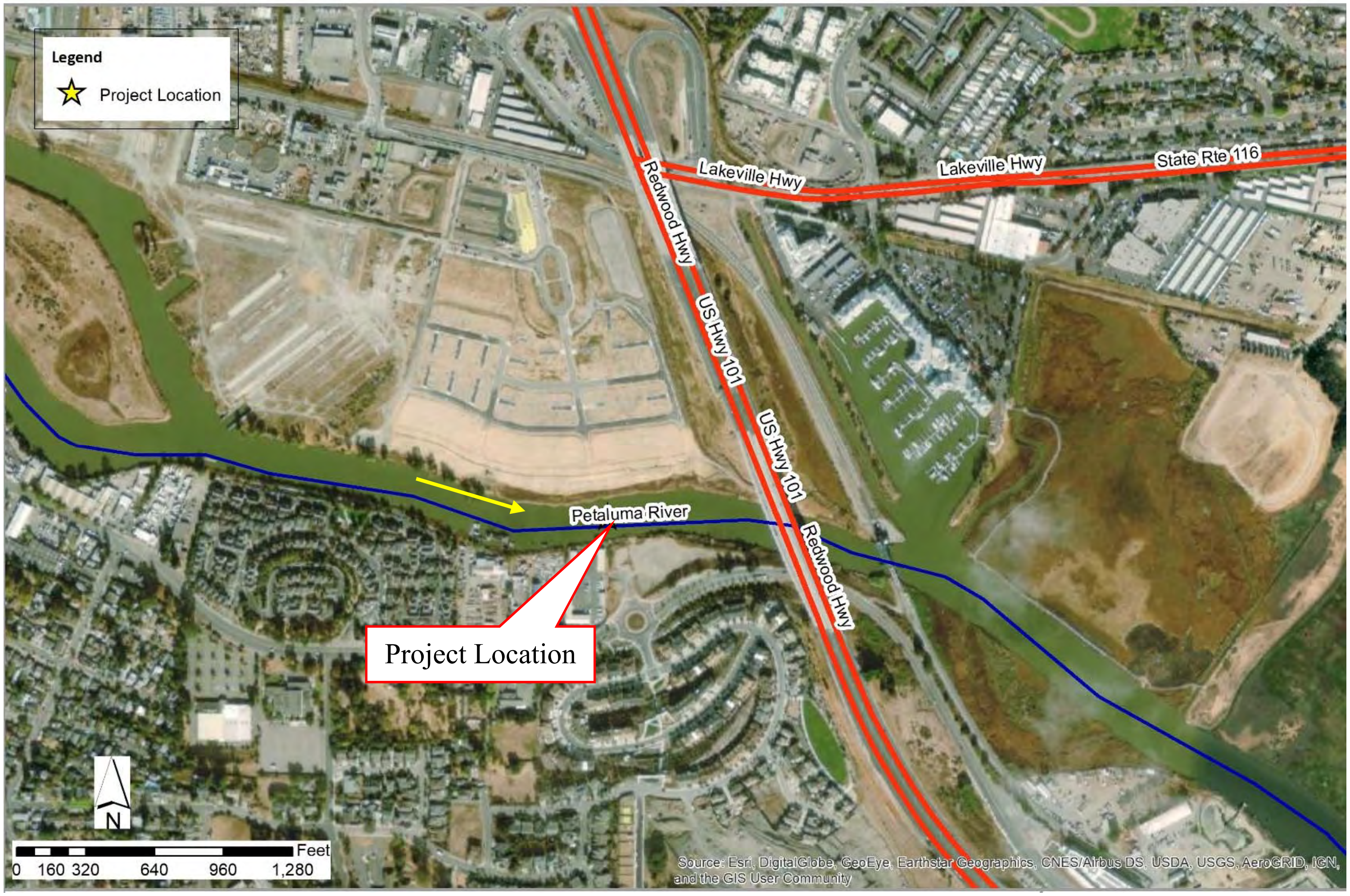

Figure 3. Project Aerial Map

Source: Environmental Systems Research Institute (ESRI, 2007)

1.3 Existing Condition

Caulfield Lane currently begins on the northeast side of Petaluma, crosses over US 101 just north of Lakeville Highway, crosses over Lakeville Street and terminates in a "T" intersection at Hopper Street immediately after crossing the Sonoma-Marin Area Rail Transit rail corridor. On the west side of US 101, Crystal Lane intersects Petaluma Boulevard South at a roundabout with a short segment of Crystal Lane extending north of the roundabout and then terminating on the south side of the Petaluma River (see Photo 1). A California Department of Transportation (Caltrans) Maintenance Facility has driveway access to Crystal Lane in this location.

Photo 1. Project Aerial View from Google Earth

Source: Google Earth (2021)

1.4 Proposed Bridge

The proposed bridge will be a single-span, two-lane, and double rolling bascule bridge, which is approximately 292 ft, and 95.6 inches in length. AECOM provided the preliminary general plan of the proposed bridge (see Figure 4). The bridge will be supported by two bascule piers, and there is 200-ft clearance navigation channel between the piers. The proposed bridge was designed to comply with the horizontal and vertical clearance criteria of Petaluma River from USCG. The minimum soffit elevation of the proposed bridge would be 16.35 ft NAVD 88 to meet the USCG clearance criteria when bridge is closed.

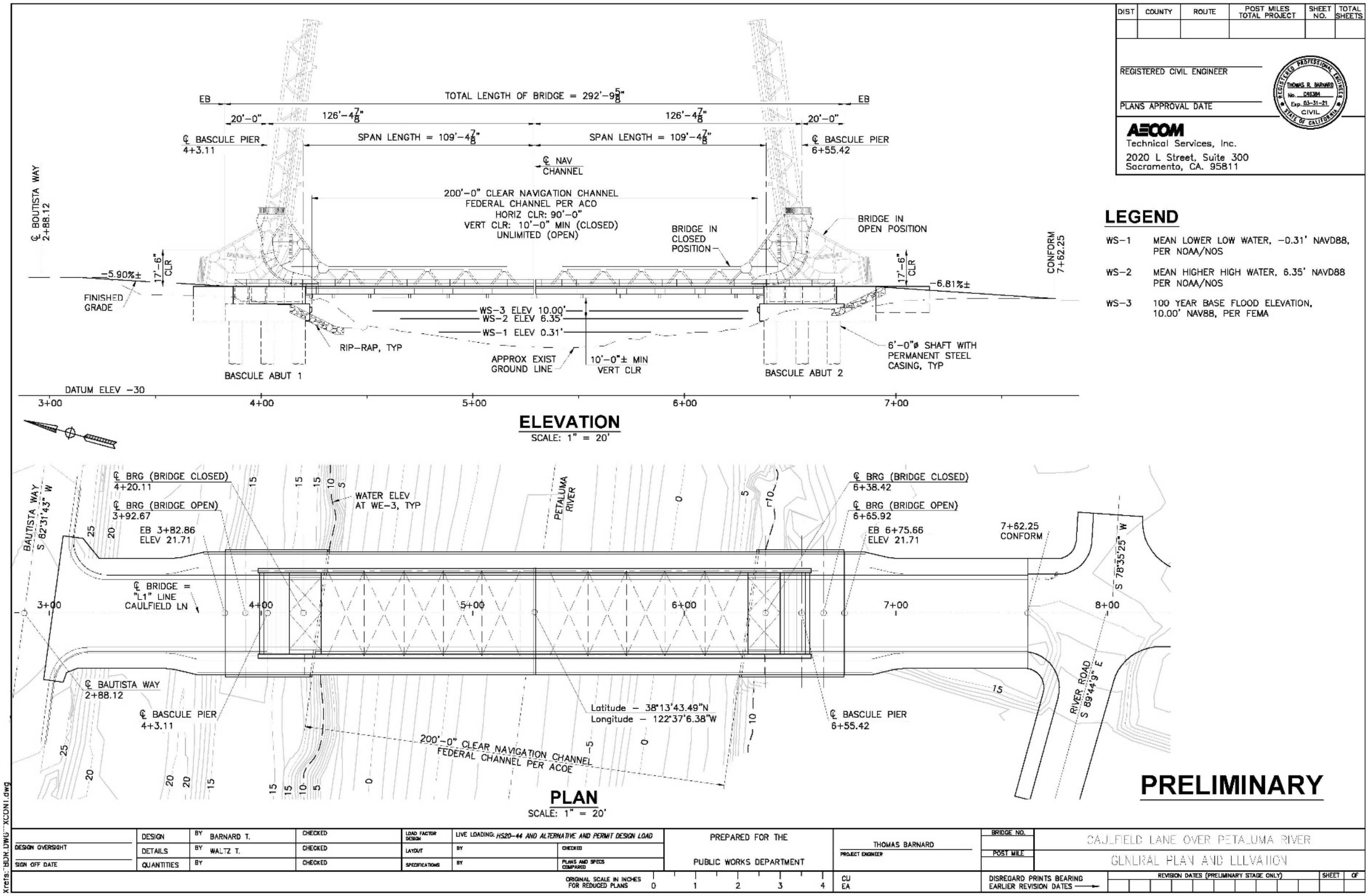

Figure 4. General Plan

Source: AECOM (2021)

1.5 Purpose of Report

The purpose of this *Bridge Design Hydraulic Study* is to present the design flow characteristics for the existing and the proposed conditions. This report provides the calculated scour potential and recommendations on the need for scour countermeasures for the proposed bridge. This report presents the hydraulic characteristics and scour potential and recommendations for the proposed bridge (see Figure 4).

1.6 Key Tasks

Key tasks performed in this study included: 1) a review of available hydrologic data, 2) a hydrologic study, 3) a hydraulic analysis to determine design water surface elevations (WSE) and flow velocities for the existing and proposed conditions, 4) a scour analysis to estimate potential scour depths for the proposed bridge, and 5) scour countermeasure analyses and recommendations for the proposed bridge.

1.7 Design Criteria

The following criteria were used in the design of the proposed bridge.

1.7.1 Hydraulic Design Criteria

1.7.1.1 Federal Highway Administration Standards

According to the *California Amendments to the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (2017), the Federal Highway Administration (FHWA) mandated that LRFD be used on all new bridge design commencing on or after October 1, 2007 (Caltrans, 2019). In 2011, the *California Amendments to the AASHTO LRFD Bridge Design Specifications* updated certain sections of the guidance, including Section 2 in its entirety.

From Section 2 of the *California Amendments to the AASHTO LRFD Bridge Design Specifications*, the proposed bridge profile should provide adequate freeboard to pass anticipated drift for the 50-year design flood, to pass the 100-year base flood without freeboard, or the flood of record without freeboard, whichever is greater (Caltrans, 2011).

Subsequent revisions to the *California Amendments to the AASHTO LRFD Bridge Design Specifications* in 2014 and 2019 did not include changes to Section 2. The sections that are not revised in subsequent versions of the *California Amendments to the AASHTO LRFD Bridge Design Specifications* are still in effect.

Per the FHWA design criteria, 100-year and 500-year storm events for the bridges in a tidal waterway must be considered in the hydraulic analysis. Scour analyses were conducted for both the 100-year and 500-year design storm events.

1.7.1.2 Caltrans Standards

From Chapter 820 of the Caltrans' *Highway Design Manual* (HDM) (2020), the criteria for the hydraulic design of bridges is that they be designed to pass the 2% probability of annual exceedance flow (50-year design discharge) with adequate freeboard to pass anticipated drift and debris (2020). Two (2) ft of freeboard is commonly used in bridge designs. The bridge should also be designed to pass the 1% probability of annual exceedance flow (100-year design discharge, or base flood). No freeboard is added to the base flood. Alternatively, the bridge can be designed for the flood of record.

1.7.1.3 USCG Standards

A preliminary clearance determination from the USCG has been obtained for the Project, which outlines the required horizontal and vertical clearances required for a bridge in this location. The required clearance for the proposed bridge is 90 ft of horizontal clearance in both open and closed bridge position (USCG, 2019a), 10 ft of vertical clearance above mean high water (MHW) elevation in the closed position. USCG's preliminary public notice (11-150) dated May 6, 2019 provided MHW elevation of 6.35 ft NAVD 88 (2019b).

1.7.1.4 Sonoma County Standards

Per Sonoma County flood control design criteria for bridges, bridges are required to have a minimum of 1 ft of freeboard above the 100-year WSEs (Sonoma County Water Agency, 2021).

1.7.1.5 Federal Emergency Management Agency Standards

The Project is located within Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM) number 06097C1001G, effective October 2, 2015 (see Figure 5). The FIRM indicates that the Project site is located in an area classified by FEMA as Special Flood Hazard Area Zone AE, which represents areas subject to flooding by the 100-year flood event determined by detailed methods where base flood elevations are shown. The Project site is located very close to cross section A.

The Project site is also within a regulatory floodway. According to Title 44, Section 60.3(d)(3) of the CFR, a community shall "prohibit encroachments, including fill, new construction, substantial improvements, and other development within the adopted regulatory floodway unless it has been demonstrated through hydrologic and hydraulic analyses performed in accordance with standard engineering practice that the proposed encroachment would not result in any increase in flood levels within the community during the occurrence of the base flood discharge."

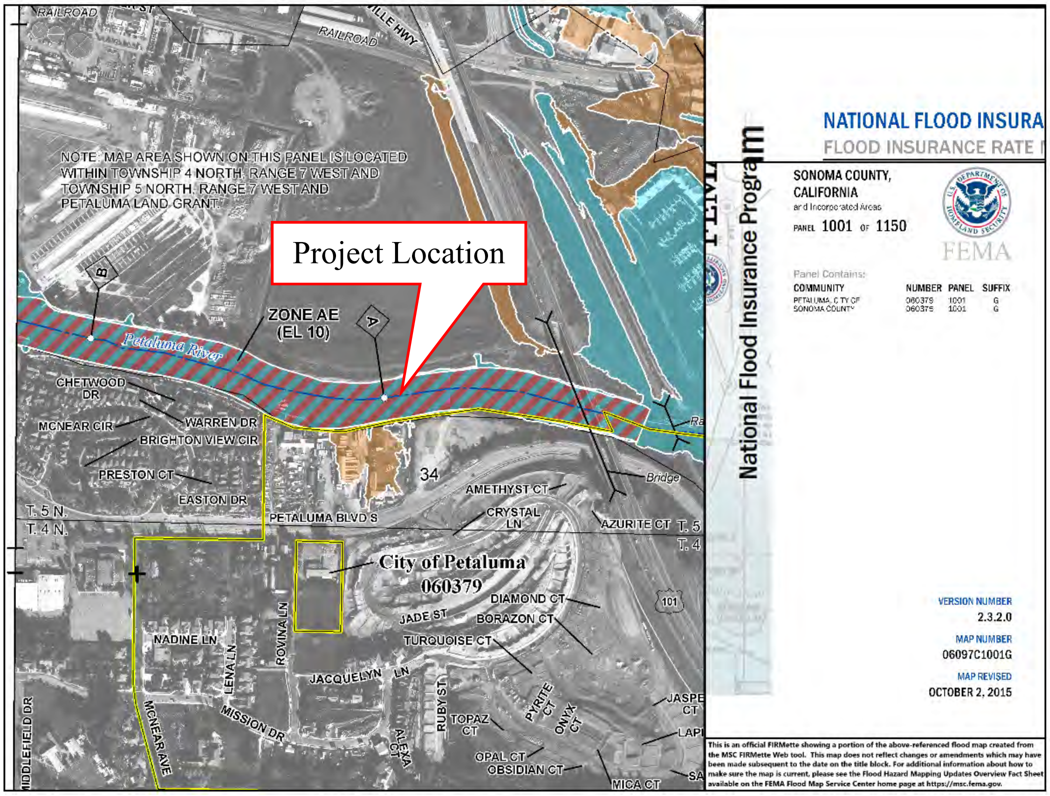

Figure 5. FEMA FIRM within the Project Vicinity

Source: FEMA (2015)

1.7.2 Foundation Criteria

Per the *California Amendments to the AASHTO LRFD Bridge Design Specifications* (Caltrans, 2014), foundations should be designed to withstand the conditions of scour. Caltrans' *Memo to Designers 16-1* (2017) provides additional guidance on foundation placement:

> The top of a spread footing must be placed at or below the anticipated total scour (Degradation + Contraction + Local) elevation (*LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-1*) unless founded on competent, scour-resistant bedrock.
>
> The top of a pile cap footing must be placed at or below the estimated degradation plus contraction scour depth (LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-2). The bottom of a pile cap footing should be placed at or below the anticipated Total Scour elevation.

1.7.3 Scour Design Criteria

The evaluation of potential scour at the proposed bridge followed the criteria described in the FHWA's *Hydraulic Engineering Circular No. 18* (HEC-18), "Evaluating Scour at Bridges" (FHWA, 2012). The evaluation of potential scour was based on hydraulic characteristics of the 100-year design discharge. The total scour was estimated based upon the cumulative effects of the long-term bed elevation change, general (contraction) scour, and local scour. The life expectancy of the bridge was considered in determining the long-term bed elevation change of the waterway; it was based on an assumed 75-year design life for a new replacement bridge.

1.7.4 Rock Slope Protection Design Criteria

Two procedures for determining rock slope protection (RSP) design were considered for the proposed structure: the FHWA's *Hydraulic Engineering Circular No. 23* (HEC-23), "Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance" (2009), and Caltrans' HDM (2020). The final selection considers both of these procedures and is based on engineering judgment.

1.8 Vertical Datum

The Project references the North American Vertical Datum of 1988 (NAVD 88).

2 GEOGRAPHIC SETTING

2.1 Geographic Location

The Project is located near the southern limits of the City of Petaluma in southern region of Sonoma County, California. It is approximately 50 miles (mi) north of San Francisco and 19 mi southeast of Santa Rosa. The proposed bridge is located at a latitude of 38°13′43.49″ North and a longitude of 122°37′6.38″ West.

2.2 Watershed Description

The Project is located in the San Pablo Hydrologic Unit (HU) of the San Francisco Bay Basin, specifically within the San Pablo Bay Watershed. This watershed falls within the jurisdiction of the San Francisco Bay Regional Water Quality Control Board. Within the San Pablo HU, the Project is in the Petaluma River Hydrologic Area.

The Petaluma River drains a watershed area of approximately 50.4 square miles (sq mi) at the Project site (see Figure 6). The upper elevation of the watershed is approximately 2,322 ft NAVD 88 and the lower elevation is 0 ft NAVD 88 (USGS StreamStats, 2020). Approximately 3.7% of the watershed is covered by forest, approximately 31.9% of watershed is developed (urban) land, and the mean annual precipitation is 33.3 inches.

2.3 Land Use

On the north side of Petaluma River between Hopper Street and the river, there is existing vacant riverfront land that has been graded for development, and the construction of a hotel is currently underway. On the south side of Petaluma River, Crystal Lane is the entrance to a residential development called Quarry Heights Subdivision on the south side of Petaluma Boulevard South. A Caltrans Maintenance Facility is located west of the proposed bridge site, while a vacant lot is to the east.

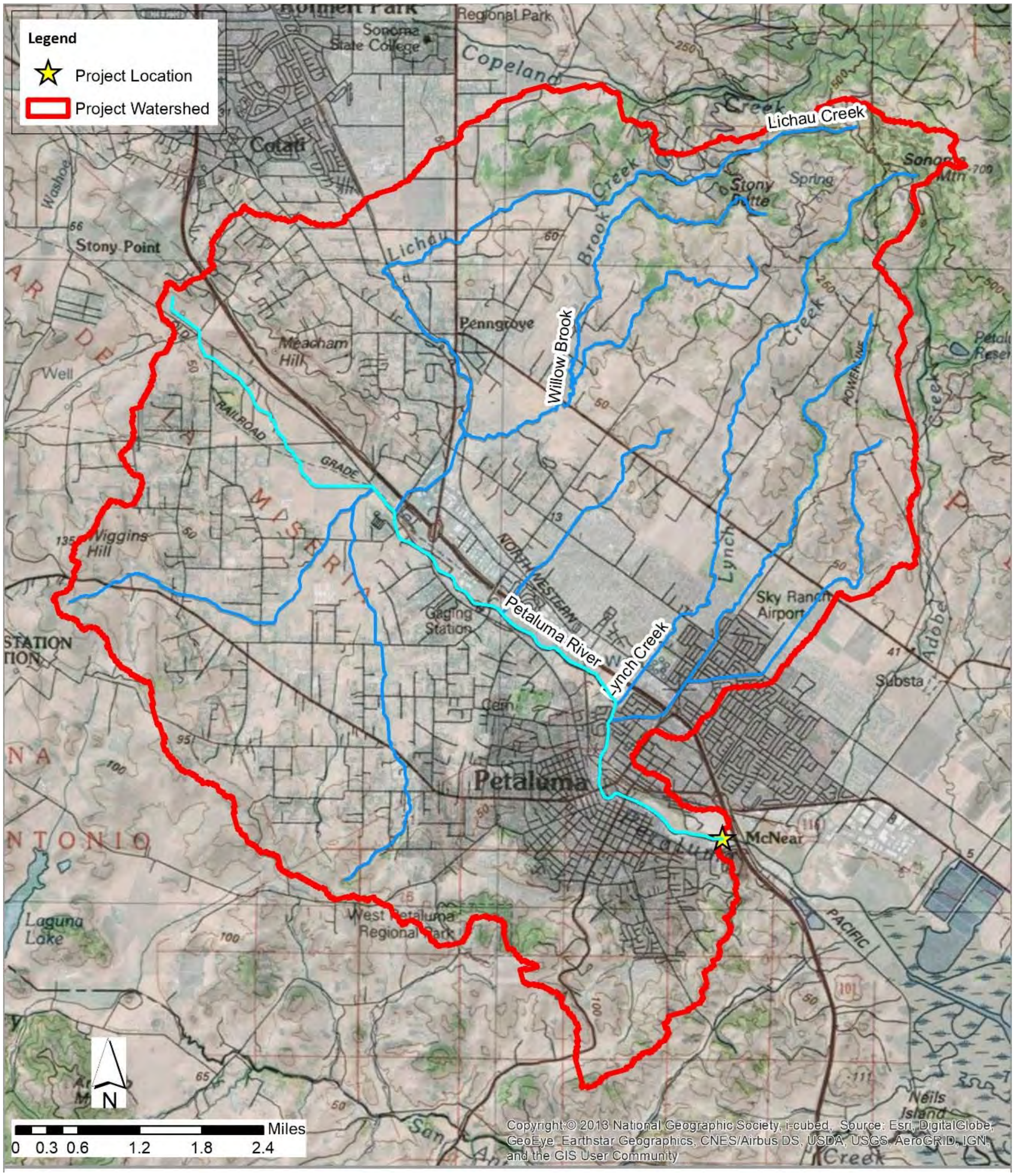

Figure 6. Project Watershed Map

Source (USGS 2021)

3 HYDROLOGIC ANALYSIS

The following sub-sections describe the hydrologic data sources that were used to estimate the flows for the Project site.

