TRAFFIC IMPACT ASSESSMENT REPORT

Traffic Impact Analysis Report
‘O‘oma Beachside Village

Kaloko, North Kona, Island of Hawai‘i, Hawai‘i

Tax Map Key Number *(3)7-3-009: 004 & 022*

MAY 2008

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‘O‘OMA BEACHSIDE VILLAGE

‘O‘oma, North Kona, Hawai‘i

Traffic Impact Analysis Report

TMK: (3)7-3-9: 004 and 022

May 2008

Expiration Date:
April 30, 2010

This Traffic Impact Analysis Report has been conducted and prepared by the undersigned professional engineer licensed in the State of Hawai‘i in accordance with the best practices of the industry.

Signature
M & E Pacific, Inc.

METCALF & EDDY | AECOM

May 7, 2008
Date

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TRAFFIC IMPACT ANALYSIS REPORT
for the
‘O‘OMA BEACHSIDE VILLAGE

‘O‘oma Beachside Village, a 302.38-acre residential and commercial mixed use community, is being planned at ‘O‘oma, North Kona, Hawai‘i. This report documents a study that was conducted to identify the traffic impacts of the proposed community and to recommend any mitigating measures.

PROJECT DESCRIPTION

‘O‘oma Beachside Village LLC intends to develop a 302.38-acre property (the Property) at ‘O‘oma, North Kona, Hawai‘i. The Property is comprised of a:

- 217.566-acre parcel identified by TMK (3)7-3-009:004 (Parcel 4);
- 83-acre parcel identified by TMK (3)7-3-009:022 (Parcel 22); and
- 1.814-acre portion of the State-owned Right-of-Way (ROW) located on by TMK (3)7-3-009: (State ROW).

The Property is on the *makai* side of Queen Ka‘ahumanu Highway about two miles south of the Kona International Airport at Keahole. Other major cross streets in the vicinity include Ka‘iminani Drive and the entrance to the Natural Energy Laboratory of Hawai‘i Authority (NELHA) to the north, and Huliko‘a Drive (the entrance to the Kohanaiki Business Park) and Hina Lani Street to the south. The Property’s location relative to these other roadway facilities is shown on **Figure 1**.

‘O‘oma Beachside Village is planned to include the following:

- Approximately 950 to 1,200 homes, including:
    - Single family units,
    - Multi-family units, and
    - “Live-work” units with commercial uses on the ground floor and residential uses above.
- Approximately 200,000 square feet of commercial space, including:
    - Space for a small grocery store,
    - Restaurants, and
    - Retail and office space.
- A private or charter school site.
- A public beach park, including a community pavilion.

Construction of ‘O‘oma Beachside Village is expected to begin in 2011 (with first occupancy projected in 2012) and will continue through approximately 2029. For the purpose of this analysis ‘O‘oma Beachside Village is roughly divided into three areas: Area A, Area B, and Area C, as shown on **Figure 2**. The development of each area could overlap into other areas at any one time. For the purpose of this analysis, the projection is to deliver about 20-40 single family residential units, 30-50 multi-family residential units, and 10-25,000 square feet of commercial space per year.

The study analyzed three forecast years to comply with the Concurrency Conditions of County of Hawai‘i Ordinance 07-99 which requires analyses for 5, 10, and 20 year forecasts. This study analyzed years 2015, 2020, and 2029 corresponding to 7, 12, and 21 year forecasts. The number of project components which were assumed to be occupied by each analysis year for purposes of conducting the traffic impact analysis is summarized on **Table 1**. The actual development schedule for the ‘O‘oma Beachside Village could deviate from the schedule shown on **Table 1**.

The State of Hawai‘i Department of Transportation (HDOT) is currently preparing for the second phase of widening of the Queen Ka‘ahumanu Highway to four lanes from Honokohau Harbor to the Kona International Airport at Keahole, with completion of construction currently scheduled for 2011. HDOT intends to restrict access to the widened highway and permit fully accessible signalized intersections only at Kealakehe Parkway (the harbor access road), Hina Lani Street, Huliko‘a Drive (Kohanaiki), Ka‘iminani Drive, and Keahole Airport Road. The developments on the *makai* side of the highway may be permitted right turn in, right turn out movements onto the highway. For this study, it was assumed that ‘O‘oma Beachside Village would have such an access.

‘O‘oma Beachside Village would also be serviced by a frontage road that would have connections to fully accessible signalized intersections. This frontage road would extend from Huliko‘a Drive at Kohanaiki Industrial subdivision (crossing Queen Ka‘ahumanu Highway into the Shores of Kohanaiki and resulting in a full, four-way intersection) to the Keahole Airport Road, and would allow vehicles from connecting makai projects direct access to the airport without having to enter the highway. The frontage road alignment has not been determined but it is not expected to be a high speed design roadway. Within ‘O‘oma Beachside Village there would be urban land uses and several intersecting streets along the roadway as traffic calming measures. ‘O‘oma Beachside Village would also be served by a transit stop.

EXISTING CONDITIONS

A survey of the existing roadway and traffic conditions was made in September 2006.

Existing Roadways

The main roadways currently in the study area include Queen Ka‘ahumanu Highway, Ka‘iminani Drive, the NELHA access road, Huliko‘a Drive, and Hina Lani Street.

Queen Ka‘ahumanu Highway is the primary arterial highway on the west side of the island of Hawai‘i. The highway passes through the North Kona and South Kohala districts and connects Kailua Village with the Kona International Airport, the Kohala resort areas, and Kawaihae. It is a two-lane Class I State Highway with limited access and a design speed of 70 miles per hour. Intersections on this highway are fully channelized and signalized where warranted, including the Ka‘iminani Drive and Hina Lani Street intersections.

Ka‘iminani Drive is a collector road within a 60-foot right-of-way that provides *mauka-makai* access between Queen Ka‘ahumanu Highway and Mamalahoa Highway and provides access to the Kona Palisades subdivision.

The NELHA access road and Huliko‘a Drive provide access to two separate industrial parks and their intersections with the highway are channelized but not signalized.

Hina Lani Street is a two-lane County secondary arterial road within an 80-foot right-of-way. It provides *mauka-makai* access between Queen Ka‘ahumanu Highway and Mamalahoa Highway and serves the Kaloko Light Industrial Subdivision at its *makai* end.

Traffic Volumes

Traffic turning movement counts were taken at the Hina Lani Street and Ka‘iminani Drive intersections on Queen Ka‘ahumanu Highway during the morning and afternoon peak periods on September 12 and 14, 2006. Traffic turning movement counts require a traffic surveyor to observe traffic flow and record the movements of each vehicle crossing the intersection as through or turning movements by 15 minute intervals. The worksheets from these traffic counts are included in **Appendix A**.

The resultant morning and afternoon peak hour traffic volumes are shown on **Figure 3**, with volumes for two consecutive morning and afternoon peak hours shown. The volumes are rounded to the nearest five vehicles per hour (vph). The northbound direction of traffic on Queen Ka‘ahumanu Highway south of Hina Lani Street is higher in the first hour of the morning peak, then about equal to the southbound flow in the second hour. The northbound volumes north of Ka‘iminani Drive are higher for both peak hours. This reflects the commute of workers from Kona to the Kohala resort area in the early morning, followed by the commute of workers to Kailua later in the morning. During the afternoon peak, the southbound volumes south of Hina Lani Street are about equal to the northbound volumes in the first hour while the northbound volumes are much higher in the second hour. The southbound volumes north of Ka‘iminani Drive are higher during both afternoon peak hours. Long traffic queues in the southbound lane were observed for short periods in the early afternoon period due to backup of traffic from Kailua Village. The existing traffic operations at the study intersections are discussed in the **Level of Service Analysis** section of this report.

The HDOT took metered traffic counts at selected locations on Hawai‘i Island roadways in even numbered years. Station T-8-M is located on Queen Ka‘ahumanu Highway 850 feet north of the NELHA access roadway. HDOT has converted this station to a telemetry station that provides continuous traffic data. The data from the previous counts and the average weekday daily traffic volumes for 2006 provides the historic trend in daily traffic volumes on the highway over a 14 year period ending in 2006. The biannual change in two way daily traffic volumes on Queen Ka‘ahumanu Highway is shown in tabular and graph form on **Figure 4**. Queen Ka‘ahumanu Highway shows a 94% increase in traffic volumes over the 14 year period, which corresponds to a 4.8% compounded annual growth rate.

The pattern of hourly traffic volumes on Queen Ka‘ahumanu Highway on June 1, 2004, is shown in tabular and graph form on **Figure 5**. Separate curves are shown for the northbound and southbound traffic volumes. The northbound traffic volumes are higher than the southbound volumes for the first two hours of the morning. The southbound traffic volumes are higher for most of the afternoon hours except the last two hours.

PROPOSED ROADWAY IMPROVEMENTS

The HDOT and County of Hawai‘i have many roadway improvements planned to meet the expected growth in the area. The “Keahole to Honaunau Regional Circulation Plan County Action Plan” (August 2006) prepared by the County of Hawai‘i Planning Department identifies several specific improvements pertinent to this study. Those improvements include the widening of Queen Ka‘ahumanu Highway from Henry Street to the airport and the development of an extensive roadway network *mauka* of the highway.

The HDOT is currently widening the highway from two to four lanes from Henry Street to Kealakehe Parkway under Phase 1 of the widening project which is expected to be completed in 2008. The second phase is expected to be completed by 2011 and would extend the four lane design past the airport access roadway. The project would also add a northbound bicycle lane and a southbound bicycle route/paved shoulder lane.

The new roadway network mauka of the highway would create more *mauka-makai* roadways between Queen Ka‘ahumanu Highway and Mamalahoa Highway and create more north-south roadways between and parallel to these two existing highways. The three important north-south roadways include the Kealaka‘a Street Extension, Ane Keohokalole Highway Extension, and Main Street (Kamanu Street) Extension. Their net effect would be the diversion of trips from the existing highways.

A timetable for the development of these new roadways has not been established but would be tied in to new projects being built along the roadway alignments. The draft Kona Community Development Plan has developed a list of roadway projects in this area:

- Keanalehu Street-Manawale‘a Street connection
- Ane Keohokalole Highway Extension (Mid-level road) in stages from Palani Road to Ka‘iminani Drive
- Kamanu Street Extension
- Kealakaa Street Extension
- Hienaloli Street Extension
- University Drive
- Frontage Road
- Queen Ka‘ahumanu Highway widening, Phase II

TRAFFIC FORECASTS

The three forecast years for the ‘O‘oma Beachside Village are 2015, 2020, and 2029. During the three periods, the ambient or background traffic on Queen Ka‘ahumanu Highway can be expected to increase due to regional growth and new projects in the area. The traffic patterns in the study area would also change as new roadways are placed in operation. The traffic that would be generated from the ‘O‘oma Beachside Village was added to the ambient traffic forecast to obtain the total with project traffic forecast.

Ambient Traffic Forecast

The results of several traffic impact analysis reports for proposed projects in the area were analyzed to develop ambient traffic forecasts on Queen Ka‘ahumanu Highway at Ka‘iminani Drive, the NELHA access roadway, Huliko‘a Drive, and Hina Lani Street for the three forecast years. The forecast procedures and summary results for each study intersection are described below.

Ka‘iminani Drive - The traffic forecasts prepared by Rowell for UH Center at West Hawai‘i Main Street Collector Road (June 2006) were used for the 2015 forecast. Other projects included in the forecast were the Makalei Estates, Palamanui and Lokahi Subdivision. Very large traffic increases were forecast for the two intersecting roadways since the mauka network of roadways were not assumed to be well developed by 2015. Also, traffic flows became significantly northbound in the AM and southbound in the PM. For the 2020 ambient traffic forecast, the 2015 traffic volumes at Ka‘iminani Drive were increased by 1.3% for the five year period. This represents a 4.83% annual growth but with 20% of the growth being routed to the by then more defined mauka roadway network. Then for 2029, the 2020 volumes were increased by 5% over the nine year period. This represents a 4.83% annual growth with 28% being routed to the mauka roadways. For each planning year, the through volumes were continued to the NELHA access road intersection. The current and ambient forecast inbound and outbound traffic volumes are summarized as follows.

YEAR	AM PEAK HOUR		PM PEAK HOUR	
	INBOUND	OUTBOUND	INBOUND	OUTBOUND
2006	155	720	595	145
2015	440	1,015	1,290	445
2020	445	1,025	1,305	450
2029	470	1,070	1,375	470

NELHA Roadway – Traffic counts were taken on the NELHA access road in 2002 by HDOT. There is a sharp peak inbound peak in the morning and a sharp outbound peak in the afternoon with less than 100 vph in the peak direction. Most of the volumes in the other hours were low. Entering and exiting peak hour volumes were increased by 3% annually as follows:

YEAR	AM PEAK HOUR		PM PEAK HOUR	
	INBOUND	OUTBOUND	INBOUND	OUTBOUND
2002	86	28	41	87
2006	96	31	46	97
2015	120	39	57	121
2020	132	43	63	134
2029	153	50	73	155

These volumes were then distributed as shown below reflecting the increasing urbanization of the area north of the Property:

YEAR	PERCENT DISTRIBUTION BY DIRECTION OF TRAVEL	
	INBOUND	OUTBOUND
2015	45%	55%
2020	48%	52%
2029	50%	50%

Huliko‘a Drive – Two separate projects are planned on the mauka and makai sides of the highway at this intersection. Only inbound and outbound traffic forecasts were made for these two projects.

The existing Kohanaiki Business Park is accessed by Huliko‘a Drive on the mauka side of the highway. This intersection is currently unsignalized but there are plans to make this a fully accessible signalized intersection with the highway widening project. In lieu of traffic counts, the traffic forecast prepared by Pacific Planning and Engineering, Inc., in 1991 for the Kohanaiki Mauka project was updated for the current land use classifications and trip generation rates. The business park project was assumed to be fully occupied by 2015 and the results of this analysis were assumed to be constant for the three forecast years as follows:

	AM PEAK HOUR				PM PEAK HOUR			
	INBOUND		OUTBOUND		INBOUND		OUTBOUND	
YEAR	North	South	North	South	North	South	North	South
2006	65	95	95	65	35	50	90	135
2015	125	190	130	195	65	100	180	270
2020	125	170	130	175	65	90	170	240
2029	125	170	130	175	65	90	170	240

For the purposes of this study, the existing 2006 volumes were assumed to be half of the 2015 forecasts. The south inbound and outbound volumes were reduced slightly for 2020 and 2029 since the Kamanu Street Extension would intersect the northern terminus of Huliko‘a Drive and provide an alternate route to the south, thereby diverting some trips.

The Shores of Kohanaiki is planned for the makai side of the highway. Its access road would intersect the highway across from Huliko‘a Drive and form the west leg of the fully accessible, signalized intersection. The access road would also serve as the southern terminus for the makai frontage road. A letter report prepared by Julian Ng, Inc., in 2003 discussed the trip generation characteristics of the Shores of Kohanaiki project with proposed new land uses (500 dwelling units, an 18-hole golf course, and

120 parking stalls for public beach access). The Shores of Kohanaiki has been approved and is expected to be in place by 2015. Only entering and exiting volumes were forecast for each analysis year:

PEAK HOUR	VEHICLE TRIPS/HOUR	
	INBOUND	OUTBOUND
AM	125	290
PM	465	235

The trips were distributed north and south on the highway and a small portion of trips was assumed to use the makai frontage road to access the airport. The through volumes on the highway were forecast at the Hina Lani Street intersection and continued to Huliko‘a Drive.

Hina Lani Street – For 2015, the existing 2006 through and turning volumes were increased by 1.529, which is the 4.83% annual growth rate compounded for 9 years. For 2020, the through volumes were increased by 1.3% similar to Ka‘iminani Drive, however turning volumes for 2020 from the TIAR prepared by Fehr & Peers/Kaku Associates for the Kula Nei Residential Development were used. This forecast also included the traffic which would be generated by the proposed Kaloko Heights subdivision. For 2029, the 5% growth factor used at Ka‘iminani Drive was also used here. The through traffic forecasts were carried to the Huliko‘a Drive intersection. The current and ambient forecast inbound and outbound traffic volumes are summarized below:

YEAR	AM PEAK HOUR		PM PEAK HOUR	
	INBOUND	OUTBOUND	INBOUND	OUTBOUND
2006	490	560	620	580
2015	740	860	960	975
2020	900	1,205	1,130	935
2029	930	1,050	1,215	995

The results of the ambient traffic forecasts are shown on **Figure 6** with the frontage road assumed in place. The AM peak hour forecasts for the three forecast years are shown on the first page of the figure, while the PM peak hour forecasts are shown on the second page. The NELHA access road was assumed to provide right turn in, right turn out access to the highway.

Project Generated Traffic

The traditional three-step process of trip generation, trip distribution, and trip assignment was used to forecast future traffic that would be generated by ‘O‘oma Beachside Village. The trip generation step forecasts the number of new trips that would be produced during each of the two study periods. The trip distribution step allocates these new trips by direction of travel. Finally, the trip assignment step assigns the trips to the specific turning movements at the study intersections.

The trip generation step forecasts the volume of vehicle trips that would be generated by ‘O‘oma Beachside Village during the morning and afternoon peak periods. The Institute of Transportation Engineers’ Trip Generation Report (Seventh Edition, 2003) has rates to calculate the number of morning and afternoon peak hour trips that would be generated by various land uses.

An initial step was to correlate the land uses proposed in ‘O‘oma Beachside Village with the land uses included in the Trip Generation Report that would have similar trip generation characteristics. The results of this analysis are summarized on Table 1 and are discussed below:

- The single family residential units utilized the equations/rates for single family detached housing (ITE land use 210).
- All multi-family residential units including the mixed use and live-work units were assumed to be low-rise condominiums/town houses (ITE land use 231) that are described as residential units that have at least one other unit located in the same building that has one or two levels.

- The makai mixed use village commercial area was assumed to be retail-oriented and was classified as a shopping center (ITE land use 820). The mauka mixed use/live-work village was assumed to be an office park (ITE land use 750). The ITE report describes the latter as suburban subdivisions or planned unit developments containing general office buildings and support services such as banks, restaurants, and service stations, arranged in a park-like setting. This was the closest land use to the suburban neighborhood commercial center envisioned for this proposed project.
- The charter school was assumed to have the trip generation characteristics of a private school with grades K-8 (ITE land use 534) and having 225 students.
- The grocery store was assumed to be a 15,000 sf supermarket (ITE land use 850).
- The restaurant and private canoe club was assumed to be a 20,000 sf quality restaurant (ITE land use 931) with turnover rates usually of one hour or longer.
- There are no trip generation rates for a public beach use. Based on the previously referenced letter report by Ng, the following number of beach use trips were forecast:

	HOURLY TRIPS			
YEAR	AM PEAK HOUR		PM PEAK HOUR	
	INBOUND	OUTBOUND	INBOUND	OUTBOUND
2015	50	10	20	50
2020	60	15	25	60
2029	70	20	25	70

The trip generation analysis for each land use in each analysis year is detailed on **Table 2**, including the trip generation equations and rates from the ITE report.

The Trip Generation Report also provides the percentage of inbound and outbound trips in each peak hour. The number of generated trips was divided into inbound and outbound trips based on the information from the report, as shown on **Table 2**.

The first forecast year (2015) of ‘O‘oma Beachside Village is summarized on the first page of **Table 2**, and it would generate 187 outbound and 131 inbound trips in the morning peak hour, and 310 inbound and 243 outbound trips in the afternoon peak hour. The second analysis year (2020) is summarized on the second page and it would generate 445 outbound and 421 inbound trips in the morning peak hour and 656 inbound and 701 outbound trips in the afternoon peak hour. The third analysis year (2029) is summarized on the third and fourth pages and it would generate 884 outbound and 906 inbound trips in the morning peak hour and 1,023 inbound and 1,128 outbound trips in the afternoon peak hour.

The project generated trips were then distributed by three primary direction of travel to and from the Property: north and south of the Property, and internal to the Property. The distribution of external trips was determined from the current distribution of population and employment in West Hawai‘i. The districts closer to the Property were weighted higher due to the propensity for shorter trips to be made more frequently. This analysis indicated that the current weighted population and employment distributions are 55% south and 45% north. These proportions were assumed for the employment distribution in all three forecast years. The proportion of population to the north was assumed to be 45% in 2015, 48% in 2020, and 50% in 2029, reflecting the trend of urbanization to the north. The morning outbound residential trips and the afternoon inbound trips were distributed based on the employment distribution. The distribution of population was used for all other trips. The percentage of internal trips were initially calculated for the non-residential land uses, and made to balance the corresponding resident-generated trips. The trip distribution rates also considered that a portion of the trips from the live-work units and to a smaller extent, the mixed use units, would not be made outside of ‘O‘oma Beachside Village and the proportion of internal trips were increased accordingly.

The results of the trip distribution analysis are shown on **Table 3** with the 2015 results on the first page, the 2020 results on the second page, and the 2029 results on the third page. The residential land uses were combined into a single land use for this calculation. Similarly, the two mixed-use village commercial uses and the live-work commercial use were combined together.

The project generated traffic volumes were assigned to the highway and frontage road network with movements as permitted. The results of the traffic assignment analysis are shown on **Figure 7** with the volumes not rounded.

A unique aspect of trips attracted by commercial centers is that a number of these trips are pass-by trips. Pass-by trips are attracted from traffic passing the site on an adjacent roadway having direct access to the commercial center. Therefore, these trips do not add to the through volumes on the roadway. They are added to the turning movements but are subtracted from the through movements where they turn off to access the commercial center. The commercial areas of ‘O‘oma Beachside Village are not expected to draw pass-by trips in the morning peak hour but would attract some pass-by trips in the afternoon peak hour, especially trips stopping for shopping purposes. These trips are shown as negative volumes on the trip assignments (**Figure 7**).

Total Forecast Volumes

The project generated traffic assignment volumes from **Figure 7** were added to their corresponding ambient traffic forecasts from **Figure 6** to obtain the total with project traffic forecasts shown on **Figure 8** for each forecast year. The traffic volumes are rounded to the nearest five vph.

LEVEL OF SERVICE ANALYSIS

The traffic forecast volumes in themselves do not indicate the quality of traffic operations. The concept of level of service is used to quantify the quality of traffic flow on roadway facilities. The Transportation Research Board (TRB) has developed procedures to calculate level of service value(s) by measuring traffic volumes against the capacities of different types of roadway facilities. Their Highway Capacity Manual 2000 (HCM2000) describes the various procedures developed for freeways, highways, signalized and unsignalized intersections, etc.

A variety of methodologies was used to analysis existing and forecast traffic conditions. The methodology for analyzing signalized intersections was used for the Ka‘iminani Drive, Huliko‘a Drive, and Hina Lani Street intersections. The methodology for analyzing unsignalized intersections was used for the existing NELHA access road and Huliko‘a Drive intersections. The methodology for analyzing highway on-ramps was used for the future right turn out movement at the NELHA and ‘O‘oma Beachside Village access roads. Finally, separate methodologies for analyzing two-lane and multi-lane highways were used for the current and forecast highway conditions fronting the Property.

Signalized Intersection Analysis

The Ka‘iminani Drive, Huliko‘a Drive and Hina Lani Street study intersections are/will be signalized. The methodology for analyzing signalized intersections calculates the levels of service for individual movements, approaches, and the intersection as a whole based on the average stopped delay per vehicle. The results range from level of service A (best with average delays less than ten seconds) to F (worst with average delays longer than 80 seconds, described as follows.

LEVEL OF SERVICE	CONTROL DELAY PER VEHICLE (Seconds/Vehicle)
A	< 10.0
B	10.1 to 20.0
C	20.1 to 35.0
D	35.1 to 55.0
E	55.1 to 80.0
F	> 80.1

The County of Hawai‘i considers levels of service A to D as acceptable by ordinance with levels of service E and F indicating the need for mitigating measures. As a matter of practice, the major streets of signalized intersections can be designed to have a higher level of service than the side streets or turning lanes with the latter having unacceptable levels of service in order to maintain an acceptable level of service on the main road. These unacceptable levels of service are often times caused by long waits for the green traffic signal phase rather than by capacity problems and are indicated by low values of the volume/capacity (V/C) ratio as described below.

The results of the signalized intersection level of service analysis for the Queen Ka‘ahumanu Highway intersections with Ka‘iminani Drive, Huliko‘a Drive, and Hina Lani Street are shown on **Tables 4, 5,** and **6,** respectively. Each table is for a single intersection and includes the results for the AM (morning) and PM (afternoon) peak hours for the intersection as a whole, each approach of the intersection, and the left turn, through and right turn movements of each approach. The results are shown for the 2006 existing conditions (for Ka‘iminani Drive on **Table 4** and for Hina Lani Street on **Table 6**) and the years 2015, 2020, and 2029 forecasts, with ambient without project and total with project results for each forecast year. The specific results data shown for each year includes the level of service (LOS), average stopped delay (DEL) and volume/capacity ratio (V/C), which is a percentage utilization of the traffic signal green time given the entire intersection and each movement. The level of service calculation worksheets are provided in **Appendix B**.

<u>Queen Ka‘ahumanu Highway/Ka‘iminani Drive</u> – The results of the signalized intersection level of service analysis for the Queen Ka‘ahumanu Highway/Ka‘iminani Drive intersection are shown on **Table 4**. The intersection is currently operating at an acceptable level of service B in the AM peak hour. With the large increases in traffic volumes forecast for 2015 ambient conditions, the Ka‘iminani Drive westbound approach would require two left turn lanes to maintain the acceptable levels of service. The frontage road approach is forecast to operate at level of service F due to the long wait for the green phase and not capacity problems, as evidenced by the low V/C ratio. The additional traffic generated by ‘O‘oma Beachside Village would cause the Ka‘iminani Drive approach to change from level of service D to E, but the intersection would continue to operate at level of service D. Similarly, the intersection levels of service would remain at acceptable levels for the 2020 and 2029 forecast years, although individual and approach levels could be at unacceptable levels.

The intersection is currently operating at an acceptable level of service B in the PM peak hour. As with the AM peak hour, the Ka‘iminani Drive westbound approach would require two left turn lanes by 2015 to maintain the acceptable levels of service for the ambient traffic forecast. The large traffic increases forecast for 2020 and 2029 would require additional mitigation in the form of two southbound left turn lanes and two northbound right turn lanes to maintain the intersection level of service D for both ambient and total with project conditions. The AM peak hour forecasts would not require these additional improvements but the AM peak hour results shown on **Table 4** do include these mitigating measures. As with the AM peak hour, several approaches/individual movements may have to operate at unacceptable levels of service to maintain an acceptable intersection level of service.

The analysis for the Queen Ka‘ahumanu Highway/Ka‘iminani Drive intersection indicates that this intersection could operate at acceptable levels of service with mitigation measures for the ambient traffic forecasts. These include having double left turn lanes on the Ka‘iminani Drive westbound approach by 2015, and double left turn lanes on the highway southbound approach and double right turn lanes on the highway northbound approach by 2020. Additional mitigating measures would not be required to accommodate traffic generated from ‘O‘oma Beachside Village.

Queen Ka‘ahumanu Highway/Huliko‘a Drive – The results for the Queen Ka‘ahumanu Highway/Huliko‘a Drive intersection are shown on **Table 5.** There is no existing analysis since the intersection is not currently signalized. The intersection is forecast to operate at an acceptable level of service C for the three ambient forecast years in the AM peak hour, although several individual movements would be at unacceptable levels. The AM peak hour 2029 ambient traffic forecast shows a double left turn lane for the northbound highway approach since it would be required for the PM peak hour condition.

The AM peak hour 2015 and 2020 total with project traffic forecasts shows a double left turn lane for the northbound highway approach since it would be required for the PM peak hour condition. With the additional traffic generated by ‘O‘oma Beachside Village in 2020 the intersection level of service would change from C to D, which is considered an acceptable level of service. The additional project generated traffic in 2029 would require a double left turn lane on the northbound highway approach to maintain the intersection level of service D. The long delays on the Huliko‘a Drive approaches are due to the long cycle lengths and not capacity problems, as noted by the low V/C ratios.

The PM peak hour has higher volumes and worse levels of service as a result. The intersection is forecast to operate at an acceptable level of service D for the three forecast year ambient conditions, although the 2029 forecast would require a double left turn lane on the northbound highway approach as a mitigating measure to maintain the intersection level of service D. The intersection levels of service for the 2015 and 2020 total with project forecasts could be maintained at D with a double left turn lane on the northbound highway approach. Additional mitigation in the form of double left turn lanes on the Huliko‘a Drive westbound approach would be needed to accommodate the 2029 total with project forecast.

The analysis for the Queen Ka‘ahumanu Highway/Huliko‘a Drive intersection indicates that this intersection would be impacted by traffic generated from ‘O‘oma Beachside

Village and would require mitigation to operate at acceptable levels of service. These measures include having double left turn lanes on the Queen Ka‘ahumanu Highway northbound approach by 2015, and double left turn lanes on Huliko‘a Drive westbound approach by 2029.

The level of service analysis indicated that the Huliko‘a Drive intersection would operate at an acceptable level D for the volumes forecast with the large conflicting volumes of southbound through traffic and northbound left turns. This assumes that sufficient traffic would be diverted to the mauka roadway network. If the highway volumes are higher than forecast due to insufficient traffic being diverted to the mauka roadway network or other unforeseen reasons, then the intersection could operate at unacceptable levels of service. As a contingency measure for this possibility, the “Michigan U-turn” should be considered as a supplemental mitigating measure to divert turning traffic movements from the intersection and reduce the conflicting movements.

