Olowalu Town
Preliminary Traffic Impact Analysis Report

Prepared for:

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Table of Contents

List of Figures

List of Tables

Introduction, Purpose and Methodology

The Olowalu Town Master Plan is proposing to re-establish the once thriving village of Olowalu, located on the west side of the island of Maui. The subject property encompasses the lower coastal reaches of Olowalu ahupua'a; between the base of the south-west facing slopes of West Maui Mountains and the shoreline of Olowalu, as shown in Figure 1. Olowalu Town will be a small-scale and mixed-use community designed to be a pedestrian-friendly community which will allow residents to live within walking distance of corner stores, schools, parks, employment opportunities, community centers, beaches, and social and civic resources, ultimately reducing reliance on automobiles. The Master Plan is guided by values and principles of sustainability by balancing the needs of Maui's growing population; yet maintaining and respecting our cultural, historical and natural resources.

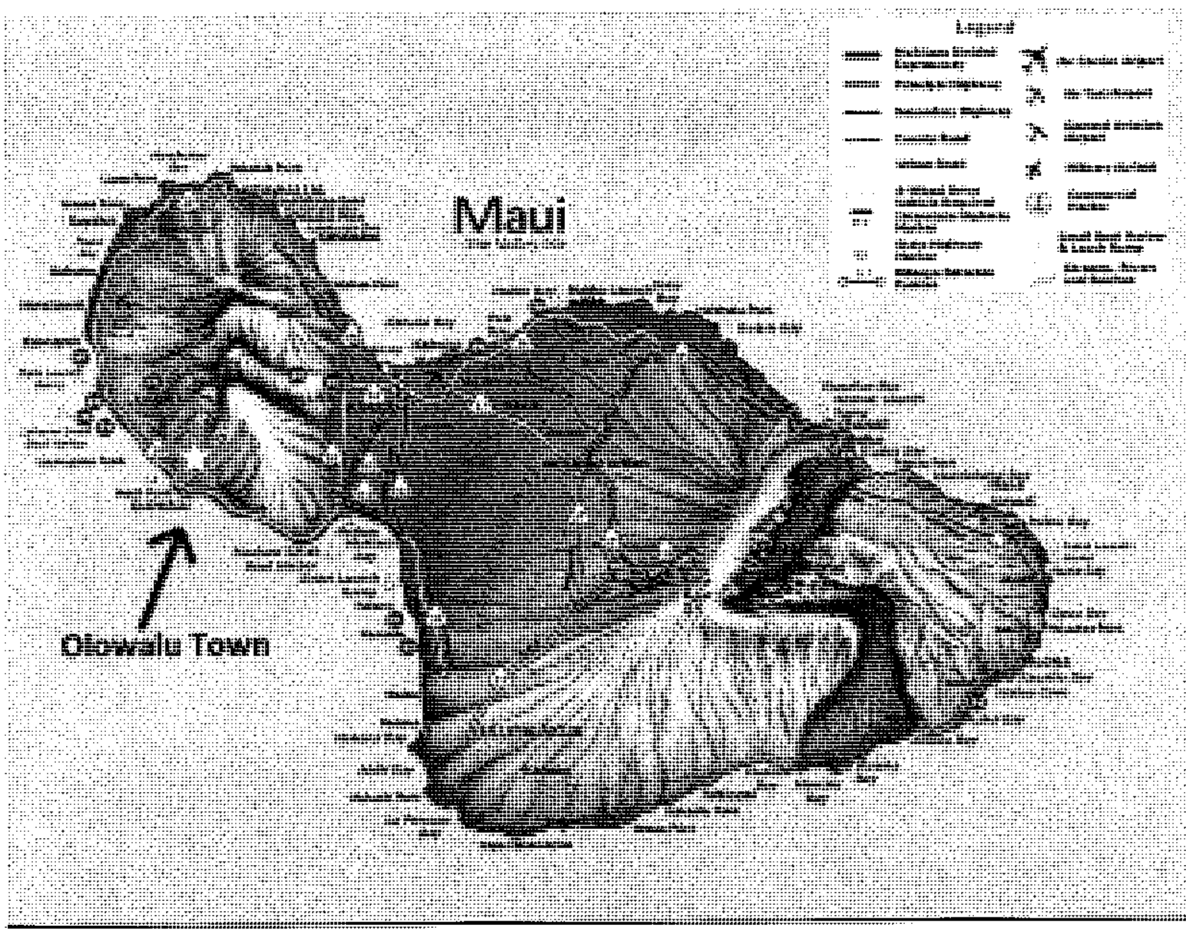
The new town will be designed to be self-sustaining in that commercial uses will in general be expected to operate to a very large degree based on the anticipated activities of the residents. At the same time, it is expected that the majority of the base of labor needed in the town can come from residents of the town.

This Preliminary Traffic Impact Analysis Report (TIAR) has been prepared to support the Draft Environmental Impact Statement (EIS). A Final TIAR will be prepared to support the Final EIS. The purpose of this preliminary TIAR is to provide a general assessment of the expected traffic impacts of the proposed project and a general framework for the anticipated traffic and transportation system mitigations that may be needed at full buildout of the project. This preliminary TIAR will review the expected traffic impacts based on existing daily traffic volumes and future traffic volumes which are predicted to occur due to the Olowalu Town project. This preliminary TIAR will concentrate on predicted daily traffic volumes and on more general traffic needs. It will utilize the same general process by providing an assessment of existing conditions, prediction of trip generation, distribution and assignment for the new town, and analysis of future-year traffic volumes and traffic flow conditions. This report utilizes data from several other TIARs which have been done for other projects on the west side of Maui over the last five years. It also uses information from previous master transportation studies of the island conducted by HDOT in 1997 and 2002, as well as studies done by Maui County.

As noted above, a Final TIAR will be prepared to address and incorporate comments received during review of the Draft EIS. The Final TIAR will address peak hour traffic flows and utilize the methods that are normally employed in standard traffic assessments. That TIAR will also analyze in detail the predicted traffic operations at the access points to Honoapi'ilani Highway. It will assess the need for any mitigation and analyze the need for traffic control measures and devices that may be required for proper functioning of the street system. This preliminary report will not cover all items that may be studied and analyzed in the future detailed TIAR and it is not intended to substitute for that more comprehensive analysis.

The methods of study employed in this report rely upon daily traffic volumes that exist or will be created by the activities of the proposed Olowalu Town. Analysis of capacity and traffic flow utilizes approaches that were developed by the Florida Department of Transportation, based on the Highway Capacity Manual. These methods take into account the peaking that occurs in morning and afternoon hours but they also consider the overall traffic flow on a facility over a 24-hour period. The level of analysis in this preliminary TIAR does not include detailed analysis of all traffic movements at individual intersections. This report is intended to illustrate that the increase in vehicular traffic along the Honoapi'ilani Highway attributed to Olowalu Town will be successfully mitigated by way of implementing the proposed transportation plan and the related improvements, including the relocation and widening of the segment of Hononpi'ilani Highway which traverses the subject property. This transportation plan is discussed in the following sections.

Figure 1 Location of Olowalu Town



As indicated in Figure 1, the proposed Olowalu Town is located about half-way between the town of Lahaina and Ma'alea along Honoapi'ilani Highway.

Description of Olowalu Town

At final build-out, Olowalu Town will consists of approximately 1,500 residential dwelling units to be built concurrent with appropriate infrastructure in phases spread out over a period of approximately 10 years. There will be a wide variety of single-family and multi-family dwelling types, including houses, apartments, live-work units, cottages, rural homes and farmsteads, to be offered at a wide-range of income levels, including both rental and fee-ownership. A substantial portion of the homes are planned for much-needed affordable housing and senior living.

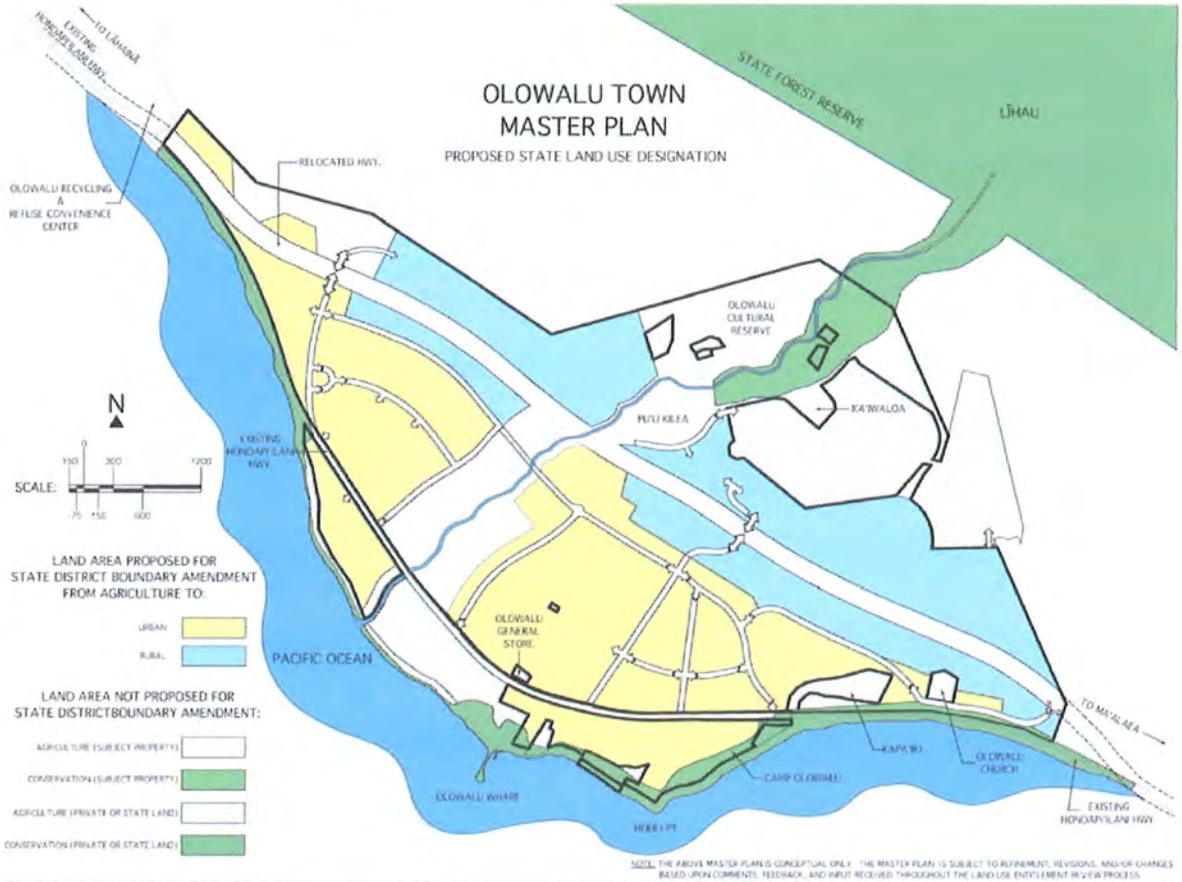
The design of Olowalu Town incorporates smart growth and sustainable land use principles of New Urbanism. As a result, Olowalu Town's spatial layout of land uses, varying density, connective transportation, parks/greenways, civic/social facilities, housing, employment and other land uses are balanced to create a mixed-use community. Neighborhood town centers provide economic sustainability with a range of business and employment opportunities. Olowalu Town is also designed to meet the certification requirements of *Leadership in Energy and Environmental Design for Neighborhood Development* (LEED ND). As such, the Master Plan will be built using strategies aimed at improving performance in regards to energy savings, water efficiency, reducing CO_2 emissions, improved indoor environmental quality, and stewardship of resources and sensitivity to their impacts.

Olowalu Town's proposed infrastructure improvements will be constructed concurrently with the project and will incorporate innovative, efficient, and sustainable technology to minimize adverse impacts upon the natural environment. Olowalu Town's Transportation system includes the relocation of the existing high speed/high volume Honoapi'ilani highway away from coastal resources to a new mauka alignment, which will be designed to accommodate mass transit or light rail, if needed in future. The existing highway corridor with monkey-pod trees will be preserved and converted to low speed/low volume coastal roadway. The project includes an internal roadway network, as well as, an assortment of interconnected greenways and bikeways links community and supports overall well-being and health of residents; reducing dependency on automobiles.

Additionally, other infrastructure system improvements will require an expansion of both the existing potable and non-potable water system, the likely addition of a second ground water well to supplement the existing well; and an extensive drainage system to capture storm-water runoff. The project will also include the construction of an onsite decentralized wastewater treatment facility, which will include R-1 water storage tank, a constructed vertical flow wetland, and a soil aquifer treatment system. The wastewater treatment facility will produce clean recycled water for irrigation, and thereby eliminate the need for injection wells.

The Olowalu Town consists of four general land use categories as defined by the state of Hawaii Land Use Commission, as shown in Figure 2.

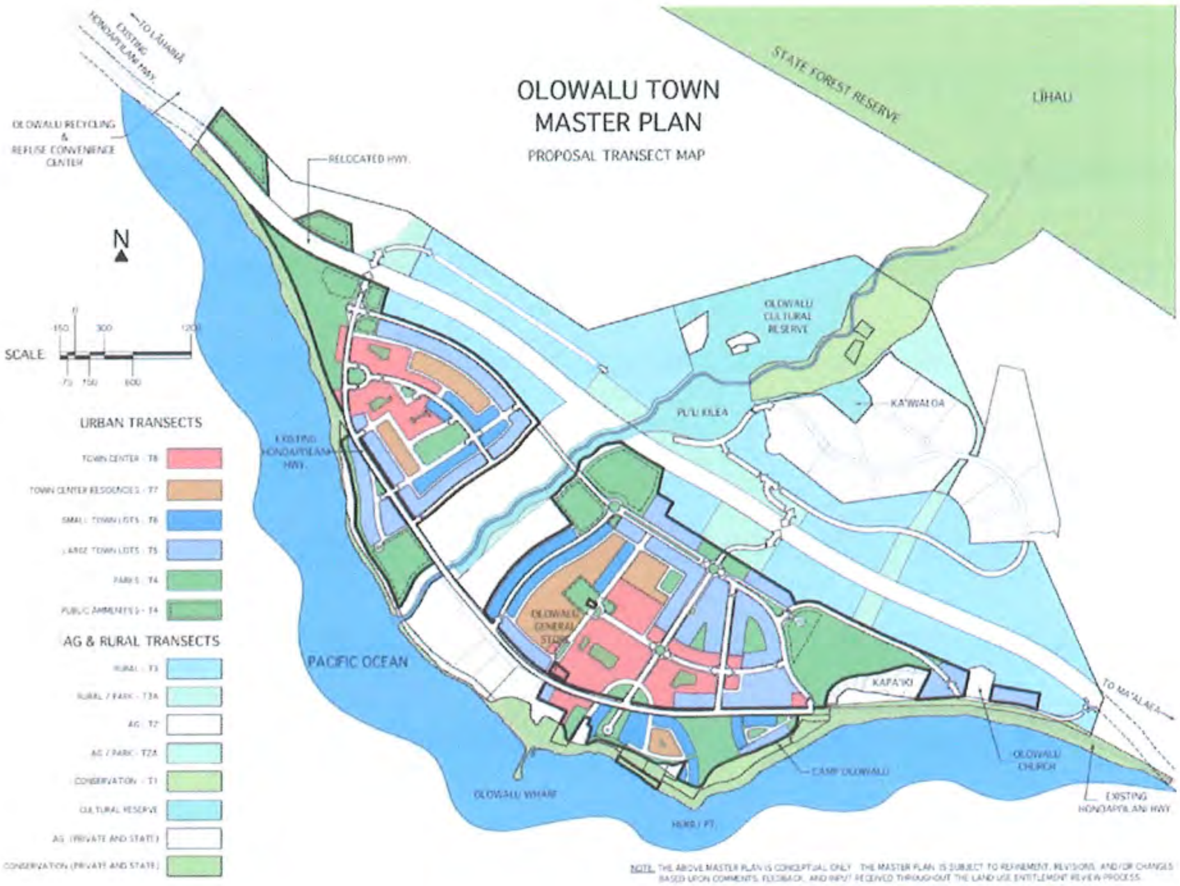
Figure 2 Land Use Categories for Olowalu Town



As shown in the above figure, there are conservation districts, shown in green, along the shore of the Pacific Ocean and in the areas upland to the mauka side of the proposed relocated Route 30. The green areas with the dots delineate the conservation areas which are not part of the subject properties. The site for the Olowalu Town includes both rural designations as shown in the blue color and urban areas as shown in the lighter beige color. Agricultural designations which are not proposed for reclassification are shown in white. Agricultural land areas which are not part of the subject properties are shown in the same colors but with dots.

Figure 3 shows the proposed land uses within the Olowalu Town.

Figure 3 Proposed Land Uses in Olowalu Town

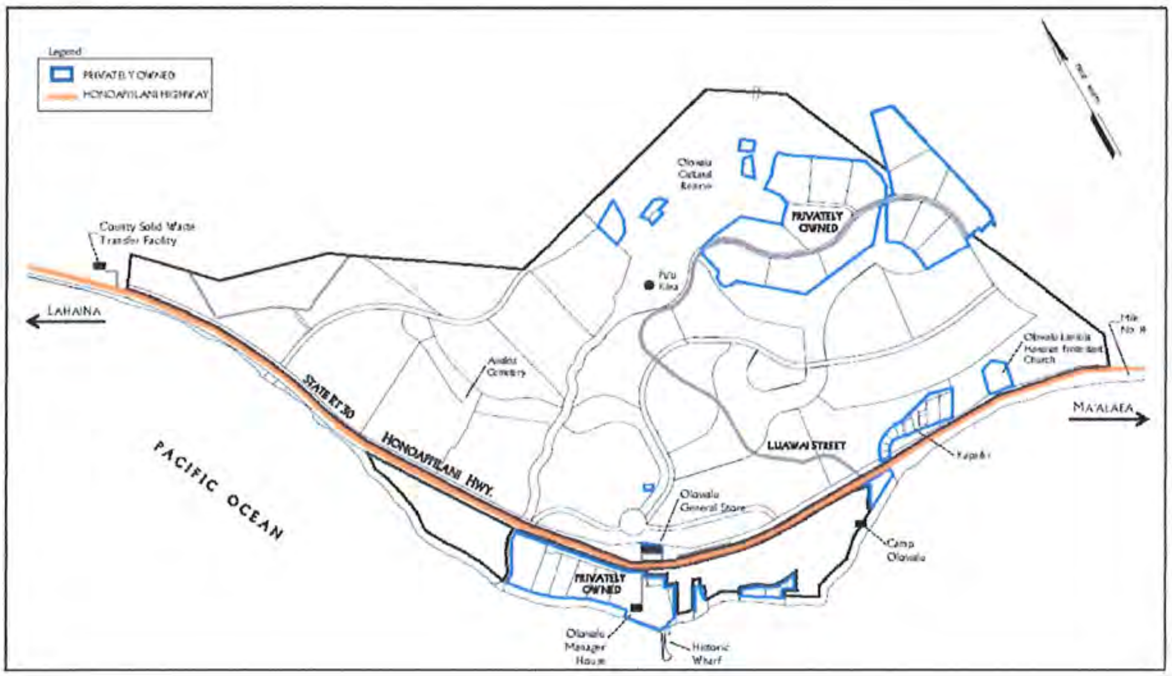


As shown in Figure 3 there are generally two neighborhood town centers on either side of Olowalu Stream, with smaller tracts of rural on the mauka side of Honoapi'ilani Highway. All the tracts on the mauka side of Honoapi'ilani Highway are either rural or agricultural lots ranging from ½ to 2 acres. The urban areas, shown in yellow, will have commercial developments along with a variety of residential units including apartments, townhouses and single-family house. Areas in white and green along the Pacific Ocean will contain recreational facilities and include beach access. The street patterns in the urban footprint areas will be modified grids following the principles of new urbanism.

Review of Existing Road and Traffic Conditions

The site of the proposed Olowalu Town is situated on both sides of Honoapi'ilani Highway on the west side of the island of Maui. Honoapi'ilani Highway is the major surface transportation route for the west side of Maui and it provides a connection from the town of Lahaina to Ma'alea. The highway is on the HDOT road system as Route 30. It is classified as a major arterial roadway and it has several different design cross-sections as it winds its way along the Pacific Ocean coastline and inland to Lahaina and Ma'alea. Within the boundaries of Olowalu Town, the roadway is primarily a two-lane highway with turn lanes in place at intersections and access points. The site for the proposed project is traversed by approximately 2.6 miles of the existing Honoapi'ilani Highway. Photographs of the roadway are included in the appendices. The following Figure 4 depicts existing conditions with area landmarks shown.

Figure 4 Existing Conditions with Area Landmarks



Trip Generation Methodology

Trip generation for Olowalu Town was estimated using the methods of the Institute of Transportation Engineers (ITE) that are generally endorsed by HDOT and Maui County. The ITE methodology uses tables that relate units of land use to predicted trips into and out a site or development for peak hours of travel and for an entire day. Trips were predicted using the latest version of the ITE Trip Generation Edition 8, as well as a software package developed by the Florida Department of Transportation (FDOT). This software provides a spreadsheet for calculation of predicted trips based on ITE average national rates, with provisions for pass-by trips and internal capture trips.

Figure 5 shows the predicted total trips for the Olowalu Town using the software from FDOT. As indicated in Figure 5, it is estimated that the Olowalu Town would generate approximately 32,800 trips per day at full buildout. This would include all trips within the town and to and from the Town via Honoapi'ilani Highway. Further analysis of internal and external trips will be made in a subsequent section. The detailed printout from the FDOT software for trip generation is provided in the appendices.

Figure 5 Summary of Trip Generation for Olowalu Town

Site Information		Adjacent Highway PassBy Information	
Name of Development	Olowalu	North/ South Roadway	Honoapi'ilani Highway
Name of Applicant	F&W	North/ South 24-Hour Volume	22,840
Name of Analyst	Dyar	East/ West Roadway	NA
Date	02/08/2011	East/ West 24-Hour Volume	NA
Development Phase	Full Buildout	Analysis Year	2020

ITE Code	Land Use	# Units	Unit	Unit Conversion	Trip Rate Per Unit	Total Trips	Directional Distribution		Trip Generation	
							In	Out	In	Out
730	Government Office Building	15,000	sq. feet	1,000	11.933	179	0.5	0.5	90	90
110	General Light Industrial	26,000	sq. feet	1,000	6.962	181	0.5	0.5	90	90
590	Library	5,000	sq. feet	1,000	54.0	270	0.5	0.5	135	135
417	Regional Park	77	acres	1	4.571	352	0.5	0.5	176	176
310	Boutique Hotels	58	rooms	1	8.172	474	0.5	0.5	237	237
732	United States Post Office	5,000	sq. feet	1,000	108.2	541	0.5	0.5	270	270
710	General Office Building	60,000	sq. feet	1,000	11.017	661	0.5	0.5	330	330
230	Residential Condo Or Townhouse	174	dwelling units	1	5.908	1028	0.5	0.5	514	514
220	Apartments	260	dwelling units	1	6.719	1747	0.5	0.5	874	874
944	Gasoline/ Service Station	20	fueling positions	1	168.5S	3371	0.5	0.5	1686	1686
220	Apartments	593	dwelling units	1	6.72	3985	0.5	0.5	1992	1992
210	Single-Family Detached Housing	523	dwelling units	1	9.109	4764	0.5	0.5	2382	2382
820	Commercial Retail	114,000	sq. feet	1,000	64.868	7395	0.5	0.5	3698	3698
820	Commercial Retail	125,000	sq. feet	1,000	62.808	7851	0.5	0.5	3926	3926
Totals						32,800	0.5	0.5	16,400	16,400

Trip Distribution

Trips were distributed based on the existing pattern of traffic on Honoapi'ilani Highway and trip data available from the most recent update of the Maui Long Range Transportation Plan (MLRTP). The MLRTP update was prepared for HDOT by a consulting firm utilizing the TransCad model. This model predicts trip generation using equations for production and attraction of trips for the traffic analysis zones (TAZs) established for the island. The gravity model distributes trips to and from the TAZs using a complicated algorithm. The study established some 400 TAZs for the island, as indicated in materials in the appendices. The model then assigns trips to the existing (and proposed) network of streets using equations that take into account the capacity of the street network and anticipated speeds on the network. One of the results of this effort is the development of a trip table that indicates the number of trips to and from each TAZ on the island. The trip distribution process will require additional work when the detailed TIAR is prepared, since trip distribution for peak hours may differ from the values for the overall daily traffic generated. This may occur especially at early morning hours when there are a greater percentage of journey-to-work trips being made and fewer trips made for shopping or recreational uses.

Available traffic counts on Honoapi'ilani Highway indicate the direction of traffic flow by hour of the day nearby the project site. This information was used as background information in determining the distribution of trips; however it is of course more useful when looking at hourly trip generation and traffic distribution.

Trip Assignment

Trips were assigned to the three proposed access points to the Olowalu Town, based on the general preliminary site development plan. Trips from the residential and commercial components of Olowalu Town were assigned mostly to the two primary access points along the relocated Honoapi'ilani Highway. These two access points are expected to be designed as non-signalized intersections with the use of median U-turns in an arrangement generally known as the "Michigan U-turn." The design has been called the "O-turn" to refer to Olowalu Town. The "Michigan U-turn" frequently utilizes traffic signals, however it is expected that the "O-turns" will operate with U-turns required for all left turns out of the site (after a right turn), for left turns into the site and for straight-through movements from one side of Honoapi'ilani Highway to the other. It is envisioned that no traffic signals will be required for through traffic along the new Honoapi'ilani Highway and that traffic control for vehicles entering or leaving Olowalu Town will be via stop signs or yield signs. The third access point planned is a right-in/right-out access point on the southern end of the site, which will have limited use compared to the other two major access points. Trip assignments were made to each of the three proposed access points based on the number of residential units and the square footage of commercial and other space planned for Olowalu Town, including the proposed recreational areas. The amount of trips generated by the mauka side of Honoapi'ilani Highway will be very small in comparison to the total trips generated by the entire site and will have a relatively small impact on traffic flow. In addition, there is a connector road planned to link the mauka and makai sides that will allow for traffic flow without the need to utilize either of the "O-turns." This connector should minimize the trips that would be made through the "O-turns."

