Supplement to Final Environmental Assessment

July 30, 2010

WAIKIKI LANDING

HONEY BEE USA, INC.
ALA WAI HARBOR FAST LANDS PROJECT

Parcel 1 (Boatyard Repair Site)
TMK: (1) 2-6-10:5 and 16 and a portion of (1) 2-6-10:3

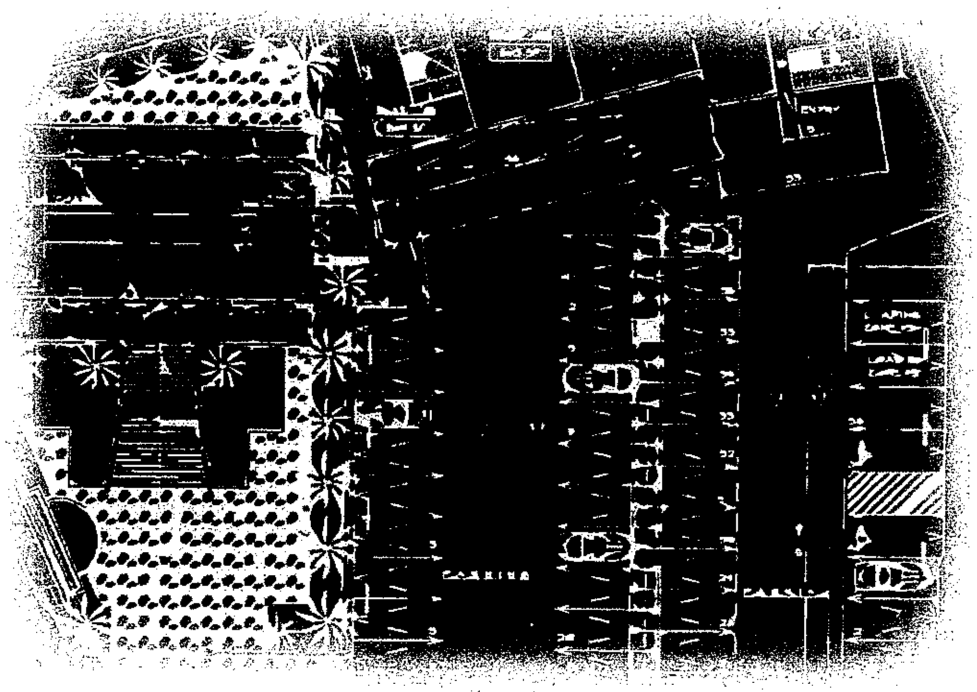
Parcel 2 (Fuel Dock Site)
TMK: (1) 2-3-37:20

This is a supplement to the Final Environmental Assessment submitted by Honey Bee USA, Inc. for its proposed project named "Waikiki Landing" located in the Ala Wai Boat Harbor. This supplement has been issued by Honey Bee USA, Inc. to attach: (a) a draft of a Traffic Impact Report prepared by Wilson Okamoto Corporation in July, 2010, which was received by Honey Bee USA, Inc. on July 28, 2010 and (b) a comment letter to the Draft Environmental Assessment that the applicant was requested to respond to.

Exhibit "AA"
Draft Traffic Impact Report – Waikiki Landing
Prepared by Wilson Okamoto Corporation
July 2010

Traffic Impact Report

Waikiki Landing



Prepared for:
Kiuchi, Nakamoto & Tanaka

Prepared by:
Wilson Okamoto Corporation

July 2010

TRAFFIC IMPACT REPORT

FOR THE

WAIKIKI LANDING

Prepared for:

Kiuchi, Nakamoto & Tanaka
1001 Bishop Street, Suite 1090
Honolulu, HI 96813

Prepared by:

Wilson Okamoto Corporation
1907 South Beretania Street, Suite 400
Honolulu, Hawaii 96826
WOC Ref: 8099-01

July 2010

TABLE OF CONTENTS

LIST OF EXHIBITS

LIST OF APPENDICES

I. INTRODUCTION

A. Purpose of Study

The purpose of this study is to identify and assess the traffic impacts resulting from the proposed Waikiki Landing development located adjacent to Ala Moana Boulevard in Waikiki on the island of Oahu. The proposed development includes retail, restaurant, and office space, as well as, facilities to hold wedding ceremonies on-site.

B. Scope of Study

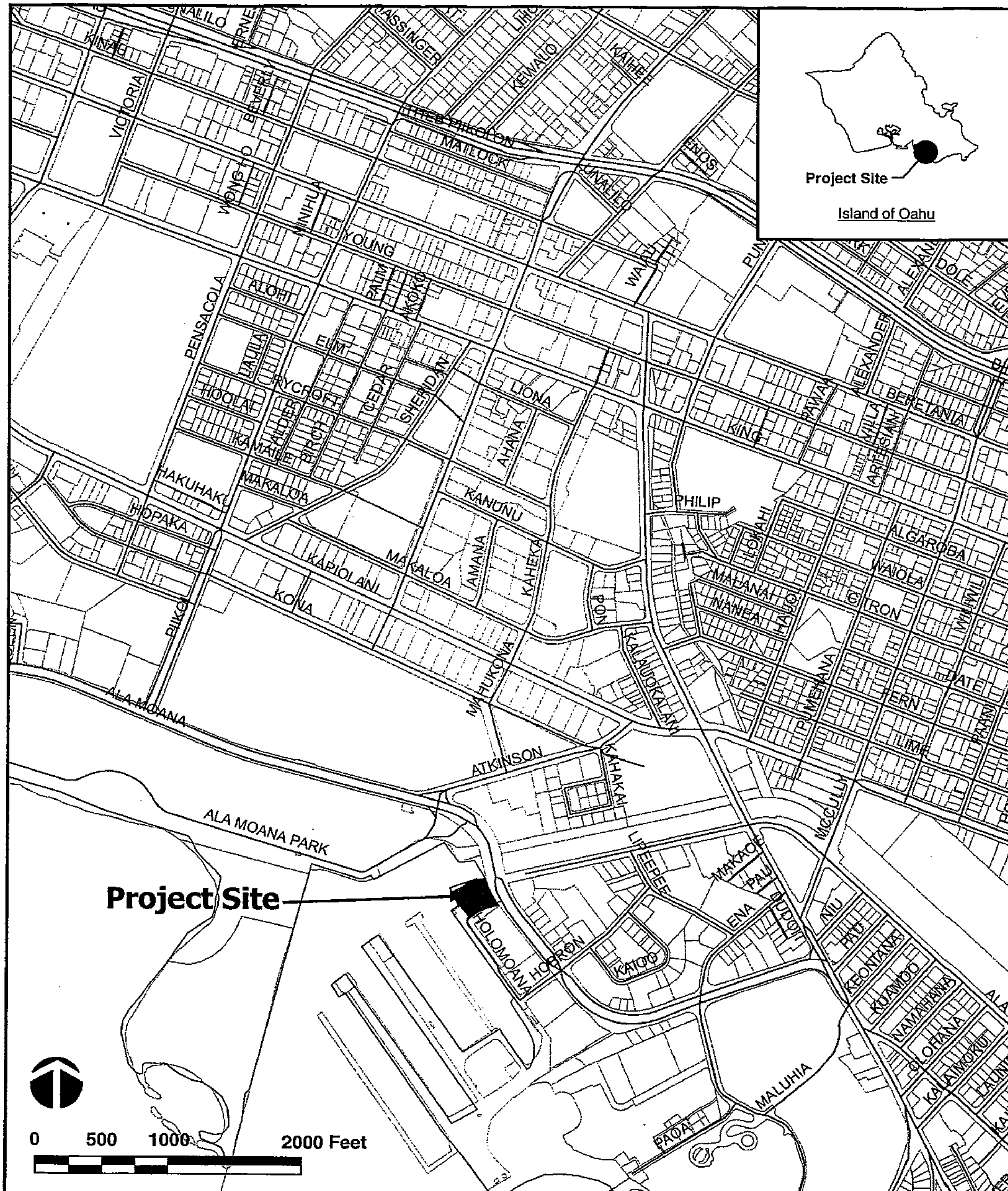
This report presents the findings and conclusions of the traffic study, the scope of which includes:

1. Description of the proposed project.
2. Evaluation of existing roadway and traffic operations in the vicinity.
3. Analysis of future roadway and traffic conditions without the proposed project.
4. Analysis and development of trip generation characteristics for the proposed project.
5. Superimposing site-generated traffic over future traffic conditions.
6. The identification and analysis of traffic impacts resulting from the proposed project.
7. Recommendations of improvements, if appropriate, that would mitigate the traffic impacts resulting from the proposed project.

II. PROJECT DESCRIPTION

A. Location

The proposed project will be located adjacent to Ala Moana Boulevard near Holomoana Street in the Ala Wai Boat Harbor on the island of Oahu (See Figure 1). The project site is further identified as Tax Map Keys: 2-6-10: por. 3, 5, and 16. Access to project site will be provided off Holomoana Street.



WILSON OKAMOTO CORPORATION ENGINEERS - PLANNERS	**WAIKIKI LANDING**	FIGURE
	LOCATION AND VICINITY MAP	1

B. Project Characteristics

The project site currently houses a boatyard repair facility, convenience store, and fueling station. The proposed project entails the renovation and upgrade of the existing boatyard repair facility and the redevelopment of the remainder of the existing site. The development is expected to include the following:

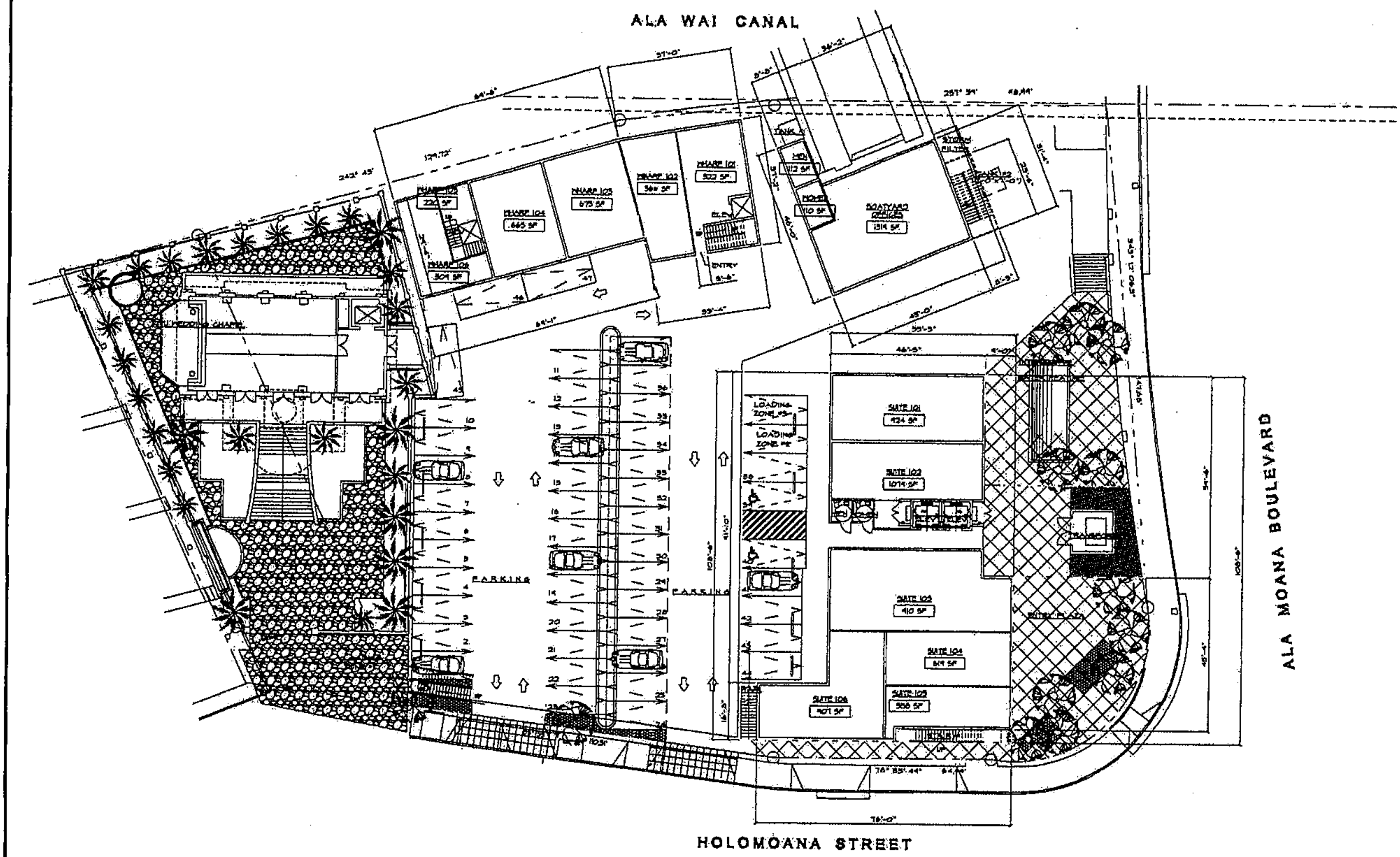
- Boatyard Building
  - ~10,894 square feet of retail space
  - ~9,287 square feet of restaurant space
  - ~1,877 square feet of office space
- Wharf Building
  - ~6,098 square feet of restaurant space
  - ~1,319 square feet of office space
- Canoe House (~4,094 square feet of space for wedding ceremonies to be held on-site)
- Diamond Vista Building
  - ~3,007 square feet of retail space
  - ~1,583 square feet of office space
  - ~6,048 square feet of space for wedding ceremonies to be held on-site
- ~At-Grade Parking

The proposed wedding facilities with the Canoe House and Diamond Vista Building should be able to accommodate up to 6 small weddings each per day (approximately 12 guests per wedding). The proposed project is expected to be completed by the Year 2013 with access to be provided off Holomoana Street. Figure 2 shows the proposed site plan.

III. EXISTING TRAFFIC CONDITIONS

A. Area Roadway System

The proposed project site is located adjacent to Ala Moana Boulevard, a predominantly six-lane, two-way State of Hawaii roadway that, with Kalakaua Avenue, provides access to and from Waikiki. Northwest of the project site, Ala Moana Boulevard intersects Atkinson Drive. At this signalized intersection, the eastbound approach of Ala Moana Boulevard has two left-turn lanes, two through lanes, and a shared through and right-turn lane while the westbound approach has exclusive turning lanes and three through lanes. Atkinson Drive serves as a connector roadway between Kalakaua Avenue and Ala Moana Boulevard and at the intersection



WILSON OKAMOTO CORPORATION
ENGINEERS - PLANNERS

WAIKIKI LANDING

PROJECT SITE PLAN

FIGURE 2

with Ala Moana Boulevard, the Atkinson Drive approach has two left-turn lanes, a channelized through and right-turn lane, and an exclusive right-turn lane. The northbound approach of the intersection is comprised of the access roadway for Ala Moana Beach Park. At the intersection with Ala Moana Boulevard and Atkinson Drive, the beach park access approach has exclusive turning lanes and one through lane.

Southeast of the intersection with Atkinson Drive, Ala Moana Boulevard intersects Holomoana Street, a predominantly two-lane, two-way roadway that serves as the access road for Ala Wai Boat Harbor. At this unsignalized T-intersection, Ala Moana Boulevard is a divided roadway with turning movements from Holomoana Street restricted to right-turn-out movements only. The eastbound approach of Ala Moana Boulevard has two through lanes and a shared through and right-turn lane while the Holomoana Street approach has one right-turn lane.

Further southeast, Ala Moana Boulevard intersects Hobron Lane. At this signalized intersection, both approaches of Ala Moana Boulevard have exclusive left-turn lanes, two through lanes, and a shared through and right-turn lane. Hobron Lane is a predominantly two-lane, two-way roadway between Ena Road and Holomoana Street. At the intersection with Ala Moana Boulevard, the northbound approach of Hobron Lane has an exclusive left-turn lane and one lane that serves all traffic movements while the southbound approach has a shared left-turn and through lane and an exclusive right-turn lane.

B. Traffic Volumes and Conditions

1. General

a. Field Investigation

The field investigations were conducted on October 28-29, 2009, and June 9, 2010, and consisted of manual turning movement count surveys and traffic flow assessments during the morning peak hours of 6:30 AM and 8:30 AM, and between the afternoon peak hours of 4:00 PM and 6:00 PM at the following intersections:

- Ala Moana Boulevard and Atkinson Drive
- Ala Moana Boulevard and Holomoana Street
- Ala Moana Boulevard and Hobron Lane

Appendix A includes the existing traffic count data.

b. Capacity Analysis Methodology

The highway capacity analysis performed in this study is based upon procedures presented in the "Highway Capacity Manual", Transportation Research Board, 2000, and the "Synchro" software, developed by Trafficware. The analysis is based on the concept of Level of Service (LOS).