The Michigan U-turn requires a U-turn facility in the highway median in concert with a right turn in, right turn out access roadway so that left turns are not made. Exiting left turns from the access roadway would make a right turn onto the highway, merge across highway traffic into the left-most lane, then make a U-turn on the highway median facility, and then proceed in the opposite direction from which they started. Similarly, incoming left turns would proceed on the opposite side of the median past the access road, make a U-turn on the highway median facility, then merge across highway traffic into the right-most lane, and then make a right turn into the access roadway. A Michigan U-turn on Queen Ka‘ahumanu Highway for the ‘O‘oma Beachside Village would eliminate some of the crossing and turning movements at the Ka‘iminani Drive and Huliko‘a Drive intersections and make them work more efficiently. The two median U-turn facilities would be located between the ‘O‘oma Beachside Village and Huliko‘a Drive and between the ‘O‘oma Beachside Village and NELHA access road. The second facility could be located further north between the NELHA access road and Ka‘iminani Drive to include NELHA in the Michigan U-turn.

Queen Ka‘ahumanu Highway/Hina Lani Street – The results of the signalized intersection level of service analysis for the Queen Ka‘ahumanu Highway/Hina Lani Street intersection are shown on **Table 6**. The intersection is currently operating at an acceptable level of service C in both peak hours, and is forecast to operate at a still acceptable level of service D for the 2015 ambient without project and total with project forecasts. The development of the mauka residential projects would generate the need for a double left turn lane on the westbound approach of Hina Lani Street by 2020. The additional traffic generated by the ‘O‘oma Beachside Village would not require any additional mitigation. Hence, the ‘O‘oma Beachside Village is not expected to contribute to adverse traffic impacts at the Hina Lani Street intersection until after 2020. However, the additional project generated traffic would require mitigation in 2029 to maintain acceptable level of service for the intersection. A double left turn lane on the southbound highway approach would improve the intersection level of service to C.

Signalized Intersection Conclusions – The preceding level of service analysis indicated the need for mitigating measures to accommodate the project generated traffic by 2029. This need should be considered as speculative due to the uncertainties associated with such a long forecast period, including regional development projects and *mauka* roadway plans that may or may not be actually accomplished. Contingencies should be made to implement these measures while recognizing that their needs may not actually occur.

Unsignalized Intersection Analysis

The NELHA access road and Huliko‘a Drive intersections are currently unsignalized. The procedure used for analyzing unsignalized intersections calculates vehicle delays and levels of service based on the distribution of gaps in traffic on the major street and driver judgment in selecting gaps through which to execute turns. For two-way stop intersections where only the minor street traffic is controlled by a stop sign, levels of service are calculated for the critical turning movements, including outbound movements from the stop-controlled approach and left turns from the major street to the minor street. The procedure does not calculate an overall intersection level of service.

The Highway Capacity Manual defines the relationship between level of service and delay (in seconds/vehicle) for unsignalized intersections as shown below:

LEVEL OF SERVICE	DELAY (Seconds/Vehicle)
A	< 10.0
B	10.1 to 15.0
C	15.1 to 25.0
D	25.1 to 35.0
E	35.1 to 50.0
F	> 50.1

The County of Hawai‘i considers levels of service A to D as acceptable for unsignalized intersections. Level of service F (with average delays longer than 50 seconds) is considered undesirable for unsignalized intersections and indicates the possible need for mitigation at that intersection.

The results of current operations at the two current unsignalized intersections are shown on **Table 7**. The critical movement at each intersection is the outbound left turn. Based on the estimated current volumes at each intersection, this movement at the NELHA access road intersection is at level of service F in the AM and E in the PM peak hour. Similarly, this movement at the Huliko‘a Drive intersection is at level of service F in both peak hours. These results indicate the current need for mitigating measures at both intersections. The level of service calculation worksheets are provided in **Appendix C**.

No future study intersections were analyzed as unsignalized intersections since none are expected to operate as unsignalized intersections.

Highway On-Ramp Analysis

The access roadways serving ‘O‘oma Beachside Village and NELHA are expected to be unsignalized and limited to right turn in, right turn out movements. The methodology for analyzing highway on-ramps was used instead of an unsignalized intersection analysis since the right turn lane would have adequate acceleration and taper lengths to perform like a highway on-ramp. The methodology for analyzing on-ramps calculates maximum flow rates in passenger cars/hour/lane based on the volumes of highway/roadway and merging traffic, and roadway capacities, and then calculates levels of service based on density as follows:

LEVEL OF SERVICE	**DENSITY (passenger car/mile/lane)**
A	≤ 10
B	> 10 - 20
C	> 20 - 28
D	> 28 - 35
E	> 35
F	Demand > Supply

The results of the on-ramp analysis are summarized on **Table 8** for the total with project forecasts only. For each of the three forecast years, both access roads (for ‘O‘oma Beachside Village and NELHA) are calculated to operate at levels of service B in the AM peak hour and C in the PM peak hours, indicating acceptable levels of service in both analysis periods. This indicates that the traffic generated by ‘O‘oma Beachside Village would not have an adverse traffic impact on this aspect of the highway operations. The level of service calculation worksheets are provided in **Appendix D**.

Highway Analysis

Queen Ka‘ahumanu Highway is currently a two-lane highway that the HDOT is currently widening to a four multi-lane highway. Separate methodologies and criteria are used for calculating levels of service for these two distinct highway types.

The ideal (maximum) capacity of a two-way, two-lane highway is 1,700 passenger car equivalents per hour per lane, and 3,200 passenger car equivalents per hour for both directions of travel. This is lower than the capacity of a multi-lane highway that can range from 2,000 to 2,200 passenger car equivalents per hour per lane. The analysis procedure for two-way, two-lane highways takes into account the more restrictive aspects of its operations relative to wider multi-lane highways. The procedure considers the impact of geometric data: lane width, shoulder width, type of terrain, free flow speed, percent no passing zones; and demand characteristics: volumes, percent of heavy vehicles; as some of the inputs. For Class I highways like Queen Ka‘ahumanu Highway where efficient mobility is important and drivers expect to drive at relatively high speeds, level of service is defined in terms of both percent time spent following other vehicles and average travel speeds. The level of service criteria for Class I two-lane highways are shown below:

LEVEL OF SERVICE	PERCENT TIME SPENT FOLLOWING	AVE. TRAVEL SPEED (Miles/Hour)
A	< 35	> 55
B	>35 to 50	>50 to 55
C	>50 to 65	>45 to 50
D	>65 to 80	>40 to 45
E	> 80.0	<40

The methodology for analyzing multi-lane highways calculates several criteria based on the capacity and design characteristics of the highway and traffic volumes. There are several sets of criteria for levels of service based on the free flow speed of the highway. The criteria for a 55 mph free flow speed (FFS) are summarized as follows.

	LOS CRITERIA FOR 55 MPH FFS				
CRITERIA	**A**	**B**	**C**	**D**	**E**
Maximum Density (passenger car /mile/lane)	11	18	26	35	41
Average speed (mph)	55.0	55.0	54.9	52.9	51.2
Max. Volume/Capacity Ratio (V/C)	0.29	0.47	0.68	0.88	1.00
Max. Service Volume Flow Rate (passenger car/hour/lane)	600	990	1,430	1,850	2,100

The results of the highway analysis are shown on **Table 9**. The first line shows that the existing two-lane highway is currently operating at level of service E in both peak periods, primarily due to the high percentage of time spent following other cars and the lack of opportunity to pass slower vehicles. The remaining lines show the results for the ambient without project and total with project forecasts for southbound traffic fronting the Property. With the highway widening, the highway is calculated to operate at levels of service B in the AM peak hours and C in the PM peak hours, indicating acceptable levels of service in both analysis periods. There is no difference between the ambient without project and the total with project results, indicating that the traffic generated by ‘O‘oma Beachside Village would not have an adverse traffic impact on this aspect of the highway operations. The level of service calculation worksheets are provided in **Appendix E**.

CONCLUSIONS

The widening of Queen Ka‘ahumanu Highway and the development of the mauka roadway network would accommodate much of the anticipated growth in the North Kona region. The highway system is expected to operate at acceptable levels of service in the forecast future.

The ‘O‘oma Beachside Village is not expected to have a fully accessible intersection connection with the widened Queen Ka‘ahumanu Highway; however, the right turn in, right turn out access roadway intersection is expected to operate at acceptable levels of service in the forecast future.

The ‘O‘oma Beachside Village is planned to include a frontage road makai of and parallel to Queen Ka‘ahumanu Highway. This frontage road would allow access to fully accessible intersections at Ka‘iminani Drive and Huliko‘a Drive, where vehicles traveling from and to ‘O‘oma would be able to make left turns onto and from the highway, respectively. These intersections would require mitigating actions to accommodate the ambient forecast traffic. The additional traffic generated by the ‘O‘oma Beachside Village would require further mitigating measures to maintain acceptable levels of service at the Huliko‘a Drive and Hina Lani Street intersections including the following:

- Huliko‘a Drive - a double left turn lane on the northbound highway approach by 2015.

- Huliko‘a Drive - a double left turn lane on the westbound approach by 2029.
- Hina Lani Street - a double left turn lane on the southbound highway approach by 2029.

However, the need for mitigating measures to accommodate the project generated traffic by 2029 should be considered as speculative due to the uncertainties associated with such a long forecast period, including: 1) whether or not other development projects in the region are built or are built with as many units as currently anticipated; 2) the implementation of the mauka roadway network as currently planned and how much turning movement traffic is diverted to the mauka roadway system as it is completed; and 3) the level of mitigating measures that would be imposed on other development projects that could mitigate the impact of ambient traffic. Contingencies should be made to implement these measures while recognizing that their needs may not actually occur. The right turn in, right turn out access roadway intersection and highway system are expected to operate at acceptable levels of service in the forecast future.

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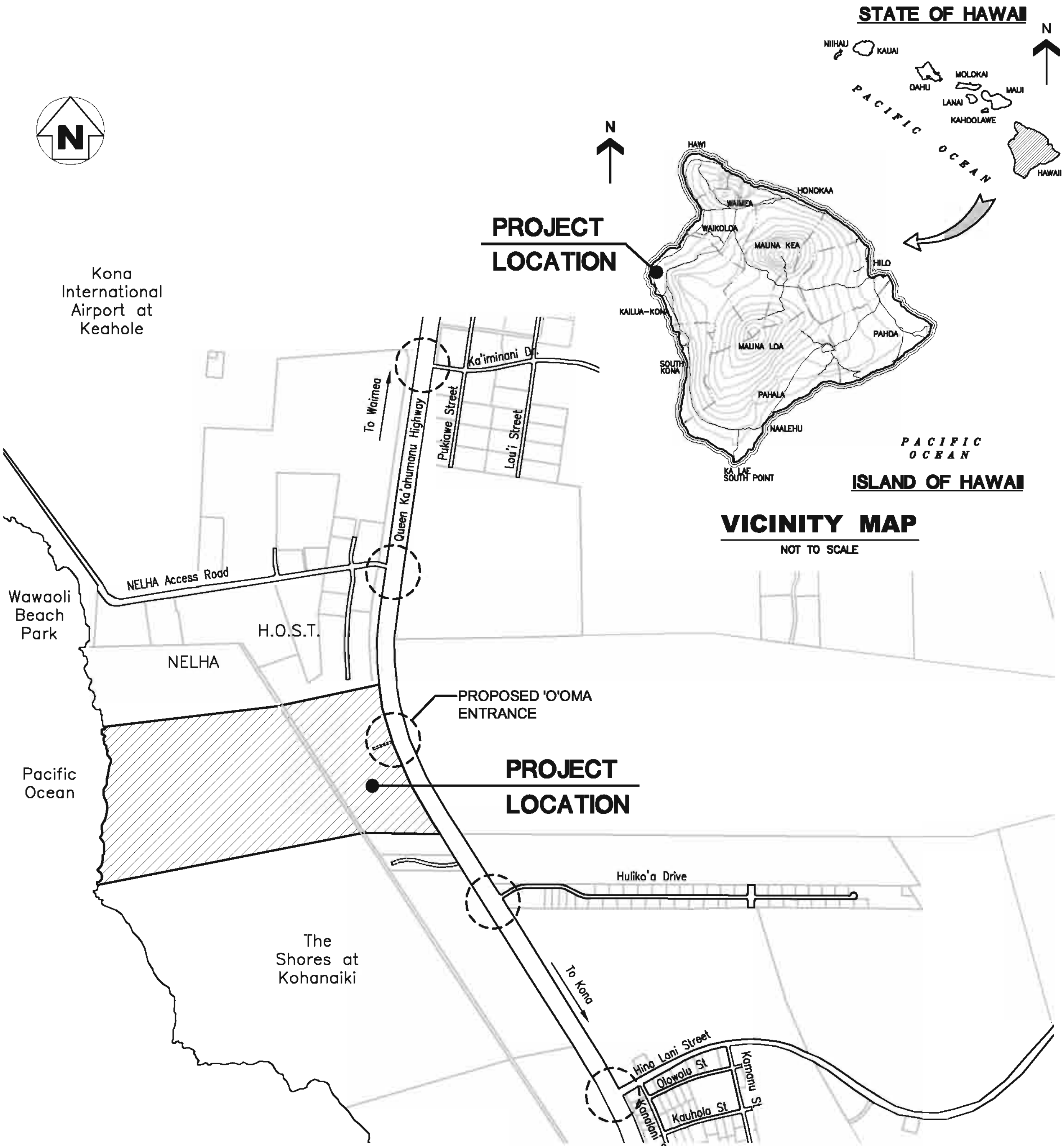
Figures



LOCATION MAP

NOT TO SCALE

STUDY INTERSECTIONS

M&E Pacific, Inc.

METCALF & EDDY | AECOM

DAVIES PACIFIC CTR, STE 1900 · 841 BISHOP ST, HONOLULU, HAWAII 96813

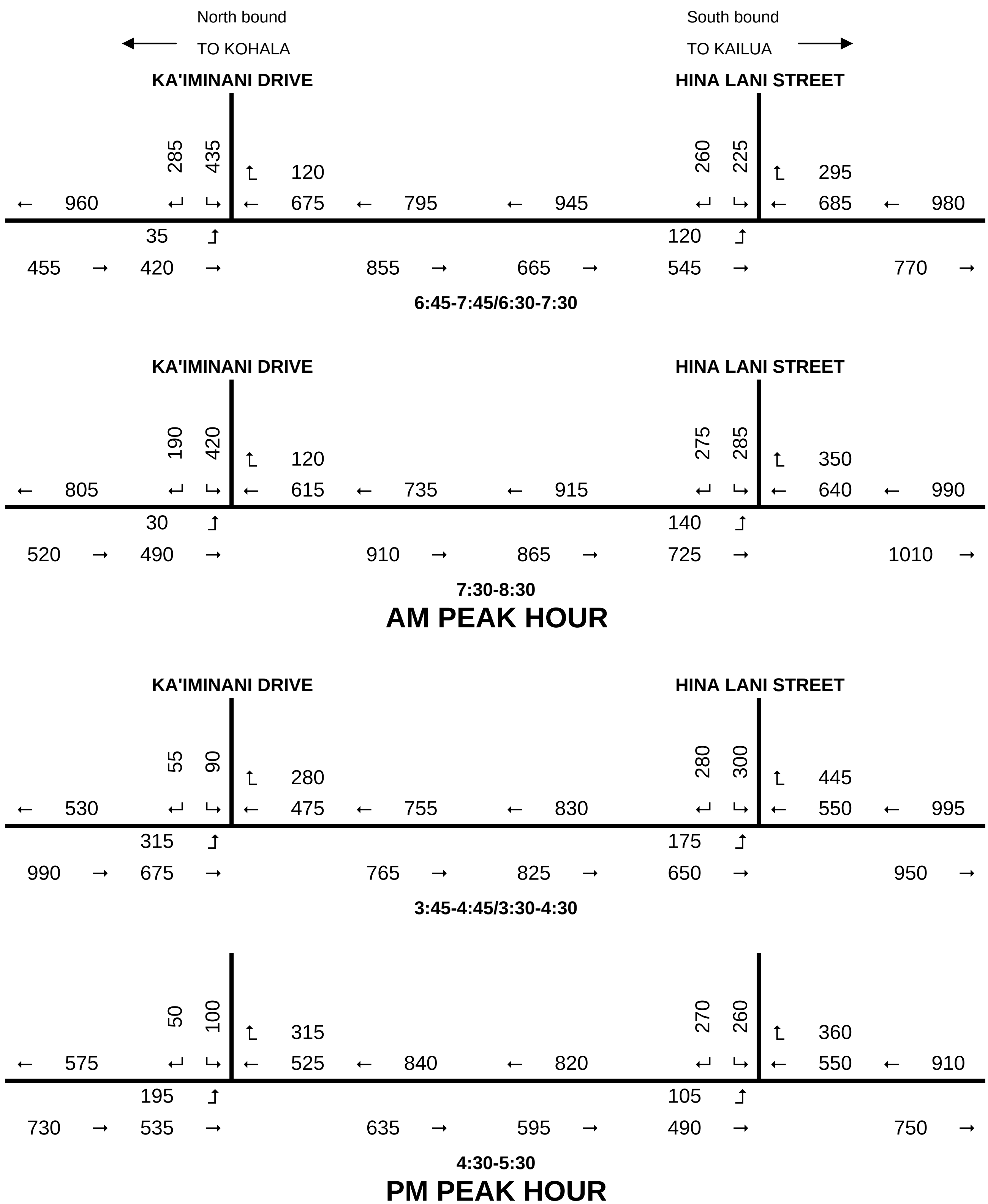
Figure 1
Location Map
Traffic Impact Analysis Report
'O'oma Beachside Village
May 2008

M&E Pacific, Inc.

METCALF & EDDY | AECOM

DAVIES PACIFIC CTR, STE 1800 · 841 BISHOP ST, HONOLULU, HAWAII 96813

Figure 2
Conceptual Master Plan
Traffic Impact Analysis Report
'O'oma Beachside Village
May 2008



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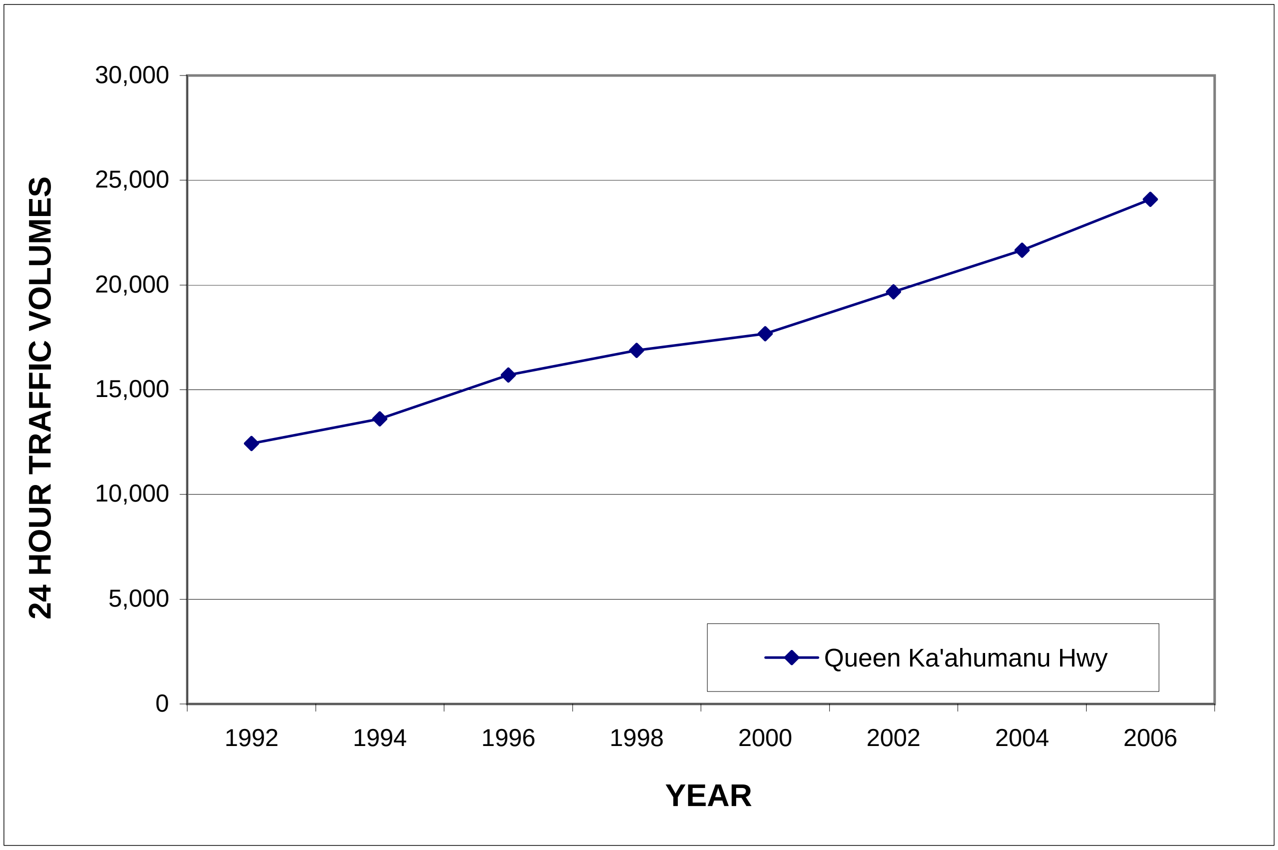
2006 EXISTING TRAFFIC VOLUMES
FIGURE 3

TWO-WAY DAILY TRAFFIC VOLUMES	
YEAR	QUEEN KA'AHUMANU HWY
1992	12,432
1994	13,610
1996	15,709
1998	16,882
2000	17,670
2002	19,678
2004	21,654
2006	24,085

*Average Weekday Daily Traffic

Source: State of Hawai'i Department of Transportation
Station T-8-M, June 1, 2004



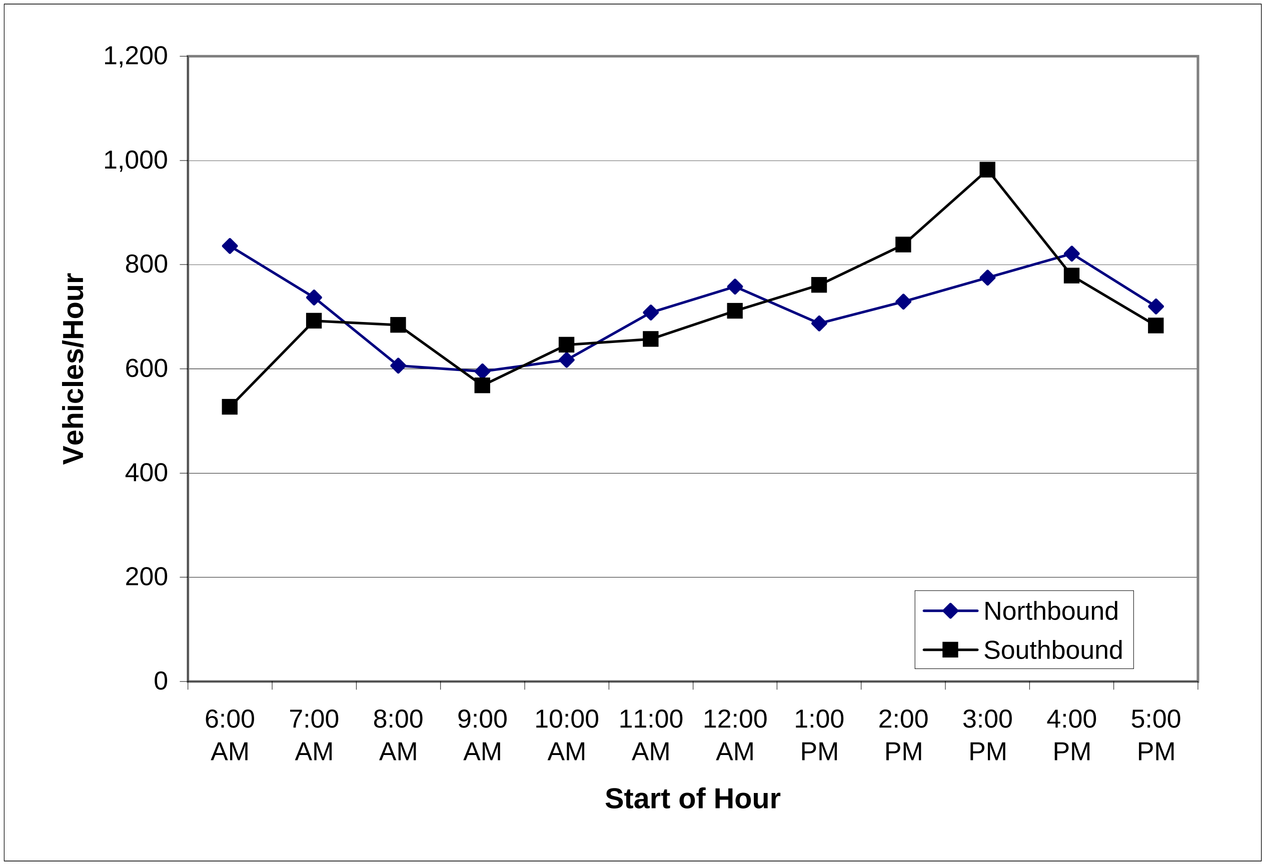
HISTORICAL TREND IN DAILY TRAFFIC VOLUMES ON QUEEN KA'AHUMANU HIGHWAY AT NELHA ACCESS ROAD

FIGURE 4

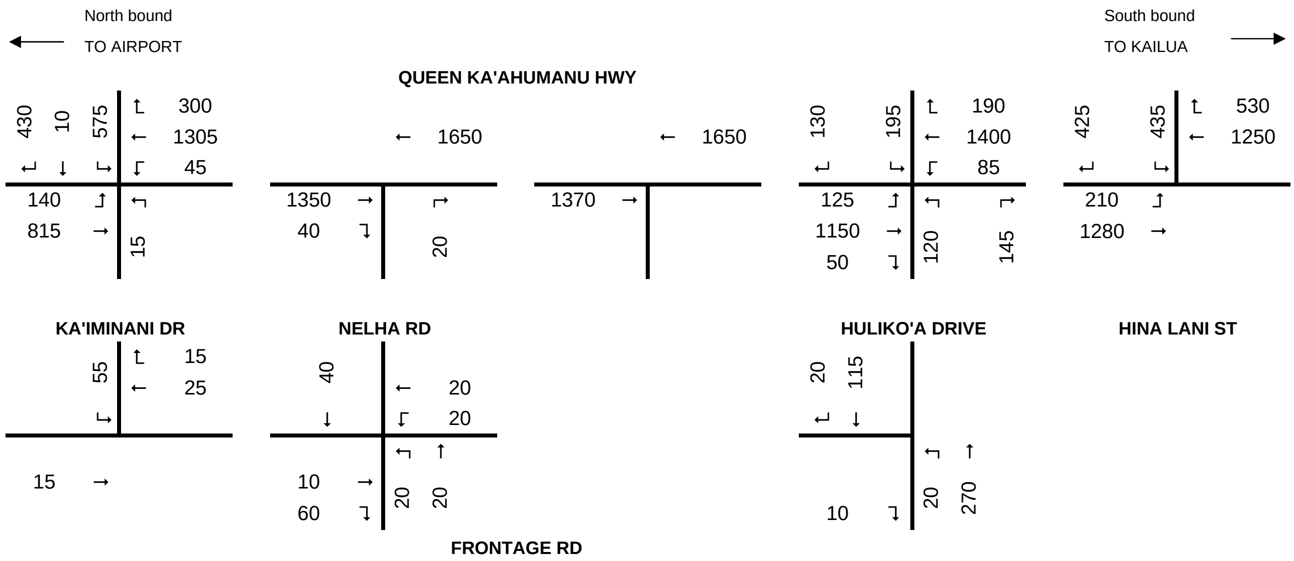
HOURLY TRAFFIC VOLUMES ON QUEEN KA'AHUMANU HIGHWAY
AT STATION T-8-M, North of NELHA access road, June 1, 2004

Vehicles/Hour		
Start of Hour	North-Bound	South-Bound
6:00 AM	836	527
7:00 AM	737	692
8:00 AM	606	684
9:00 AM	595	568
10:00 AM	617	646
11:00 AM	708	657
12:00 AM	758	711
1:00 PM	687	761
2:00 PM	729	838
3:00 PM	775	982
4:00 PM	821	779
5:00 PM	720	683