Background Traffic Growth

Several studies were made available which analyzed traffic growth trends on Honoapi'ilani Highway and in the west Maui area. Data from these studies are included in the appendices.

Based on a review of available data, it was decided that an annual traffic volume growth rate of approximately 1% would be appropriate for Honoapi'ilani Highway, resulting in a total growth of 8% between 2011 and full buildout of the Olowalu Town in 2020. This 8% growth rate was applied to the existing traffic volumes on Honoapi'ilani Highway to derive future year traffic volumes without the Olowalu Town project in place. Detailed analysis of peak hour traffic growth rates will result in the application of hourly factors when the final TIAR is completed.

The current average annual daily traffic on Honoapi'ilani Highway was estimated with a 24-hour machine traffic count made in October 2010. Other studies have indicated there are approximately 5% trucks in the traffic stream on Honoapi'ilani Highway. Using the assumption of 5% trucks, the count made in October 2010 indicates a daily traffic volume of 22,840. This count was made slightly north of the solid waste transfer station which is just beyond the northern boundary of the proposed Olowalu Town site. Assuming the growth rate of 8%, the background traffic volume growth on Honoapi'ilani Highway would be 1798 vehicles resulting in an average daily traffic volume of 24,667 (or approximately 24,700) in the future year of 2020 without the project in place.

Traffic Analysis in Year 2020 without Olowalu Town Project

An analysis was made of the traffic flow on Honoapi'ilani Highway in the year 2020 without the project in place. This analysis assumed the background traffic volume growth would continue, resulting in a total increase of 8% in the daily traffic volumes on Honoapi'ilani Highway. The analysis assumed that that all peak hour and directional factors and truck factors remained the same. The results of the use of the Highplan software are given in Figure 6. As noted in Figure 6, the volume to capacity ratio would increase to 0.73 and the level of service when considering speed would be at an E. The volume to capacity ratio of 0.73 indicates there would be the ability to add more traffic to the roadway on a daily basis, although peak hour traffic speeds would continue to decrease.

Additional information on the Highplan software and its outputs is available in the appendices. See Figure 7 for existing 24-hour traffic volumes on Honoapi'ilani Highway and Figure 8 for the predicted traffic volumes in the year 2020 without the project in place.

Figure 6 Output from Highplan Software for Honoapi'ilani Highway for Year 2020 without Project in Place

HIGHPLAN 2009 Conceptual Planning Analysis									
Project Information									
Analyst	RDD	Highway Name	Honoapi'ilani Highway	Study Period	K100				
Date Prep.	4/22/2011	From	Old Landfill	Program	HIGHPLAN 2009				
Agency	For F&W	To	Mile 14	Version	7/17/2010				
Area Type	Rural Undeveloped	Peak Direction	Northbound						
File Name	C:\Users\rogerdyar\AppData\Local\Temp\preview.xml								
User Notes	Future Year 2020 without Project in Place								
Highway Data									
Roadway Variables				Traffic Variables					
Area Type	Rural Undeveloped	Segment Length	2.6 miles	AADT	24,470	PHF	0.85		
# Thru Lanes	2	Median	Yes	K factor	0.078	% Heavy Vehicles	5		
Terrain	Level	Left Turn Impact	No	D factor	0.58	Base Capacity	1700		
Posted Speed	55	Pass Lane Spacing	N/A	Peak Dir. Hourly Vol.	1107	Local Adj. Factor	1		
Free Flow Speed	50	% No Passing Zones	100	Off Peak Dir. Hourly Vol.	802	Adjusted Capacity	16S9		
LOS Results									
v/c Ratio	0.73	Density	NA	PTSF	91.83	ATS	42.0	% FFS	70.04
FFS Delay	66.73	LOS Thresh. Delay	35.53	Service Measure	PTSF	LOS	E		
Service Volumes									
Note: The maximum normally acceptable directional service volume for LOS E in Florida for this facility type and area type is 1500 veh/h/ln.									
		LOS	A	B	C	D	E		
		Lanes	Hourly Volume In Peak Direction						
		2	490	140	280	590	1500		
		Lanes	Hourly Volume In Both Directions						
		4	850	250	490	1020	2590		
		Lanes	Annual Average Daily Traffic						
		4	9,500	3300	6300	13100	33300		

Traffic Generation for Olowalu Town

Traffic generation was predicted for Olowalu Town using the methods of the ITE that were discussed earlier. To account for the principles of New Urbanism, an adjustment was made to trip generation rates. Based on research of other New Urbanism style developments in a number of states, it is apparent that there is a reduction in total travel in communities with this type of design and layout. These communities are designed to encourage walking and cycling and they are often attractive to people whose lifestyles include less traditional travel for various needs, including for employment, recreation and shopping. In these communities there are frequently recreational opportunities that reduce the need for off-site travel; and there are many cases of live-work uses of residences. A review of this phenomenon in several other locations led to an estimate that these communities have a typical reduction in total trip generation ranging between 15% and 45%. There are local governments such as Frederick, Maryland, which typically allow for a reduction in trip generation rates, in recognition that this phenomenon does occur, as noted in the appendices. Using this methodology, the land use mix for Olowalu Town indicates that the trip generation should be reduced to about 85% of the ITE values. Therefore the ITE trip generation rates for all the residential land uses were decreased by applying a factor of 85% to the published ITE rates. It should be remembered the ITE publishes ranges but also publishes average rates. So, the rates for trip generation used in this report are at essentially at 85% of the national averages.

It was also assumed that many of the trips generated would be made to and from the residences and commercial and recreational establishments in the town by residents of Olowalu Town. This phenomenon is generally referred to as "internal capture." Based on the anticipated plan for the proposed project, it was determined that significant proportions of total travel could and would be made within the town itself, without any requirement to travel on Honoapi'ilani Highway to Lahaina, Ma'alea or elsewhere on the island. The plan for the town is for a substantial amount of the business activity to be supported by town residents and for many if not the majority of the potential employment opportunities to be taken by town residents. Facilities such as the proposed library, the post office, many of the retail establishments and many of the recreational opportunities would be used primarily by town residents. As a result, many of the trips generated will have both origin and destination in the town. Due to the design of the town and its street network, many of the trips within the town will likely be made via walking or cycling and not require use of the automobile. This element will be addressed in detail in the final TIAR.

For this preliminary TIAR, it was assumed that the "internal capture" of trips generated would align with and be affected by the various land uses. Some land uses would have higher proportions of trips made within the town with others having more trips made outside the town via Honoapi'ilani Highway to west Maui or other locations. The following Table 1 shows the anticipated "internal capture" of trips generated by the proposed land uses in Olowalu Town.

Table 1 Internal Capture of Trips in Olowalu Town

ITE Land Use Code	Land Use Description	Percentage of Trips Internal to Olowalu Town	Percentage of Trips External to Olowalu Town
730	Government Office Building	85%	15%
110	General Light Industrial	30%	70%
590	Library	90%	10%
520	Elementary School	90%	10%
415/417	Regional Park with Beach	50%	50%
310	Hotel	10%	90%
732	United States Post Office	95%	5%
230	Condominium/Townhouse	45%	55%
944	Gasoline/Service Station	95%	5%
220	Apartments	45%	55%
210	Single-Family Detached Housing	45%	55%
820	Commercial Retail	75%	25%
710	General Office	30%	70%

As seen in Table 1, some land uses, such as the library and the gasoline station would have almost all their trips made by residents of the Olowalu Town. Others, such as the general light industrial and general office land uses, would depend more heavily on residents and commercial ventures in other parts of the island and would have much lower "internal capture" percentages.

In addition to the "internal capture", there was also a major consideration of the effect of existing traffic on Honoapi'ilani Highway for the trips that are external to the Olowalu Town. Due to the isolated nature of the site, with long travel distance to other populated areas on the island, it is less likely that island residents on other parts of the island would make special trips to Olowalu Town for employment, shopping opportunities, or other services that could be found closer to their respective home region. However, it is also recognized that a significant number of trips that will be made to and from Olowalu Town that are not generated within Olowalu Town will be from travelers already on Honoapi'ilani Highway. Generally, this phenomenon is referred to as "pass-by trips or diverted trips." In other words, many trips made to Olowalu Town will be made by drivers who are already on Honoapi'ilani Highway and will divert into Olowalu Town for their business, employment, recreation of for other purposes. An analysis of trip length frequency curves from the Maui LRTP was made to assist in estimating the amount of "pass-by" trips that would have destinations or origins in Olowalu Town that would already be on Honoapi'ilani Highway.

The following Table 2 shows the estimated proportions of trips that would essentially be pass-by trips on Honoapi'ilani Highway to and from Olowalu Town.

Table 2 Pass-by and Diverted Trips on Honoapi'ilani Highway

ITE Land Use Code	Land Use Description	Percentage of Trips to/from Olowalu Town Already On Honoapi'ilani Hwy	Percentage of Trips to/from Olowalu Town Not Already On Honoapi'ilani Hwy
730	Government Office Building	50%	50%
110	General Light Industrial	20%	80%
590	Library	20%	80%
520	Elementary School	80%	20%
415/417	Regional Park with Beach	20%	80%
310	Boutique Hotel	20%	80%
732	United States Post Office	80%	20%
230	Condominium/Townhouse	20%	80%
944	Gasoline/Service Station	90%	10%
220	Apartments	20%	80%
210	Single-Family Detached Housing	20%	80%
820	Commercial Retail	80%	20%
710	General Office	20%	80%

As indicated in Table 2, some land uses have high rates of pass-by trips on Honoapi'ilani Highway. For example, it is estimated that 90% of all trips to the gasoline station in Olowalu Town that are not from within Olowalu Town would be trips that are already on Honoapi'ilani Highway. In other words, only 10% of the trips that come from external to Olowalu Town would be by traffic that is not already on Honoapi'ilani Highway. A similar situation is shown for the post office. It is not likely that very many trips to the post office that are not from Olowalu Town would be made by the casual traveler on Honoapi'ilani Highway. In other words, it is unlikely that many special trips will be made via Honoapi'ilani Highway to get to Olowalu Town to use the post office. This is of course due to there being sufficient other post office locations already on the island of Maui. At Olowalu Town, some land uses, like the hotels, would generate more new trips (80%) that are not already on Honoapi'ilani Highway than are already on Honoapi'ilani Highway (20%). This would reflect the fact that hotel guests may well come from other portions of the island and would not be as likely to be traveling on Honoapi'ilani Highway already. The percentages used in Tables 1 and 2 were used to allocate new trips and pass-by trips to the access points on Honoapi'ilani Highway for the Olowalu Town.

Trip Distribution

Trips were distributed using information from the Maui LRTP and a review of the trip length frequency curves from the latest update of the island's LRTP. By analyzing this data and by being aware of the location of various traffic generators on the island, distribution values for each land use were developed. The trip distribution pattern for external trips shows trips that would leave Olowalu Town to travel to or from the north (Lahaina and beyond) or the central and south portions of Maui. The following Table 3 shows the distribution of trips generated for each land use in the Olowalu Town.

Table 3 Trip Distribution for New External Trips Generated in Olowalu Town

ITE Land Use Code	Land Use Description	Percentage on Honoapi'ilani Hwy To/from Lahaina	Percentage on Honoapi'ilani Hwy To/from Ma'alea
730	Government Office Building	46%	54%
110	General Light Industrial	58%	42%
590	Library	54%	46%
520	Elementary School	54%	46%
415/417	Regional Park with Beach	46%	54%
310	Hotel	72%	28%
732	United States Post Office	54%	46%
230	Condominium/Townhouse	60%	40%
944	Gasoline/Service Station	50%	50%
220	Apartments	60%	40%
210	Single-Family Detached Housing	60%	40%
820	Commercial Retail	60%	40%
710	General Office	60%	40%

A similar process was used for pass-by or diverted trips to and from Olowalu Town that would be added to traffic on Honoapi'ilani Highway. The following Table 4 shows the distribution of pass-by trips for the various land uses in Olowalu Town.

Table 4 Trip Distribution for Passby Trips Generated to and from Olowalu Town

ITE Land Use Code	Land Use Description	Percentage on Honoapi'ilani Hwy To/from Lahaina	Percentage on Honoapi'ilani Hwy To/from Ma'alea
730	Government Office Building	46%	54%
110	General Light Industrial	58%	42%
590	Library	54%	46%
520	Elementary School	54%	46%
415/417	Regional Park with Beach	46%	54%
310	Hotel	72%	28%
732	United States Post Office	54%	46%
230	Condominium/Townhouse	60%	40%
944	Gasoline/Service Station	50%	50%
220	Apartments	60%	40%
210	Single-Family Detached Housing	60%	40%
820	Commercial Retail	60%	40%
710	General Office	60%	40%

Traffic Assignment

The Wintass software program was used to assign trips to the street network with trips allocated based on the distribution data described earlier and the allocation of trips to the three proposed access points. Generally, trips were allocated by land use to the most logical access point, depending upon the direction of travel away from Olowalu Town. These trips are only for travel external to Olowalu Town. Overall, a small percentage of trips were assigned to the proposed right-in/right-out access point that will be located on the southern end of the project. The remaining trips were allocated to the two major access points that will operated as the "O-turns." The following Table 5 shows the general allocation of trips to the three access points.

Table 5 Allocation of External Trips to Proposed Olowalu Town Access Points

Access Point	Percentage of External Trips Entering Olowalu Town	Percentage of External Trips Exiting Olowalu Town	Comments
O-turn 1	35%	35%	Values vary slightly for individual land uses
O-turn 2	54%	58%	Values vary slightly for individual land uses
RIRO	11%	7%	Assumes fewer entries than exits due to location and design of access point

Detailed percentage values for each land use for each access point and direction of travel are provided in the appendices. This analysis does not include estimation of travel from the mauka side to and from the makai side of the Olowalu Town. The amount of land development on the mauka side of Honoapi'ilani Highway compared to the makai side is very low, so that a very small percentage of trips will be made from one side to the other. This item will be reviewed in detail in the final TIAR which will be prepared at a later date. This analysis also does not address the internal trips made from the mauka side to the makai side of Honoapi'ilani Highway via a connector that does not require access to Honoapi'ilani Highway. This item will also be reviewed in the final TIAR to be prepared at a later date.

Development of Future Traffic Data

Traffic volumes were predicted for the future development of the entire Olowalu Town using the Wintass software mentioned earlier. This software takes existing traffic volumes and adds background growth and trips that are assigned to the street network at each node and for each turning movement. This preliminary TIAR focuses only on the total daily traffic volumes for the proposed Olowalu Town. Trip generation for the Olowalu Town included new external trips that would use Honoapi'ilani Highway either for entry to or exit from Olowalu Town. It also included the allocation of pass-by trips that would have already been on Honoapi'ilani Highway. The following schematic Figure 6 shows the street network with the Olowalu Town in place, not including a proposed internal connector from the mauka side to the makai side of Honoapi'ilani Highway.

As discussed earlier, a background growth factor of 8% was used to account for growth in traffic on Honoapi'ilani Highway without the Olowalu Town being in place. This factor was applied to all traffic movements on Honoapi'ilani Highway that would exist without Olowalu Town. Trips were allocated as indicated in Tables 1 – 5. Detailed printouts from the Wintass software are included in the appendices.

Figure 7 provides a schematic diagram showing the existing average daily traffic volumes on Honoapi'ilani Highway for the study network. Figure 8 shows the predicted traffic volumes on Honoapi'ilani Highway in the year 2020 without the project in place. Figure 9 shows the new trips added to the street network with Olowalu Town in place at its full buildout in the year 2020, with the relocated and widened Honoapi'ilani Highway in place. Figure 10 shows the total traffic volumes on the street network at full buildout of the Olowalu Town in the year 2020 with the background growth of 15%. Again, Figure 10 includes the relocation of Honoapi'ilani Highway mauka of the existing highway along with its widening to four lanes.

Figure 7 Existing Traffic Volumes on Honoapi'ilani Highway

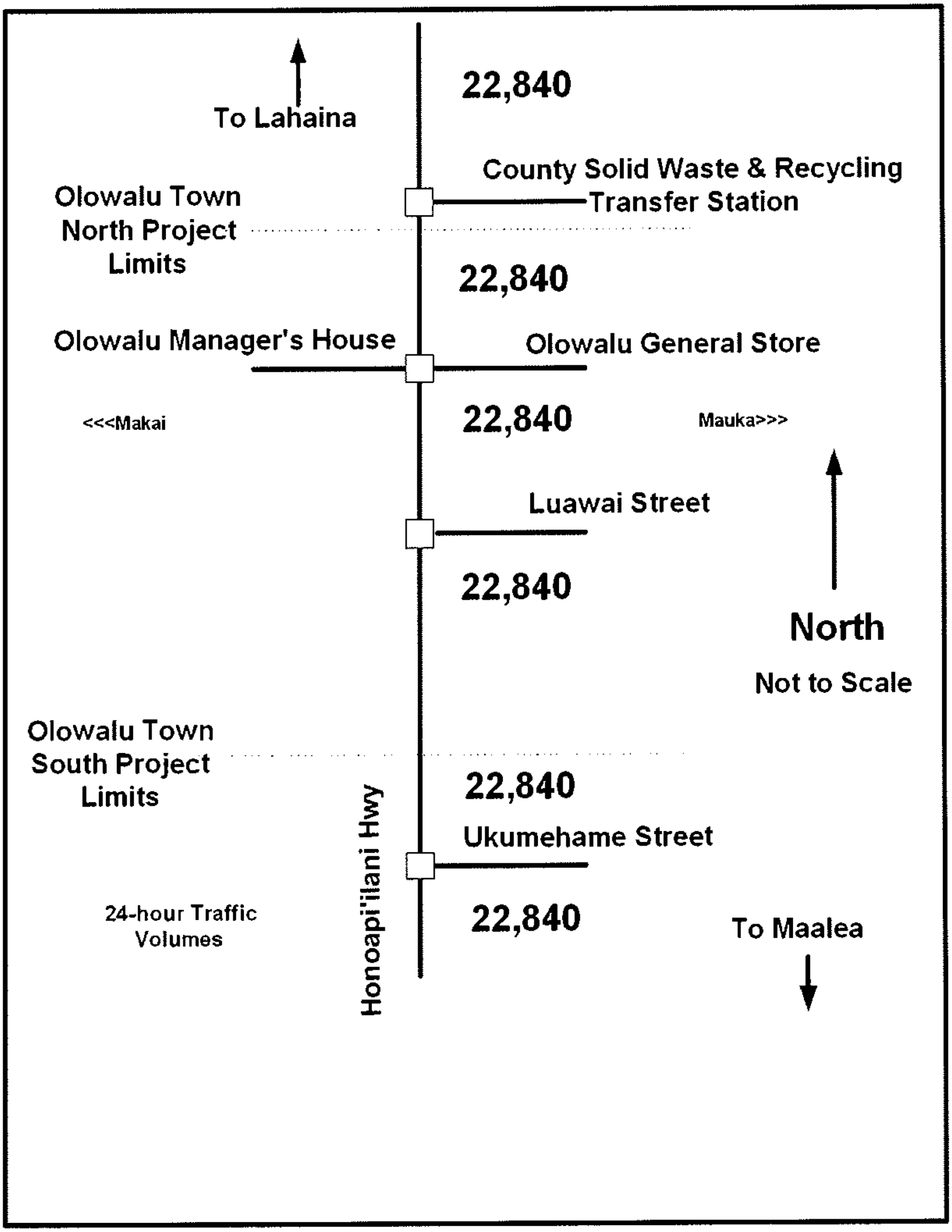


Figure 8 Future Year 2020 Traffic Volumes without Project on Honoapi'ilani Highway