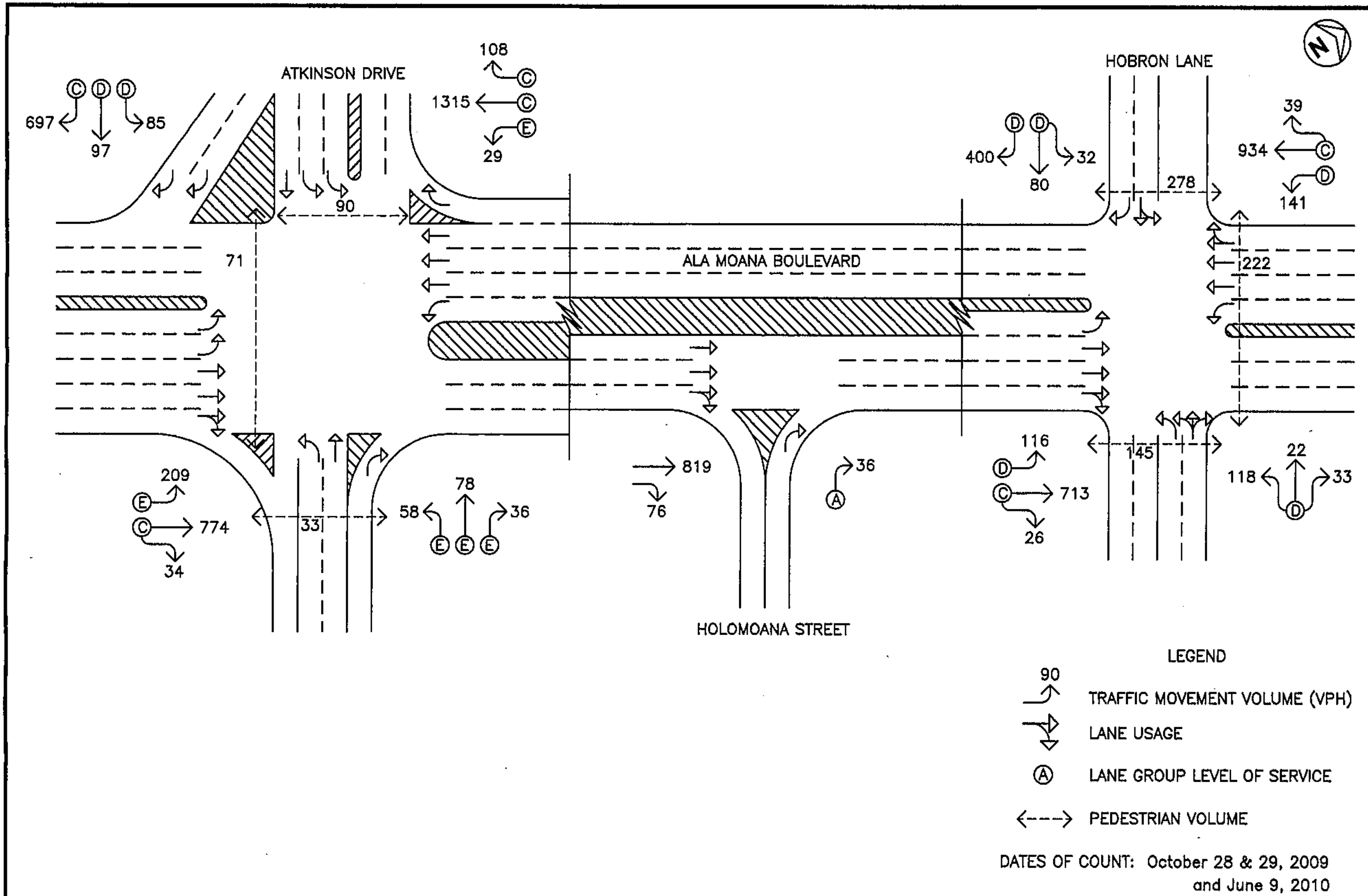
LOS is a quantitative and qualitative assessment of traffic operations. Levels of Service are defined by LOS "A" through "F"; LOS "A" representing ideal or free-flow traffic operating conditions and LOS "F" representing unacceptable or potentially congested traffic operating conditions.

"Volume-to-Capacity" (v/c) ratio is another measure indicating the relative traffic demand to the roadway carrying capacity. A v/c ratio of one (1.00) indicates that the roadway is operating at or near capacity. A v/c ratio of greater than 1.00 generally indicates that the traffic demand exceeds the road's carrying capacity. The LOS definitions are included in Appendix B.

2. Existing Peak Hour Traffic

a. General

Figures 3 and 4 show the existing AM and PM peak period traffic volumes and traffic operating conditions. The AM peak hour of traffic generally occurs between the hours of 7:15 AM and 8:15 AM. During the afternoon, the PM peak hour of traffic generally occurs between the hours of 4:00 PM and 5:00 PM. Although the peak hours of traffic generally occur around the same time periods at each of the study intersections, the absolute commuter peak hour time periods for

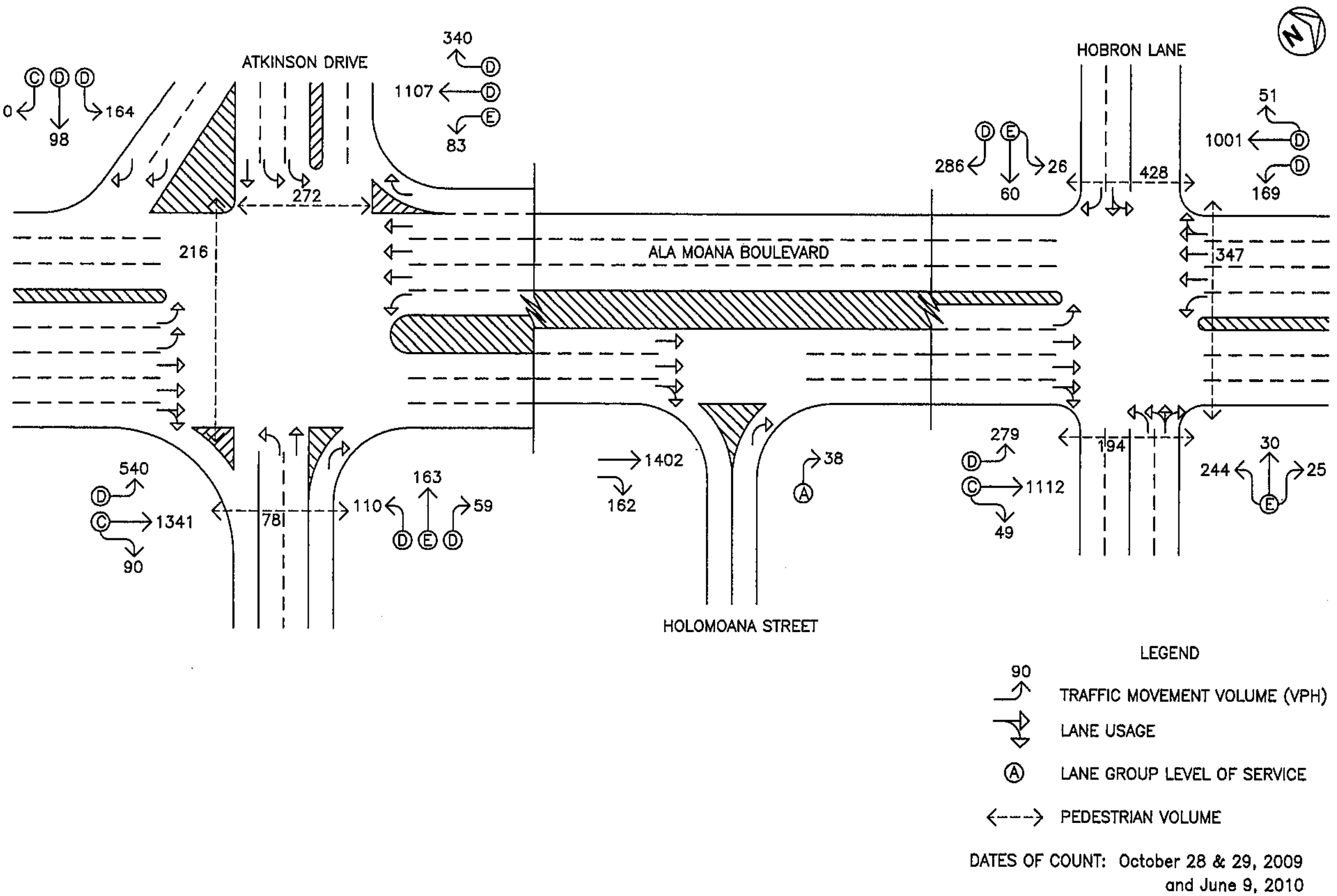


WILSON OKAMOTO CORPORATION
ENGINEERS • PLANNERS

WAIKIKI LANDING

EXISTING AM PEAK HOUR OF TRAFFIC

FIGURE 3



WILSON OKAMOTO CORPORATION
ENGINEERS · PLANNERS

WAIKIKI LANDING

EXISTING PM PEAK HOUR OF TRAFFIC

FIGURE 4

each intersection may differ slightly as shown in Table 1.

Table 1: Peak Hours of Traffic

Intersection	AM Peak	PM Peak
Ala Moana Blvd/ Atkinson Dr	7:00 – 8:00 AM	4:30 – 5:30 PM
Ala Moana Blvd/ Kahanamoku St	7:15 – 8:15 AM	4:00 – 5:00 PM
Ala Moana Blvd/ Hobron Ln	7:15 – 8:15 AM	4:00 – 5:00 PM

The analysis is based on the above absolute commuter peak hour time periods for each intersection to identify the traffic impacts resulting from the proposed project. LOS calculations are included in Appendix C.

b. Ala Moana Boulevard and Atkinson Drive

At the intersection with Atkinson Drive, Ala Moana Boulevard carries 1,017 vehicles eastbound and 1,452 vehicles westbound during the AM peak period. During the PM peak period, traffic volumes are higher with 1,971 vehicles traveling eastbound and 1,530 vehicles traveling westbound. On the eastbound approach of Ala Moana Boulevard, the left-turn traffic movement operates at LOS "E" and LOS "D" during the AM and PM peak periods, respectively, while the through and right-turn traffic movement operates at LOS "C" during both peak periods. On the westbound approach, the left-turn traffic movement operates at LOS "E" during both peak periods while the through and right-turn traffic movements operate at LOS "C" and LOS "D" during the AM and PM peak periods, respectively. Vehicular queues formed periodically on the Ala Moana Boulevard approaches with the most significant queuing occurring during the PM peak period. During this period, average queue lengths of 15-17 vehicles were observed on both approaches. Most of these queues cleared the intersection after each traffic signal cycle change, but occasionally vehicles had to wait for more than one traffic signal cycle length.

The Atkinson Drive approach of the intersection carries 879 vehicles southbound during the AM peak period. During the PM peak period, the traffic volume is less with 472 vehicles travelling southbound. The southbound left-turn and through traffic movements operate at LOS "D" during both peak periods while the southbound right-turn traffic movement operates at LOS "C" during both peak periods. Vehicular queues formed periodically on the Atkinson Drive approaches of the intersection with average queue lengths of 5-7 vehicles observed during both peak periods. Most of these queues were observed to clear the intersection after each traffic signal cycle change.

The northbound approach of the intersection is comprised of the access roadway for Ala Moana Beach Park which carries 172 vehicles and 332 vehicles northbound during the AM and PM peak periods, respectively. The northbound left-turn and right-turn traffic movements operate at LOS "E" and LOS "D" during the AM and PM peak periods, respectively, while the through traffic movement operates at LOS "E" during both peak periods. Vehicular queues formed periodically on the beach park access approach of the intersection with average queue lengths of 5-7 vehicles observed during both peak periods. Most of these queues were observed to clear the intersection after each traffic signal cycle change.

Pedestrian crossings are provided at this intersection across the eastbound approach of Ala Moana Boulevard, Atkinson Drive, and the beach park access approach. During the AM peak period, 71 pedestrians were observed crossing Ala Moana Boulevard, 90 pedestrians were observed crossing Atkinson Drive, and 33 pedestrians were observed crossing the beach park access. During the PM peak period, 216 pedestrians were observed crossing Ala Moana Boulevard, 272 pedestrians were observed crossing Atkinson Drive, and 78 pedestrians were observed crossing the beach park access.

c. Ala Moana Boulevard and Holomoana Street

At the intersection with Holomoana Street, Ala Moana Boulevard carries 895 vehicles and 1,564 vehicles eastbound during the AM and PM peak periods, respectively. The westbound direction of traffic along Ala Moana Boulevard travels through the intersection unimpeded.

The Holomoana Street approach of the intersection carries 36 vehicles northbound during the AM peak period. During the PM peak period, the traffic volume is approximately the same with 38 vehicles travelling northbound. The Holomoana Street approach operates at LOS "A" during both peak periods.

d. Ala Moana Boulevard and Hobron Lane

At the intersection with Hobron Lane, Ala Moana Boulevard carries 855 vehicles eastbound and 1,114 vehicles westbound during the AM peak period. During the PM peak period, the traffic volumes are higher with 1,440 vehicles traveling eastbound and 1,221 vehicles traveling westbound. On the eastbound approach of Ala Moana Boulevard, the left-turn traffic movement operates at LOS "D" during both peak periods while the through and right-turn traffic movement operates at LOS "C" during both peak periods. On the westbound approach, the left-turn traffic movement operates at LOS "D" during both peak periods while the through and right-turn traffic movement operates at LOS "C" and LOS "D" during the AM and PM peak periods, respectively. Vehicular queues formed periodically on the Ala Moana Boulevard approaches with the most significant queuing occurring during the PM peak period. During this period, average queue lengths of 15-17 vehicles were observed on both approaches with westbound queues occasionally extending through the upstream intersection with Kahanamoku Street. Most of these queues cleared the intersection after each traffic signal cycle change, but occasionally vehicles had to wait for more than one traffic signal cycle length.

The Hobron Lane approaches of the intersection carry 173 vehicles northbound and 512 vehicles southbound during the AM peak period. During the PM peak period, the overall traffic volume is approximately the same with 299 vehicles travelling northbound and 372 vehicles traveling southbound. The northbound approach and the left-turn and through traffic movement on the southbound approach operate at LOS "D" and LOS "E" during both peak periods while the southbound right-turn traffic movement operates at LOS "D" during both peak periods. Vehicular queues formed periodically on the Hobron Lane approaches of the intersection with the most significant queuing occurring on the southbound approach. Average queue lengths of 5-7 vehicles were observed during both peak periods. However, these queues cleared the intersection after each traffic signal cycle change.

Pedestrian crossings are provided at this intersection across the westbound approach of Ala Moana Boulevard and both approaches of Hobron Lane. During the AM peak period, 222 pedestrians were observed crossing Ala Moana Boulevard while 145 pedestrians and 278 pedestrians were observed crossing the northbound and southbound approaches of Hobron Lane, respectively. During the PM peak period, 347 pedestrians were observed crossing Ala Moana Boulevard while 194 pedestrians and 428 pedestrians were observed crossing the northbound and southbound approaches of Hobron Lane, respectively.

IV. PROJECTED TRAFFIC CONDITIONS

A. Site-Generated Traffic

1. Trip Generation Methodology

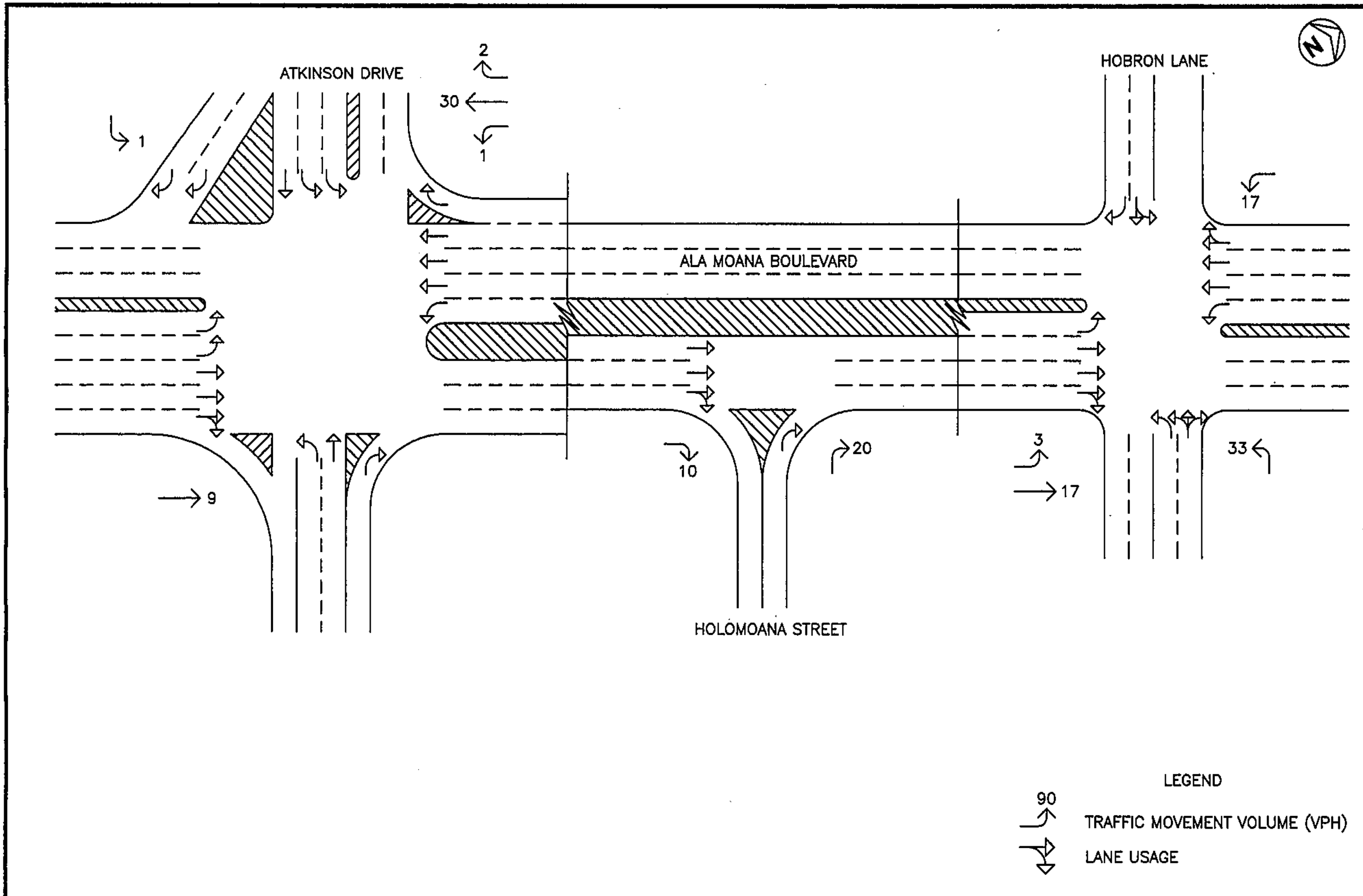
The trip generation methodology used in this study is based upon generally accepted techniques developed by the Institute of Transportation Engineers (ITE) and published in "Trip Generation, 8th Edition," 2008. The ITE trip generation rates are developed empirically by correlating the vehicle trip generation data with various land use characteristics such as the number of vehicle trips generated per 1,000 square feet of development. The proposed office areas on-site are expected to be utilized as administration space for the development, as well as, State of Hawaii Department of Boating and Ocean Recreation (DOBOR). For the purpose of this report, these office spaces are expected to function as an amenity to the development and not expected to generate additional trips to/from the project site. With regards to the trips generated by the proposed restaurant uses, since the proposed development will be located in a neighborhood with limited parking, high volumes of pedestrian traffic, and a high density of attractive destinations, patrons may elect to walk to their destinations rather than drive. In addition, a portion of the patrons are assumed to already be accessing the adjacent marina areas or businesses and consequently not expected to generate new trips. As such, approximately 30% of the trips generated by the proposed restaurant uses are assumed to result in new trips in the project vicinity. With regards to the proposed wedding facilities, for the purpose of this report, the functions at these facilities are assumed to be held during off-peak periods when traffic along the adjacent roadways is less. Table 2 summarizes the project site trip generation characteristics applied to the AM and PM peak periods of traffic.