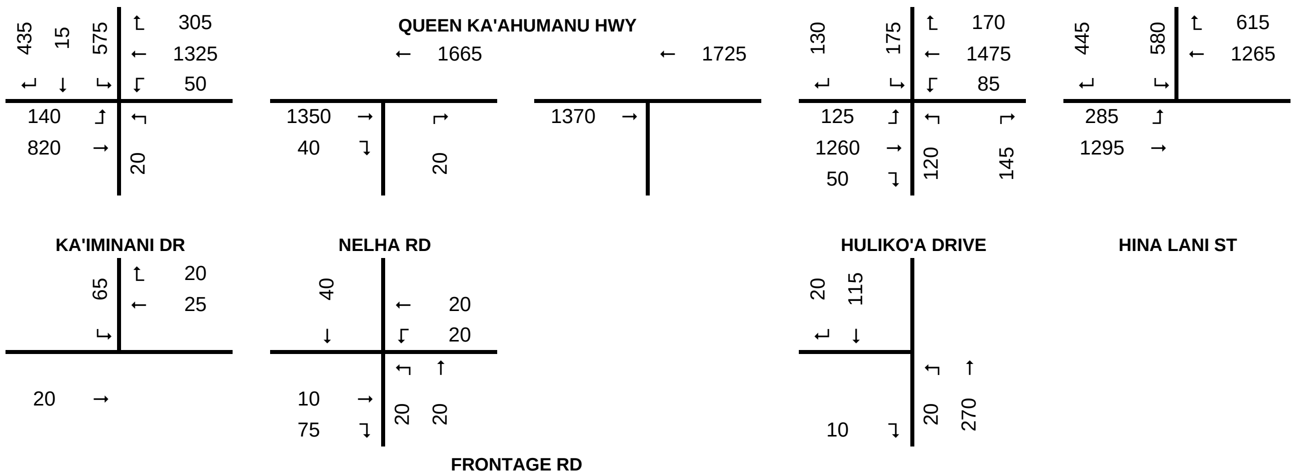
Source: State of Hawaii
Department of Transportation



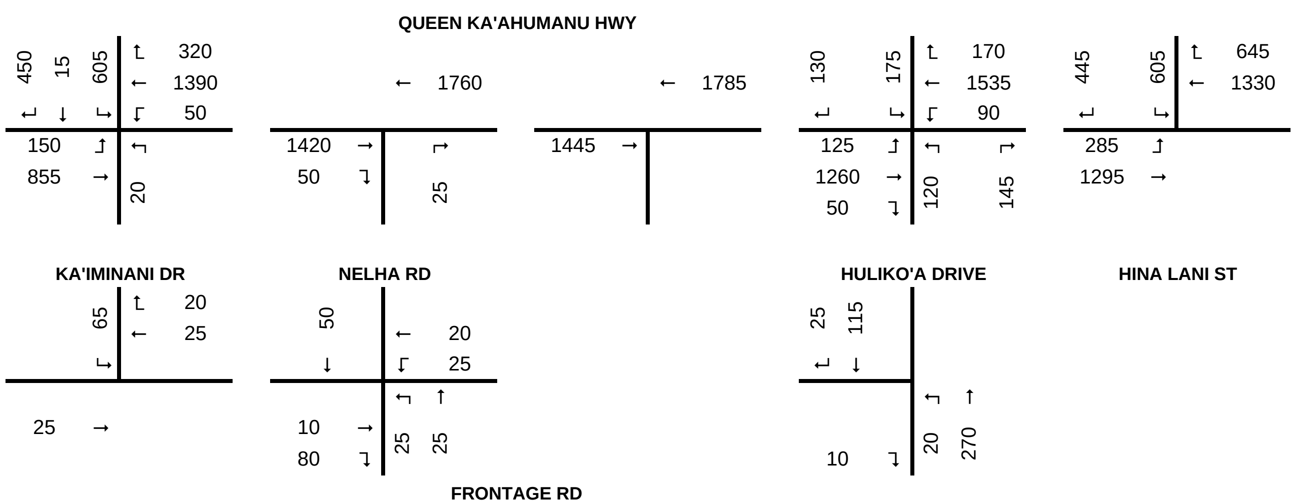
HOURLY TRAFFIC VOLUMES ON
QUEEN KA'AHUMANU HIGHWAY AT NELHA ACCESS ROAD
FIGURE 5



PLANNING YEAR 2015



PLANNING YEAR 2020

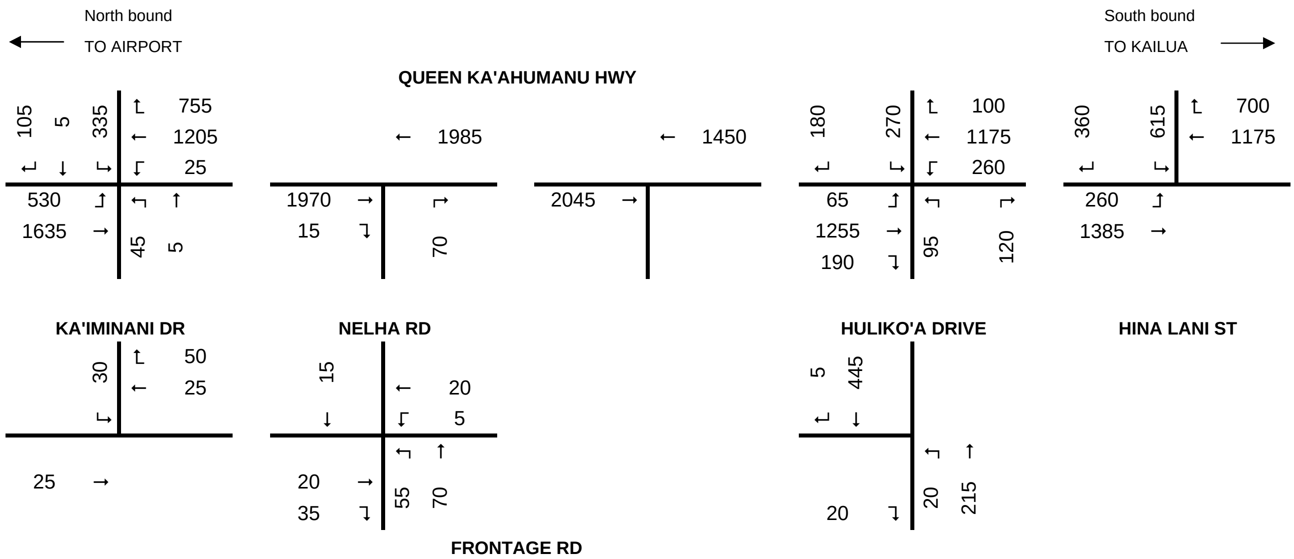


PLANNING YEAR 2029

AM PEAK HOUR

Not To Scale

AMBIENT TRAFFIC FORECAST
FIGURE 6



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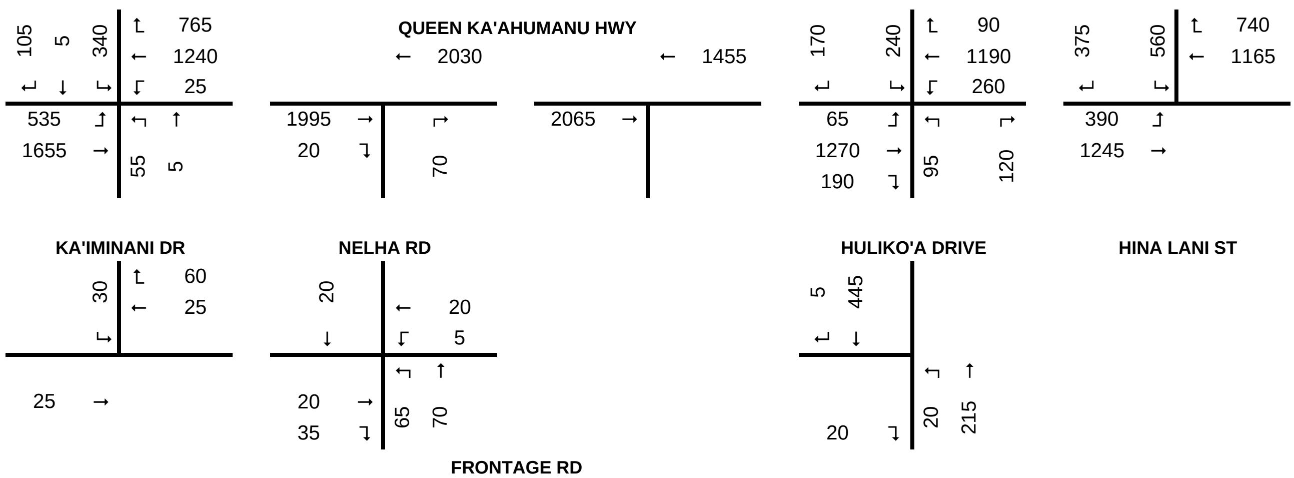
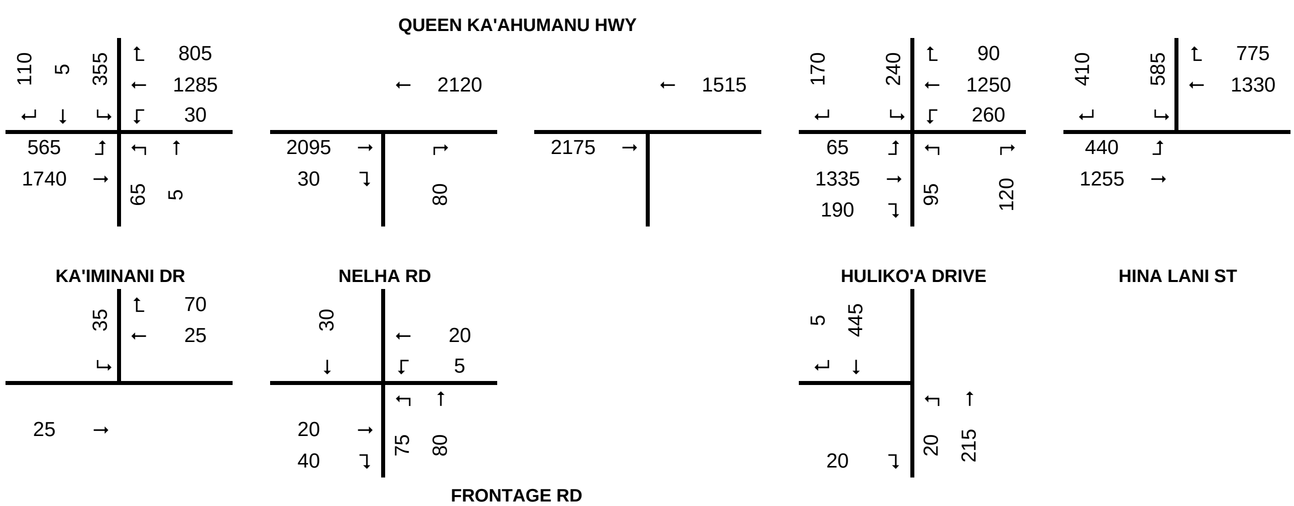


PLANNING YEAR 2020

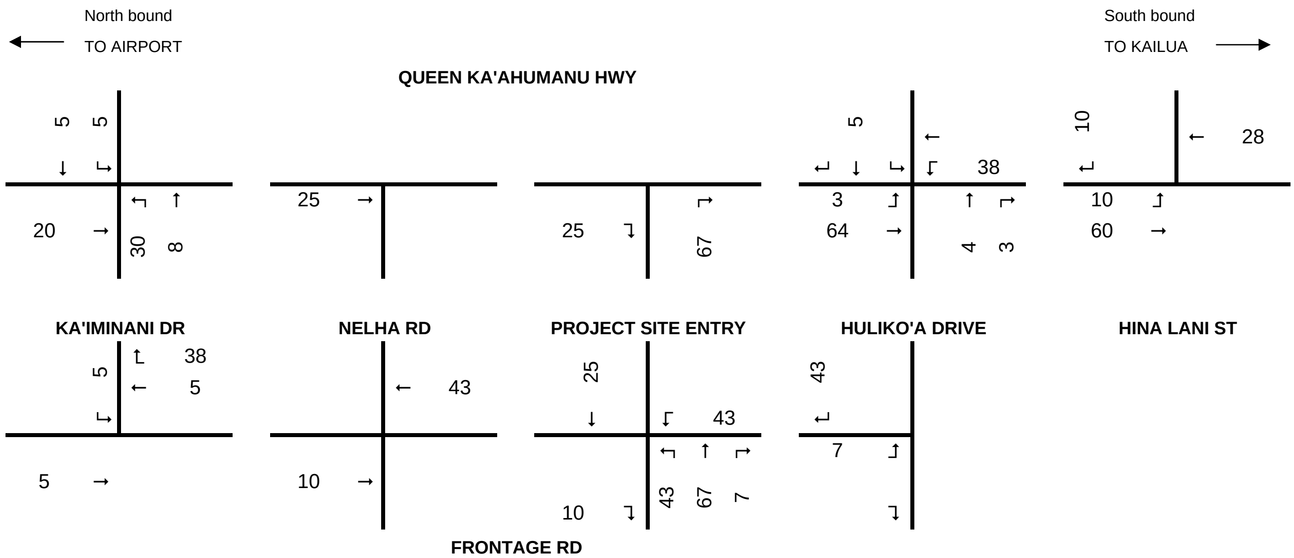


PLANNING YEAR 2029

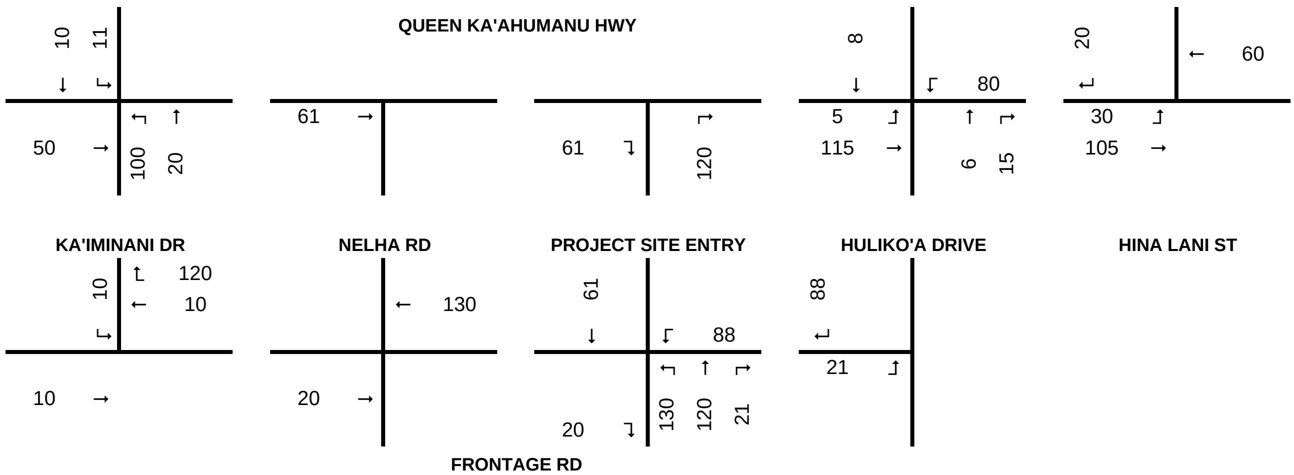
PM PEAK HOUR

Not To Scale

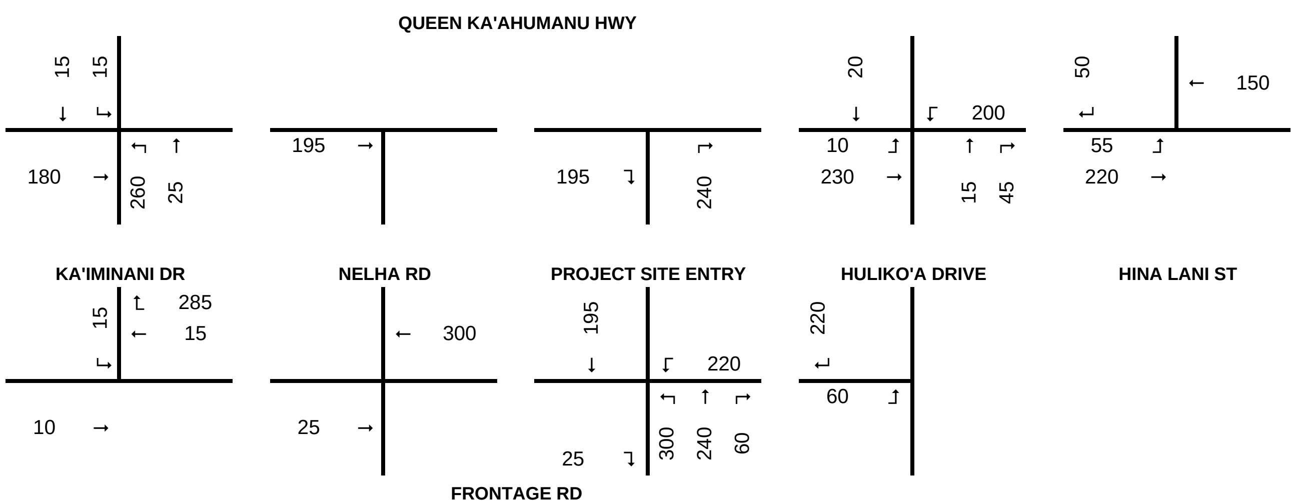
AMBIENT TRAFFIC FORECAST
FIGURE 6



PLANNING YEAR 2015



PLANNING YEAR 2020

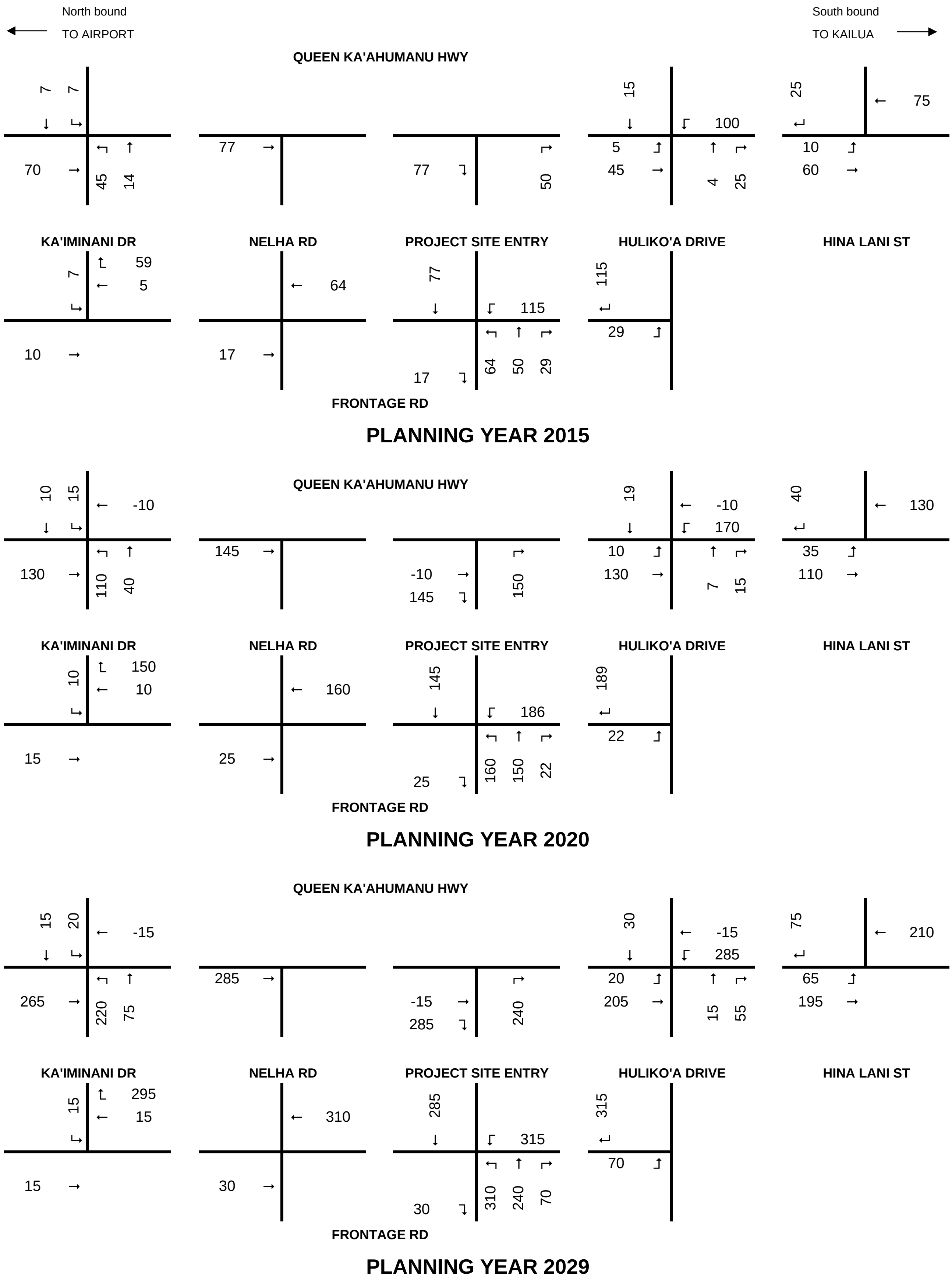


PLANNING YEAR 2029

AM PEAK HOUR

Not To Scale

PROJECT GENERATED TRAFFIC ASSIGNMENT
FIGURE 7



PM PEAK HOUR

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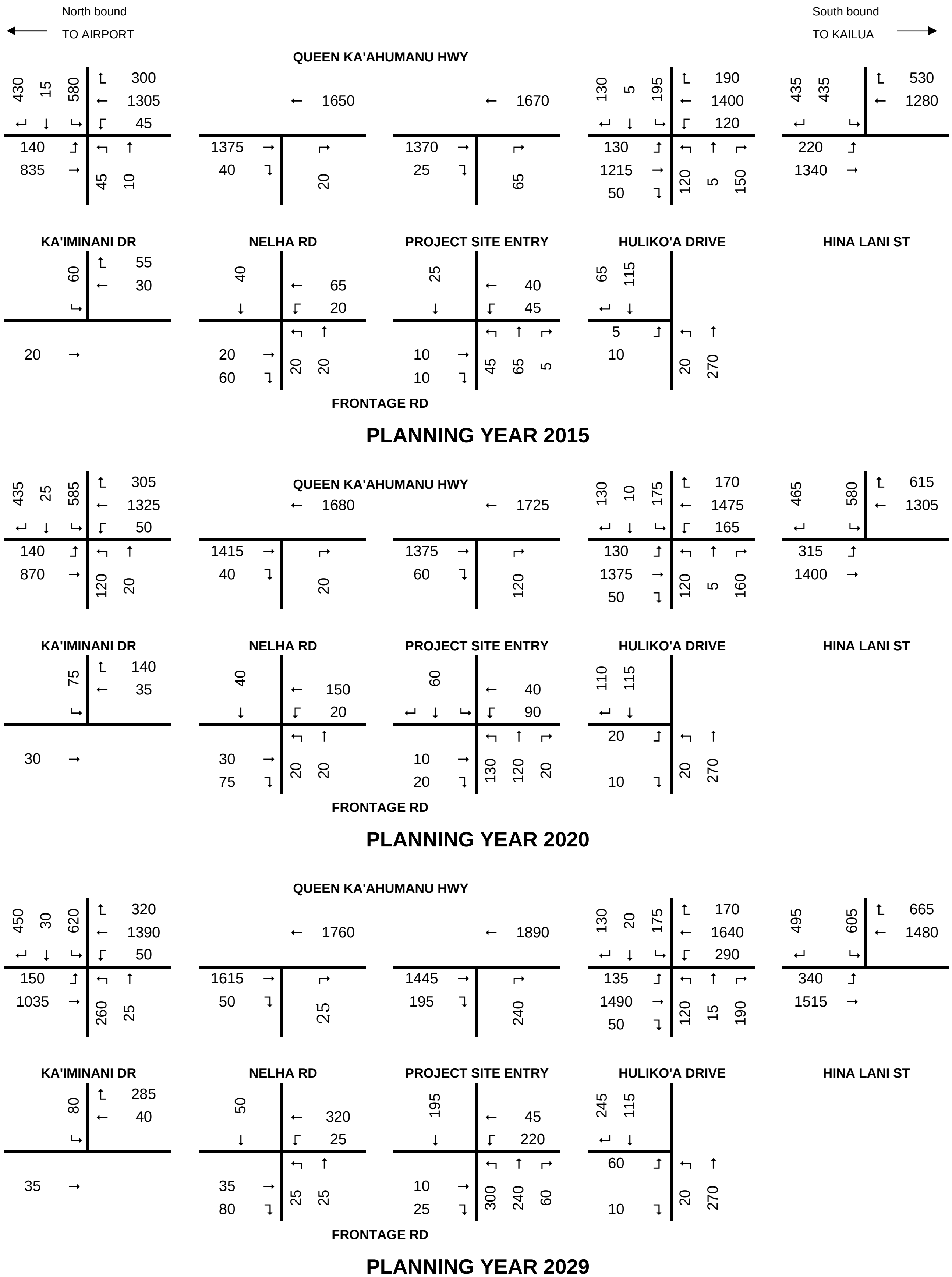
PROJECT GENERATED TRAFFIC ASSIGNMENT
FIGURE 7



AM PEAK HOUR

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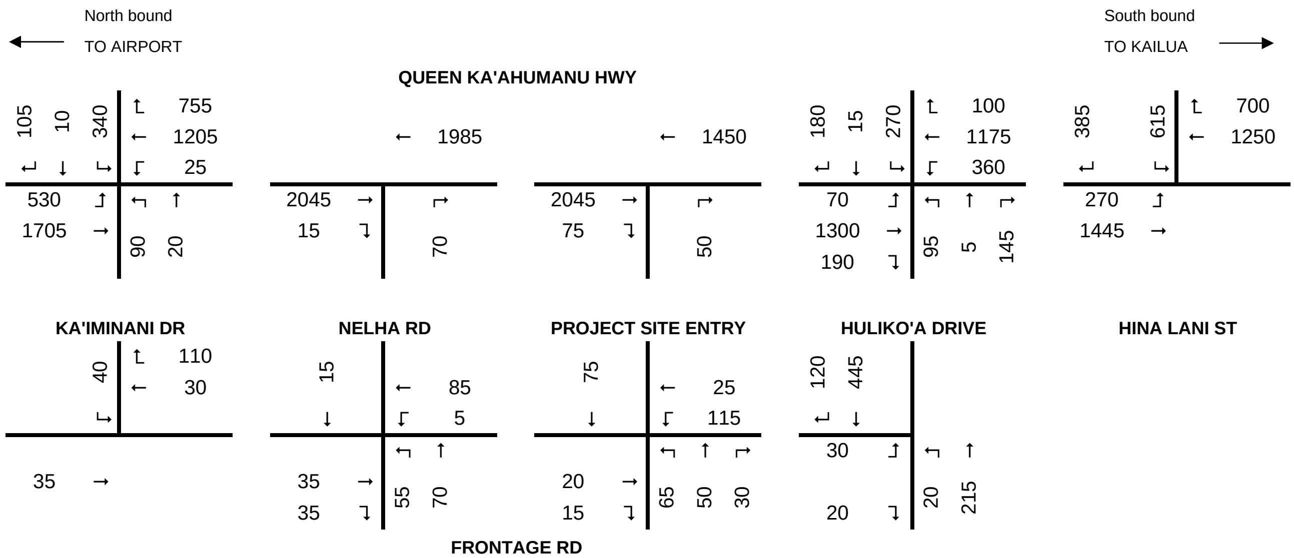
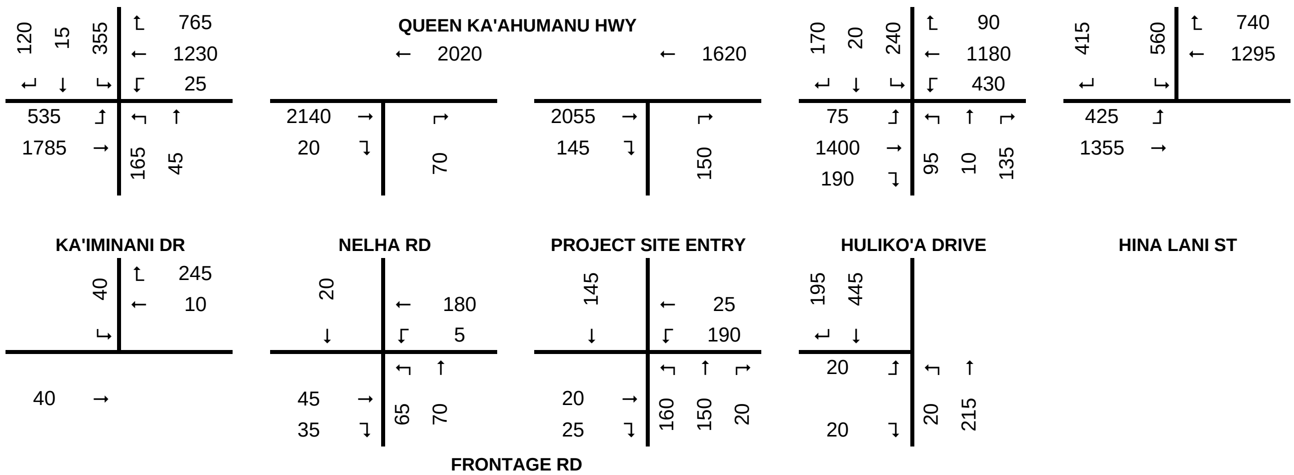
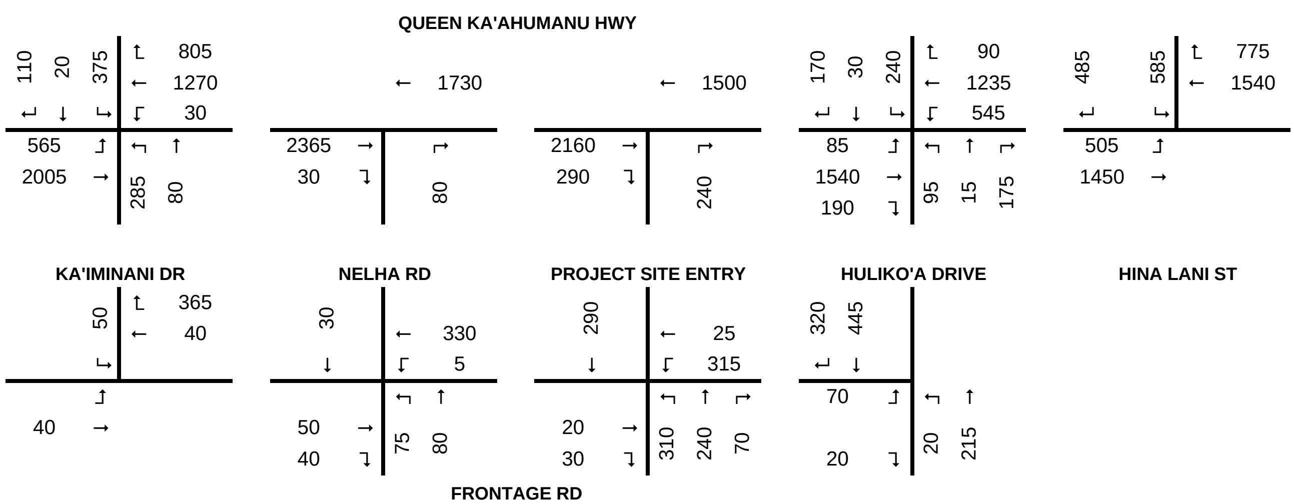
TOTAL WITH PROJECT FORECAST
FIGURE 8



PLANNING YEAR 2015



PLANNING YEAR 2020



PLANNING YEAR 2029

PM PEAK HOUR

Not To Scale

TOTAL WITH PROJECT FORECAST
FIGURE 8

Tables

TABLE 1
PROJECT MILESTONE SCHEDULE

LAND USE	PLANNING YEAR MILESTONE			TG REPORT LAND USE
	2015	2020	2029	
	Cumulative Number of Units			
Single Family DU Residential	120	275	475	SFDU (210)
Multi-family DU Residential	115	355	715	Low-rise Townhome (231)
TOTAL RESIDENTIAL	**235**	**630**	**1,190**	
Makai Village - MU Commercial (sf)	30,000	30,000	30,000	Shopping Center (820)
Restaurant & Canoe Club (sf)	20,000	20,000	20,000	Quality Restaurant (931)
TOTAL COMMERCIAL - Area A (sf)	**50,000**	**50,000**	**50,000**	
Mauka Village - MU&LW Commercial (sf)	0	35,000	135,000	Office Park (750)
Grocery Store (sf)		15000	15,000	Supermarket (850)
TOTAL COMMERCIAL - Area B (sf)	**0**	**50,000**	**150,000**	
Charter School (students)			225	Private School (534)
Public Beach Clubhouse (ac)	1	1	1	Constant assumed

Proposed development schedule assumed for forecasting project generated traffic.
This schedule does not reflect the actual project development schedule.