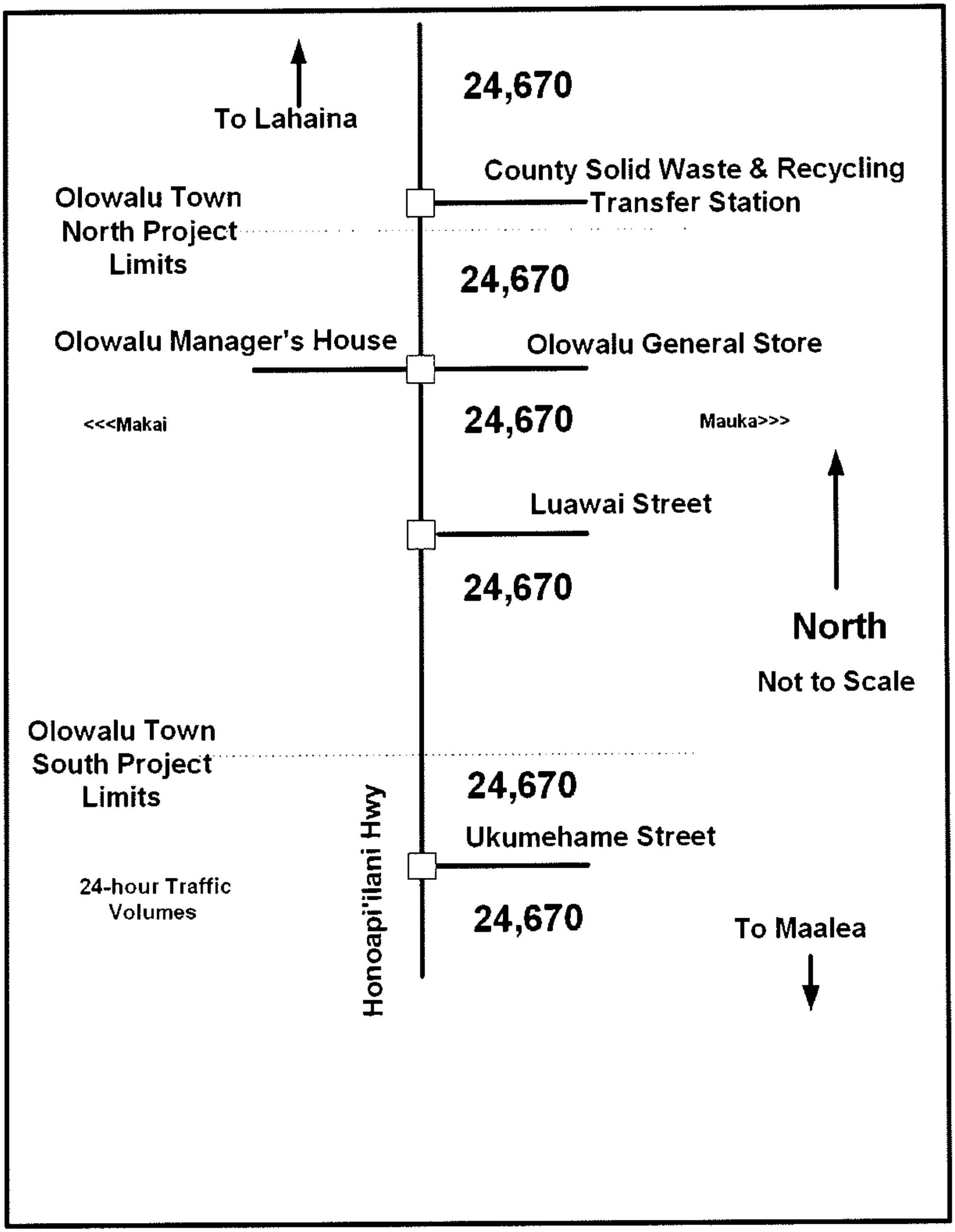


Figure 9 Traffic Added from Olowalu Town Project

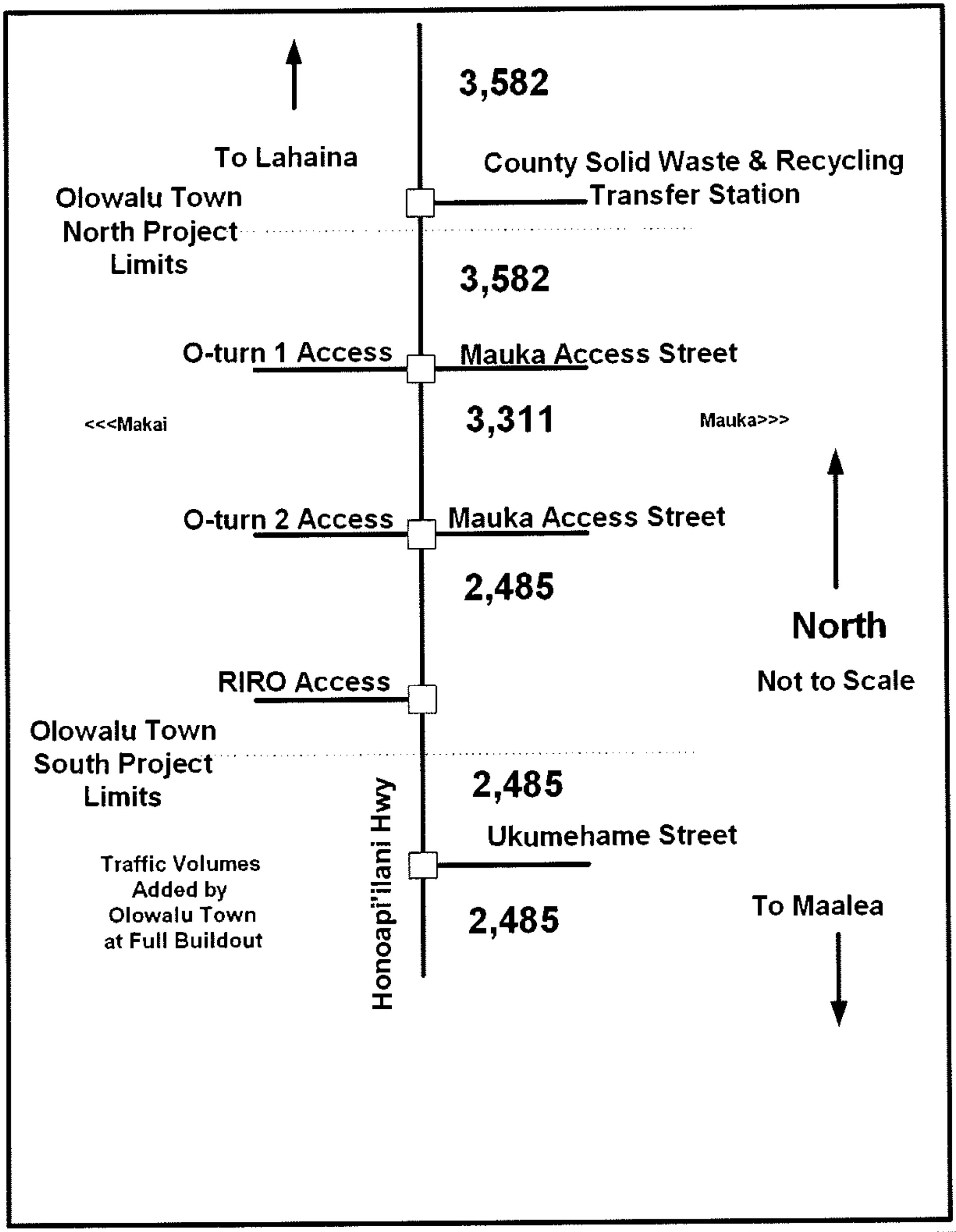
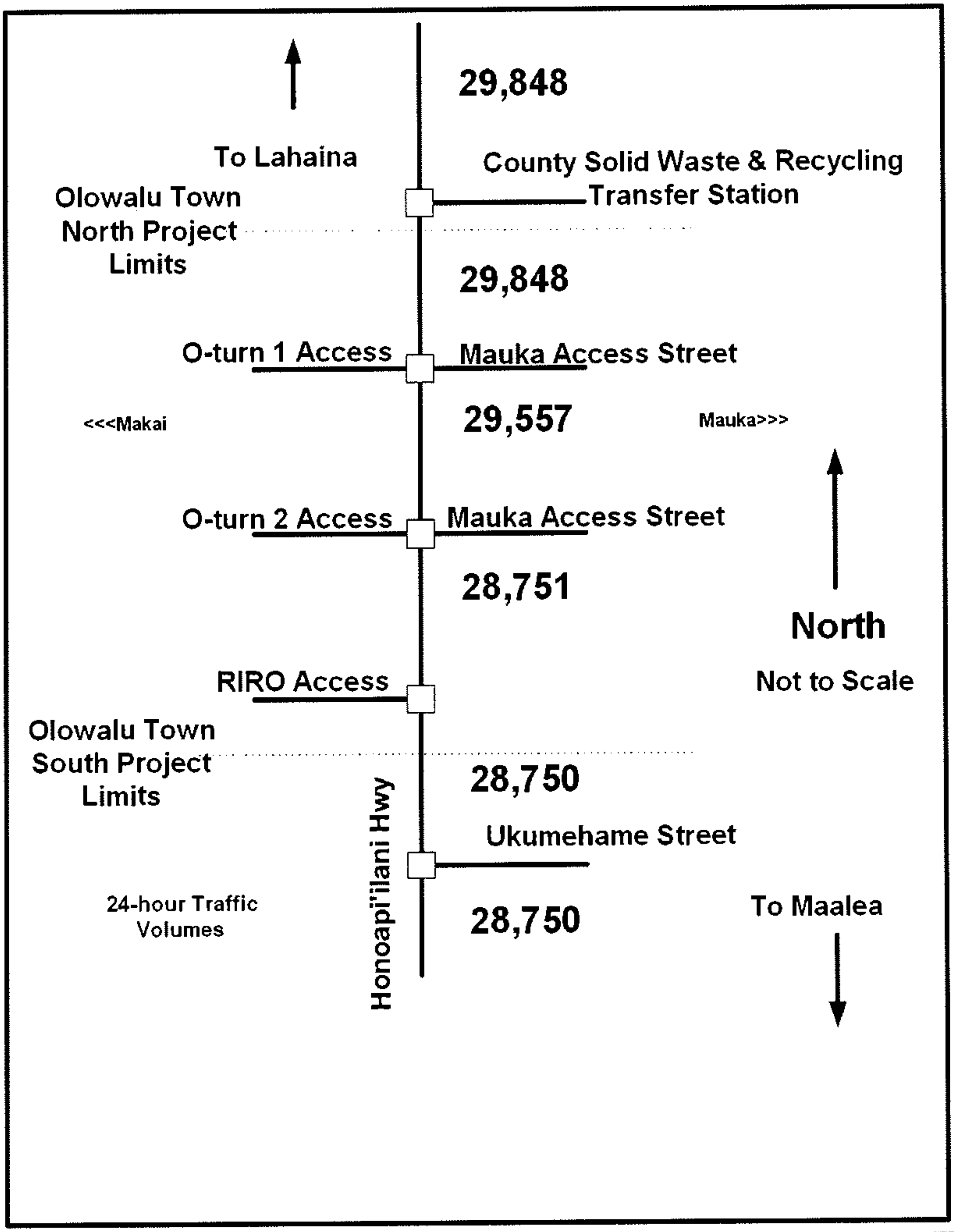


Figure 10 Olowalu Town Study Network Traffic with Full Buildout of Project in Place



Future Roadway Network

The following Figure 11 shows the conceptual design of the O-turns for the relocated Honoapi'ilani Highway (Route 30). As shown in the Figure 11, the roadway will be widened to four lanes with two through lanes plus necessary acceleration and deceleration lanes in each direction. Access to Olowalu Town will be via three new intersections. Two of the intersections will operate with the modified "Michigan U-turn," named the "O-Turn." The remaining access point will be a right-in/right-out connection with limited traffic predicted.

As shown in Figure 11, the "O-Turn" would have a deceleration lane and a stacking lane for right turns into each new access point. For right turns from the "O-Turn," a driver simply makes a right turn and uses an acceleration lane to reach highway speed. For left turns out of Olowalu Town to go to Lahaina, a driver makes a right turn and then moves to the left lane and enters a left turn deceleration and stacking area and then makes a U-turn to go north. For left turns into Olowalu Town from the Ma'alea direction, the reverse maneuver is made, with a left turn followed by a U-turn, then with travel to the south followed by a right turn into Olowalu Town. Detailed analysis will be made with the final TIAR to assess detailed traffic operations for the peak hours. Based on preliminary reviews it appears the future traffic volumes predicted for the various movements at the "O-Turns" on a daily basis are well within the expected capacity of the design for an acceptable level of service.

Figure 11 Anticipated Roadway Connections for O-Turns on Relocated Honoapi'ilani Highway

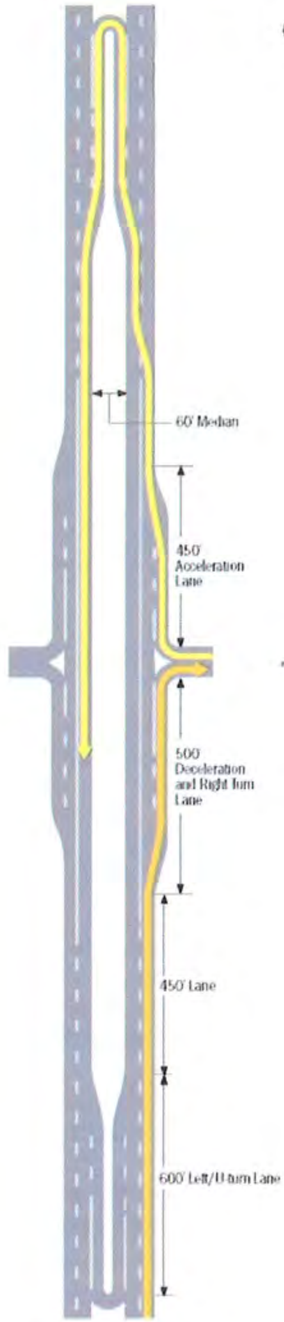
Olowalu Town PRELIMINARY STUDIES FOR POTENTIAL HIGHWAY LOCATIONS

The Olowalu O-turn keeps traffic flowing continuously without stoplights or over/underpasses. The design features long merge lanes which allow for easy turns.

How the O-Turn Works

The Olowalu Turn or O-Turn works by preventing drivers from making left turns across traffic. Drivers safely take a U-turn with the help of merge lanes and enter into the flow of traffic going in the reverse direction. Then, by merging to the right lane, drivers may turn right and reach their destination. Meetings with the Department of Transportation have been productive and they have been receptive to these innovative ideas.

O-TURN EXPLAINED



Analysis of Impacts of Olowalu Town Project

As seen in Figure 10, the predicted average daily traffic volumes on Honoapi'ilani Highway with the Olowalu Town in place at its full buildout will vary from 28,750 to about 29,850. With the proposed Olowalu Town project, there will be a relocation of Honoapi'ilani Highway and widening to provide for two through lanes in each direction for the extent of the project. South of the project, the roadway would return to its current status with one through lane in each direction plus turn lanes at intersections. A review was made of the general overall impacts of Olowalu Town by analyzing the predicted ADTs versus the daily capacity of the roadway. To achieve this preliminary analysis, the methods of the Florida Department of Transportation (FDOT) were used. The FDOT methodology is based on the Highway Capacity Manual (HCM) which is published by the Federal Highway Administration (FHWA) as well as extensive research in the state of Florida on capacity of highway facilities. The software modules available from FDOT include Highplan, which provides estimated daily capacities for highway facilities such as Honoapi'ilani Highway. While Honoapi'ilani Highway in its current state does have direct access, the number of access points is limited and their approach volumes are quite low. All major intersections have turn lanes in place. Generally, Honoapi'ilani Highway operates more like a controlled access highway rather than an arterial street. Therefore it appears to be appropriate to use the Highplan software to estimate capacity of Honoapi'ilani Highway. Additional information on the Highplan software is provided in the appendices.

Figure 12 shows the output from the Highplan software for the existing configuration of Honoapi'ilani Highway while Figure 13 provides the output from the Highplan software for Honoapi'ilani Highway with the relocation and widening in place. As indicated in Figure 13, the estimated daily maximum capacity for Honoapi'ilani Highway is approximately 56,600. For the portion of Honoapi'ilani Highway south of Olowalu Town with its current status of two through lanes and turn lanes, the estimated maximum daily capacity is shown to be 33,300. The following Table 6 shows the predicted daily capacities and ADTs without and with the Olowalu Town project in place at its potential full buildout in the year 2020. As seen in Table 6, the widened and relocated Honoapi'ilani Highway will have more than adequate capacity to handle the existing traffic plus the background growth of 8% plus the new traffic added from the Olowalu Town. At the junction of the widened and relocated section to the existing section, the level of service will reduce to E but the calculations indicate the total capacity will not be exceeded at full buildout of the Olowalu Town. As seen in the appendices and Figures 12 - 14 the predicted overall average speed for the portion of Honoapi'ilani Highway south of Olowalu Town is approximately 29 mph, while the predicted speed in the relocated and widened segment of the highway is approximately 50 mph. The speeds in the proposed relocated and widened segment assume a design free-flow speed of 50 mph with a posted speed limit of 45 mph. Actual speeds will be affected by the speeds of the merging and diverging areas and will be modeled in detail in the final TIAR. See the appendices for detailed program outputs.

Figure 12 Output from Highplan Software for Honoapi'ilani Highway with Existing Roadway Configuration

HIGHPLAN 2009 Conceptual Planning Analysis									
Project Information									
Analyst	RDD	Highway Name	Honoapi'ilani Highway	Study Period	K100				
Date Prep.	4/22/2011	From	Old Landfill	Program	HIGHPLAN 2009				
Agency	For F&W	To	Mile 14	Version	7/17/2010				
Area Type	Rural Undeveloped	Peak Direction	Northbound						
File Name	C:\Users\rogerdyar\AppData\Local\Temp\preview.xml								
User Notes	Preliminary TIAR								
Highway Data									
Roadway Variables				Traffic Variables					
Area Type	Rural Undeveloped	Segment Length	5 miles	AADT	22,840	PHF	0.85		
# Thru Lanes	2	Median	Yes	K factor	0.088	% Heavy Vehicles	5		
Terrain	Level	Left Turn Impact	No	D factor	0.58	Base Capacity	1700		
Posted Speed	45	Pass Lane Spacing	N/A	Peak Dir. Hourly Vol.	1258	Local Adj. Factor	1		
Free Flow Speed	50	% No Passing Zones	100	Off Peak Dir. Hourly Vol.	911	Adjusted Capacity	1659		
LOS Results									
v/c Ratio	0.77	Density	N/A	PTSF	90.5	ATS	43.1	% FFS	71.8
FFS Delay	55.4	LOS Thresh. Delay	27.2	Service Measure	PTSF	LOS	E		
Service Volumes									
Note: The maximum normally acceptable directional service volume for LOS E in Florida for this facility type and area type is 1500 veh/h/ln.									
		LOS	A	B	C	D	E		
		Lanes	Hourly Volume In Peak Direction						
		1	*	140	280	590	1500		
		Lanes	Hourly Volume In Both Directions						
		2	*	250	490	1020	2590		
		Lanes	Annual Average Daily Traffic						
		2	*	3300	6300	13100	33300		

Figure 13 Output from Highplan Software with Relocated and Widened Honoapi'ilani Highway in Place at Full Buildout of Olowalu Town

HIGHPLAN 2009 Conceptual Planning Analysis									
Project Information									
Analyst	RDD	Highway Name	Honoapi'ilani Highway	Study Period	K100				
Date Prep.	4/22/2011	From	Old Landfill	Program	HIGHPLAN 2009				
Agency	For F&W	To	Mile 14	Version	7/17/2010				
Area Type	Rural Undeveloped	Peak Direction	Northbound						
File Name	C:\Users\rogerdyar\AppData\Local\Temp\preview.xml								
User Notes	Preliminary TIAR								
Highway Data									
Roadway Variables				Traffic Variables					
Area Type	Rural Undeveloped	Segment Length	5 miles	AADT	29,850	PHF	0.85		
# Thru Lanes	4	Median	Yes	K factor	0.090	% Heavy Vehicles	5		
Terrain	Level	Left Turn Impact	No	D factor	0.58	Base Capacity	2000		
Posted Speed	45	Pass Lane Spacing	N/A	Peak Dir. Hourly Vol.	1558	Local Adj. Factor	1		
Free Flow Speed	50	% No Passing Zones	N/A	Off Peak Dir. Hourly Vol.	1128	Adjusted Capacity	1659		
LOS Results									
v/c Ratio	0.47	Density	18.8	PTSF	N/A	ATS	50.0	% FFS	100
FFS Delay	0	LOS Thresh. Delay	60.0	Service Measure	Density	LOS	C		
Service Volumes									
Note: The maximum normally acceptable directional service volume for LOS E in Florida for this facility type and area type is 1500 veh/h/ln.									
		LOS	A	B	C	D	E		
		Lanes	Hourly Volume In Peak Direction						
		2	490	1,160	1,820	2,390	2,950		
		Lanes	Hourly Volume In Both Directions						
		4	850	2,010	3,140	4,130	5,090		
		Lanes	Annual Average Daily Traffic						
		4	9,500	22,400	34,900	45,900	56,600		

Figure 14 Output from Highplan Software for Portion of Honoapi'ilani Highway South of the Project Site at Full Buildout of Olowalu Town

HIGHPLAN 2009 Conceptual Planning Analysis									
Project Information									
Analyst	RDD	Highway Name	Honoapi'ilani Highway	Study Period	K100				
Date Prep.	4/22/2011	From	Mile 14	Program	HIGHPLAN 2009				
Agency	For F&W	To	Mile 13	Version	7/17/2010				
Area Type	Rural Undeveloped	Peak Direction	Northbound						
File Name	C:\Users\rogerdyar\AppData\Local\Temp\preview.xml								
User Notes	Two Lane Portion South of Site								
Highway Data									
Roadway Variables				Traffic Variables					
Area Type	Rural Undeveloped	Segment Length	1 miles	AADT	28,750	PHF	0.85		
# Thru Lanes	4	Median	Yes	K factor	0.090	% Heavy Vehicles	5		
Terrain	Level	Left Turn Impact	No	D factor	0.58	Base Capacity	1700		
Posted Speed	45	Pass Lane Spacing	N/A	Peak Dir. Hourly Vol.	1301	Local Adj. Factor	1		
Free Flow Speed	50	% No Passing Zones	N/A	Off Peak Dir. Hourly Vol.	942	Adjusted Capacity	1659		
LOS Results									
v/c Ratio	0.86	Density	NA	PTSF	94.30	ATS	29.3	% FFS	58.7
FFS Delay	50.7	LOS Thresh. Delay	50.7	Service Measure	Density	LOS	E		
Service Volumes									
Note: The maximum normally acceptable directional service volume for LOS E in Florida for this facility type and area type is 1500 veh/h/ln.									
		LOS	A	B	C	D	E		
		Lanes	Hourly Volume In Peak Direction						
		2	*	140	280	590	1500		
		Lanes	Hourly Volume In Both Directions						
		4	*	250	490	1020	2590		
		Lanes	Annual Average Daily Traffic						
		4	*	3300	6300	13100	33300		

Table 6 Capacity, ADTs and Levels of Service for Honoapi'ilani Highway

In Full Buildout Year of 2020

Segment of Honoapi'ilani Highway	Daily Maximum Capacity(1)	Predicted ADT	Volume to Capacity Ratio	Predicted LOS	Comments
North of Transfer Station	56,600	29,850	0.53	C	Assumes widening to two through lanes in each direction
Transfer Station to O-turn 1	56,600	29,870	0.53	C	
O-turn 1 to O-turn 2	56,600	29,580	0.52	C	
O-turn 2 To RIRO	56,600	28,800	0.51	C	
RIRO to Existing Roadway	56,600	28,750	0.51	C	
Existing Roadway South of Olowalu Town Project	33,300	28,750	0.86	E	Under capacity on daily basis

(1) From Highplan calculations.

Conclusions

Based on a review of the preliminary development plan for Olowalu Town and a review of the traffic data for daily 24-hour traffic volumes and daily 24-hour trip generation, the following conclusions are made:

1. The Olowalu Town will generate about 32,800 total trips per day with about 26,700 of these trips being internal to the town.

2. The Olowalu Town will generate about 6,100 new trips per day external to the town with this traffic using Honoapi'ilani Highway to travel to or from other island destinations.

3. The new trips added to HPH will be distributed with a ratio of approximately 60% towards Lahaina and 40% towards Ma'alea. On the Lahaina side, it is expected that the new Olowalu Town will add about 3,600 vehicles per day to Honoapi'ilani Highway. On the Ma'alea side, it is expected the new Olowalu Town will add about 2,500 vehicles per day to Honoapi'ilani Highway.

4. The proposed relocation and widening of Honoapi'ilani Highway will provide significant additional capacity to Honoapi'ilani Highway. Within the widened portion of the highway, the predicted volume to capacity ratio is no more than 0.47, resulting in a predicted LOS of C or better.

5. Within the portion of Honoapi'ilani Highway that is not proposed for widening at this time, the predicted future volume to capacity ratio is higher at 0.86 with an expected LOS of E. However, this analysis indicates that the segment of roadway south of the proposed Olowalu Town would still operate at less than capacity. Also, it is predicted that overall speeds in this segment of the highway will still be approximately 29.3 mph, even with the full buildout of the Olowalu Town. The additional time to drive the entire route from the Lahaina end towards the Ma'alea end with the widening in place is predicted to be just slightly more than with the existing roadway. This is due to the increased speeds in the portion of the highway which will be widened to four lanes, as well as the shorter distance of that portion of the roadway combined with the slightly reduced speeds in the existing portion of the highway that will not be widened. For the total roadway from the County Solid Waste and Recycling Transfer Center to a point one mile south of the site the total travel time is predicted to increase by only about 5 seconds.

Recommendations

Based on a review of the data analyzed in this report and the above conclusions, the following recommendations are made:

1. Approval of the preliminary plan and roadway access plan should be approved since the analysis indicates that the roadway capacity will be sufficient to accommodate the traffic added by Olowalu Town.

2. The analysis of the daily capacity and traffic flow indicates that the proposed roadway and access system will be sufficient.

3. The Final TIAR will include detailed analysis of peak hour conditions for Olowalu Town. Detailed analysis in the Final TIAR of the morning and afternoon peak hours will reveal

how well the system will operate at those hours. It is expected the results will indicate the proposed system with O-turns will work well.

4. Additional options for access and traffic control should also be examined in the Final TIAR to include the possibility of traffic signals. Mitigations that would include the required number of through lanes and auxiliary lanes, signal coordination, signal phasing and design for safety will be included in the Final TIAR.

List of Appendices

1. Additional Site Plan and Trip Generation Information
2. Trip Distribution Information
3. Trip Assignment and Urban Transportation Demand Model Information
4. Traditional Neighborhood Design and Internal Capture Information
5. O-turn and Michigan U-turn Information
6. Florida D.O.T. Highway Capacity Tables

1

TIPS Site Summary Worksheet

Site Information		**Adjacent Highways Passby Information**	
Name of Development	Olowalu	**North/ South Roadway**	
Name of Applicant	F&W	**North/ South Daily Hour Volume**	
Name of Analyst	Dyar		
Date	02/08/2011	**East/ West Roadway**	
Development Phase	Full Buildout	**East/ West Daily Hour Volume**	
Analysis Year	2030		

ITE Code	Land Type	# Units	Independent Variable	Total Single Use Trips	Trip Generation		IC Trips Based on IC Rate		Balanced IC Trips w/Reason Check		Total IC Trips	Real IC %	Trips on External Roadway		Total Trips	Result Pass By	Street Dir	MOD
					In	Out	In	Out	In	Out			In	Out				
730	Government Office Building	15000	sq. feet gross floor area	179	90	89	0	0	0	0	0	0	90	89	179	-	N/S	*
110	General Light Industrial	26000	sq. feet gross floor area	181	90	91	0	0	0	0	0	0	90	91	181	-	N/S	*
590	Library	5000	sq. feet gross floor area	270	135	135	0	0	0	0	0	0	135	135	270	-	N/S	*
417	Regional Park	77	acres	352	176	176	0	0	0	0	0	0	176	176	352	-	N/S	*
310	Hotel	58	rooms	474	237	237	0	0	0	0	0	0	237	237	474	-	N/S	*
732	United States Post Office	5000	sq. feet gross floor area	541	270	271	128	200	128	200	328	61	142	71	213	0	N/S	
710	General Office Building	60000	sq. feet gross floor area	661	330	331	155	239	155	239	394	60	175	92	267	-	N/S	
230	Residential Condominium/Townhouse	174	dwelling units	1028	514	514	489	492	489	492	981	95	25	22	47	-	N/S	*
220	Apartments	260	dwelling units	1747	874	873	756	816	756	816	1572	90	118	57	175	-	N/S	*
944	Gasoline/Service Station	20	vehicle fueling positions	3371	1686	1685	1493	1496	1493	1496	2989	89	193	189	382	0	N/S	*
220	Apartments	593	dwelling units	3985	1992	1993	0	0	0	0	0	0	1992	1993	3985	-	N/S	*
210	Single-Family Detached Housing	523	dwelling units	4764	2382	2382	1019	838	1019	838	1857	39	1363	1544	2907	-	N/S	*
820	Shopping Center	114000	sq. feet gross leasable area	7395	3698	3697	2458	2499	2458	2499	4957	67	1240	1198	2438	0	N/S	
820	Shopping Center	125000	sq. feet gross leasable area	7851	3926	3925	2542	2460	2542	2460	5002	64	1384	1465	2849	0	N/S	
	Total Volume			32799			9040	9040	9040	9040	18080	55%	7360	7359	14719			