Table 2: Peak Hour Trip Generation

SPECIALTY RETAIL CENTER INDEPENDENT VARIABLE: 1,000 sf of development = 13.901		
		PROJECTED TRIP ENDS
AM PEAK	ENTER	0
	EXIT	0
	TOTAL	0
PM PEAK	ENTER	17
	EXIT	21
	TOTAL	38
HIGH-TURNOVER (SIT-DOWN) RESTAURANT (ADJUSTED) INDEPENDENT VARIABLE: 1,000 sf of development = 15.385		
		PROJECTED TRIP ENDS
AM PEAK	ENTER	28
	EXIT	25
	TOTAL	53
PM PEAK	ENTER	30
	EXIT	21
	TOTAL	51
TOTALS		
		PROJECTED TRIP ENDS
AM PEAK	ENTER	28
	EXIT	25
	TOTAL	53
PM PEAK	ENTER	47
	EXIT	42
	TOTAL	89

2. Trip Distribution

Figures 5 and 6 show the distribution of site-generated vehicular trips at the study intersections during the AM and PM peak hours of traffic. Access to the proposed project will be provided off Holomoana Street. The directional distribution of site-generated vehicles was based upon the prevailing directional distribution of traffic along Ala Moana Boulevard. As such, 37.1% were assumed to be traveling eastbound during the AM peak period while 62.9% were assumed to be traveling westbound. During the PM

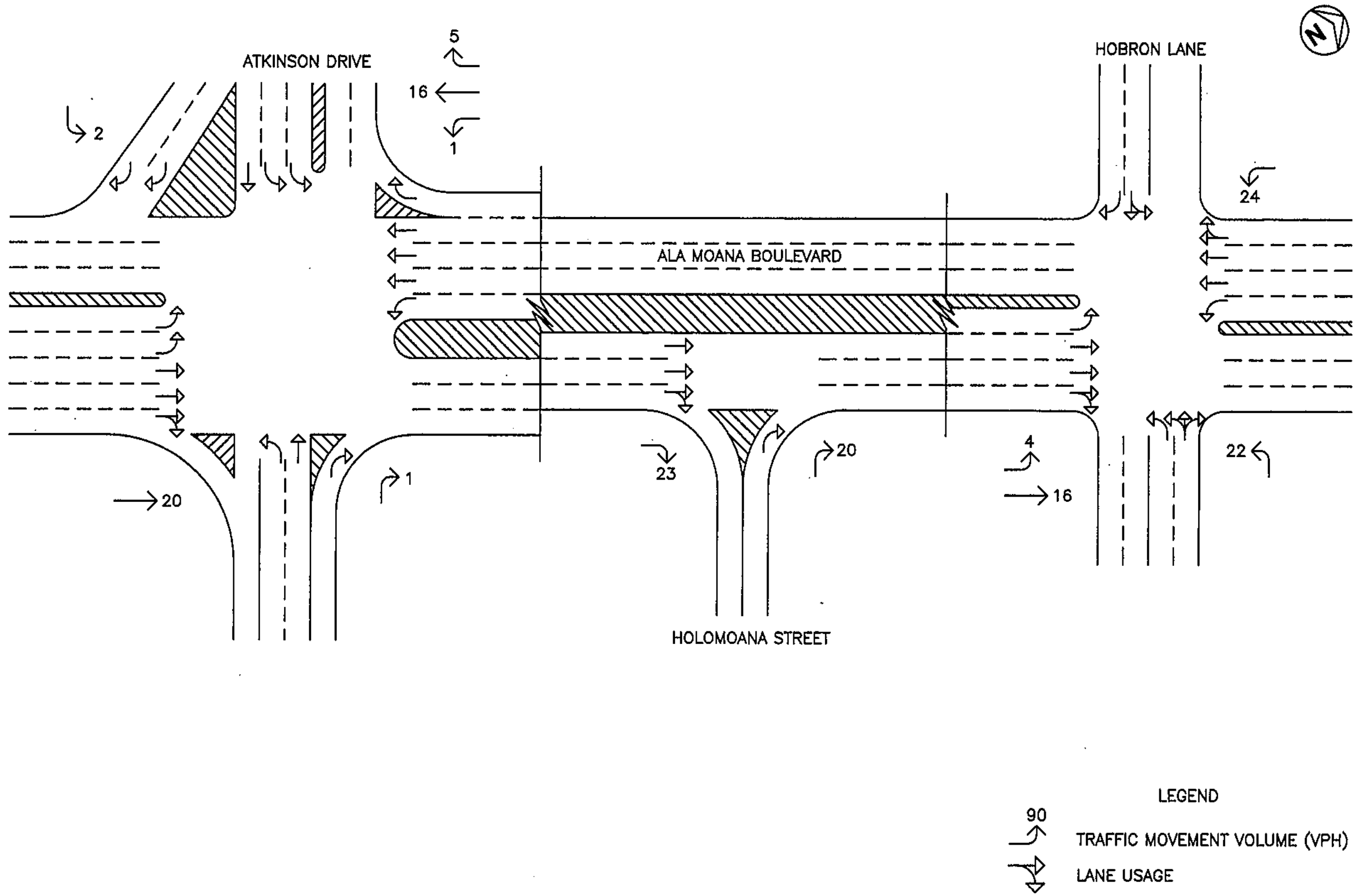


WILSON OKAMOTO CORPORATION
ENGINEERS · PLANNERS

WAIKIKI LANDING

DISTRIBUTION OF SITE-GENERATED VEHICLES
AM PEAK HOUR OF TRAFFIC

FIGURE 5



WILSON OKAMOTO CORPORATION
ENGINEERS • PLANNERS

WAIKIKI LANDING

DISTRIBUTION OF SITE-GENERATED VEHICLES
PM PEAK HOUR OF TRAFFIC

FIGURE 6

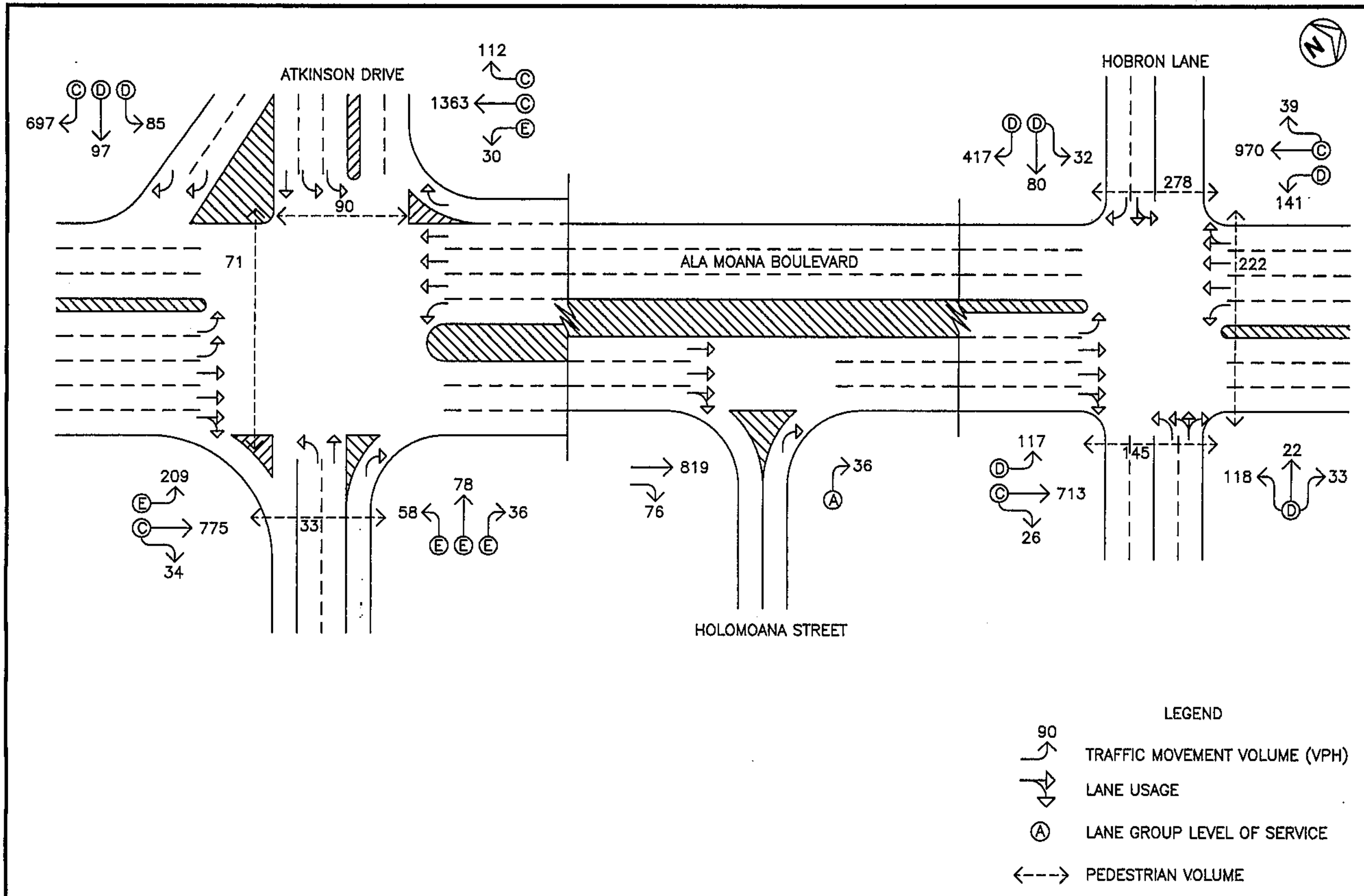
peak period, 48.5% were assumed to be traveling eastbound while 51.5% were assumed to be traveling westbound. All eastbound entering and westbound exiting vehicles were assumed to utilize Hobron Lane to travel between Ala Moana Boulevard and Holomoana Street while all eastbound exiting and westbound entering vehicles were assumed to directly access Ala Moana Boulevard via Holomoana Street. At the other study intersections along their route, the directional distribution of the site-generated vehicles was assumed to remain similar to existing conditions.

B. Other Considerations

There have been a number of new developments in Waikiki in the vicinity of the Hilton Hawaiian Village in recent years including the Waikiki Beach Walk and Watermark. One of the developments in the vicinity is still under construction, but is expected to be completed by the Year 2013. This development is the Waikiki Allure condominium located adjacent to Kalakaua Avenue north of Ena Road. This development will include 315 residential condominium units and a restaurant. As detailed in the "Traffic Impact Report for the Waikiki Allure Condominium," the development is expected to generate 130 trips and 213 trips during the AM and PM peak periods, respectively. The trips associated with the Waikiki Allure condominium were incorporated with the Year 2013 without project conditions to account for the additional traffic generated by this development.

C. Total Traffic Volumes Without Project

The projected year 2013 AM and PM peak period traffic volumes and operating conditions without the proposed Hilton Hawaiian Village renovations are shown in Figures 7 and 8, and summarized in Table 3. The existing levels of service are included for comparison purposes. LOS calculations are included in Appendix D.

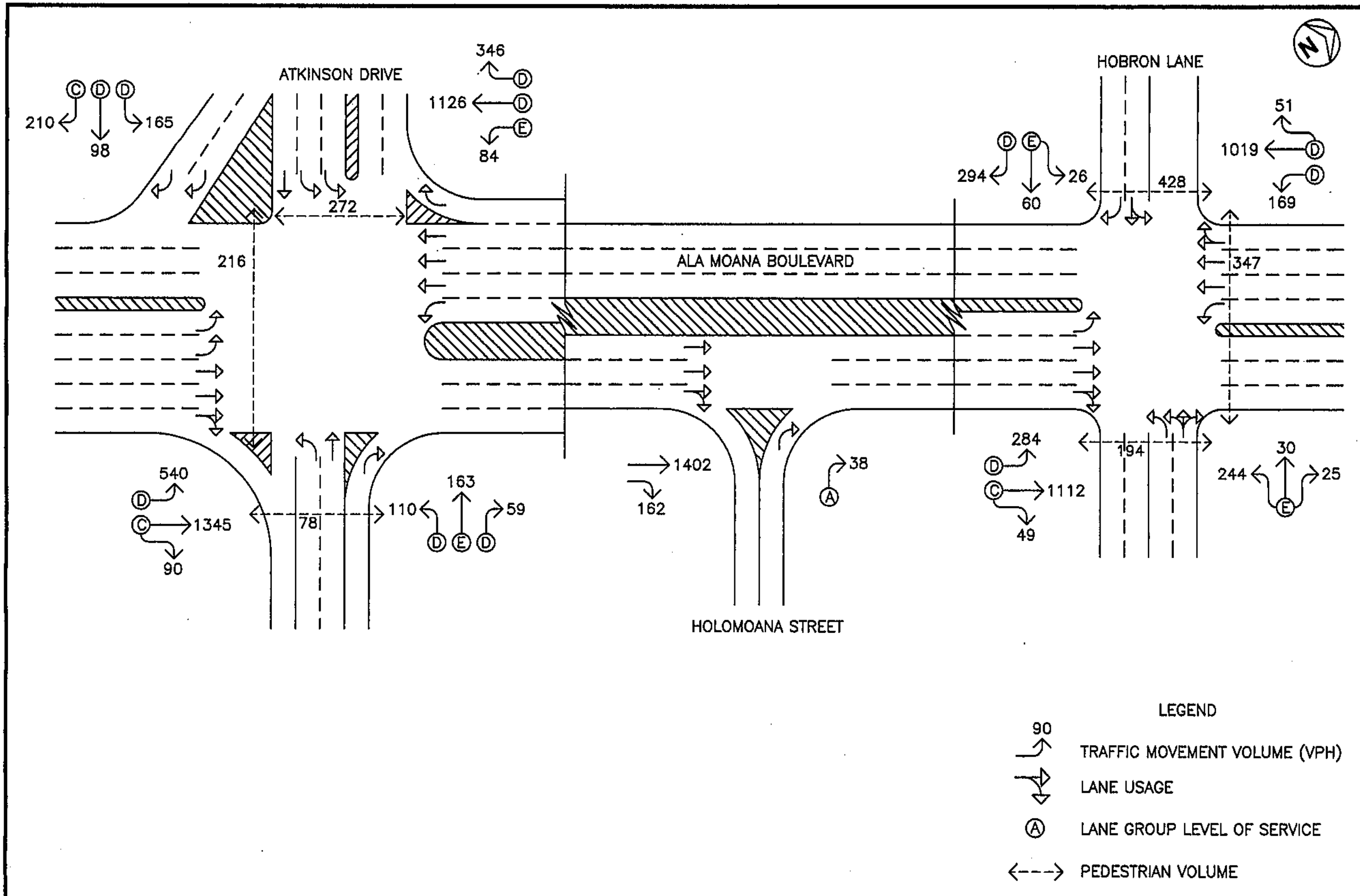


WILSON OKAMOTO CORPORATION
ENGINEERS · PLANNERS

WAIKIKI LANDING

YEAR 2013 AM PEAK HOUR OF TRAFFIC
WITHOUT PROJECT

FIGURE
7



WILSON OKAMOTO CORPORATION
ENGINEERS • PLANNERS

WAIKIKI LANDING

YEAR 2013 PM PEAK HOUR OF TRAFFIC
WITHOUT PROJECT

FIGURE 8

Table 3: Existing and Projected (Without Project) LOS Traffic Operating Conditions

Intersection	Critical Traffic Movement		AM		PM	
			Exist	Year 2013 w/out Proj	Exist	Year 2013 w/out Proj
Ala Moana Blvd/ Atkinson Dr	Eastbound	LT	E	E	D	D
		TH-RT	C	C	C	C
	Westbound	LT	E	E	E	E
		TH	C	C	D	D
		RT	C	C	D	D
	Northbound	LT	E	E	D	D
		TH	E	E	E	E
		RT	E	E	D	D
	Southbound	LT	D	D	D	D
		TH	D	D	D	D
		RT	C	C	C	C
Ala Moana Blvd/ Holomoana St	Northbound	RT	A	A	A	A
Ala Moana Blvd/ Hobron Ln	Eastbound	LT	D	D	D	D
		TH-RT	C	C	C	C
	Westbound	LT	D	D	D	D
		TH-RT	C	C	D	D
	Northbound	LT-TH-RT	D	D	E	E
	Southbound	LT-TH	D	D	E	E
		RT	D	D	D	D

Under Year 2013 without project conditions, traffic operations are expected to remain similar to existing conditions. Despite the anticipated increases in traffic due to the completion of other projects in the vicinity, the critical traffic movements at the intersection of Ala Moana Boulevard with Atkinson Drive are expected to continue operating at LOS "E" or better during both peak periods while those at the intersection with Hobron Lane are expected to continue operating at LOS "D" or better during the AM peak period and LOS "E" or better during the PM peak period.