TABLE 2
TRIP GENERATION ANALYSIS

TIME PERIOD Land Use	Cumulative Units	Trip Generation Equation	Ln(T)	T = Number of Trips	Direction of Travel	Percent	Number of Trips
PLANNING YEAR 2015							
WEEKDAY AM PEAK HOUR							
Single Family Residential	120 units	T = 0.7(x) + 12.05		96	Enter	26%	25
					Leave	74%	71
MF & Mixed Use Vill Residential	115 units	T = 0.88(x) - 49.7		115	Enter	25%	29
					Leave	75%	86
Mixed Use Commercial (Area A)	30 ksf GLA	T = 1.03(X)		31	Enter	61%	19
					Leave	39%	12
Restaurant	20 ksf GLA	T = 0.81(X)		16	Enter	50%	8
					Leave	50%	8
Public Beach Clubhouse					Enter		50
					Leave		10
TOTAL					**Enter**		**131**
					Leave		**187**
WEEKDAY PM PEAK HOUR							
Single Family Residential	120 units	Ln(T)=0.89Ln(X)+0.61	4.87	130	Enter	64%	83
					Leave	36%	47
MF & Mixed Use Vill Residential	115 units	T = 0.78(X)		90	Enter	58%	52
					Leave	42%	38
Mixed Use Commercial (Area A)	30 ksf GLA	T = 3.75(X)		113	Enter	48%	54
					Leave	52%	59
Restaurant	20 ksf GLA	T = 7.49(X)		150	Enter	67%	100
					Leave	33%	49
Public Beach Clubhouse					Enter		20
					Leave		50
TOTAL					**Enter**		**310**
					Leave		**243**

TABLE 2 (continued)
TRIP GENERATION ANALYSIS

TIME PERIOD Land Use	Cumulative Units		Trip Generation Equation	Ln(T)	T = Number of Trips	Direction of Travel	Percent	Number of Trips
PLANNING YEAR 2020								
WEEKDAY AM PEAK HOUR								
Single Family Residential	275	units	T = 0.7(x) + 12.05		205	Enter	26%	53
						Leave	74%	151
MF, M/U, L/W Residential	355	units	T = 0.88(x) - 49.7		263	Enter	25%	66
						Leave	75%	197
Mixed Use Commercial (Area A)	30	ksf GLA	T = 1.03(X)		31	Enter	61%	19
						Leave	39%	12
M/U, L/W Commercial (Area B)	35	ksf GLA	Ln(T)=0.84Ln(X)+1.51	4.50	90	Enter	89%	80
						Leave	11%	10
Grocery Store	15	ksf GLA	Ln(T)=01.70Ln(X)-1.42	3.18	24	Enter	61%	15
						Leave	39%	9
Restaurant	20	ksf GLA	T = 0.81(X)		16	Enter	50%	8
						Leave	50%	8
Public Beach Clubhouse						Enter		60
						Leave		15
TOTAL						**Enter**		**300**
						Leave		**403**
WEEKDAY PM PEAK HOUR								
Single Family Resident	275	units	Ln(T)=0.89Ln(X)+0.61	5.61	273	Enter	64%	175
						Leave	36%	98
MF, M/U, L/W Residential	355	units	T = 0.78(X)		277	Enter	58%	161
						Leave	42%	116
Mixed Use Commercial (Area A)	30	ksf GLA	T = 3.75(X)		113	Enter	48%	54
						Leave	52%	59
M/U, L/W Commercial (Area B)	35	ksf GLA	T = 1.21(x) + 106.22		149	Enter	14%	21
						Leave	86%	128
Grocery Store	15	ksf GLA	Ln(T)=0.79Ln(X)+3.20	5.34	208	Enter	51%	106
						Leave	49%	102
Restaurant	20	ksf GLA	T = 7.49(X)		150	Enter	67%	100
						Leave	33%	49
Public Beach Clubhouse						Enter		25
						Leave		60
TOTAL						**Enter**		**642**
						Leave		**612**

TABLE 2 (continued)
TRIP GENERATION ANALYSIS

TIME PERIOD Land Use	Cumulative Units		Trip Generation Equation	Ln(T)	T = Number of Trips	Direction of Travel	Percent	Number of Trips
PLANNING YEAR 2029								
WEEKDAY AM PEAK HOUR								
Single Family Residential	475	units	T = 0.7(x) + 12.05		345	Enter	26%	90
						Leave	74%	255
MF, M/U, L/W Residential	715	units	T = 0.88(x) - 49.7		580	Enter	25%	145
						Leave	75%	435
Mixed Use Commercial (Area A)	30	ksf GLA	T = 1.03(X)		31	Enter	61%	19
						Leave	39%	12
M/U, L/W Commercial (Area B)	135	ksf GLA	Ln(T)=0.84Ln(X)+1.51	5.63	279	Enter	89%	248
						Leave	11%	31
Grocery Store	15	ksf GLA	Ln(T)=01.70Ln(X)-1.42	3.18	24	Enter	61%	15
						Leave	39%	9
Restaurant	20	ksf GLA	T = 0.81(X)		16	Enter	50%	8
						Leave	50%	8
Charter School (K-8)	225	students	Ln(T)=Ln(X)-0.13	5.29	198	Enter	55%	109
						Leave	45%	89
Public Beach Clubhouse						Enter		70
						Leave		20
TOTAL						**Enter**		**703**
						Leave		**859**

TABLE 2 (continued)
TRIP GENERATION ANALYSIS

TIME PERIOD Land Use	Cumulative Units		Trip Generation Equation	Ln(T)	T = Number of Trips	Direction of Travel	Percent	Number of Trips
PLANNING YEAR 2029								
WEEKDAY PM PEAK HOUR								
Single Family Residential	475	units	Ln(T)=0.89Ln(X)+0.61	6.10	444	Enter	64%	284
						Leave	36%	160
MF, M/U, L/W Residential	715	units	T = 0.78(X)		558	Enter	58%	323
						Leave	42%	234
Mixed Use Commercial (Area A)	30	ksf GLA	T = 3.75(X)		113	Enter	48%	54
						Leave	52%	59
M/U, L/W Commercial (Area B)	135	ksf GLA	T = 1.21(x) + 106.22		270	Enter	14%	38
						Leave	86%	232
Grocery Store	15	ksf GLA	Ln(T)=0.79Ln(X)+3.20	5.34	208	Enter	51%	106
						Leave	49%	102
Restaurant	20	ksf GLA	T = 7.49(X)		150	Enter	67%	100
						Leave	33%	49
Charter School (K-8)	225	students	T = 0.58(x) + 14.03		145	Enter	47%	68
						Leave	53%	77
Public Beach Clubhouse						Enter		25
						Leave		70
TOTAL						**Enter**		**999**
						Leave		**982**

TABLE 3
TRIP DISTRIBUTION ANALYSIS

TIME PERIOD Land Use	Direction of Travel	No. of Trips	NORTH %	NORTH No. of Trips	SOUTH %	SOUTH No. of Trips	INTERNAL %	INTERNAL No. of Trips
PLANNING YEAR 2015								
WEEKDAY AM PEAK HOUR								
Single Family Residential	Enter	25						
	Leave	71						
MF & Mixed Use Vill Residential	Enter	29						
	Leave	86						
COMBINED RESIDENTIAL	Enter	54	17%	9	20%	11	61%	33
	Leave	157	34%	53	41%	64	25%	40
Mixed Use Commercial (Area A)	Enter	19	32%	6	42%	8	26%	5
	Leave	12	25%	3	33%	4	42%	5
Restaurant	Enter	8	25%	2	25%	2	50%	4
	Leave	8	25%	2	25%	2	50%	4
Public Beach Clubhouse	Enter	50	36%	18	44%	22	20%	10
	Leave	10	30%	3	40%	4	30%	3
TOTAL	Enter	131	27%	35	33%	43	40%	52
	Leave	187	33%	61	40%	74	28%	52
WEEKDAY PM PEAK HOUR								
Single Family Residential	Enter	83						
	Leave	47						
MF & Mixed Use Vill Residential	Enter	52						
	Leave	38						
COMBINED RESIDENTIAL	Enter	135	32%	43	39%	52	30%	40
	Leave	85	18%	15	21%	18	61%	52
Mixed Use Commercial (Area A)	Enter	54	33%	18	41%	22	26%	14
	Leave	59	36%	21	42%	25	24%	14
Restaurant	Enter	100	27%	27	33%	33	40%	40
	Leave	49	27%	13	33%	16	41%	20
Public Beach Clubhouse	Enter	20	30%	6	40%	8	30%	6
	Leave	50	30%	15	40%	20	30%	15
TOTAL	Enter	309	30%	94	37%	115	32%	100
	Leave	243	26%	64	33%	79	42%	101

TABLE 3 (continued)
TRIP DISTRIBUTION ANALYSIS

TIME PERIOD Land Use	Direction of Travel	No. of Trips	NORTH %	NORTH No. of Trips	SOUTH %	SOUTH No. of Trips	INTERNAL %	INTERNAL No. of Trips
PLANNING YEAR 2020								
WEEKDAY AM PEAK HOUR								
Single Family Residential	Enter	53						
	Leave	151						
MF, M/U, L/W Residential	Enter	66						
	Leave	197						
COMBINED RESIDENTIAL	Enter	119	38%	45	42%	50	20%	24
	Leave	348	33%	115	36%	125	31%	108
Mixed Use Commercial (Area A)	Enter	19						
	Leave	12						
M/U, L/W Commercial (Area B)	Enter	80						
	Leave	10						
COMBINED COMMERCIAL	Enter	99	10%	10	11%	11	79%	78
	Leave	22	26%	6	29%	6	45%	10
Grocery Store	Enter	15	0%	0	0%	0	100%	15
	Leave	9	0%	0	0%	0	100%	9
Restaurant	Enter	8	48%	4	52%	4	0%	0
	Leave	8	48%	4	52%	4	0%	0
Public Beach Clubhouse	Enter	60	36%	22	39%	23	25%	15
	Leave	15	33%	5	33%	5	33%	5
TOTAL	Enter	301	27%	81	29%	88	44%	132
	Leave	402	32%	130	35%	141	33%	132
WEEKDAY PM PEAK HOUR								
Single Family Resident	Enter	175						
	Leave	98						
MF, M/U, L/W Residential	Enter	161						
	Leave	116						
COMBINED RESIDENTIAL	Enter	336	23%	77	25%	84	52%	174
	Leave	214	24%	51	26%	56	50%	108
Mixed Use Commercial (Area A)	Enter	54						
	Leave	59						
M/U, L/W Commercial (Area B)	Enter	21						
	Leave	128						
COMBINED COMMERCIAL	Enter	75	36%	27	44%	33	20%	15
	Leave	187	32%	60	34%	64	34%	64
Grocery Store	Enter	106	28%	30	31%	33	41%	43
	Leave	102	15%	15	16%	16	69%	70
Restaurant	Enter	100	29%	29	31%	31	40%	40
	Leave	49	29%	14	31%	15	41%	20
Public Beach Clubhouse	Enter	25	29%	7	31%	8	40%	10
	Leave	60	32%	19	35%	21	33%	20
TOTAL	Enter	642	26%	170	29%	189	44%	282
	Leave	612	26%	160	28%	172	46%	282

TABLE 3 (continued)
TRIP DISTRIBUTION ANALYSIS

TIME PERIOD Land Use	Direction of Travel	No. of Trips	NORTH %	NORTH No. of Trips	SOUTH %	SOUTH No. of Trips	INTERNAL %	INTERNAL No. of Trips
PLANNING YEAR 2029								
WEEKDAY AM PEAK HOUR								
Single Family Residential	Enter	90						
	Leave	255						
MF, M/U, L/W Residential	Enter	145						
	Leave	435						
COMBINED RESIDENTIAL	Enter	235	41%	96	41%	96	18%	43
	Leave	690	34%	235	34%	235	32%	220
Mixed Use Commercial (Area A)	Enter	19						
	Leave	12						
M/U, L/W Commercial (Area B)	Enter	248						
	Leave	31						
COMBINED RESIDENTIAL	Enter	267	18%	47	18%	47	65%	174
	Leave	43	33%	14	33%	14	35%	15
Grocery Store	Enter	15	0%	0	0%	0	100%	15
	Leave	9	0%	0	0%	0	100%	9
Restaurant	Enter	8	50%	4	50%	4	0%	0
	Leave	8	50%	4	50%	4	0%	0
Public Beach Clubhouse	Enter	70	36%	25	36%	25	29%	20
	Leave	20	30%	6	30%	6	40%	8
Charter School (K-8)	Enter	109	45%	49	45%	49	10%	11
	Leave	89	44%	39	44%	39	12%	11
TOTAL	Enter	704	31%	221	31%	221	37%	263
	Leave	859	35%	298	35%	298	31%	263
WEEKDAY PM PEAK HOUR								
Single Family Resident	Enter	284						
	Leave	160						
MF, M/U, L/W Residential	Enter	323						
	Leave	234						
COMBINED RESIDENTIAL	Enter	607	30%	179	30%	179	41%	247
	Leave	394	35%	138	35%	138	30%	120
Mixed Use Commercial (Area A)	Enter	54						
	Leave	59						
M/U, L/W Commercial (Area B)	Enter	38						
	Leave	232						
COMBINED COMMERCIAL	Enter	92	39%	36	39%	36	22%	20
	Leave	291	28%	81	28%	81	44%	129
Grocery Store	Enter	106	30%	31	30%	31	41%	43
	Leave	102	16%	16	16%	16	69%	70
Restaurant	Enter	100	30%	30	30%	30	40%	40
	Leave	49	30%	14	30%	15	41%	20
Public Beach Clubhouse	Enter	25	30%	8	30%	7	40%	10
	Leave	70	36%	25	36%	25	29%	20
Charter School (K-8)	Enter	68	45%	30	45%	31	10%	7
	Leave	77	45%	34	45%	35	10%	8
TOTAL	Enter	998	31%	314	31%	314	37%	367
	Leave	983	31%	308	32%	310	37%	367

TABLE 4
LEVEL OF SERVICE ANALYSIS (SIGNALIZED)
QUEEN KA'AHUMANU HIGHWAY AT KAI'MINANI DRIVE

	2006			2015						2020						2029					
	EXISTING			AMBIENT[1]			TOTAL[1]			AMBIENT[2]			TOTAL[2]			AMBIENT[2]			TOTAL[2]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
AM PEAK HOUR	B	17.3	0.79	D	41.8	0.70	D	45.2	0.72	D	40.1	0.7	D	45.2	0.77	D	41.6	0.74	D	41.6	0.95
Frontage Rd Eastbound	NA	-	-	F	86.9	-	F	86.8	0.57	E	78.7	-	F	82.0	-	F	88.7	-	E	71.2	-
Left	-	-	-	F	86.9	0.34	F	87.1	0.57	E	78.7	0.19	F	82.8	0.90	F	88.7	0.46	E	69.9	0.99
Through/Right	-	-	-	F	86.1	0.00	F	85.1	0.1	E	77.8	0	E	77.7	0.16	F	87.6	0	E	77.8	0.55
Ka'iminani Dr WB	C	25.7	-	D	54.8	-	E	57.4	-	E	55.4	-	E	59.3	-	E	59.4	-	E	70.5	-
Left	C	26.8	0.98	E	63.9	0.75	E	67	0.78	E	60.0	0.73	E	64.8	0.80	E	66.5	0.81	E	70.0	1.17
Through	NA	-	-	D	52.6	0.02	E	55.2	0.04	D	49.5	0.04	D	53.4	0.08	D	54.2	0.04	E	78.3	0.64
Right	C	23.8	0.65	D	39.8	0.57	D	41.7	0.58	D	48.2	0.69	D	50.5	0.69	D	47.8	0.67	E	70.0	1.43
Queen Ka'ahumanu Hwy NB	B	14.9	-	D	42.5	-	D	45.6	-	D	35.9	-	D	40.1	-	D	37.4	-	C	28.7	-
Left	NA	-	-	F	86.6	0.74	F	85.2	0.48	F	83.2	0.77	F	84.8	0.62	F	83.5	0.47	E	78.7	0.58
Through	B	16.9	0.83	D	43.1	0.86	D	46.4	0.88	D	36.4	0.81	D	40.9	0.84	D	38.1	0.83	C	28.8	0.76
Right	A	1.2	0.83	C	30.9	0.35	C	33.3	0.36	C	23.9	0.19	C	26.8	0.20	C	24.3	0.2	B	18.4	0.18
Queen Ka'ahumanu Hwy SB	A	8.6	-	C	27.1	-	C	30.7	-	C	31.2	-	C	34.6	-	C	30.3	-	C	25.8	-
Left	A	8.6	0.13	E	71.3	0.56	E	73	0.56	E	78.5	0.6	F	80.2	0.55	E	76.3	0.42	E	74.2	0.54
Through	A	8.6	0.42	B	19.5	0.42	C	23.6	0.45	C	23.2	0.45	C	27.5	0.52	C	22.3	0.45	B	19.1	0.54
Right	NA	-	-	C	25.9	0.00	C	27.9	0.01	C	21.7	0.01	C	24.3	0.01	C	21.9	0.01	B	16.6	0.01

	2006			2015						2020						2029					
	EXISTING			AMBIENT[1]			TOTAL[1]			AMBIENT[2]			TOTAL[2]			AMBIENT[2]			TOTAL[2]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
PM PEAK HOUR	B	11.5	0.67	D	38.6	0.97	D	39.2	1.00	D	42.7	0.71	D	42.0	0.93	D	48.2	0.75	D	51.5	1.00
Frontage Rd Eastbound	NA	-	-	E	73.6	-	E	75.5	-	F	82.2	0.71	F	115	-	F	100	-	F	84.4	-
Left	-	-	-	E	73.8	0.44	E	76	0.89	F	82.5	0.46	F	123	0.92	F	101	0.63	F	82.9	0.79
Through/Right	-	-	-	E	72	0.05	E	72.4	0.13	E	78.2	0.04	F	87.0	0.48	F	87.2	0.05	F	89.8	0.57
Ka'iminani Dr WB	C	26	-	E	62.9	-	E	63	-	E	75.5	-	F	91.8	-	E	78.6	-	F	85.0	-
Left	C	26.3	0.24	E	65.3	0.69	E	65.5	0.71	F	83.4	0.78	F	98.7	0.89	F	86.6	0.75	F	84.3	0.77
Through	NA	-	-	E	58.6	0.02	E	58.8	0.05	E	67.4	0.02	E	76.7	0.10	E	72.0	0.02	F	96.60	0.67
Right	C	25.5	0.12	C	31.1	0.04	C	31.1	0.04	D	40.5	0.15	E	67.3	0.37	D	43.5	0.16	F	85.7	0.54
Queen Ka'ahumanu Hwy NB	B	12	-	D	40.5	-	D	40.5	-	D	36.9	-	C	30.4	-	D	41.6	-	D	36.1	-
Left	NA	-	-	E	75.9	0.37	E	75.9	0.37	F	87.1	0.43	F	83.9	0.34	F	156	0.78	F	90.8	0.42
Through	B	16.4	0.59	D	37.7	0.80	D	37.7	0.8	D	39.0	0.76	D	37.9	0.75	D	42.9	0.78	D	46.2	0.82
Right	A	2.5	0.20	D	44	1.04	D	44	1.04	C	31.0	0.51	B	14.0	0.38	C	34.1	0.54	B	14.9	0.41
Queen Ka'ahumanu Hwy SB	A	9.2	-	C	32.1	-	C	32.5	-	D	40.5	-	D	35.2	-	D	46.4	-	D	52.6	-
Left	A	8.9	0.67	E	65	0.20	E	65	1.61	F	93.7	0.93	E	76.3	0.82	F	114	0.99	F	91.1	0.9
Through	A	9.3	0.62	C	21.4	0.79	C	22.4	0.82	C	23.3	0.77	C	22.9	0.81	C	24.6	0.79	D	41.9	0.95
Right	NA	-	-	C	23.8	0.01	C	23.8	0.01	C	22.9	0.01	A	3.0	0.01	C	24.7	0.01	A	1.2	0.01

[1] With 2 left turn lanes on westbound approach

[2] With 2 northbound right turn, 2 southbound left turn lanes and 2 westbound left turn lanes

TABLE 5
LEVEL OF SERVICE ANALYSIS (SIGNALIZED)
QUEEN KA'AHUMANU HIGHWAY AT HULIKO'A DRIVE

	2006			2015						2020						2029					
	EXISTING			AMBIENT			TOTAL[1]			AMBIENT			TOTAL[1]			AMBIENT[1]			TOTAL[1]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
AM PEAK HOUR				C	29.2	0.74	C	29.7	0.71	C	29.6	0.72	D	37.0	0.72	C	30.2	0.74	D	46.4	0.77
Frontage Road Eastbound				D	47.7	-	D	47.7	-	D	47.7	-	D	45.0	-	D	47.7	-	E	59.7	-
Left				D	48.3	0.44	D	48.3	0.44	D	48.3	0.44	D	45.1	0.36	D	48.3	0.44	E	59.3	0.40
Through				D	43.2	0.01	D	43.3	0.01	D	43.2	0.01	D	40.8	0.01	D	43.2	0.01	D	54.0	0.04
Right				D	47.1	0.38	D	47.2	0.38	D	47.1	0.38	D	45.0	0.36	D	47.1	0.38	E	60.6	0.45
Huliko'a Drive Westbound				D	54.6	-	D	54.7	-	D	51.6	-	D	46.6	-	D	51.6	-	E	61.5	-
Left				E	59.0	0.72	E	59.3	0.72	D	54.8	0.64	D	48.6	0.52	D	54.8	0.64	E	64.6	0.58
Through				D	43.3	0.01	D	43.3	0.01	D	43.3	0.04	D	41.2	0.04	D	43.3	0.01	D	54.1	0.05
Right				D	46.5	0.33	D	46.5	0.33	D	46.5	0.33	D	43.9	0.26	D	46.5	0.33	E	57.5	0.29
Queen Ka'ahumanu Hwy NB				C	25.0	-	C	25.7	-	C	26.4	-	D	36.8	-	C	27.7	-	D	38.9	-
Left				E	60.5	0.48	E	58.0	0.36	E	60.5	0.48	E	62.7	0.55	E	57.4	0.26	E	72.2	0.55
Through				C	24.1	0.75	C	24.1	0.75	C	25.7	0.79	C	34.5	0.85	C	27.2	0.82	C	34.8	0.85
Right				B	14.4	0.19	B	14.4	0.19	B	14.2	0.17	B	18.5	0.18	B	14.2	0.17	B	17.2	0.16
Queen Ka'ahumanu Hwy SB				C	25.3	-	C	26.0	-	C	26.1	-	C	34.2	-	C	26.1	-	D	50.7	-
Left				E	73.2	0.72	E	74.9	0.74	E	73.2	0.72	E	67.4	0.63	E	73.2	0.72	F	98.3	0.77
Through				C	20.5	0.62	C	21.3	0.65	C	21.9	0.68	C	31.4	0.80	C	21.9	0.68	D	47.2	0.89
Right				B	13.2	0.05	B	13.2	0.05	B	13.2	0.05	B	17.2	0.05	B	13.2	0.05	C	22.9	0.05

	2006			2015						2020						2029					
	EXISTING			AMBIENT			TOTAL[1]			AMBIENT			TOTAL[1]			AMBIENT[1]			TOTAL[2]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
PM PEAK HOUR				D	45.3	0.80	D	43.6	0.77	D	41.6	0.78	D	50.3	0.79	D	36.1	0.73	D	45.7	0.85
Frontage Road Eastbound				D	53.4	-	D	53.5	-	D	51.3	-	E	63.7	-	D	45.5	-	E	75.9	-
Left				D	53.6	0.27	D	53.3	0.28	D	51.6	0.28	E	64.3	0.29	D	46.1	0.27	F	105.0	0.74
Through				D	49.7	0.01	D	49.4	0.01	D	47.8	0.01	E	59.7	0.01	D	42.7	0.01	F	86.5	0.17
Right				D	53.2	0.24	D	53.7	0.31	D	51.1	0.25	E	63.2	0.23	D	44.8	0.17	D	52.2	0.29
Huliko'a Drive Westbound				E	67.6	-	E	67.7	-	E	61.8	-	E	75.6	-	D	54.2	-	E	77.0	-
Left				E	74.2	0.79	E	75.6	0.81	E	66.9	0.74	F	82.1	0.75	E	58.1	0.69	F	82.0	0.62
Through				D	49.8	0.01	D	49.8	0.03	D	47.8	0.01	E	60.3	0.05	D	42.7	0.01	E	78.9	0.18
Right				E	55.7	0.39	E	55.3	0.40	D	53.2	0.38	E	65.1	0.33	D	46.7	0.31	E	66.5	0.42
Queen Ka'ahumanu Hwy NB				D	35.1	-	C	34.4	-	D	35.8	-	D	44.2	-	C	32.4	-	C	33.7	-
Left				F	87.1	0.83	E	72.2	0.65	F	113	0.96	F	97.8	0.84	E	78.1	0.75	E	77.6	0.8
Through				C	24.8	0.61	C	24	0.61	C	20.1	0.59	C	26.1	0.59	C	23.8	0.66	B	16.0	0.56
Right				B	16.3	0.09	B	15.8	0.08	B	13.1	0.07	B	17.2	0.07	B	14.8	0.07	A	2.8	0.05
Queen Ka'ahumanu Hwy SB				D	48.4	-	D	44.9	-	D	41.1	-	D	49.2	-	C	34.6	-	D	47.4	-
Left				F	84.7	0.50	F	83.7	0.53	F	100	0.71	F	99.3	0.56	E	77.8	0.55	F	97.6	0.67
Through				D	48.8	0.84	D	44.9	0.83	D	40.0	0.8	D	48.6	0.84	C	33.9	0.79	D	47.6	0.9
Right				C	30.8	0.24	C	27.9	0.23	C	25.6	0.23	C	28.4	0.19	C	20.6	0.18	B	14.8	0.15