(indicates the land use was modified from the original rates.)*

2

Trip Distribution Calculations Olowalu Town

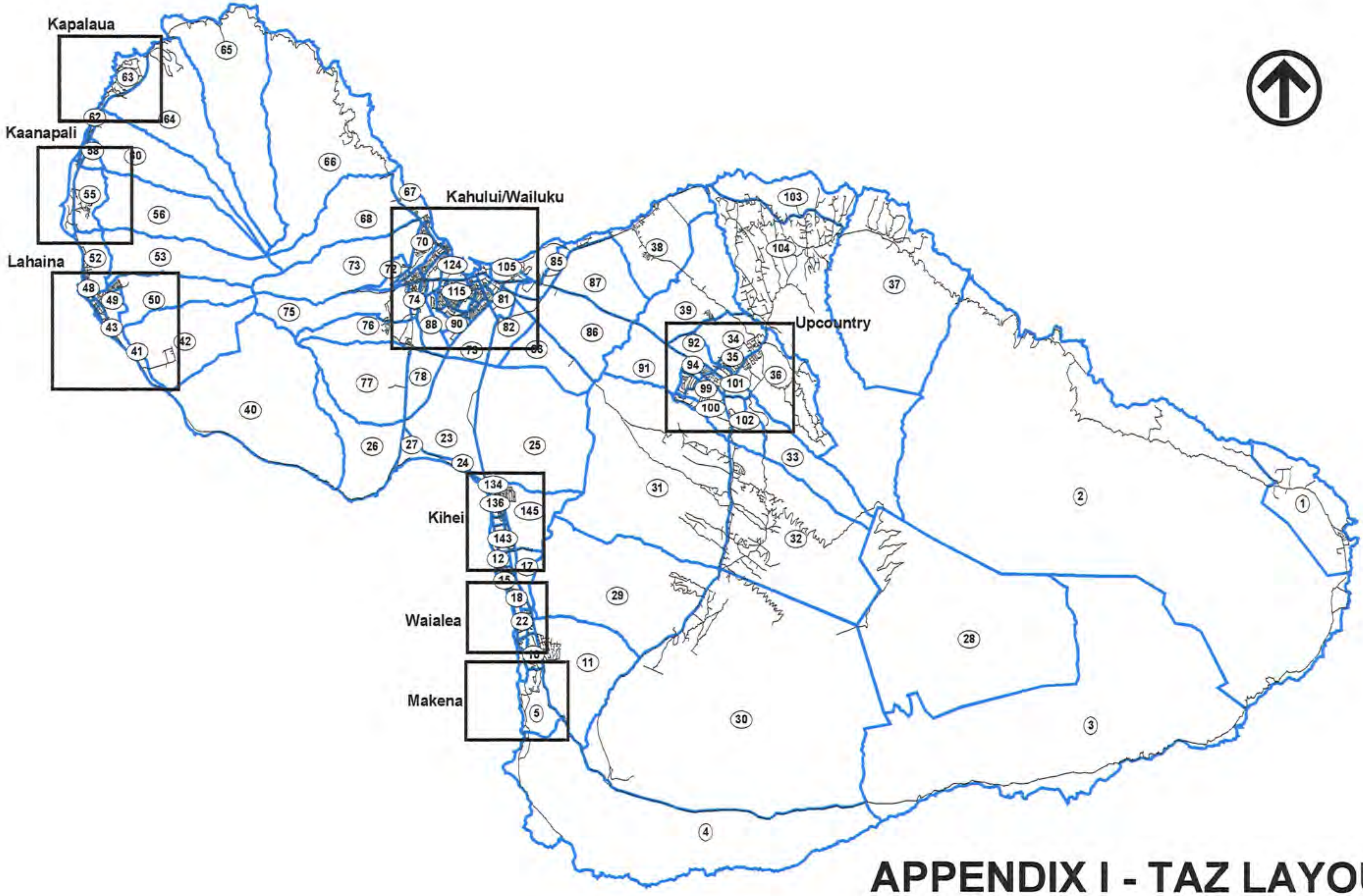
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC
1	ITE Land Use Code	Land Use Type	# Units	Independent Variable	Internal Capture Rate Est.	Unit Conversion	Trip Rate Per Unit Daily	Total Single Use Trips Daily	Directional Distribution		Trip Generation		Town Only		External of Town		External Passby Estimate	External New Only Added		External Passby Only		Internal Total	External Total	External New	External Passby	External New		External Passby	
2					(Town)		(ITE)		In	Out	In	Out	In	Out	In	Out	Tot %	In	Out	In	Out					In	Out	In	Out
3											C/F*G*I	C/F*G*J	K*E	L*E	K-M	L-N		(1-Q)*O	(1-Q)*P	O-R	O-R								
4	230	Government Office Building	15,000	sq. feet gross floor area	85%	1000	11.9	179	50%	50%	89	89	76	76	13	13	50%	7	7	7	7	152	27	13	13	7	7	7	7
5	110	General Light Industrial	26,000	sq. feet gross floor area	30%	1000	7.0	181	50%	50%	91	91	27	27	63	63	20%	51	51	13	13	54	127	101	25	51	51	13	13
6	590	Library	5,000	sq. feet gross floor area	90%	1000	54.0	270	50%	50%	135	135	122	122	14	14	20%	11	11	3	3	243	27	22	5	11	11	3	3
7	417	Regional Park	77	acres	20%	1	4.6	352	50%	50%	176	176	35	35	141	141	20%	113	113	28	28	70	282	225	56	113	113	28	28
8	310	Hotel	58	rooms	20%	1	8.2	474	50%	50%	237	237	47	47	190	190	20%	152	152	38	38	95	379	303	76	152	152	38	38
9	732	United States Post Office	5,000	sq. feet gross floor area	95%	1000	108.2	541	50%	50%	271	271	257	257	14	14	80%	3	3	11	11	514	27	5	22	3	3	11	11
10	230	Residential Condominium/Townhouse	174	dwelling units	40%	1	5.9	874	50%	50%	437	437	149	149	288	288	20%	231	231	58	58	350	524	419	105	210	210	52	52
11	220	Apartments	260	dwelling units	40%	1	6.7	1,485	50%	50%	742	742	297	297	445	445	20%	356	356	89	89	594	891	713	178	356	356	89	89
12	944	Gasoline/Service Station	20	vehicle fueling positions	90%	1	168.6	3,371	50%	50%	1,686	1,686	1,517	1,517	169	169	90%	17	17	152	152	3,034	337	34	303	17	17	152	152
13	220	Apartments	594	dwelling units	40%	1	6.3	3,162	50%	50%	1,581	1,581	632	632	949	949	20%	759	759	190	190	1,265	1,897	1,518	379	759	759	190	190
14	210	Single-Family Detached Housing	262	dwelling units	40%	1	9.1	2,027	50%	50%	1,013	1,013	405	405	608	608	20%	486	486	122	122	811	1,216	973	243	486	486	122	122
15	210	Single-Family Detached Housing	261	dwelling units	40%	1	9.1	2,021	50%	50%	1,010	1,010	404	404	606	606	20%	485	485	121	121	808	1,212	970	242	485	485	121	121
16	820	Shopping Center (Retail)	188,000	sq. feet gross leasable area	50%	1000	50.1	9,419	50%	50%	4,709	4,709	2,355	2,355	2,355	2,355	80%	471	471	1,884	1,884	4,709	4,709	942	3,768	471	471	1,884	1,884
17	820	Shopping Center (Retail)	187,000	sq. feet gross leasable area	50%	1000	50.1	9,362	50%	50%	4,681	4,681	2,340	2,340	2,340	2,340	80%	468	468	1,872	1,872	4,681	4,681	936	3,745	468	468	1,872	1,872
18								33,717			16,858	16,858	8,664	8,664	8,195	8,195		3,608	3,608	4,586	4,586	17,380	16,337	7,175	9,162	3,587	3,587	4,581	4,581
19																													
20																							33,717		16,337	7,175	9,162		
21																													
22																													
23	Column	Description and Meaning								Trips		Formula			% Trips	Amt.	% Increase												
24	A	ITE land use code								Total for site	33,717	H16			100%														
25	B	Land use description				Distribution				Internal to site	17,380	M16 + n16			52%														
26	C	Units (provided by F&W)				Maalea	40%			External	16,337	K22 - K23			48%											33,717			
27	D	Variable for calculations				Lahaina	60%			External new	7,175	R16 + S16			21%											32,673			
28	E	Internal capture rate for each use (estimated)								External passby	9,162				27%														
29	F	Unit conversion factor																											
30	G	ITE Trip Rate per unit (national average)								Existing ADT	22,900																		
31	H	Trips generated by each land use								Add new ADT	7,175	K25							Capacity	Spare	%	Yrs at 1%							
32	I	Percentage of trips into the land use				Internal Capture %		52%	K25/K24	Lahaina Side	27,205	K28 + (K29*G25)				4,305	19%		31,700	4,495	20%	16							
33	J	Percentage of trips out of the land use			12.73581	New Lahaina Side %		13%		Maalea Side	25,770	K28 + (K29*G24)				2,870	13%		31,700	5,930	26%	22							
34	K	Trips generated into each land use (C * G * I)			16.31965	Passby Lahaina Side %		16%		Total ADT						7,175	31%												
35	L	Trips generated out of each land use (C * G * J)			8.490543	New Maalea Side %		8%																					
36	M	Trips to/from the town itself (K * E) inbound			10.87972	Passby Maalea Side %		11%																					
37	N	Trips to/from the town itself (L * E) outbound						100%																					

Island Trip Distribution From Urban Model

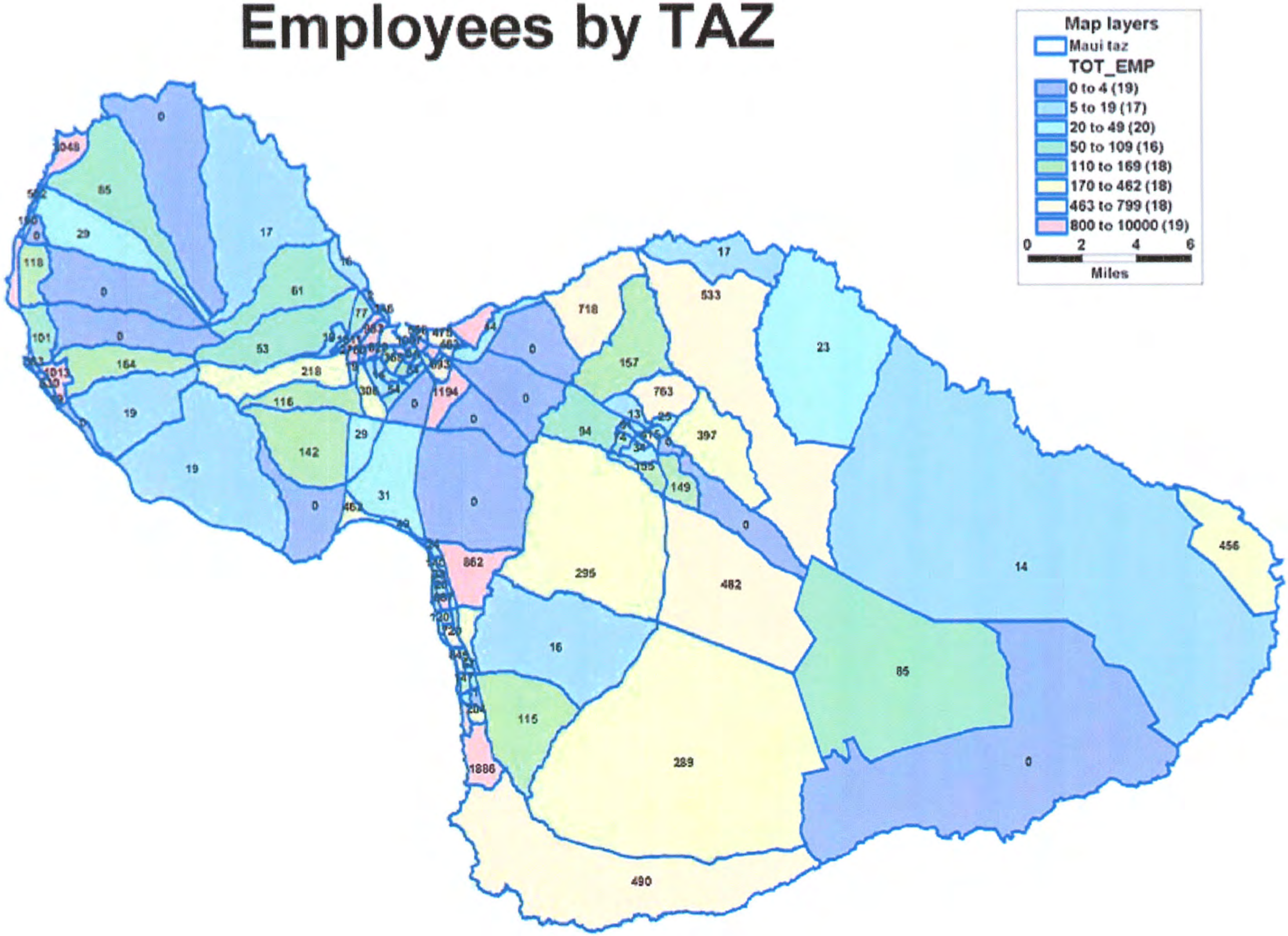
	A	B	C	D	E	F	G
2	Section	Lahaina	Kaanapali	Honokowai Napili	Kahana	Kapalua	Total
3	West Maui	17%	28%	3%	5%	7%	60%
4	Kahului	4%	6%	0%	0%	1%	11%
5	Wailuku	5%	8%	1%	0%	1%	15%
6	Kihei	3%	4%	0%	0%	1%	8%
7	Up Country	2%	3%	0%	0%	1%	6%
8	Other	0%	0%	0%	0%	0%	0%
9	Total	31%	49%	4%	5%	11%	100%

3

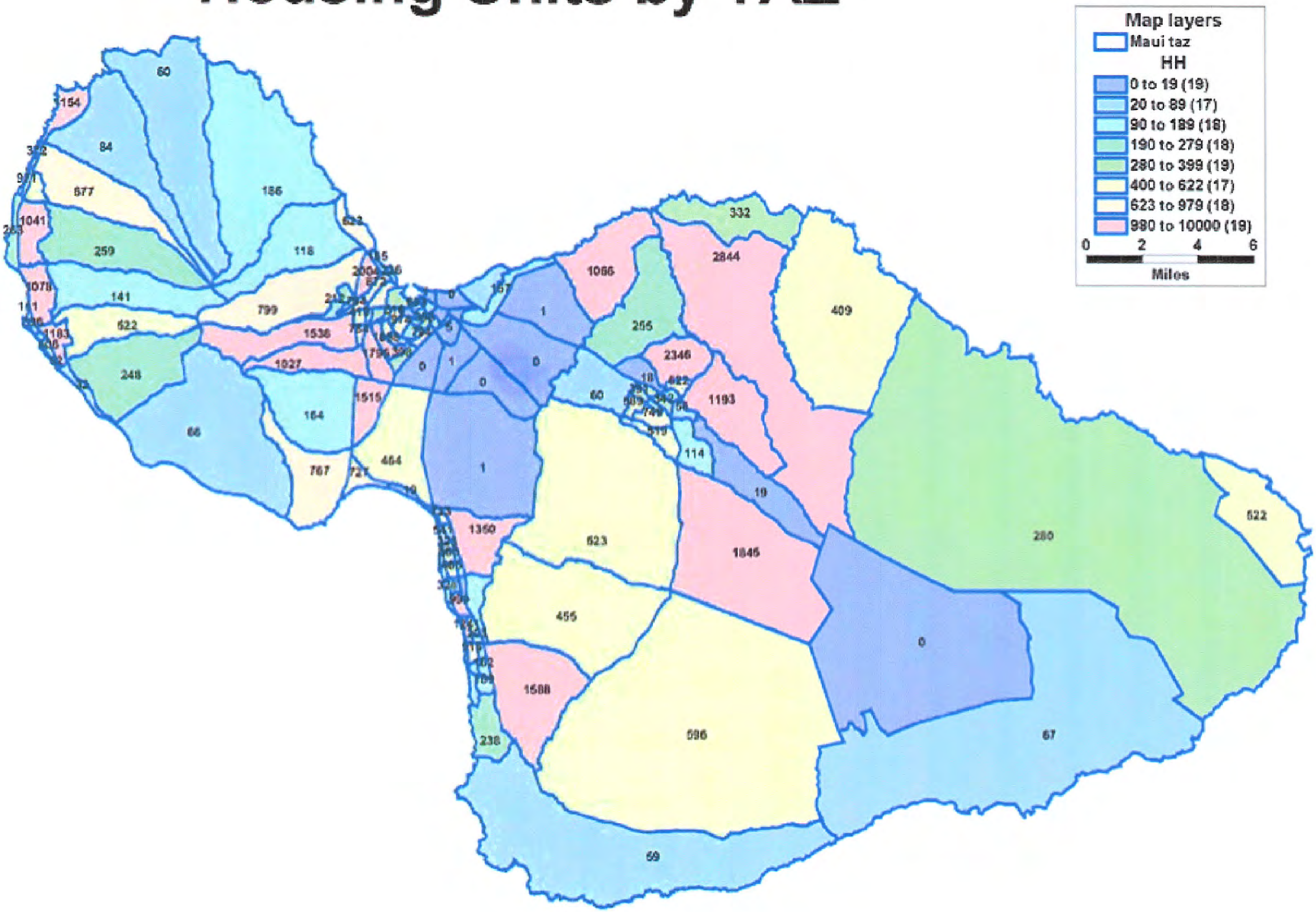
APPENDIX I - TAZ LAYOUT



Employees by TAZ



Housing Units by TAZ



Trips in 1990 and 2020 By Community Name and By Purpose

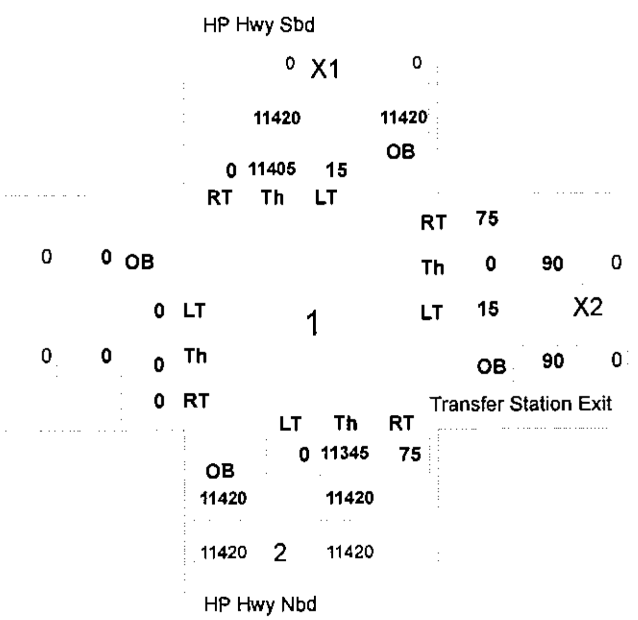
	A	B	C	D	E	F	G	H	I	J	K	L
1												
2												
3			1990					2020				
4			Home Based Work	Home Based Other	Non-Home Based	Visitors	Total	Home Based Work	Home Based Other	Non-Home Based	Visitors	Total
5	**Community Name**	**aka 1997 LRTP**										
6	West Maui	Lahaina										
7	Kihei-Makena	Kihei-Makena										
8	Wailuku-Kahului	Wailuku-Kahului										
9	Makawao-Pukalani-Kula	Makawao-Pukalani-Kula										
10	Paia-Haiku	Paia-Haiku										
11	Hana	Hana										
12	Total Daily Trips		43,260	89,660	65,590	35,140	233,650	69,050	143,130	114,250	59,850	386,280
13	A.M. Peak Hour		8,980	4,490	1,450	2,600	17,520	14,340	7,170	2,520	4,430	28,460
14	P.M. Peak Hour		9,070	5,050	2,210	4,220	20,550	14,470	8,050	3,840	7,190	33,550
15												
16	**Percentage of Total Trips by Purpose Daily**		**18.5%**	**38.4%**	**28.1%**	**15.0%**		**17.9%**	**37.1%**	**29.6%**	**15.5%**	
17	**Percentage of Total Trips by Purpose AM**		**51.3%**	**25.6%**	**8.3%**	**14.8%**		**50.4%**	**25.2%**	**8.9%**	**15.6%**	
18	**Percentage of Total Trips by Purpose PM**		**44.1%**	**24.6%**	**10.8%**	**20.5%**		**43.1%**	**24.0%**	**11.4%**	**21.4%**	
19												
20												
21			1990 LRTP Daily Trips		2020 LRTP Daily Trips							
22		From	To/from	To/from	To/from	To/from						
23		Site by Purpose	Lahaina	All others	Lahaina	All others						
24		HBW	12,899	30,361	19,426	49,624						
25		HBO	16,431	73,229	20,230	122,900						
26		NHB	10,475	55,115	20,969	93,281						
27		Visitor	21,547	13,593	32,943	26,907						
28		Total	61,352	172,298	93,568	292,712						
29			26.3%	73.7%	26.3%	73.7%						
30	Daily 1990 % by Purpose							2020 % by Purpose				
31	18.5%	HBW	29.8%	70.2%	28.1%	71.9%		17.9%				
32	38.4%	HBO	18.3%	81.7%	14.1%	85.9%		37.1%				
33	28.1%	NHB	16.0%	84.0%	18.4%	81.6%		29.6%				
34	15.0%	Visitor	61.3%	38.7%	55.0%	45.0%		15.5%				
35		Total	26.3%	73.7%	24.2%	75.8%						
36												
37												
38												
39												
40												
41												
42		Example of 5000	To Lahaina (left out)	All others (right out)								
43		5000	276	650								
44		5000	352	1567								
45		5000	224	1179								
46		5000	461	291								
47		Total	1313	3687								

Intersection Turning Movements

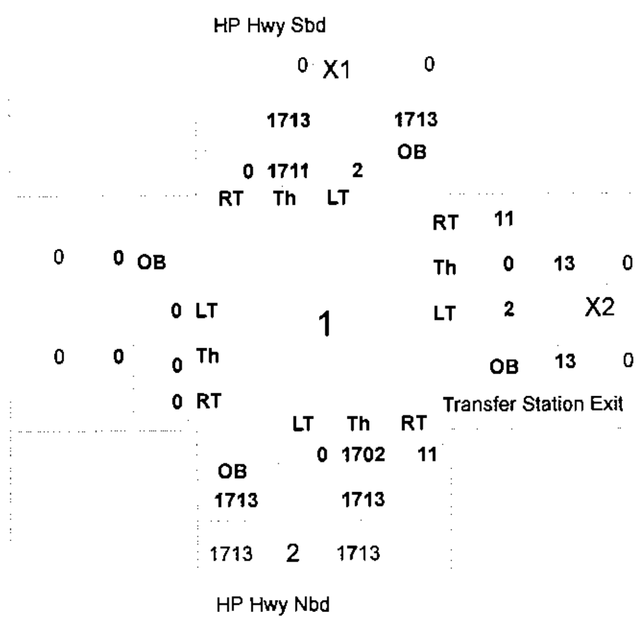
Project: Olowalu New Town
Scenario: Preliminary Traffic Report Full Buildout + 15% Growth
Time Period: Daily

Intersection Name: HP Hwy at Transfer Station
Intersection ID: 1

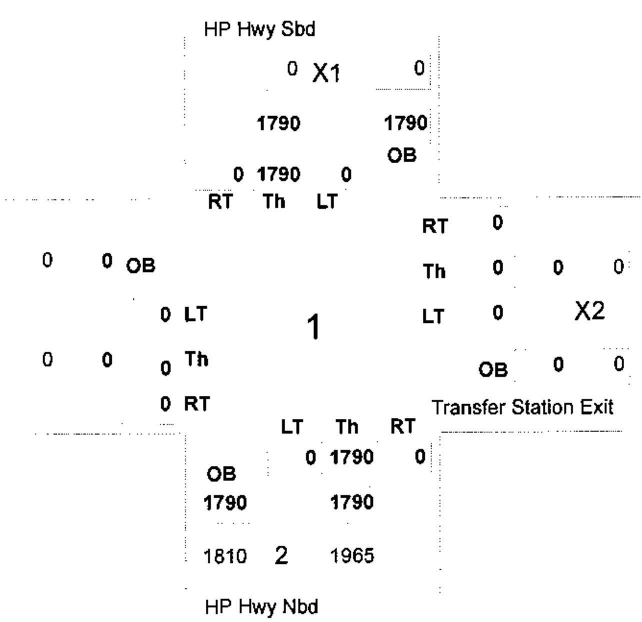
Existing



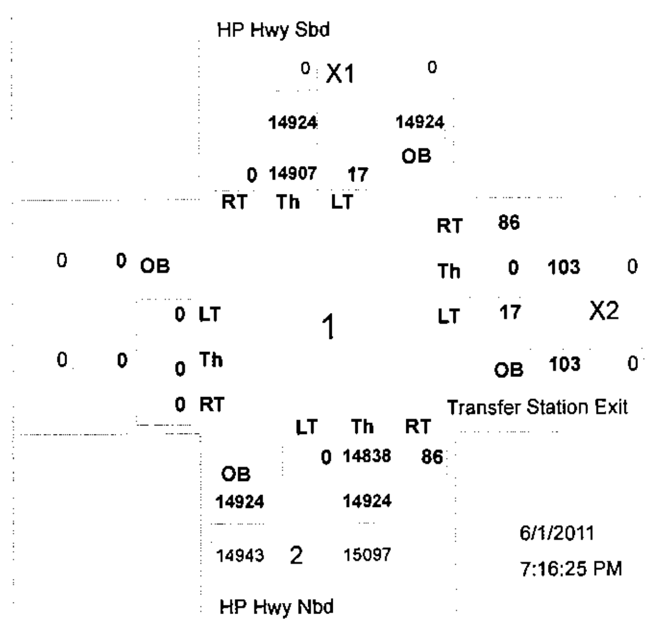
Growth



Site



Future



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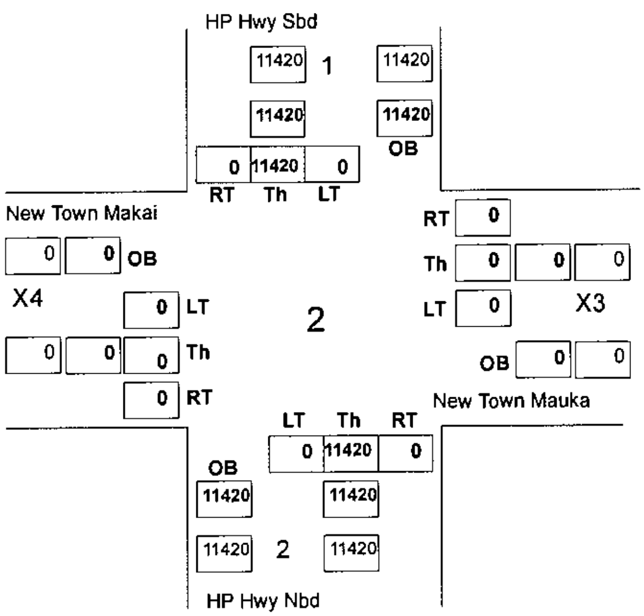
Intersection Turning Movements