At the intersection with Holomoana Street, the northbound approach is expected to continue operating at LOS "A" during both peak periods.

D. Total Traffic Volumes With Project

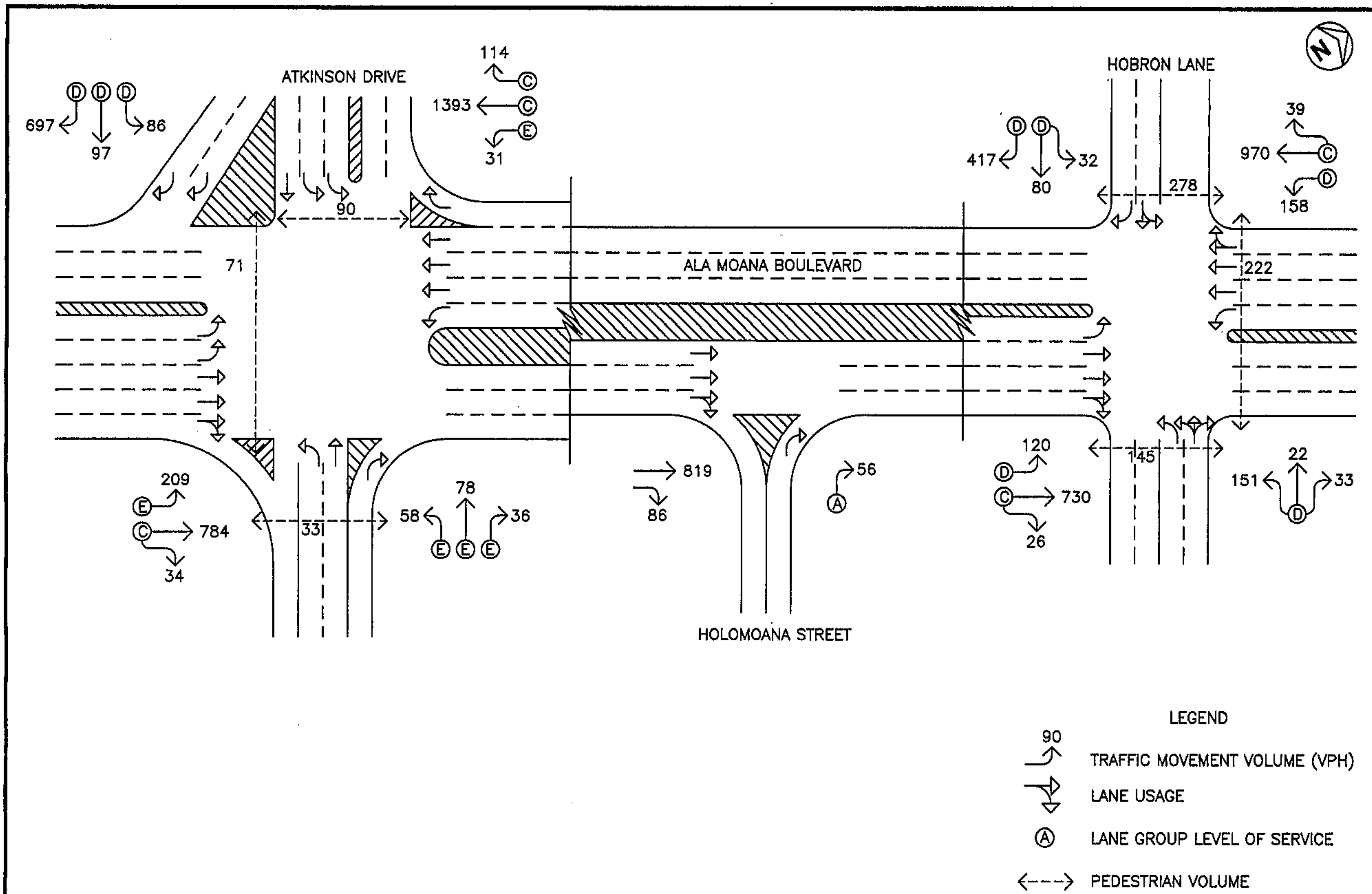
Figures 9 and 10 show the Year 2019 cumulative AM and PM peak hour traffic conditions resulting from the completion of other developments in the vicinity and the proposed Waikiki Landing development. The cumulative volumes consist of site- generated traffic superimposed over Year 2013 projected traffic demands. The traffic impacts resulting from the proposed project are addressed in the following section.

V. TRAFFIC IMPACT ANALYSIS

The Year 2013 cumulative AM and PM peak hour traffic conditions with the proposed Waikiki Landing development are summarized in Table 4. The existing and projected Year 2013 (Without Project) operating conditions are provided for comparison purposes. LOS calculations are included in Appendix E.

Table 4: Existing and Projected (Without and With Project) LOS Traffic Operating Conditions

Intersection	Critical Traffic Movement		AM			PM		
			Exist	Year 2013		Exist	Year 2013	
				w/out Proj	w/ Proj		w/out Proj	w/ Proj
Ala Moana Blvd/ Atkinson Dr	EB	LT	E	E	E	D	D	D
		TH-RT	C	C	C	C	C	C
	WB	LT	E	E	E	E	E	E
		TH	C	C	C	D	D	D
		RT	C	C	C	D	D	D
	NB	LT	E	E	E	D	D	D
		TH	E	E	E	E	E	E
		RT	E	E	E	D	D	D
	SB	LT	D	D	D	D	D	D
		TH	D	D	D	D	D	D
		RT	C	C	D	C	C	C

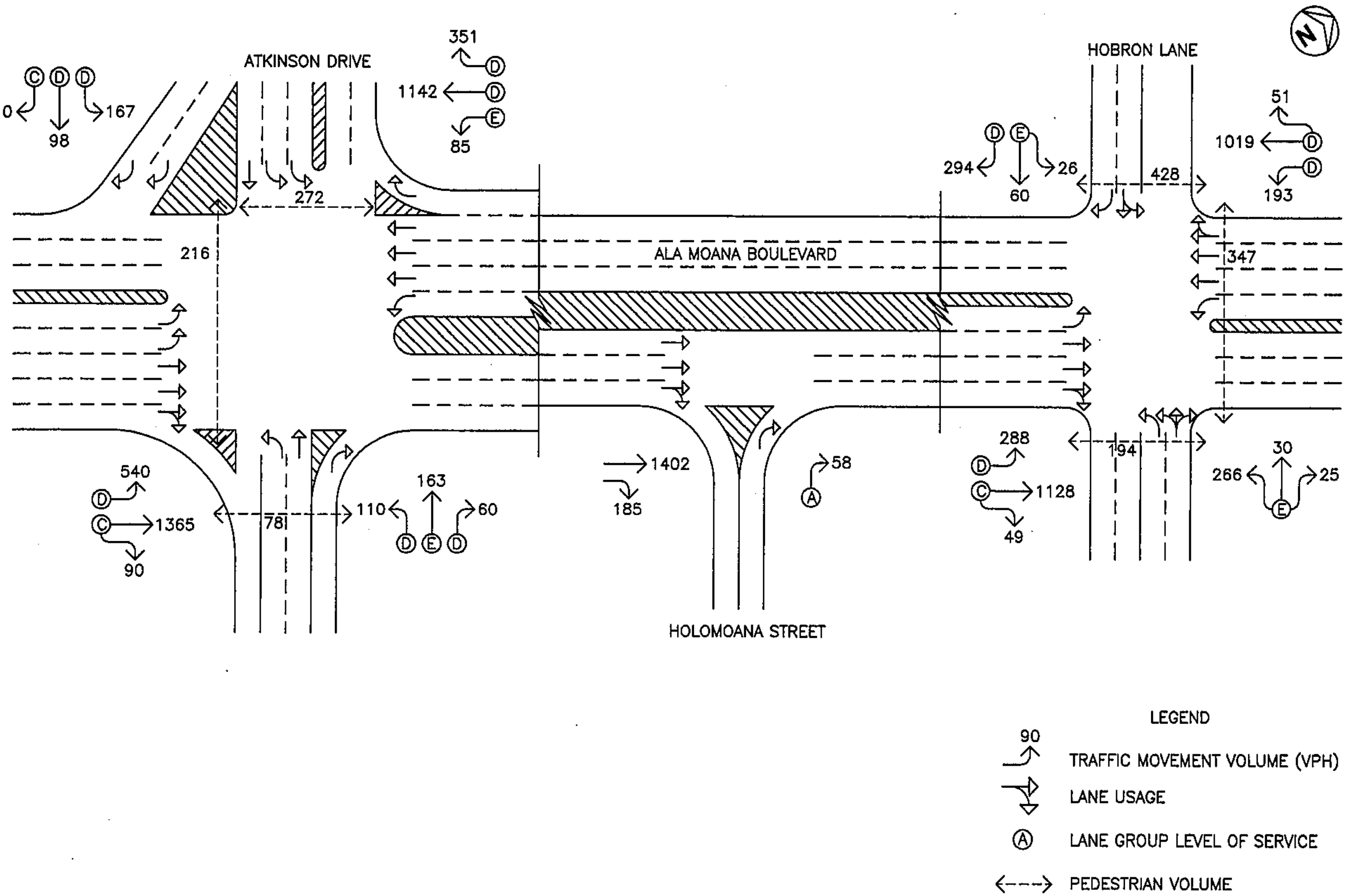


WILSON OKAMOTO CORPORATION
ENGINEERS • PLANNERS

WAIKIKI LANDING

YEAR 2013 AM PEAK HOUR OF TRAFFIC
WITH PROJECT

FIGURE
9



WILSON OKAMOTO CORPORATION
ENGINEERS • PLANNERS

WAIKIKI LANDING

YEAR 2013 PM PEAK HOUR OF TRAFFIC WITH PROJECT

FIGURE 10

Table 4: Existing and Projected (Without and With Project) LOS Traffic Operating Conditions (Cont'd)

Intersection	Critical Traffic Movement		AM			PM		
			Exist	Year 2013		Exist	Year 2013	
				w/out Proj	w/ Proj		w/out Proj	w/ Proj
Ala Moana Blvd/ Holomoana St	NB	RT	A	A	A	A	A	A
Ala Moana Blvd/ Hobron Ln	EB	LT	D	D	D	D	D	D
		TH-RT	C	C	C	C	C	C
	WB	LT	D	D	D	D	D	D
		TH-RT	C	C	C	D	D	D
	NB	LT-TH-RT	D	D	D	E	E	E
	SB	LT-TH	D	D	D	E	E	E
		RT	D	D	D	D	D	D

Traffic operations under Year 2013 with project conditions are expected, in general, to remain similar to existing and without project conditions during both peak periods despite the addition of site-generated vehicles to the surrounding roadways. The southbound right-turn traffic movement at the intersection of Ala Moana Boulevard with Atkinson Drive is expected to operate at a slightly lower level-of-service during the AM peak period. The remaining critical traffic movements at this intersection, as well as, the other study intersections are expected to continue operating at levels-of-service similar to existing and without project conditions. In addition, the total traffic volumes entering the study intersections along Ala Moana Boulevard are expected to increase by approximately 2-3% during both peak periods with the proposed development. These increases in the total traffic volumes are in the range of daily volume fluctuations along that roadway and represent a minimal increase in the overall traffic volumes.

VI. RECOMMENDATIONS

Based on the analysis of the traffic data, the following are the recommendations of this study associated with the development of the proposed Waikiki Landing:

1. Maintain sufficient sight distance for motorists to safely enter and exit all driveways and roadways.

2. Maintain adequate on-site loading and off-loading service areas and prohibit off-site loading operations.

3. Maintain adequate turn-around area for service, delivery, and refuse collection vehicles to maneuver on the project site to avoid vehicle-reversing maneuvers onto public roadways.

4. Maintain sufficient turning radii at all driveways and roadways to avoid or minimize vehicle encroachments to oncoming traffic lanes.

VII. CONCLUSION

The proposed project entails the renovation and upgrade of an existing boatyard repair facility and the redevelopment of adjacent areas to create an attractive waterfront development containing a variety of retail and restaurant uses. With the implementation of the aforementioned recommendations, the proposed project is not expected to have a significant impact on traffic operations in the project vicinity. Traffic operations at the study intersections are expected to remain similar to existing and without project conditions. In addition, the total traffic volumes entering the study intersections along Ala Moana Boulevard are expected to increase by approximately 2-3% during both peak periods with the proposed development. These increases in the total traffic volumes are in the range of daily volume fluctuations along that roadway and represent a minimal increase in the overall traffic volumes.

APPENDIX A

EXISTING TRAFFIC COUNT DATA

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-3888, D4-5673
Counted By:AR, KP
Weather:Clear

File Name : AlaAtk AM
Site Code : 00000092
Start Date : 10/28/2009
Page No : 1

Groups Printed- Unshifted

	Atkinson Drive Southbound					Ala Moana Blvd. Westbound					Ala Moana Park Drive Northbound					Ala Moana Blvd. Eastbound					
Start Time	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
06:30 AM	15	38	108	19	180	13	214	7	0	234	13	24	9	7	53	29	171	7	22	229	696
06:45 AM	20	14	144	35	213	7	242	9	0	258	18	15	11	4	48	58	192	6	22	278	797
Total	35	52	252	54	393	20	456	16	0	492	31	39	20	11	101	87	363	13	44	507	1493
07:00 AM	24	25	133	21	203	7	300	25	0	332	12	18	11	7	48	62	199	11	26	298	881
07:15 AM	13	26	173	29	241	8	391	23	0	422	19	14	12	15	60	45	204	3	13	265	988
07:30 AM	23	23	191	17	254	4	360	33	0	397	16	29	5	7	57	42	179	7	21	249	957
07:45 AM	26	23	200	23	272	11	327	32	0	370	11	17	9	4	41	60	206	13	11	290	973
Total	86	97	697	90	970	30	1378	113	0	1521	58	78	37	33	206	209	788	34	71	1102	3799
08:00 AM	22	13	147	32	214	6	321	34	0	361	10	15	9	4	38	41	160	7	21	229	842
08:15 AM	31	20	145	32	228	12	278	29	0	319	22	25	13	4	64	44	181	5	25	255	866
Grand Total	174	182	1241	208	1805	68	2433	192	0	2693	121	157	79	52	409	381	1492	59	161	2093	7000
Apprch %	9.6	10.1	68.8	11.5		2.5	90.3	7.1	0		29.6	38.4	19.3	12.7		18.2	71.3	2.8	7.7		
Total %	2.5	2.6	17.7	3	25.8	1	34.8	2.7	0	38.5	1.7	2.2	1.1	0.7	5.8	5.4	21.3	0.8	2.3	29.9	

	Atkinson Drive Southbound				Ala Moana Blvd. Westbound				Ala Moana Park Drive Northbound				Ala Moana Blvd. Eastbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 06:30 AM to 08:15 AM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 07:00 AM																	
07:00 AM	24	25	133	182	7	300	25	332	12	18	11	41	**62**	199	11	272	827
07:15 AM	13	**26**	173	212	8	**391**	23	**422**	**19**	14	**12**	45	45	204	3	252	931
07:30 AM	23	23	191	237	4	360	**33**	397	16	**29**	5	**50**	42	179	7	228	912
07:45 AM	**26**	23	**200**	**249**	**11**	327	32	370	11	17	9	37	60	**206**	**13**	**279**	**935**
Total Volume	86	97	697	880	30	1378	113	1521	58	78	37	173	209	788	34	1031	3605
% App. Total	9.8	11	79.2		2	90.6	7.4		33.5	45.1	21.4		20.3	76.4	3.3		
PHF	.827	.933	.871	.884	.682	.881	.856	.901	.763	.672	.771	.865	.843	.956	.654	.924	.964

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-3888, D4-5673
Counted By:LE. RK
Weather:Clear

File Name : AlaAtk PM
Site Code : 00000092
Start Date : 10/28/2009
Page No : 1

Groups Printed- Unshifted

	Atkinson Drive Southbound					Ala Moana Blvd. Westbound					Ala Moana Park Drive Northbound					Ala Moana Blvd. Eastbound					
Start Time	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
04:00 PM	29	22	39	43	133	20	299	71	2	392	24	65	18	25	132	112	340	17	18	487	1144
04:15 PM	32	15	43	75	165	20	350	70	0	440	10	14	8	37	69	119	311	24	75	529	1203
04:30 PM	48	19	57	69	193	22	261	79	0	362	37	33	13	25	108	133	301	11	51	496	1159
04:45 PM	29	25	44	72	170	24	307	85	0	416	23	73	15	19	130	135	308	20	45	508	1224
Total	138	81	183	259	661	86	1217	305	2	1610	94	185	54	106	439	499	1260	72	189	2020	4730
05:00 PM	44	26	58	70	198	15	241	84	3	343	32	31	17	18	98	136	305	26	63	530	1169
05:15 PM	37	28	51	61	177	22	298	92	0	412	18	26	12	17	73	136	378	33	57	604	1266
05:30 PM	68	48	48	64	228	10	263	74	0	347	30	34	12	79	155	112	260	37	44	453	1183
05:45 PM	58	44	49	70	221	34	254	68	0	356	24	24	21	28	97	134	349	28	78	589	1263
Total	207	146	206	265	824	81	1056	318	3	1458	104	115	62	142	423	518	1292	124	242	2176	4881
Grand Total	345	227	389	524	1485	167	2273	623	5	3068	198	300	116	248	862	1017	2552	196	431	4196	9611
Apprch %	23.2	15.3	26.2	35.3		5.4	74.1	20.3	0.2		23	34.8	13.5	28.8		24.2	60.8	4.7	10.3		
Total %	3.6	2.4	4	5.5	15.5	1.7	23.6	6.5	0.1	31.9	2.1	3.1	1.2	2.6	9	10.6	26.6	2	4.5	43.7	