[1] With 2 left turn lanes on northbound approach

[2] With 2 left turn lanes on westbound approach

TABLE 6
LEVEL OF SERVICE ANALYSIS (SIGNALIZED)
QUEEN KA'AHUMANU HIGHWAY AT HINA LANI STREET

	2006			2015						2020						2029					
	EXISTING			AMBIENT			TOTAL			AMBIENT[1]			TOTAL[1]			AMBIENT[1]			TOTAL[2]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
AM PEAK HOUR	C	27.5	0.75	D	36.9	0.92	D	36.9	0.93	C	33.2	1.04	D	38.4	1.06	D	35.5	1.09	C	30.9	1.07
Hina Lani St WB	C	29.5	-	D	38.6	-	D	41.1	-	D	46.1	-	D	52.1	-	D	47.4	-	D	52.1	-
Left	C	29.0	0.47	D	50.0	0.77	D	53.5	0.8	E	57.6	0.74	E	65.9	0.75	E	59.2	0.78	E	62.8	0.84
Right	C	30.0	0.52	C	24.1	0.47	C	25.9	0.49	C	27.7	0.52	C	31.5	0.54	C	27.7	0.52	D	36.5	0.67
Queen Ka'ahumanu Hwy NB	D	37.3	-	D	41.2	-	D	39.6	-	D	36.6	-	D	43.3	-	D	40.7	-	C	33.9	-
Through	D	43.2	0.93	D	54.0	0.93	D	51.7	0.93	D	47.7	0.89	E	56.1	0.92	D	53.5	0.94	D	44.0	0.93
Right	B	18.5	0.34	A	3.9	0.37	A	3.9	0.37	A	9.1	0.459	B	10.8	0.50	A	9.5	0.52	A	6.6	0.50
Queen Ka'ahumanu Hwy SB	B	13.0	-	C	31.1	-	C	31.7	-	C	21.8	-	C	25.5	-	C	21.8	-	B	15.9	-
Left	B	13.0	0.38	E	71.1	0.85	E	78.9	0.89	E	56.1	0.76	E	67.5	0.81	E	56.5	0.76	C	23.6	0.21
Through	B	13.0	0.55	C	24.5	0.70	C	23.9	0.71	B	14.2	0.60	B	16.1	0.64	B	14.2	0.60	B	14.2	0.68

	2006			2015						2020						2029					
	EXISTING			AMBIENT			TOTAL			AMBIENT[1]			TOTAL[1]			AMBIENT[1]			TOTAL[2]		
APPROACH & MOVEMENTS	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**	**LOS**	**Delay**	**V/C**
PM PEAK HOUR	C	27.9	0.69	D	49.3	0.60	D	53.5	1.11	C	33.8	0.74	D	39.7	0.76	D	41.3	0.77	C	33.0	1.16
Hina Lani St WB	D	42.6	-	E	76.4	-	E	75.1	-	D	52.7	-	D	54.6	-	D	54.8	-	E	60.8	-
Left	D	44.1	0.74	F	100.8	1.04	F	100.8	1.04	E	67.2	0.83	E	72.1	0.85	E	70.9	0.87	E	66.5	0.72
Right	D	40.8	0.66	C	22.7	0.36	C	23.3	0.39	C	25.2	0.41	C	25.6	0.46	C	26.3	0.46	D	52.7	0.72
Queen Ka'ahumanu Hwy NB	C	33.6	-	D	41.6	-	D	49.7	-	C	33.7	-	D	46.2	-	D	42.9	-	C	30.3	-
Through	D	36.8	0.82	E	59.7	0.92	E	70.5	0.98	D	44.0	0.83	E	60.2	0.96	E	56.4	0.95	D	41.4	0.87
Right	C	28.8	0.64	A	6.2	0.52	A	6.2	0.52	B	15.0	0.65	B	17.7	0.67	B	16.2	0.69	A	4.8	0.56
Queen Ka'ahumanu Hwy SB	B	11.9	-	D	42.9	-	D	46.0	-	C	24.0	-	C	25.2	-	C	32.3	-	C	21.8	-
Left	B	11.3	0.36	F	107.3	0.98	F	118.6	1.02	E	65.2	0.88	E	68.8	0.89	F	92.8	0.99	C	31.1	0.34
Through	B	12	0.60	C	30.8	0.77	C	32.4	0.80	B	11.1	0.55	B	11.5	0.59	B	11.2	0.55	B	18.6	0.67

[1] With 2 left turn lanes on westbound approach

[2] With 2 southbound left turn lanes and 2 westbound left turn lanes

TABLE 7
LEVEL OF SERVICE ANALYSIS (UNSIGNALIZED)
QUEEN KA'AHUMANU HIGHWAY AT EXISTING (2006) INTERSECTIONS

			AM PEAK HOUR		PM PEAK HOUR	
			LOS	DELAY	LOS	DELAY
NELHA ACCESS RD INTERSECTION						
	NELHA Access Rd	EB Approach	D	34.9	D	35
		EB RT	C	17.6	B	14.4
		EB LT	F	64.2	E	47.3
	Queen Ka'ahumanu Hwy	NB LT	B	10.6	A	9.1
HULIKOA DRIVE INTERSECTION						
	Hulikoa Drive	WB Approach	F	107.3	F	104
		WB RT	C	21.1	C	19
		WB LT	F	237	F	161
	Queen Ka'ahumanu Hwy	SB LT	B	10.9	A	9.8

TABLE 8
LEVEL OF SERVICE ANALYSIS (ON-RAMP)
QUEEN KA'AHUMANU HIGHWAY AT
'O'OMA BEACHSIDE VILLAGE AND NELHA ACCESS ROADS

PEAK	2015		2020		2029	
HOUR	LOS	DENSITY	LOS	DENSITY	LOS	DENSITY
At 'O'oma Beachside Village Access Road						
AM	B	17.3	B	17.9	B	19.5
PM	C	23.2	C	24.1	C	25.8
At NELHA Access Road						
AM	B	16.9	B	17.3	B	19.2
PM	C	23.3	C	24.2	C	26.3

Legend:
LOS = Level of Service for vehicles entering Queen Ka'ahumanu Highway from access road
DENSITY = Passenger Cars/Mile/Lane

TABLE 9
LEVEL OF SERVICE ANALYSIS (HIGHWAY)
QUEEN KA'AHUMANU HIGHWAY SOUTHBOUND AT
'O'OMA BEACHSIDE VILLAGE ACCESS ROAD

		AM PEAK HOUR			PM PEAK HOUR		
2-LANE HIGHWAY ANALYSIS							
		LOS	**% PASS**	**ATS**	**LOS**	**% PASS**	**ATS**
2006 Existing		E	91.4	46	E	91.1	47.2
		LOS	**DENSITY**	**VOLUME**	**LOS**	**DENSITY**	**VOLUME**
MULTI-LANE HIGHWAY ANALYSIS							
2015	Ambient	B	14.24	783	C	21.26	1,169
2015	Total	B	14.5	797	C	22.04	1,212
2020	Ambient	B	14.24	783	C	21.46	1,180
2020	Total	B	14.92	820	C	22.87	1,258
2029	Ambient	B	15.02	826	C	22.61	1,243
2029	Total	B	17.05	938	C	25.47	1,401

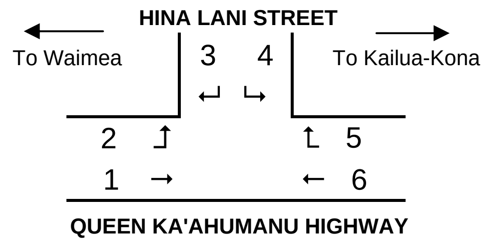
Legend:
LOS = Level of Service
% PASS = Percent Time Spent Following
ATS = Average Travel Speed (mi/hr)
DENSITY = Passenger Cars/Mile/Lane
VOLUME = Hourly Passenger Cars/Hour/Lane

Appendix A

Traffic Turning Movement Counts

TRAFFIC TURNING MOVEMENT COUNT
O'OMA TIAR

	Queen K'aahumanu Highway/
LOCATION:	Hina Lani Street
DATE:	September 14, 2006
TIME:	6:30a-8:30a / 11:00a-1:00p / 3:30p-5:30p
WEATHER:	Clear
RECORDER:	C. Darby

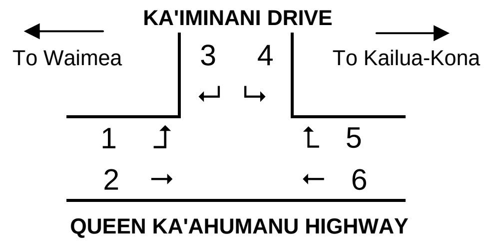


TIME PERIOD	MOVEMENT NUMBER 1	2	3	4	5	6	TOTAL
6:30-6:45a	120	17	79	58	50	207	531
6:45-7:00a	130	35	68	72	90	177	572
7:00-7:15a	134	24	49	44	70	154	475
7:15-7:30a	162	42	64	53	86	147	554
7:30-7:45a	184	36	65	62	106	180	633
7:45-8:00a	171	34	88	82	72	133	580
8:00-8:15a	182	33	63	66	88	170	602
8:15-8:30a	186	35	61	74	82	156	594
6:30-8:30a	1269	256	537	511	644	1324	4541
7:30-8:30a	723	138	277	284	348	639	2409
PHF	0.98				0.86		
11:00-11:15a	149	33	47	109	116	139	593
11:15-11:30a	173	49	59	97	126	138	642
11:30-11:45a	147	43	64	89	94	105	542
11:45-12:00n	174	45	65	107	121	124	636
12:00n-12:15p	130	31	58	91	113	133	556
12:15-12:30p	109	32	58	110	104	113	526
12:30-12:45p	144	28	58	85	123	147	585
12:45-1:00p	145	15	67	96	141	136	600
11:00a-1:00p	1171	276	476	784	938	1035	4680
11:00a-12:00p	643	170	235	402	457	506	2413
3:30-3:45p	150	33	65	64	118	141	571
3:45-4:00p	193	60	90	89	138	155	725
4:00-4:15p	210	52	89	106	128	175	760
4:15-4:30p	95	31	36	42	61	79	344
4:30-4:45p	150	30	63	57	114	141	555
4:45-5:00p	137	36	63	82	119	146	583
5:00-5:15p	122	26	58	73	65	151	495
5:15-5:30p	80	14	84	50	63	110	401
3:30-5:30p	1137	282	548	563	806	1098	4434
3:30-4:30p	648	176	280	301	445	550	2400
PHF	0.79				0.82		

Traffic accident from 5:15-5:30 pm, affected movements 1 & 6
Long traffic queues on movements 1 & 4 from 3:35 to 4:10 pm

TRAFFIC TURNING MOVEMENT COUNT
O'OMA TIAR

LOCATION: Queen Ka'ahumanu Highway/ Ka'iminani Drive
DATE: September 12, 2006
TIME: 6:30a-8:30a / 11:00a-1:00p / 3:30p-5:30p
WEATHER: Clear
RECORDER: C. Darby, R. Miguel



TIME PERIOD	MOVEMENT NUMBER 1	2	3	4	5	6	TOTAL
6:30-6:45a	7	54	92	86	29	184	452
6:45-7:00a	7	89	83	95	36	180	490
7:00-7:15a	13	92	84	114	37	181	521
7:15-7:30a	9	113	73	96	22	152	465
7:30-7:45a	6	124	46	130	26	162	494
7:45-8:00a	6	100	62	126	34	144	472
8:00-8:15a	7	129	37	89	37	135	434
8:15-8:30a	12	139	44	74	23	176	468
6:30-8:30a	67	840	521	810	244	1314	3796
6:45-7:45a	35	418	286	435	121	675	1970
PHF	0.87				0.91		
11:00-11:15a	13	141	21	42	34	141	392
11:15-11:30a	16	147	27	39	35	117	381
11:30-11:45a	13	157	13	26	22	123	354
11:45-12:00n	12	124	20	35	33	143	367
12:00n-12:15p	26	154	16	39	37	141	413
12:15-12:30p	12	130	11	17	35	126	331
12:30-12:45p	9	130	25	32	32	125	353
12:45-1:00p	29	136	17	28	41	143	394
11:00a-1:00p	130	1119	150	258	269	1059	2985
11:15a-12:15p	67	582	76	139	127	524	1515
3:30-3:45p	49	133	15	33	59	122	411
3:45-4:00p	102	171	13	21	69	128	504
4:00-4:15p	99	197	10	21	70	101	498
4:15-4:30p	64	153	19	23	73	115	447
4:30-4:45p	48	155	14	24	69	133	443
4:45-5:00p	44	115	13	25	80	134	411
5:00-5:15p	52	147	13	17	72	122	423
5:15-5:30p	51	117	12	33	92	134	439
3:30-5:30p	509	1188	109	197	584	989	3576
3:45-4:45p	313	676	56	89	281	477	1892
PHF	0.84				0.96		

Appendix B

Signalized Intersection Level of Service (LOS) Calculations

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: EX AM #1 2006
Comment: 2006 EXISTING 6:30-7:30 AM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other | Analysis period: .25 h | Signal type: Actuated-Field | % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)				435		285		675	120	35	420	
RTOR volume (veh/h)						30			20			0
Peak-hour factor				.92		.92		.92	.92	.92	.92	
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB	R		TR					
SB		LT	LT					
Green (s)	20	4	35					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s): 73.9 | Lost time per cycle (s): 9.8 | Critical v/c Ratio: .787

Intersection Performance

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				473		277		734	109	38	457	
Capacity (veh/h)				479		429		882	1303	301	1084	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.987		.647		.832	.083	.126	.421	
g/C ratio				.271		.271		.474	.823	.606	.582	
Average back of queue (veh)				14.1		5.7		15.6	.5	.4	5.7	
Uniform delay (s)				26.8		23.8		16.9	1.2	8.6	8.6	
Incremental delay (s)												
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				26.8		23.8		16.9	1.2	8.6	8.6	
LOS				C		C		B	A	A	A	
Approach delay (s)/LOS		/		25.7	/	C	14.9	/	B	8.6	/	A
Intersection delay (s)/ LOS				17.3			/			B		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: EX AM #2 2006
Comment: 2006 EXISTING 7:30-8:30 AM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other | Analysis period: .25 h | Signal type: Actuated-Field | % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)				420		190		615	120	30	490	
RTOR volume (veh/h)						30			20			0
Peak-hour factor				.92		.92		.92	.92	.92	.92	
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB	R		TR					
SB		LT	LT					
Green (s)	20	4	35					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s): 73.9 | Lost time per cycle (s): 9.8 | Critical v/c Ratio: .732

Intersection Performance

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				457		174		668	109	33	533	
Capacity (veh/h)				479		429		882	1303	328	1084	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.953		.406		.758	.083	.1	.491	
g/C ratio				.271		.271		.474	.823	.606	.582	
Average back of queue (veh)				12.8		3.2		12.9	.5	.3	7	
Uniform delay (s)				26.5		22.1		16	1.2	8.2	9	
Incremental delay (s)												
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				26.5		22.1		16	1.2	8.2	9	
LOS				C		C		B	A	A	A	
Approach delay (s)/LOS		/		25.3	/	C	13.9	/	B	9	/	A
Intersection delay (s)/ LOS				16.1			/			B		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	3/26/2008
Agency or Company		EB/WB Street	KAIMINANI
Analysis Period/Year	AMB AM 2015	NB/SB Street	QUEEN KAAH
Comment	2015 AMBIENT AM		

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	15	0	0	575	10	430	46	1307	300	140	815	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	5	44	7	20	85			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s) 182 Lost time per cycle (s) 5.8 Critical v/c Ratio .7

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	1	1	2	1
Flow rate (veh/h)	16	0		625	11	380	50	1421	261	152	886	3
Capacity (veh/h)	49	52		831	450	671	68	1656	739	272	2124	739
Adjusted saturation flow (veh/h)	1770	1885		3437	1863	1583	1770	3547	1583	1770	3547	1583
v/c ratio	.335	0		.752	.024	.567	.735	.858	.353	.559	.417	.004
g/C ratio	.027	.027		.242	.242	.424	.038	.467	.467	.154	.599	.467
Average back of queue (veh)	.9			16.8	.4	15.6	3	38.1	8.9	7.7	13.4	.1
Uniform delay (s)	86.9	86.1		63.9	52.6	39.8	86.6	43.1	30.9	71.3	19.5	25.9
Incremental delay (s)												
Initial queue delay (s)	0			0	0	0	0	0	0	0	0	0
Delay (s)	86.9	86.1		63.9	52.6	39.8	86.6	43.1	30.9	71.3	19.5	25.9
LOS	F	F		E	D	D	F	D	C	E	B	C
Approach delay (s)/LOS	86.9	/	F	54.8	/	D	42.5	/	D	27.1	/	C
Intersection delay (s)/ LOS			41.8				/			D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	3/26/2008
Agency or Company		EB/WB Street	KAIMINANI
Analysis Period/Year	TOT AM 2015	NB/SB Street	QUEEN KAAH
Comment	2015 TOTAL AM		

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	45	8	0	580	15	430	46	1307	300	140	835	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	9	44	11	17	85			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s) 187 Lost time per cycle (s) 5.8 Critical v/c Ratio .72

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	1	1	2	1
Flow rate (veh/h)	49	9		630	16	380	50	1421	261	152	908	3
Capacity (veh/h)	85	89		809	438	661	104	1612	720	274	2010	720
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	1583	1770	3547	1583
v/c ratio	.574	.098		.78	.037	.575	.48	.881	.362	.555	.451	.005
g/C ratio	.048	.048		.235	.235	.418	.059	.455	.455	.155	.567	.455
Average back of queue (veh)	2.8	.5		17.7	.7	16.2	2.8	40.4	9.3	7.9	15.3	.1
Uniform delay (s)	87.1	85.1		67	55.2	41.7	85.2	46.4	33.3	73	23.6	27.9
Incremental delay (s)												
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	87.1	85.1		67	55.2	41.7	85.2	46.4	33.3	73	23.6	27.9
LOS	F	F		E	E	D	F	D	C	E	C	C
Approach delay (s)/LOS	86.8	/	F	57.4	/	E	45.6	/	D	30.7	/	C
Intersection delay (s)/ LOS			45.2				/			D		

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: AMB AM 2 2020
Comment: 2020 AMBIENT AM W/3 DOUBLE TURNS

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	18	0	0	577	15	435	50	1324	305	140	820	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	10	44	7	5	88			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 175 Lost time per cycle (s): 10.9 Critical v/c Ratio: .704

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	20	0		627	16	386	54	1439	266	152	891	3
Capacity (veh/h)	101	108		864	468	562	71	1783	1409	255	1966	796
Adjusted saturation flow (veh/h)	1770	1885		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.193	0		.726	.035	.687	.768	.807	.189	.596	.453	.004
g/C ratio	.057	.057		.251	.251	.355	.04	.503	.503	.074	.554	.503
Average back of queue (veh)	1			15.9	.6	17.5	3.2	34.3	4.2	4.1	14.4	.1
Uniform delay (s)	78.7	77.8		60	49.5	48.2	83.2	36.4	23.9	78.5	23.2	21.7
Incremental delay (s)												
Initial queue delay (s)	0			0	0	0	0	0	0	0	0	0
Delay (s)	78.7	77.8		60	49.5	48.2	83.2	36.4	23.9	78.5	23.2	21.7
LOS	E	E		E	D	D	F	D	C	E	C	C
Approach delay (s)/LOS	78.7	/	E	55.4	/	E	35.9	/	D	31.2	/	C
Intersection delay (s)/ LOS		40.1				/				D		

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: TOT AM 2 2020
Comment: 2020 TOTAL AM W/3 DOUBLE TURNS

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	121	23	0	610	35	435	50	1324	305	142	910	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	15	44	9	5	88			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 182 Lost time per cycle (s): 10.9 Critical v/c Ratio: .77

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	132	25		663	38	386	54	1439	266	154	989	3
Capacity (veh/h)	146	152		831	450	558	88	1715	1355	283	1890	766
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.902	.164		.798	.084	.692	.621	.839	.197	.545	.523	.004
g/C ratio	.082	.082		.242	.242	.352	.049	.484	.484	.082	.533	.484
Average back of queue (veh)	8.2	1.2		18.3	1.5	18.3	3.1	37.4	4.6	4.2	18.1	.1
Uniform delay (s)	82.8	77.7		64.8	53.4	50.5	84.8	40.9	26.8	80.2	27.5	24.3
Incremental delay (s)												
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	82.8	77.7		64.8	53.4	50.5	84.8	40.9	26.8	80.2	27.5	24.3
LOS	F	E		E	D	D	F	D	C	F	C	C
Approach delay (s)/LOS	82	/	F	59.3	/	E	40.1	/	D	34.6	/	C
Intersection delay (s)/ LOS		45.2				/				D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: AMB AM 2 2029
Comment: 2029 AMB AM W/ 3 DOUBLE TURNS

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	20	0	0	607	15	450	50	1390	320	150	855	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	5	44	12	8	95			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 185 Lost time per cycle (s): 10.9 Critical v/c Ratio: .736

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	22	0		660	16	402	54	1511	283	163	929	3
Capacity (veh/h)	48	51		817	443	600	115	1821	1439	390	2051	813
Adjusted saturation flow (veh/h)	1770	1885		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.455	0		.807	.037	.67	.473	.83	.196	.418	.453	.004
g/C ratio	.027	.027		.238	.238	.379	.065	.514	.514	.114	.578	.514
Average back of queue (veh)	1.2			18.6	.7	18.7	2.9	38.7	4.7	4.3	15.2	.1
Uniform delay (s)	88.7	87.6		66.5	54.2	47.8	83.5	38.1	24.3	76.3	22.3	21.9
Incremental delay (s)												
Initial queue delay (s)	0			0	0	0	0	0	0	0	0	0
Delay (s)	88.7	87.6		66.5	54.2	47.8	83.5	38.1	24.3	76.3	22.3	21.9
LOS	F	F		E	D	D	F	D	C	E	C	C
Approach delay (s)/LOS	88.7	/	F	59.4	/	E	37.4	/	D	30.3	/	C
Intersection delay (s)/ LOS		41.6				/				D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: TOT AM 2 2029
Comment: 2029 TOTAL AM W/3 DOUBLE TURN LANE

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	285	55	0	655	65	450	50	1390	320	150	1075	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	L	TR	R					
WB	L	TR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	30	10	9	5	95			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 170 Lost time per cycle (s): 10.9 Critical v/c Ratio: .948

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	310	60		712	71	402	54	1511	283	163	1168	3
Capacity (veh/h)	312	109		606	110	280	94	1982	1566	303	2170	885
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.992	.55		1.174	.645	1.435	.58	.762	.18	.538	.539	.004
g/C ratio	.176	.059		.176	.059	.177	.053	.559	.559	.088	.612	.559
Average back of queue (veh)	18.9	3.1		26.6	3.7	35.6	2.8	31.8	3.9	4.2	18	.1
Uniform delay (s)	69.9	77.8		70	78.3	70	78.7	28.8	18.4	74.2	19.1	16.6
Incremental delay (s)												
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	69.9	77.8		70	78.3	70	78.7	28.8	18.4	74.2	19.1	16.6
LOS	E	E		E	E	E	E	C	B	E	B	B
Approach delay (s)/LOS	71.2	/	E	70.5	/	E	28.7	/	C	25.8	/	C
Intersection delay (s)/ LOS		41.6				/				D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year EX PM #1 2006
Comment 2006 EXISTING 3:30-4:30 PM

Site Information

Jurisdiction/Date 3/26/2008
EB/WB Street KAIMINANI
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				90		55		475	280	315	675	
RTOR volume (veh/h)						15			60			0
Peak-hour factor				.92		.92		.92	.92	.92	.92	
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB	R		TR					
SB		LT	LT					
Green (s)	20	10	40					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s) 84.9 Lost time per cycle (s) 15.6 Critical v/c Ratio .669

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				98		43		516	239	342	734	
Capacity (veh/h)				417		373		878	1227	508	1185	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.235		.117		.588	.195	.673	.619	
g/C ratio				.236		.236		.471	.775	.657	.636	
Average back of queue (veh)				2		.9		9.8	1.7	5.5	11.6	
Uniform delay (s)				26.3		25.5		16.4	2.5	8.9	9.3	
Incremental delay (s)												
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				26.3		25.5		16.4	2.5	8.9	9.3	
LOS				C		C		B	A	A	A	
Approach delay (s)/LOS		/		26	/	C	12	/	B	9.2	/	A
Intersection delay (s)/ LOS		11.5				/				B		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year EX PM #2 2006
Comment 2006 EXISTING 4:30-5:30 PM

Site Information

Jurisdiction/Date 3/26/2008
EB/WB Street KAIMINANI
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				100		50		525	315	195	535	
RTOR volume (veh/h)						15			60			0
Peak-hour factor				.92		.92		.92	.92	.92	.92	
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB	R		TR					
SB		LT	LT					
Green (s)	20	5	40					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s) 79.9 Lost time per cycle (s) 15.6 Critical v/c Ratio .676

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				109		38		571	277	212	582	
Capacity (veh/h)				443		396		933	1304	419	1142	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.245		.096		.612	.213	.506	.509	
g/C ratio				.25		.25		.501	.824	.636	.613	
Average back of queue (veh)				2.1		.7		10.1	1.5	2.7	8	
Uniform delay (s)				23.9		23		14.4	1.5	8.3	8.7	
Incremental delay (s)												
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				23.9		23		14.4	1.5	8.3	8.7	
LOS				C		C		B	A	A	A	
Approach delay (s)/LOS		/		23.7	/	C	10.2	/	B	8.6	/	A
Intersection delay (s)/ LOS		10.6				/				B		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: AMB PM 2015
Comment: 2015 AMBIENT PM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other | Analysis period: .25 h | Signal type: Actuated-Field | % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	44	5	0	335	5	105	26	1205	755	530	1635	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	10	25	7	25	75			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 163 | Lost time per cycle (s): 5.8 | Critical v/c Ratio: .97

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	1	1	2	1
Flow rate (veh/h)	48	5		364	5	27	28	1310	755	576	1777	3
Capacity (veh/h)	109	113		527	286	613	76	1632	729	358	2263	729
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	1583	1770	3547	1583
v/c ratio	.441	.048		.691	.019	.044	.372	.803	1.037	1.608	.785	.004
g/C ratio	.061	.061		.153	.153	.387	.043	.46	.46	.202	.638	.46
Average back of queue (veh)	2.3	.2		9	.2	.8	1.4	29.8	44.8	54.7	34.3	.1
Uniform delay (s)	73.8	72		65.3	58.6	31.1	75.9	37.7	44	65	21.4	23.8
Incremental delay (s)												
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	73.8	72		65.3	58.6	31.1	75.9	37.7	44	65	21.4	23.8
LOS	E	E		E	E	C	E	D	D	E	C	C
Approach delay (s)/LOS	73.6	/	E	62.9	/	E	40.5	/	D	32.1	/	C
Intersection delay (s)/ LOS		38.6					/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: TOT PM 2015
Comment: 2015 TOTAL PM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other | Analysis period: .25 h | Signal type: Actuated-Field | % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	89	14	0	342	12	105	26	1205	755	530	1705	3
RTOR volume (veh/h)			0			80			60			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	10	25	7	25	75			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 163 | Lost time per cycle (s): 5.8 | Critical v/c Ratio: 1.001

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	1	1	2	1
Flow rate (veh/h)	97	15		372	13	27	28	1310	755	576	1853	3
Capacity (veh/h)	109	113		527	286	613	76	1632	729	358	2263	729
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	1583	1770	3547	1583
v/c ratio	.891	.134		.705	.046	.044	.372	.803	1.037	1.608	.819	.004
g/C ratio	.061	.061		.153	.153	.387	.043	.46	.46	.202	.638	.46
Average back of queue (veh)	5.5	.7		9.2	.5	.8	1.4	29.8	44.8	54.7	37.7	.1
Uniform delay (s)	76	72.4		65.5	58.8	31.1	75.9	37.7	44	65	22.4	23.8
Incremental delay (s)												
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	76	72.4		65.5	58.8	31.1	75.9	37.7	44	65	22.4	23.8
LOS	E	E		E	E	C	E	D	D	E	C	C
Approach delay (s)/LOS	75.5	/	E	63	/	E	40.5	/	D	32.5	/	C
Intersection delay (s)/ LOS		39.2					/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: AMB PM 2 2020
Comment: 2020 AMBIENT PM W/2LT SB

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	54	5	0	340	5	106	27	1240	765	537	1656	3
RTOR volume (veh/h)			0			30			110			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	13	25	7	25	90			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s) 181 Lost time per cycle (s) 5.8 Critical v/c Ratio .713

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	59	5		370	5	83	29	1348	712	584	1800	3
Capacity (veh/h)	127	133		475	257	552	68	1764	1394	627	2332	787
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.462	.041		.779	.021	.15	.429	.764	.511	.932	.772	.004
g/C ratio	.072	.072		.138	.138	.349	.039	.497	.497	.182	.657	.497
Average back of queue (veh)	3.1	.3		10.6	.2	3	1.6	31.8	14.6	18.2	36.7	.1
Uniform delay (s)	80.6	78.2		75.3	67.4	40.5	85	36.9	30.7	72.9	21.6	22.9
Incremental delay (s)	1.9	0		8.1	0	0	2.1	2.1	.3	20.8	1.7	0
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	82.5	78.2		83.4	67.4	40.5	87.1	39	31	93.7	23.3	22.9
LOS	F	E		F	E	D	F	D	C	F	C	C
Approach delay (s)/LOS	82.2	/	F	75.5	/	E	36.9	/	D	40.5	/	D
Intersection delay (s)/ LOS			42.7				/			D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: TOT PM 2 2020
Comment: 2020 TOTAL PM W/2LT SB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	164	45	0	355	15	120	27	1230	765	537	1790	3
RTOR volume (veh/h)			0			30			110			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	L		TR	R				
WB	L	LTR	TR	R				
NB	R	R		L		TR		
SB	R			L	LTR	TR		
Green (s)	20	2	10	9	28	92		
Yellow + All red (s)	1	4	5.8	1	4	5.8		