3.1 Hydrologic Design Methods

WRECO evaluated the hydrology at the Project site using the following hydrologic design methods:

1. USGS Regional Regression Equations;
2. FEMA Flood Insurance Study (FIS).

3.2 Design Discharge Summary

3.2.1 United States Geological Survey Regional Regression Equations

Flood-frequency equations were developed by the USGS and are based on an analysis of data from gaging stations. The USGS has divided California into six hydrologic regions; the Project site is within the North Coast region. These flood frequency equations are generally used to estimate stream flow for ungaged sites that are not affected by substantial urban development and that are natural (unregulated) streams.

The USGS issued *Methods for Determining Magnitude and Frequency of Floods in California, Based on Data through Water Year 2006* (Gotvald et al., 2012), which contains updated regional flood-frequency equations, and revised the boundaries of the six unique regions within California. These equations are based on annual peak-flow data through water year 2006 for 771 streamflow-gaging stations in California having 10 or more years of data. The Project site is within the North Coast region, and the 100- and 500-year peak flow equations for the North Coast region are the following:

$$Q_{50} = 36.3DRNAREA^{0.87}PRECIP^{0.589}$$
$$Q_{100} = 48.5DRNAREA^{0.866}PRECIP^{0.556}$$
$$Q_{500} = 79.3DRNAREA^{0.86}PRECIP^{0.503}$$

Where:

Q_n = peak flow rate for return period of n-year (cubic feet per second [cfs])
DRNAREA = watershed area (sq mi)
PRECIP = average yearly precipitation (inches)

The 50-, 100-, and 500-year peak discharges at the Project site were calculated using the USGS Regional Regression Equations and summarized in Table 1.

Table 1. Flow Rates for Petaluma River at the Project Site Using USGS Regional Regression Equations

Peak Discharge (cfs)		
50-year	**100-year**	**500-year**
8,660	13,500	10,200

3.2.2 FEMA FIS

FEMA has issued an effective FIS for Sonoma County, California unincorporated areas (Community Number 060376), which was effective on March 7, 2017. There are five volumes of the FIS (No. 06097CV001E, 06097CV002D, 06097CV003E, 06097CV004B, 06097CV005A, and 06023CV002B). The effective FEMA FIS for Sonoma County provides detailed hydrologic and hydraulic information for Petaluma River is from the confluence of the river with San Pablo Bay to approximately 12.8 mi upstream of the confluence. The Project site is located within the detailed study segments along the Petaluma River, and a flood profile for the Petaluma River at the Project site is included in the FIS.

The first volume of the effective FIS for Sonoma County includes the peak flows for the Petaluma River at seven locations (see Appendix A). The Project site is located between two locations (see Table 2), but is much closer to the location at US 101 Bridge.

Table 2. Peak Discharges for the Petaluma River from Sonoma County FIS

Location/Distance from Project Location	Drainage Area (sq mi)	Peak Discharges (cfs)		
		50-year	**100-year**	**500-year**
At US 101 Bridge (700 ft Downstream)	50.8	9,149	10,494	13,694
Confluence with Washington Creek (11,260 ft Upstream)	44.55	8,459	9,757	13,056

Source: FEMA FIS, 2017

3.2.3 Selected Design Discharges

The 50-, 100- and 500-year peak discharges from the FEMA FIS at US 101 Bridge were selected for the hydraulic analysis at the Project site. The selected design peak discharges are summarized in Table 2 and highlighted in the red rectangle.

3.3 Hydrologic Stability

Due to the nature of the work proposed by the Project, the Project would not significantly change the overall land use within the watershed basin. If future development is limited within the watershed due to Sonoma County’s growth constraints, the design discharges would not be anticipated to significantly change during the lifetime of the bridge at the Project location. However, future developments within the watershed would have the potential to impact the hydrologic conditions of the watershed and at the Project site.

4 HYDRAULIC ANALYSIS

The following sections discuss the development of the hydraulic models and summarize the results for the existing and proposed conditions. The water surface profile plots, hydraulic summary tables, and channel cross sections are included in Appendix B for the existing condition and Appendix C for the proposed condition.

4.1 Study Tool

The hydraulic analyses were performed for the existing and proposed conditions using the United States Army Corps of Engineers (USACE)'s Hydrologic Engineering Centers River Analysis System (HEC-RAS) modeling software, Version 5.0.7.

4.2 Hydraulic Model Development

4.2.1 Cross Section Data

The channel geometry for the hydraulic model was developed using topographic survey data provided by AECOM. The locations of the cross sections are shown in Figure 7. The cross sections extend approximately 650 ft upstream and 1,208 ft downstream of the proposed bridge along the Petaluma River. The cross-section naming convention is by river station (RS) with the cross-section number increasing in RS (measured in feet) going upstream.

4.2.2 Modeled Hydraulic Structures

There is no existing bridge at the Project site, but US 101 Bridge was included in the hydraulic model due to the proximity. The geometry of the existing US 101 Bridge in the hydraulic model was based on the information from a previous hydraulic study for the bridge (WRECO, 2011).

The proposed structural design and roadway elevations for the proposed bridge were based on General Plan documents provided by AECOM (see Figure 4). The minimum soffit elevation of the proposed bridge is 16.35 ft NAVD 88. The proposed bridge was modeled with a 200-ft hydraulic clearance.

Figure 7. Cross Section Locations

4.2.3 Model Boundary Condition

Three downstream boundary conditions were selected to conduct the hydraulic analyses: the 50-, 100-, and 500-year stillwater elevation from FEMA FIS (see Appendix A); the Mean Higher-High Water (MHHW) elevation at the Project location was from the effective FEMA FIS (2017) for the Petaluma River; and the Mean Low Water (MLW) elevation at the Project location was based on the water elevations recorded in National Oceanic and Atmospheric Administration (NOAA)'s tidal station No. 9415584 (NOAA, 2021), which is located approximately 1,000ft downstream of the Project location, and approximately 300 ft downstream of US 101 bridge (see Figure 8 for the location of the station and see Table 3 for the recorded water elevations).

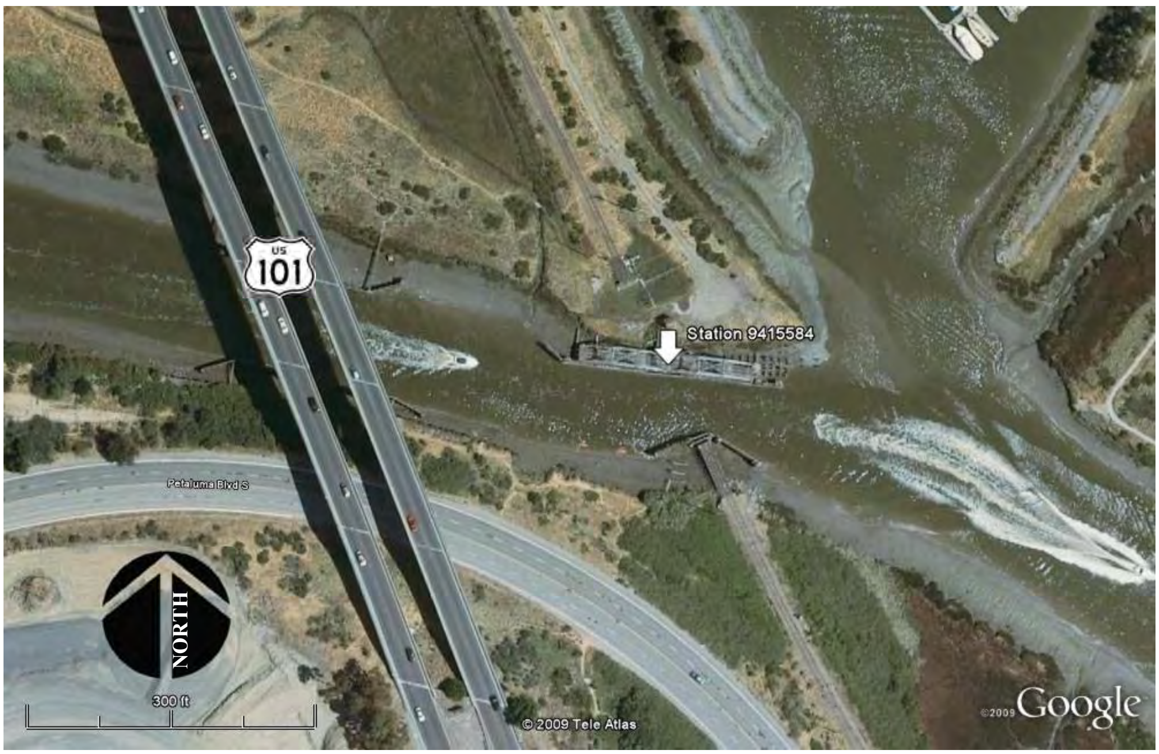

Figure 8. NOAA Tidal Station Location

Source: Google Earth

Table 3. Recorded Water Elevations at NOAA Tidal Station (No. 9415584, Petaluma River, Upper Drawbridge, CA)

Station Location	Latitude	38°13′42″ North
	Longitude	122°36′48″ West
Tidal Period	1983-2001	
Water Elevations (ft NAVD 88)	MHHW	6.35
	MHW	5.88
	MLH	0.67
	Mean Lower-Low Water (MLLW)	-0.31

Source: NOAA (2021)

The selected downstream boundary conditions for the 50-, 100-, and 500-year storm events are summarized in Table 4.

Table 4. Selected Downstream Boundary Conditions

Boundary Conditions	Known WSE (ft NAVD 88)
50-Year Stillwater Elevation	9.5
100-Year Stillwater Elevation	10.0
500-Year Stillwater Elevation	11.0
MHHW Elevation	6.53
MLH Elevation	0.67

Source: FEMA, 2017; NOAA, 2021

4.2.4 Manning's Roughness Coefficients

Manning's roughness coefficients were used in the hydraulic model to estimate energy losses in the flow due to friction. Per the effective FEMA FIS for Sonoma County (2017), the roughness coefficient for the main channel of the Petaluma River ranges from 0.025 to 0.055, and the roughness coefficient for the overbank areas range from 0.025 to 0.07. A roughness coefficient of 0.035 was used to describe the generally straight channel with very little vegetation channel within the Project vicinity, and a roughness coefficient of 0.05 was used to describe the overbank areas with light brush and trees. These values fall within the roughness coefficient ranges included in FEMA FIS, and were selected based on the aerial photograph and field investigation conducted by the Project team.

4.2.5 Expansion and Contraction Coefficients

Expansion and contraction coefficients were used in the hydraulic model to represent energy losses in the channel. An expansion coefficient of 0.3 and a contraction coefficient of 0.1 were used to represent the channel. These values represent a channel with gradual transitions between cross sections. The expansion and contraction coefficients used in the vicinity of the bridge were 0.5 and 0.3, respectively. These values represent the flow interference caused by the bridge structure.

4.3 Hydraulic Model Results

4.3.1 Water Surface Elevations

The WSEs for the Petaluma River were estimated for the existing and proposed conditions using the hydraulic models created in HEC-RAS. See Table 5, Table 6, and Table 7 for the comparison of the WSEs in the vicinity of the bridges during the 50-, 100- and 500-year storm events with different downstream boundary conditions. The cross section facing downstream at the upstream side of the proposed bridge is shown in Figure 9. The water surface profiles along the studied stream reach are presented for the existing and proposed bridges in Figure 10 through Figure 12 for the 50-, 100-, and 500-year storms, respectively.

Table 5. The Petaluma River 50-Year Water Surface Elevations with Stillwater Elevation as Downstream Boundary Condition

RS	Location/Distance from Proposed Bridge Centerline	WSE* (ft NAVD 88)		WSE Difference (ft)
		Existing	Proposed	
1324	Approximately 32 ft Upstream	9.8	9.8	0.0
1300 BR U	Proposed Bridge Upstream Face	-	9.8	-
1300 BR D	Proposed Bridge Downstream Face	-	9.8	-
1254	Approximately 33 ft Downstream	9.8	9.8	0.0

Note: * WSEs were rounded to nearest 0.1 ft.

Table 6. The Petaluma River 100-Year Water Surface Elevations

RS	Location/Distance from Proposed Bridge Centerline	WSE* (ft NAVD 88)		WSE Difference (ft)
		Existing	Proposed	
With Stillwater as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	10.4	10.4	10.4
1300 BR U	Proposed Bridge Upstream Face	-	10.4	-
1300 BR D	Proposed Bridge Downstream Face	-	10.4	-
1254	Approximately 33 ft Downstream	10.4	10.4	10.4
With MHHW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	7.4	7.4	0.0
1300 BR U	Proposed Bridge Upstream Face	-	7.4	-
1300 BR D	Proposed Bridge Downstream Face	-	7.3	-
1254	Approximately 33 ft Downstream	7.3	7.3	0.0
With MLW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	4.0	4.0	0.0
1300 BR U	Proposed Bridge Upstream Face	-	4.0	-
1300 BR D	Proposed Bridge Downstream Face	-	3.9	-
1254	Approximately 33 ft Downstream	3.8	3.8	0.0

Note: * WSEs were rounded to nearest 0.1 ft.

Table 7. The Petaluma River 500-Year Water Surface Elevations

RS	Location/Distance from Proposed Bridge Centerline	WSE* (ft NAVD 88)		WSE Difference (ft)
		Existing	Proposed	
With Stillwater as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	11.6	11.6	0.0
1300 BR U	Proposed Bridge Upstream Face	-	11.6	-
1300 BR D	Proposed Bridge Downstream Face	-	11.5	-
1254	Approximately 33 ft Downstream	11.5	11.5	0.0
With MHHW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	7.9	7.9	0.0
1300 BR U	Proposed Bridge Upstream Face	-	7.9	-
1300 BR D	Proposed Bridge Downstream Face	-	7.8	-
1254	Approximately 33 ft Downstream	7.8	7.8	0.0
With MLW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	5.5	5.5	0.0
1300 BR U	Proposed Bridge Upstream Face	-	5.5	-
1300 BR D	Proposed Bridge Downstream Face	-	5.3	-
1254	Approximately 33 ft Downstream	5.3	5.3	0.0

Note: * WSEs were rounded to nearest 0.1 ft.

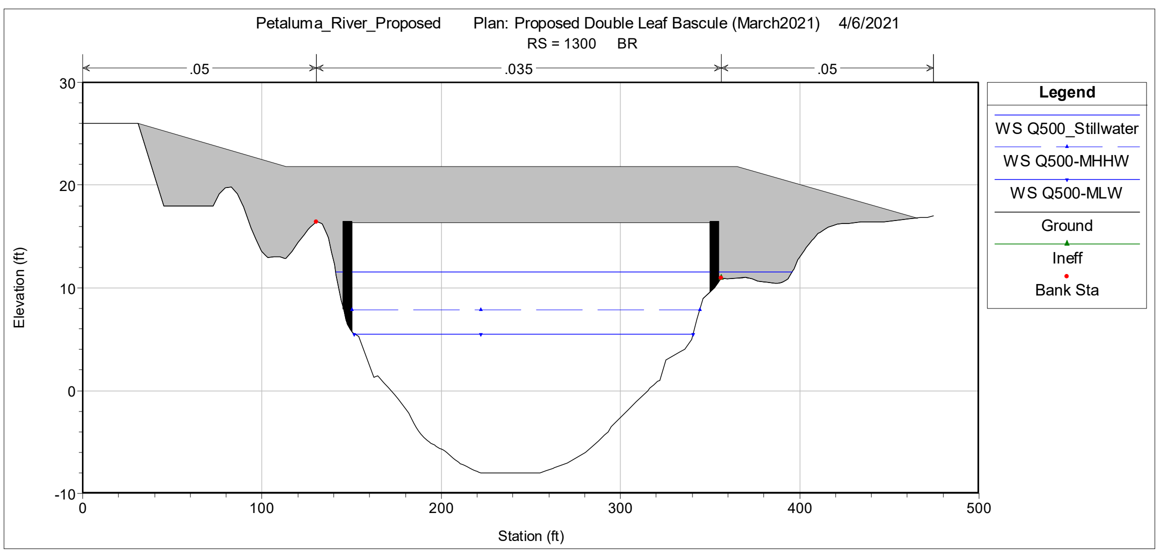

Figure 9. Upstream Face of Proposed Bridge, Looking Downstream (East)

The results of the hydraulic modeling indicated that the proposed condition would not result in an increase in WSEs relative to the existing condition with the studied reach of the river during the 50-, 100-, and 500-year storm evens with stillwater, MHHW, or MLW as downstream boundary condition.