	Atkinson Drive Southbound				Ala Moana Blvd. Westbound				Ala Moana Park Drive Northbound				Ala Moana Blvd. Eastbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 04:30 PM																	
04:30 PM	**48**	19	57	124	22	261	79	362	**37**	33	13	83	133	301	11	445	1014
04:45 PM	29	25	44	98	**24**	**307**	85	**416**	23	**73**	15	**111**	135	308	20	463	1088
05:00 PM	44	26	**58**	**128**	15	241	84	340	32	31	**17**	80	**136**	305	26	467	1015
05:15 PM	37	**28**	51	116	22	298	**92**	412	18	26	12	56	136	**378**	**33**	**547**	**1131**
Total Volume	158	98	210	466	83	1107	340	1530	110	163	57	330	540	1292	90	1922	4248
% App. Total	33.9	21	45.1		5.4	72.4	22.2		33.3	49.4	17.3		28.1	67.2	4.7		
PHF	.823	.875	.905	.910	.865	.901	.924	.919	.743	.558	.838	.743	.993	.854	.682	.878	.939

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-3890
Counted By:RJ
Weather:Clear

File Name : AlaHolo AM
Site Code : 00000001
Start Date : 6/9/2010
Page No : 1

Groups Printed- Unshifted

	Southbound	Westbound	Holomoana Street Northbound					Ala Moana Blvd. Eastbound					
Start Time	App. Total	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
06:30 AM	0	0	0	0	6	1	7	0	0	25	0	25	32
06:45 AM	0	0	0	0	1	0	1	0	0	17	0	17	18
Total	0	0	0	0	7	1	8	0	0	42	0	42	50
07:00 AM	0	0	0	0	6	2	8	0	0	20	0	20	28
07:15 AM	0	0	0	0	11	0	11	0	0	18	0	18	29
07:30 AM	0	0	0	0	6	6	12	0	0	23	0	23	35
07:45 AM	0	0	0	0	9	2	11	0	0	12	0	12	23
Total	0	0	0	0	32	10	42	0	0	73	0	73	115
08:00 AM	0	0	0	0	10	8	18	0	0	23	0	23	41
08:15 AM	0	0	0	0	9	4	13	0	0	18	0	18	31
Grand Total	0	0	0	0	58	23	81	0	0	156	0	156	237
Apprch %			0	0	71.6	28.4		0	0	100	0		
Total %	0	0	0	0	24.5	9.7	34.2	0	0	65.8	0	65.8	

	Southbound	Westbound	Holomoana Street Northbound				Ala Moana Blvd. Eastbound				
Start Time	App. Total	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 06:30 AM to 08:15 AM - Peak 1 of 1											
Peak Hour for Entire Intersection Begins at 07:15 AM											
07:15 AM	0	0	0	0	11	11	0	0	18	18	29
07:30 AM	0	0	0	0	6	6	0	0	23	23	29
07:45 AM	0	0	0	0	9	9	0	0	12	12	21
08:00 AM	0	0	0	0	10	10	0	0	23	23	33
Total Volume	0	0	0	0	36	36	0	0	76	76	112
% App. Total			0	0	100		0	0	100		
PHF	.000	.000	.000	.000	.818	.818	.000	.000	.826	.826	.848

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-3890
Counted By:RJ
Weather:Clear

File Name : AlaHolo PM
Site Code : 00000001
Start Date : 6/9/2010
Page No : 1

Groups Printed- Unshifted

	Southbound	Westbound	Holomoana Street Northbound					Ala Moana Blvd. Eastbound					
Start Time	App. Total	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
04:00 PM	0	0	0	0	7	5	12	0	0	41	0	41	53
04:15 PM	0	0	0	0	8	11	19	0	0	32	0	32	51
04:30 PM	0	0	0	0	8	9	17	0	0	32	0	32	49
04:45 PM	0	0	0	0	7	13	20	0	0	57	0	57	77
Total	0	0	0	0	30	38	68	0	0	162	0	162	230
05:00 PM	0	0	0	0	6	12	18	0	0	45	0	45	63
05:15 PM	0	0	0	0	2	8	10	0	0	50	0	50	60
05:30 PM	0	0	0	0	8	7	15	0	0	47	0	47	62
05:45 PM	0	0	0	0	6	5	11	0	0	47	0	47	58
Total	0	0	0	0	22	32	54	0	0	189	0	189	243
Grand Total	0	0	0	0	52	70	122	0	0	351	0	351	473
Apprch %			0	0	42.6	57.4		0	0	100	0		
Total %	0	0	0	0	11	14.8	25.8	0	0	74.2	0	74.2	

	Southbound	Westbound	Holomoana Street Northbound				Ala Moana Blvd. Eastbound				
Start Time	App. Total	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1											
Peak Hour for Entire Intersection Begins at 04:45 PM											
04:45 PM	0	0	0	0	7	7	0	0	57	57	64
05:00 PM	0	0	0	0	6	6	0	0	45	45	51
05:15 PM	0	0	0	0	2	2	0	0	50	50	52
05:30 PM	0	0	0	0	8	8	0	0	47	47	55
Total Volume	0	0	0	0	23	23	0	0	199	199	222
% App. Total			0	0	100		0	0	100		
PHF	.000	.000	.000	.000	.719	.719	.000	.000	.873	.873	.867

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-5677, D4-3890
Counted By:SM, VL
Weather:Clear

File Name : AlaHob AM
Site Code : 00000093
Start Date : 10/29/2009
Page No : 1

Groups Printed- Unshifted

	Hobron Lane Southbound					Ala Moana Blvd. Westbound					Hobron Lane Northbound					Ala Moana Blvd. Eastbound					
Start Time	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
06:30 AM	6	19	78	57	160	11	125	7	55	198	28	3	5	30	66	13	159	12	0	184	608
06:45 AM	5	17	83	44	149	20	155	5	47	227	31	3	8	21	63	22	154	16	0	192	631
Total	11	36	161	101	309	31	280	12	102	425	59	6	13	51	129	35	313	28	0	376	1239
07:00 AM	1	16	78	73	168	19	221	1	48	289	32	3	4	46	85	17	197	10	0	224	766
07:15 AM	5	22	91	56	174	33	239	11	46	329	24	6	8	38	76	26	187	7	0	220	799
07:30 AM	10	21	99	63	193	41	221	7	61	330	26	4	13	24	67	31	170	6	0	207	797
07:45 AM	10	20	98	67	195	28	225	17	55	325	21	6	5	34	66	18	176	5	0	199	785
Total	26	79	366	259	730	121	906	36	210	1273	103	19	30	142	294	92	730	28	0	850	3147
08:00 AM	7	17	93	92	209	39	205	4	60	308	41	6	7	49	103	39	166	7	0	212	832
08:15 AM	8	13	78	71	170	27	199	8	55	289	30	1	11	25	67	29	191	9	0	229	755
Grand Total	52	145	698	523	1418	218	1590	60	427	2295	233	32	61	267	593	195	1400	72	0	1667	5973
Apprch %	3.7	10.2	49.2	36.9		9.5	69.3	2.6	18.6		39.3	5.4	10.3	45		11.7	84	4.3	0		
Total %	0.9	2.4	11.7	8.8	23.7	3.6	26.6	1	7.1	38.4	3.9	0.5	1	4.5	9.9	3.3	23.4	1.2	0	27.9	

	Hobron Lane Southbound				Ala Moana Blvd. Westbound				Hobron Lane Northbound				Ala Moana Blvd. Eastbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 06:30 AM to 08:15 AM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 07:15 AM																	
07:15 AM	5	**22**	91	118	33	**239**	11	**283**	24	**6**	8	38	26	**187**	**7**	**220**	**659**
07:30 AM	**10**	21	**99**	**130**	**41**	221	7	269	26	4	**13**	43	31	170	6	207	649
07:45 AM	10	20	98	128	28	225	**17**	270	21	6	5	32	18	176	5	199	629
08:00 AM	7	17	93	117	39	205	4	248	**41**	6	7	**54**	**39**	166	7	212	631
Total Volume	32	80	381	493	141	890	39	1070	112	22	33	167	114	699	25	838	2568
% App. Total	6.5	16.2	77.3		13.2	83.2	3.6		67.1	13.2	19.8		13.6	83.4	3		
PHF	.800	.909	.962	.948	.860	.931	.574	.945	.683	.917	.635	.773	.731	.934	.893	.952	.974

Wilson Okamoto Corporation

1907 S. Beretania Street Suite 400
Honolulu, Hi 96826

Counter:D4-5677, D4-3890
Counted By:SM, VL
Weather:Clear

File Name : AlaHob PM
Site Code : 00000093
Start Date : 10/29/2009
Page No : 1

Groups Printed- Unshifted

	Hobron Lane Southbound					Ala Moana Blvd. Westbound					Hobron Lane Northbound					Ala Moana Blvd. Eastbound					
Start Time	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Left	Thru	Right	Peds	App. Total	Int. Total
04:00 PM	11	22	71	95	199	22	239	10	101	372	61	10	4	55	130	69	292	14	0	375	1076
04:15 PM	8	14	51	104	177	60	257	18	76	411	78	4	3	41	126	80	314	13	0	407	1121
04:30 PM	6	10	85	124	225	63	217	11	99	390	57	8	8	31	104	77	239	5	0	321	1040
04:45 PM	1	14	79	105	199	24	288	12	71	395	48	8	10	67	133	64	311	19	0	394	1121
Total	26	60	286	428	800	169	1001	51	347	1568	244	30	25	194	493	290	1156	51	0	1497	4358
05:00 PM	5	18	71	95	189	39	218	9	90	356	53	12	9	87	161	68	263	9	0	340	1046
05:15 PM	7	13	57	60	137	30	221	17	85	353	35	12	7	75	129	90	285	7	0	382	1001
05:30 PM	6	22	74	59	161	26	220	5	110	361	54	9	8	71	142	67	311	5	0	383	1047
05:45 PM	11	11	43	90	155	27	170	13	96	306	27	10	4	112	153	68	294	15	0	377	991
Total	29	64	245	304	642	122	829	44	381	1376	169	43	28	345	585	293	1153	36	0	1482	4085
Grand Total	55	124	531	732	1442	291	1830	95	728	2944	413	73	53	539	1078	583	2309	87	0	2979	8443
Apprch %	3.8	8.6	36.8	50.8		9.9	62.2	3.2	24.7		38.3	6.8	4.9	50		19.6	77.5	2.9	0		
Total %	0.7	1.5	6.3	8.7	17.1	3.4	21.7	1.1	8.6	34.9	4.9	0.9	0.6	6.4	12.8	6.9	27.3	1	0	35.3	

	Hobron Lane Southbound				Ala Moana Blvd. Westbound				Hobron Lane Northbound				Ala Moana Blvd. Eastbound				
Start Time	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Left	Thru	Right	App. Total	Int. Total
Peak Hour Analysis From 04:00 PM to 05:45 PM - Peak 1 of 1																	
Peak Hour for Entire Intersection Begins at 04:00 PM																	
04:00 PM	**11**	**22**	71	**104**	22	239	10	271	61	**10**	4	75	69	292	14	375	825
04:15 PM	8	14	51	73	60	257	**18**	**335**	**78**	4	3	**85**	**80**	**314**	13	**407**	**900**
04:30 PM	6	10	**85**	101	**63**	217	11	291	57	8	8	73	77	239	5	321	786
04:45 PM	1	14	79	94	24	**288**	12	324	48	8	**10**	66	64	311	**19**	394	878
Total Volume	26	60	286	372	169	1001	51	1221	244	30	25	299	290	1156	51	1497	3389
% App. Total	7	16.1	76.9		13.8	82	4.2		81.6	10	8.4		19.4	77.2	3.4		
PHF	.591	.682	.841	.894	.671	.869	.708	.911	.782	.750	.625	.879	.906	.920	.671	.920	.941

APPENDIX B

LEVEL OF SERVICE DEFINITIONS

LEVEL OF SERVICE DEFINITIONS

LEVEL-OF-SERVICE CRITERIA FOR SIGNALIZED INTERSECTIONS

Level of Service (LOS) for signalized intersections is defined in terms of delay, which is a measure of driver discomfort, frustration, fuel consumption, and increased travel time. Specifically, level-of-service (LOS) criteria are stated in terms of the average control delay per vehicle, typically a 15-min analysis period. The criteria are given in the following table.

Table 1: Level-of-Service Criteria for Signalized Intersections

Level of Service	Control Delay per Vehicle (sec/veh)
A	≤ 10.0
B	>10.0 and ≤ 20.0
C	>20.0 and ≤ 35.0
D	>35.0 and ≤ 55.0
E	>55.0 and ≤ 80.0
F	>80.0

Delay is a complex measure and depends on a number of variables, including the quality of progression, the cycle length, the green ratio, and the v/c ratio for the lane group.

Level of Service A describes operations with low control delay, up to 10 sec per vehicle. This level of service occurs when progression is extremely favorable and most vehicles arrive during the green phase. Many vehicles do not stop at all. Short cycle lengths may tend to contribute to low delay values.

Level of Service B describes operations with control delay greater than 10 and up to 20 sec per vehicle. This level generally occurs with good progression, short cycle lengths, or both. More vehicles stop than with LOS A, causing higher levels of delay.

Level of Service C describes operations with control delay greater than 20 and up to 35 sec per vehicle. These higher delays may result from only fair progression, longer cycle lengths, or both. Individual cycle failures may begin to appear at this level. Cycle failure occurs when a given green phase does not serve queued vehicles and overflows occur. The number of vehicles stopping is significant at this level, though many still pass through the intersection without stopping.

Level of Service D describes operations with control delay greater than 35 and up to 55 sec per vehicle. At level of service D, the influence of congestion becomes more noticeable. Longer delays may result from some combination of unfavorable progression, long cycle lengths, or high v/c ratios. Many vehicles stop, and the proportion of vehicles not stopping declines. Individual cycle failures are noticeable.

"Highway Capacity Manual," Transportation Research Board, 2000.

Level of Service E describes operation with control delay greater than 55 and up to 80 sec per vehicle. These high delay values generally indicate poor progression, long cycle lengths, and high v/c ratios. Individual cycle failures are frequent.

Level of Service F describes operations with control delay in excess of 80 sec per vehicle. This level, considered to be unacceptable to most drivers, often occurs with oversaturation, that is, when arrival flow rates exceed the capacity lane groups. It may also occur at high v/c ratios with many individual cycle failures. Poor progression and long cycle lengths may also contribute significantly to high delay levels.

"Highway Capacity Manual," Transportation Research Board, 2000.

LEVEL OF SERVICE DEFINITIONS

LEVEL-OF-SERVICE CRITERIA FOR UNSIGNALIZED INTERSECTIONS

Level of Service (LOS) criteria are given in Table 1. As used here, control delay is defined as the total elapsed time from the time a vehicle stops at the end of the queue to the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of vehicles from free-flow speed to the speed of vehicles in the queue.

The average total delay for any particular minor movement is a function of the service rate or capacity of the approach and the degree of saturation. If the degree of saturation is greater than about 0.9, average control delay is significantly affected by the length of the analysis period.

Table 1: Level-of-Service Criteria for Unsignalized Intersections

Level of Service	Average Control Delay (Sec/Veh)
A	≤10.0
B	>10.0 and ≤15.0
C	>15.0 and ≤25.0
D	>25.0 and ≤35.0
E	>35.0 and ≤50.0
F	>50.0

"Highway Capacity Manual," Transportation Research Board, 2000.

APPENDIX C

CAPACITY ANALYSIS CALCULATIONS
EXISTING PEAK HOUR TRAFFIC ANALYSIS

Lane Configurations	↰↰	↑↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	209	774	34	29	1315	108	58	78	36	85	97	697
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	1.00		1.00	1.00	0.85	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	5037		1770	5085	1350	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	5037		1770	5085	1350	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.92	0.92	0.92	0.90	0.90	0.90	0.87	0.87	0.87	0.88	0.88	0.88
Adj. Flow (vph)	227	841	37	32	1461	120	67	90	41	97	110	792
RTOR Reduction (vph)	0	3	0	0	0	55	0	0	36	0	0	104
Lane Group Flow (vph)	227	875	0	32	1461	65	67	90	5	97	110	688
Confl. Peds. (#/hr)			33			90						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	16.1	63.3		6.0	54.2	54.2	11.3	11.3	11.3	31.7	31.7	52.8
Effective Green, g (s)	16.1	63.3		6.0	54.2	54.2	11.3	11.3	11.3	31.7	31.7	52.8
Actuated g/C Ratio	0.12	0.47		0.05	0.41	0.41	0.08	0.08	0.08	0.24	0.24	0.40
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	415	2392		80	2068	549	150	158	134	816	443	1104
v/s Ratio Prot	0.07	0.17		0.02	c0.29		0.04	c0.05		0.03	0.06	c0.25
v/s Ratio Perm						0.05			0.00			
v/c Ratio	0.55	0.37		0.40	0.71	0.12	0.45	0.57	0.04	0.12	0.25	0.62
Uniform Delay, d1	55.2	22.2		61.9	32.9	24.7	58.0	58.7	56.0	39.8	41.1	32.3
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	5.1	0.4		14.2	2.1	0.4	2.1	4.7	0.1	0.1	0.3	1.1
Delay (s)	60.3	22.7		76.1	35.0	25.1	60.1	63.3	56.1	39.9	41.4	33.4
Level of Service	E	C		E	C	C	E	E	E	D	D	C
Approach Delay (s)		30.4			35.1			60.8			34.9	
Approach LOS		C			D			E			C	

HCM Average Control Delay	35.0	HCM Level of Service	D
HCM Volume to Capacity ratio	0.66		
Actuated Cycle Length (s)	138.3	Sum of lost time (s)	15.0
Intersection Capacity Utilization	66.5%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