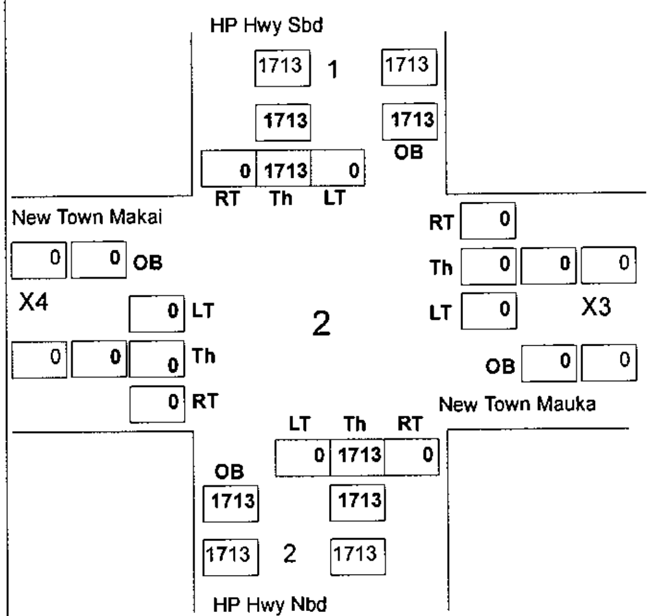
Project: Olowalu New Town
Scenario: Preliminary Traffic Report Full Buildout + 15% Growth
Time Period: Daily

Intersection Name: HP Hwy at New Town Access 1
Intersection ID: 2

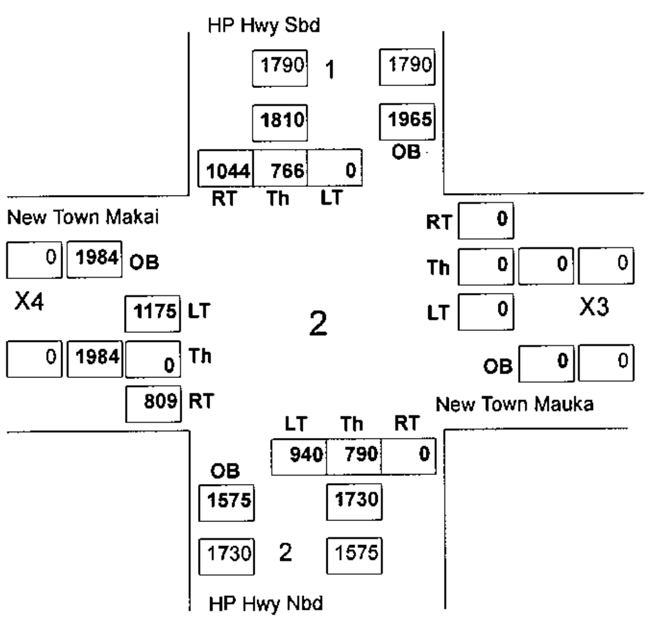
Existing



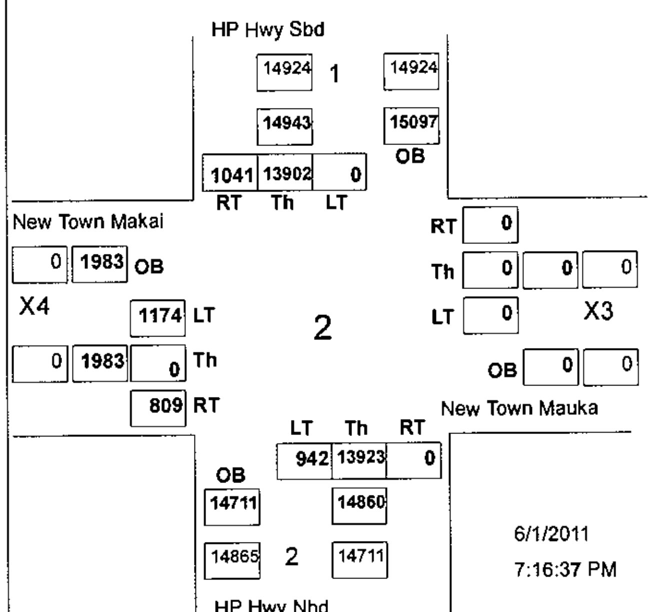
Growth



Site



Future



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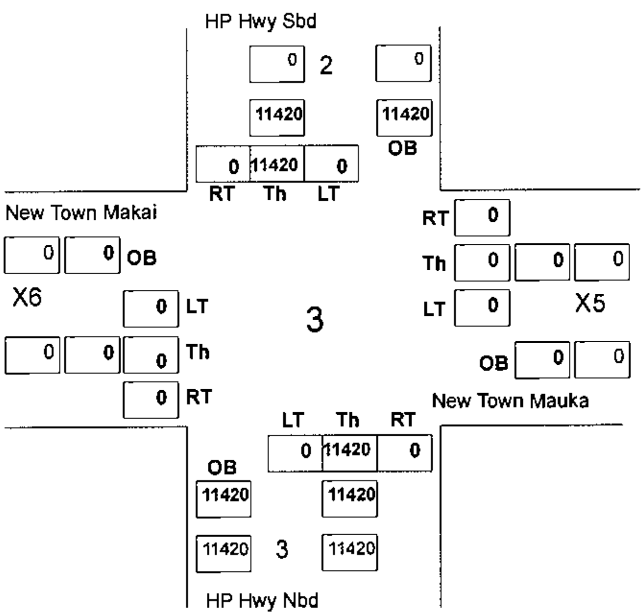
Intersection Turning Movements

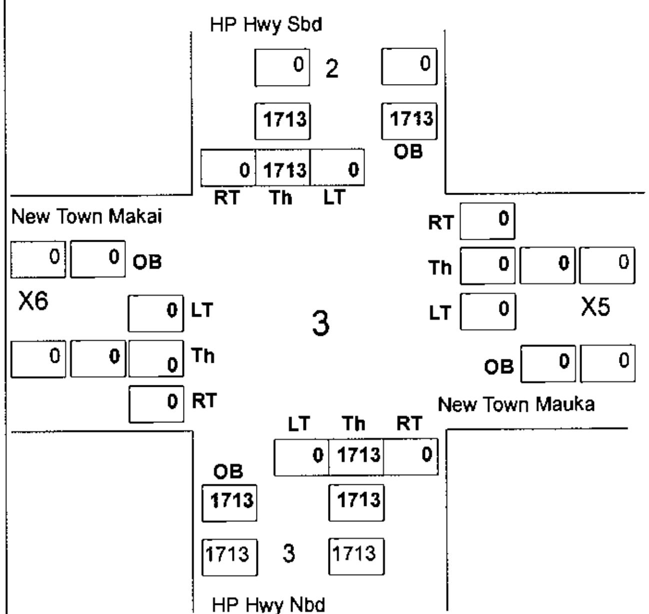
Project: Olowalu New Town
Scenario: Preliminary Traffic Report Full Buildout + 15% Growth
Time Period: Daily

Intersection Name: HP Hwy at New Town Access 2
Intersection ID: 3

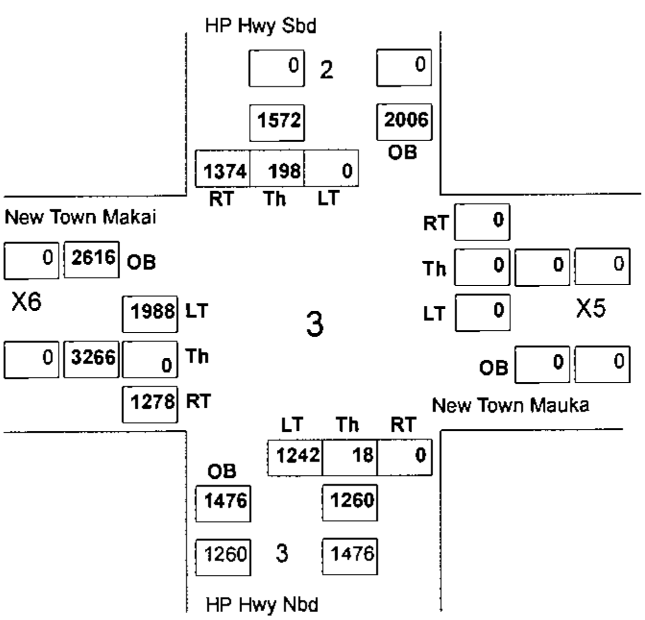
Existing



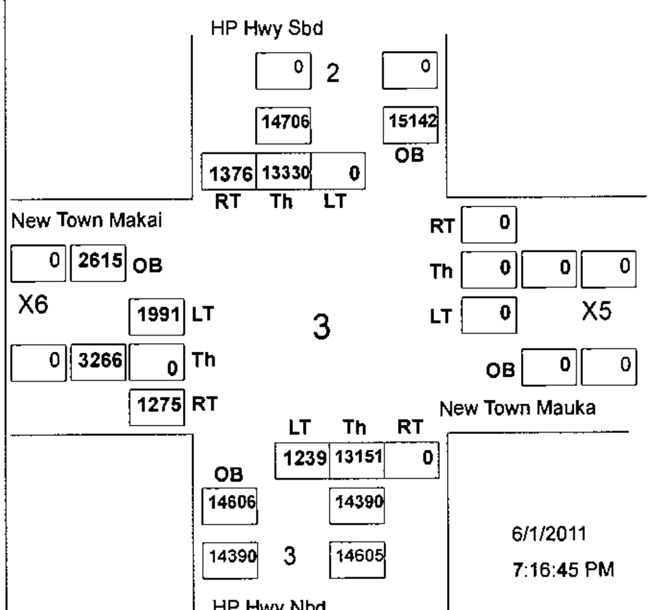
Growth



Site



Future



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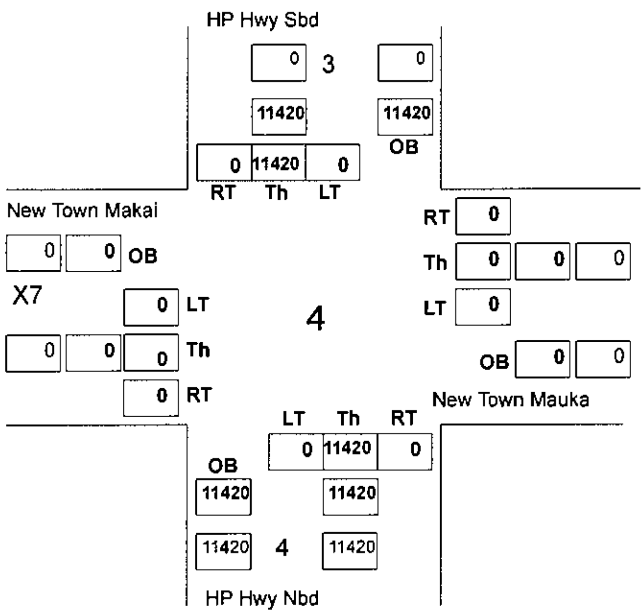
Intersection Turning Movements

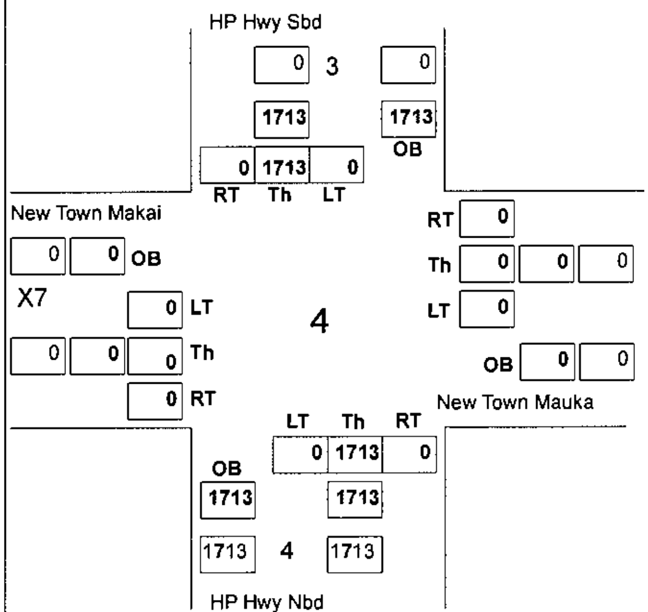
Project: Olowalu New Town
Scenario: Preliminary Traffic Report Full Buildout + 15% Growth
Time Period: Daily

Intersection Name: HP Hwy at New Town Access RIRO
Intersection ID: 4

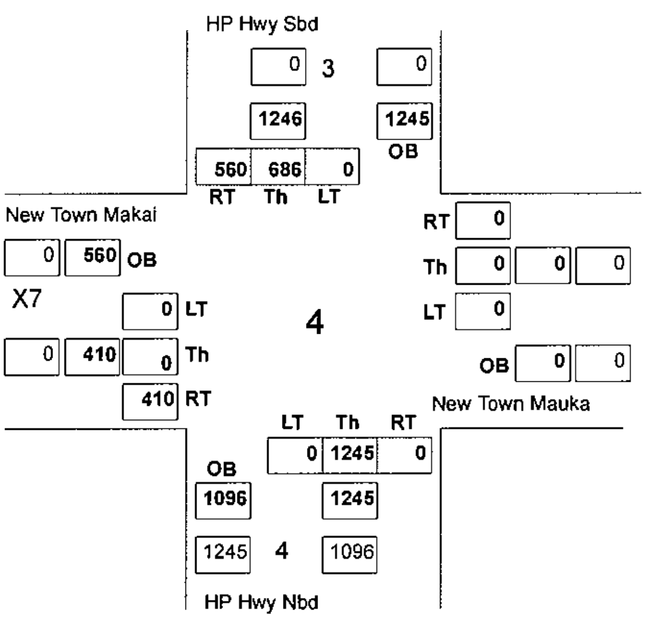
Existing



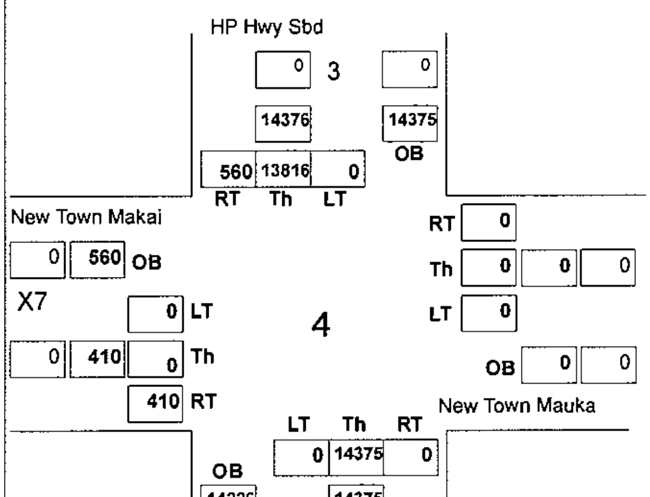
Growth



Site



Future



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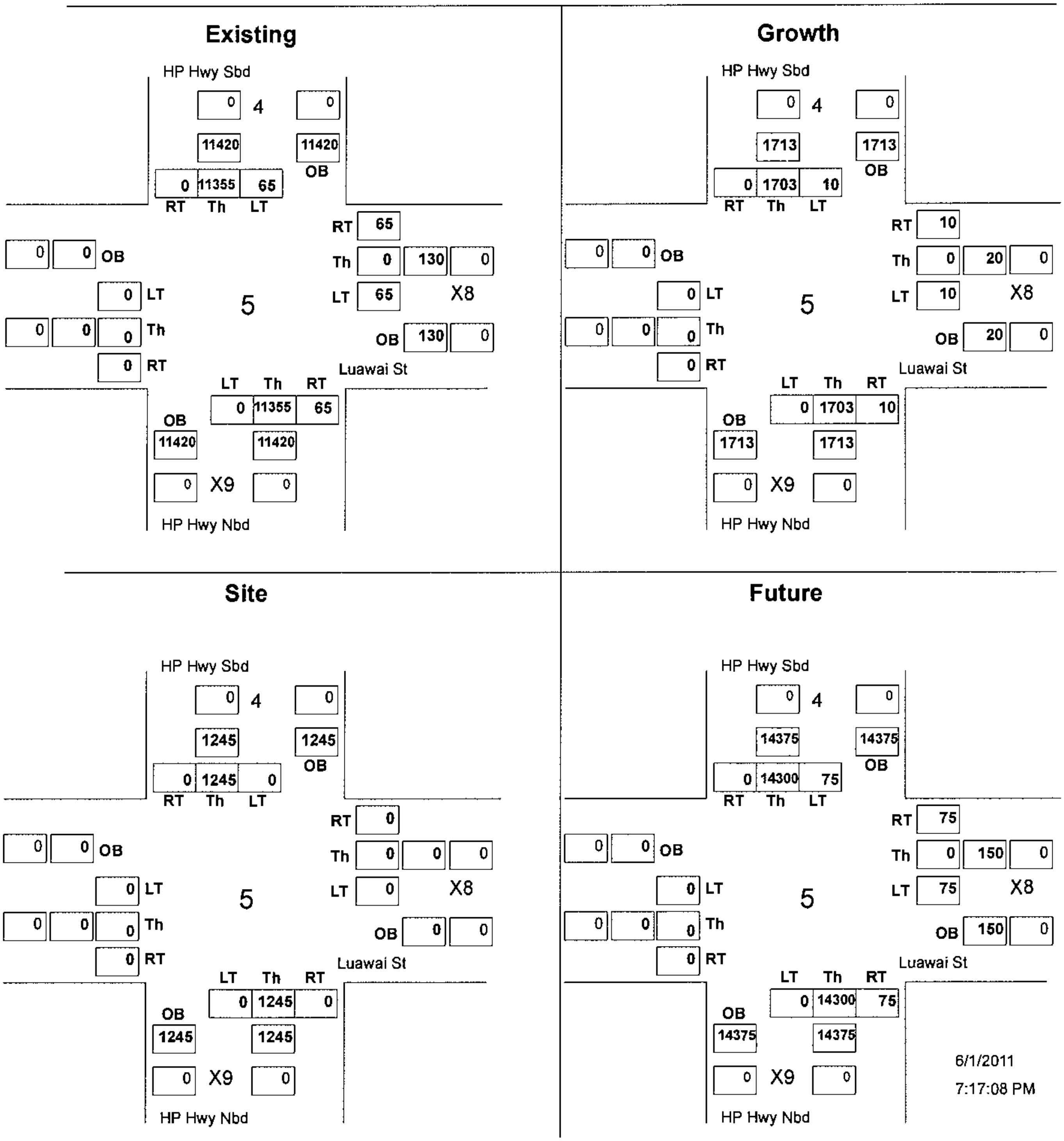
Intersection Turning Movements

Project: Olowalu New Town
Scenario: Preliminary Traffic Report Full Buildout + 15% Growth
Time Period: Daily

Intersection Name: HP Hwy at Luawai
Intersection ID: 5



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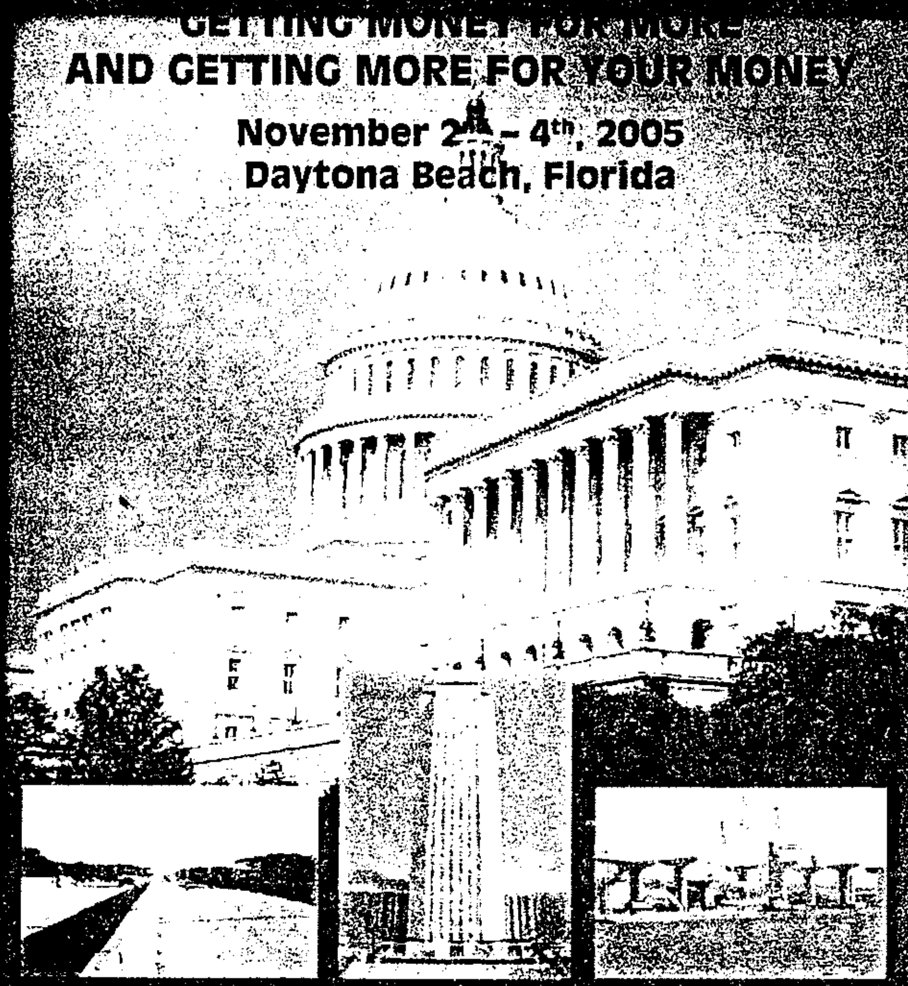

INSIDE

- PREDICTING FUTURE TRAVEL PATTERNS IN FLORIDA MPO AREAS
- TRADITIONAL DEVELOPMENT TRIP GENERATION CHARACTERISTICS
- FSITE STRATEGIC PLAN
- FSITE CHAPTER NEWS
- AND MORE

Traditional Development Trip Generation Characteristics

I. Introduction

Planned communities today are widely employing a New Urbanism approach in design. These concepts are hailed as one of the remedies to the rapid utilization of resources due to suburban sprawl. Traditional neighborhood developments incorporate a varied mixture of land uses usually within walking distance of one another. The proximity of these easily accessible facilities is intended to promote pedestrian, bicycle, and shorter internal auto trips. A large percentage of internal trips should be the result and lessen the impact of the community on the surrounding roadway network. Internal trips are defined as those that have both trip ends within the development project. Although they often utilize one or more segments of a public roadway, there is no net increase in traffic volume on the external roadway system outside the boundary of the project.

These benefits of internal capture and the level of trip reduction associated with traditional land use development, however, are supported by a minimal amount of documented research. Estimations for traffic characteristics such as trip generation and internal capture are sometimes challenged due to this lack of information. This report presents available information from the research conducted for traditional neighborhood developments and provides sources for greater inquiry.

II. Study Review

A. Celebration Monitoring & Modeling Study

Published in September 2003, the Celebration Monitoring & Modeling Study provides a quantified insight into an established traditional community. The report was mandated by the Third Amended and Restated Development Order of the Celebration Development of Regional Impact (DRI). The report contributes information on many aspects of the community. Relevant information includes the development program, traffic counts, trip generation, and internal capture percentages.

Celebration is located in Osceola County, Florida. Interstate 4 (1-4) and US 192 form the community's north and west borders, respectively. Other communities in the area include Lake Buena Vista at approximately 7 miles, and Kissimmee at approximately 10 miles away.

B. Traditional Neighborhood Development Trip Generation Study

Traditional Neighborhood Development Trip Generation Study was prepared by Dr. Asad J. Khattak, Dr. John R. Stone, William E. Letchworth, Ben K. Rasmussen and Bastian J. Schroeder in February 2005. The report was completed for the North Carolina Department of Transportation by the Department of City and Regional Planning of the University of North Carolina in Chapel Hill and the Department of Civil, Construction, and Environmental Engineering at North Carolina State. The report uses local traffic impact analysis methods to estimate trip generation rates. These estimated trip generation rates were compared with actual traffic counts produced by the existing developments.

The Traditional Neighborhood Development surveyed for this study is Southern Village, which is located within the city limits of the City of Chapel Hill. The other suburban development surveyed is located within the Northern Carrboro area. The developments are approximately 7 miles apart. The nearest commercial district is Franklin Street of Chapel Hill and the University of North Carolina and both developments are located equal distances away.

C. Haile Plantation Traffic Study

The Haile Plantation Traffic Study was prepared in November 15, 1996 by Buckholz Traffic. The study was completed to monitor the effect of changes to the Haile Plantation Master Plan and to document the actual project traffic characteristics. Both machine and manual intersection turning movement counts were conducted for the study. The manual counts were conducted at the intersection of SW 46th Boulevard and SW 75th Street, and SW 91st Street and Archer Road. The machine counts were taken at SW 46th Boulevard and SW 91st Street. These locations were determined sufficient to record all traffic movements from the traditional project elements.

Haile Plantation Village is located in Alachua County, FL. The development is approximately 7 miles from the commercial district of the City of Gainesville.