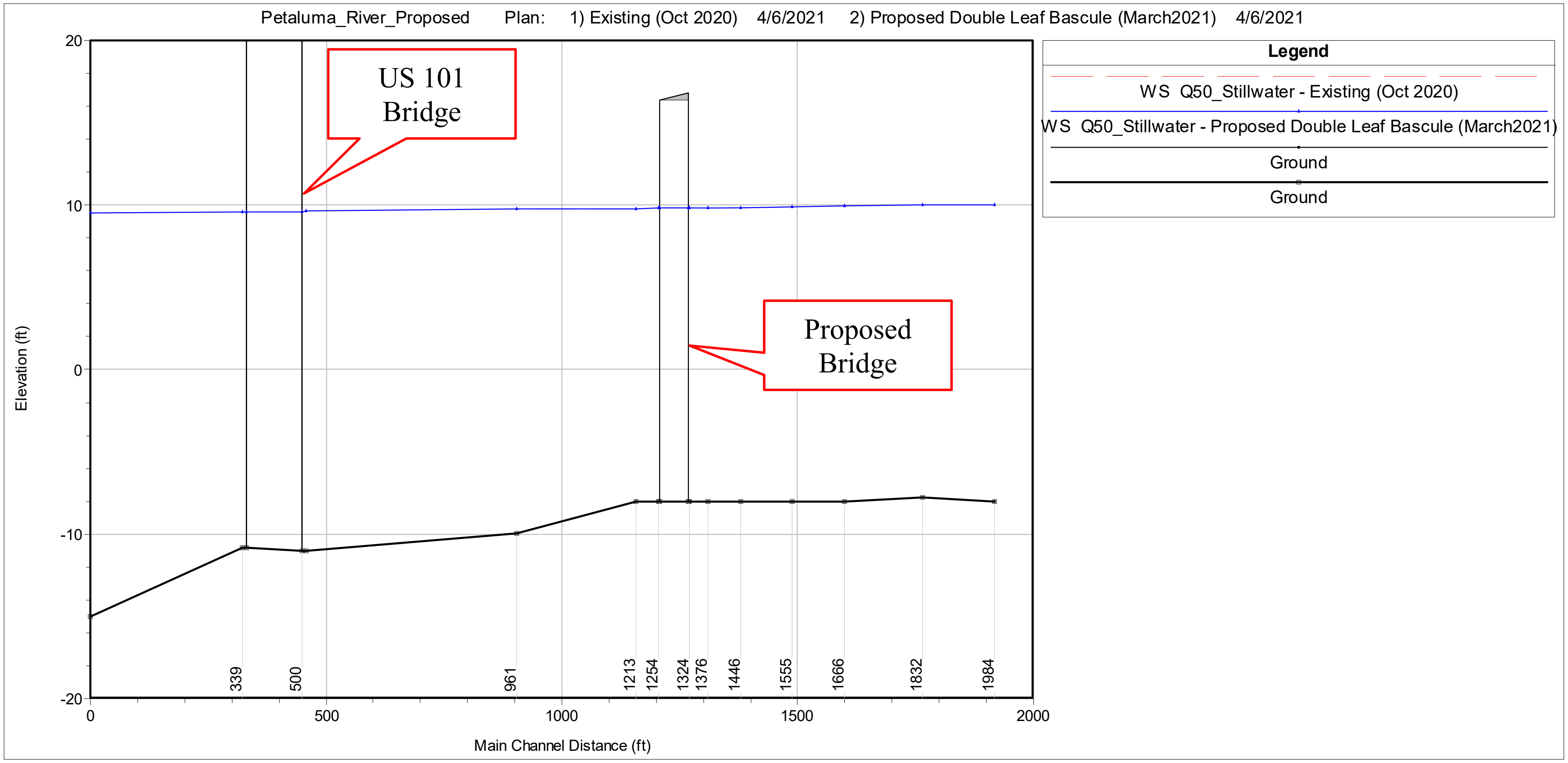

Figure 10. The Petaluma River 50-Year Water Surface Profile at Caulfield Lane

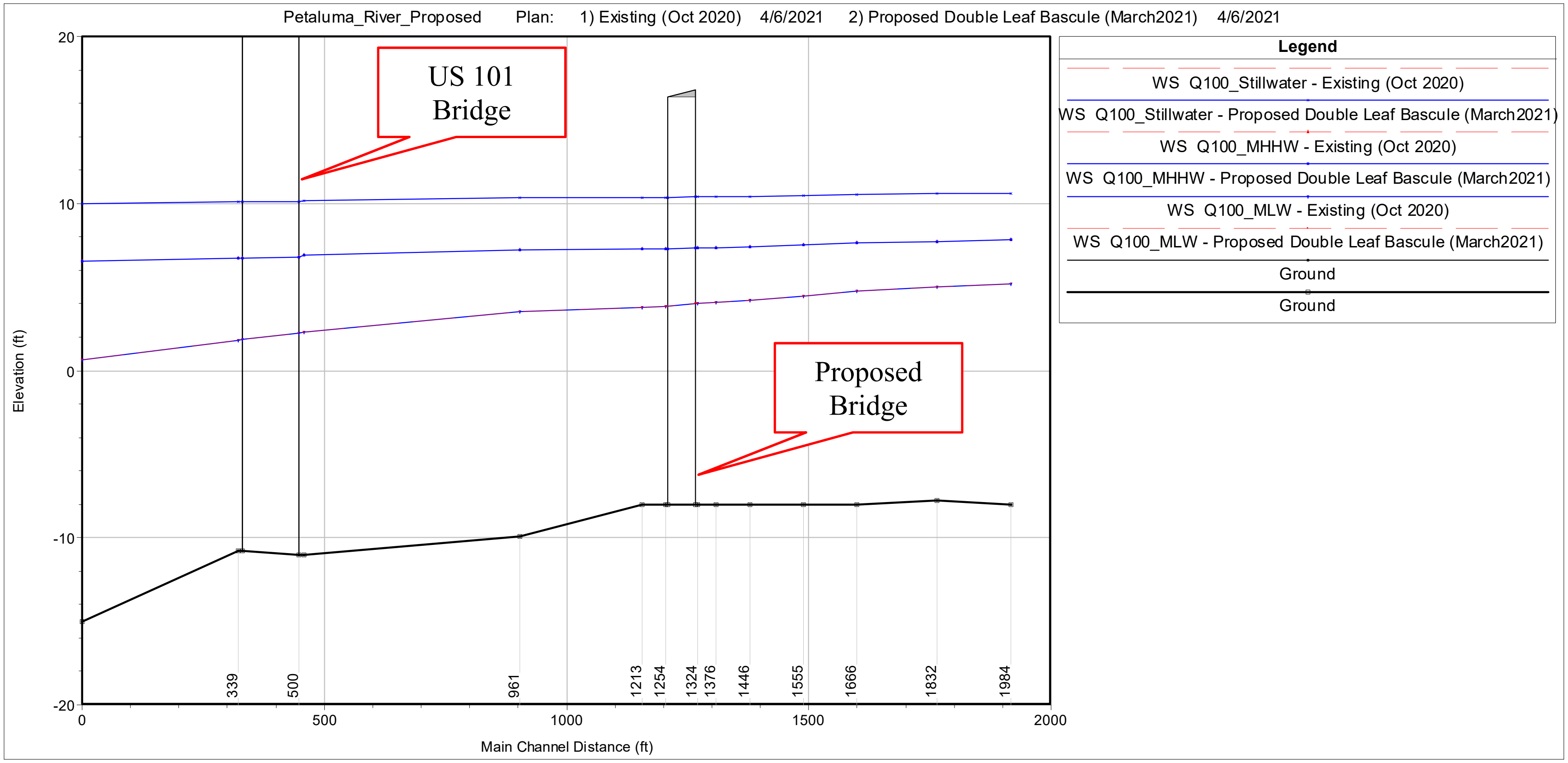

Figure 11. The Petaluma River 100-Year Water Surface Profile at Caulfield Lane

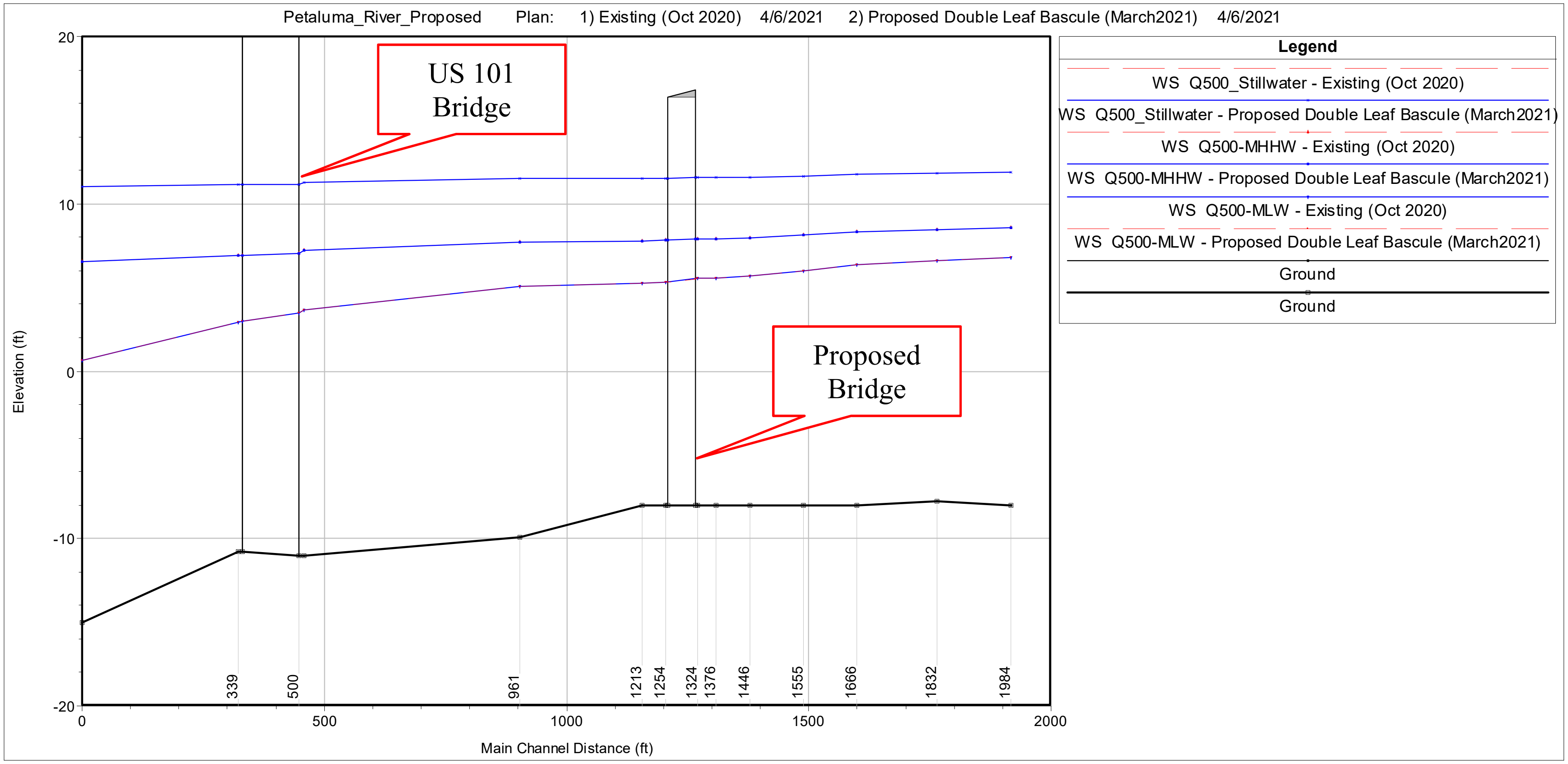

Figure 12. The Petaluma River 500-Year Water Surface Profile at Caulfield Lane

Per Caltrans' *Memo to Designers 16-1* (2017), the following Hydrologic Summary Table should be included on the proposed bridge general plan.

Table 8. Hydrologic Summary Table

Hydrologic Summary for Bridge No. xx-xxxx			
Drainage Area: 50.4 mi^2			
Frequency	**Design Flood**	**Base Flood**	**Flood or Record**
	50-Year	100-Year	N/A
Discharge	9,149 cfs	10,494 cfs	N/A
Water Surface Elevation at Bridge	9.8*	10.4*	N/A
Floodplain data are based upon information available when the plans were prepared and are shown to meet federal requirements. The accuracy of said information is not warranted by the State and interested or affected parties should make their own investigation.			

Note: *Water surface elevations are at the upstream face of the proposed bridge.
Bridge No. needs to be provided by Caltrans.

4.3.2 Freeboard

The freeboard requirements applicable to the Project are discussed in Section 1.7.1. The available freeboard for the proposed bridge is summarized in Table 9, which are representative of the minimum soffit elevations and WSEs at the upstream cross section of the proposed bridge with the 50-, 100- and 500-year storms with stillwater elevation as downstream boundary condition.

The hydraulic analysis results indicated that the proposed bridge has enough freeboard to pass the 50-, 100- and 500-year storm events. Therefore, the proposed bridge would meet the freeboard criteria of FHWA, Caltrans, USCG, and Sonoma County.

Table 9. 50-, 100-, and 500-Year Water Surface Elevations and Available Freeboard

Storm Event	**Bridge Soffit Elevation (ft NAVD 88)**	**WSE* (ft NAVD 88)**	**Available Freeboard* (ft)**
MHW	16.35	5.9	10.5
50-Year	16.35	9.8	6.6
100-Year	16.35	10.4	6.0
500-Year	16.35	11.6	4.8

Note: * WSEs and available freeboard were rounded to nearest 0.1 ft.

4.3.3 Flow Velocities

The 100- and 500-year average channel flow velocities were estimated for the existing and proposed conditions from the developed hydraulic models, which are summarized in Table 10 and Table 11, respectively, for the locations in the vicinity of the proposed bridge. Based on the results of the analysis, the proposed bridge would not result in an increase in average channel velocities during both the 100-year and 500-year storm events.

Table 10. The Petaluma River 100-Year Average Channel Velocities

RS	Location/Distance from Proposed Bridge Centerline	Average Channel Velocities* (ft/sec)		Velocity Difference (ft)
		Existing	Proposed	
With Stillwater as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	3.6	3.6	0.0
1300 BR U	Proposed Bridge Upstream Face	-	3.6	-
1300 BR D	Proposed Bridge Downstream Face	-	3.6	-
1254	Approximately 33 ft Downstream	3.6	3.6	0.0
With MHHW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	5.1	5.1	0.0
1300 BR U	Proposed Bridge Upstream Face	-	5.1	-
1300 BR D	Proposed Bridge Downstream Face	-	5.1	-
1254	Approximately 33 ft Downstream	5.1	5.1	0.0
With MLW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	7.3	7.3	0.0
1300 BR U	Proposed Bridge Upstream Face	-	7.3	-
1300 BR D	Proposed Bridge Downstream Face	-	7.4	-
1254	Approximately 33 ft Downstream	7.5	7.5	0.0

Note: * Average channel velocities were rounded to nearest 0.1 ft/sec.

Table 11. The Petaluma River 500-Year Average Channel Velocities

RS	Location/Distance from Proposed Bridge Centerline	Average Channel Velocities* (ft/sec)		Velocity Difference (ft)
		Existing	Proposed	
With Stillwater as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	4.7	4.7	0.0
1300 BR U	Proposed Bridge Upstream Face	-	4.7	-
1300 BR D	Proposed Bridge Downstream Face	-	4.8	-
1254	Approximately 33 ft Downstream	4.7	4.8	0.0

Table 11 (Continued). The Petaluma River 500-Year Average Channel Velocities

RS	Location/Distance from Proposed Bridge Centerline	Average Channel Velocities* (ft/sec)		Velocity Difference (ft)
		Existing	Proposed	
With Stillwater as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	4.7	4.7	0.0
1300 BR U	Proposed Bridge Upstream Face	-	4.7	-
1300 BR D	Proposed Bridge Downstream Face	-	4.8	-
1254	Approximately 33 ft Downstream	4.7	4.8	0.0
With MHHW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	7.9	6.3	-1.6
1300 BR U	Proposed Bridge Upstream Face	-	6.3	-
1300 BR D	Proposed Bridge Downstream Face	-	6.4	-
1254	Approximately 33 ft Downstream	7.8	6.4	-1.4
With MLW Elevation as Downstream Boundary Condition				
1324	Approximately 32 ft Upstream	8.0	8.0	0.0
1300 BR U	Proposed Bridge Upstream Face	-	8.0	-
1300 BR D	Proposed Bridge Downstream Face	-	8.2	-
1254	Approximately 33 ft Downstream	8.2	8.2	0.0

Note: * Average channel velocities were rounded to nearest 0.1 ft/sec.

5 SCOUR ANALYSIS

WRECO evaluated bridge scour per the criteria described in "Evaluating Scour at Bridges" (FHWA, 2012). The minimum design criterion for bridge scour is the 100-year design storm. The proposed bridge is in a tidally influenced area. Therefore, scour analysis was also performed for the 500-year storm even. WRECO evaluated the scour potential and scour countermeasure analysis using the results of the steady-state flow analysis from HEC-RAS for the proposed bridge. The scour calculations assume that the channel bed material is erodible. The following sub-sections summarize the results of the analysis.

5.1 Existing Channel Bed

Based on the particle size distribution of the bed sample collected at the railroad bridge over Petaluma River (URS Corporation, 2010), located approximately 1,100 ft downstream of the Project site, the existing channel bed varied from fine sand to coarse gravel (see Appendix D). The median particle size (D_{50}) from the distribution curve is 3.3 millimeters (mm), which is larger than 2 mm and can be considered as cohesionless bed material. The potential scour was assessed using the cohesionless equations.

5.2 Long-Term Bed Elevation Change

Long-term bed elevation changes can be due to either aggradation or degradation. Aggradation at the bridge site is a result of the deposition of material eroded from the channel. Degradation at the bridge site is a result of scouring of the channel due to sediment deficit. Only degradation is accounted for in scour calculations. Because the proposed bridge is located at a location where the river channel is navigable and the main channel will be dredged periodically to maintain channel depth for navigation, therefore, no long-term bed elevation change was considered for the Project.

5.3 Contraction Scour

For estimating contraction scour of cohesionless bed materials, HEC-18 recommends using the live-bed contraction scour equation when the critical velocity of the bed material is less than the mean velocity in the main channel, and considers clear-water contraction scour when the critical velocity of the bed material is greater than the mean velocity. The critical velocity was computed using the following equation:

$$V_c = K_u y^{1/6} D_{50}^{1/3}$$

Where:

V_c = Critical velocity above which bed material size of D and smaller will be transported (ft/s)
y = Average depth of flow upstream of the bridge (ft)
D_{50} = Particle size in a mixture of which 50 percent are smaller (ft)
K_u = 6.19 for SI unit and 11.17 for English units

Only live-bed contraction scour was estimated because the critical velocity of the channel was less than the mean approach velocity upstream of the proposed bridge for 100- and 500-year storms with different downstream boundary conditions such as stillwater, MHHW, or MLW elevations.

HEC-18 suggests a modified version of Laursen's 1960 equation to predict the depth of scour in a contracted section for live-bed scour. The equation assumes bed material is transported from the upstream section, and the equation is listed as follows:

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1}$$

$$y_s = y_2 - y_1$$

Where:

y_1 = Average depth in the upstream main channel (ft)
y_2 = Average depth in the contracted section (ft)
y_s = Scour depth (ft)
Q_1 = Flow in the upstream channel transporting sediment (ft^3/s)
Q_2 = Flow in the contracted channel (ft^3/s)
W_1 = Bottom width of the upstream main channel that is transporting bed material (ft)
W_2 = Bottom width of main channel in contracted section less pier width(s) (ft)
k_1 = Exponent determined on next page

V*/ω	k_1	Scour Hole Elevation
<0.50	0.59	Mode of Bed Material Transport
0.50 to 2.0	0.64	Mostly contact bed material discharge
>2.0	0.69	Some suspended bed material discharge

V^* = $(\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section (ft/s)
ω = Fall velocity of bed material based on D_{50}
g = Acceleration of gravity (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel (ft/ft)
τ_o = Shear stress on the bed (lb/ft^2)
ρ = Density of water (1.94 $slugs/ft^3$)

The estimated contraction scour depth is summarized in Table 12. The calculation of the cohesionless contraction scour for the proposed bridge is presented in Appendix E.

Table 12. Contraction Scour Summary

Storm Event	Downstream Boundary Condition	Contractor Scour Depth (ft)
100-Year	Stillwater Elevation	0.0
	MHHW Elevation	0.4
	MLW Elevation	0.6
500-Year	Stillwater Elevation	0.0
	MHHW Elevation	0.0
	MLW Elevation	0.2

5.4 Abutment Scour

Abutment scour occurs when the bridge abutments and roadway embankment block approaching flow. According to HEC-18, local scour at the bridge abutment is commonly evaluated using either the Froehlich or HIRE live-bed scour equation. The Froehlich equation is applicable when the ratio of the projected abutment length to the flow depth is less than 25. The HIRE equation is applicable when the ratio of the projected abutment length to the flow depth is greater than or equal to 25. Both equations assume the bed material around the bridge abutment is erodible during the 100-year storm event.

The Froehlich equation is given below:

$$y_s = y_a \left[2.27 K_1 K_2 \left(\frac{L'}{y_a} \right)^{0.43} Fr^{0.61} + 1 \right]$$

Where:

$y_s =$ Scour depth (ft)

$K_1 =$ Coefficient for abutment shape (from Table 8.1 of HEC-18)

$K_2 =$ Coefficient for angle of embankment to flow

$L' =$ Length of active flow obstructed by the embankment (ft)

$Fr =$ Froude number, based on the velocity and depth adjacent to and upstream of the abutment

$y_a =$ Average depth of flow at the abutment = A_e/L (ft)

- $L =$ Length of embankment projected normal to the flow (ft)
- $A_e =$ Flow area of the approach cross section obstructed by the embankment (ft^2)

The HIRE equation is listed below:

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} K_2$$

Where:

$y_s =$ Scour depth (ft)

y_1 = Depth of flow at the abutment on the overbank or in the main channel (ft)

Fr = Froude number, based on the velocity and depth adjacent to and upstream of the abutment

K_1 = Abutment shape coefficient (from Table 8.1 of HEC-18)

K_2 = Coefficient for skew angle of abutment to flow

The estimated local scour depth is summarized in Table 13. The calculation of the local scour for the proposed bridge is presented in Appendix E.

Table 13. Local Scour Depth

Storm Event	Downstream Boundary Condition	Calculated Local Scour Depth (ft)	
		Abutment 1 (Northwest)	Abutment 2 (Southeast)
100-Year	Stillwater Elevation	8.7	5.2
	MHHW Elevation	5.4	-*
	MLW Elevation	-*	-*
500-Year	Stillwater Elevation	11.4	6.1
	MHHW Elevation	6.8	2.4
	MLW Elevation	-*	-*

Note: *The ground elevation at the abutment is above the WSE.

5.5 Total Scour

The total scour is the sum of the local scour and contraction scour, and long-term bed elevation change, which was not considered for the proposed bridge. The calculated scour depths and elevations for the proposed bridge are summarized in Table 14 for Abutment 1, and Table 15 for Abutment 2. The total scour depths listed in the Table 14 and Table 15 are a combination of all scour components, assuming bed materials are erodible up to the depth of calculated scour. The scour elevations reference the finish grade elevations at the abutments, which requires that bank and embankment slope protection measures have been placed around Abutments 1 and 2.

Table 14. Total Scour Depth for Abutment 1

Storm Event	Downstream Boundary Condition	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Calculated Scour Elevation* (ft NAVD 88)
100-Year	Stillwater Elevation	0.0	8.7	8.7	-2.9
	MHHW Elevation	0.4	5.4	5.8	0.4
	MLW Elevation	0.6	-**	0.6	5.8
500-Year	Stillwater Elevation	0.0	11.4	11.4	-5.6
	MHHW Elevation	0.0	6.8	6.8	-1.0
	MLW Elevation	0.2	-**	0.2	5.8

Note: *The finished grade elevation at Abutment 1 is 5.8 ft NAVD 88.
**The ground elevation at the abutment is above the WSE.

Table 15. Total Scour Depth for Abutment 2

Storm Event	Downstream Boundary Condition	Contraction Scour (ft)	Local Scour (ft)	Total Scour (ft)	Calculated Scour Elevation* (ft NAVD 88)
100-Year	Stillwater Elevation	0.0	5.2	5.2	4.4
	MHHW Elevation	0.4	-**	0.4	9.6
	MLW Elevation	0.6	-**	0.6	9.6
500-Year	Stillwater Elevation	0.0	6.1	6.1	3.5
	MHHW Elevation	0.0	2.4	2.4	7.2
	MLW Elevation	0.2	-**	0.2	9.6

Note: *The finished grade elevation at Abutment 2 is 9.6 ft NAVD 88.
**The ground elevation at the abutment is above the WSE.

According to the Caltrans *Bridge Memo to Designers*, bridge footings supported on soil or degradable rock should be embedded below the maximum computed scour depth (2003). If the footing is supported by massive, competent rock formations resistant to scour, the footing should be placed directly on the cleaned rock surface.

If there is no bedrock existence at the Project site, the foundations should be designed to withstand the conditions of scour. Caltrans' *Memo to Designers 16-1* (2017) provides additional guidance on foundation placement:

> The top of a spread footing must be placed at or below the anticipated total scour (Degradation + Contraction + Local) elevation (*LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-1*) unless founded on competent, scour-resistant bedrock.
>
> The top of a pile cap footing must be placed at or below the estimated degradation plus contraction scour depth (LRFD 2.6.4.4.2 and LRFD-BDS-CA Figure C2.6.4.4.2-2). The bottom of a pile cap footing should be placed at or below the anticipated Total Scour elevation.

The scour data table (see Table 16) is the format Caltrans requires on the foundation plans. The scour data table was prepared by including the maximum contractor scour depth and local scour depth during 100- and 500-year storm events with different downstream boundary conditions to be conservative.

Table 16. Scour Data Table

Support No.	Calculated Long-Term (Long-Term Degradation and Contraction) Scour Elevation (ft NAVD 88)	Short-Term (Local) Scour Depth (ft)
Abutment 1	5.2	11.4
Abutment 2	9.0	6.1

6 SCOUR AND EROSION COUNTERMEASURES

RSP generally consists of rocks on channel and structure boundaries to limit the effects of erosion. It is the most common type of scour countermeasure due to its general availability, ease of installation, and relatively low cost. RSP sizing calculations were performed to estimate a minimum recommended rock class to protect the embankment slopes of the proposed bridge from scour and erosion.

RSP calculations estimate a minimum recommended rock size/class to protect the embankment slopes at the abutments from scour and erosion. Two procedures were considered to determine the RSP size for the proposed bridge: HEC-23 (FHWA, 2009) and the HDM (Caltrans, 2020). The calculation following both the HEC-23 and HDM resulted in Class I RSP (20 lb median particle weight) (see Appendix F).

A minimum size of Class IV RSP is recommended to protect the embankment slopes at the proposed bridge based on the engineering judgement. RSP is typically placed at the embankment slopes at the abutments. Class IV RSP has a median particle weight of 300 lb and a median particle diameter of 15 inches. The minimum layer thickness of the Class IV (300 lb) RSP is 3 ft. The RSP should extend from 2 ft above the 100-year WSE and from the face of the abutments to the toe of slope. The RSP should extend horizontally 25 ft from upstream and downstream faces of the bridge. The RSP should be placed using Method B, which involves dumping rock near its planned location, and working the rock to its final position with machinery. A Class 8 RSP geotextile filter fabric should be placed on the bank as the initial filter separator material between the layer of RSP and the channel bank. If the Class 8 RSP geotextile filter fabric is not feasible at the Project location, a layer of gravel filter composed of well compacted gravels with thickness of approximately 6 to 8 inches should be placed on the bank as a separate material between the RSP and the existing bed material.