	↗	→	↘	↙	←	↖	↖	↑	↗	↘	↓	↙
Lane Configurations	↰↰	↑↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	540	1341	90	83	1107	340	110	163	59	164	98	210
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	0.99		1.00	1.00	0.68	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	4995		1770	5085	1078	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	4995		1770	5085	1078	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.88	0.88	0.88	0.92	0.92	0.92	0.74	0.74	0.74	0.91	0.91	0.91
Adj. Flow (vph)	614	1524	102	90	1203	370	149	220	80	180	108	231
RTOR Reduction (vph)	0	5	0	0	0	196	0	0	28	0	0	106
Lane Group Flow (vph)	614	1621	0	90	1203	174	149	220	52	180	108	125
Confl. Peds. (#/hr)			78			272						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	29.1	55.2		10.0	37.1	37.1	18.1	18.1	18.1	13.0	13.0	47.1
Effective Green, g (s)	29.1	55.2		10.0	37.1	37.1	18.1	18.1	18.1	13.0	13.0	47.1
Actuated g/C Ratio	0.25	0.47		0.09	0.32	0.32	0.15	0.15	0.15	0.11	0.11	0.40
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	852	2351		151	1608	341	273	287	244	380	206	1119
v/s Ratio Prot	c0.18	0.32		0.05	c0.24		0.08	c0.12		0.05	c0.06	0.04
v/s Ratio Perm						0.16			0.03			
v/c Ratio	0.72	0.69		0.60	0.75	0.51	0.55	0.77	0.21	0.47	0.52	0.11
Uniform Delay, d1	40.4	24.3		51.7	35.9	32.7	45.8	47.6	43.4	48.9	49.2	22.0
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	5.2	1.7		16.2	3.2	5.4	2.2	11.6	0.4	0.9	2.4	0.0
Delay (s)	45.6	26.0		67.9	39.1	38.0	48.0	59.1	43.8	49.9	51.6	22.0
Level of Service	D	C		E	D	D	D	E	D	D	D	C
Approach Delay (s)		31.4			40.5			52.7			37.8	
Approach LOS		C			D			D			D	

HCM Average Control Delay	37.1	HCM Level of Service	D
HCM Volume to Capacity ratio	0.71		
Actuated Cycle Length (s)	117.3	Sum of lost time (s)	20.0
Intersection Capacity Utilization	67.9%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Lane Configurations						
Volume (veh/h)	819	76	0	0	0	36
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.95	0.95	0.95	0.95	0.82	0.82
Hourly flow rate (vph)	862	80	0	0	0	44
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.93		0.93	0.93
vC, conflicting volume			862		902	327
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			608		650	35
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	95
cM capacity (veh/h)			904		375	962

Volume Total	345	345	252	0	0	0	44
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	80	0	0	0	44
cSH	1700	1700	1700	1700	1700	1700	962
Volume to Capacity	0.20	0.20	0.15	0.00	0.00	0.00	0.05
Queue Length 95th (ft)	0	0	0	0	0	0	4
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	8.9
Lane LOS							A
Approach Delay (s)	0.0			0.0			8.9
Approach LOS							A

Average Delay	0.4		
Intersection Capacity Utilization	27.5%	ICU Level of Service	A
Analysis Period (min)	15		

Lane Configurations	↑↑↑↱			↑↑↑		↱
Volume (veh/h)	1402	162	0	0	0	38
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.92	0.92	0.91	0.91	0.72	0.72
Hourly flow rate (vph)	1524	176	0	0	0	53
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.78		0.78	0.78
vC, conflicting volume			1524		1612	596
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			685		798	0
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	94
cM capacity (veh/h)			706		252	846

Volume Total	610	610	481	0	0	0	53
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	176	0	0	0	53
cSH	1700	1700	1700	1700	1700	1700	846
Volume to Capacity	0.36	0.36	0.28	0.00	0.00	0.00	0.06
Queue Length 95th (ft)	0	0	0	0	0	0	5
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	9.5
Lane LOS							A
Approach Delay (s)	0.0			0.0			9.5
Approach LOS							A

Average Delay	0.3		
Intersection Capacity Utilization	40.7%	ICU Level of Service	A
Analysis Period (min)	15		

Lane Configurations	↰	↑↑↱		↰	↑↑↱		↰	↔			↰	↱
Volume (vph)	116	716	26	141	934	39	118	22	33	32	80	400
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.98		1.00	0.86			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.94			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (prot)	1770	4970		1770	4933		1681	1404			1836	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (perm)	1770	4970		1770	4933		1681	1404			1836	1583
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.77	0.77	0.70	0.95	0.95	0.95
Adj. Flow (vph)	122	754	27	148	983	41	153	29	47	34	84	421
RTOR Reduction (vph)	0	3	0	0	3	0	0	19	0	0	0	293
Lane Group Flow (vph)	122	778	0	148	1021	0	116	94	0	0	118	128
Confl. Peds. (#/hr)			145			278			222			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	19.2	39.5		22.2	42.5		13.1	13.1			15.3	15.3
Effective Green, g (s)	19.2	39.5		22.2	42.5		13.1	13.1			15.3	15.3
Actuated g/C Ratio	0.17	0.36		0.20	0.39		0.12	0.12			0.14	0.14
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	309	1783		357	1904		200	167			255	220
v/s Ratio Prot	0.07	0.16		c0.08	c0.21		c0.07	0.07			0.06	
v/s Ratio Perm												c0.08
v/c Ratio	0.39	0.44		0.41	0.54		0.58	0.56			0.46	0.58
Uniform Delay, d1	40.3	26.8		38.3	26.2		45.9	45.8			43.6	44.4
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	3.7	0.8		3.5	1.1		4.0	4.3			1.3	3.9
Delay (s)	44.0	27.6		41.8	27.3		49.9	50.0			44.9	48.3
Level of Service	D	C		D	C		D	D			D	D
Approach Delay (s)		29.8			29.1			50.0			47.6	
Approach LOS		C			C			D			D	

HCM Average Control Delay	34.5	HCM Level of Service	C
HCM Volume to Capacity ratio	0.54		
Actuated Cycle Length (s)	110.1	Sum of lost time (s)	20.0
Intersection Capacity Utilization	69.9%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Lane Configurations	↰	↑↑↑↱		↰	↑↑↑↱		↰	↰↑↱			↰↑	↱
Volume (vph)	279	1112	49	169	1001	51	244	30	25	26	60	286
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.96		1.00	0.93			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.97			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (prot)	1770	4934		1770	4867		1681	1558			1835	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (perm)	1770	4934		1770	4867		1681	1558			1835	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.91	0.91	0.91	0.88	0.88	0.88	0.89	0.89	0.89
Adj. Flow (vph)	303	1209	53	186	1100	56	277	34	28	29	67	321
RTOR Reduction (vph)	0	3	0	0	4	0	0	5	0	0	0	290
Lane Group Flow (vph)	303	1259	0	186	1152	0	172	162	0	0	96	31
Confl. Peds. (#/hr)			194			428			347			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	34.1	52.2		24.1	42.2		17.5	17.5			12.2	12.2
Effective Green, g (s)	34.1	52.2		24.1	42.2		17.5	17.5			12.2	12.2
Actuated g/C Ratio	0.27	0.41		0.19	0.33		0.14	0.14			0.10	0.10
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	479	2044		339	1630		233	216			178	153
v/s Ratio Prot	c0.17	0.26		0.11	c0.24		0.10	c0.10			c0.05	
v/s Ratio Perm												0.02
v/c Ratio	0.63	0.62		0.55	0.71		0.74	0.75			0.54	0.20
Uniform Delay, d1	40.4	29.0		46.0	36.5		52.1	52.1			54.2	52.4
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	6.2	1.4		6.3	2.6		11.5	13.3			3.1	0.7
Delay (s)	46.7	30.4		52.3	39.1		63.6	65.4			57.3	53.1
Level of Service	D	C		D	D		E	E			E	D
Approach Delay (s)		33.6			40.9			64.5			54.1	
Approach LOS		C			D			E			D	

HCM Average Control Delay	41.5	HCM Level of Service	D
HCM Volume to Capacity ratio	0.67		
Actuated Cycle Length (s)	126.0	Sum of lost time (s)	20.0
Intersection Capacity Utilization	64.6%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

APPENDIX D

CAPACITY ANALYSIS CALCULATIONS
PROJECTED YEAR 2013 PEAK HOUR TRAFFIC
ANALYSIS WITHOUT PROJECT

Lane Configurations	↰↰	↑↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	209	775	34	30	1363	112	58	78	38	85	97	697
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	1.00		1.00	1.00	0.85	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	5037		1770	5085	1347	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	5037		1770	5085	1347	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.92	0.92	0.92	0.90	0.90	0.90	0.87	0.87	0.87	0.88	0.88	0.88
Adj. Flow (vph)	227	842	37	33	1514	124	67	90	44	97	110	792
RTOR Reduction (vph)	0	3	0	0	0	55	0	0	39	0	0	91
Lane Group Flow (vph)	227	876	0	33	1514	69	67	90	5	97	110	701
Confl. Peds. (#/hr)			33			90						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	16.0	65.2		6.0	56.2	56.2	11.0	11.0	11.0	31.8	31.8	52.8
Effective Green, g (s)	16.0	65.2		6.0	56.2	56.2	11.0	11.0	11.0	31.8	31.8	52.8
Actuated g/C Ratio	0.12	0.48		0.04	0.42	0.42	0.08	0.08	0.08	0.24	0.24	0.39
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	407	2433		79	2117	561	144	152	129	809	439	1090
v/s Ratio Prot	0.07	0.17		0.02	c0.30		0.04	c0.05		0.03	0.06	c0.25
v/s Ratio Perm						0.05			0.00			
v/c Ratio	0.56	0.36		0.42	0.72	0.12	0.47	0.59	0.04	0.12	0.25	0.64
Uniform Delay, d1	56.2	21.8		62.8	32.7	24.2	59.2	59.8	57.1	40.6	41.9	33.4
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	5.4	0.4		15.4	2.1	0.5	2.4	6.1	0.1	0.1	0.3	1.3
Delay (s)	61.6	22.3		78.2	34.8	24.7	61.6	65.9	57.3	40.7	42.2	34.8
Level of Service	E	C		E	C	C	E	E	E	D	D	C
Approach Delay (s)		30.3			35.0			62.6			36.1	
Approach LOS		C			C			E			D	

HCM Average Control Delay	35.4	HCM Level of Service	D
HCM Volume to Capacity ratio	0.67		
Actuated Cycle Length (s)	135.0	Sum of lost time (s)	15.0
Intersection Capacity Utilization	67.4%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

	↗	→	↘	↙	←	↖	↖	↑	↗	↘	↓	↙
Lane Configurations	↰↰	↑↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	540	1345	90	84	1126	346	110	163	62	165	98	210
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	0.99		1.00	1.00	0.68	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	4995		1770	5085	1077	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	4995		1770	5085	1077	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.88	0.88	0.88	0.92	0.92	0.92	0.74	0.74	0.74	0.91	0.91	0.91
Adj. Flow (vph)	614	1528	102	91	1224	376	149	220	84	181	108	231
RTOR Reduction (vph)	0	5	0	0	0	196	0	0	29	0	0	106
Lane Group Flow (vph)	614	1625	0	91	1224	180	149	220	55	181	108	125
Confl. Peds. (#/hr)			78			272						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	29.1	55.2		10.0	37.1	37.1	18.1	18.1	18.1	13.2	13.2	47.3
Effective Green, g (s)	29.1	55.2		10.0	37.1	37.1	18.1	18.1	18.1	13.2	13.2	47.3
Actuated g/C Ratio	0.25	0.47		0.09	0.32	0.32	0.15	0.15	0.15	0.11	0.11	0.40
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	850	2347		151	1606	340	273	287	244	386	209	1122
v/s Ratio Prot	c0.18	0.33		0.05	c0.24		0.08	c0.12		0.05	c0.06	0.04
v/s Ratio Perm						0.17			0.03			
v/c Ratio	0.72	0.69		0.60	0.76	0.53	0.55	0.77	0.23	0.47	0.52	0.11
Uniform Delay, d1	40.5	24.5		51.8	36.2	33.0	45.9	47.7	43.6	48.9	49.1	22.0
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	5.3	1.7		16.5	3.5	5.8	2.2	11.6	0.5	0.9	2.2	0.0
Delay (s)	45.8	26.2		68.4	39.7	38.8	48.1	59.2	44.0	49.8	51.3	22.0
Level of Service	D	C		E	D	D	D	E	D	D	D	C
Approach Delay (s)		31.5			41.0			52.8			37.8	
Approach LOS		C			D			D			D	

HCM Average Control Delay	37.4	HCM Level of Service	D
HCM Volume to Capacity ratio	0.72		
Actuated Cycle Length (s)	117.5	Sum of lost time (s)	20.0
Intersection Capacity Utilization	67.9%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

[redacted]

Lane Configurations	↑↑↱			↑↑↑		↱
Volume (veh/h)	820	76	0	0	0	36
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.95	0.95	0.95	0.95	0.82	0.82
Hourly flow rate (vph)	863	80	0	0	0	44
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.94		0.94	0.94
vC, conflicting volume			863		903	328
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			615		658	43
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	95
cM capacity (veh/h)			899		372	952

[redacted]

Volume Total	345	345	253	0	0	0	44
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	80	0	0	0	44
cSH	1700	1700	1700	1700	1700	1700	952
Volume to Capacity	0.20	0.20	0.15	0.00	0.00	0.00	0.05
Queue Length 95th (ft)	0	0	0	0	0	0	4
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	9.0
Lane LOS							A
Approach Delay (s)	0.0			0.0			9.0
Approach LOS							A

[redacted]

Average Delay	0.4		
Intersection Capacity Utilization	27.5%	ICU Level of Service	A
Analysis Period (min)	15		

Lane Configurations	↑↑↱			↑↑↑		↱
Volume (veh/h)	1407	162	0	0	0	38
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.92	0.92	0.91	0.91	0.72	0.72
Hourly flow rate (vph)	1529	176	0	0	0	53
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.78		0.78	0.78
vC, conflicting volume			1529		1617	598
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			687		800	0
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	94
cM capacity (veh/h)			703		251	845

Volume Total	612	612	482	0	0	0	53
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	176	0	0	0	53
cSH	1700	1700	1700	1700	1700	1700	845
Volume to Capacity	0.36	0.36	0.28	0.00	0.00	0.00	0.06
Queue Length 95th (ft)	0	0	0	0	0	0	5
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	9.5
Lane LOS							A
Approach Delay (s)	0.0			0.0			9.5
Approach LOS							A

Average Delay	0.3		
Intersection Capacity Utilization	40.8%	ICU Level of Service	A
Analysis Period (min)	15		

Lane Configurations	↰	↑↑↱		↰	↑↑↱		↰	✥			↰↑	↱
Volume (vph)	117	713	26	141	970	39	118	22	33	32	80	417
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.98		1.00	0.86			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.94			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (prot)	1770	4969		1770	4937		1681	1401			1836	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (perm)	1770	4969		1770	4937		1681	1401			1836	1583
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.77	0.77	0.70	0.95	0.95	0.95
Adj. Flow (vph)	123	751	27	148	1021	41	153	29	47	34	84	439
RTOR Reduction (vph)	0	3	0	0	2	0	0	19	0	0	0	260
Lane Group Flow (vph)	123	775	0	148	1060	0	116	94	0	0	118	179
Confl. Peds. (#/hr)			145			278			222			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	18.2	42.6		20.3	44.7		13.1	13.1			18.3	18.3
Effective Green, g (s)	18.2	42.6		20.3	44.7		13.1	13.1			18.3	18.3
Actuated g/C Ratio	0.16	0.37		0.18	0.39		0.11	0.11			0.16	0.16
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	282	1852		314	1931		193	161			294	253
v/s Ratio Prot	0.07	0.16		c0.08	c0.21		c0.07	0.07			0.06	
v/s Ratio Perm												c0.11
v/c Ratio	0.44	0.42		0.47	0.55		0.60	0.59			0.40	0.71
Uniform Delay, d1	43.4	26.6		42.2	27.0		48.1	48.0			43.1	45.5
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	4.8	0.7		5.0	1.1		5.2	5.4			0.9	8.8
Delay (s)	48.3	27.3		47.2	28.1		53.3	53.4			44.0	54.3
Level of Service	D	C		D	C		D	D			D	D
Approach Delay (s)		30.2			30.4			53.3			52.1	
Approach LOS		C			C			D			D	

HCM Average Control Delay	36.3	HCM Level of Service	D
HCM Volume to Capacity ratio	0.55		
Actuated Cycle Length (s)	114.3	Sum of lost time (s)	15.0
Intersection Capacity Utilization	71.6%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

	↗	→	↘	↙	←	↖	↖	↑	↗	↘	↓	↙
Lane Configurations	↰	↑↑↑↱		↰	↑↑↑↱		↰	↰↑↱			↰↑	↱
Volume (vph)	284	1112	49	169	1019	51	244	30	25	26	60	294
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.96		1.00	0.93			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.97			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (prot)	1770	4934		1770	4871		1681	1559			1835	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (perm)	1770	4934		1770	4871		1681	1559			1835	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.91	0.91	0.91	0.88	0.88	0.88	0.89	0.89	0.89
Adj. Flow (vph)	309	1209	53	186	1120	56	277	34	28	29	67	330
RTOR Reduction (vph)	0	3	0	0	3	0	0	5	0	0	0	298
Lane Group Flow (vph)	309	1259	0	186	1173	0	172	162	0	0	96	32
Confl. Peds. (#/hr)			194			428			347			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	34.1	51.2		24.1	41.2		17.4	17.4			12.3	12.3
Effective Green, g (s)	34.1	51.2		24.1	41.2		17.4	17.4			12.3	12.3
Actuated g/C Ratio	0.27	0.41		0.19	0.33		0.14	0.14			0.10	0.10
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	483	2021		341	1605		234	217			181	156
v/s Ratio Prot	c0.17	0.26		0.11	c0.24		0.10	c0.10			c0.05	
v/s Ratio Perm												0.02
v/c Ratio	0.64	0.62		0.55	0.73		0.74	0.75			0.53	0.21
Uniform Delay, d1	40.0	29.2		45.5	37.0		51.6	51.7			53.6	51.9
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	6.4	1.5		6.1	3.0		11.3	13.0			3.0	0.7
Delay (s)	46.4	30.7		51.7	40.0		62.9	64.7			56.6	52.5
Level of Service	D	C		D	D		E	E			E	D
Approach Delay (s)		33.8			41.6			63.8			53.4	
Approach LOS		C			D			E			D	