Continued on page 17

D. Internalizing Travel by Mixing Land Uses: A Study of Master-Planned Communities in South Florida

Internalizing Travel by Mixing Land Uses: A Study of Master-Planned Communities in South Florida, a study completed by Reid Ewing, Eric Dumbaugh, and Mike Brown, was presented at the 80th Transportation Research Board Conference in Washington D.C. The travel data originated form the Southeast Florida Travel 2000 Survey conducted by the Florida Department of Transportation. The land use data originated from metropolitan planning organizations and the Florida Department of Transportation. The information for Palm Beach and Broward counties was updated in 1999; the information for Dade County in 1996. Land use measures employed in the study were size, density, entropy, balance and accessibility.

Twenty different neighborhood developments from Palm Beach, Broward and Dade counties are included in this study. These communities were built over the last 40 years, and were both family and retirement oriented. The land uses included in each development included "housing, shopping, services, and recreational facilities."

E. Other Traditional Neighborhood Development Resources

Using the New Urban News' list of traditional neighborhood communities as a starting point, a list of additional nearly or recently completed developments was compiled. Local jurisdictions and development companies were contacted in reference to traffic studies for each of these projects but no usable information was available. These communities could provide opportunities for study in the future.

- Seaside, Walton County, FL. 350 Single Family (SF), 60 ksf commercial, charter school, chapel
- Kentlands, Gaithersburg, MD. 520 SF, 1539 Multi Family (MF), 1,000 ksf commercial & office, elementary school, convenience store/gas station/car wash
- Birkdale Village, Huntersville, NC. 230 SF, 360 MF, 234,921 sf Specialty Retail, 52,202 sf Office
- Daniel Island, Charleston, SC. 10,000 seat Tennis Stadium, 16 tennis courts, 5,100 seat Soccer Stadium, Private High School, K-8 Public, Preschool/Daycare
- I'ON, Mt. Pleasant, SC. 762 SF, 30,000 sf Commercial
- Harbor Town, Memphis, TN. 550 SF, 345 MF, 43 ksf commercial
- Middleton Hills, Middleton, WI. 327 SF, 87 MF, 150 ksf commercial

III. Traffic Generation & Internal Capture

A. Celebration, Osceola County, FL

The land use composition at the time of the Monitoring & Modeling (M&M) study is listed below. In keeping with the traditional community philosophy, the development program is quite diverse. Since Celebration development is still under way, this mixture of uses will change with time.

- Single Family — 2,232 DU
- Retail — 65,687 square feet
- Multi-Family — 1,868 DU
- Hospital — 100 beds
- Hotel — 90 rooms
- Medical Office — 204,940 square feet
- Office — 922,857 square feet
- Golf Course — 18 holes

Data was collected for the Celebration M&M during 72-hour machine cordon line counts. These counts revealed a significant difference from the internal trip values predicted by the ITE Trip Generation Handbook 6th Edition. This difference was evident in both the peak hour and daily trips and created significant internal capture percentages.

- Total Peak Hour Project Trips (actual counts): 3,458
- Total Peak Hour Project Trips (ITE, 6th Edition): 5,044
- Total Daily Project Trips (actual counts): 40,912
- Total Daily Project Trips (ITE, 6th Edition): 56,544
- Calculated Peak Hour Internal Capture: 31.8%
- Calculated Daily Internal Capture: 27.7%

B. Southern Village, Chapel Hill, NC

The study of Southern Village compares the development to a similar conventional neighborhood. Both development programs are depicted below and the multi-use housing is included for Southern Village.

Southern Village:

- Single Family — 611 DU
- Church — 27,000 square feet
- Multi-family — 309 DU
- Retail — 30,000 square feet
- School — 90,000 square feet
- Office — 95,000 square feet
- Day Care Center — 6,000 square feet

Northern Carrboro:

- Single Family — 891 DU

Continued on page 18

Traditional Development Trip Generation Characteristics, *continued from page 17*

Traffic counts for Southern Village single-family households recorded a value of 1,336 for the PM peak-hour and a value of 12,609 for the daily period. Using ITE trip generation methods, estimated trip generation values were calculated at 1,363 for the PM peak and 12,250 for the daily. The percent difference between actual and estimated is negligible. Thus, the findings confirm that the ITE trip generation methods were very reasonable for Southern Village.

The authors concluded that the residents of Southern Village did not make significantly fewer trips than residents of the conventional neighborhood. However, the trips by residents of Southern Village were shorter in time and distances, used different modes, and were less frequently external. For instance, each household in Southern Village was responsible for 7.7 auto trips per day. A single-family household in the conventional neighborhood created approximately 10 auto trips per day. The study also found that Southern Village residents traveled 28 fewer miles per day. In Southern Village, 78.4% of all trips were made by automobile. This compares favorably with an average of 89.9% for the Northern Carrboro suburbs, a 92.4% regional average, and a national average of 87%. The internal capture percentage for Southern Village was calculated to be 20.2 %. Southern Village residents also produced 25.8% fewer external trips and 30.3% fewer regional trips.

The intent of this study was to produce a comparison of traffic studies for traditional neighborhood development and conventional communities. The report suggests that no matter the type of residential use, tradition neighborhood developments produce different traffic behaviors than the conventional community and that those behaviors include fewer external and long distance trips.

C. Haile Village Plantation Center, Alachua County, FL

The development program for Haile Plantation is listed below. The community was not complete when the study was conducted, however, a variety of land uses were represented to a large extent.

- Single Family 1,070 DU
- Multi Family 2,460 DU
- Retail 175,000 square feet
- Office 175,000 square feet
- Church 18,000 square feet
- Church/School 12,000 square feet & 600 Students
- School Elementary/Middle 1,650 Students

The traffic study conducted in 1996 concluded that Haile Village Plantation Center is exhibiting an internal capture rate of 24% of the daily trips, and 28% during the PM peak hour. The ITE trip generation manual estimated that a traditional neighborhood project like Haile Plantation would have an internal capture rate of approximately 32% of daily trips and 36% of PM peak hour trips. Based upon the comparison of these two sets of internal capture percentages, it was determined that due to the existing mixture and density of development , the percentages of 24% for the daily and 28% of the PM peak hour were an appropriate expectation for the project at its current state of completion.

D. Southern Florida – Palm Beach, Broward, and Dade Counties

The 20 Traditional Neighborhood Developments for this study were chosen based upon the mix of land uses. Each development was required to have housing, shopping, services, and recreational facilities. All were constructed after 1965. The size of the developments, in population and acreage, ranged significantly. Each of the developments was analyzed according to its gross acreage, population, and employment.

Community	Gross Acreage	Population	Employment
Aventura	692	8,303	5,965
California Club	1,234	13,649	1,869
Century Village	934	12,781	534
Century Village North	716	10,246	331
The Crossings	662	6,036	965
The Hammocks	863	13,801	1,338
Jonathan's Landing	1,205	4,211	3,127
Kendale Lakes	985	12,207	2,588
Kings Point	845	12,523	771
Miami Lakes	2,541	12,918	17,862
Mission Bay	3,851	10,598	7,869
Pembroke Meadows	1,687	5,638	1,032
PGA National	2,421	9,178	2,324
Sabel Chase	325	4,984	1,120
Silver Lakes	3,210	11,329	1,593
The Township	715	4,267	556
Village of Palm Beach Lakes	1,475	8,215	1,818
Wellington	10,727	34,267	5,220
Weston	15,517	44,199	9,206
Winston Park	1,464	8,017	440

The focus of this paper was to evaluate and report internal capture rates from a multitude of traditional neighborhood developments. The land use measures considered in calculations were size, density, entropy, balance, and accessibility. The scale of the development was directly related to capture levels and regional accessibility was inversely related to internal capture rates.

The internal capture percentages for the twenty "traditional" neighborhood developments varied greatly. However, the authors highlighted the trend of the largest traditional neighborhood developments also employing the largest internal capture percentages.

• Wellington, Palm Beach County	57 %
• Weston, Broward County	52 %
• Century Village Broward, Broward County	43 %
• The Township, Broward County	41 %
• Century Village North, Palm Beach County	40 %
• Village of Palm Beach Lakes, Palm Beach County	34 %
• Winston Park, Broward County	30 %
• The Hammocks, Dade County	28 %
• Silver Lakes, Broward County	27 %
• Miami Lakes, Dade County	25 %
• Mission Bay, Palm Beach County	18 %
• PGA National, Palm Beach County	17 %
• Aventura, Dade County	17 %
• Jonathan's Landing, Palm Beach County	13 %
• Sabel Chase, Dade County	13 %
• Kendale Lakes, Dade County	12 %
• Kings Point, Palm Beach County	10 %
• Pembroke Meadows, Broward County	9 %
• The Crossings, Dade County	6 %
• California Club, Dade County	0 %

The mean internal capture percentage was 25% and the median was 22%. The authors concluded that the traditional neighborhood developments most successful in obtaining increased internal capture were large in size and distant from similar regional trip attractions.

IV. Summary

Most of the studies reviewed in this report indicated support for more research into the effect of traditional neighborhood developments. The studies confirm that while traditional neighborhood developments lower trip distances and encourage other modes, the actual number of total trips produced does not decrease but the net number that reach the external roadway network are reduced by up to over 50% with an average of between 25% and 30%. Each of the studies supports the trend of increasing internal capture with increasing size and diversity of land uses. Many questions still remain regarding the effects on trip length and geographic effect on trip production, but the majority of researchers agree that the ITE trip generation estimates and the results produced using the methodologies are reasonably accurate for these communities.

In conclusion, the data available for use in the development of this paper supports the use of internal capture estimates produced using the ITE Trip Generation Handbook methodologies and that results ranging between 25% and 50% should not be questioned if the land use composition is favorable in size and diversity to supporting the estimated capture rate. Therefore, placing a cap on the level of internal capture that can be assumed for purposes of development planning, a common approach in some jurisdictions due to limited documented data, is not a practice that can be supported by this research.

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Case Study – From ITE Website

ITE Journal - Surface Transportation Security Lessons Learned from 9/11

The terrorist attacks of Sept. 11, 2001 exacted a terrible toll on the United States and fundamentally changed the way of life in America. Surface transportation has changed and continues to change in response to the attack. Agencies that own and operate surface transportation systems must understand the relevant lessons from the 9/11 experience and respond accordingly so that we as a nation are well prepared should we be attacked again. The U.S. Federal Highway Administration (FHWA) commissioned the John A. Volpe National Transportation Systems Center (Cambridge, MA, USA) and Science Applications International Corporation (SAIC) to prepare detailed case studies of surface transportation in the New York City (NYC) and Washington, DC metro areas on the day of and in the days and weeks following the attacks. Published material was reviewed, participants were interviewed and internal agency working documents were analyzed. Extensive chronologies of actions were prepared for each study and impacts of the actions were identified whenever possible. The final two case studies total more than 150 pages. A panel of participants in the actual events have reviewed and approved the studies. The following material summarizes the lessons learned, exploring what did and did not work. This article synthesizes the findings from both studies and presents lessons that can be made available to the transportation profession as a whole. http://www.ite.org/membersonly/itejournal/itejournal/pdf/2002/JB02IA38.pdf

A. Following approval of a final site plan and subdivision plat, the first seventy five percent (75%) of all certificates of occupancy for dwelling units shall be issued prior to the establishment of any non-residential use.

B. No certificate of use and occupancy may be issued for the remaining dwelling units until a certificate of use and occupancy has been issued for one-hundred percent (100%) of the non-residential floor area.

Table 407-2 Trip Reductions for Mixed Use Development

Percent Residential Equivalent Units	Percent Non-residential Equivalent Units	Percent Trips Reduced
85-100%	0-14%	Not Applicable
75-84%	15-25%	10%
65-74%	25-35%	20%
35-65%	35-74%	30%
25-34%	65-74%	20%
15-24%	75-84%	10%
0-14%	85-100%	Not Applicable

Rules of Interpretation for Table 407-2:

For purposes of computing the percentage established above, one dwelling unit or 800 square feet of non-residential space shall equal one (1) equivalent unit. The equivalent units shall be located within the boundaries of the proposed development.

(3) For purposes of this section, the overall trip generation for an eligible use (see subsection (4), below) in the DR, DB, or DBO district shall be reduced by thirty percent (30%).

(4) For purposes of this subsection, an "eligible use" includes any residential, retail, institutional or industrial use except Auto-Oriented Uses as defined in Article 10 of this Code.

(f) Stormwater management

Stormwater credits are defined in the Maryland Department of Environment, 2000 Maryland Stormwater Design Manual, which is hereby incorporated by reference. Credits are calculated for using non-structural practices including Natural Area Conservation, Disconnection of Rooftop Runoff, Disconnection of Non Rooftop Runoff, Sheet Flow to Buffers, Open Channel Use, and Environmentally Sensitive Development. The percentage refers to the reduction in Water Quality Volume (WQv) from a development.

Improved Estimation Of Internal Trip Capture For Mixed-Use Developments

THIS ARTICLE DESCRIBES THE KEY TASKS AND FINDINGS OF NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM PROJECT 8-51, WHICH DEVELOPED IMPROVEMENTS TO THE EXISTING ITE METHODOLOGY FOR ESTIMATING INTERNAL CAPTURE. VALIDATION OF THE PROPOSED METHOD WAS BASED ON COMPARISON OF ESTIMATED EXTERNAL VEHICLE TRIP GENERATION WITH OBSERVED CORDON COUNTS.

BY BRIAN S. BOCHNER, P.E., PTOE, PTP, KEVIN G. HOOPER, P.E., AND BENJAMIN R. SPERRY, E.I.

INTRODUCTION

One of the key components of traffic impact analyses (TIA) for proposed land developments is the estimation of the number of trips that are expected to be generated by the site. For developments composed of a single land use, trip generation can be estimated using the Institute of Transportation Engineers' (ITE's) *Trip Generation*, 8th Edition or similar reference, based on certain characteristics of the proposed site.[1] For multi-use land developments, the process of estimating trip generation is slightly more complex due to the internalization of some trips between on-site land uses. ITE defines a "multi-use development" as a single real-estate project that consists of two or more ITE land-use classifications between which trips can be made without using the off-site road system.[2] Internal trip capture is used as a measure of the extent to which trips made at multi-use developments are internalized with both origin and destination within the development. Accurate estimation of internal trip capture for mixed-use developments benefits all parties involved with the development review process. In recent years, a new form of multi-use development, the mixed-use development (MXD), has emerged as a popular development style in both suburban greenfield and urban infill areas. MXD sites are developed in conformance with a single master plan, including fully integrated on-site land uses and deliberate layout and design of buildings and streets.

RESEARCH PROBLEM

Despite the importance of accurately estimating internal trip capture for MXDs, very little research exists on the subject. Prior to 2005, the most robust research effort examining internal trip capture was two studies from the early 1990s funded by the Florida (USA) Department of Transportation (FDOT). These studies identified internal capture using intercept surveys of residents, employees, and visitors at six developments in south Florida. The FDOT studies would ultimately form the basis of the ITE-recommended procedure for estimating internal trip capture at multi-use developments, which is described in Chapter 7 of the ITE *Trip Generation Handbook*.[3] For more details of the FDOT studies, the reader is referred to summaries located in Appendix C of the *Trip Generation Handbook*.[4]

While the ITE-recommended internal trip capture estimation procedure is the most frequently used method in practice, there are several limitations to the method. For example, the method includes data for only three land uses: residential, office, and retail. Although the data allow for estimation of internal capture during the p.m. peak hour, it does not allow data for the a.m. peak hour, so the current method does not support completion of traffic impact analyses for both street peak hours. Finally, there are no provisions in the method to account for certain features of the MXD that may have a profound impact on trip internalization, including the proximity of on-site land uses or the emphasis on pedestrian-oriented design.

Recognizing these limitations, the National Cooperative Highway Research Program (NCHRP) initiated Project 8-51 in 2005 to improve the methodology used to estimate internal trip capture witbin mixed-use developments.[5] Among the project outputs were the following:

- A data collection framework and methodology to estimate internal trip capture at an existing MXD site; and
- A defensible improved methodology for estimating internal trip capture to determine appropriate reductions below single-use trip generation estimates.

The improved estimation method was developed using existing survey data from prior studies plus surveys of three additional MXDs conducted in the NCHRP project. The estimation method is based on the existing ITE procedure but expands it to cover both a.m. and p.m. peak periods, six primary land uses most frequently found at MXDs, and proximity of interacting land uses.

This article describes the key tasks and findings of NCHRP Project 8-51. For more details on the study methodology and analysis findings, readers are encouraged to read the project report, which is available from the Transportation Research Board.[6]

Table 1. Characteristics of NCHRP 8-51 study MXD sites.

Characteristic[1]	Mockingbird Station	Atlantic Station	Legacy Town Center
Location	Dallas, Texas	Atlanta, Georgia	Plano, Texas
MXD Style	Urban Infill Transit-Oriented Development	Urban Infill	Suburban
Site Size (Acres)	7	120	75
Maximum Walk Distance (Feet)	700	1,600[2]	2,000
Residential (DU)	191	798	1,360
Retail (GLA)	156,100	434,500	196,300
Office (GSF)	114,600	550,600	310,800
Restaurant (GLA)	28,900	64,600	69,300
Hotel (Rooms)	*No On-Site Hotel*	101	404
Cinema (GLA/Screens)	31,500/8	87,000/16	27,100/5
Parking Spaces	1,528	7,300	6,070
Transit Service	Adjacent DART Light Rail Station	Heavy Rail via Shuttle; City Bus	Limited Suburban Bus/Hotel Shuttle

Source: Cited Reference 5

[1]DU = dwelling units; GLA = gross leasable floor area; GSF = gross square feet of floor area.

[2]Reflects walking distance across main town center district.

DATA COLLECTION METHODOLOGY

The first key task of the NCHRP 8-51 project was development of a methodology and procedural instructions for the data collection process as well as for selection of MXD sites that would be used for data collection. Preliminary steps of the data collection process included the following:

- Selection of representative MXDs that could be sufficiently isolated to collect data to determine both total external trip generation by mode and to determine internal trip making by interviewing the trip makers to determine trip origins, destinations, modes of travel, and related information; and
- Acquisition of owner/manager permissions to conduct the surveys and to obtain data describing each MXD and its land uses, including numbers of occupied development units.

Data were collected at the subject MXD sites during typical weekdays (Tuesday–Thursday) during the morning peak period (6:30 to 10:00 a.m.) and the weekday afternoon peak period (4:00 to 7:00 p.m.). Data obtained at each site included the following:

- Origin-destination intercept interviews of persons exiting land uses within the MXD; where not all doors could be covered due to limitations of resources, representative establishments were selected;
- Door counts of the number of people entering and exiting each MXD establishment at which interviews were conducted;
- Cordon line counts, including numbers of persons and vehicles by mode and direction; and
- Where specifics of a development required, additional interviews or counts to cover the characteristics of that development (for example, interviews of transit patrons accessing a rail station via an on-site access point).

Counts were used to factor interview data to represent the full population of travelers at the MXD site. Observed cordon counts were checked to ensure that total factored interview data compared logically with counts.

STUDY SITES

Three MXDs in the United States were selected for study in the NCHRP 8-51 project: Mockingbird Station in Dallas, Texas; Atlantic Station in Atlanta, Georgia; and Legacy Town Center in Plano, Texas. Each site contained at least five of the six land uses most frequently found in major MXDs: office, retail, restaurant, residential, cinema, and hotel. All were developed from a single master plan with fully integrated component land uses. Table 1 reports selected characteristics of the three MXD sites surveyed in NCHRP 8-51, including the quantities of each of the six land uses being examined for internal trip capture relationships.

Office properties at the study sites were generally mid-rise office towers, although some offices were vertically integrated above ground-level retail space or restaurants. Retail, restaurant, and cinema land uses were generally grouped in a "town center" configuration. Retail properties ranged from specialty stores to convenience/service shops. The retail component of the study sites included a large department store and grocery store at Atlantic Station and a large furniture store at Legacy Town Center. On-site restaurants ranged from high-turnover fast food to exclusive. Residential units at the study sites consisted of free-standing multi-family apartment buildings or owner-occupied townhomes. As with the offices, vertical integration between on-site residences and ground-level retail or restaurant was present at the study sites.

In general, the walkability of the study sites was excellent; maximum walking distances across the study sites were no more than one-half mile, although a majority of the walking activity occurred at much shorter distances. Parking (on-street and garage/surface lots) at the study sites was generally available to visitors but also included reserved parking spaces for on-site residents and employees. Observed vehicle occupancy rates were higher during

Table 2. Proposed revision to ITE *Trip Generation Handbook* Table 7.1—unconstrained internal trip capture rates for trip origins within a mixed-use development.

Land-Use Pairs		Weekday Peak Hours a.m.	Weekday Peak Hours p.m.
From OFFICE	To Office		
	To Retail	28%	*20%*
	To Restaurant	63%	*4%*
	To Cinema/Entertainment		0%
	To Residential	1%	*2%*
	To Hotel	0%	0%
From RETAIL	To Office	29%	2%
	To Retail		
	To Restaurant	13%	29%
	To Cinema/Entertainment		4%
	To Residential	14%	*26%*
	To Hotel	0%	5%
From RESTAURANT	To Office	31%	3%
	To Retail	14%	41%
	To Restaurant		
	To Cinema/Entertainment		8%
	To Residential	4%	*18%*
	To Hotel	3%	7%
From CINEMA/ENTERTAINMENT	To Office		2%
	To Retail		21%
	To Restaurant		31%
	To Cinema/Entertainment		
	To Residential		*8%*
	To Hotel		2%
From RESIDENTIAL	To Office	2%	4%
	To Retail	1%	*42%*
	To Restaurant	20%	*21%*
	To Cinema/Entertainment		0%
	To Residential		
	To Hotel	0%	3%
From HOTEL	To Office	75%	0%
	To Retail	14%	16%
	To Restaurant	9%	68%
	To Cinema/Entertainment		0%
	To Residential	0%	*2%*
	To Hotel		

Note: Blank cells indicate intra-land-use trips or locations not open in a.m. peak hours.
Italicized values indicate internal capture rates eligible for proximity adjustment.
Source: Cited Reference 5

the p.m., ranging between 1.07 and 1.12 in the a.m. and between 1.09 and 1.40 in the p.m. peak hour. Transit service at the study sites was as follows:

- Mockingbird Station: light-rail station adjacent to site; transit mode share approximately 15 percent of external trips;
- Atlantic Station: frequent circulator shuttle connecting site and nearby heavy rail station; transit mode share approximately 5 percent; and
- Legacy Town Center: infrequent suburban bus route adjacent to site plus private hotel shuttle to nearby business park; negligible transit mode share (less than 1 percent).

INTERNAL TRIP CAPTURE ANALYSIS

Using the factored interview data, researchers developed origin-destination (O-D) matrices that reported travel between each of the six land uses being examined as well as trips external to the site. Researchers excluded trips between the same land uses because ITE trip generation estimates already reflect trips within the same land use on the same site. From the O-D matrices, researchers computed internal trip capture percentages for origin and destination ends at land uses in the MXD. Researchers reviewed the internal trip capture rates for the three NCHRP study sites (and three FDOT sites where applicable) for logic related to nearby competing opportunities, proximity, connectivity, and specifics of the establishments to ensure the results were consistent with expectations. Researchers identified both consistencies and differences between the internal capture percentages for the same land-use pairs and directions. These consistencies and differences result from differences in proximity of the interacting land uses as well as the balance of development units. Other factors may also affect these percentages.

For each land-use pair and peak period, researchers identified up to six unique internal trip capture percentages, one for each of the three NCHRP sites plus up to three from the FDOT sites (where applicable). The highest percentages for each land-use pair examined represent the highest directional interaction shown by the survey data. The researchers considered these percentages to be unconstrained (or at least the least-constrained found): That is, they result from the most favorable balance of land-use development units as well as relatively close proximity. Tables 2 and 3 report the unconstrained internal trip capture percentages for trip origins and destinations within an MXD identified in this research. The data in Tables 2 and 3 are reported in a similar format as Tables 7.1 and 7.2 in the *Trip Generation Handbook.*[7]

IMPROVED ESTIMATION METHODOLOGY

The second key task of the NCHRP 8-51 project was to develop an improved methodology for estimating internal trip capture for proposed MXD sites. Figure 1 shows a flowchart of the improved methodology, which is closely patterned after and builds upon the ITE-recommended methodology found in the *Trip Generation Handbook*.[8] The ITE-recommended method applies internal trip capture percentages to estimated vehicle trips generated by each on-site land use (for which data are available) and distributes these trips among the other land uses at the site. In order to reflect the balance between quantities of each land use, directional trips between each land-use pair are constrained so the internal capture estimate includes only those trips for which there are enough of both origin and destination land uses to support trips. External travel is then computed by subtracting the estimated internal trips between on-site land uses from the single-site trip generation estimates. The NCHRP 8-51 project improves upon the ITE methodology by adding the following elements:

- Internal trip capture rates for the weekday a.m. peak hour;
- Internal trip capture rates for restaurant, cinema, and hotel land uses; and
- Adjustments to certain unconstrained internal trip capture rates (italicized values in Tables 2 and 3) to account for the effects of proximity (convenient walking distance) between interacting land uses to represent both compactness and design.

In addition to these improvements, researchers developed a convenient spreadsheet-based estimator tool to apply the methodology in practice. As shown in Figure 1, the improved method uses the following inputs:

- User-estimated a.m. and p.m. inbound and outbound vehicle trip generation for the six land uses as single-use, free-standing sites;
- Mode split for MXD trips to/from each land use—percent by auto, transit, and nonmotorized travel modes (walk/bicycle);
- Average vehicle occupancy to/from each land use; and
- Average walking distance (feet) between land-use pairs.