7 REFERENCES

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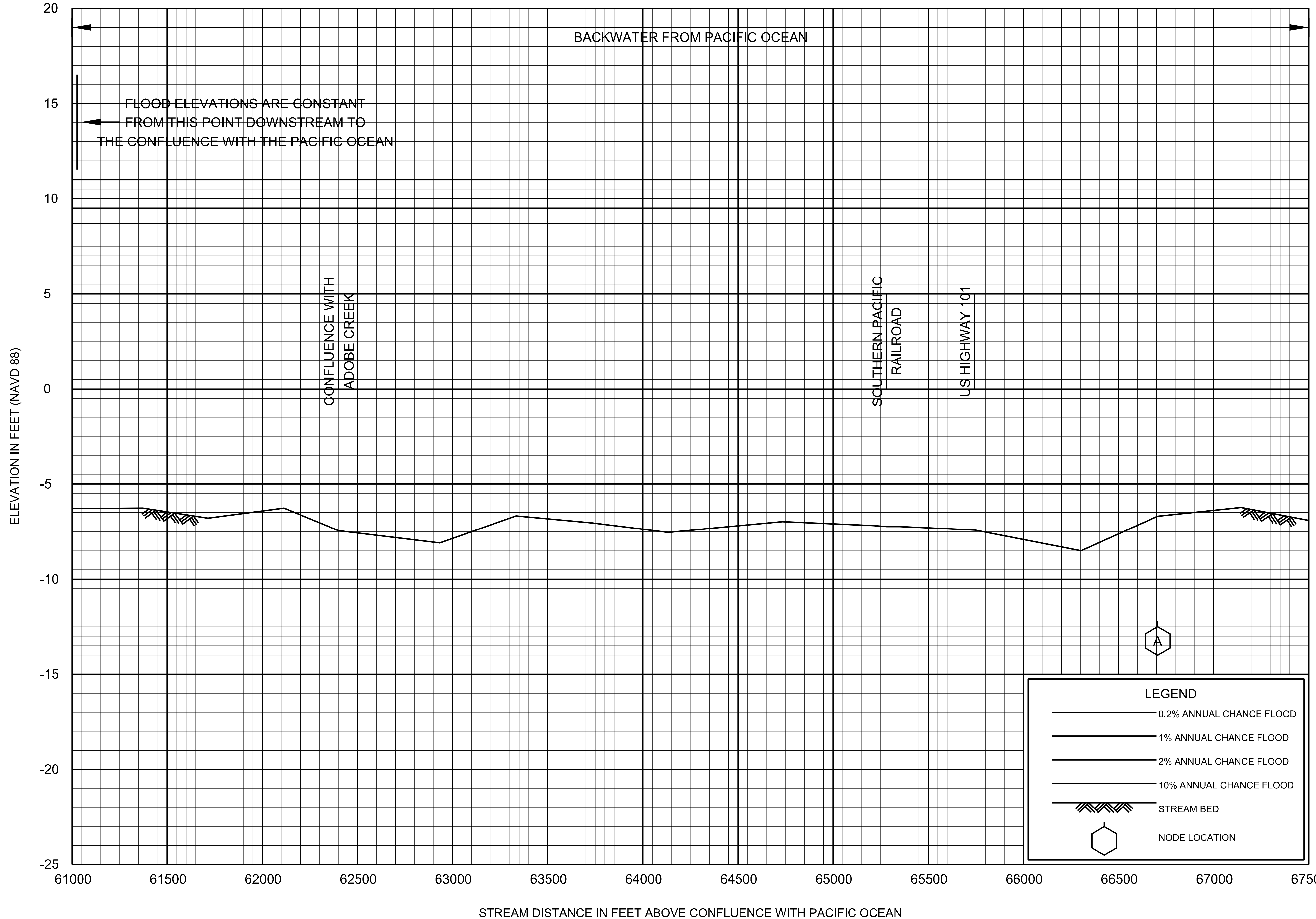
Appendix A FEMA FIS

Table 4. Summary of Discharges (continued)

Flooding Source and Location	Drainage Area (Square miles)	Peak Discharges (Cubic Feet per Second)			
		10-percent	2-percent	1-percent	0.2-percent
PETALUMA RIVER					
Downstream of confluence with Adobe Creek	58.93	8,672	11,034	11,910	15,044
At Highway 101 bridge	50.8	6,675	9,149	10,494	13,694
Downstream of confluence of Washington Creek	44.55	5,758	8,459	9,757	13,056
Downstream of confluence of Lynch Creek	39.55	5,246	7,492	8,671	11,563
Downstream of confluence of Capri Creek	34.60	4,653	6,583	7,728	10,523
Downstream of confluence of Willow Brook	29.31	3,587	4,825	5,360	6,733
Upstream of confluence of Willow Brook	14.97	1,701	2,947	3,529	4,801
POCKET CANYON					
Upstream of confluence with Russian River	6.56	1,790	2,650	3,050	3,880
Downstream of confluence of Mays Canyon	5.06	1,390	2,050	2,360	2,990
Downstream of confluence of Oregon Canyon	2.42	840	1,230	1,400	1,780
Upstream of confluence of Oregon Canyon	1.75	580	850	970	1,230
POOL CREEK					
Upstream of confluence with Windsor Creek	10.3	2,283	2,970[1]	3,258[1]	3,815[1]
Upstream of confluence of Pruitt Creek	4.91	1,358	1,874[1]	1,906[1]	2,165[1]
At Highway 101	3.75	1,152	1,520[1]	1,677[1]	2,030[1]
At Chalk Hill Road	1.67	684	895	987	1,200[1]
PRUITT CREEK					
Upstream of confluence with Pool Creek	3.99	925[1]	1,210[1]	1,311[1]	1,540[1]
At Shiloh Road (upstream crossing)	2.36	876[1]	1,146[1]	1,240[1]	1,455[1]
At Faught Road	1.94	535	618	767	930
REDWOOD CREEK					
Upstream of confluence with Airport Creek	1.12	490	640	715	880
At NWPER	0.30	115	150	160	205

[1]Reduced flows due to upstream losses



FLOOD PROFILES

PETALUMA RIVER

FEDERAL EMERGENCY MANAGEMENT AGENCY

SONOMA COUNTY, CA

AND INCORPORATED AREAS

breakout is possible for the 50-, 100-, and 500-year events just upstream of McDowell Boulevard in the left overbank, and for the 500-year event in the right overbank.

Corona Creek

Corona Creek is located northwest of Capri Creek and the model reach extends from approximately 3,000 feet upstream of Riesling Road to the confluence with Capri Creek at Highway 101.

Model results for the 100-year flood indicate a flow direction from the east side of Highway 101 to the west side (Petaluma River side) during peak flow conditions.

There are several locations where breakout flow could occur along Corona Creek. A breakout with depths greater than one foot is possible between Telford Lane and Andover Way for the 100- and 500- year events in the right overbank. Downstream of the Southern Pacific Railroad overflows are possible for the 50-, 100-, and 500- year events to the left into the Petaluma Estates Mobile Home Park and to the right into the Youngstown Senior Mobile home Park.

East Washington Creek

East Washington Creek study reach is from north end of the golf course, under the Petaluma Airport to the confluence with Washington Creek between McDowell Boulevard and Maria Drive. The available storage for the golf course upstream of the airport was entered as a stepwise linear storage relationship. Some of the links also convey flow from the left overbank breakout from the left overbank breakout from Washington Creek at McDowell Boulevard.

Lynch Creek

Lynch Creek is located northwest of Washington Creek. The model reach extends from the north end of the golf course to the confluence with Petaluma River. There are several areas where breakout flow could occur along Lynch Creek. The most upstream breakout is in the vicinity of the golf course. Breakout flow to the golf course occurs for all modeled events (10-, 50-, 100-, and 500-year events). The next four downstream breakouts occur at Sonoma Mountain Parkway, Sheila Street Court, Flanigan Way, and Maria Drive. The next breakout downstream occurs adjacent to Luchessi Park Lake. This breakout occurred during the New Year's Flood and, in addition, the pond itself overflowed into McDowell Boulevard and across the street into the Plaza North Shopping Center parking lot. The available storage for Luchessi Park Lake was entered as a stepwise linear storage relationship. Links were added to the model to evaluate flooding depth along the potential flow paths – some of these links also convey flow from the left overbank breakouts from Washington Creek.

Petaluma River

The model reach of the Petaluma River extends from the confluence of Liberty and Wiggins Creeks to the City limit downstream of the Adobe Creek confluence near the City limit. Several additional model elements were added to extend the model upstream to include the Liberty, Marin, Wiggins, and Wilson Creek watersheds for hydrologic purposes. The best available topography upstream of the confluence of the Petaluma River and Liberty and Wiggins Creek is USGS 7.5-minute topography at a 10-foot contour interval. Therefore, model cross sections and slopes in this area may not accurately describe the actual stream characteristics.

When the flood peak arrives there is some detention volume available in the Benson and Hummel properties as water overtops Stony Point Road into the Benson property. This detention effect significantly reduces the peak flow rate of the Petaluma River.

The upstream end of the recent USACE project creates a lateral constriction to direct flow away from nearby housing without flooding areas upstream.

The downstream boundary condition for the XP-SWMM model is the Mean Higher High Water (MHHW) tide condition equal to 6.53 feet NAVD.

Washington Creek

Washington Creek is northwest of Adobe Creek and includes a tributary, East Washington Creek. The study reach includes the north end of the Rooster Run Golf Course to the confluence with Petaluma River.

There are some areas of breakout flow that could occur along Washington Creek. The most upstream breakout is located between Sparrow and Songbird Streets in the right overbank for the 100- and 500-year events. A portion of this breakout flow returns to the channel downstream of Ely Road. The next downstream breakout occurs at Rene and Lauren Streets in the right overbank for the 500-year event only, followed by Maria Drive in the right overbank for the 500-year event only. Downstream of East Washington Creek the next breakout is at McDowell Boulevard to the right and left overbanks for the 100- and 500-year events between McDowell and Highway 101. The last breakout for Washington Creek is located at Madison Avenue in the right and left overbanks for both the 100- and 500-year events.

Willow Brook

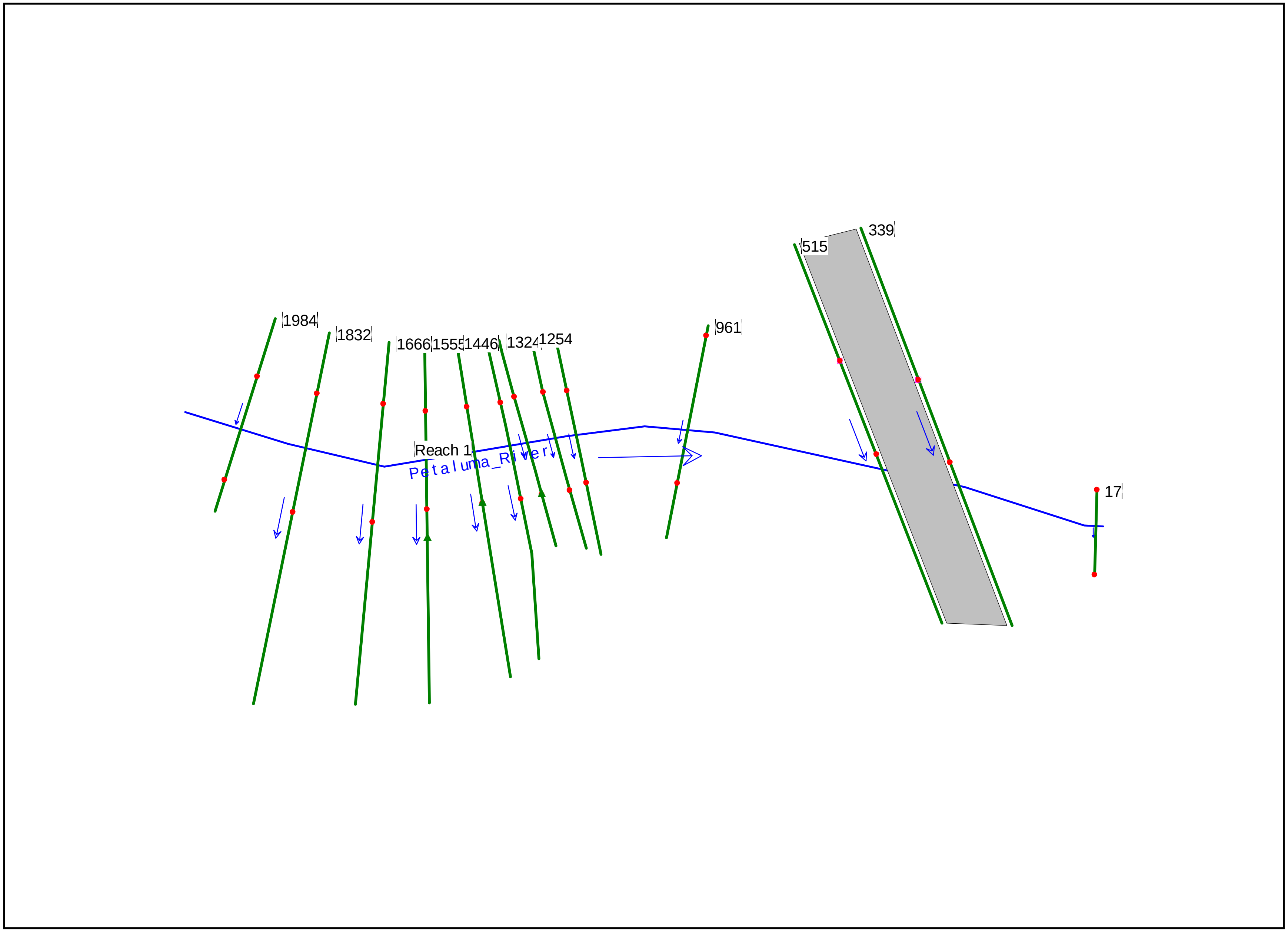
The Willow Brook model extends from Ely Road to the confluence with Petaluma River. There are several locations where breakout flow occurs downstream of Ely Road. The model was revised to evaluate overflows in the Willow Brook Breakout Area and Old Redwood Highway.

The North Corona Channel (a ditch along the north side of Corona Road) is being removed as a flooding source in this revision because the majority of flooding in the North Corona Channel area originates from Willow Brook breaking out of bank. Therefore, the floodplain in the vicinity of the North Corona Channel will be mapped as part of the Willow Brook breakout (Zone AE). The North Corona Channel also receives flow from a roadside ditch along Corona Road upstream of the Southern Pacific Railroad. This flow contribution is accounted for in the model.

Floodway Revisions

The Petaluma River floodway was revised along the entire floodway length to reflect better topography, to avoid maximum surcharges greater than one foot, and to represent equal conveyance reduction. Floodway revisions followed the FEMA guidelines for

Appendix B HEC-RAS Results: Existing Condition



1984
1832
1666
1555
1446
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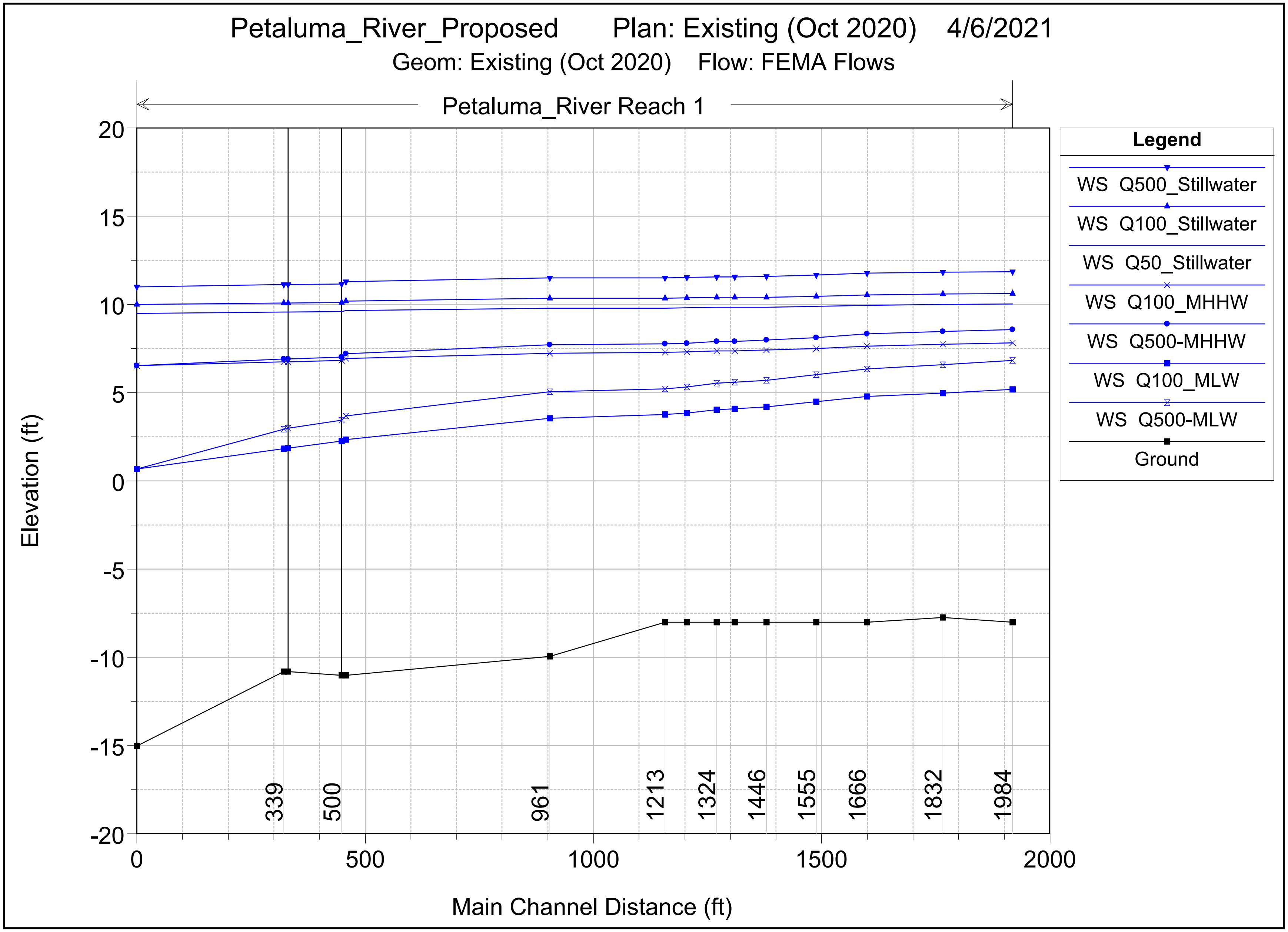
Reach 1
Petaluma_River



Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
Petaluma_River Reach 1
Legend
WS Q500_Stillwater
WS Q100_Stillwater
WS Q50_Stillwater
WS Q100_MHHW
WS Q500-MHHW
WS Q100_MLW
WS Q500-MLW
Ground
Elevation (ft)
Main Channel Distance (ft)
20
15
10
5
0
-5
-10
-15
-20
0
500
1000
1500
2000
339
500
961
1213
1324
1446
1555
1666
1832
1984

HEC-RAS Plan: Existing (Oct 2020) River: Petaluma_River Reach: Reach 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1984	Q50_Stillwater	9149.00	-8.00	10.02		10.21	0.000286	3.55	2575.66	229.29	0.19
Reach 1	1984	Q100_Stillwater	10494.00	-8.00	10.61		10.84	0.000321	3.87	2712.62	231.49	0.20
Reach 1	1984	Q500_Stillwater	13694.00	-8.00	11.86		12.18	0.000403	4.56	3005.87	242.56	0.23
Reach 1	1984	Q100_MHHW	10494.00	-8.00	7.82		8.21	0.000727	5.05	2079.49	221.19	0.29
Reach 1	1984	Q100_MLW	10494.00	-8.00	5.20		5.93	0.001520	6.84	1534.04	179.74	0.41
Reach 1	1984	Q500-MHHW	13694.00	-8.00	8.57		9.15	0.000976	6.09	2247.59	224.56	0.34
Reach 1	1984	Q500-MLW	13694.00	-8.00	6.82		7.66	0.001701	7.35	1862.49	213.30	0.44
Reach 1	1832	Q50_Stillwater	9149.00	-7.75	9.99		10.16	0.000256	3.33	2751.03	248.13	0.18
Reach 1	1832	Q100_Stillwater	10494.00	-7.75	10.58		10.79	0.000286	3.62	2898.69	250.13	0.19
Reach 1	1832	Q500_Stillwater	13694.00	-7.75	11.83		12.11	0.000355	4.26	3213.13	254.96	0.21
Reach 1	1832	Q100_MHHW	10494.00	-7.75	7.74		8.09	0.000674	4.77	2200.75	240.14	0.28
Reach 1	1832	Q100_MLW	10494.00	-7.75	4.98		5.67	0.001649	6.68	1570.89	202.96	0.42
Reach 1	1832	Q500-MHHW	13694.00	-7.75	8.47		8.98	0.000902	5.76	2377.18	242.88	0.32
Reach 1	1832	Q500-MLW	13694.00	-7.75	6.60		7.38	0.001724	7.10	1929.53	235.05	0.44
Reach 1	1666	Q50_Stillwater	9149.00	-8.00	9.95		10.12	0.000229	3.30	2776.19	234.47	0.17
Reach 1	1666	Q100_Stillwater	10494.00	-8.00	10.54		10.74	0.000261	3.60	2914.47	237.08	0.18
Reach 1	1666	Q500_Stillwater	13694.00	-8.00	11.77		12.05	0.000352	4.26	3215.00	272.02	0.21
Reach 1	1666	Q100_MHHW	10494.00	-8.00	7.64		7.98	0.000580	4.68	2244.37	225.90	0.26
Reach 1	1666	Q100_MLW	10494.00	-8.00	4.78		5.41	0.001339	6.39	1642.29	193.98	0.39
Reach 1	1666	Q500-MHHW	13694.00	-8.00	8.34		8.84	0.000803	5.70	2401.37	228.68	0.31
Reach 1	1666	Q500-MLW	13694.00	-8.00	6.36		7.11	0.001416	6.99	1959.60	210.88	0.40
Reach 1	1555	Q50_Stillwater	9149.00	-8.00	9.89		10.09	0.000262	3.59	2549.40	207.54	0.18
Reach 1	1555	Q100_Stillwater	10494.00	-8.00	10.47		10.71	0.000303	3.93	2671.21	273.02	0.19
Reach 1	1555	Q500_Stillwater	13694.00	-8.00	11.67		12.01	0.000384	4.68	3043.57	569.26	0.22
Reach 1	1555	Q100_MHHW	10494.00	-8.00	7.51		7.91	0.000636	5.07	2069.10	195.79	0.28
Reach 1	1555	Q100_MLW	10494.00	-8.00	4.48		5.24	0.001612	6.98	1503.78	178.09	0.42
Reach 1	1555	Q500-MHHW	13694.00	-8.00	8.13		8.74	0.000916	6.25	2190.91	198.92	0.33
Reach 1	1555	Q500-MLW	13694.00	-8.00	6.01		6.93	0.001677	7.69	1781.12	187.18	0.44
Reach 1	1446	Q50_Stillwater	9149.00	-8.00	9.85		10.06	0.000271	3.71	2469.01	196.40	0.18
Reach 1	1446	Q100_Stillwater	10494.00	-8.00	10.41		10.67	0.000317	4.07	2581.39	206.50	0.20
Reach 1	1446	Q500_Stillwater	13694.00	-8.00	11.60		11.96	0.000425	4.84	2963.21	580.73	0.23
Reach 1	1446	Q100_MHHW	10494.00	-8.00	7.41		7.84	0.000668	5.24	2002.50	187.17	0.28
Reach 1	1446	Q100_MLW	10494.00	-8.00	4.20		5.04	0.001818	7.34	1429.17	171.81	0.45
Reach 1	1446	Q500-MHHW	13694.00	-8.00	7.97		8.63	0.000972	6.50	2107.97	188.97	0.34
Reach 1	1446	Q500-MLW	13694.00	-8.00	5.71		6.73	0.001848	8.10	1691.11	176.88	0.46