Cycle (s) 182.6 Lost time per cycle (s) 15.6 Critical v/c Ratio .925

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	178	49		386	16	98	29	1337	712	584	1946	3
Capacity (veh/h)	194	101		433	163	267	87	1787	1854	715	2408	1299
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.92	.483		.891	.1	.366	.336	.748	.384	.816	.808	.003
g/C ratio	.11	.055		.126	.088	.169	.049	.504	.662	.208	.679	.82
Average back of queue (veh)	11.1	2.6		12	.8	4.7	1.6	31.1	9.9	16.6	41.3	0
Uniform delay (s)	80.5	83.8		78.6	76.7	67.3	83.9	36.1	14	69	20.8	3
Incremental delay (s)	42.7	3.2		20.1	0	0	0	1.8	0	7.3	2.1	0
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	123.2	87		98.7	76.7	67.3	83.9	37.9	14	76.3	22.9	3
LOS	F	F		F	E	E	F	D	B	E	C	A
Approach delay (s)/LOS	115.4	/	F	91.8	/	F	30.4	/	C	35.2	/	D
Intersection delay (s)/ LOS			42				/			D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: AMB PM 2 2029
Comment: 2029 AMBIENT PM W/2LT SB

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other; Analysis period: .25 h; Signal type: Actuated-Field; % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	67	5	0	355	5	111	32	1283	803	564	1740	3
RTOR volume (veh/h)			0			30			110			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	LTR		R					
WB		LTR	R	R				
NB			L		TR			
SB			L	LT	TR			
Green (s)	13	30	5	30	100			
Yellow + All red (s)	5.1	5.1	1	4	5.8			

Cycle (s): 199; Lost time per cycle (s): 5.8; Critical v/c Ratio: .747

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	73	5		386	5	88	35	1395	753	613	1891	3
Capacity (veh/h)	116	121		518	281	566	44	1782	1408	622	2388	796
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.63	.045		.745	.019	.156	.782	.782	.535	.986	.792	.004
g/C ratio	.065	.065		.151	.151	.357	.025	.503	.503	.181	.673	.503
Average back of queue (veh)	4.4	.3		11.8	.3	3.5	2.4	36.6	17.1	21.8	42.7	.1
Uniform delay (s)	90.7	87.2		80.8	72	43.5	96.5	40.6	33.7	81.3	22.7	24.7
Incremental delay (s)	10.5	0		5.8	0	0	59.7	2.3	.4	32.5	1.9	0
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	101.2	87.2		86.6	72	43.5	156.2	42.9	34.1	113.8	24.6	24.7
LOS	F	F		F	E	D	F	D	C	F	C	C
Approach delay (s)/LOS	100.2	/	F	78.6	/	E	41.6	/	D	46.4	/	D
Intersection delay (s)/ LOS		48.2				/			D			

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company:
Analysis Period/Year: TOT PM 2 2029
Comment: 2029 TOTAL PM W/2LT SB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KAIMINANI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other; Analysis period: .25 h; Signal type: Actuated-Field; % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	275	80	0	375	20	111	32	1270	803	564	2005	3
RTOR volume (veh/h)			0			30			110			0
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	2	2	2	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	L	LTR	TR	R				
WB	L		TR	R				
NB	R			L		TR		
SB	R	R		L	LTR	TR		
Green (s)	30	10	5	9	28	92		
Yellow + All red (s)	1	1	5.8	1	4	5.8		

Cycle (s): 192.6; Lost time per cycle (s): 10.8; Critical v/c Ratio: 1.006

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	TR		L	T	R	L	T	R	L	T	R
No. of lanes	1	1		2	1	1	1	2	2	2	2	1
Flow rate (veh/h)	299	87		408	22	88	35	1380	753	613	2179	3
Capacity (veh/h)	377	154		535	48	163	83	1694	1860	678	2283	1404
Adjusted saturation flow (veh/h)	1770	1848		3437	1863	1583	1770	3547	2803	3437	3547	1583
v/c ratio	.793	.566		.761	.45	.541	.421	.815	.405	.904	.954	.002
g/C ratio	.213	.083		.156	.026	.103	.047	.478	.664	.197	.644	.887
Average back of queue (veh)	17.1	4.9		12.1	1.3	4.9	2	36.9	11.2	19.6	66.8	0
Uniform delay (s)	71.8	85		77.9	92.4	82.1	89.3	43	14.9	75.5	31.7	1.2
Incremental delay (s)	11.1	4.8		6.4	4.2	3.6	1.5	3.2	0	15.6	10.2	0
Initial queue delay (s)	0	0		0	0	0	0	0	0	0	0	0
Delay (s)	82.9	89.8		84.3	96.6	85.7	90.8	46.2	14.9	91.1	41.9	1.2
LOS	F	F		F	F	F	F	D	B	F	D	A
Approach delay (s)/LOS	84.4	/	F	85	/	F	36.1	/	D	52.6	/	D
Intersection delay (s)/ LOS		51.5				/				D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2015
Comment: 2015 AMBIENT AM

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	122	1	147	193	5	129	84	1400	190	126	1150	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB			LTR					
WB			LTR					
NB	L	TR						
SB	L	TR						
Green (s)	15	80	30					
Yellow + All red (s)	5	5	5					

Cycle (s): 140 — Lost time per cycle (s): 10 — Critical v/c Ratio: .71

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	1	2	1	1	2	1
Flow rate (veh/h)	133	1	127	210	5	108	91	1522	174	137	1250	43
Capacity (veh/h)	301	399	339	294	388	330	190	2027	905	190	2027	905
Adjusted saturation flow (veh/h)	1405	1863	1583	1370	1810	1538	1770	3547	1583	1770	3547	1583
v/c ratio	.441	.003	.375	.715	.014	.326	.482	.751	.192	.722	.617	.048
g/C ratio	.214	.214	.214	.214	.214	.214	.107	.571	.571	.107	.571	.571
Average back of queue (veh)	4.8	0	4.5	8.6	.2	3.8	3.6	25.9	3.5	5.9	18.4	.8
Uniform delay (s)	47.7	43.2	47	51	43.3	46.5	58.8	22.5	14.4	60.5	19.9	13.2
Incremental delay (s)	.6	0	.1	8	0	0	1.7	1.6	0	12.7	.6	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	48.3	43.2	47.1	59	43.3	46.5	60.5	24.1	14.4	73.2	20.5	13.2
LOS	D	D	D	E	D	D	E	C	B	E	C	B
Approach delay (s)/LOS	47.7	/	D	54.6	/	D	25	/	C	25.3	/	C
Intersection delay (s)/ LOS		29.2				/				C		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2 2015
Comment: 2015 TOTAL AM W/2LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	122	4	150	193	5	129	122	1400	190	129	1214	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB			LTR					
WB			LTR					
NB	L	TR						
SB	L	TR						
Green (s)	15	80	30					
Yellow + All red (s)	5	5	5					

Cycle (s): 140 — Lost time per cycle (s): 10 — Critical v/c Ratio: .713

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	133	4	130	210	5	108	133	1522	174	140	1320	43
Capacity (veh/h)	301	399	339	293	388	330	368	2027	905	190	2027	905
Adjusted saturation flow (veh/h)	1405	1863	1583	1366	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.441	.011	.384	.717	.014	.326	.36	.751	.192	.74	.651	.048
g/C ratio	.214	.214	.214	.214	.214	.214	.107	.571	.571	.107	.571	.571
Average back of queue (veh)	4.8	.1	4.6	8.6	.2	3.8	2.7	25.9	3.5	6.1	20.1	.8
Uniform delay (s)	47.7	43.3	47.1	51.1	43.3	46.5	58	22.5	14.4	60.6	20.5	13.2
Incremental delay (s)	.6	0	.1	8.2	0	0	0	1.6	0	14.3	.8	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	48.3	43.3	47.2	59.3	43.3	46.5	58	24.1	14.4	74.9	21.3	13.2
LOS	D	D	D	E	D	D	E	C	B	E	C	B
Approach delay (s)/LOS	47.7	/	D	54.7	/	D	25.7	/	C	26	/	C
Intersection delay (s)/ LOS		29.7				/				C		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2020
Comment: 2020 AMBIENT AM

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	122	1	147	173	5	129	84	1475	170	126	1261	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB			LTR					
WB			LTR					
NB	L	TR						
SB	L	TR						
Green (s)	15	80	30					
Yellow + All red (s)	5	5	5					

Cycle (s): 140   Lost time per cycle (s): 10   Critical v/c Ratio: .718

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	1	2	1	1	2	1
Flow rate (veh/h)	133	1	127	188	5	108	91	1603	152	137	1371	43
Capacity (veh/h)	301	399	339	294	388	330	190	2027	905	190	2027	905
Adjusted saturation flow (veh/h)	1405	1863	1583	1370	1810	1538	1770	3547	1583	1770	3547	1583
v/c ratio	.441	.003	.375	.64	.014	.326	.482	.791	.168	.722	.676	.048
g/C ratio	.214	.214	.214	.214	.214	.214	.107	.571	.571	.107	.571	.571
Average back of queue (veh)	4.8	0	4.5	7.4	.2	3.8	3.6	28.8	3	5.9	21.4	.8
Uniform delay (s)	47.7	43.2	47	50.1	43.3	46.5	58.8	23.5	14.2	60.5	21	13.2
Incremental delay (s)	.6	0	.1	4.7	0	0	1.7	2.2	0	12.7	.9	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	48.3	43.2	47.1	54.8	43.3	46.5	60.5	25.7	14.2	73.2	21.9	13.2
LOS	D	D	D	D	D	D	E	C	B	E	C	B
Approach delay (s)/LOS	47.7	/	D	51.6	/	D	26.4	/	C	26.1	/	C
Intersection delay (s)/ LOS		29.6				/			C			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2020
Comment: 2020 TOTAL AMW/2 LT NB LANES

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	122	1	167	173	16	129	230	1475	170	136	1376	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB			LTR					
WB			LTR					
NB	L	TR						
SB	L	TR						
Green (s)	20	80	40					
Yellow + All red (s)	1	5	5					

Cycle (s): 151   Lost time per cycle (s): 10   Critical v/c Ratio: .721

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	133	1	149	188	17	108	250	1603	152	148	1496	43
Capacity (veh/h)	368	493	419	363	479	407	455	1879	839	234	1879	839
Adjusted saturation flow (veh/h)	1390	1863	1583	1370	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.36	.002	.355	.518	.036	.264	.549	.853	.181	.631	.796	.052
g/C ratio	.265	.265	.265	.265	.265	.265	.132	.53	.53	.132	.53	.53
Average back of queue (veh)	4.8	0	5.4	7.3	.6	3.8	5.5	34.7	3.5	6.5	30	.9
Uniform delay (s)	45.1	40.8	45	47.3	41.2	43.9	61.3	30.5	18.5	62	28.9	17.2
Incremental delay (s)	0	0	0	1.3	0	0	1.4	4	0	5.4	2.5	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	45.1	40.8	45	48.6	41.2	43.9	62.7	34.5	18.5	67.4	31.4	17.2
LOS	D	D	D	D	D	D	E	C	B	E	C	B
Approach delay (s)/LOS	45	/	D	46.6	/	D	36.8	/	D	34.2	/	C
Intersection delay (s)/ LOS		37				/			D			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2 2029
Comment: 2029 AMB AM W/2LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	122	1	147	173	5	129	89	1535	170	126	1261	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB			LTR					
WB			LTR					
NB	L	TR						
SB	L	TR						
Green (s)	15	80	30					
Yellow + All red (s)	5	5	5					

Cycle (s): 140   Lost time per cycle (s): 10   Critical v/c Ratio: .738

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	133	1	127	188	5	108	97	1668	152	137	1371	43
Capacity (veh/h)	301	399	339	294	388	330	368	2027	905	190	2027	905
Adjusted saturation flow (veh/h)	1405	1863	1583	1370	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.441	.003	.375	.64	.014	.326	.263	.823	.168	.722	.676	.048
g/C ratio	.214	.214	.214	.214	.214	.214	.107	.571	.571	.107	.571	.571
Average back of queue (veh)	4.8	0	4.5	7.4	.2	3.8	1.9	31.3	3	5.9	21.4	.8
Uniform delay (s)	47.7	43.2	47	50.1	43.3	46.5	57.4	24.3	14.2	60.5	21	13.2
Incremental delay (s)	.6	0	.1	4.7	0	0	0	2.9	0	12.7	.9	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	48.3	43.2	47.1	54.8	43.3	46.5	57.4	27.2	14.2	73.2	21.9	13.2
LOS	D	D	D	D	D	D	E	C	B	E	C	B
Approach delay (s)/LOS	47.7	/	D	51.6	/	D	27.7	/	C	26.1	/	C
Intersection delay (s)/ LOS			30.2				/			C		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2 2029
Comment: 2029 TOTAL AM W/2 LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	122	17	190	173	20	129	290	1641	170	135	1490	50
RTOR volume (veh/h)			30			30			30			10
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	20	10	95	45				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 186   Lost time per cycle (s): 10   Critical v/c Ratio: .766

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	133	18	174	188	22	108	315	1784	152	147	1620	43
Capacity (veh/h)	335	451	383	326	438	372	573	2097	936	190	1811	809
Adjusted saturation flow (veh/h)	1384	1863	1583	1349	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.396	.041	.454	.576	.05	.289	.55	.85	.163	.771	.894	.054
g/C ratio	.242	.242	.242	.242	.242	.242	.167	.591	.591	.108	.511	.511
Average back of queue (veh)	6.1	.8	8.1	9.3	.9	4.8	8.3	44.9	3.8	8.5	45.5	1.2
Uniform delay (s)	59.1	54	60	62.1	54.1	57.5	71.1	31.2	17.2	80.8	41	22.9
Incremental delay (s)	.2	0	.6	2.5	0	0	1.1	3.6	0	17.5	6.2	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	59.3	54	60.6	64.6	54.1	57.5	72.2	34.8	17.2	98.3	47.2	22.9
LOS	E	D	E	E	D	E	E	C	B	F	D	C
Approach delay (s)/LOS	59.7	/	E	61.5	/	E	38.9	/	D	50.7	/	D
Intersection delay (s)/ LOS			46.4				/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB PM 2015
Comment: 2015 AMBIENT PM

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	95	1	119	268	1	179	260	1175	98	66	1255	190
RTOR volume (veh/h)			25			30			25			30
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	15	20	85	50				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 186 Lost time per cycle (s): 10 Critical v/c Ratio: .8

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	1	2	1	1	2	1
Flow rate (veh/h)	103	1	102	291	1	162	283	1277	79	72	1364	174
Capacity (veh/h)	379	501	426	368	486	413	343	2097	936	143	1621	724
Adjusted saturation flow (veh/h)	1410	1863	1583	1370	1810	1538	1770	3547	1583	1770	3547	1583
v/c ratio	.272	.002	.24	.791	.002	.392	.825	.609	.085	.503	.842	.24
g/C ratio	.269	.269	.269	.269	.269	.269	.194	.591	.591	.081	.457	.457
Average back of queue (veh)	4.4	0	4.3	15.8	0	7.2	16.1	23.8	1.9	3.9	36.8	5.8
Uniform delay (s)	53.6	49.7	53.2	63.1	49.8	55.6	72	24.3	16.3	81.9	44.6	30.8
Incremental delay (s)	0	0	0	11.1	0	.1	15.1	.5	0	2.8	4.2	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	53.6	49.7	53.2	74.2	49.8	55.7	87.1	24.8	16.3	84.7	48.8	30.8
LOS	D	D	D	E	D	E	F	C	B	F	D	C
Approach delay (s)/LOS	53.4	/	D	67.6	/	E	35.1	/	D	48.4	/	D
Intersection delay (s)/ LOS			45.3			/			D			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2 2015
Comment: 2015 TOTAL PM W/2LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other Analysis period: .25 h Signal type: Actuated-Field % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	95	4	144	268	15	179	360	1175	98	71	1300	190
RTOR volume (veh/h)			25			30			25			30
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	15	16	87	48				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 182 Lost time per cycle (s): 10 Critical v/c Ratio: .768

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	103	4	129	291	16	162	391	1277	79	77	1413	174
Capacity (veh/h)	367	491	418	360	477	406	604	2105	940	146	1695	757
Adjusted saturation flow (veh/h)	1391	1863	1583	1366	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.281	.009	.31	.809	.034	.399	.648	.607	.084	.529	.833	.23
g/C ratio	.264	.264	.264	.264	.264	.264	.176	.593	.593	.082	.478	.478
Average back of queue (veh)	4.4	.2	5.5	15.7	.6	7.1	10.4	23.2	1.8	4.1	36.6	5.4
Uniform delay (s)	53.3	49.4	53.7	62.7	49.8	55.1	69.8	23.5	15.8	80.1	41.2	27.9
Incremental delay (s)	0	0	0	12.9	0	.2	2.4	.5	0	3.6	3.7	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	53.3	49.4	53.7	75.6	49.8	55.3	72.2	24	15.8	83.7	44.9	27.9
LOS	D	D	D	E	D	E	E	C	B	F	D	C
Approach delay (s)/LOS	53.5	/	D	67.7	/	E	34.4	/	C	44.9	/	D
Intersection delay (s)/ LOS			43.6			/			D			

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB PM 2020
Comment: 2020 AMBIENT PM

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	95	1	119	240	1	170	260	1190	88	66	1270	190
RTOR volume (veh/h)			25			30			25			30
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	10	18	85	45				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 174   Lost time per cycle (s): 10   Critical v/c Ratio: .784

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	1	2	1	1	2	1
Flow rate (veh/h)	103	1	102	261	1	152	283	1293	68	72	1380	174
Capacity (veh/h)	365	482	409	354	468	398	295	2201	983	102	1733	773
Adjusted saturation flow (veh/h)	1410	1863	1583	1370	1810	1538	1770	3547	1583	1770	3547	1583
v/c ratio	.283	.002	.25	.736	.002	.383	.958	.588	.07	.705	.797	.225
g/C ratio	.259	.259	.259	.259	.259	.259	.167	.621	.621	.057	.489	.489
Average back of queue (veh)	4.2	0	4.1	12.9	0	6.4	17	21.2	1.4	3.9	32.7	5.1
Uniform delay (s)	51.6	47.8	51.1	59.1	47.8	53.1	71.9	19.7	13.1	80.6	37.3	25.6
Incremental delay (s)	0	0	0	7.8	0	.1	41.1	.4	0	19.9	2.7	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	51.6	47.8	51.1	66.9	47.8	53.2	113	20.1	13.1	100.5	40	25.6
LOS	D	D	D	E	D	D	F	C	B	F	D	C
Approach delay (s)/LOS	51.3	/	D	61.8	/	E	35.8	/	D	41.1	/	D
Intersection delay (s)/ LOS	41.6			/			D					

HiCAP 2000 ™   1 of 1

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2 2020
Comment: 2020 TOTAL PM W 2LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Volume (veh/h)	95	4	135	240	20	170	430	1180	88	76	1400	190
RTOR volume (veh/h)			50			50			25			50
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT   T: TH   R: RT   P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	18	16	110	55				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 215   Lost time per cycle (s): 10   Critical v/c Ratio: .793

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	103	4	92	261	22	130	467	1283	68	83	1522	152
Capacity (veh/h)	354	477	405	349	463	393	559	2161	965	148	1815	810
Adjusted saturation flow (veh/h)	1384	1863	1583	1366	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.292	.009	.228	.747	.047	.332	.835	.594	.071	.558	.839	.188
g/C ratio	.256	.256	.256	.256	.256	.256	.163	.609	.609	.084	.512	.512
Average back of queue (veh)	5.2	.2	4.6	15.9	1	6.7	16	26.4	1.8	5.2	45.5	5.1
Uniform delay (s)	64.3	59.7	63.2	73.6	60.3	65.1	87.2	25.7	17.2	94.7	44.9	28.4
Incremental delay (s)	0	0	0	8.5	0	0	10.6	.4	0	4.6	3.7	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	64.3	59.7	63.2	82.1	60.3	65.1	97.8	26.1	17.2	99.3	48.6	28.4
LOS	E	E	E	F	E	E	F	C	B	F	D	C
Approach delay (s)/LOS	63.7	/	E	75.6	/	E	44.2	/	D	49.2	/	D
Intersection delay (s)/ LOS	50.3			/			D					

HiCAP 2000 ™   1 of 1

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB PM 2 2029
Comment: 2029 AMBIENT PM W/2LT NB

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	95	1	119	240	1	170	260	1250	88	66	1335	190
RTOR volume (veh/h)			50			50			25			50
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB				LTR				
WB				LTR				
NB	L	LTR	TR					
SB	L		TR					
Green (s)	12	5	85	45				
Yellow + All red (s)	1	5	5	5				

Cycle (s): 163 — Lost time per cycle (s): 10 — Critical v/c Ratio: .726

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	1	1	1	2	2	1	1	2	1
Flow rate (veh/h)	103	1	75	261	1	130	283	1359	68	72	1451	152
Capacity (veh/h)	389	514	437	378	500	425	379	2067	923	130	1849	826
Adjusted saturation flow (veh/h)	1410	1863	1583	1370	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.265	.002	.172	.69	.002	.307	.745	.657	.074	.551	.785	.184
g/C ratio	.276	.276	.276	.276	.276	.276	.11	.583	.583	.074	.521	.521
Average back of queue (veh)	3.9	0	2.7	11.7	0	4.9	7.3	23.8	1.4	3.5	31.1	3.8
Uniform delay (s)	46.1	42.7	44.8	52.8	42.7	46.7	70.3	23	14.8	72.9	31.6	20.6
Incremental delay (s)	0	0	0	5.3	0	0	7.8	.8	0	4.9	2.3	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	46.1	42.7	44.8	58.1	42.7	46.7	78.1	23.8	14.8	77.8	33.9	20.6
LOS	D	D	D	E	D	D	E	C	B	E	C	C
Approach delay (s)/LOS	45.5	/	D	54.2	/	D	32.4	/	C	34.6	/	C
Intersection delay (s)/ LOS	36.1					/			D			

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 8PH 2029
Comment: 2029 TOTAL PM W 8 PH SIG

Site Information

Jurisdiction/Date: 4/13/2008
EB/WB Street: KOHANA IKI
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)	95	15	175	240	30	170	545	1235	88	86	1540	190
RTOR volume (veh/h)			50			50			25			50
Peak-hour factor	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92	.92
Heavy vehicles (%)	2	2	2	5	5	5	2	2	2	2	2	2
Start-up lost time, l_1 (s)	2	2	2	2	2	2	2	2	2	2	2	2
Extension of effective green, e (s)	2	2	2	2	2	2	2	2	2	2	2	2
Arrival type, AT	3	3	3	3	3	3	3	3	3	3	3	3
Approach pedestrian volume (p/h)		0			0			0			0	
Approach bicycle volume (bic/h)		0			0			0			0	
Left/right parking (Y or N)	N	/	N	N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB	R	R		L		TR		
WB	R			L	LTR	TR		
NB	L	LTR	TR	R	R			
SB	L		TR	R				
Green (s)	15	25	100	15	8	10		
Yellow + All red (s)	1	5	5	1	1	5		

Cycle (s): 191 — Lost time per cycle (s): 10 — Critical v/c Ratio: .853

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration	L	T	R	L	T	R	L	T	R	L	T	R
No. of lanes	1	1	1	2	1	1	2	2	1	1	2	1
Flow rate (veh/h)	103	16	136	261	33	130	592	1342	68	93	1674	152
Capacity (veh/h)	139	98	464	419	180	314	738	2414	1318	139	1857	995
Adjusted saturation flow (veh/h)	1770	1863	1583	3338	1810	1538	3437	3547	1583	1770	3547	1583
v/c ratio	.743	.167	.293	.622	.181	.415	.803	.556	.052	.673	.901	.153
g/C ratio	.079	.052	.293	.126	.099	.204	.215	.681	.832	.079	.524	.628
Average back of queue (veh)	6.2	.9	5.9	7.4	1.7	6.4	17.4	20.8	.7	5.4	48.5	3.5
Uniform delay (s)	86.1	86.5	52.2	79.2	78.9	66.1	71.2	15.7	2.8	85.6	41.1	14.6
Incremental delay (s)	19.2	0	0	2.8	0	.4	6.4	.3	0	12	6.5	0
Initial queue delay (s)	0	0	0	0	0	0	0	0	0	0	0	0
Delay (s)	105.3	86.5	52.2	82	78.9	66.5	77.6	16	2.8	97.6	47.6	14.6
LOS	F	F	D	F	E	E	E	B	A	F	D	B
Approach delay (s)/LOS	75.9	/	E	77	/	E	33.7	/	C	47.4	/	D
Intersection delay (s)/ LOS	45.7					/			D			

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year EX AM #1 2006
Comment 2006 EXIST 6:30-7:30AM

Site Information

Jurisdiction/Date 3/26/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				225		260		685	295	120	545	
RTOR volume (veh/h)						40			80			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB			TR					
SB		LT	LT					
Green (s)	30	11	44					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 100 Lost time per cycle (s) 15 Critical v/c Ratio .751

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				250		244		761	239	133	606	
Capacity (veh/h)				531		475		820	697	352	1101	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.471		.515		.929	.343	.378	.55	
g/C ratio				.3		.3		.44	.44	.608	.591	
Average back of queue (veh)				6.1		6.1		25	4.7	2.1	11.1	
Uniform delay (s)				28.5		29		26.5	18.5	12.9	12.4	
Incremental delay (s)				.5		1		16.7	0	.1	.6	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				29		30		43.2	18.5	13	13	
LOS				C		C		D	B	B	B	
Approach delay (s)/LOS		/		29.5	/	C	37.3	/	D	13	/	B
Intersection delay (s)/ LOS		27.5				/			C			

HiCAP 2000 ™

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year EX AM #2 2006
Comment 2006 EXIST 7:30-8:30AM

Site Information

Jurisdiction/Date 3/26/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				285		275		640	340	140	725	
RTOR volume (veh/h)						40			80			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB			TR					
SB		LT	LT					
Green (s)	30	11	44					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 100 Lost time per cycle (s) 9.9 Critical v/c Ratio .72

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				317		261		711	289	156	806	
Capacity (veh/h)				531		475		820	697	370	1101	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.596		.55		.868	.415	.42	.732	
g/C ratio				.3		.3		.44	.44	.608	.591	
Average back of queue (veh)				8.2		6.6		21.3	5.9	2.5	18.1	
Uniform delay (s)				29.8		29.3		25.4	19.2	12.7	14.7	
Incremental delay (s)				1.8		1.4		9.8	.2	.3	2.5	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				31.6		30.7		35.2	19.4	13	17.2	
LOS				C		C		D	B	B	B	
Approach delay (s)/LOS		/		31.2	/	C	30.6	/	C	16.6	/	B
Intersection delay (s)/ LOS		25.4				/			C			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year AMB AM 2015
Comment 2015 AMB AM

Site Information

Jurisdiction/Date 4/17/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other | Analysis period .25 h | Signal type Actuated-Field | % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				434		423		1250	532	211	1280	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	55	20	65					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 155 | Lost time per cycle (s) 15.7 | Critical v/c Ratio .917

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		2	1	1	2	
Flow rate (veh/h)				482		381		1389	480	234	1422	
Capacity (veh/h)				628		818		1487	1285	276	2039	
Adjusted saturation flow (veh/h)				1770		1583		3547	1583	1770	3547	
v/c ratio				.768		.466		.934	.374	.848	.698	
g/C ratio				.355		.517		.419	.812	.586	.575	
Average back of queue (veh)				20.6		11.2		36	6.2	10	25	
Uniform delay (s)				44.3		23.8		42.9	3.9	49.9	23.4	
Incremental delay (s)				5.7		.3		11.1	0	21.2	1.1	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				50		24.1		54	3.9	71.1	24.5	
LOS				D		C		D	A	E	C	
Approach delay (s)/LOS		/		38.6	/	D	41.2	/	D	31.1	/	C
Intersection delay (s)/ LOS		36.9				/				D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year TOT AM 2015
Comment 2015 TOT W/PROJ AM

Site Information

Jurisdiction/Date 4/17/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other | Analysis period .25 h | Signal type Actuated-Field | % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				434		433		1278	532	221	1340	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	53	20	67					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 155 | Lost time per cycle (s) 15.7 | Critical v/c Ratio .926

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		2	1	1	2	
Flow rate (veh/h)				482		392		1420	480	246	1489	
Capacity (veh/h)				605		798		1533	1285	276	2085	
Adjusted saturation flow (veh/h)				1770		1583		3547	1583	1770	3547	
v/c ratio				.797		.492		.926	.374	.888	.714	
g/C ratio				.342		.504		.432	.812	.599	.588	
Average back of queue (veh)				21.2		11.9		36.3	6.2	11.1	26.3	
Uniform delay (s)				46.1		25.4		41.7	3.9	51.3	22.7	
Incremental delay (s)				7.4		.5		10	0	27.6	1.2	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				53.5		25.9		51.7	3.9	78.9	23.9	
LOS				D		C		D	A	E	C	
Approach delay (s)/LOS		/		41.1	/	D	39.6	/	D	31.7	/	C
Intersection delay (s)/ LOS		36.9				/				D		

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: AMB AM 2 2020
Comment: 2020 AMB AM W/2 LT WB

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				578		443		1266	613	285	1296	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	40	33	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 158   Lost time per cycle (s): 15.7   Critical v/c Ratio: 1.039