HCM Average Control Delay	41.7	HCM Level of Service	D
HCM Volume to Capacity ratio	0.68		
Actuated Cycle Length (s)	125.0	Sum of lost time (s)	20.0
Intersection Capacity Utilization	65.3%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

APPENDIX E

CAPACITY ANALYSIS CALCULATIONS
PROJECTED YEAR 2013 PEAK HOUR TRAFFIC
ANALYSIS WITH PROJECT

	↗	→	↘	↙	←	↖	↖	↑	↗	↘	↓	↙
Lane Configurations	↰↰	↑↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	209	784	34	31	1393	114	58	78	38	86	97	697
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	1.00		1.00	1.00	0.85	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	5038		1770	5085	1346	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	5038		1770	5085	1346	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.92	0.92	0.92	0.90	0.90	0.90	0.87	0.87	0.87	0.88	0.88	0.88
Adj. Flow (vph)	227	852	37	34	1548	127	67	90	44	98	110	792
RTOR Reduction (vph)	0	3	0	0	0	55	0	0	39	0	0	90
Lane Group Flow (vph)	227	886	0	34	1548	72	67	90	5	98	110	702
Confl. Peds. (#/hr)			33			90						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	16.0	66.1		6.0	57.1	57.1	11.0	11.0	11.0	31.4	31.4	52.4
Effective Green, g (s)	16.0	66.1		6.0	57.1	57.1	11.0	11.0	11.0	31.4	31.4	52.4
Actuated g/C Ratio	0.12	0.49		0.04	0.42	0.42	0.08	0.08	0.08	0.23	0.23	0.39
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	405	2458		78	2143	567	144	151	129	796	432	1078
v/s Ratio Prot	0.07	0.18		0.02	c0.30		0.04	c0.05		0.03	0.06	c0.25
v/s Ratio Perm						0.05			0.00			
v/c Ratio	0.56	0.36		0.44	0.72	0.13	0.47	0.60	0.04	0.12	0.25	0.65
Uniform Delay, d1	56.4	21.6		63.1	32.6	24.0	59.4	60.1	57.4	41.2	42.5	34.1
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	5.5	0.4		16.7	2.1	0.5	2.4	6.2	0.1	0.1	0.3	1.4
Delay (s)	61.9	22.0		79.8	34.8	24.4	61.8	66.3	57.5	41.2	42.8	35.5
Level of Service	E	C		E	C	C	E	E	E	D	D	D
Approach Delay (s)		30.1			34.9			62.9			36.8	
Approach LOS		C			C			E			D	

HCM Average Control Delay	35.4	HCM Level of Service	D
HCM Volume to Capacity ratio	0.68		
Actuated Cycle Length (s)	135.5	Sum of lost time (s)	15.0
Intersection Capacity Utilization	68.0%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Lane Configurations	↰↰	↑↑↱		↰	↑↑↑	↱	↰	↑	↱	↰↰	↑	↱↱
Volume (vph)	540	1365	90	85	1142	351	110	163	62	167	98	210
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0
Lane Util. Factor	0.97	0.91		1.00	0.91	1.00	1.00	1.00	1.00	0.97	1.00	0.88
Frpb, ped/bikes	1.00	0.99		1.00	1.00	0.68	1.00	1.00	1.00	1.00	1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Frt	1.00	0.99		1.00	1.00	0.85	1.00	1.00	0.85	1.00	1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (prot)	3433	4996		1770	5085	1077	1770	1863	1583	3433	1863	2787
Flt Permitted	0.95	1.00		0.95	1.00	1.00	0.95	1.00	1.00	0.95	1.00	1.00
Satd. Flow (perm)	3433	4996		1770	5085	1077	1770	1863	1583	3433	1863	2787
Peak-hour factor, PHF	0.88	0.88	0.88	0.92	0.92	0.92	0.74	0.74	0.74	0.91	0.91	0.91
Adj. Flow (vph)	614	1551	102	92	1241	382	149	220	84	184	108	231
RTOR Reduction (vph)	0	5	0	0	0	196	0	0	29	0	0	93
Lane Group Flow (vph)	614	1648	0	92	1241	186	149	220	55	184	108	138
Confl. Peds. (#/hr)			78			272						
Turn Type	Prot			Prot		Perm	Split		Perm	Split		pt+ov
Protected Phases	5	2		1	6		3	3		4	4	4 5
Permitted Phases						6			3			
Actuated Green, G (s)	28.1	56.2		10.0	39.1	39.1	17.8	17.8	17.8	13.2	13.2	46.3
Effective Green, g (s)	28.1	56.2		10.0	39.1	39.1	17.8	17.8	17.8	13.2	13.2	46.3
Actuated g/C Ratio	0.24	0.48		0.08	0.33	0.33	0.15	0.15	0.15	0.11	0.11	0.39
Clearance Time (s)	5.0	6.0		5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	
Vehicle Extension (s)	2.0	3.0		2.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	
Lane Grp Cap (vph)	816	2375		150	1682	356	267	281	238	383	208	1092
v/s Ratio Prot	c0.18	0.33		0.05	c0.24		0.08	c0.12		0.05	c0.06	0.05
v/s Ratio Perm						0.17			0.03			
v/c Ratio	0.75	0.69		0.61	0.74	0.52	0.56	0.78	0.23	0.48	0.52	0.13
Uniform Delay, d1	41.8	24.3		52.2	35.0	32.0	46.6	48.3	44.2	49.3	49.5	23.0
Progression Factor	1.00	1.00		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Incremental Delay, d2	6.3	1.7		17.3	2.9	5.4	2.5	13.3	0.5	1.0	2.2	0.1
Delay (s)	48.2	26.0		69.5	38.0	37.4	49.1	61.6	44.7	50.2	51.7	23.1
Level of Service	D	C		E	D	D	D	E	D	D	D	C
Approach Delay (s)		32.0			39.5			54.3			38.5	
Approach LOS		C			D			D			D	

HCM Average Control Delay	37.3	HCM Level of Service	D
HCM Volume to Capacity ratio	0.72		
Actuated Cycle Length (s)	118.2	Sum of lost time (s)	20.0
Intersection Capacity Utilization	68.1%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Lane Configurations	↑↑↑>			↑↑↑		↱
Volume (veh/h)	820	86	0	0	0	56
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.95	0.95	0.95	0.95	0.82	0.82
Hourly flow rate (vph)	863	91	0	0	0	68
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.94		0.94	0.94
vC, conflicting volume			863		908	333
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			611		659	44
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	93
cM capacity (veh/h)			902		371	951

Volume Total	345	345	263	0	0	0	68
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	91	0	0	0	68
cSH	1700	1700	1700	1700	1700	1700	951
Volume to Capacity	0.20	0.20	0.15	0.00	0.00	0.00	0.07
Queue Length 95th (ft)	0	0	0	0	0	0	6
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	9.1
Lane LOS							A
Approach Delay (s)	0.0			0.0			9.1
Approach LOS							A

Average Delay	0.6		
Intersection Capacity Utilization	27.9%	ICU Level of Service	A
Analysis Period (min)	15		

HCM Unsignalized Intersection Capacity Analysis
34: Ala Moana Blvd & 7/23/2010

	↘	↲	↖ (left)	↖	↰	↗ (right)
Lane Configurations	↑↑↑↱			↑↑↑		↱
Volume (veh/h)	1407	185	0	0	0	58
Sign Control	Free			Free	Yield	
Grade	0%			0%	0%	
Peak Hour Factor	0.92	0.92	0.91	0.91	0.72	0.72
Hourly flow rate (vph)	1529	201	0	0	0	81
Pedestrians						
Lane Width (ft)						
Walking Speed (ft/s)						
Percent Blockage						
Right turn flare (veh)						
Median type	None			None		
Median storage veh)						
Upstream signal (ft)	1227			648		
pX, platoon unblocked			0.78		0.78	0.78
vC, conflicting volume			1529		1630	610
vC1, stage 1 conf vol						
vC2, stage 2 conf vol						
vCu, unblocked vol			668		798	0
tC, single (s)			4.1		6.8	6.9
tC, 2 stage (s)						
tF (s)			2.2		3.5	3.3
p0 queue free %			100		100	90
cM capacity (veh/h)			711		251	841

Volume Total	612	612	507	0	0	0	81
Volume Left	0	0	0	0	0	0	0
Volume Right	0	0	201	0	0	0	81
cSH	1700	1700	1700	1700	1700	1700	841
Volume to Capacity	0.36	0.36	0.30	0.00	0.00	0.00	0.10
Queue Length 95th (ft)	0	0	0	0	0	0	8
Control Delay (s)	0.0	0.0	0.0	0.0	0.0	0.0	9.7
Lane LOS							A
Approach Delay (s)	0.0			0.0			9.7
Approach LOS							A

Average Delay	0.4		
Intersection Capacity Utilization	41.6%	ICU Level of Service	A
Analysis Period (min)	15		

	↗	→	↘	↙	←	↖	↖	↑	↗	↘	↓	↙
Lane Configurations	↰	↑↑↱		↰	↑↑↱		↰	↰↑↱			↰↑	↱
Volume (vph)	120	730	26	158	970	39	151	22	33	32	80	417
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.98		1.00	0.88			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.95			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (prot)	1770	4971		1770	4938		1681	1444			1836	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.98			0.99	1.00
Satd. Flow (perm)	1770	4971		1770	4938		1681	1444			1836	1583
Peak-hour factor, PHF	0.95	0.95	0.95	0.95	0.95	0.95	0.77	0.77	0.70	0.95	0.95	0.95
Adj. Flow (vph)	126	768	27	166	1021	41	196	29	47	34	84	439
RTOR Reduction (vph)	0	3	0	0	3	0	0	15	0	0	0	262
Lane Group Flow (vph)	126	792	0	166	1059	0	139	118	0	0	118	177
Confl. Peds. (#/hr)			145			278			222			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	18.3	38.6		22.3	42.6		14.7	14.7			18.1	18.1
Effective Green, g (s)	18.3	38.6		22.3	42.6		14.7	14.7			18.1	18.1
Actuated g/C Ratio	0.16	0.34		0.20	0.37		0.13	0.13			0.16	0.16
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	285	1688		347	1850		217	187			292	252
v/s Ratio Prot	0.07	0.16		c0.09	c0.21		c0.08	0.08			0.06	
v/s Ratio Perm												c0.11
v/c Ratio	0.44	0.47		0.48	0.57		0.64	0.63			0.40	0.70
Uniform Delay, d1	43.1	29.5		40.5	28.3		47.0	46.9			43.0	45.2
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	4.9	0.9		4.7	1.3		6.3	6.8			0.9	8.5
Delay (s)	48.0	30.4		45.2	29.6		53.3	53.7			43.9	53.7
Level of Service	D	C		D	C		D	D			D	D
Approach Delay (s)		32.8			31.7			53.5			51.7	
Approach LOS		C			C			D			D	

HCM Average Control Delay	37.8	HCM Level of Service	D
HCM Volume to Capacity ratio	0.60		
Actuated Cycle Length (s)	113.7	Sum of lost time (s)	20.0
Intersection Capacity Utilization	71.6%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Lane Configurations	↰	↑↑↱		↰	↑↑↱		↰	✥			↲	↱
Volume (vph)	288	1128	49	193	1019	51	266	30	25	26	60	294
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Lane Util. Factor	1.00	0.91		1.00	0.91		0.95	0.95			1.00	1.00
Frpb, ped/bikes	1.00	0.98		1.00	0.96		1.00	0.94			1.00	1.00
Flpb, ped/bikes	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Frt	1.00	0.99		1.00	0.99		1.00	0.98			1.00	0.85
Flt Protected	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (prot)	1770	4936		1770	4870		1681	1567			1835	1583
Flt Permitted	0.95	1.00		0.95	1.00		0.95	0.97			0.99	1.00
Satd. Flow (perm)	1770	4936		1770	4870		1681	1567			1835	1583
Peak-hour factor, PHF	0.92	0.92	0.92	0.91	0.91	0.91	0.88	0.88	0.88	0.89	0.89	0.89
Adj. Flow (vph)	313	1226	53	212	1120	56	302	34	28	29	67	330
RTOR Reduction (vph)	0	3	0	0	3	0	0	5	0	0	0	298
Lane Group Flow (vph)	313	1276	0	212	1173	0	184	175	0	0	96	32
Confl. Peds. (#/hr)			194			428			347			
Turn Type	Prot			Prot			Split			Split		Perm
Protected Phases	5	2		1	6		3	3		4	4	
Permitted Phases												4
Actuated Green, G (s)	34.1	48.2		27.1	41.2		18.5	18.5			12.3	12.3
Effective Green, g (s)	34.1	48.2		27.1	41.2		18.5	18.5			12.3	12.3
Actuated g/C Ratio	0.27	0.38		0.21	0.33		0.15	0.15			0.10	0.10
Clearance Time (s)	5.0	5.0		5.0	5.0		5.0	5.0			5.0	5.0
Vehicle Extension (s)	3.0	3.0		3.0	3.0		3.0	3.0			3.0	3.0
Lane Grp Cap (vph)	479	1887		380	1591		247	230			179	154
v/s Ratio Prot	c0.18	c0.26		0.12	c0.24		0.11	c0.11			c0.05	
v/s Ratio Perm												0.02
v/c Ratio	0.65	0.68		0.56	0.74		0.74	0.76			0.54	0.21
Uniform Delay, d1	40.8	32.4		44.2	37.6		51.5	51.7			54.2	52.4
Progression Factor	1.00	1.00		1.00	1.00		1.00	1.00			1.00	1.00
Incremental Delay, d2	6.8	2.0		5.8	3.1		11.5	13.7			3.1	0.7
Delay (s)	47.6	34.4		50.0	40.7		63.1	65.4			57.3	53.1
Level of Service	D	C		D	D		E	E			E	D
Approach Delay (s)		37.0			42.1			64.2			54.0	
Approach LOS		D			D			E			D	

HCM Average Control Delay	43.4	HCM Level of Service	D
HCM Volume to Capacity ratio	0.73		
Actuated Cycle Length (s)	126.1	Sum of lost time (s)	25.0
Intersection Capacity Utilization	66.0%	ICU Level of Service	C
Analysis Period (min)	15		

c Critical Lane Group

Exhibit "Y1"
Comment to Draft Environmental Assessment Submitted by Dave Cooper and Response to Comment

Honey Bee USA, Inc. **18 MAY 2010**

c/o Kiuchi, Nakamoto and Tanaka
1001 Bishop Street ASB Tower
Suite 1090
Honolulu, HI 96813
Attn: Keith Kiuchi

Subject: Honey Bee USA, Inc. Waikiki Landing Draft Environmental Assessment

Copies to:

Division of Boating and Natural Resources (DOBOR)
333 Queen St, Suite 300
Honolulu, HI 96813

Department of Transportation, Highways Division
Aliiaaimoku Building
869 Punchbowl St, Rm 513
Honolulu, Hi 96813
Attn: Jamie Ho

Office of Environmental Quality Control
235 South Beretania Street, Suite 702
Honolulu, Hawaii 96813
Ph. 586-4185

Department of Health
Environmental Planning Office
919 Ala Moana Blvd Rm 312
Honolulu, HI 96814
(808)586-4337

Department of Planning and Permitting
650 S King St
Honolulu, HI 96813

US Army Corps of Engineers
Regulatory Branch
Building 230
Fort Shafter, HI 96858-5440

Waikiki Landing Proposal

Environmental Issues and site concerns.

Based on the DEA posted by the QEQC April 23rd, 2010

Honey Bee's proposal per the DEA calls for them to demolish the current buildings, do a little superficial site work and prepare the sites for the construction of three buildings. All this with no mention of soil loading tests, excavating the sites, demolition & rebuilding of seawalls, UST's, etc. The statement that the waters surrounding these sites are polluted and is not a natural wildlife habitat is a bit of a stretch. The Ala Wai Boat Harbor is home to many different aquatic species. Disturbing the seabed and submerged lands will have an effect on them if this project goes forward as planned. It begs for good discovery, well thought out environmental planning and careful execution of the development. We see little of this in reading Honey Bee's Proposal and DEA submittal. Their DEA ends with a summary sentence that "the approving agency should determine that this project has no significant impact on the environment".

However, spending a minimal amount of time on these sites will show that this is not the case. Both sites will require removal of UST's or AST's, extensive excavation of soils contaminated from residual petroleum products and heavy metals, removal of the old shipyard rail and rail beds, the removal/rebuilding of seawalls, removal and reconstruction of the travel lift ways in and over submerged lands, dock construction and the demolition of 1950's era buildings which may contain asbestos. The soils will require pressure testing to insure that it can withstand the loading of the proposed building on these filled sites. Neither of the bulkheads surrounding either site was designed nor was the fill used to create them done with the thought of constructing multi-story buildings on this land in the future.

This project begs further site discovery, disclosure and a full EIS for the issues sited below and others that may exist and not listed here.