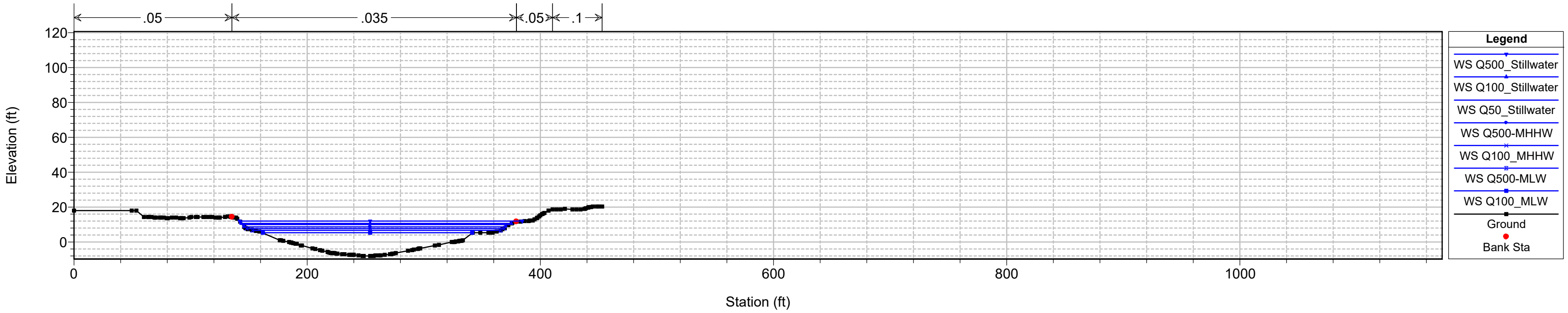
HEC-RAS Plan: Existing (Oct 2020) River: Petaluma_River Reach: Reach 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1376	Q50_Stillwater	9149.00	-8.00	9.83		10.04	0.000268	3.70	2474.39	195.49	0.18
Reach 1	1376	Q100_Stillwater	10494.00	-8.00	10.39		10.65	0.000313	4.06	2585.46	199.30	0.20
Reach 1	1376	Q500_Stillwater	13694.00	-8.00	11.57		11.93	0.000432	4.84	2895.52	425.48	0.23
Reach 1	1376	Q100_MHHW	10494.00	-8.00	7.37		7.79	0.000635	5.22	2010.69	181.30	0.28
Reach 1	1376	Q100_MLW	10494.00	-8.00	4.09		4.91	0.001680	7.25	1446.79	166.44	0.43
Reach 1	1376	Q500-MHHW	13694.00	-8.00	7.91		8.56	0.000945	6.49	2109.05	184.52	0.34
Reach 1	1376	Q500-MLW	13694.00	-8.00	5.59		6.60	0.001741	8.06	1698.01	170.12	0.45
Reach 1	1324	Q50_Stillwater	9149.00	-8.00	9.83		10.03	0.000254	3.55	2574.87	208.40	0.18
Reach 1	1324	Q100_Stillwater	10494.00	-8.00	10.40		10.63	0.000294	3.90	2693.39	211.60	0.19
Reach 1	1324	Q500_Stillwater	13694.00	-8.00	11.57		11.91	0.000381	4.65	2967.25	254.65	0.22
Reach 1	1324	Q100_MHHW	10494.00	-8.00	7.36		7.76	0.000632	5.05	2076.54	197.34	0.27
Reach 1	1324	Q100_MLW	10494.00	-8.00	4.02		4.84	0.001856	7.27	1444.27	179.58	0.45
Reach 1	1324	Q500-MHHW	13694.00	-8.00	7.90		8.51	0.000923	6.27	2183.16	199.01	0.33
Reach 1	1324	Q500-MLW	13694.00	-8.00	5.53		6.52	0.001888	7.95	1722.28	189.04	0.46
Reach 1	1254	Q50_Stillwater	9149.00	-8.00	9.81		10.01	0.000256	3.61	2537.70	202.05	0.18
Reach 1	1254	Q100_Stillwater	10494.00	-8.00	10.37		10.61	0.000297	3.96	2651.45	205.35	0.19
Reach 1	1254	Q500_Stillwater	13694.00	-8.00	11.53		11.88	0.000396	4.73	2894.60	212.51	0.23
Reach 1	1254	Q100_MHHW	10494.00	-8.00	7.30		7.71	0.000648	5.14	2042.93	193.38	0.28
Reach 1	1254	Q100_MLW	10494.00	-8.00	3.84		4.71	0.001901	7.45	1408.34	171.84	0.46
Reach 1	1254	Q500-MHHW	13694.00	-8.00	7.81		8.44	0.000957	6.40	2140.65	195.00	0.34
Reach 1	1254	Q500-MLW	13694.00	-8.00	5.32		6.37	0.001994	8.21	1668.57	182.17	0.48
Reach 1	1213	Q50_Stillwater	9149.00	-8.00	9.80		10.00	0.000251	3.60	2540.93	199.16	0.18
Reach 1	1213	Q100_Stillwater	10494.00	-8.00	10.35		10.60	0.000290	3.96	2652.36	200.93	0.19
Reach 1	1213	Q500_Stillwater	13694.00	-8.00	11.51		11.86	0.000383	4.74	2887.44	212.47	0.22
Reach 1	1213	Q100_MHHW	10494.00	-8.00	7.27		7.68	0.000638	5.12	2047.68	191.42	0.28
Reach 1	1213	Q100_MLW	10494.00	-8.00	3.75		4.62	0.001912	7.45	1408.10	172.03	0.46
Reach 1	1213	Q500-MHHW	13694.00	-8.00	7.76		8.40	0.000949	6.39	2141.50	193.17	0.34
Reach 1	1213	Q500-MLW	13694.00	-8.00	5.23		6.28	0.001964	8.22	1665.46	178.49	0.47
Reach 1	961	Q50_Stillwater	9149.00	-9.94	9.79		9.93	0.000160	2.95	3102.19	235.34	0.14
Reach 1	961	Q100_Stillwater	10494.00	-9.94	10.35		10.51	0.000185	3.24	3234.21	237.35	0.15
Reach 1	961	Q500_Stillwater	13694.00	-9.94	11.51		11.75	0.000247	3.90	3512.47	242.26	0.18
Reach 1	961	Q100_MHHW	10494.00	-9.94	7.24		7.51	0.000400	4.17	2514.02	226.19	0.22
Reach 1	961	Q100_MLW	10494.00	-9.94	3.55		4.14	0.001303	6.16	1704.54	209.27	0.38
Reach 1	961	Q500-MHHW	13694.00	-9.94	7.72		8.15	0.000598	5.22	2622.54	227.90	0.27
Reach 1	961	Q500-MLW	13694.00	-9.94	5.07		5.77	0.001316	6.75	2029.85	218.36	0.39

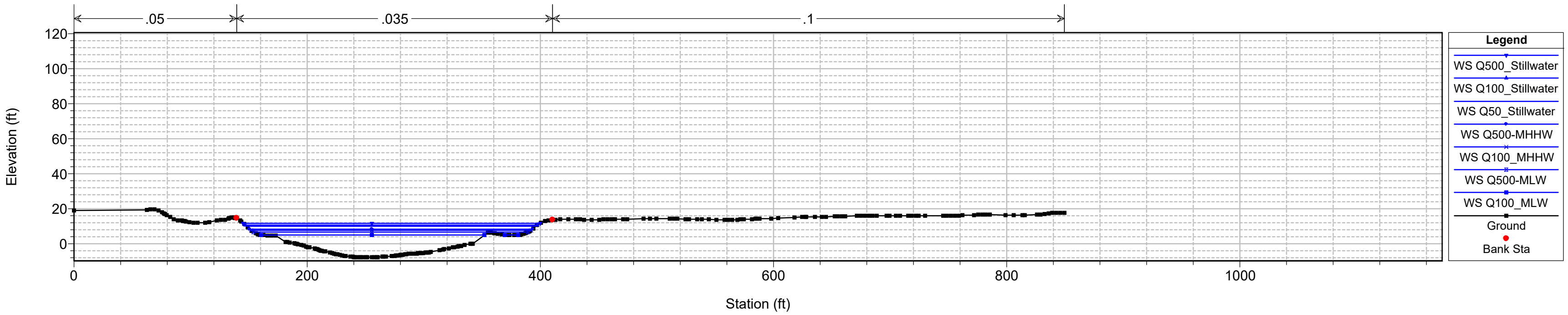
HEC-RAS Plan: Existing (Oct 2020)   River: Petaluma_River   Reach: Reach 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	515	Q50_Stillwater	9149.00	-11.02	9.66	-2.31	9.84	0.000212	3.41	2681.22	198.80	0.16
Reach 1	515	Q100_Stillwater	10494.00	-11.02	10.19	-1.67	10.41	0.000247	3.76	2787.58	200.08	0.18
Reach 1	515	Q500_Stillwater	13694.00	-11.02	11.29	-0.30	11.61	0.000353	4.55	3008.34	212.11	0.21
Reach 1	515	Q100_MHHW	10494.00	-11.02	6.92	-1.67	7.30	0.000552	4.89	2146.32	192.24	0.26
Reach 1	515	Q100_MLW	10494.00	-11.02	2.33	-1.67	3.34	0.002287	8.05	1303.17	161.98	0.50
Reach 1	515	Q500-MHHW	13694.00	-11.02	7.21	-0.30	7.81	0.000870	6.22	2200.64	192.92	0.32
Reach 1	515	Q500-MLW	13694.00	-11.02	3.68	-0.30	4.91	0.002625	8.92	1535.55	181.63	0.54
Reach 1	500 BR U	Q50_Stillwater	9149.00	-11.02	9.60	-2.31	9.82	0.000329	3.82	2394.10	159.31	0.17
Reach 1	500 BR U	Q100_Stillwater	10494.00	-11.02	10.11	-1.67	10.39	0.000394	4.24	2476.51	160.03	0.19
Reach 1	500 BR U	Q500_Stillwater	13694.00	-11.02	11.16	-0.30	11.58	0.000598	5.18	2644.43	149.48	0.22
Reach 1	500 BR U	Q100_MHHW	10494.00	-11.02	6.82	-1.67	7.27	0.000766	5.36	1956.95	155.45	0.27
Reach 1	500 BR U	Q100_MLW	10494.00	-11.02	2.26	-1.67	3.31	0.002440	8.19	1281.13	137.09	0.47
Reach 1	500 BR U	Q500-MHHW	13694.00	-11.02	7.02	-0.30	7.76	0.001247	6.89	1988.09	155.73	0.34
Reach 1	500 BR U	Q500-MLW	13694.00	-11.02	3.44	-0.30	4.84	0.003009	9.47	1445.74	143.64	0.53
Reach 1	500 BR D	Q50_Stillwater	9149.00	-10.80	9.58	-2.36	9.78	0.000252	3.62	2526.89	178.48	0.17
Reach 1	500 BR D	Q100_Stillwater	10494.00	-10.80	10.09	-1.72	10.34	0.000300	4.01	2618.67	179.55	0.18
Reach 1	500 BR D	Q500_Stillwater	13694.00	-10.80	11.12	-0.36	11.49	0.000425	4.88	2805.41	183.36	0.22
Reach 1	500 BR D	Q100_MHHW	10494.00	-10.80	6.75	-1.72	7.17	0.000628	5.17	2029.16	174.11	0.27
Reach 1	500 BR D	Q100_MLW	10494.00	-10.80	1.86	-1.72	2.99	0.002525	8.54	1228.98	149.33	0.52
Reach 1	500 BR D	Q500-MHHW	13694.00	-10.80	6.91	-0.36	7.60	0.001030	6.66	2056.57	174.61	0.34
Reach 1	500 BR D	Q500-MLW	13694.00	-10.80	2.98	-0.36	4.46	0.003018	9.76	1403.31	158.77	0.58
Reach 1	339	Q50_Stillwater	9149.00	-10.80	9.58	-2.36	9.77	0.000225	3.51	2608.42	190.48	0.17
Reach 1	339	Q100_Stillwater	10494.00	-10.80	10.09	-1.72	10.33	0.000268	3.88	2706.74	193.45	0.18
Reach 1	339	Q500_Stillwater	13694.00	-10.80	11.13	-0.36	11.48	0.000390	4.70	2915.06	207.47	0.22
Reach 1	339	Q100_MHHW	10494.00	-10.80	6.76	-1.72	7.15	0.000601	5.05	2076.74	186.13	0.27
Reach 1	339	Q100_MLW	10494.00	-10.80	1.82	-1.72	2.96	0.002547	8.58	1223.76	148.70	0.53
Reach 1	339	Q500-MHHW	13694.00	-10.80	6.92	-0.36	7.57	0.000981	6.50	2106.55	186.62	0.34
Reach 1	339	Q500-MLW	13694.00	-10.80	2.94	-0.36	4.43	0.003186	9.77	1401.06	165.67	0.59
Reach 1	17	Q50_Stillwater	9149.00	-15.03	9.50	-3.81	9.67	0.000364	3.29	2776.96	216.51	0.16
Reach 1	17	Q100_Stillwater	10494.00	-15.03	10.00	-3.10	10.21	0.000430	3.64	2885.83	218.98	0.18
Reach 1	17	Q500_Stillwater	13694.00	-15.03	11.00	-1.61	11.30	0.000595	4.41	3107.28	223.93	0.21
Reach 1	17	Q100_MHHW	10494.00	-15.03	6.53	-3.10	6.90	0.000970	4.87	2156.06	199.77	0.26
Reach 1	17	Q100_MLW	10494.00	-15.03	0.67	-3.10	1.91	0.004116	8.92	1176.16	138.35	0.54
Reach 1	17	Q500-MHHW	13694.00	-15.03	6.53	-1.61	7.16	0.001651	6.35	2156.06	199.77	0.34
Reach 1	17	Q500-MLW	13694.00	-15.03	0.67	-1.62	2.78	0.007009	11.64	1176.16	138.35	0.70

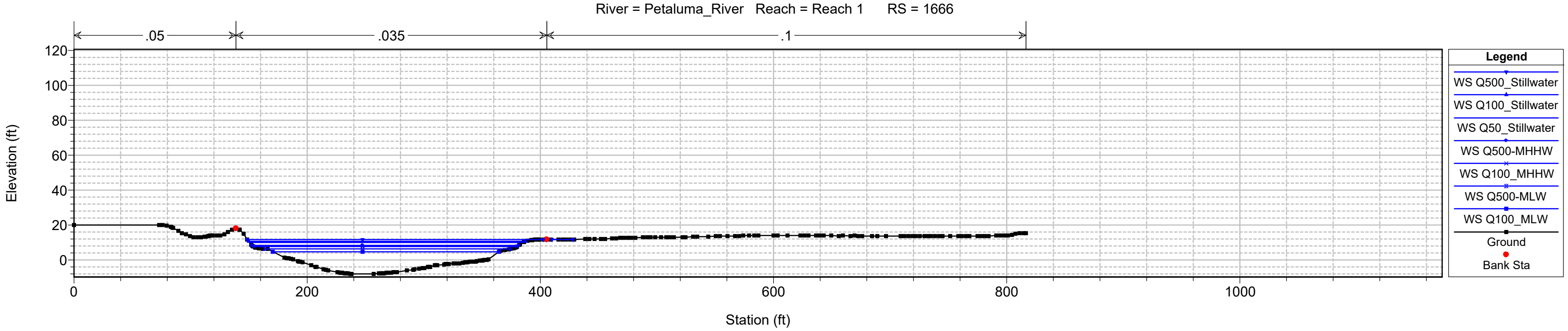
Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 1984   At "Guadalupe Ct"



Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 1832

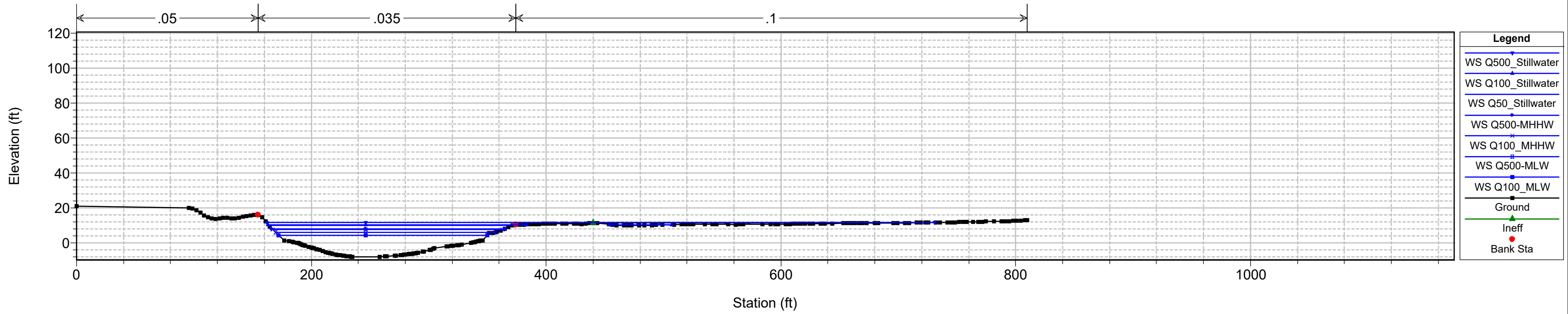


Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 1666

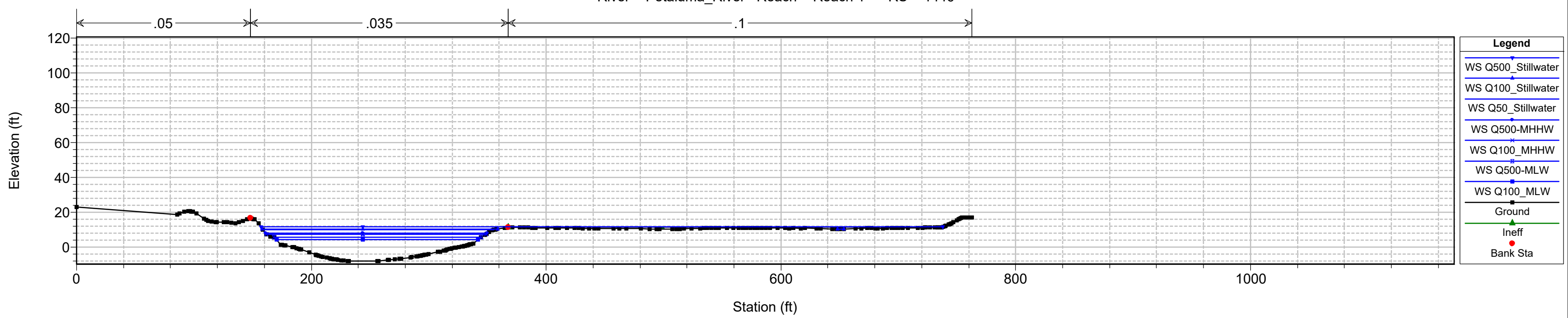


1 in Horiz. = 150 ft   1 in Vert. = 100 ft

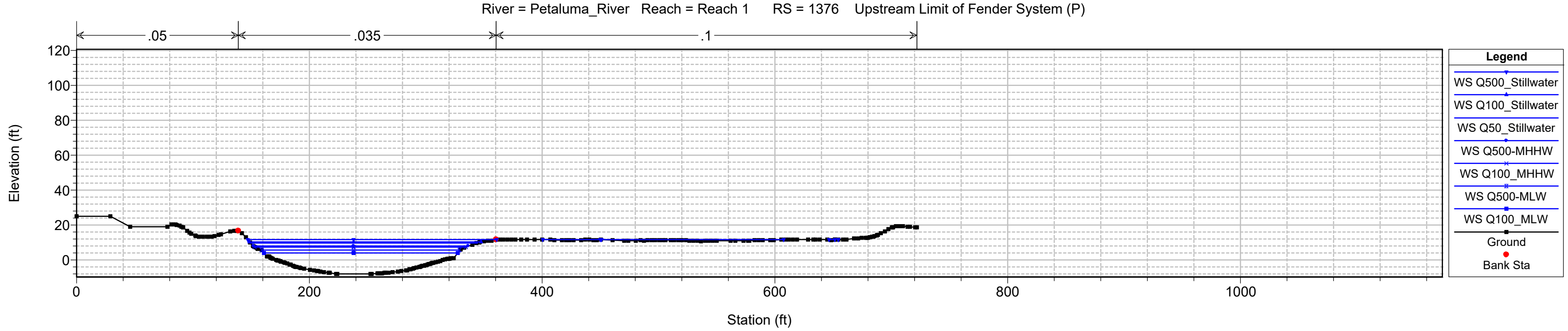
Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1555



Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1446

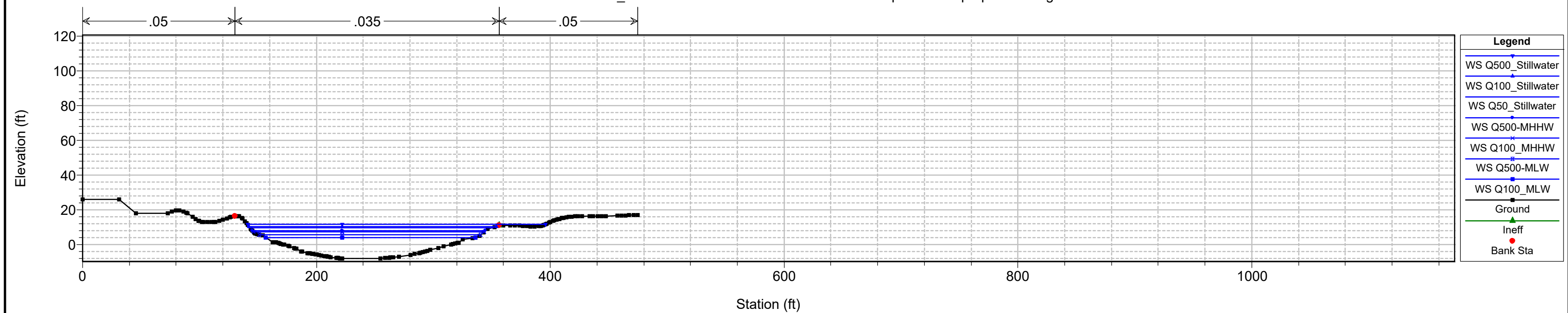


Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1376 Upstream Limit of Fender System (P)