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				642		403		1407	570	317	1440	
Capacity (veh/h)				870		783		1571	1160	417	2404	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.738		.515		.895	.491	.76	.599	
g/C ratio				.253		.494		.443	.733	.689	.678	
Average back of queue (veh)				14.9		12.9		35	11.4	10.6	19.6	
Uniform delay (s)				54.2		27.1		40.6	8.8	48.1	13.8	
Incremental delay (s)				3.4		.6		7.1	.3	8	.4	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				57.6		27.7		47.7	9.1	56.1	14.2	
LOS				E		C		D	A	E	B	
Approach delay (s)/LOS		/		46.1	/	D	36.6	/	D	21.8	/	C
Intersection delay (s)/ LOS				33.2			/			C		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: TOT AM 2 2020
Comment: 2020 TOT AM W/2 LT WB

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				578		465		1305	613	315	1401	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	45	40	80					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 180   Lost time per cycle (s): 15.7   Critical v/c Ratio: 1.059

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				642		428		1450	570	350	1557	
Capacity (veh/h)				859		793		1576	1151	435	2445	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.748		.54		.92	.495	.805	.637	
g/C ratio				.25		.501		.444	.727	.699	.689	
Average back of queue (veh)				16.9		15.7		41.9	13.3	14.1	24.7	
Uniform delay (s)				62.3		30.8		47	10.5	56.9	15.5	
Incremental delay (s)				3.6		.7		9.1	.3	10.6	.6	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				65.9		31.5		56.1	10.8	67.5	16.1	
LOS				E		C		E	B	E	B	
Approach delay (s)/LOS		/		52.1	/	D	43.3	/	D	25.5	/	C
Intersection delay (s)/ LOS				38.4			/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: AMB AM 2 2029
Comment: 2029 AMB AM W/2 LT WB

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				607		443		1330	644	285	1296	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	40	33	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 158   Lost time per cycle (s): 15.7   Critical v/c Ratio: 1.085

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				674		403		1478	604	317	1440	
Capacity (veh/h)				870		783		1571	1160	417	2404	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.775		.515		.94	.521	.76	.599	
g/C ratio				.253		.494		.443	.733	.689	.678	
Average back of queue (veh)				16		12.9		39.1	12.6	10.6	19.6	
Uniform delay (s)				54.8		27.1		42	9.1	48.5	13.8	
Incremental delay (s)				4.4		.6		11.5	.4	8	.4	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				59.2		27.7		53.5	9.5	56.5	14.2	
LOS				E		C		D	A	E	B	
Approach delay (s)/LOS		/		47.4	/	D	40.7	/	D	21.8	/	C
Intersection delay (s)/ LOS		35.5				/			D			

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: TOT AM 2 2029
Comment: 2029 TOT AM W/2 LT WB&SB

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other   Analysis period: .25 h   Signal type: Actuated-Field   % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				607		495		1480	644	340	1515	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	35	25	75					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 150   Lost time per cycle (s): 15.7   Critical v/c Ratio: 1.067

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	2	2	
Flow rate (veh/h)				674		461		1644	604	378	1683	
Capacity (veh/h)				802		687		1773	1222	1833	2461	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	3437	3547	
v/c ratio				.841		.671		.927	.495	.206	.684	
g/C ratio				.233		.434		.5	.772	.705	.694	
Average back of queue (veh)				16.1		16.8		40	10.3	2.9	23.7	
Uniform delay (s)				54.8		33.9		35	6.3	23.6	13.4	
Incremental delay (s)				8		2.6		9	.3	0	.8	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				62.8		36.5		44	6.6	23.6	14.2	
LOS				E		D		D	A	C	B	
Approach delay (s)/LOS		/		52.1	/	D	33.9	/	C	15.9	/	B
Intersection delay (s)/ LOS		30.9				/			C			

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: ANALYST
Agency or Company: M&E PAC
Analysis Period/Year: EX PM #1 2006
Comment: 2006 EXIST 3:30-4:30 PM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				300		280		550	445	175	650	
RTOR volume (veh/h)						40			80			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB			TR					
SB		LT	LT					
Green (s)	28	23.1	44					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s): 110 — Lost time per cycle (s): 9.8 — Critical v/c Ratio: .688

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				333		267		611	406	194	722	
Capacity (veh/h)				450		403		745	633	541	1204	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.74		.662		.82	.64	.36	.6	
g/C ratio				.255		.255		.4	.4	.663	.646	
Average back of queue (veh)				10.6		8.1		19.3	11	2.9	14	
Uniform delay (s)				37.7		36.8		29.5	26.6	11.3	11.2	
Incremental delay (s)				6.4		4		7.3	2.2	0	.8	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				44.1		40.8		36.8	28.8	11.3	12	
LOS				D		D		D	C	B	B	
Approach delay (s)/LOS		/		42.6	/	D	33.6	/	C	11.9	/	B
Intersection delay (s)/ LOS		27.9			/				C			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: ANALYST
Agency or Company: M&E PAC
Analysis Period/Year: EX PM #2 2006
Comment: 2006 EXIST 4:30-5:30 PM

Site Information

Jurisdiction/Date: 3/26/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other — Analysis period: .25 h — Signal type: Actuated-Field — % Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				260		270		550	360	105	490	
RTOR volume (veh/h)						40			80			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR							
NB			TR					
SB		LT	LT					
Green (s)	28	23.1	44					
Yellow + All red (s)	5.1	4	5.8					

Cycle (s): 110 — Lost time per cycle (s): 9.8 — Critical v/c Ratio: .612

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		1	1	1	1	
Flow rate (veh/h)				289		256		611	311	117	544	
Capacity (veh/h)				450		403		745	633	541	1204	
Adjusted saturation flow (veh/h)				1770		1583		1863	1583	1770	1863	
v/c ratio				.641		.634		.82	.491	.216	.452	
g/C ratio				.255		.255		.4	.4	.663	.646	
Average back of queue (veh)				8.7		7.7		19.3	7.7	1.5	9	
Uniform delay (s)				36.5		36.4		29.5	24.6	10.5	9.7	
Incremental delay (s)				3.1		3.3		7.3	.6	0	.2	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				39.6		39.7		36.8	25.2	10.5	9.9	
LOS				D		D		D	C	B	A	
Approach delay (s)/LOS		/		39.7	/	D	32.9	/	C	10	/	A
Intersection delay (s)/ LOS		27.5			/				C			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/17/2008
Agency or Company	M&E PAC	EB/WB Street	HINALANI D
Analysis Period/Year	AMB PM 2015	NB/SB Street	QUEEN KAAH
Comment	2015 AMB PM W/ 1LT WB LANE		

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				615		360		1174	699	260	1383	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	65	25	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 175 Lost time per cycle (s) 9.9 Critical v/c Ratio .597

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		2	1	1	2	
Flow rate (veh/h)				683		311		1304	666	289	1537	
Capacity (veh/h)				657		860		1419	1274	295	2008	
Adjusted saturation flow (veh/h)				1770		1583		3547	1583	1770	3547	
v/c ratio				1.04		.362		.919	.522	.978	.765	
g/C ratio				.371		.543		.4	.805	.576	.566	
Average back of queue (veh)				43.3		9.1		37.2	12.2	17.3	33.2	
Uniform delay (s)				55		22.7		49.8	5.8	61.1	29	
Incremental delay (s)				45.8		0		9.9	.4	46.2	1.8	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				100.8		22.7		59.7	6.2	107.3	30.8	
LOS				F		C		E	A	F	C	
Approach delay (s)/LOS		/		76.4	/	E	41.6	/	D	42.9	/	D
Intersection delay (s)/ LOS		49.3				/			D			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/17/2008
Agency or Company	M&E PAC	EB/WB Street	HINALANI D
Analysis Period/Year	TOT PM 2015	NB/SB Street	QUEEN KAAH
Comment	2015 TOT PM W/ 1LT WB LANE		

Intersection Data

Area type Other Analysis period .25 h Signal type Actuated-Field % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				615		385		1249	699	270	1443	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	65	25	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 175 Lost time per cycle (s) 9.9 Critical v/c Ratio 1.106

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				1		1		2	1	1	2	
Flow rate (veh/h)				683		339		1388	666	300	1603	
Capacity (veh/h)				657		860		1419	1274	295	2008	
Adjusted saturation flow (veh/h)				1770		1583		3547	1583	1770	3547	
v/c ratio				1.04		.394		.978	.522	1.016	.798	
g/C ratio				.371		.543		.4	.805	.576	.566	
Average back of queue (veh)				43.3		10.2		42.8	12.2	19	36.1	
Uniform delay (s)				55		23.2		51.7	5.8	62.2	30	
Incremental delay (s)				45.8		.1		18.8	.4	56.4	2.4	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				100.8		23.3		70.5	6.2	118.6	32.4	
LOS				F		C		E	A	F	C	
Approach delay (s)/LOS		/		75.1	/	E	49.7	/	D	46	/	D
Intersection delay (s)/ LOS		53.5				/			D			

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CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year AMB PM 2 2020
Comment 2020 AMB PM W/ 2LT WB LANE

Site Information

Jurisdiction/Date 4/17/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other   Analysis period .25 h   Signal type Actuated-Field   % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				559		374		1165	739	392	1247	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	35	40	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 160   Lost time per cycle (s) 9.9   Critical v/c Ratio .74

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				621		327		1294	710	436	1386	
Capacity (veh/h)				752		793		1552	1096	495	2529	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.826		.412		.834	.648	.88	.548	
g/C ratio				.219		.501		.438	.693	.724	.713	
Average back of queue (veh)				15.7		9.7		30.3	19.4	17.8	16.6	
Uniform delay (s)				59.6		25.1		39.9	13.7	48.7	10.8	
Incremental delay (s)				7.6		.1		4.1	1.3	16.5	.3	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				67.2		25.2		44	15	65.2	11.1	
LOS				E		C		D	B	E	B	
Approach delay (s)/LOS		/		52.7	/	D	33.7	/	C	24	/	C
Intersection delay (s)/ LOS		33.8					/			C		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst WY
Agency or Company M&E PAC
Analysis Period/Year TOT PM 2 2020
Comment 2020 TOT PM W/ 2LT WB LANE

Site Information

Jurisdiction/Date 4/17/2008
EB/WB Street HINALANI D
NB/SB Street QUEEN KAAH

Intersection Data

Area type Other   Analysis period .25 h   Signal type Actuated-Field   % Back of queue 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				559		417		1295	739	425	1355	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	35	45	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s) 165   Lost time per cycle (s) 9.9   Critical v/c Ratio .761

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				621		374		1439	710	472	1506	
Capacity (veh/h)				729		817		1505	1063	528	2560	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.852		.459		.956	.668	.895	.588	
g/C ratio				.212		.516		.424	.672	.732	.722	
Average back of queue (veh)				16.5		11.6		40.7	21.4	20.4	19.2	
Uniform delay (s)				62.5		25.3		46	16.1	51.1	11.1	
Incremental delay (s)				9.6		.3		14.2	1.6	17.7	.4	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				72.1		25.6		60.2	17.7	68.8	11.5	
LOS				E		C		E	B	E	B	
Approach delay (s)/LOS		/		54.6	/	D	46.2	/	D	25.2	/	C
Intersection delay (s)/ LOS		39.7					/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: AMB PM 2 2029
Comment: 2029 AMB PM W/ 2LT WB LANE

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other
Analysis period: .25 h
Signal type: Actuated-Field
% Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				587		412		1328	776	438	1256	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	35	40	70					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 160 Lost time per cycle (s): 9.9 Critical v/c Ratio: .772

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	1	2	
Flow rate (veh/h)				652		369		1476	751	487	1396	
Capacity (veh/h)				752		793		1552	1096	489	2529	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	1770	3547	
v/c ratio				.868		.465		.951	.685	.995	.552	
g/C ratio				.219		.501		.438	.693	.724	.713	
Average back of queue (veh)				17		11.4		40.2	21.7	27.3	16.8	
Uniform delay (s)				60.3		26		43.3	14.4	53.4	10.9	
Incremental delay (s)				10.6		.3		13.1	1.8	39.4	.3	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				70.9		26.3		56.4	16.2	92.8	11.2	
LOS				E		C		E	B	F	B	
Approach delay (s)/LOS		/		54.8	/	D	42.9	/	D	32.3	/	C
Intersection delay (s)/ LOS			41.3				/			D		

CHAPTER 16 - OPERATIONAL ANALYSIS - SUMMARY WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PAC
Analysis Period/Year: TOT PM 2 2029
Comment: 2029 TOT PM W/ 2LT WB &SB LANES

Site Information

Jurisdiction/Date: 4/17/2008
EB/WB Street: HINALANI D
NB/SB Street: QUEEN KAAH

Intersection Data

Area type: Other
Analysis period: .25 h
Signal type: Actuated-Field
% Back of queue: 95

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume (veh/h)				587		487		1540	776	505	1450	
RTOR volume (veh/h)						80			100			0
Peak-hour factor				.9	.9	.9	.9	.9	.9	.9	.9	.9
Heavy vehicles (%)				2		2		2	2	2	2	
Start-up lost time, l_1 (s)				2		2		2	2	2	2	
Extension of effective green, e (s)				2		2		2	2	2	2	
Arrival type, AT				3		3		3	3	3	3	
Approach pedestrian volume (p/h)					0			0			0	
Approach bicycle volume (bic/h)					0			0			0	
Left/right parking (Y or N)		/		N	/	N	N	/	N	N	/	N

Signal Phasing Plan

L: LT T: TH R: RT P: Peds

	Phase 1	Phase 2	Phase 3	Phase 4	Phase 5	Phase 6	Phase 7	Phase 8
EB								
WB	LR	R						
NB	R		TR					
SB		LT	LT					
Green (s)	50	20	105					
Yellow + All red (s)	5.1	4.1	5.8					

Cycle (s): 190 Lost time per cycle (s): 15.7 Critical v/c Ratio: 1.158

Intersection Performance

	EB			WB			NB			SB		
Lane group configuration				L		R		T	R	L	T	
No. of lanes				2		1		2	1	2	2	
Flow rate (veh/h)				652		452		1711	751	561	1611	
Capacity (veh/h)				904		626		1960	1340	1627	2410	
Adjusted saturation flow (veh/h)				3437		1583		3547	1583	3437	3547	
v/c ratio				.721		.723		.873	.561	.345	.669	
g/C ratio				.263		.395		.553	.846	.688	.679	
Average back of queue (veh)				17.7		22.2		46.6	13.2	6.4	28.6	
Uniform delay (s)				63.7		48.6		36.7	4.3	31.1	17.9	
Incremental delay (s)				2.8		4.1		4.7	.5	0	.7	
Initial queue delay (s)				0		0		0	0	0	0	
Delay (s)				66.5		52.7		41.4	4.8	31.1	18.6	
LOS				E		D		D	A	C	B	
Approach delay (s)/LOS		/		60.8	/	E	30.3	/	C	21.8	/	C
Intersection delay (s)/ LOS			33				/			C		

Appendix C

Unsignalized Intersection
Level of Service (LOS) Calculations

CHAPTER 17 - TWSC - UNSIGNALIZED INTERSECTIONS WORKSHEET

Analysis Summary

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/16/2008
Agency or Company	M&E PACIFIC	Major Street	QUEEN KAAHUMANU HWY
Analysis Period/Year	EXIST AM 2006	Minor Street	NELHA RD
Comment	2006 EXIST AM		

Input Data

Lane Configuration	SB	NB	EB	WB
Lane 1 (curb)	R	T	R	
Lane 2	T	L	L	
Lane 3				

	SB			NB			EB			WB		
Movement	1 (LT)	2 (TH)	3 (RT)	4 (LT)	5 (TH)	6 (RT)	7 (LT)	8 (TH)	9 (RT)	10 (LT)	11 (TH)	12 (RT)
Volume (veh/h)		870	40	60	675		12		20			
PHF		.9	.9	.9	.9		.9		.9			
Proportion of heavy vehicles, HV		3	3	3	3		3		3			
Flow rate		967	44	67	750		13		22			
Flare storage (# of vehs)									0			
Median storage (# of vehs)								0				

Signal upstream of Movement 2 ________ ft   Movement 5 ________ ft

Length of study period (h) .25

Output Data

	Lane	Movement	Flow Rate (veh/h)	Capacity (veh/h)	v/c	Queue Length (veh)	Control Delay (s)	LOS	Approach Delay and LOS
EB	1	R	22	307	.072	<1	17.6	C	34.9
	2	L	13	74	.177	1	64.2	F	D
	3								
WB	1								
	2								
	3								
		1							
		4	67	709	.094	<1	10.6	B	

CHAPTER 17 - TWSC - UNSIGNALIZED INTERSECTIONS WORKSHEET

Analysis Summary

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/16/2008
Agency or Company	M&E PACIFIC	Major Street	QUEEN KAAHUMANU HWY
Analysis Period/Year	EXIST PM 2006	Minor Street	NELHA RD
Comment	2006 EXIST PM		

Input Data

Lane Configuration	SB	NB	EB	WB
Lane 1 (curb)	R	T	R	
Lane 2	T	L	L	
Lane 3				

	SB			NB			EB			WB		
Movement	1 (LT)	2 (TH)	3 (RT)	4 (LT)	5 (TH)	6 (RT)	7 (LT)	8 (TH)	9 (RT)	10 (LT)	11 (TH)	12 (RT)
Volume (veh/h)		615	18	28	840		39		58			
PHF		.9	.9	.9	.9		.9		.9			
Proportion of heavy vehicles, HV		3	3	3	3		3		3			
Flow rate		683	20	31	933		43		64			
Flare storage (# of vehs)									0			
Median storage (# of vehs)								0				

Signal upstream of Movement 2 ________ ft   Movement 5 ________ ft

Length of study period (h) .25

Output Data

	Lane	Movement	Flow Rate (veh/h)	Capacity (veh/h)	v/c	Queue Length (veh)	Control Delay (s)	LOS	Approach Delay and LOS
EB	1	R	64	447	.143	<1	14.4	B	35
	2	L	107	187	.572	3	47.3	E	D
	3								
WB	1								
	2								
	3								
		1							
		4	31	905	.034	<1	9.1	A	

CHAPTER 17 - TWSC - UNSIGNALIZED INTERSECTIONS WORKSHEET

Analysis Summary

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/16/2008
Agency or Company	M&E PACIFIC	Major Street	QUEEN KAAHUMANU HWY
Analysis Period/Year	EXIST AM 2006	Minor Street	HULIKOA DR
Comment	2006 EXISTING AM		

Input Data

Lane Configuration	SB			NB			EB			WB		
Lane 1 (curb)	T			R						R		
Lane 2	L			T						L		
Lane 3												
	SB			NB			EB			WB		
Movement	1 (LT)	2 (TH)	3 (RT)	4 (LT)	5 (TH)	6 (RT)	7 (LT)	8 (TH)	9 (RT)	10 (LT)	11 (TH)	12 (RT)
Volume (veh/h)	63	770			820	95				65		97
PHF	.9	.9			.9	.9				.9		.9
Proportion of heavy vehicles, HV	3	3			3	3				3		3
Flow rate	70	856			911	106				72		108
Flare storage (# of vehs)												0
Median storage (# of vehs)										0		

Signal upstream of Movement 2 ______ ft Movement 5 ______ ft

Length of study period (h) .25

Output Data

	Lane	Movement	Flow Rate (veh/h)	Capacity (veh/h)	v/c	Queue Length (veh)	Control Delay (s)	LOS	Approach Delay and LOS
EB	1								
	2								
	3								
WB	1	R	108	331	.326	1	21.1	C	107.3
	2	L	72	67	1.073	6	236.8	F	F
	3								
		①	70	678	.103	<1	10.9	B	
		④							

CHAPTER 17 - TWSC - UNSIGNALIZED INTERSECTIONS WORKSHEET

Analysis Summary

General Information		Site Information	
Analyst	WY	Jurisdiction/Date	4/16/2008
Agency or Company	M&E PACIFIC	Major Street	QUEEN KAAHUMANU HWY
Analysis Period/Year	EXIST PM 2006	Minor Street	HULIKOA DR
Comment	2006 EXISTING PM		

Input Data

Lane Configuration	SB			NB			EB			WB		
Lane 1 (curb)	T			R						R		
Lane 2	L			T						L		
Lane 3												
	SB			NB			EB			WB		
Movement	1 (LT)	2 (TH)	3 (RT)	4 (LT)	5 (TH)	6 (RT)	7 (LT)	8 (TH)	9 (RT)	10 (LT)	11 (TH)	12 (RT)
Volume (veh/h)	33	460			770	50				134		90
PHF	.9	.9			.9	.9				.9		.9
Proportion of heavy vehicles, HV	3	3			3	3				3		3
Flow rate	37	511			856	56				149		100
Flare storage (# of vehs)												0
Median storage (# of vehs)										0		

Signal upstream of Movement 2 ______ ft Movement 5 ______ ft

Length of study period (h) .25

Output Data

	Lane	Movement	Flow Rate (veh/h)	Capacity (veh/h)	v/c	Queue Length (veh)	Control Delay (s)	LOS	Approach Delay and LOS
EB	1								
	2								
	3								
WB	1	R	100	356	.281	1	19	C	104
	2	L	149	139	1.075	8	161.1	F	F
	3								
		①	37	780	.047	<1	9.8	A	
		④							

Appendix D

On-Ramp
Level of Service (LOS) Calculations

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2029
Comment:

Site Information

Jurisdiction/Date: 4/14/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 270 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ft
V_D = veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\, f_{HV}\, f_p}$
v_F	16100	.09	1	1449	.9	5	.976	1	1650
v_R	2700	.09		243	.9	5	.976	1	277
v_U		.09		270	.9	5	.976	1	307
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$
L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1650 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1927	See Exhibit 25-7	
v_{R12}	1927	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\, v_R + 0.0078\, v_{12} - 0.00627\, L_A$
D_R = 19.5 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .338 (Exhibit 25-19)
S_R = 60.5 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.5 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$
L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\, v_{12} - 0.009\, L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2015
Comment: 2015 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/9/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 77 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ft
V_D = veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\, f_{HV}\, f_p}$
v_F	22700	.09	1	2043	.9	5	.976	1	2327
v_R	555	.09		50	.9	5	.976	1	57
v_U		.09		77	.9	5	.976	1	87
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$
L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 2327 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2384	See Exhibit 25-7	
v_{R12}	2384	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\, v_R + 0.0078\, v_{12} - 0.00627\, L_A$
D_R = 23.2 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .353 (Exhibit 25-19)
S_R = 60.1 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.1 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$
L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\, v_{12} - 0.009\, L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2020
Comment: 2020 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/9/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 153 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ______ ft
V_D = ______ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	22850	.09	1	2057	.9	5	.976	1	2342
v_R	1670	.09		150	.9	5	.976	1	171
v_U		.09		153	.9	5	.976	1	174
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ______ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ______ (Exhibit 25-5)
v_{12} = 2342 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2513	See Exhibit 25-7	
v_{R12}	2513	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 24.1 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .359 (Exhibit 25-19)
S_R = 59.9 mi/h (Exhibit 25-19)
S_0 = ______ mi/h (Exhibit 25-19)
S = 59.9 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ______ (Equation 25-8 or 25-9)
P_{FD} = ______ using Equation ______ (Exhibit 25-12)
v_{12} = ______ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ______ pc/mi/ln
LOS = ______ (Exhibit 25-4)

Speed Estimation

D_s = ______ (Exhibit 25-19)
S_R = ______ mi/h (Exhibit 25-19)
S_0 = ______ mi/h (Exhibit 25-19)
S = ______ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2029
Comment: 2029 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/14/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 288 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ______ ft
V_D = ______ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	24000	.09	1	2160	.9	5	.976	1	2460
v_R	2700	.09		243	.9	5	.976	1	277
v_U		.09		288	.9	5	.976	1	328
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ______ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ______ (Exhibit 25-5)
v_{12} = 2460 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2737	See Exhibit 25-7	
v_{R12}	2737	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 25.8 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .371 (Exhibit 25-19)
S_R = 59.6 mi/h (Exhibit 25-19)
S_0 = ______ mi/h (Exhibit 25-19)
S = 59.6 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ______ (Equation 25-8 or 25-9)
P_{FD} = ______ using Equation ______ (Exhibit 25-12)
v_{12} = ______ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ______ pc/mi/ln
LOS = ______ (Exhibit 25-4)

Speed Estimation

D_s = ______ (Exhibit 25-19)
S_R = ______ mi/h (Exhibit 25-19)
S_0 = ______ mi/h (Exhibit 25-19)
S = ______ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2015
Comment: 2015 TOTAL AM ON-RAMP

Site Information

Jurisdiction/Date: 4/14/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 23 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	15200	.09	1	1368	.9	5	.976	1	1558
v_R	730	.09		66	.9	5	.976	1	75
v_U		.09		23	.9	5	.976	1	26
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1558 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1633	See Exhibit 25-7	
v_{R12}	1633	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 17.3 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .331 (Exhibit 25-19)
S_R = 60.7 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.7 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2020
Comment: 2020 TOAL AM ON-RAMP

Site Information

Jurisdiction/Date: 4/14/2008
Freeway/Direction of Travel: QKH SOUTHBOUND
Junction: OOMA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 117 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	15300	.09	1	1377	.9	5	.976	1	1568
v_R	1400	.09		126	.9	5	.976	1	143
v_U		.09		117	.9	5	.976	1	133
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1568 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1712	See Exhibit 25-7	
v_{R12}	1712	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 17.9 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .333 (Exhibit 25-19)
S_R = 60.7 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.7 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2015
Comment: 2015 TOTAL AM ON-RAMP

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 23 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft

S_{FF} = 70 mi/h S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	15200	.09	1	1368	.9	5	.976	1	1558
v_R	220	.09		20	.9	5	.976	1	23
v_U		.09		23	.9	5	.976	1	26
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1558 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1581	See Exhibit 25-7	
v_{R12}	1581	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 16.9 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .33 (Exhibit 25-19)
S_R = 60.8 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.8 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2020
Comment: 2020 TOAL AM ON-RAMP

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 117 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge
☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft

S_{FF} = 70 mi/h S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	15700	.09	1	1413	.9	5	.976	1	1609
v_R	220	.09		20	.9	5	.976	1	23
v_U		.09		117	.9	5	.976	1	133
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1609 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1632	See Exhibit 25-7	
v_{R12}	1632	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 17.3 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .331 (Exhibit 25-19)
S_R = 60.7 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.7 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2029
Comment:

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 270 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	17950	.09	1	1616	.9	5	.976	1	1840
v_R	275	.09		25	.9	5	.976	1	28
v_U		.09		270	.9	5	.976	1	307
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$
L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 1840 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	1868	See Exhibit 25-7	
v_{R12}	1868	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 19.2 pc/mi/ln
LOS = B (Exhibit 25-4)

Speed Estimation

M_s = .336 (Exhibit 25-19)
S_R = 60.6 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.6 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$
L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2015
Comment: 2015 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 77 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	22700	.09	1	2043	.9	5	.976	1	2327
v_R	770	.09		69	.9	5	.976	1	79
v_U		.09		77	.9	5	.976	1	87
v_D								1	

Merge Areas

Estimation of v_{12}

$v_{12} = v_F * P_{FM}$
L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 2327 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2406	See Exhibit 25-7	
v_{R12}	2406	4600: All	

Level-of-Service Determination (if not F)

$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$
D_R = 23.3 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .354 (Exhibit 25-19)
S_R = 60.1 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 60.1 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$v_{12} = v_R + (v_F - v_R)P_{FD}$
L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$
D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2020
Comment: 2020 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 153 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	23800	.09	1	2142	.9	5	.976	1	2439
v_R	770	.09		69	.9	5	.976	1	79
v_U		.09		153	.9	5	.976	1	174
v_D								1	

Merge Areas

Estimation of v_{12}

$$v_{12} = v_F * P_{FM}$$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 2439 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2518	See Exhibit 25-7	
v_{R12}	2518	4600: All	

Level-of-Service Determination (if not F)

$$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$$

D_R = 24.2 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .36 (Exhibit 25-19)
S_R = 59.9 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 59.9 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$$

D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

CHAPTER 25 - RAMPS AND RAMP JUNCTIONS WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2029
Comment: 2029 TOTAL PM ON-RAMP

Site Information

Jurisdiction/Date: 4/19/2008
Freeway/Direction of Travel: QKH SB NELHA
Junction: NELHA ACCESS

☐ Operational (LOS) ☐ Design (L_A, L_D, or N) ☑ Planning (LOS) ☐ Planning (L_A, L_D, or N)

Inputs

Upstream Adjacent Ramp: ☑ Yes ☐ On ☐ No ☑ Off
L_{up} = 450 ft
V_u = 288 veh/h

Freeway terrain: Level
Ramp terrain: Level
Ramp Type: ☑ Merge ☐ Diverge ☑ Right side ☐ Left side
Number of freeway lanes: 2
Number of ramp lanes: 1
Length of ramp roadway: 140 ft
S_{FF} = 70 mi/h
S_{FR} = 35 mi/h

Downstream Adjacent Ramp: ☐ Yes ☐ On ☑ No ☐ Off
L_{down} = ____ ft
V_D = ____ veh/h

Conversion to pc/h Under Base Conditions

(pc/h)	AADT (veh/day)	K	D	V (veh/h)	PHF	% HV	f_{HV}	f_p	$v = \frac{V}{PHF\ f_{HV}\ f_p}$
v_F	26300	.09	1	2367	.9	5	.976	1	2696
v_R	840	.09		76	.9	5	.976	1	86
v_U		.09		288	.9	5	.976	1	328
v_D								1	

Merge Areas

Estimation of v_{12}

$$v_{12} = v_F * P_{FM}$$

L_{EQ} = ____ (Equation 25-2 or 25-3)
P_{FM} = 1 using Equation ____ (Exhibit 25-5)
v_{12} = 2696 pc/h