<u>Boatyard site</u> (also known as the Boatyard building & Canoe House site)

(a) The boatyard site has been used as a boatyard for more than 50 years. During this time boats have been scraped of bottom fouling and paints containing lead in the old days and more recently copper and TBT. These heavy metals have been washed into the soils of the entire boatyard and the adjacent seabed. The area used over the years for these activities comprise over 90% of the land proposed to be used this project. *{The DEA makes no mention of these issues. Honey Bee should address these in a more detailed EA or EIS}*

(b) Honey Bee claims that no grading will be required yet the current site has old railways and ties embed in it, the utility lines that run to the building including sewer, electrical, water and air which will all need to be dug up and removed. The site must be graded to slope such that runoff water will flow to the collection system. This will disturb the contaminated soil on the whole site which will need to be properly contained, both airborne and runoff, and disposed of. *{**<u>DEA page 10 top of the page</u>** "..the property will be graded but very little excavation of the soil on the base level of the property is anticipated." Yet on **<u>page 23 2nd para</u>** it states "pile driving driving.....grading and earth moving", which is it? Removal of the concrete and asphalt cap on the site will be required to do their proposed work. Can we have a more detailed disclosure of what this will entail?}*

(c) The demolition of the existing building is to be done with a hydraulic crusher. Yet no tests for potential asbestos, petroleum products, lead paints, varnish, oils and other contaminates routine used and stored in a typical boatyard storage building has been outlined in the DEA. *{**<u>DEA pg 23 para B</u>** " ..a dust fabric barrier will be erected..." How does this address the fine airborne contaminates and the heavy metal waste contaminates? A informed and detailed discussion is required to insure that these contaminates are not dispersed into the community and adjacent waters during the demolition phase}*

(d) Where is the soil test report? They propose to use "spread footings" foundations for a 3 (4?) story building on filled land that has not been load tested. The main building is sited across old seawalls and two/three sets of rails. Again this will call for substantial excavation of the contaminated soils. *{**<u>DEA pg 10 middle of page</u>** "Footings for this building will most likely be a spread footing." **<u>DEA report page 14 para</u>** 2 "at least two-thirds of the Boatyard Repair Site is also filled land....... Light brown silty sand with coral fragments."}*

(e) There is evidence of more than one UST's still on the site yet no mention of them in the DEA. *{**<u>DEA pg 6 bottom of page</u>** "There are no known Underground Storage Tanks (UST's) on the site". In fact there is at least one with a fill bung and possibly more.}*

(f) There was cesspool on the site when the building was constructed in 1955 which was there before the sewer connection was there. {*No mention of it in the DEA.*}

(g) The seawall, built before 1948 according to the Corp of Engineers survey mark, is in dis-repair in many sections and will need to be repaired. It was not designed/constructed for the lateral loading of a multi-story building on spread footings. {*Finding anything other than a single mention of this seawall project is impossible in this DEA. This reconstruction alone should require an EIS to make sure that the impact on the seabed and runoff is properly contained.*}

(h) The current condemned wooden dock on the EWA side is to be replaced according to the DEA. This will requires removing the over water structure that is there, driving new pilings and the construction of a new dock. {*There is no mention of what impact that this part of the project might have to the seabed/submerged lands in their DEA.*}

(i) The new boatyard which is to have a sloping surface to drain into a state of the art waste water treatment system will certainly have to be dug into to removed current concrete foundations /footing to "grade' & install such a system/device again releasing contaminates in the soil. *{**__DEA pg 14 para 4__** .."a re-grading of the site....." No mention of the excavation required in the DEA.}*

(j) The area where the wedding chapel sits appears to have some underground structures that the site plan shows the building spanning. There are two access plates there and a subsurface concrete berm. { *The DEA makes no mention of these or what may lie beneath them.}*

__Fuel Dock site__ (also known as the Diamond Vista site)

(a) This site has been in use as fueling site for over 50 years. It's different than an old gas station with underground tanks and all that entail to allow the site to re-developed. The developer includes a copy of a report from BEI which made no tests on this site. {***__Exhibit U__*** *from BEI done in 2003 and released in 2004*}

(b) There were 7 UST's which were removed however there were never soil samples taken when they removed and as far as can be determined by BEI none up to 2004. It would be reasonable to assume that the reason the tanks were removed was that one or more of them were leaking. It is therefore reasonable to assume that soil samples should be taken before demolition begins to better understand the scope of the task. DEA mentions little of this or glosses over issues. {***__DEA page 7 para b__***. *Refers to the BEI report Exhibit U pg 11 which makes no statement that the tanks were not leaking nor were any soil samples taken*}

(c) There is mention of a cesspool on the site but no definitive statement as to status other than it is not in us. The DEA makes no mention of a plan to address this. {***__DEA page 7 para b__*** *refers to the BEI report Exhibit U page 17 re the cesspool. The BEI report makes such claim that it was filled in*}

(d) The proposal states that this site will not be graded, however it currently has a crown profile and slopes generally from the mole Ewa to the sea. In order to put a building of 55' plus a tower you need substantial secure footings which will require you to dig into the contaminated soil. No mention of this excavation or the disposal of the petroleum contaminated soil in the DEA. {***__DEA page 10 Para b__*** *refers to "Minimal grading......to excavate the site." This is cannot be true given the current observable condition of the site, ref Para e, f, g & h below*}.

(e) The present underground utilities must be excavated from this soil.

(f) There is a blowhole on the site which begs the question, what is under the current asphalt cap? {*Nothing in the DEA.*}

(g) The sites fill and rip -rap has been eroded by the sea as the seawall needs replacement in many places. This will require the site to be opened and the base material to be replaced. {*No mention in their DEA.*}

(h) The 10' wide concrete perimeter apron that runs on 3 sides of the property is in poor condition with concrete and rebar falling to the seabed. It will need 100% replacement before building construction. {*No mention of this and or its effects on the seabed/submerged lands.*}

(i) The existing two 2000 gal fuel tanks are to be removed along with the concrete containment building, pipes and other fueling equipment. No mention of a plan to mitigate this demolition even to it is mere inches from the sea. {***DEA page 10 para b*** *"The existing aboveground storage tanks will be removed…."*]

(j) We also note that 16 boat slips will be part of the project yet no mention of the impact on the seabed/submerged lands in the DEA. {***DEA pg 4 top of page*** *"Honey Bee will lease16 newly renovated boat slips……….facility"*. }

In summary, I am having trouble understanding how this project has gotten this far without the basic foundations of standard site discovery, land owner disclosure requirements and good engineering practices. It is filled with jaw dropping issues that will require substantial time, money and a

processes to mitigate them. For the developer, Honey Bee, to state in their DEA that they believe this is a FONSI project vs. requiring a full EIS indicates that they have little experience in the re-development of or construction on contaminated and/or prior usage sites like the Ala Wai boatyard and fuel dock.

Dave Cooper

1777 Ala Moana #1132

Honolulu, HI 96815

80 864-8995

captdave@boats4u.com

KIUCHI, NAKAMOTO & TANAKA
ATTORNEYS AT LAW

AMERICAN SAVINGS BANK TOWER • 1001 BISHOP STREET, SUITE 1090
HONOLULU, HAWAII 96813 • (808) 521-7465 FAX: (808) 521-5873

July 30, 2010

Mr. Dave Cooper
1777 Ala Moana Blvd., #1132
Honolulu, HI 96815

Dear Mr. Cooper:

This responds, on behalf of Honey Bee USA, Inc., to your comments to the Draft Environmental Assessment dated May 18, 2010. When we received the log of the comments from the Dept. of Transportation ("DOT") on July 7, 2010, your letter entitled "Environmental Issues and Site Concerns" dated May 18, 2010 was not shown as being received by DOT and your letter addressed to the Office of Environmental Quality Control ("OEQC") was received by DOT. The letter to OEQC was specifically addressed to OEQC and was not a comment on the project. Nevertheless, pursuant to the request of DOT and OEQC we are sending this response to your comments. Your comments and this response letter are included in a supplement to the Final Environmental Assessment and it is the understanding of Honey Bee USA that it will be published with the Final Environmental Assessment.

This project has presented several challenges to Honey Bee USA, Inc. The primary challenge was creating an attractive development with a "Hawaiian sense of place" while combining that with an industrial boat repair yard and fuel dock and producing the revenue that was needed to pay DOBOR the rent set forth in its RFP. Contrary to what has been said in two community meetings, that rent is not too high given the site's location in Waikiki and as important, DOBOR has recognized that in order to maximize the return on its lands that this site must become a viable commercial development and not just a boat repair facility and fuel dock. It is simply not fair that this site is underutilized at a time when the State of Hawai`i is in a budget crisis. The rent from this site goes into the DOBOR fund and without these funds there is not enough money in the state budget to repair harbors throughout the state.

Honey Bee has, in its draft environmental assessment, complied with the contents of an environmental assessment as set forth in §11-200-10 of the Hawaii Administrative Rules. That rule states that what is required, as it relates to a description of the project, is a "general description" of the action's technical, economic, social and environmental characteristics.

With the foregoing in mind, Honey Bee's response to your comments are as follows:

Contrary to your statement there are no Underground Storage Tanks ("UST") on either the Fuel Dock Site or the Boatyard Repair Site. The 2004 BEI Phase I Environmental Study, which was attached to the EA, specifically states that all USTs were removed from that site by 1987 and that there are no remaining USTs on that site. The Executive Summary to the 2004

BEI Phase I Environmental Study sets forth all of the USTs within .25 miles of the Fuel Dock site (which would include the Boatyard Repair Site) and no UST for the Boatyard Repair Site is shown on that list. Nor is any UST for either site shown on the Dept. of Health current list of USTs. A copy of the relevant page of the Executive Summary is attached to this letter. As to the above ground storage tanks, it is Honey Bee's understanding that those tanks belong to Magic Island Petroleum, Inc. and that these above ground storage tanks will be removed by the owner at the termination of that tenant's month to month license. If these tanks are not removed by Magic Island Petroleum, Inc. then Honey Bee will remove the above ground storage tanks in accordance with all environmental regulations.

Boatyard Repair Site

On the Boatyard Repair Site Honey Bee had access to the water quality reports prepared by the previous tenant, Ala Wai Marine, Ltd. to determine the level of contaminants on the surface of the site. These water quality reports, which were submitted up to the time that Ala Wai Marine, Ltd. closed, showed that there were no contaminants being carried by stormwater from the surface of the Boatyard Repair Site into the Ala Wai Boat Harbor. Environmental regulations for the removal of any hazardous substances will be followed and a licensed company will be hired to remove these substances. Soil tests will be conducted on this property to determine if there are any contaminants in the soil and Honey Bee has budgeted for the removal if any such contaminants are found.

There will be minimal grading on both sites, but as stated in the EA a grading permit will be obtained, and a concrete pad will be poured for the new buildings. As stated above, there is no evidence, based upon these water quality studies, that there are no contaminants on the surface of the property and any other contaminants found will be removed under the applicable environmental regulations. Earth moving will only be required, as stated in the EA, to move some dirt near a retaining wall that will be constructed on the Boatyard Repair Site. There may be pile driving for the wedding facility, which will depend on the actual construction of this building (which will only require concrete piles if the construction is reinforced concrete) and for the concrete deck for the boatyard. The contaminants you discuss are not airborne in nature, except for asbestos, which can be appropriately contained.

The geotechnical soil test report will be conducted in August and that report will be given to Honey Bee by early September. The proposal to use "spread footings" arose after Honey Bee initially consulted the soils engineers. At the same time that the soil tests are being conducted an environmental company will conduct tests on the soil to determine if there are any hazardous substances in the soil and will make recommendations regarding the removal of any such hazardous substances. A fill bung is not an underground storage tank. Honey Bee has been unable to find any evidence of a fill bung on the Boatyard Repair Site and would appreciate receiving information as to the specific location of such a site so that testing can be done to determine the level, if any, of any migration of hazardous substances from this fill bung.

There is no record on file with any governmental agency showing that the Boatyard Repair Site ever had a cesspool.

Honey Bee is well aware of the condition of the seawall on the Boatyard Repair Site and will be repairing the same in accordance with all environmental regulations, which will involve a permit from the Army Corps of Engineers. Honey Bee has also retained a structural engineering firm to determine the structural load appropriate for each location on this site.

The wooden dock on the Ewa side of the Boatyard Repair Site is cantilevered over a concrete lip that is above the water. The wooden dock will be removed and reinforced from this concrete lip. No driving of new pilings will be required. The haul-out repair slip will not require new construction but will require repair. That haul-out repair slip was in use by Ala Wai Marine, Ltd. up to the date that the company closed.

The revised plan for the boatyard places the boatyard on a deck above the property and thus there will be no excavation of the property for this purpose. The filtration system is located above ground and not below ground per the recommendations of the manufacturer.

There is no record of any underground structures on the Boatyard Repair Site. We would welcome any information that you have on such structures.

Fuel Dock Site

The 2004 BEI Phase I Environmental Assessment did not test the soil on the site (which Honey Bee will) but the report notes that such a soil test report was not required by the Dept. of Health. Your assumption that the tanks were leaking is not correct given that the BEI report shows no such leaks being reported. As shown in the attached portion of the Executive Summary from that report, the Fuel Dock Site was not listed as having a Leaking Underground Storage Tank ("LUST"). Soil samples will be taken at the Fuel Dock Site, as they will with the Boatyard Repair Site.

The same 2004 report does not state that the cesspool area requires any action.

The height of the majority of the building will be 25 feet, not 55 feet. The tower is atop the 25 feet but that tower has been reduced to a single story. Very little excavation will be done but if any contaminants are found in the soil after testing, as noted above, they will be removed in accordance with the law.

We are aware of a "sink hole" at the Fuel Dock. It has been examined and the opinion is that it is caused by a potential leak space in the rock wall surrounding the Fuel Dock. This can be repaired. Honey Bee has reviewed the 2000 Engineering Report done on this site, will supplement that with an updated report and will make the necessary repairs, including repairing the concrete apron surrounding the site.

As stated above, the present operator of the fuel dock has informed the state that he owns all of the equipment on the site, including the above ground storage tanks, and has said that he will remove the same upon the termination of his month to month license . If these tanks are not removed by Magic Island Petroleum, Inc. then Honey Bee will remove the above ground storage tanks in accordance with all environmental regulations.

If floating docks are constructed for the Fuel Dock piles will have to be driven into the water which can only be approved by the Army Corps of Engineers pursuant to a separate permit. It is likely that this work will be done later in the project as such approval to drive in these pilings could take as long as a year. Honey Bee is presently looking into installing floating docks, such as those docks recently installed in Keehi Lagoon, that do not require pilings.

Finally, as to your comment that Honey Bee's statement that the Ala Wai Boat Harbor is not a "natural wildlife habitat is a bit of a stretch", the rules regarding the content of an Environmental Assessment, which is §11-200-10, do not require a study of flora and fauna in the area and §11-200-12, which is the significance criteria for evaluating an EA, looks at whether the proposed action "substantially affects a rare, threatened, or endangered species, or it habitat". Thus Honey Bee has complied with addressing this significance criteria in its EA.

Thank you for your comments.

Sincerely,

Keith M. Kiuchi

Encl.

EXECUTIVE SUMMARY

Equal/Higher Elevation	Address	Dist / Dir	Map ID	Page
RESERVE HOUSING TOWER SOIL CON	*1141 WAIMANU ST.*	*1/2 - 1 NW*	*30*	*43*
MEADOW GOLD ICE CREAM PLANT FU	*1418 YOUNG ST*	*1/2 - 1 N*	*31*	*45*
KAPIOLANI MEDICAL CENTER FOT W	1319 PUNAHOU STREET	1/2 - 1 NNE	32	53
GROVER BUILDING	*1046 WAIMANU STREET*	*1/2 - 1 NW*	*33*	*54*

LUST: The Leaking Underground Storage Tank Incident Reports contain an inventory of reported leaking underground storage tank incidents. The data come from the Department of Health's Active Leaking Underground Storage Tank Log Listing.

A review of the LUST list, as provided by EDR, and dated 08/01/2003 has revealed that there are 13 LUST sites within approximately 0.5 miles of the target property.

Equal/Higher Elevation	Address	Dist / Dir	Map ID	Page
DISCOVERY BAY	*1778 ALA MOANA BLVD*	*1/8 - 1/4SE*	*B7*	*14*
ILIKAI HOTEL	*1777 ALA MOANA BLVD*	*1/8 - 1/4SE*	*B8*	*14*
KALAKAUA TRANSMISSION INC.	*1665 KALAKAUA AVE*	*1/4 - 1/2NE*	*D14*	*20*
SEARS ROEBUCK & COMPANY #1158	*1450 ALA MOANA BLVD 10*	*1/4 - 1/2WNW*	*15*	*20*
ALOHA MOTORS	*KAPIOLANI BLVD X KALAKA*	*1/4 - 1/2NE*	*D16*	*23*
FUNAI'S UNION SERVICE L-5583	*1810 KAPIOLANI BLVD*	*1/4 - 1/2NE*	*17*	*24*
PAWAA FIRE STATION	*1610 MAKALOA ST*	*1/4 - 1/2NNE*	*E18*	*25*
7-11 MOANALUA	C/O MOANALUA SHOPPING C	1/4 - 1/2NW	19	25
GAS 'N GLO INC	*1670 MAKALOA ST*	*1/4 - 1/2NNE*	*E20*	*25*
U.S. ARMED FORCES RECREATION C	*2055 KALIA RD*	*1/4 - 1/2SE*	*21*	*26*
HOLIDAY ACTION GAS - KAHEKA	*801 KAHEKA ST*	*1/4 - 1/2N*	*F22*	*28*
FIRESTONE STORE	*801 KAHEKA ST*	*1/4 - 1/2N*	*F23*	*29*
JIMMY S CHEVRON SERVICE INC	*1958 KALAKAUA AVE*	*1/4 - 1/2ESE*	*24*	*29*

UST: The Underground Storage Tank database contains registered USTs. USTs are regulated under Subtitle I of the Resource Conservation and Recovery Act (RCRA). The data come from the Department of Health's Listing of Underground Storage Tanks.

A review of the UST list, as provided by EDR, and dated 08/01/2003 has revealed that there are 5 UST sites within approximately 0.25 miles of the target property.

Equal/Higher Elevation	Address	Dist / Dir	Map ID	Page
ALOH MARINE	1651 ALA MOANA BLVD	0 - 1/8 NW	A5	10
DISCOVERY BAY	*1778 ALA MOANA BLVD*	*1/8 - 1/4SE*	*B7*	*14*
ILIKAI HOTEL	*1777 ALA MOANA BLVD*	*1/8 - 1/4SE*	*B8*	*14*
ALA MOANA HOTEL	410 ATKINSON DR	1/8 - 1/4N	C10	15
BUDGET RENT-A-CAR SYSTEMS, INC	1837 ALA MOANA BLVD	1/8 - 1/4SE	11	15