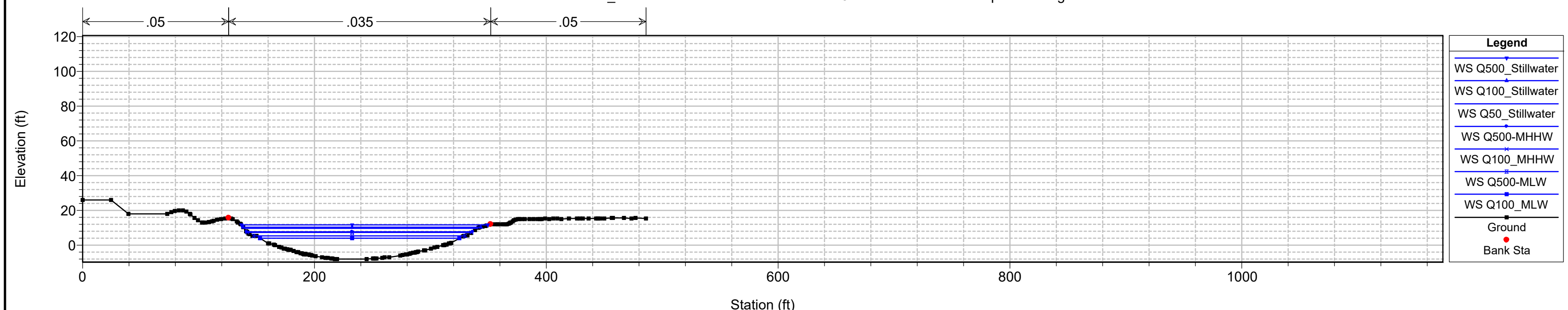


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

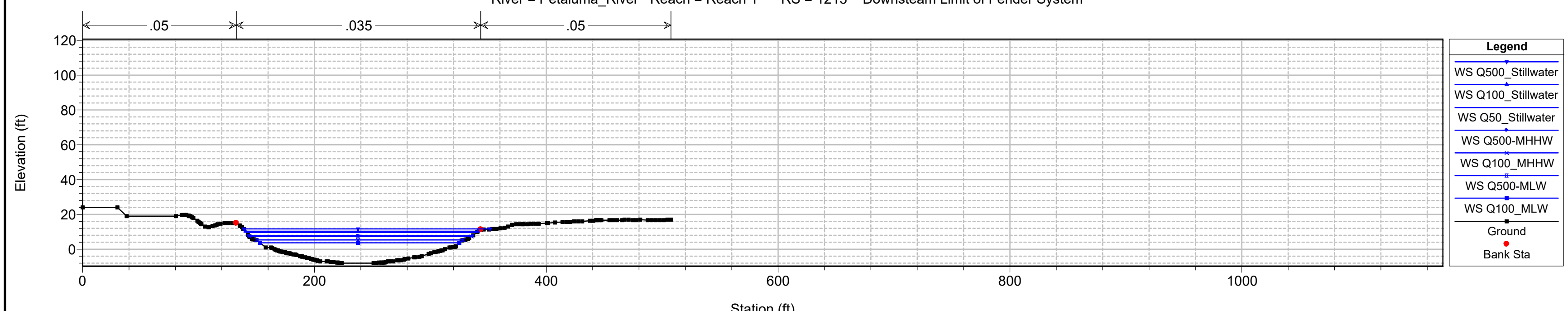
Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1324 Upstream of proposed bridge



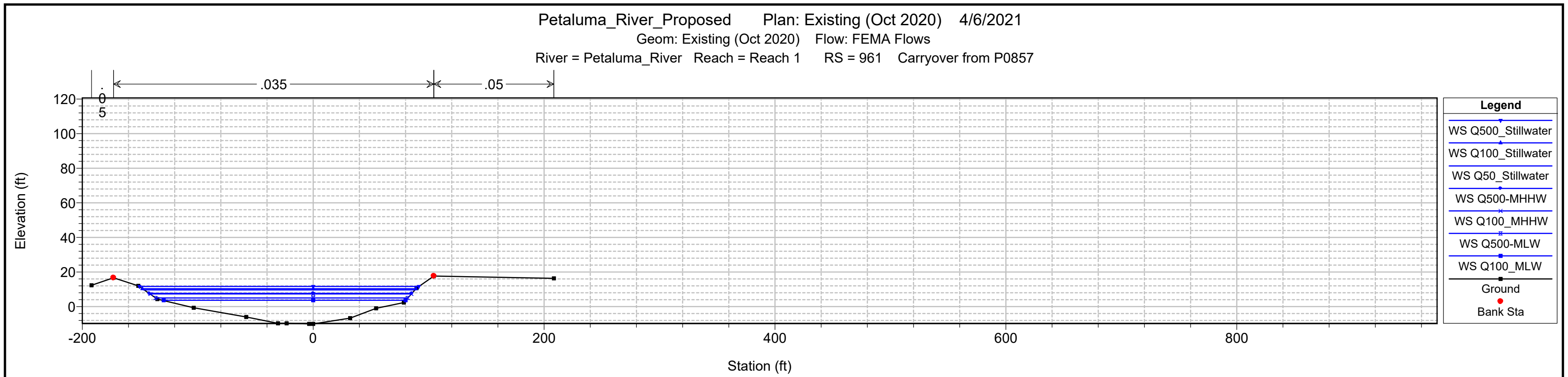
Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1254 Downstream of Proposed Bridge

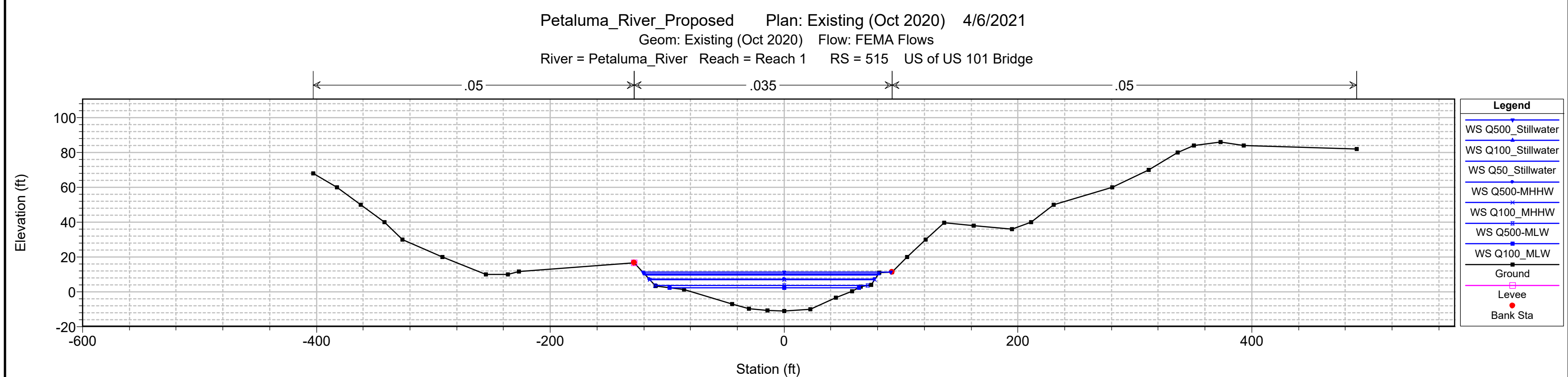


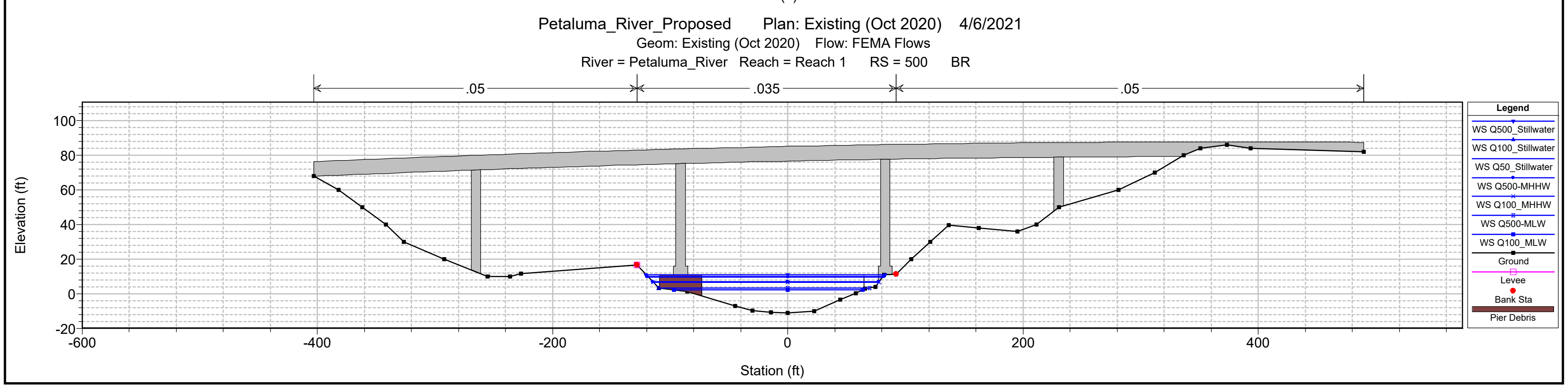
Petaluma_River_Proposed Plan: Existing (Oct 2020) 4/6/2021
Geom: Existing (Oct 2020) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1213 Downsteam Limit of Fender System



1 in Horiz. = 150 ft 1 in Vert. = 100 ft

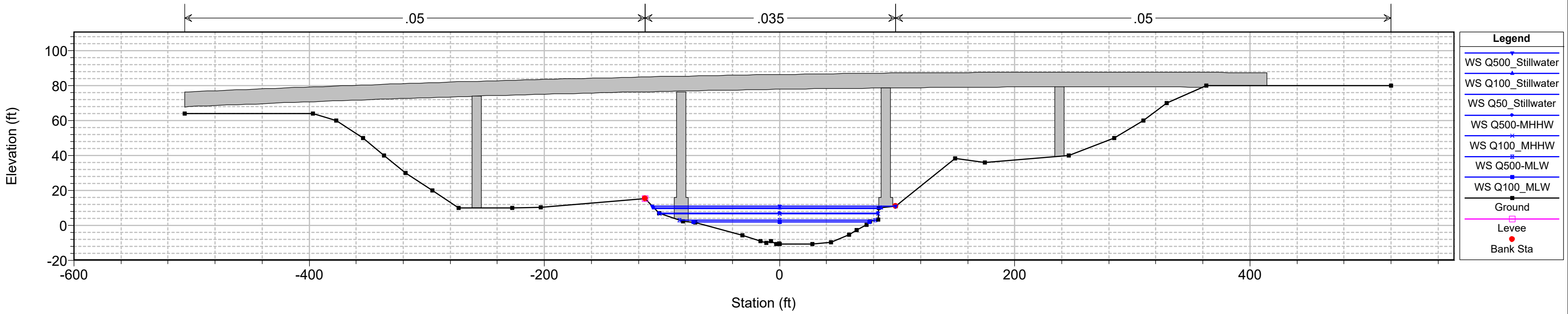




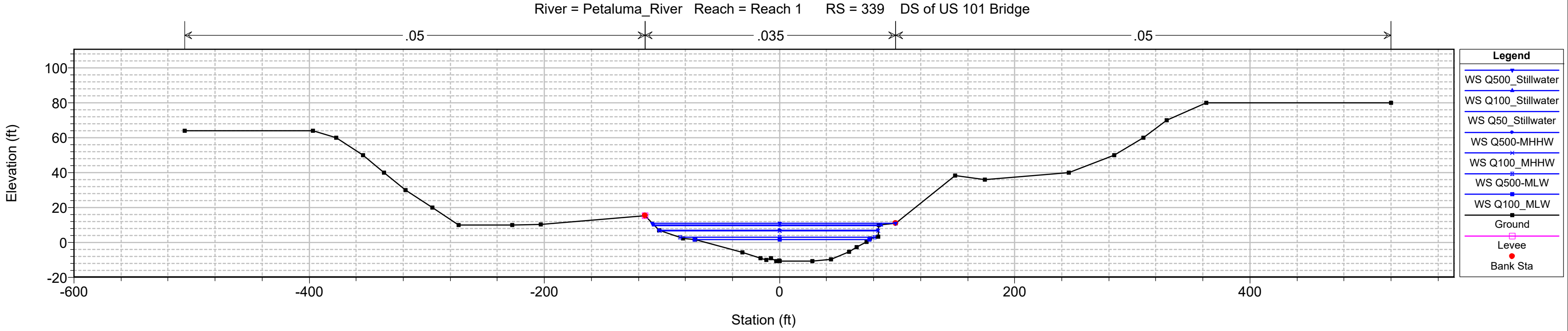


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

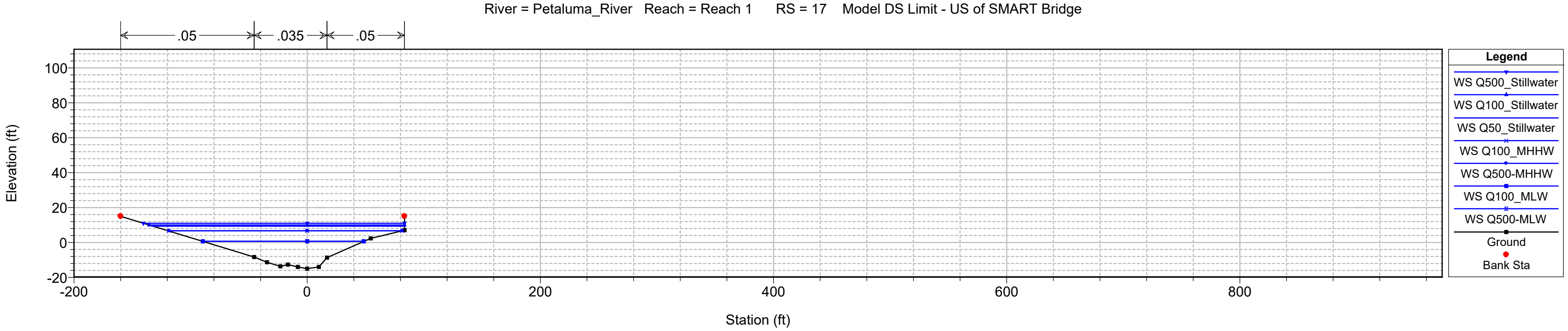
Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 500   BR



Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 339   DS of US 101 Bridge

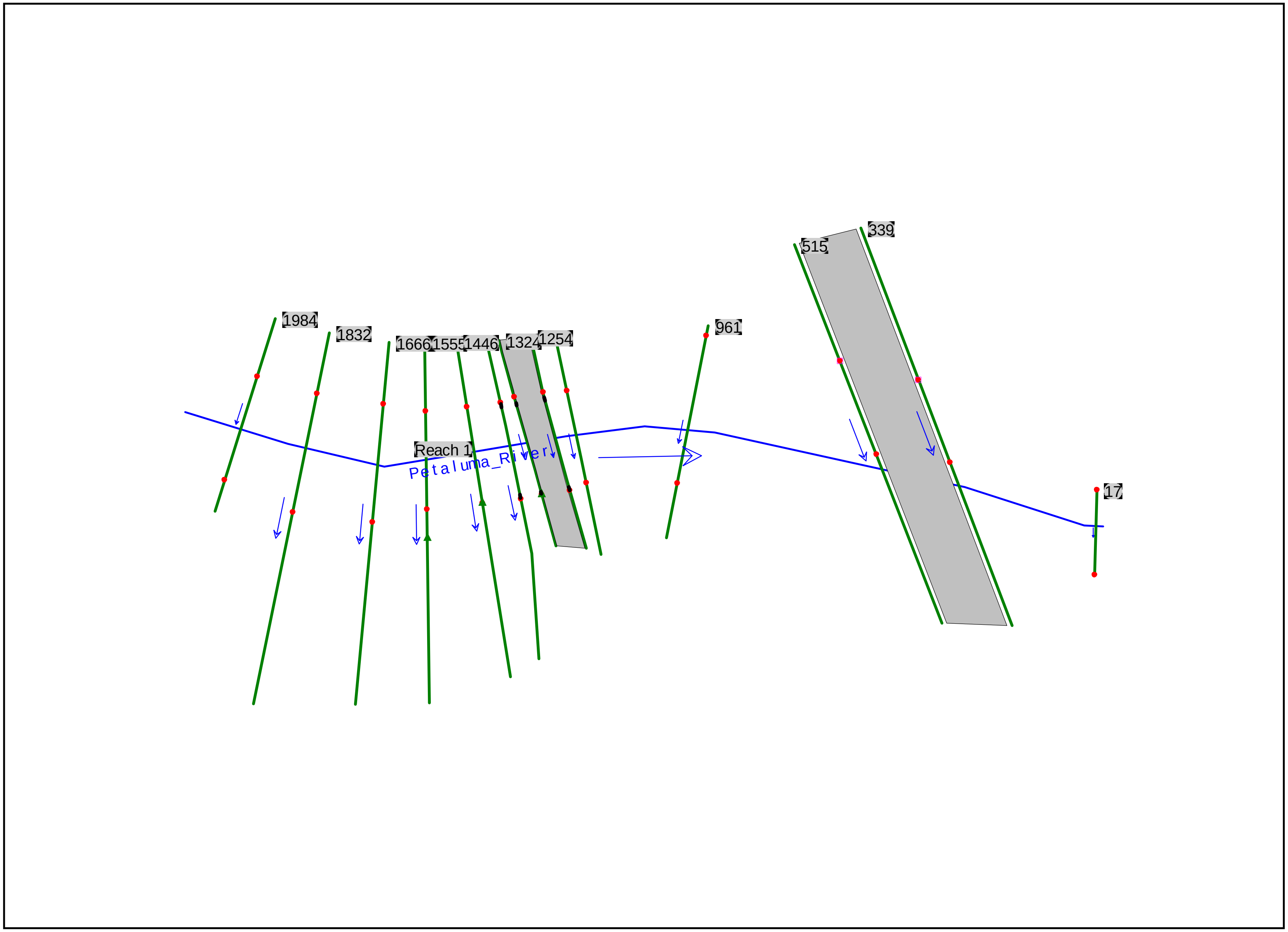


Petaluma_River_Proposed   Plan: Existing (Oct 2020)   4/6/2021
Geom: Existing (Oct 2020)   Flow: FEMA Flows
River = Petaluma_River   Reach = Reach 1   RS = 17   Model DS Limit - US of SMART Bridge

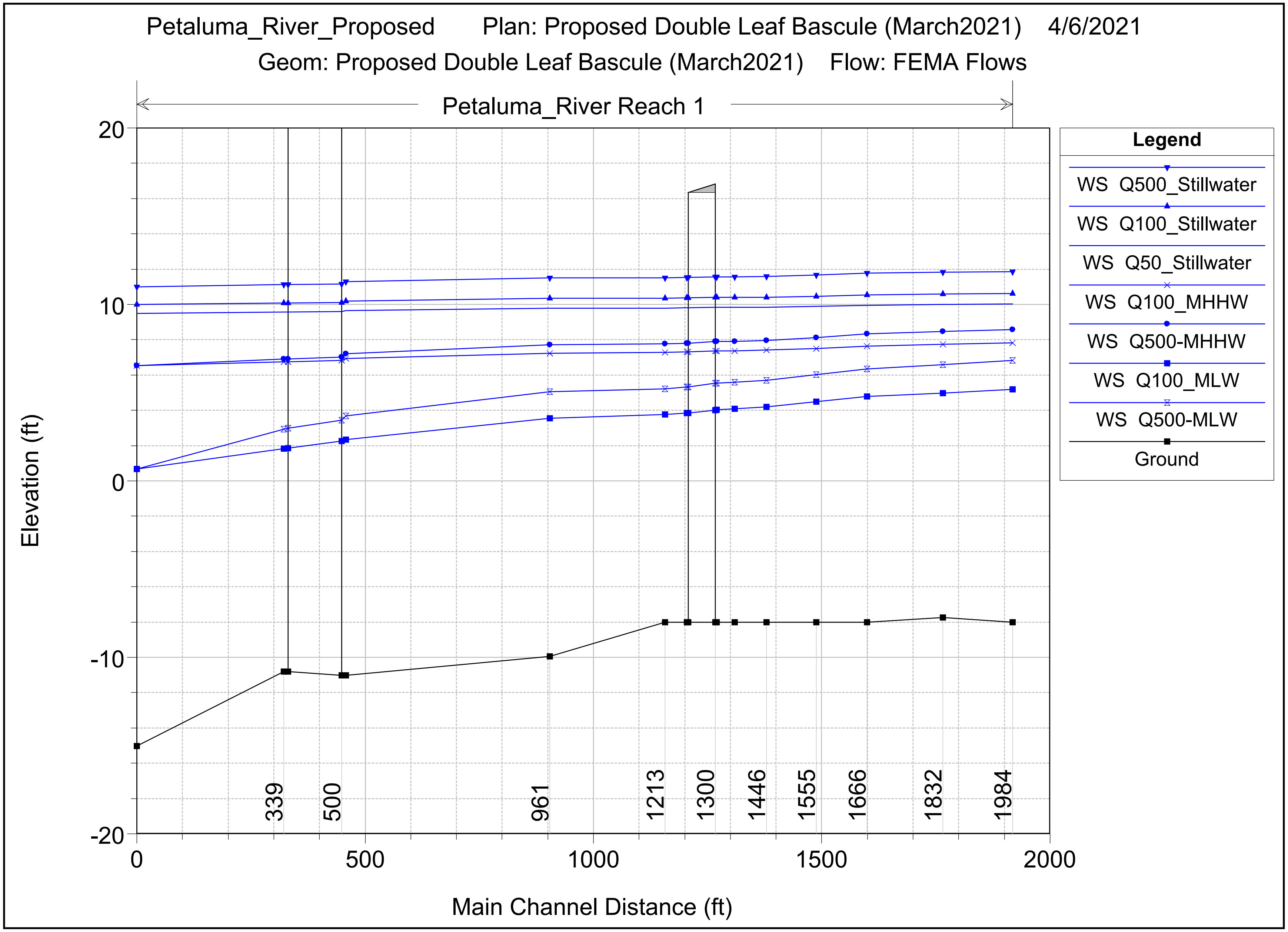


1 in Horiz. = 150 ft   1 in Vert. = 100 ft

Appendix C HEC-RAS Results: Proposed Condition



1984
1832
1666
1555
1446
1324
1254
961
515
339
17
Reach 1
Petaluma_River


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
Petaluma_River Reach 1
Elevation (ft)
Main Channel Distance (ft)
20
10
0
-10
-20
0
500
1000
1500
2000
339
500
961
1213
1300
1446
1555
1666
1832
1984
Legend
WS Q500_Stillwater
WS Q100_Stillwater
WS Q50_Stillwater
WS Q100_MHHW
WS Q500-MHHW
WS Q100_MLW
WS Q500-MLW
Ground