From these inputs, the following outputs are produced:

- A.m. and p.m. peak hour internal person trips by land use in origin-destination form;
- A.m. and p.m. peak hour percent internal capture (person trips); and
- A.m. and p.m. peak hour inbound,

Table 3. Proposed Revision to ITE *Trip Generation Handbook* Table 7.2—unconstrained internal trip capture rates for trip destinations within a mixed-use development.

Land-Use Pairs		Weekday Peak Hours a.m.	Weekday Peak Hours p.m.
To OFFICE	From Office		
	From Retail	32%	8%
	From Restaurant	23%	2%
	From Cinema/Entertainment		1%
	From Residential	0%	4%
	From Hotel	0%	0%
To RETAIL	From Office	4%	*31%*
	From Retail		
	From Restaurant	50%	29%
	From Cinema/Entertainment		26%
	From Residential	2%	*46%*
	From Hotel	0%	17%
To RESTAURANT	From Office	14%	*30%*
	From Retail	8%	50%
	From Restaurant		
	From Cinema/Entertainment		32%
	From Residential	5%	*16%*
	From Hotel	4%	71%
To CINEMA/ENTERTAINMENT	From Office		6%
	From Retail		4%
	From Restaurant		3%
	From Cinema/Entertainment		
	From Residential		4%
	From Hotel		1%
To RESIDENTIAL	From Office	3%	57%
	From Retail	17%	10%
	From Restaurant	20%	14%
	From Cinema/Entertainment		0%
	From Residential		
	From Hotel	0%	12%
To HOTEL	From Office	3%	0%
	From Retail	4%	2%
	From Restaurant	6%	5%
	From Cinema/Entertainment		0%
	From Residential	0%	0%
	From Hotel		

Note: Blank cells indicate intra-land-use trips or locations not open in a.m. peak hours.
Italicized values indicate internal capture rates eligible for proximity adjustment.
Source: Cited Reference 5

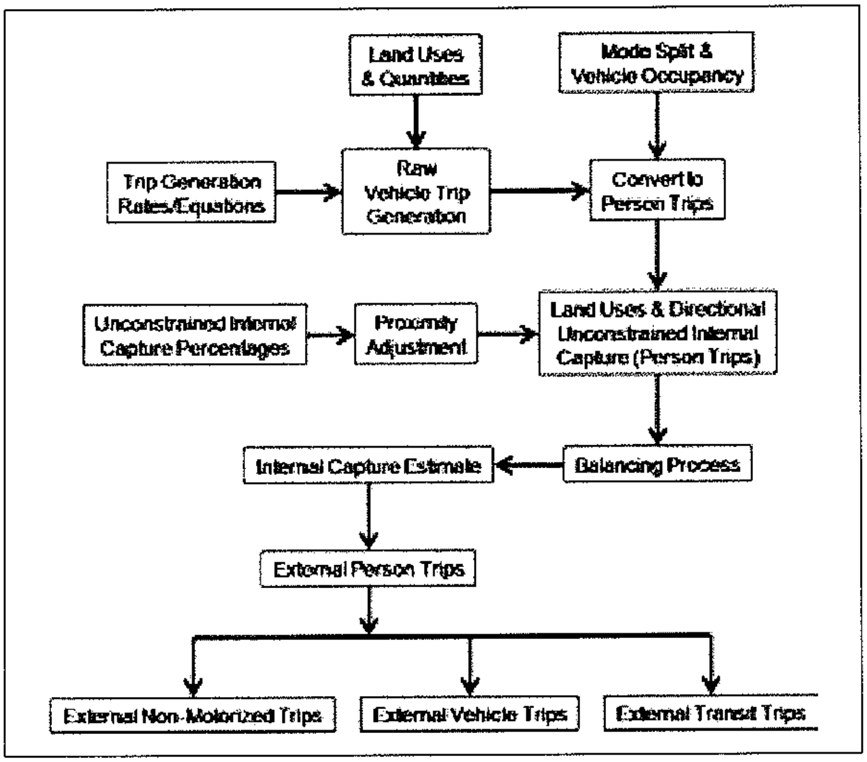

Figure 1. Generalized NCHRP 8-51 internal capture estimation method.

outbound, and total external person trips (trips to and from the development being analyzed) by mode (vehicles, transit, or nonmotorized).

VALIDATION

The validity of the improved method was tested by comparing the estimated trip generation from the method with observed cordon count data from MXDs. Cordon count data from the three NCHRP sites, the three FDOT sites, and one additional MXD site were used in the validation. For the seven MXD sites included in the validation, the proposed method was found to reduce estimation error by about half as compared to the existing ITE method and by three-fourths compared to raw ITE-based trip generation estimates.

RECOMMENDED MODIFICATIONS TO EXISTING ITE PROCEDURES

The recommended estimation method developed by NCHRP Project 8-51 does build on the current ITE internal trip capture procedures contained in the second edition of the *Trip Generation Handbook*. Incorporation of the NCHRP project's recommendations could be accomplished by doing the following:

- Expand Tables 7.1 and 7.2 of the *Trip Generation Handbook* to include all six land uses examined in the NCHRP project;
- Add a proximity adjustment to the unconstrained internal capture rates before estimating directional internal trips by land use and the balancing process; and
- Modify the data collection procedures to include those recommended in this project.

It should be noted that the findings of NCHRP Project 8-51 are being published for informational purposes; this research has yet not been advanced through ITE's process for development of recommended practices and therefore should not yet be considered an ITE-recommended methodology. This research will be considered for inclusion in an update to the ITE *Trip Generation Handbook*.

SUGGESTED FUTURE RESEARCH

The NCHRP 8-51 project made progress to improve estimation of internal capture for MXDs. However, the database is still sparse, and much that is thought to be logical about MXD travel characteristics is still unproven and largely untested. The results presented in this paper are based on surveys of six mixed-use developments, and validation was limited to seven such developments. Caution should be exercised in application of this methodology. For example, it cannot be concluded that the methodology will be appropriate for mixed-use developments that differ substantially from those surveyed.

Two of the research efforts recommended by the research team are as follows:

- Collect more data at MXDs; the researchers think data are needed from at least six more sites that have five to six land uses; and
- Independent of the additional data collection, test the applicability of the proposed methodology using data from MXDs of different sizes, character, and land-use components; use validation tests similar to those used in NCHRP 8-51. To do so, the only data needed are a complete directional cordon count for morning and afternoon peak hours plus development data and a good site plan from which to estimate proximities.

Practitioners, researchers, and other interested parties are encouraged to collect and contribute additional data using the data collection procedures described in research report. Those data could be used to further enhance the accuracy of the proposed methodology and/or expand the number of land-use classifications covered by the methodology. New data should be forwarded to the Institute of Transportation Engineers. ■

References

1. *Trip Generation*, 8th Edition, Institute of Transportation Engineers, Washington, DC, December 2008.
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4. Ibid.
5. Bochner, Brian, Kevin Hooper, and Benjamin Sperry, *Improved Estimation Method for Estimating Internal Capture for Mixed-Use Developments*, draft final report, NCHRP Project 8-51, Texas Transportation Institute, November 2009.
6. Ibid.
7. ITE, 2004.
8. Ibid.

BRIAN S. BOCHNER, *P.E., PTOE, PTP, senior research engineer, Texas Transportation Institute, College Station, Texas, USA, served as principal investigator for NCHRP Project 8-51,* Enhancing Internal Trip Capture Estimation for Mixed-Use Developments. *He has served on ITE's Trip Generation Advisory Committee for more than two decades and was a member of the committee that wrote the* Trip Generation Handbook *as well as several other publications. Mr. Bochner has also performed traffic impact and site access studies for single- and mixed-use developments. He is a fellow of ITE.*

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Table 411-7 TND Trip Reductions for Traffic Studies

% Residential Equivalent Units	% Commercial Equivalent Units	% Trips Reduced
76-85%	15-24%	7.5%
66-75%	25-34%	15%
56-65%	35-44%	22.5%
46-55%	45-54%	30%
36-45%	55-64%	22.5%
26-35%	65-74%	15%
16-25%	75-84%	7.5%

For purposes of computing the percentage established above, one dwelling unit or 800 square feet of non-residential space equals one (1) equivalent unit.

Sec. 412 – 416 [RESERVED]

Pages 169 – 170 Reserved

Background and Overview of
Validation of Potential Smart Growth Trip Generation Methodologies

for the California Smart Growth Trip Generation Rate (SGTGR)
Methodology and Validation Subcommittees
(prepared by Rick Lee – July 28, 2010)

Overview

As we have seen, most traffic studies conducted in California base their estimates of development impacts on the Institute of Transportation Engineers' *Trip Generation* manual, or on the San Diego Association of Governments' (SANDAG) similarly constructed *Traffic Generators*. The ITE *Trip Generation Handbook* (a companion volume to *Trip Generation*), describes procedures for adjusting the manual's vehicle trip generation rates to account for the effects of mixed use development, based on limited data. While neither the ITE manual nor the handbook present validation data of their reported rates and equations, they do describe procedures for collecting traffic generation data at developed sites in order to validate, refine or replace the published rates.

The methods that are being studied (or possibly developed) by the SGTGR effort differ substantially from conventional ITE trip-generation methods. In order to provide practitioners with substantiation of the validity of the methods and their estimates, the study team plans to compare results of each potential method against empirical traffic data gathered at developed smart growth sites within California. Site identification is underway, and data collection will hopefully begin under separate contract by early 2011 (if the recession subsides sufficiently to allow data collection).

The study team is also using traffic count data already available from the EPA and SANDAG studies for 10 of the California sites that was used to evaluate these agencies' versions of the MXD method. These sites include one in Northern California (in Sacramento), three in Orange County, and six in San Diego County. A limitation of this existing set of data is that it includes vehicle trips only; also it does not report peak hours in some cases.

The SGTGR evaluation process (whether using pre-existing or new data) will be to compare the empirical data collected at each site to estimates produced by the various "candidate" methodologies for that site. Key steps the UCD project team anticipates conducting in this approach are as follows:

1. Calculate each site's ITE gross trip generation based on: a) the reported ITE average trip rates, and b) the reported ITE trip generation equations. (i.e., summing the trips from the individual land use categories). Also apply the ITE Handbook's multi-use internal capture rates equation for mixed-use sites.
2. Identify each candidate method's needed input variables on a consistent basis for each validation site.
3. For all candidate methods, compute the estimate of external vehicle trip generation, and compare to both the gross and multi-use ITE trip generation estimates to the cordon traffic counts for the site.
4. The study team and Methodology Subcommittee may consider possible refinements to the candidate methodologies to improve the validation against the empirical traffic count data, or may decide to develop a new methodology if no existing methodologies are sufficiently accurate.

This approach represents a substantial advance over the approach taken in the existing ITE trip generation guidelines. The methodologies tested in this study will be validated via comparison of method estimates against the best available empirical field data gathered from a range of case study sites. When the validation process is completed, it will help select or devise a new analysis technique for traffic engineers that can more accurately and defensibly quantify impacts and size infrastructure for smart growth development projects.

The remainder of this document summarizes the "validation" approach taken by three key prior tool development projects: the EPA MXD spreadsheet, the SANDAG MXD spreadsheet, and the finalized NCHRP 8-51 report *Enhancing Internal Trip Capture Estimation for Mixed-Use Developments*.

I. EPA MXD Spreadsheet Validation Approach (based on the draft *Research-Based National Method of Estimating Trip Reduction Attributable to Multi-Use Development Research Report,* US EPA, n.d.)

The EPA MXD spreadsheet model was applied to 16 multi-use developments (MXDs) in various parts of the US for which counts of external vehicle trips were available AND which were not one of the 239 mixed-use sites used to develop the underlying equations. Seven of these sites are in Florida: six of these Florida sites are described in Appendix C of the ITE Trip Generation *Handbook*, and the seventh is the new neo-traditional town of Celebration, Florida. Six of the remaining sites are located in northern and southern California, and the remaining three are in Texas and Georgia; cordon counts at these last three sites were originally conducted for the NCHRP 8-51 study (see below).

Overall, the EPA validation sites are comprised of mixed-use developments and areas ranging in size from approximately 5 acres to over 1,000 acres. The sites represent a wide range of densities, land use mixes, and development scales. Populations of the validation MXDs range from zero (Crocker Center in Boca Raton, FL, containing commercial and office uses only) to nearly 17,000 (the entire town of Moraga, CA). Employment levels range from near zero (in The Villages in Irvine, CA) to more than 6,000 (Jamboree Center in Irvine, CA). Some sites are well served by transit, including one built around a rail station. Others are located in suburban areas and are poorly served by transit. The diversity of the comparison sites suggests that a tool that matches all would help establish its validity for analyzing a wide variety of projects.

Data were collected for all model variables for each of the 16 sites. The variables "Employment within one-mile" and "Employment within 30 minutes by transit" were estimated from regional travel models, and verified from aerial photos. Some of the site data was confirmed from development websites. Trip purpose percentages were estimated based data presented in NCHRP Report 365 for the combination of land uses contained within each MXD.

These validation tests produced two types of performance measures: root mean squared error (RMSE) and pseudo R-squared. RMSE is a measure of the percentage by which the trip generation estimates produced by the method deviate from the actual trip generation counted at each of the study sites. The lower the RMSE deviation, the more accurate is the prediction method. R-squared is a measure of how well the prediction method accounts for the degree of variation in trip generation from one site to another, An R-squared of 0.5 indicates the method explains 50 percent of the variation among cases and a value of 1.0 would imply a perfect ability to capture the variation in trips from one site to another.

Among the validation sites, use of the EPA Mixed-Use Method produced a significantly better root mean squared error (RMSE) and pseudo-R squared than traditional methods when comparing estimated to observed external vehicle trips. Estimates from the ITE *Trip Generation* manual had an RMSE of 40 percent and pseudo-R squared of 0.58, and modified estimates using ITE's current trip internalization techniques had an RMSE of 32 percent and pseudo-R squared of 0.73. Estimates produced by the Mixed-Use Method had an RMSE of only 26 percent and pseudo-R squared of 0.82. This means that the Mixed-Use Method explains roughly 82 percent of the variation in trip generation among the 16 sites, with the remaining 18 percent attributable to variables not included in the method.

II. VALIDATION of the EPA MXD METHOD for SAN DIEGO SITES (Based on *Trip Generation for Smart Growth, Final Report*, SANDAG, 2010)

To evaluate the EPA MXD method (referred to as the Mixed-Use Method in this section) for use in the San Diego region, a series of tests were performed comparing the method's estimations with actual traffic count data from a number of sites within the region. This included comparisons at both large designated Smart Growth Opportunity Areas (SGOAs, defined below) and six smaller mixed-use and TOD sites.

Smart Growth Opportunity Areas. The SANDAG Smart Growth Concept Map identifies a list of SGOAs classified into one of seven place types (Metropolitan Center, Urban Center, Town Center, Community Center, Rural Village, Mixed-Use Transit Corridor, and Special Use Center). SANDAG identified a list of 57 existing SGOAs to be studied in this analysis. These 57 SGOAs were chosen by virtue of having residential and employment densities on the ground that currently meet the prescribed thresholds

Analysis of SGOAs. For most of the SGOAs, obtaining traffic counts entering and exiting the areas was not feasible due to the size of the SGOAs and the inability to filter out through trips. An alternative method to cordon counts was therefore identified.

Fehr & Peers (consultant to SANDAG) used available data from the *SANDAG Regional Household Travel Behavior Survey* (conducted mainly in 2006) to collect observed trip reduction percentages, which could be compared to the Mixed-Use Method's predicted trip reduction percentages. SANDAG staff provided Fehr & Peers with a data set of "flags" identifying which trips from the survey began and/or ended in one of the SGOAs. The trip data also included travel modes and travel party sizes. From this information, the total number of origins, destinations, and internalized trips (trips that begin and end in the same SGOA) by auto, walk, bicycle, and transit modes was computed for each SGOA. This was translated into observed values of probability of a trip being Internal, or external Walk/Bike, or Transit, the three key outputs of the Mixed-Use Method.

The analysis was performed for each of the twenty SGOAs that had at least 100 trips recorded in the survey. A cutoff of 100 trip records was chosen because in general, a sample size of at least 30 to 40 for each probability being estimated (i.e., the probability of a trip being either 1) internal; 2) external Walk/Bike; or 3) external Transit) is necessary for meaningful sample probabilities that are close to their true values.

Vehicle trip reduction at the SGOAs studied averaged 24 percent relative to raw trip calculations using SANDAG trip rates, ranging from as high as 47 percent in downtown San Diego to 32 percent in North Park/City Heights, and as low as 5 percent in Mira Mesa. Overall, the Mixed-Use Method proved to be a conservative predictor of trip reduction, underestimating trip reduction by about 10 percent on average, but the estimated and observed trip reductions are highly correlated.

Smaller Mixed-Use/TOD Sites. Six additional smaller mixed-use/TOD sites were identified for comparing the Mixed-Use Method estimates to actual counts of vehicles entering and exiting each site. The sites were selected from a list of developments representing typical smart development mixes and density in the region. The sites are all cohesive developments rather than historic neighborhood, because so as to be more representative of new development. Sites were also selected to be as consistent as possible with ITE data collection recommendations, e.g. the ability to identify and count all vehicular access/egress points.

The six sites selected and counted were as follows:

1. Station Village at Rio Vista Trolley Station, bounded by Camino Del Este, Rio San Diego Drive, Qualcomm Way, and the trolley tracks (residential and retail; trolley station and local bus)
2. La Mesa Village Plaza, bounded by La Mesa Boulevard, Acacia Avenue, Orange Avenue, and the train tracks (residential, retail, and office; trolley station)
3. The Uptown Center in the Hillcrest neighborhood, bound by University Avenue, Cleveland Avenue, Richmond Street, Washington Street, and SR-163 (residential & retail; high frequency local bus)
4. The Village at Morena Linda Vista Trolley Station, bound by Morena Boulevard, Linda Vista Road, Napa Street, and the train tracks (residential and retail; trolley station)
5. Hazard Center, bound by SR-163, Friars Road, Frazee Road, and Hazard Center Drive (retail and office; trolley station)
6. Heritage Town Center at Otay Ranch in Chula Vista, bound by Santa Rita Street, Palomar Street, Santa Andrea Street, and the southern end of the parking lot, not including the houses on Fieldbrook Street (residential, retail, and medical office).

A set of maps were created illustrating the sites' locations and driveways where traffic counts were made. Continuous 24-hour traffic counts were conducted at the six small mixed-use/TOD sites on typical midweek weekdays: Tuesday, Wednesday, or Thursday. Counts were conducted in October of 2008 for Otay Ranch, and in May and early June of 2009 (prior to the end of the K-12 school year) for all other sites.

Analysis: Small Mixed-Use/TOD Sites With Counts. For the small mixed-use/TOD sites, preliminary estimates of site trip generation were calculated from *San Diego Traffic Generators* trip rates and site land uses. These estimates of raw trips use suburban trip generation rates for single use sites and do not consider the effects of mixed-use development or transit access. The Mixed-Use Method was applied to each site and the trip reduction percentages were applied to the raw trips to obtain Mixed-Use Method net trips. SANDAG staff provided site land uses and values for most of the Mixed-Use Method input variables.

The results of the small sites analysis suggest that the Mixed-Use Method is an excellent predictor of external vehicle trips generated by smart growth development, tending to be slightly conservative, but without overestimating smart growth trips to the same degree as conventional trip generation methods. At all six sites, the Mixed-Use Method results in an estimation of external vehicle trips that is below the levels of estimated trip generation using raw trips alone and at or above the level of trips that was determined through actual counts. On average, the *San Diego Traffic Generators* trip generation rates for suburban development overestimate traffic from the six sites by 29 percent, while the Mixed-Use Method reduces the average overestimate to 9 percent.

In the study of the six small SANDAG sites some of the input variables were determined by estimation methods, as follows:

- Due to confidentiality restrictions associated with California Employment Development Department data, employment levels for some sites were not always reflective of current land uses in the SANDAG databases; in those cases, they were determined from the building areas and jobs per 1,000 square foot conversion ratios.
- VRPA Technologies performed an independent set of land use data checks, collecting data from traffic studies wherever possible, and estimated building occupancy. Those estimates were taken into account in the calculation of raw trips.
- Vehicle ownership per capita was calculated from 2000 Census data using the census block group(s) that most closely matched the sites' locations. SANDAG staff estimated employment within 30 minutes by transit using their regional travel demand model.

III. VALIDATION of the NCHRP 8-51 Estimation Procedure (Based on pending NCHRP 8-51 Final Report Appendix F)

The NCHRP 8-51 estimation procedure was applied to seven different developments for which at least land use information, peak hour cordon counts, and proximity information were available. Four of these developments provided data for this study; the other three did not. The validation test was to see how well the estimation procedure could begin with ITE trip generation data and reproduce the external vehicular cordon volumes. Five of the developments had directional cordon traffic volumes available for both peaks. These developments included:

- Mockingbird Station (Dallas, TX)
- Legacy Town Center (Plano, TX)
- Atlantic Station (Atlanta, GA)
- Crocker Center (independent site, Boca Raton, Florida), and
- Mizner Center (independent site, Boca Raton, Florida).

Two developments had only non-directional PM peak period counts available. They were:

- Boca del Mar (FL) and
- Southern Village (independent site, Chapel Hill, North Carolina).

The validation test compared four different estimation methods to determine which method produced the results closest to the cordon counts:

- the estimator described in the NCHRP 8-51 report,
- the estimator, but without a proximity adjustment,
- the existing ITE multi-use estimation method, and
- unadjusted ITE trip generation.

Development data and approximations of surveyed mode split and vehicle occupancies were input to the estimation procedure.

Figures F-1 through F-4 (that follow) show the comparisons of vehicle trips for both AM and PM peak periods and both inbound and outbound directions.

In Figure F-1, it is evident that for the AM peak hour inbound vehicle trips, the NCHRP estimation methods—both with and without the proximity adjustment—produce the best results for three of the five developments; the current ITE multi-use method is closest for one site and slightly better than the NCHRP method for another site. Atlantic Station is more closely estimated by both unadjusted trip generation and the current ITE multi-use method. The current ITE multi-use method is better than raw trip generation, but the method developed in this project is even closer to the counts.

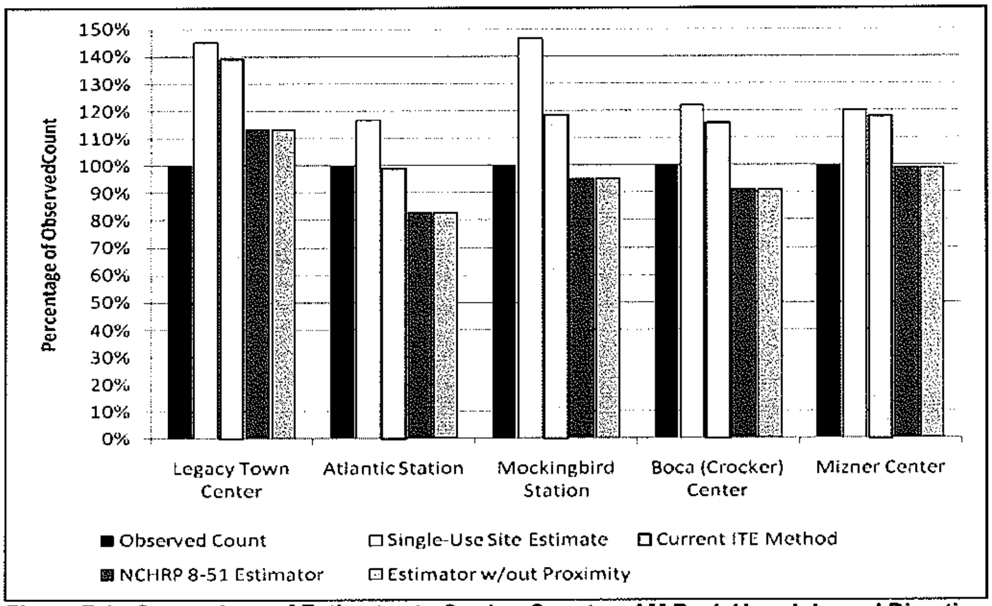

Figure F-1. Comparison of Estimates to Cordon Counts – AM Peak Hour Inbound Direction

Figure F-2 shows similar results for AM peak hour outbound vehicle trips with the recommended estimator (both with and without the proximity adjustment) producing the best results for four of the five developments. This time Mizner Center is better estimated by raw trip generation and the current ITE method. As with the previous comparison, the ITE multi-use method is an improvement on raw trip generation.