Capacity Checks

	Actual	Maximum	LOS F?
v_{FO}	2782	See Exhibit 25-7	
v_{R12}	2782	4600: All	

Level-of-Service Determination (if not F)

$$D_R = 5.475 + 0.00734\ v_R + 0.0078\ v_{12} - 0.00627\ L_A$$

D_R = 26.3 pc/mi/ln
LOS = C (Exhibit 25-4)

Speed Estimation

M_s = .374 (Exhibit 25-19)
S_R = 59.5 mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = 59.5 mi/h (Equation 25-14)

Diverge Areas

Estimation of v_{12}

$$v_{12} = v_R + (v_F - v_R)P_{FD}$$

L_{EQ} = ____ (Equation 25-8 or 25-9)
P_{FD} = ____ using Equation ____ (Exhibit 25-12)
v_{12} = ____ pc/h

Capacity Checks

	Actual	Maximum	LOS F?
$v_{FI} = v_F$		See Exhibit 25-14	
v_{12}		4400: All	
$v_{FO} = v_F - v_R$		See Exhibit 25-14	
v_R		See Exhibit 25-3	

Level-of-Service Determination (if not F)

$$D_R = 4.252 + 0.0086\ v_{12} - 0.009\ L_D$$

D_R = ____ pc/mi/ln
LOS = ____ (Exhibit 25-4)

Speed Estimation

D_s = ____ (Exhibit 25-19)
S_R = ____ mi/h (Exhibit 25-19)
S_0 = ____ mi/h (Exhibit 25-19)
S = ____ mi/h (Equation 25-15)

Appendix E

Highway
Level of Service (LOS) Calculations

CHAPTER 20 - DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: EX AM 2006
Comment: 2006 EXISTING AM SB

Site Information

Jurisdiction/Date: 4/19/2008
Highway: Q. KAAHUMANU HWY SB
From/To: KAIMINANI TO KOHANAIKI

☐ Operational (LOS) ☐ Design (v_p) ☑ Planning (LOS) ☐ Planning (v_p)

Input Data

Opposing direction / Analysis direction
Shoulder width 6 ft
Lane width 12 ft
Lane width 12 ft
Shoulder width 6 ft
Segment length, L_t 2 mi

Show North Arrow

☑ Class I highway ☐ Class II highway
Terrain ☐ Level ☐ Rolling
Grade Length ____ mi Up/down ____ %
Peak-hour factor, PHF .95
% Trucks and buses, P_T 3 %
% Recreational vehicles, P_R 0 %
% No-passing zone 50 %
Access points/mi 2 /mi

Analysis direction volume, V_d 865 veh/h
Opposing direction volume, V_o 915 veh/h

Average Travel Speed

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-9 or 20-15)	1.1	1.1
Passenger-car equivalent for RVs, E_R (Exhibit 20-9 or 20-17)	1	1
Heavy-vehicle adjustment factor,[5] f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	.997	.997
Grade adjustment factor,[1] f_G (Exhibit 20-6 or 20-12)	1	1
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$	913	966

Free-Flow Speed from Field Measurement	Estimated Free-Flow Speed
Field measured speed,[3] S_{FM} 55 mi/h	Base free-flow speed,[3] BFFS ____ mi/h
Observed volume,[3] V_f 865 veh/h	Adj. for lane width and shoulder width,[3] f_{LS} (Exh. 20-5) 0 mi/h
Free-flow speed, FFS_d 61.7 mi/h	Adj. for access points,[3] f_A (Exhibit 20-6) .5 mi/h
$FFS_d = S_{FM} + 0.00776\left(\frac{V_f}{f_{HV}}\right)$	Free-flow speed, FFS_d 61.7 mi/h
	$FFS_d = BFFS - f_{LS} - f_A$

Adjustment for no-passing zones, f_{np} (mi/h) (Exhibit 20-19)	.9
Average travel speed, ATS_d (mi/h) $ATS_d = FFS_d - 0.00776(v_d + v_o) - f_{np}$	46.3

Percent Time-Spent-Following

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-10 or 20-16)	1	1
Passenger-car equivalent for RVs, E_R (Exhibit 20-10 or 20-16)	1	1
Heavy-vehicle adjustment factor, f_{HV} $f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$	1	1
Grade adjustment factor,[1] f_G (Exhibit 20-8 or 20-14)	1	1
Directional flow rate,[2] v_i (pc/h) $v_i = \frac{V_i}{PHF * f_{HV} * f_G}$	911	963

Base percent time-spent-following,[4] $BPTSF_d$ (%) $BPTSF_d = 100(1 - e^{av_d^b})$	87.8
Adjustment for no-passing zone, f_{np} (Exhibit 20-20)	3.6
Percent time-spent-following, $PTSF_d$ (%) $PTSF_d = BPTSF_d + f_{np}$	91.4

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 or 20-4)	E
Volume to capacity ratio, v/c $v/c = \frac{v_d}{1700}$	.54
Peak 15-min vehicle-miles of travel, VMT_{15} (veh-mi) $VMT_{15} = 0.25L_t\left(\frac{V_d}{PHF}\right)$	455
Peak-hour vehicle-miles of travel, VMT_{60} (veh-mi) $VMT_{60} = V_d * L_t$	1730
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VMT_{15}}{ATS_d}$	9.8

Notes

1. If the highway is extended segment (level) or rolling terrain, f_G = 1.0
2. If v_i (v_d or v_o) ≥ 1,700 pc/h, terminate analysis—the LOS is F.
3. For the analysis direction only.
4. Exhibit 20-21 provides factors a and b.
5. Use alternative Equation 20-14 if some trucks operate at crawl speeds on a specific downgrade.

CHAPTER 20 - DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WITH PASSING LANE WORKSHEET

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: EX AM 2006
Comment: 2006 EXISTING AM SB

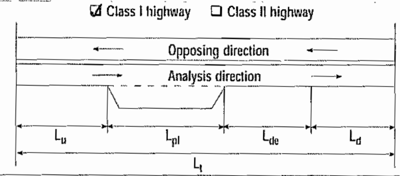
Site Information

Jurisdiction/Date: 4/19/2008
Highway: Q. KAAHUMANU HWY SB
From/To: KAIMINANI TO KOHANAIKI

☐ Operational (LOS) ☐ Design (v_p) ☑ Planning (LOS) ☐ Planning (v_p)

Input Data

☑ Class I highway ☐ Class II highway



Show North Arrow

Total length of analysis segment, L_t (mi)	2
Length of two-lane highway upstream of the passing lane, L_u (mi)	0
Length of passing lane including tapers, L_{pl} (mi)	
Average travel speed, ATS_d (from Directional Two-Lane Highway Segment Worksheet)	46.3
Percent time-spent-following, $PTSF_d$ (from Directional Two-Lane Highway Segment Worksheet)	91.4
Level of service,[1] LOS_d (from Directional Two-Lane Highway Segment Worksheet)	E

Average Travel Speed

Downstream length of two-lane highway within effective length of passing lane for average travel speed, L_{de} (mi) (Exhibit 20-23)	1.7
Length of two-lane highway downstream of effective length of the passing lane for average travel speed, L_d (mi) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on average speed, f_{pl} (Exhibit 20-24)	1.11
Average travel speed including passing lane,[2] ATS_{pl} $ATS_{pl} = \frac{ATS_d * L_t}{L_u + L_d + \frac{L_{pl}}{f_{pl}} + \frac{2L_{de}}{1 + f_{pl}}}$	

Percent Time-Spent-Following

Downstream length of two-lane highway within effective length of passing lane for percent time-spent-following, L_{de} (mi) (Exhibit 20-23)	5.7
Length of two-lane highway downstream of effective length of the passing lane for percent time-spent-following, L_d (mi) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on percent time-spent-following, f_{pl} (Exhibit 20-24)	.62
Percent time-spent-following including passing lane,[3] $PTSF_{pl}$ (%) $PTSF_{pl} = \frac{PTSF_d\left[L_u + L_d + f_{pl}L_{pl} + \left(\frac{1 + f_{pl}}{2}\right)L_{de}\right]}{L_t}$	

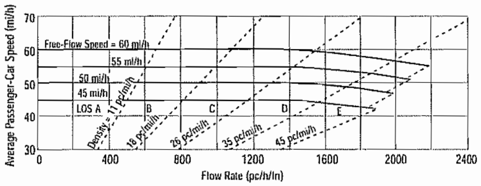
Level of Service and Other Performance Measures [4]

Level of service including passing lane, LOS_{pl} (Exhibits 20-3 or 20-4)	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VMT_{15}}{ATS_{pl}}$	

Notes

1. If LOS_d = F, passing lane analysis cannot be performed.
2. If L_d < 0, use alternative Equation 20-22.
3. If L_d < 0, use alternative Equation 20-20.
4. v/c, VMT_{15}, and VMT_{60} are calculated on Directional Two-Lane Highway Segment Worksheet.

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2015
Comment: 2015 AM AMBIENT

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1370 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☐ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 783 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 14.24

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

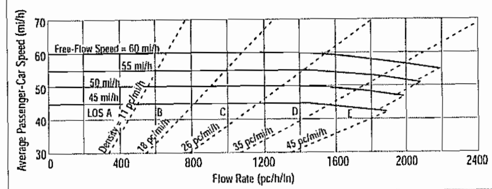
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2015
Comment: 2015 AM TOTAL

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1395 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☐ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 797 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 14.5

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

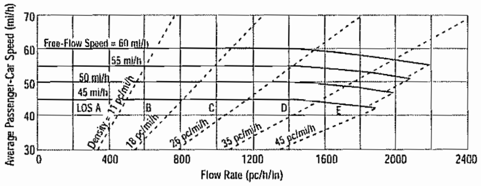
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2020
Comment: 2020 AM AMBIENT

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1370 veh/h
Annual avg. daily traffic, AADT: ______ veh/day
Peak-hour proportion of AADT, K: ______
Peak-hour direction proportion, D: ______
DDHV = AADT * K * D: ______ veh/h
Driver type: ☐ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ______ mi Up/Down ______ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ______ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ______ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ______ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 783 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 14.24

Design (v_p) or Planning (v_p)
LOS: ______
v_p: ______ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ______ veh/h
S: ______ mi/h
$D = v_p/S$: ______ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ______ pc/h/ln
LOS: ______

Design (N) or Planning (N) 2nd Iteration
N: ______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ______ pc/h/ln
LOS: ______
S: ______ mi/h
$D = v_p/S$: ______ pc/mi/ln

Glossary

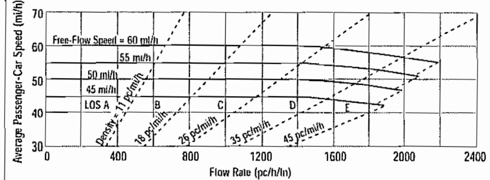
N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

HiCAP 2000 ™

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2020
Comment: 2020 AM TOTAL

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1435 veh/h
Annual avg. daily traffic, AADT: ______ veh/day
Peak-hour proportion of AADT, K: ______
Peak-hour direction proportion, D: ______
DDHV = AADT * K * D: ______ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ______ mi Up/Down ______ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ______ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ______ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ______ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 820 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 14.92

Design (v_p) or Planning (v_p)
LOS: ______
v_p: ______ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ______ veh/h
S: ______ mi/h
$D = v_p/S$: ______ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ______ pc/h/ln
LOS: ______

Design (N) or Planning (N) 2nd Iteration
N: ______ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ______ pc/h/ln
LOS: ______
S: ______ mi/h
$D = v_p/S$: ______ pc/mi/ln

Glossary

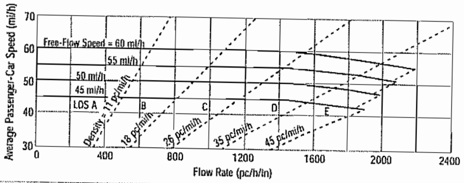
N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

HiCAP 2000 ™

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB AM 2029
Comment: 2029 AM AMBIENT

Site Information

Jurisdiction/Date: 4/9/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1445 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 826 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 15.02

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

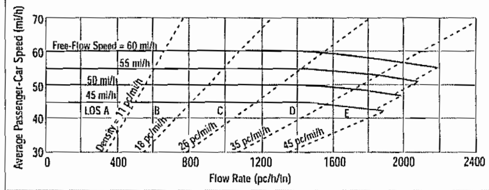
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT AM 2029
Comment: 2029 AM TOTAL

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 1640 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 938 pc/h/ln
S: B mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 17.05

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 20 - DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WORKSHEET

General Information

Analyst WY
Agency or Company M&E PACIFIC
Analysis Period/Year EX PM 2006
Comment 2006 EXISTING PM SB

Site Information

Jurisdiction/Date 4/19/2008
Highway Q. KAAHUMANU HWY SB
From/To KAIMINANI TO KOHANAIKI

☐ Operational (LOS) ☐ Design (v_p) ☑ Planning (LOS) ☐ Planning (v_p)

Input Data

Shoulder width 6 ft
Opposing direction — Lane width 12 ft
Analysis direction — Lane width 12 ft
Shoulder width 6 ft
Segment length, L_t 2 mi

Show North Arrow

☑ Class I highway ☐ Class II highway
Terrain ☐ Level ☐ Rolling
Grade Length ____ mi Up/down ____ %
Peak-hour factor, PHF .95
% Trucks and buses, P_T 3 %
% Recreational vehicles, P_R 0 %
% No-passing zone 50 %
Access points/mi 2 /mi

Analysis direction volume, V_d 825 veh/h
Opposing direction volume, V_o 830 veh/h

Average Travel Speed

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-9 or 20-15)	1.1	1.1
Passenger-car equivalent for RVs, E_R (Exhibit 20-9 or 20-17)	1	1
Heavy-vehicle adjustment factor,[5] f_{HV} $f_{HV}=\frac{1}{1+P_T(E_T-1)+P_R(E_R-1)}$	.997	.997
Grade adjustment factor,[1] f_G (Exhibit 20-6 or 20-12)	1	1
Directional flow rate,[2] v_i (pc/h) $v_i=\frac{V_i}{PHF * f_{HV} * f_G}$	871	876

Free-Flow Speed from Field Measurement		Estimated Free-Flow Speed	
Field measured speed,[3] S_{FM}	55 mi/h	Base free-flow speed,[3] BFFS	mi/h
Observed volume,[3] V_f	865 veh/h	Adj. for lane width and shoulder width,[3] f_{LS} (Exh. 20-5)	0 mi/h
Free-flow speed, FFS_d $FFS_d = S_{FM} + 0.00776(\frac{V_f}{f_{HV}})$	61.7 mi/h	Adj. for access points,[3] f_A (Exhibit 20-6)	.5 mi/h
		Free-flow speed, FFS_d $FFS_d = BFFS - f_{LS} - f_A$	61.7 mi/h

Adjustment for no-passing zones, f_{np} (mi/h) (Exhibit 20-19)	1
Average travel speed, ATS_d (mi/h) $ATS_d = FFS_d - 0.00776(v_d + v_o) - f_{np}$	47.2

Percent Time-Spent-Following

	Analysis Direction (d)	Opposing Direction (o)
Passenger-car equivalent for trucks, E_T (Exhibit 20-10 or 20-16)	1	1
Passenger-car equivalent for RVs, E_R (Exhibit 20-10 or 20-16)	1	1
Heavy-vehicle adjustment factor, f_{HV} $f_{HV}=\frac{1}{1+P_T(E_T-1)+P_R(E_R-1)}$	1	1
Grade adjustment factor,[1] f_G (Exhibit 20-8 or 20-14)	1	1
Directional flow rate,[2] v_i (pc/h) $v_i=\frac{V_i}{PHF * f_{HV} * f_G}$	868	874
Base percent time-spent-following,[4] $BPTSF_d$ (%) $BPTSF_d = 100(1 - e^{av_d^b})$	86.6	
Adjustment for no-passing zone, f_{np} (Exhibit 20-20)	4.5	
Percent time-spent-following, $PTSF_d$ (%) $PTSF_d = BPTSF_d + f_{np}$	91.1	

Level of Service and Other Performance Measures

Level of service, LOS (Exhibit 20-3 or 20-4)	E
Volume to capacity ratio, v/c $v/c = \frac{V_d}{1700}$	.51
Peak 15-min vehicle-miles of travel, VMT_{15} (veh-mi) $VMT_{15} = 0.25L_t(\frac{V_d}{PHF})$	434
Peak-hour vehicle-miles of travel, VMT_{60} (veh-mi) $VMT_{60} = V_d * L_t$	1650
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VMT_{15}}{ATS_d}$	9.2

Notes

1. If the highway is extended segment (level) or rolling terrain, $f_G = 1.0$
2. If v_i (v_d or v_o) ≥ 1,700 pc/h, terminate analysis—the LOS is F.
3. For the analysis direction only.
4. Exhibit 20-21 provides factors a and b.
5. Use alternative Equation 20-14 if some trucks operate at crawl speeds on a specific downgrade.

CHAPTER 20 - DIRECTIONAL TWO-LANE HIGHWAY SEGMENT WITH PASSING LANE WORKSHEET

General Information

Analyst WY
Agency or Company M&E PACIFIC
Analysis Period/Year EX PM 2006
Comment 2006 EXISTING PM SB

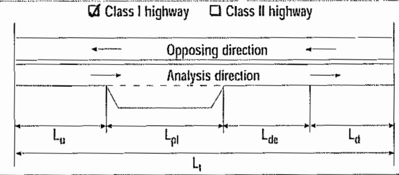
Site Information

Jurisdiction/Date 4/19/2008
Highway Q. KAAHUMANU HWY SB
From/To KAIMINANI TO KOHANAIKI

☐ Operational (LOS) ☐ Design (v_p) ☑ Planning (LOS) ☐ Planning (v_p)

Input Data

☑ Class I highway ☐ Class II highway



Show North Arrow

Total length of analysis segment, L_t (mi)	2
Length of two-lane highway upstream of the passing lane, L_u (mi)	
Length of passing lane including tapers, L_{pl} (mi)	
Average travel speed, ATS_d (from Directional Two-Lane Highway Segment Worksheet)	47.2
Percent time-spent-following, $PTSF_d$ (from Directional Two-Lane Highway Segment Worksheet)	91.1
Level of service,[1] LOS_d (from Directional Two-Lane Highway Segment Worksheet)	E

Average Travel Speed

Downstream length of two-lane highway within effective length of passing lane for average travel speed, L_{de} (mi) (Exhibit 20-23)	1.7
Length of two-lane highway downstream of effective length of the passing lane for average travel speed, L_d (mi) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on average speed, f_{pl} (Exhibit 20-24)	1.11
Average travel speed including passing lane,[2] ATS_{pl} $ATS_{pl} = \frac{ATS_d * L_t}{L_u + L_d + \frac{L_{pl}}{f_{pl}} + \frac{2L_{de}}{1+f_{pl}}}$	

Percent Time-Spent-Following

Downstream length of two-lane highway within effective length of passing lane for percent time-spent-following, L_{de} (mi) (Exhibit 20-23)	5.7
Length of two-lane highway downstream of effective length of the passing lane for percent time-spent-following, L_d (mi) $L_d = L_t - (L_u + L_{pl} + L_{de})$	
Adj. factor for the effect of passing lane on percent time-spent-following, f_{pl} (Exhibit 20-24)	.62
Percent time-spent-following including passing lane,[3] $PTSF_{pl}$ (%) $PTSF_{pl} = \frac{PTSF_d\left[L_u + L_d + f_{pl}L_{pl} + \left(\frac{1+f_{pl}}{2}\right)L_{de}\right]}{L_t}$	

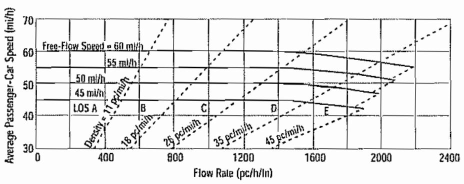
Level of Service and Other Performance Measures [4]

Level of service including passing lane, LOS_{pl} (Exhibits 20-3 or 20-4)	
Peak 15-min total travel time, TT_{15} (veh-h) $TT_{15} = \frac{VMT_{15}}{ATS_{pl}}$	

Notes

1. If LOS_d = F, passing lane analysis cannot be performed.
2. If L_d < 0, use alternative Equation 20-22.
3. If L_d < 0, use alternative Equation 20-20.
4. v/c, VMT_{15}, and VMT_{60} are calculated on Directional Two-Lane Highway Segment Worksheet.

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB PM 2015
Comment: 2015 PM AMBIENT

Site Information

Jurisdiction/Date: 4/9/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 2045 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☐ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 1169 pc/h/ln
S: C mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 21.26

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

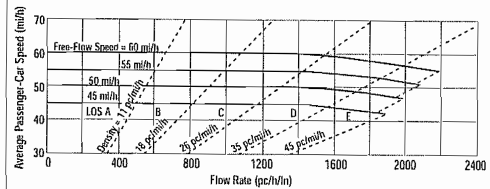
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2015
Comment: 2015 PM TOTAL

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 2120 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 1212 pc/h/ln
S: C mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 22.04

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

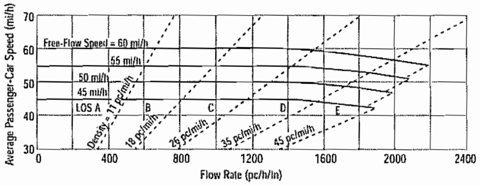
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst WY
Agency or Company M&E PACIFIC
Analysis Period/Year AMB PM 2020
Comment 2020 PM AMBIENT

Site Information

Jurisdiction/Date 4/9/2008
Highway/Direction of Travel QUEEN KAAHUMANU HWY
From/To KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V 2065 veh/h
Annual avg. daily traffic, AADT ______ veh/day
Peak-hour proportion of AADT, K ______
Peak-hour direction proportion, D ______
DDHV = AADT * K * D ______ veh/h
Driver type
☐ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF .9
% Trucks and buses, P_T 5
% RVs, P_R 2
General terrain
☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ______ mi Up/Down ______ %
Number of lanes 2

Calculate Flow Adjustments

f_p 1
E_T 1.5
E_R 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ .972

Speed Inputs

Lane width, LW 10 ft
Total lateral clearance, TLC 12 ft
Access points, A ______ A/mi
Median type, M ☐ Undivided ☑ Divided
FFS (measured) ______ mi/h
Base free-flow Speed, BFFS 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW} 6.6 mi/h
f_{LC} 0 mi/h
f_A ______ mi/h
f_M 0 mi/h
FFS = BFFS $- f_{LW} - f_{LC} - f_A - f_M$ 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ 1180 pc/h/ln
S C mi/h
D = v_p/S 55 pc/mi/ln
LOS 21.46

Design (v_p) or Planning (v_p)
LOS ______
v_p ______ pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p ______ veh/h
S ______ mi/h
D = v_p/S ______ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N ______ assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ ______ pc/h/ln
LOS ______

Design (N) or Planning (N) 2nd Iteration
N ______ assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ ______ pc/h/ln
LOS ______
S ______ mi/h
D = v_p/S ______ pc/mi/ln

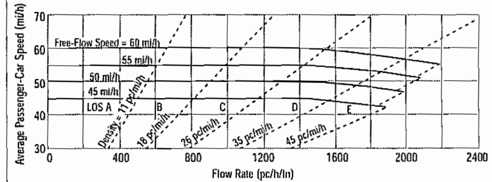
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst WY
Agency or Company M&E PACIFIC
Analysis Period/Year TOT PM 2020
Comment 2020 PM TOTAL

Site Information

Jurisdiction/Date 4/14/2008
Highway/Direction of Travel QUEEN KAAHUMANU HWY
From/To KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V 2200 veh/h
Annual avg. daily traffic, AADT ______ veh/day
Peak-hour proportion of AADT, K ______
Peak-hour direction proportion, D ______
DDHV = AADT * K * D ______ veh/h
Driver type
☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF .9
% Trucks and buses, P_T 5
% RVs, P_R 2
General terrain
☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ______ mi Up/Down ______ %
Number of lanes 2

Calculate Flow Adjustments

f_p 1
E_T 1.5
E_R 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$ .972

Speed Inputs

Lane width, LW 10 ft
Total lateral clearance, TLC 12 ft
Access points, A ______ A/mi
Median type, M ☐ Undivided ☑ Divided
FFS (measured) ______ mi/h
Base free-flow Speed, BFFS 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW} 6.6 mi/h
f_{LC} 0 mi/h
f_A ______ mi/h
f_M 0 mi/h
FFS = BFFS $- f_{LW} - f_{LC} - f_A - f_M$ 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ 1258 pc/h/ln
S C mi/h
D = v_p/S 55 pc/mi/ln
LOS 22.87

Design (v_p) or Planning (v_p)
LOS ______
v_p ______ pc/h/ln
V = v_p * PHF * N * f_{HV} * f_p ______ veh/h
S ______ mi/h
D = v_p/S ______ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N ______ assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ ______ pc/h/ln
LOS ______

Design (N) or Planning (N) 2nd Iteration
N ______ assumed
$v_p = \frac{V \text{ or DDHV}}{PHF * N * f_{HV} * f_p}$ ______ pc/h/ln
LOS ______
S ______ mi/h
D = v_p/S ______ pc/mi/ln

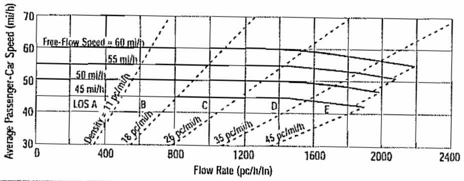
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: AMB PM 2029
Comment: 2029 PM AMBIENT

Site Information

Jurisdiction/Date: 4/9/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 2175 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 1243 pc/h/ln
S: C mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 22.61

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

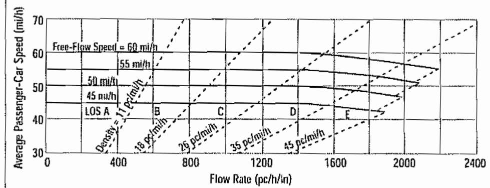
Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7

CHAPTER 21 - MULTILANE HIGHWAYS WORKSHEET



Application	Input	Output
Operational (LOS)	FFS, N, v_p	LOS, S, D
Design (N)	FFS, LOS, v_p	N, S, D
Design (v_p)	FFS, LOS, N	v_p, S, D
Planning (LOS)	FFS, N, AADT	LOS, S, D
Planning (N)	FFS, LOS, AADT	N, S, D
Planning (v_p)	FFS, LOS, N	v_p, S, D

General Information

Analyst: WY
Agency or Company: M&E PACIFIC
Analysis Period/Year: TOT PM 2029
Comment: 2029 PM TOTAL

Site Information

Jurisdiction/Date: 4/14/2008
Highway/Direction of Travel: QUEEN KAAHUMANU HWY
From/To: KAIMINANI TO KOHANAIK

☑ Oper. (LOS) ☐ Des. (N) ☐ Des. (v_p) ☐ Plan. (LOS) ☐ Plan. (N) ☐ Plan. (v_p)

Flow Inputs

Volume, V: 2450 veh/h
Annual avg. daily traffic, AADT: ____ veh/day
Peak-hour proportion of AADT, K: ____
Peak-hour direction proportion, D: ____
DDHV = AADT * K * D: ____ veh/h
Driver type: ☑ Commuter/Weekday ☐ Recreational/Weekend

Peak-hour factor, PHF: .9
% Trucks and buses, P_T: 5
% RVs, P_R: 2
General terrain: ☑ Level ☐ Rolling ☐ Mountainous
Grade: Length ____ mi Up/Down ____ %
Number of lanes: 2

Calculate Flow Adjustments

f_p: 1
E_T: 1.5
E_R: 1.2
$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$: .972

Speed Inputs

Lane width, LW: 10 ft
Total lateral clearance, TLC: 12 ft
Access points, A: ____ A/mi
Median type, M: ☐ Undivided ☑ Divided
FFS (measured): ____ mi/h
Base free-flow Speed, BFFS: 60 mi/h

Calculate Speed Adjustments and FFS

f_{LW}: 6.6 mi/h
f_{LC}: 0 mi/h
f_A: ____ mi/h
f_M: 0 mi/h
$FFS = BFFS - f_{LW} - f_{LC} - f_A - f_M$: 55 mi/h

Operational, Planning (LOS); Design, Planning (v_p)

Operational (LOS) or Planning (LOS)
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: 1401 pc/h/ln
S: C mi/h
$D = v_p/S$: 55 pc/mi/ln
LOS: 25.47

Design (v_p) or Planning (v_p)
LOS: ____
v_p: ____ pc/h/ln
$V = v_p * PHF * N * f_{HV} * f_p$: ____ veh/h
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Design, Planning (N)

Design (N) or Planning (N) 1st Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____

Design (N) or Planning (N) 2nd Iteration
N: ____ assumed
$v_p = \frac{V \text{ or } DDHV}{PHF * N * f_{HV} * f_p}$: ____ pc/h/ln
LOS: ____
S: ____ mi/h
$D = v_p/S$: ____ pc/mi/ln

Glossary

N - Number of lanes
V - Hourly volume
v_p - Flow rate
LOS - Level of service
DDHV - Directional design-hour volume
S - Speed
D - Density
FFS - Free-flow speed
BFFS - Base free-flow speed

Factor Location

E_T - Exhibit 21-8, 21-9, 21-11
E_R - Exhibit 21-8, 21-10
f_p - Page 21-11
LOS, S, FFS, v_p - Exhibit 21-2, 21-3
f_{LW} - Exhibit 21-4
f_{LC} - Exhibit 21-5
f_M - Exhibit 21-6
f_A - Exhibit 21-7