HEC-RAS Plan: Proposed Double Leaf Bascule (March2021) River: Petaluma_River Reach: Reach 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1984	Q50_Stillwater	9149.00	-8.00	10.02		10.21	0.000286	3.55	2575.76	229.29	0.19
Reach 1	1984	Q100_Stillwater	10494.00	-8.00	10.61		10.84	0.000321	3.87	2712.70	231.49	0.20
Reach 1	1984	Q500_Stillwater	13694.00	-8.00	11.86		12.18	0.000403	4.56	3005.88	242.56	0.23
Reach 1	1984	Q100_MHHW	10494.00	-8.00	7.82		8.21	0.000727	5.05	2079.32	221.19	0.29
Reach 1	1984	Q100_MLW	10494.00	-8.00	5.20		5.92	0.001521	6.84	1533.89	179.73	0.41
Reach 1	1984	Q500-MHHW	13694.00	-8.00	8.57		9.15	0.000977	6.09	2247.20	224.55	0.34
Reach 1	1984	Q500-MLW	13694.00	-8.00	6.82		7.66	0.001702	7.35	1862.30	213.29	0.44
Reach 1	1832	Q50_Stillwater	9149.00	-7.75	9.99		10.16	0.000256	3.33	2751.13	248.13	0.18
Reach 1	1832	Q100_Stillwater	10494.00	-7.75	10.58		10.79	0.000286	3.62	2898.77	250.14	0.19
Reach 1	1832	Q500_Stillwater	13694.00	-7.75	11.83		12.11	0.000355	4.26	3213.14	254.96	0.21
Reach 1	1832	Q100_MHHW	10494.00	-7.75	7.74		8.09	0.000674	4.77	2200.57	240.14	0.28
Reach 1	1832	Q100_MLW	10494.00	-7.75	4.98		5.67	0.001649	6.68	1570.71	202.92	0.42
Reach 1	1832	Q500-MHHW	13694.00	-7.75	8.47		8.98	0.000903	5.76	2376.75	242.87	0.32
Reach 1	1832	Q500-MLW	13694.00	-7.75	6.60		7.38	0.001725	7.10	1929.30	235.04	0.44
Reach 1	1666	Q50_Stillwater	9149.00	-8.00	9.95		10.12	0.000229	3.30	2776.29	234.47	0.17
Reach 1	1666	Q100_Stillwater	10494.00	-8.00	10.54		10.74	0.000261	3.60	2914.55	237.09	0.18
Reach 1	1666	Q500_Stillwater	13694.00	-8.00	11.77		12.05	0.000352	4.26	3215.01	272.03	0.21
Reach 1	1666	Q100_MHHW	10494.00	-8.00	7.64		7.98	0.000580	4.68	2244.20	225.90	0.26
Reach 1	1666	Q100_MLW	10494.00	-8.00	4.77		5.41	0.001339	6.39	1642.10	193.97	0.39
Reach 1	1666	Q500-MHHW	13694.00	-8.00	8.33		8.84	0.000803	5.70	2400.94	228.68	0.31
Reach 1	1666	Q500-MLW	13694.00	-8.00	6.35		7.11	0.001416	6.99	1959.38	210.85	0.40
Reach 1	1555	Q50_Stillwater	9149.00	-8.00	9.89		10.09	0.000262	3.59	2549.48	207.55	0.18
Reach 1	1555	Q100_Stillwater	10494.00	-8.00	10.47		10.71	0.000303	3.93	2671.28	273.05	0.19
Reach 1	1555	Q500_Stillwater	13694.00	-8.00	11.67		12.01	0.000384	4.68	3043.60	569.27	0.22
Reach 1	1555	Q100_MHHW	10494.00	-8.00	7.51		7.91	0.000636	5.07	2068.94	195.79	0.28
Reach 1	1555	Q100_MLW	10494.00	-8.00	4.48		5.23	0.001613	6.98	1503.60	178.09	0.42
Reach 1	1555	Q500-MHHW	13694.00	-8.00	8.13		8.73	0.000916	6.25	2190.52	198.91	0.33
Reach 1	1555	Q500-MLW	13694.00	-8.00	6.01		6.93	0.001678	7.69	1780.90	187.17	0.44
Reach 1	1446	Q50_Stillwater	9149.00	-8.00	9.85		10.06	0.000271	3.71	2469.10	196.40	0.18
Reach 1	1446	Q100_Stillwater	10494.00	-8.00	10.42		10.67	0.000317	4.07	2581.46	206.54	0.20
Reach 1	1446	Q500_Stillwater	13694.00	-8.00	11.60		11.96	0.000425	4.84	2963.26	580.73	0.23
Reach 1	1446	Q100_MHHW	10494.00	-8.00	7.41		7.84	0.000668	5.24	2002.35	187.17	0.28
Reach 1	1446	Q100_MLW	10494.00	-8.00	4.20		5.04	0.001819	7.34	1428.97	171.80	0.45
Reach 1	1446	Q500-MHHW	13694.00	-8.00	7.97		8.63	0.000973	6.50	2107.59	188.97	0.34
Reach 1	1446	Q500-MLW	13694.00	-8.00	5.71		6.73	0.001848	8.10	1690.88	176.87	0.46

HEC-RAS Plan: Proposed Double Leaf Bascule (March2021) River: Petaluma_River Reach: Reach 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1376	Q50_Stillwater	9149.00	-8.00	9.83		10.04	0.000268	3.70	2474.22	194.64	0.18
Reach 1	1376	Q100_Stillwater	10494.00	-8.00	10.39		10.65	0.000312	4.06	2584.57	197.74	0.20
Reach 1	1376	Q500_Stillwater	13694.00	-8.00	11.57		11.93	0.000416	4.85	2886.17	414.15	0.23
Reach 1	1376	Q100_MHHW	10494.00	-8.00	7.37		7.79	0.000635	5.22	2010.54	181.30	0.28
Reach 1	1376	Q100_MLW	10494.00	-8.00	4.09		4.91	0.001681	7.25	1446.59	166.43	0.43
Reach 1	1376	Q500-MHHW	13694.00	-8.00	7.90		8.56	0.000945	6.49	2108.67	184.51	0.34
Reach 1	1376	Q500-MLW	13694.00	-8.00	5.59		6.60	0.001741	8.07	1697.79	170.11	0.45
Reach 1	1324	Q50_Stillwater	9149.00	-8.00	9.83	-0.63	10.03	0.000259	3.57	2559.33	202.31	0.18
Reach 1	1324	Q100_Stillwater	10494.00	-8.00	10.39	-0.08	10.63	0.000299	3.93	2673.46	202.91	0.19
Reach 1	1324	Q500_Stillwater	13694.00	-8.00	11.57	1.16	11.91	0.000400	4.70	2935.36	245.13	0.22
Reach 1	1324	Q100_MHHW	10494.00	-8.00	7.36	-0.08	7.76	0.000625	5.06	2072.60	193.45	0.27
Reach 1	1324	Q100_MLW	10494.00	-8.00	4.02	-0.08	4.84	0.001857	7.27	1444.07	179.57	0.45
Reach 1	1324	Q500-MHHW	13694.00	-8.00	7.89	1.15	8.51	0.000914	6.29	2176.23	194.33	0.33
Reach 1	1324	Q500-MLW	13694.00	-8.00	5.53	1.15	6.51	0.001888	7.95	1722.05	189.02	0.46
Reach 1	1300 BR U	Q50_Stillwater	9149.00	-8.00	9.83	-0.62	10.03	0.000252	3.58	2556.79	200.00	0.18
Reach 1	1300 BR U	Q100_Stillwater	10494.00	-8.00	10.39	-0.05	10.63	0.000289	3.93	2669.35	200.00	0.19
Reach 1	1300 BR U	Q500_Stillwater	13694.00	-8.00	11.56	1.16	11.91	0.000377	4.72	2903.08	200.00	0.22
Reach 1	1300 BR U	Q100_MHHW	10494.00	-8.00	7.36	-0.05	7.76	0.000626	5.06	2072.26	193.45	0.27
Reach 1	1300 BR U	Q100_MLW	10494.00	-8.00	4.01	-0.05	4.84	0.001861	7.27	1443.01	179.54	0.45
Reach 1	1300 BR U	Q500-MHHW	13694.00	-8.00	7.89	1.16	8.51	0.000915	6.29	2175.72	194.33	0.33
Reach 1	1300 BR U	Q500-MLW	13694.00	-8.00	5.53	1.16	6.51	0.001892	7.96	1720.90	188.95	0.46
Reach 1	1300 BR D	Q50_Stillwater	9149.00	-8.00	9.81	-0.52	10.01	0.000255	3.62	2524.22	196.04	0.18
Reach 1	1300 BR D	Q100_Stillwater	10494.00	-8.00	10.37	0.02	10.61	0.000297	3.98	2634.34	198.64	0.19
Reach 1	1300 BR D	Q500_Stillwater	13694.00	-8.00	11.53	1.21	11.88	0.000390	4.78	2866.19	200.00	0.22
Reach 1	1300 BR D	Q100_MHHW	10494.00	-8.00	7.30	0.02	7.72	0.000643	5.14	2040.96	190.56	0.28
Reach 1	1300 BR D	Q100_MLW	10494.00	-8.00	3.85	0.02	4.71	0.001895	7.44	1409.80	171.88	0.46
Reach 1	1300 BR D	Q500-MHHW	13694.00	-8.00	7.81	1.21	8.45	0.000949	6.41	2137.20	191.52	0.34
Reach 1	1300 BR D	Q500-MLW	13694.00	-8.00	5.33	1.21	6.38	0.001990	8.20	1670.21	182.37	0.48
Reach 1	1254	Q50_Stillwater	9149.00	-8.00	9.81		10.01	0.000255	3.62	2524.04	196.03	0.18
Reach 1	1254	Q100_Stillwater	10494.00	-8.00	10.37		10.61	0.000297	3.98	2634.13	198.63	0.19
Reach 1	1254	Q500_Stillwater	13694.00	-8.00	11.52		11.88	0.000393	4.78	2866.07	200.63	0.22
Reach 1	1254	Q100_MHHW	10494.00	-8.00	7.30		7.71	0.000643	5.14	2040.49	190.56	0.28
Reach 1	1254	Q100_MLW	10494.00	-8.00	3.84		4.71	0.001901	7.45	1408.34	171.84	0.46
Reach 1	1254	Q500-MHHW	13694.00	-8.00	7.80		8.44	0.000950	6.41	2136.47	191.51	0.34
Reach 1	1254	Q500-MLW	13694.00	-8.00	5.32		6.37	0.001994	8.21	1668.57	182.17	0.48

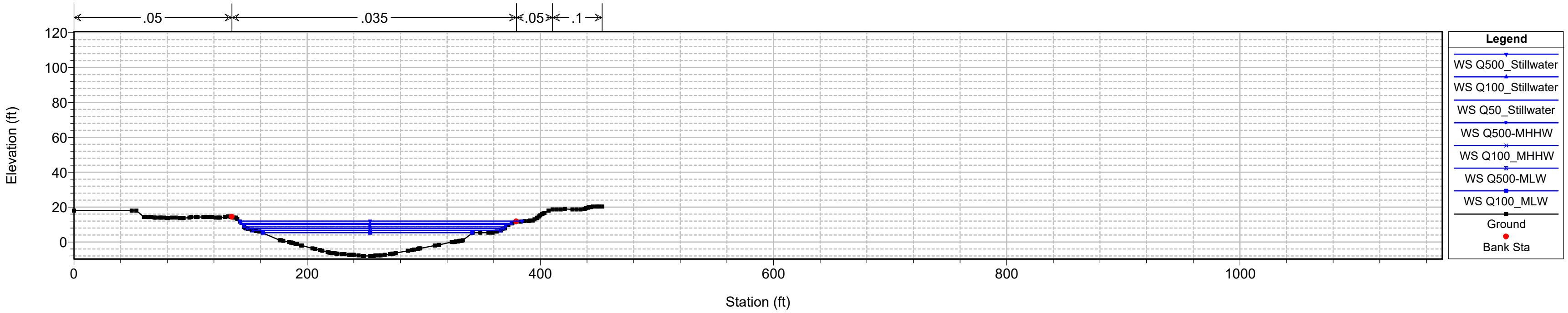
HEC-RAS Plan: Proposed Double Leaf Bascule (March2021) River: Petaluma_River Reach: Reach 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1213	Q50_Stillwater	9149.00	-8.00	9.80		10.00	0.000251	3.60	2540.93	199.16	0.18
Reach 1	1213	Q100_Stillwater	10494.00	-8.00	10.35		10.60	0.000290	3.96	2652.36	200.93	0.19
Reach 1	1213	Q500_Stillwater	13694.00	-8.00	11.51		11.86	0.000383	4.74	2887.44	212.47	0.22
Reach 1	1213	Q100_MHHW	10494.00	-8.00	7.27		7.68	0.000638	5.12	2047.68	191.42	0.28
Reach 1	1213	Q100_MLW	10494.00	-8.00	3.75		4.62	0.001912	7.45	1408.10	172.03	0.46
Reach 1	1213	Q500-MHHW	13694.00	-8.00	7.76		8.40	0.000949	6.39	2141.50	193.17	0.34
Reach 1	1213	Q500-MLW	13694.00	-8.00	5.23		6.28	0.001964	8.22	1665.46	178.49	0.47
Reach 1	961	Q50_Stillwater	9149.00	-9.94	9.79		9.93	0.000160	2.95	3102.19	235.34	0.14
Reach 1	961	Q100_Stillwater	10494.00	-9.94	10.35		10.51	0.000185	3.24	3234.21	237.35	0.15
Reach 1	961	Q500_Stillwater	13694.00	-9.94	11.51		11.75	0.000247	3.90	3512.47	242.26	0.18
Reach 1	961	Q100_MHHW	10494.00	-9.94	7.24		7.51	0.000400	4.17	2514.02	226.19	0.22
Reach 1	961	Q100_MLW	10494.00	-9.94	3.55		4.14	0.001303	6.16	1704.54	209.27	0.38
Reach 1	961	Q500-MHHW	13694.00	-9.94	7.72		8.15	0.000598	5.22	2622.54	227.90	0.27
Reach 1	961	Q500-MLW	13694.00	-9.94	5.07		5.77	0.001316	6.75	2029.85	218.36	0.39
Reach 1	515	Q50_Stillwater	9149.00	-11.02	9.66	-2.31	9.84	0.000212	3.41	2681.22	198.80	0.16
Reach 1	515	Q100_Stillwater	10494.00	-11.02	10.19	-1.67	10.41	0.000247	3.76	2787.58	200.08	0.18
Reach 1	515	Q500_Stillwater	13694.00	-11.02	11.29	-0.30	11.61	0.000353	4.55	3008.34	212.11	0.21
Reach 1	515	Q100_MHHW	10494.00	-11.02	6.92	-1.67	7.30	0.000552	4.89	2146.32	192.24	0.26
Reach 1	515	Q100_MLW	10494.00	-11.02	2.33	-1.67	3.34	0.002287	8.05	1303.17	161.98	0.50
Reach 1	515	Q500-MHHW	13694.00	-11.02	7.21	-0.30	7.81	0.000870	6.22	2200.64	192.92	0.32
Reach 1	515	Q500-MLW	13694.00	-11.02	3.68	-0.30	4.91	0.002625	8.92	1535.55	181.63	0.54
Reach 1	500 BR U	Q50_Stillwater	9149.00	-11.02	9.60	-2.31	9.82	0.000329	3.82	2394.10	159.31	0.17
Reach 1	500 BR U	Q100_Stillwater	10494.00	-11.02	10.11	-1.67	10.39	0.000394	4.24	2476.51	160.03	0.19
Reach 1	500 BR U	Q500_Stillwater	13694.00	-11.02	11.16	-0.30	11.58	0.000598	5.18	2644.43	149.48	0.22
Reach 1	500 BR U	Q100_MHHW	10494.00	-11.02	6.82	-1.67	7.27	0.000766	5.36	1956.95	155.45	0.27
Reach 1	500 BR U	Q100_MLW	10494.00	-11.02	2.26	-1.67	3.31	0.002440	8.19	1281.13	137.09	0.47
Reach 1	500 BR U	Q500-MHHW	13694.00	-11.02	7.02	-0.30	7.76	0.001247	6.89	1988.09	155.73	0.34
Reach 1	500 BR U	Q500-MLW	13694.00	-11.02	3.44	-0.30	4.84	0.003009	9.47	1445.74	143.64	0.53
Reach 1	500 BR D	Q50_Stillwater	9149.00	-10.80	9.58	-2.36	9.78	0.000252	3.62	2526.89	178.48	0.17
Reach 1	500 BR D	Q100_Stillwater	10494.00	-10.80	10.09	-1.72	10.34	0.000300	4.01	2618.67	179.55	0.18
Reach 1	500 BR D	Q500_Stillwater	13694.00	-10.80	11.12	-0.36	11.49	0.000425	4.88	2805.41	183.36	0.22
Reach 1	500 BR D	Q100_MHHW	10494.00	-10.80	6.75	-1.72	7.17	0.000628	5.17	2029.16	174.11	0.27
Reach 1	500 BR D	Q100_MLW	10494.00	-10.80	1.86	-1.72	2.99	0.002525	8.54	1228.98	149.33	0.52
Reach 1	500 BR D	Q500-MHHW	13694.00	-10.80	6.91	-0.36	7.60	0.001030	6.66	2056.57	174.61	0.34
Reach 1	500 BR D	Q500-MLW	13694.00	-10.80	2.98	-0.36	4.46	0.003018	9.76	1403.31	158.77	0.58

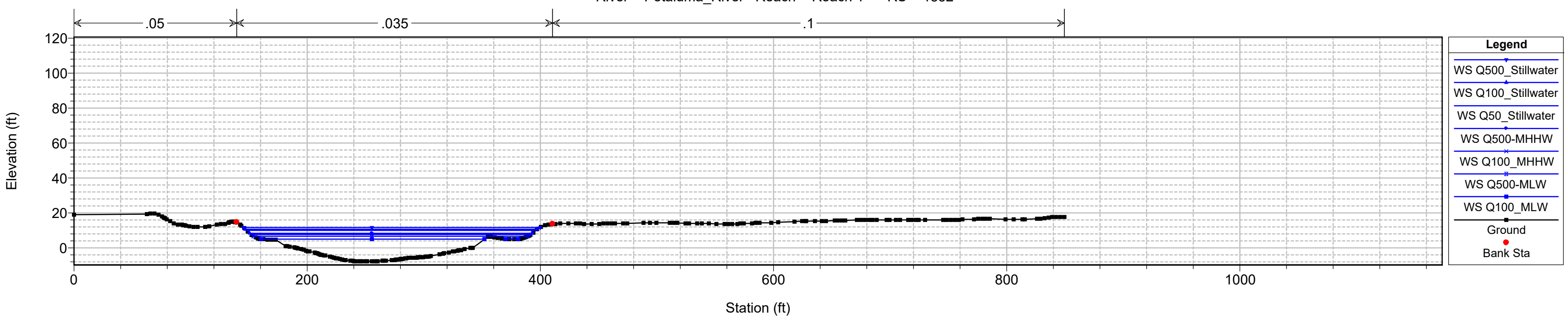
HEC-RAS Plan: Proposed Double Leaf Bascule (March2021) River: Petaluma_River Reach: Reach 1 (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	339	Q50_Stillwater	9149.00	-10.80	9.58	-2.36	9.77	0.000225	3.51	2608.42	190.48	0.17
Reach 1	339	Q100_Stillwater	10494.00	-10.80	10.09	-1.72	10.33	0.000268	3.88	2706.74	193.45	0.18
Reach 1	339	Q500_Stillwater	13694.00	-10.80	11.13	-0.36	11.48	0.000390	4.70	2915.06	207.47	0.22
Reach 1	339	Q100_MHHW	10494.00	-10.80	6.76	-1.72	7.15	0.000601	5.05	2076.74	186.13	0.27
Reach 1	339	Q100_MLW	10494.00	-10.80	1.82	-1.72	2.96	0.002547	8.58	1223.76	148.70	0.53
Reach 1	339	Q500-MHHW	13694.00	-10.80	6.92	-0.36	7.57	0.000981	6.50	2106.55	186.62	0.34
Reach 1	339	Q500-MLW	13694.00	-10.80	2.94	-0.36	4.43	0.003186	9.77	1401.06	165.67	0.59
Reach 1	17	Q50_Stillwater	9149.00	-15.03	9.50	-3.81	9.67	0.000364	3.29	2776.96	216.51	0.16
Reach 1	17	Q100_Stillwater	10494.00	-15.03	10.00	-3.09	10.21	0.000430	3.64	2885.83	218.98	0.18
Reach 1	17	Q500_Stillwater	13694.00	-15.03	11.00	-1.62	11.30	0.000595	4.41	3107.28	223.93	0.21
Reach 1	17	Q100_MHHW	10494.00	-15.03	6.53	-3.10	6.90	0.000970	4.87	2156.06	199.77	0.26
Reach 1	17	Q100_MLW	10494.00	-15.03	0.67	-3.10	1.91	0.004116	8.92	1176.16	138.35	0.54
Reach 1	17	Q500-MHHW	13694.00	-15.03	6.53	-1.62	7.16	0.001651	6.35	2156.06	199.77	0.34
Reach 1	17	Q500-MLW	13694.00	-15.03	0.67	-1.62	2.78	0.007009	11.64	1176.16	138.35	0.70

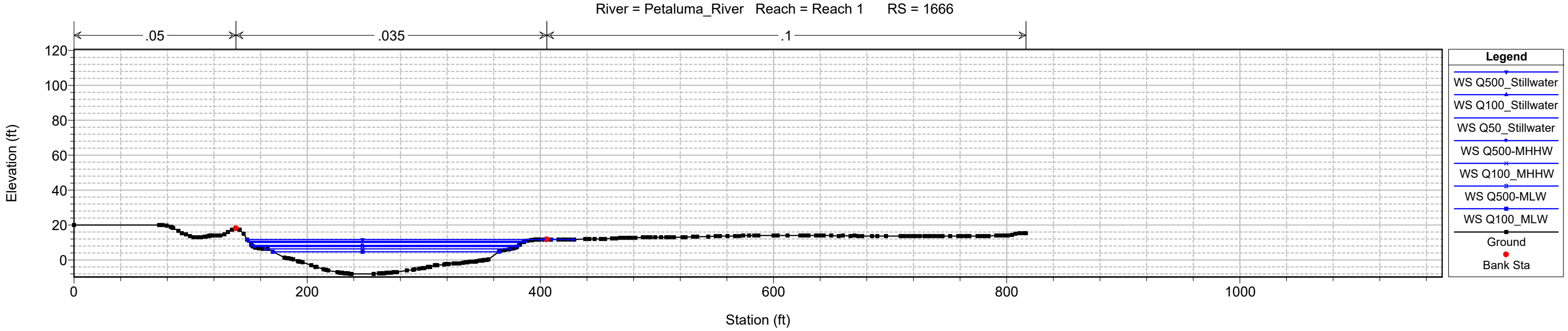
Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1984 At "Guadalupe Ct"



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1832

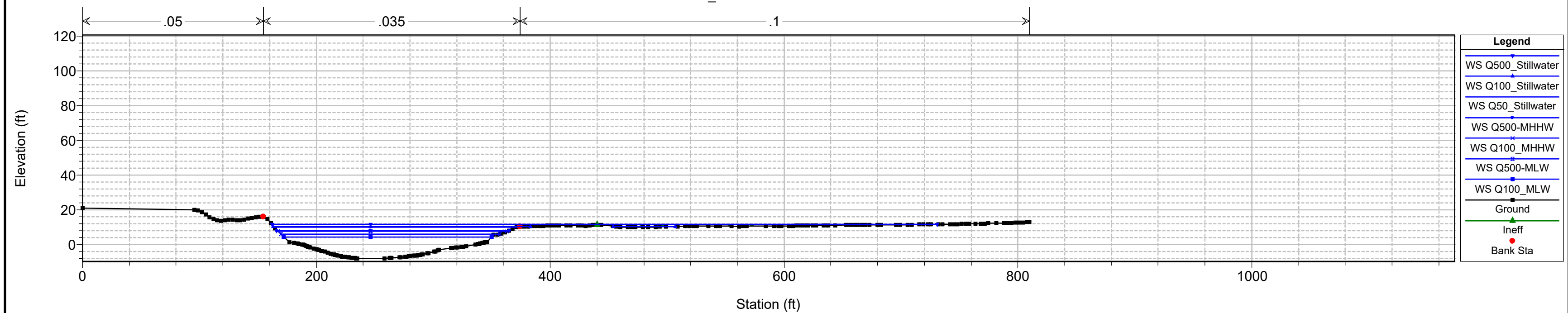


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1666

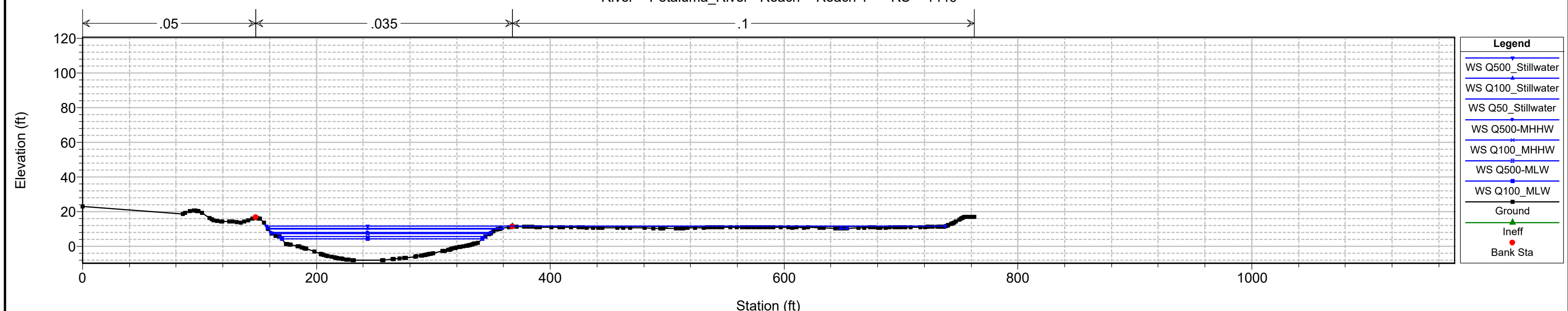


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

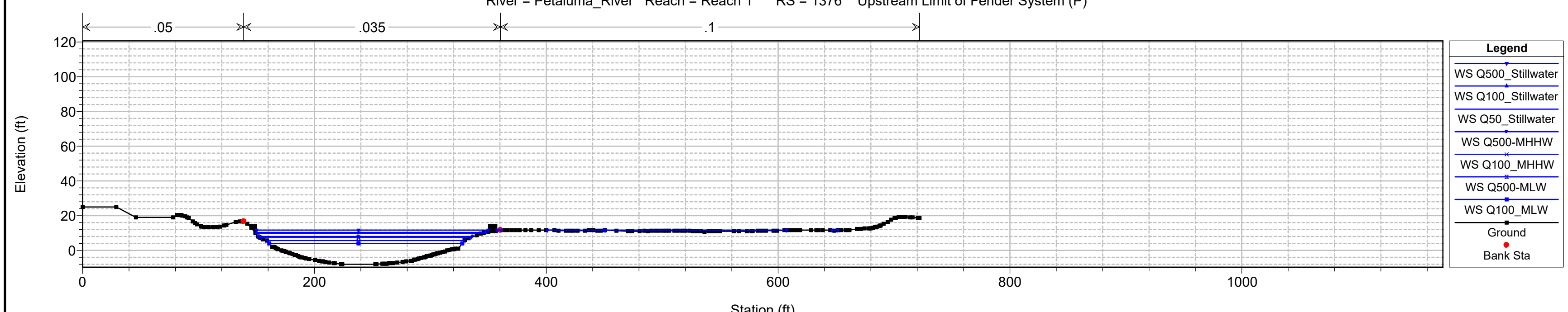
Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1555



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1446

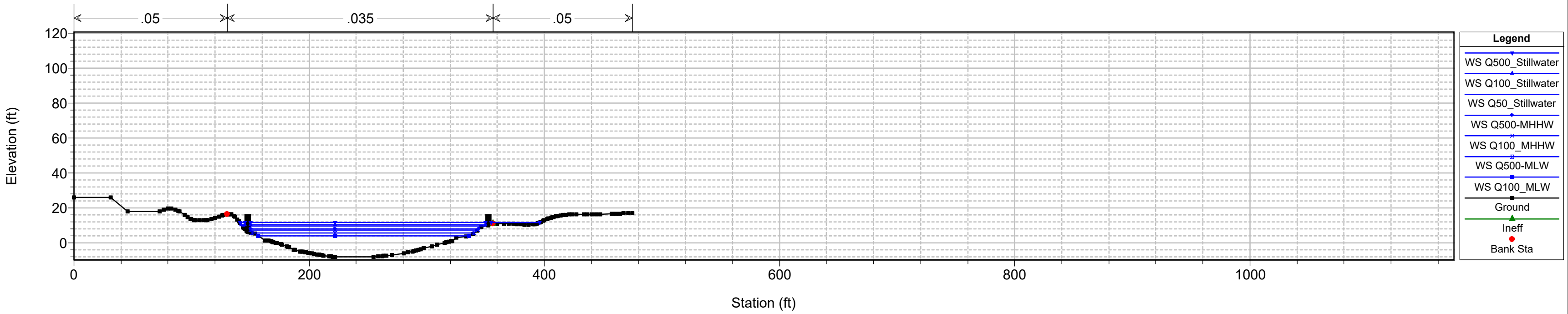


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1376 Upstream Limit of Fender System (P)

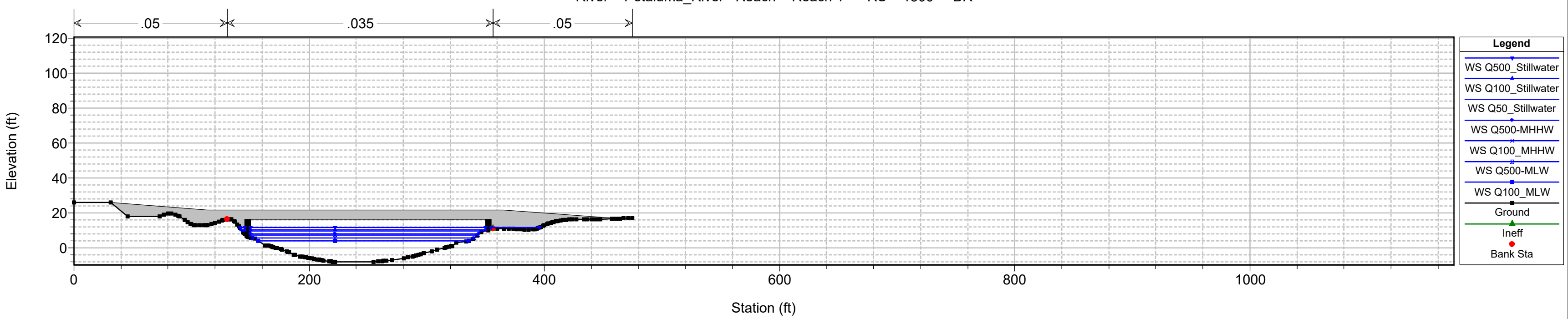


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

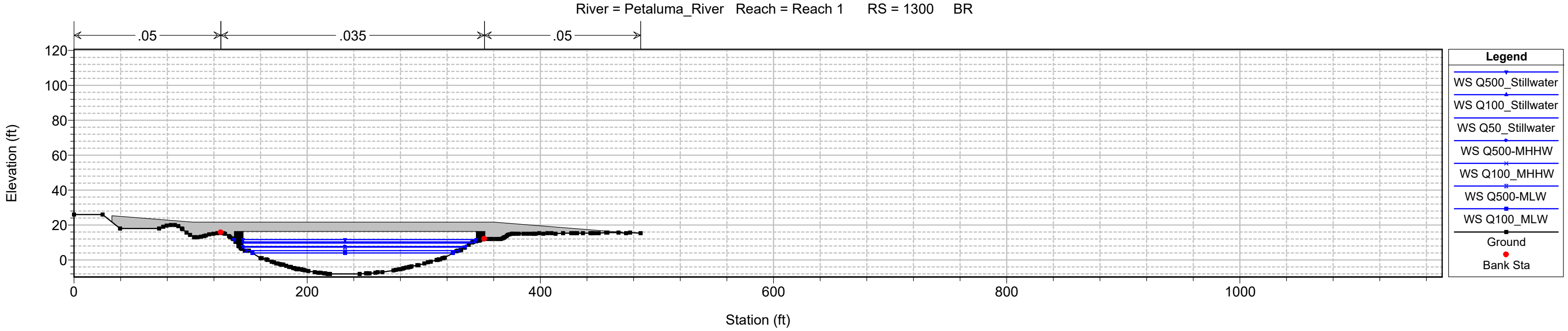
Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1324 Upstream of proposed bridge



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1300 BR

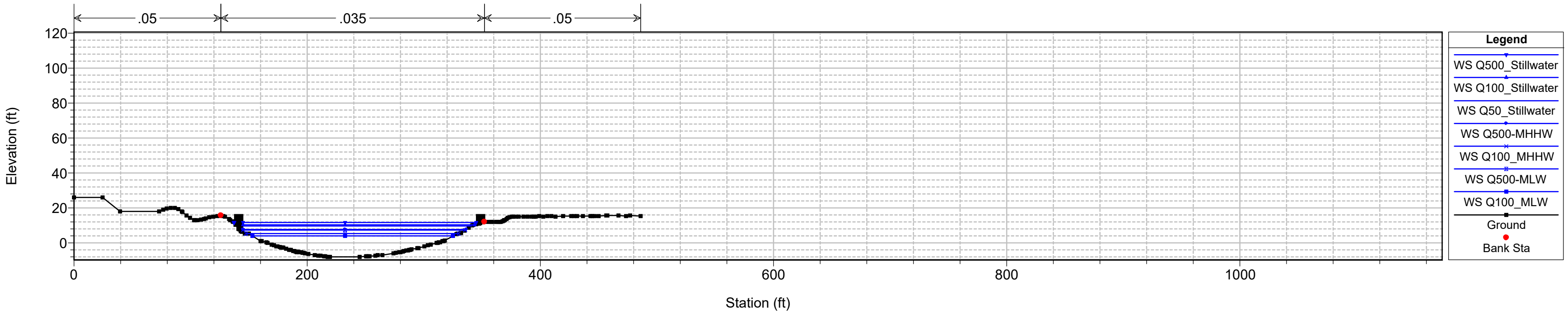


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1300 BR

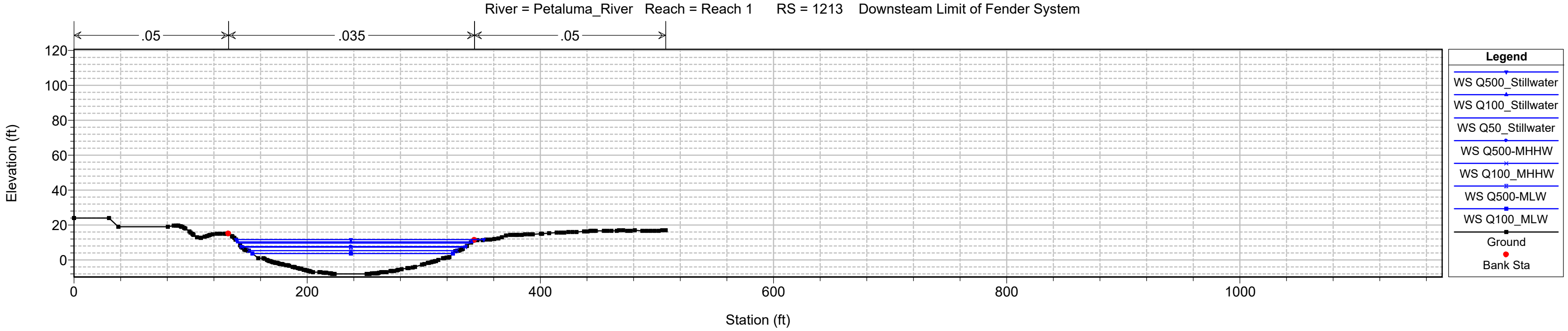


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

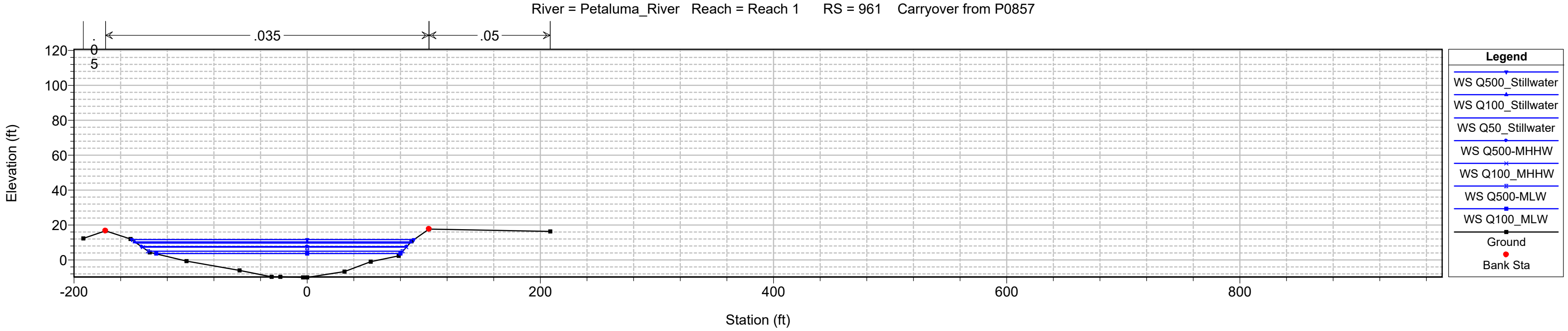
Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1254 Downstream of Proposed Bridge



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 1213 Downsteam Limit of Fender System

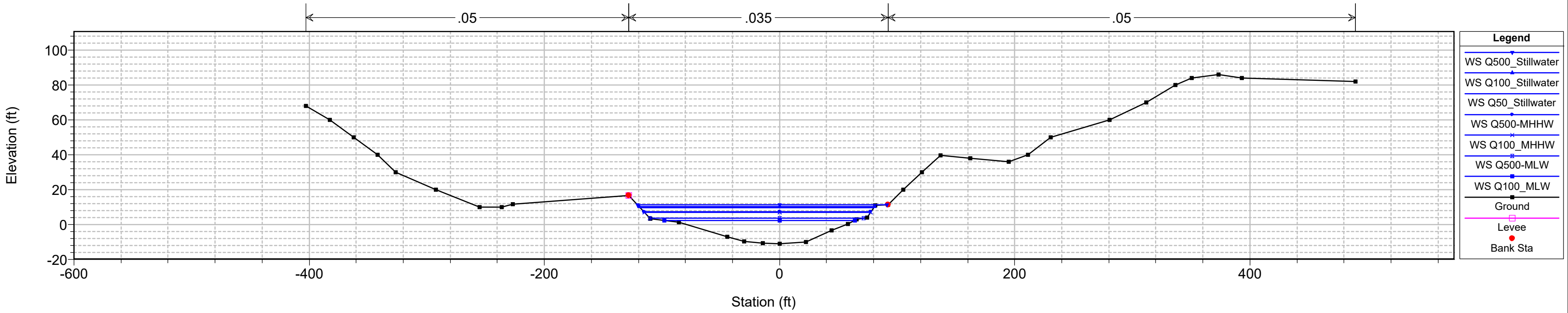


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 961 Carryover from P0857

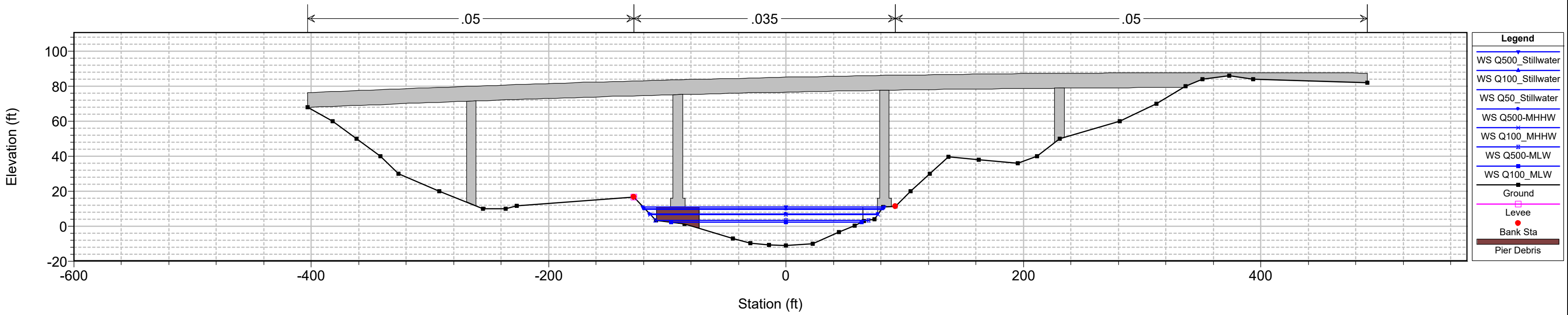


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

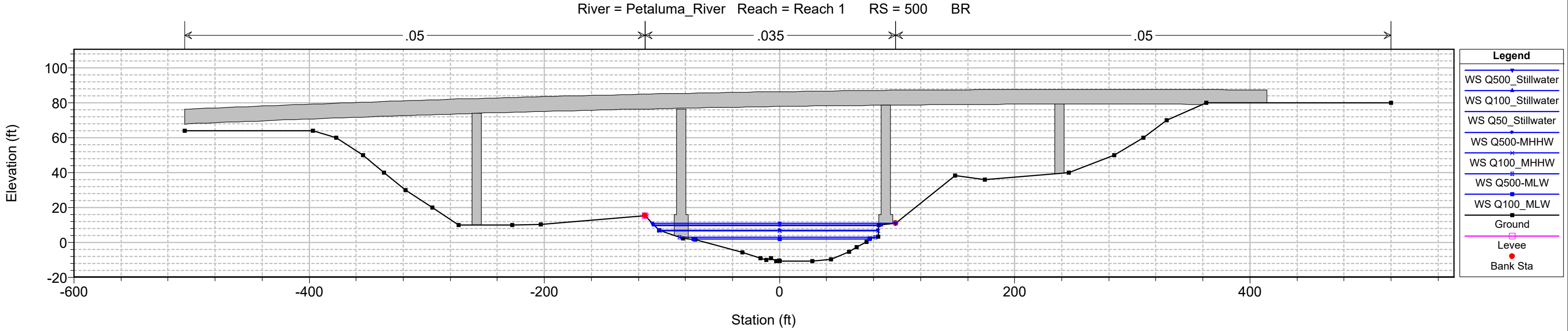
Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 515 US of US 101 Bridge



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 500 BR

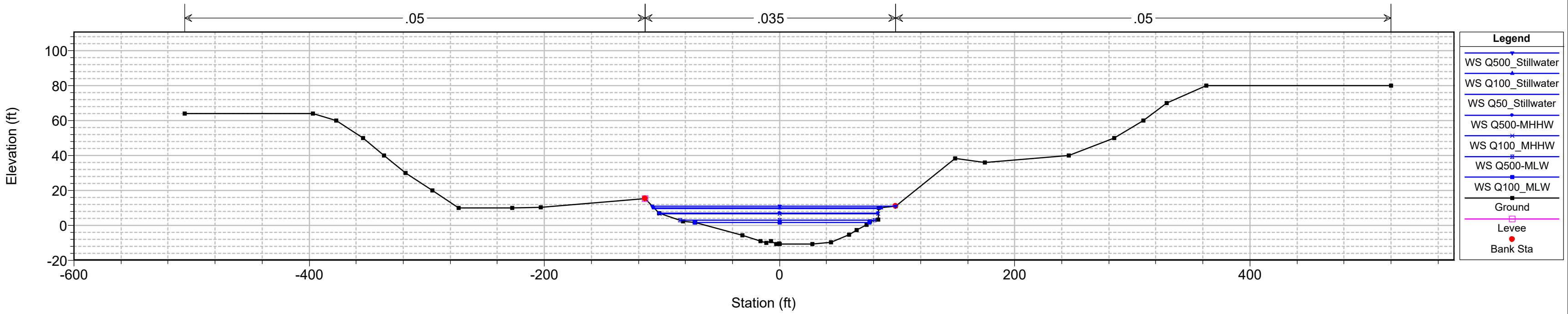


Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 500 BR

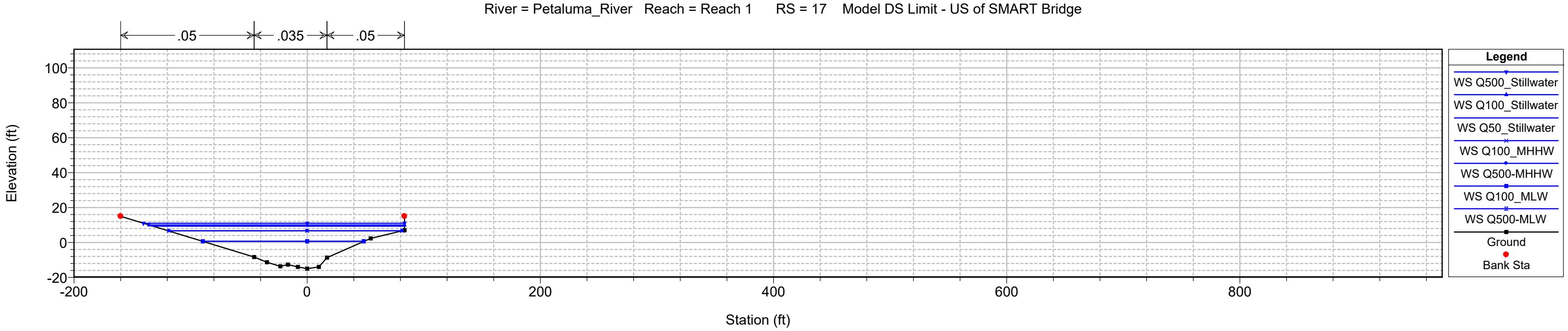


1 in Horiz. = 150 ft 1 in Vert. = 100 ft

Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 339 DS of US 101 Bridge



Petaluma_River_Proposed Plan: Proposed Double Leaf Bascule (March2021) 4/6/2021
Geom: Proposed Double Leaf Bascule (March2021) Flow: FEMA Flows
River = Petaluma_River Reach = Reach 1 RS = 17 Model DS Limit - US of SMART Bridge



1 in Horiz. = 150 ft 1 in Vert. = 100 ft

Appendix D Soil Particle Distribution Curve