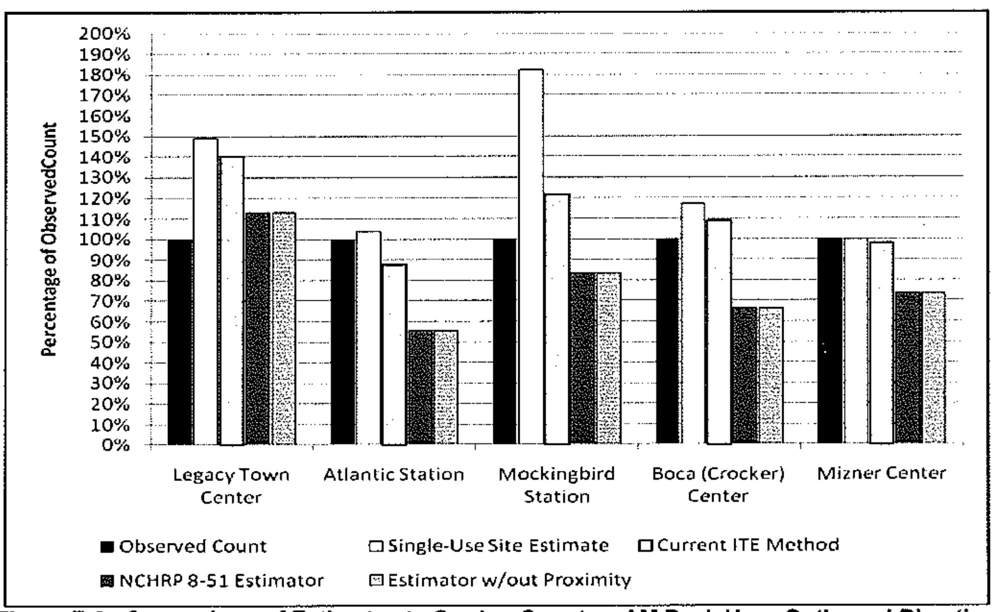

Figure F-2. Comparison of Estimates to Cordon Counts – AM Peak Hour Outbound Direction

The PM inbound comparison in Figure F-3 shows that the NCHRP method with the proximity adjustment produces the closest estimates for two sites, with the methods with and without proximity about equal for the two sites, and the raw ITE trip generation closest for one site. Again, Mizner Center was better estimated by another method (this time raw trip generation), but the other four are best estimated by the recommended method.

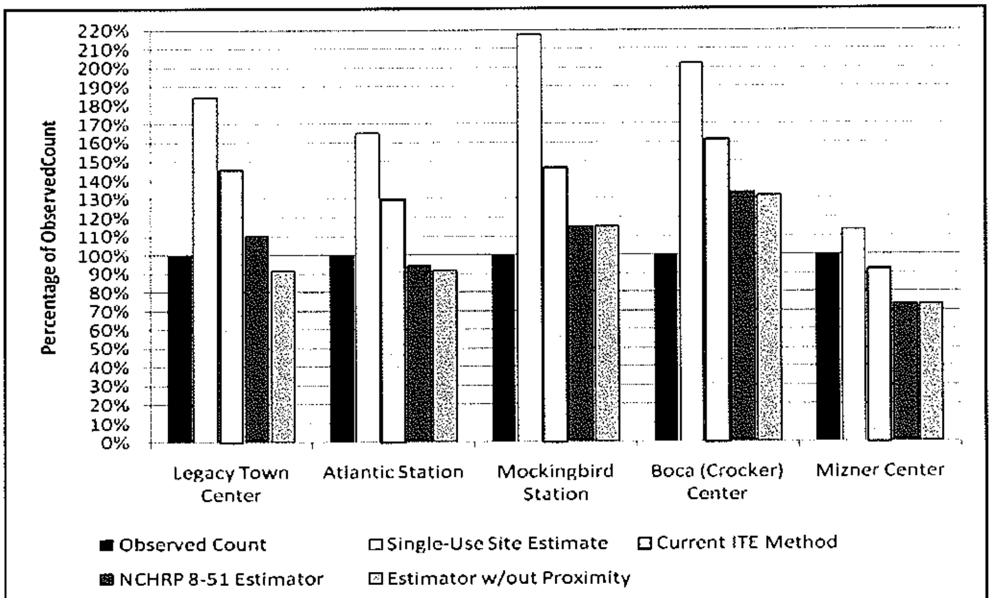

Figure F-3. Comparison of Estimates to Cordon Counts – PM Peak Hour Inbound Direction

Figure F-4 shows the comparison for PM peak hour inbound trips. As for the other time periods and directions, one or the other of the NCHRP 8-51 methods produces the closest estimates in four of the five cases. The methods with and without proximity adjustments are each best for one MXD while both yield approximately the same results for two MXDs. In this case, Boca Center is better estimated using the existing ITE method.

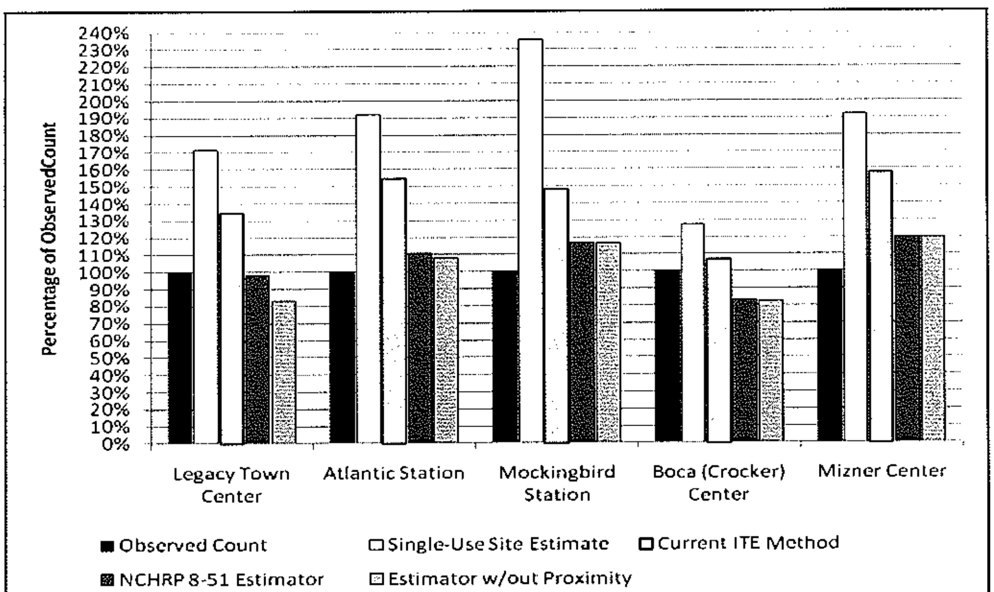

Figure F-4. Comparison of Estimates to Cordon Counts – PM Peak Hour Outbound Direction

In total, the recommended method—with or without the proximity adjustment—produces more reliable estimates for four of the five developments.

The results for the other two developments—Boca del Mar and Southern Village—show two different patterns. For Boca del Mar, both the existing ITE and recommended methods produce significantly low estimates, but are closer than the recommended method without proximity adjustments or the ITE method. The raw estimate is above the actual external count, but it and the ITE method are the closest of the estimates (about 4 percent closer than the recommended method with proximity adjustment). For Southern Village, the results are very different. The recommended method (both with and without proximity adjustments) produce estimates very close to the counts.

Table F-3 (note this is the original table number in the NCHRP 8-51 report) may quantify the degree of accuracy or error more clearly, recognizing that the statistics presented represent the sum of combined results. The average error shown is the simple sum of the percent deviations from the counts as derived in Table F-2. On average, as a group the estimates all exceed the counts (for example, the recommended method with proximity adjustment is an average of 4 percent). This is very misleading and not relevant for single developments, because overestimates and underestimates tend to cancel each other out. What may be of value in those percentages is that they could result in the sum total trip generation of several developments in an area. However, that is not what is being validated here.

More applicable is the absolute average error, which is the sum of the magnitudes of the errors averaged over the five developments. This shows more clearly what deviations—above or below actual—were found. Clearly, by examining the figures and Table F-3, it is easy to determine that the raw trip generation greatly overestimates external vehicle trip generation for the validation sites. The existing ITE method is a major improvement from raw trip generation. The recommended NCHRP 8-51 method brings the estimates significantly closer to actual. (Note that the difference between the actual and absolute value of the errors shows that there are both overestimates and underestimates occurring.)

Table F-3. Comparison of Error Statistics

Error Type	**Raw ITE Trip Generation**	**Existing ITE Multi-use Method**	**Recommended NCHRP 8-51 Method**		**Explanation**
			With Proximity Adjustment	No Proximity Adjustment	
Average error	+55%	+26%	-4%	−7%	Average error for sum of all sites
Absolute average error	55	28	17	17	Average magnitude of error per site
Standard deviation	68	34	20	19	Expect two-thirds of site estimates within this error range

The standard deviation shown in Table F-3 better represents the estimated probable magnitude of error that might occur using these estimation methods. Again, the relative magnitudes of error among the methods place them consistently in the same order.

It is clear that the NCHRP 8-51 recommended method provides more accurate estimates. Since the existing ITE multi-use adjustment method was developed from data from three of the six developments that were also used in this NCHRP project, the recommended method can only be viewed as being a further improvement.

The standard deviations for the recommended NCHRP 8-51 method, both with and without proximity adjustment, are about 20 percent of the actual external inbound and outbound volumes. This is less than the variations in the raw ITE nondirectional trip generation rates for the component land uses. For example, for the land uses listed in Table F-1, the standard deviations for their AM and PM peak hour trip generation rates are all in excess of 50 percent of the mean.

Not clear, however, is whether or not the proximity adjustment adds any current value. The validation results show no significant statistical benefit from the proximity adjustment. It has sufficient data only for the PM peak period (and less of that than would be desired). There is no AM proximity adjustment recommended at this time. On the other hand, the only examples for which the results were better without the proximity adjustment was when both variations of the new method were overestimating. In all cases the proximity adjustment either has no significant effect or renders the estimate more conservative (higher).

Conclusions Regarding Validation of the NCHRP 8-51 Method

The validation supports two principal findings:

1. The recommended method does produce noticeably more accurate results than either raw ITE trip generation estimates from the ITE *Trip Generation* report or the existing method described in the *Trip Generation Handbook*. This is true with or without the proximity adjustment.

2. The proximity adjustment, available at this time for the PM peak period, tends to make slightly more conservative estimates but overall does not, at this time, improve accuracy over a group of estimates. It can produce significant effects for larger developments.

5

TECHBRIEF

This techbrief is an archived publication and may contain dated technical, contact, and link information

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Synthesis of the Median U-Turn Intersection Treatment

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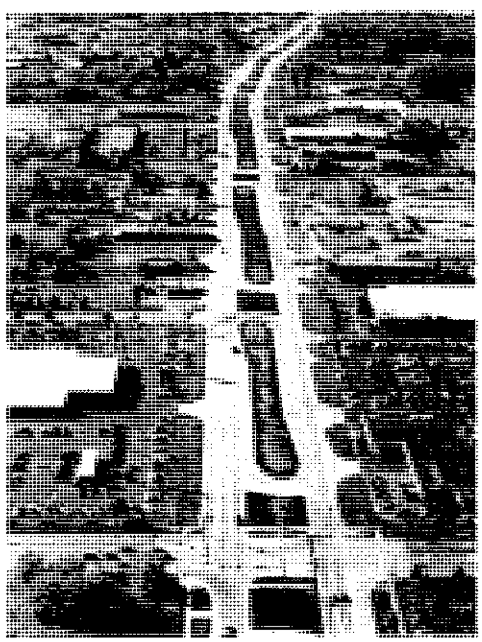
PDF Version (919 KB)

PDF files can be viewed with the Acrobat® Reader®

Objective

In the United States, congestion at intersections throughout urban and suburban areas continues to worsen. Crashes reported at intersections have continued to increase. One potential treatment to combat congestion and safety problems at intersections is the Median U-Turn Intersection Treatment (MUTIT), which has been used extensively in Michigan for many years and has been implemented successfully in Florida, Maryland, New Jersey, and Louisiana in recent years (Figure 1).

Figure 1. Example of MUTIT on Michigan corridors
(Source: AAA, Michigan).



The treatment involves the elimination of direct left turns at signal-controlled intersections from major and/or minor approaches. Drivers desiring to turn left from the major road onto an intersecting cross street must first travel through the at-grade, signal-controlled intersection and then execute a U turn at the median opening downstream of the intersection. These drivers then can turn right at the crossstreet. For drivers on the sidestreet desiring to turn left onto the major road, they must first turn right at the signal-controlled intersection and then execute a U turn at the downstream median opening and proceed back through the signalized intersection. The MUTIT can be implemented with and without signal control at the median openings on the major road.

This synthesis summarizes the advantages and disadvantages of the MUTIT compared to conventional, at-grade signal-controlled intersections with left turns permitted from all approaches. The synthesis presents design guidelines including the location and design of the median crossovers on the major roads. Many of the guidelines presented in the synthesis are from the Michigan Department of Transportation (MDOT), and address directional and bidirectional crossovers and widened areas called "loons" that facilitate the U-turn maneuver by larger vehicles and at roads with narrow medians. The synthesis also discusses application criteria for the MUTIT, and presents information on the capacity and crash experience at these intersections relative to traditional intersections. Special considerations related to signal phasing at the median openings and signal phasing at the at-grade intersection also are discussed. Empirical evidence supports the practice that the reduction in signal phases at intersections can have higher vehicle-processing capacity and better level-of-service. In terms of safety, past research has shown that the reported numbers of crashes at MUTITs are 20 to 50 percent lower than comparable conventional intersections. The major safety benefit is a reduction in the probability of head-on and angle crashes that typically have high percentages of injury severity. Although the MUTIT typically is considered a corridor-wide treatment, the concept has been used successfully at isolated intersections to improve traffic flow and enhance safety.

Introduction

The MUTIT eliminates left turns at intersections and allows the maneuver to be made via median crossovers beyond the intersection. Drivers desiring to turn left at the subject intersection from the major road first must travel through the intersection, execute a U turn at the median crossover, and then make a right turn at the crossroad. Drivers on the minor road desiring to make a left at the subject intersection first make a right turn at the intersection onto the major road, and then make a U turn at the median crossover, and subsequently go straight through the intersection. Figure 1 shows an illustrative photograph of the MUTIT implementation in Michigan, and figure 2 shows the schematic for a typical MUTIT. The MUTIT is typically a corridor treatment. However, the concept is used at isolated intersections to alleviate specific traffic operational and safety problems. Levinson et al. (1) recommended that the application of MUTIT along the corridor should not be mixed with other indirect left-turn treatments or conventional left-turn treatments, thereby meeting driver expectancy. Figure 3 shows the MUTIT movements corresponding to left turns at conventional at-grade intersections.

The MUTIT has been used widely in the State of Michigan. Several highways in Michigan, particularly in the Detroit Metropolitan area, were constructed with wide medians on wide rights-of-way. Many of these medians are 18.3 to 30.5 meters (m) (60 to 100 feet (ft)) wide and were built decades ago in semirural areas to separate opposing directions of traffic and to provide an adequate median width for landscaping and beautification. The wide rights-of-way were originally established for "super highways," as they were called in the 1920s. By the early 1960s, many of these highways had capacity problems, generally because of interlocking left turns at the conventional intersections. To address this capacity problem, MUTITs replaced conventional intersections on various corridors. Today, there are more than 684 kilometers (km) (425 miles (mi)) of "boulevards" with over 700 directional crossovers on the Michigan State highway system. Partial implementations or designs with similar concepts have appeared in Florida, Maryland, New Mexico, and New Orleans. Hummer and Reid (2) and Levinson et al. (1) compared the MUTITs to conventional intersections. Hummer and Reid recommended that agencies consider the median U-turn alternative for junctions on high design arterials where relatively high through volumes conflict with moderate or low left-turn volumes, regardless of the cross-street through volumes.

Some of the advantages cited include:

- Reduced delay and better progression for through traffic on the major arterial.
- Increased capacity at the main intersection.
- Fewer stops for through traffic, especially where there are STOP-controlled directional crossovers.
- Reduced risk to crossing pedestrians.
- Fewer and more separated conflict points.
- Two-phase signal control allows shorter cycle lengths, thereby permitting more flexibility in traffic signal progression.

Figure 2. Typical schematics of MUTIT.

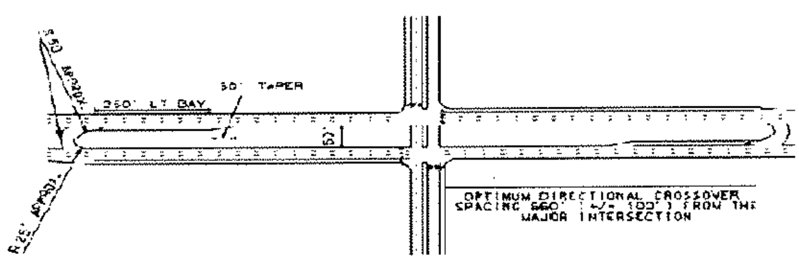
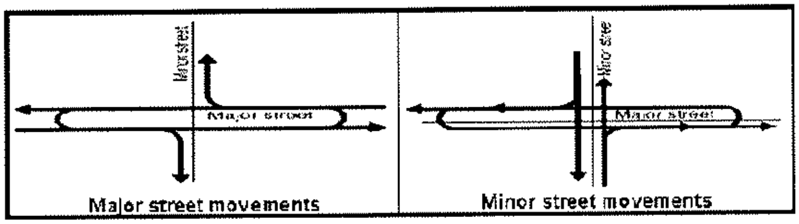


Figure 3. Vehicular movements at a MUTIT
(Source: Signalized Intersections Information Guide, FHWA-HRT-04-091, pg. 243).



Some disadvantages include:

- Possible driver confusion and disregard of left-turn prohibition at the main intersection.
- Possible increased delay, travel distances, and stops for left-turning traffic.
- Larger rights-of-way required for the arterial, although this potentially could be mitigated by the provision of loons (discussed later in this document) on roads with narrow medians.
- Higher operation and maintenance costs attributable to additional traffic signal control equipment if the directional crossovers are signalized.
- Longer minimum green times for cross-street phases or two-cycle pedestrian crossing.

MUTIT Design Guidelines

The 2004 AASHTO Green Book (3) recommends a distance of 122 to 183 m (400 to 600 ft) for the minimum spacing between the median crossover and the MUTIT intersection. The Michigan Department of Transportation (MDOT) recommends a distance of 201 m (660 ft) (+/- 30.5 m (100 ft)) for the median crossover from the MUTIT intersection. The distances recommended by the MDOT were established to accommodate drivers desiring to turn left from the crossroad. The longer distance facilitates the completion of the U-turn maneuver at the median crossover and subsequent right turn maneuver at the intersection of the major road and cross street for a 72 km/hour (h) (45 mi/h) posted speed limit on the major road. The selection of the spacing from the median crossover to the intersection is also a tradeoff between preventing spillback from the main intersection and the adverse impacts of additional travel for the left-turning vehicles. The Access Management Manual recommends an access spacing of 201 m (660 ft) on minor arterials and 402.3 m (1320 ft) on principal arterials between consecutive directional median openings on divided highways. Figures 4a, 4b, and 4c below show typical U-turn maneuvers. Table 1 gives the minimum median widths required for U turns from the major road as suggested by the MDOT.

Figure 4a. Left lane to inner lane maneuver.

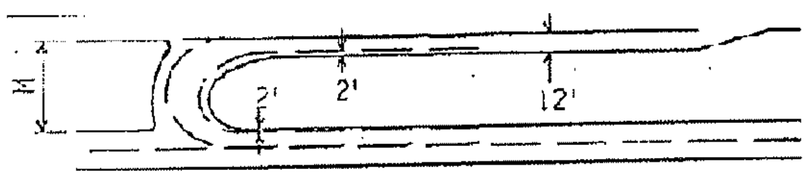


Figure 4b. Left lane to second lane maneuver.

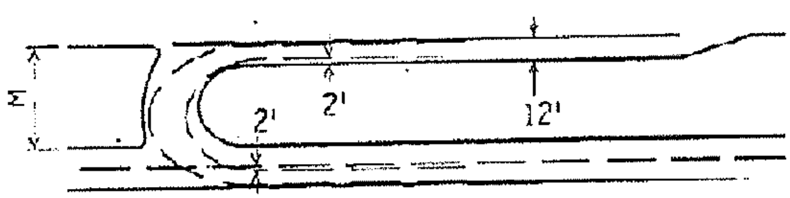


Figure 4c. Left lane to third lane maneuver.

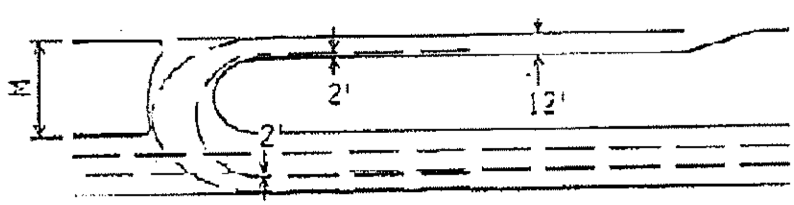


Table 1. Minimum median widths M for U-turn maneuvers suggested by MDOT.

Type of Maneuver	P	SU	BUS	WB-50	WB-60
	Length of Design Vehicle, m (ft)				
	5.8 (19)	9.1 (30)	12.2 (40)	16.8 (55)	21.3 (70)
Left Lane to Inner Lane	13.4 (44)	23.2 (76)	24.4 (80)	25 (82)	25 (82)
Left Lane to 2nd Lane	9.8 (32)	19.5 (64)	20.7 (68)	21.3 (70)	21.3 (70)
Left Lane to 3rd Lane	6.7 (22)	16.5 (54)	17.7 (58)	18.3 (60)	18.3 (60)
Where: P = passenger car SU = Single-unit truck WB-50 = Semitruc medium size WB-60 = Semitruck large size					

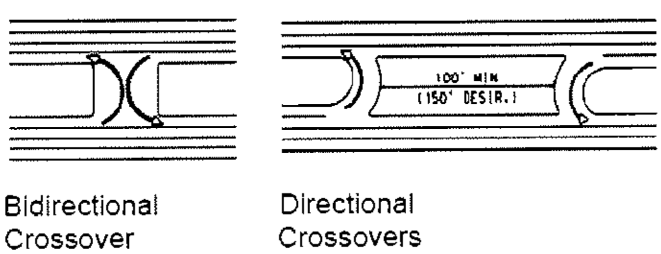
Location and Design of Median Crossovers

Figure 5 shows the two types of median crossovers, the "bidirectional" and the "directional." A bidirectional crossover is simply an opening in the median for vehicles to make U turns from either direction. Cars may enter from either direction. Bidirectional crossovers are sometimes installed without any deceleration or storage lanes. Most bidirectional median crossovers without deceleration/storage lanes can only store one or two vehicles. With high turning volumes, an interlocking effect is sometimes created. The vehicles queued to enter the crossover cannot do so until the vehicles in the crossover move out of the opening and merge into the travel lanes. A directional crossover is a one-way crossover with a deceleration/storage lane. This type of median crossover allows vehicles traveling in one direction of the boulevard to enter. As a result, motorists at a properly designed directional crossover should never experience the interlocking effect found at medians with a bidirectional crossover.

Taylor et al. (4) studied the effects of replacing existing bidirectional crossovers with directional crossovers on eight roadway sections in Michigan between 1991 and 1997. The study investigated crash frequency on roadway segments for two datasets. The study did not adjust for regression to the mean using control sites. One dataset included all the intersection crashes in the study segment, and the other dataset excluded intersection crashes from the study segment. The important findings of this study were:

- In total crash frequencies, 4 percent to 60 percent reductions were observed for the eight sections examined. The average reduction in total crash frequencies was 31 percent.
- In injury crash frequencies, 3 percent to 71 percent reductions were observed for the eight sections examined. The average reduction in injury crash frequencies was 32 percent.
- The crash types that experienced the largest decreases in crash frequency were rear-end and angle crashes. This effect was attributed to the lack of storage space and restricted visibility associated with bidirectional crossovers. There was an average 37 percent reduction in rear-end crashes when the bidirectional median crossovers were converted to directional median crossovers.
- Replacing bidirectional median crossovers at four-legged intersections and three-legged intersections produced reductions in total crash frequencies of 58 percent and 34 percent, respectively.

Figure 5. Directional and bidirectional crossovers.



Scheuer and Kunde (5) studied the effects of replacing existing bidirectional crossovers with directional crossovers on two segments of Grand River Avenue in Wayne County, MI, totaling 6.78 km (4.21 mi). The study segment was an eight-lane boulevard in a commercialized area with many driveways and minor crossroads. Three years of "before" crash data and approximately 2 years of "after" crash data were used in the analysis. The project achieved a total crash reduction of 24 percent. When the intersections where the crossovers were in-line with a crossroad are omitted, the crash reduction was 29 percent. Head-on and angle crashes showed the greatest reduction. The sideswipe crashes did increase, but the decrease in the heads-on and angle crashes far outweighed the increase of sideswipe crashes.

Castronovo et al. (6) studied the safety performance of divided highways with directional median crossovers versus bidirectional median crossovers. The key findings were:

- Divided highways with exclusive directional median crossovers have approximately the same crash rates as divided highways with exclusive bidirectional median crossovers for those sections without traffic signals.
- As the traffic signal density increases, divided highways with exclusive directional crossovers had 50 percent lower crash rates than crashes rates for divided highways with exclusive bidirectional median crossovers.

Figure 6a. Cured section of directional crossover
(Source: MDOT Geometric Design Guide 670).

CREST OF MOUND, FOR DRAINAGE AND AETHETICS, SHOULD SHOULD NOT EXCEED 1' ABOVE TOP OF CURB. IF NOT PAVED, VEGETATION MUST NOT OBSTRUCT DRIVER SIGHT DISTANCE (TYP.)

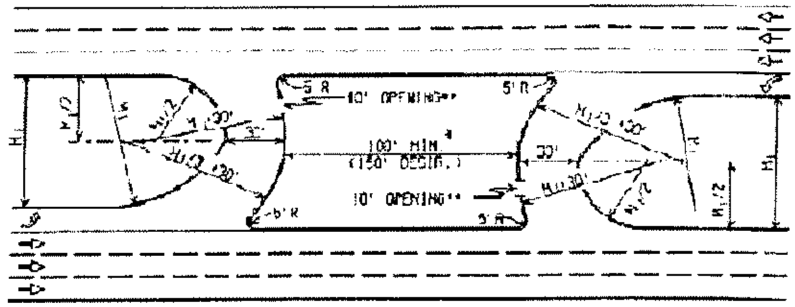
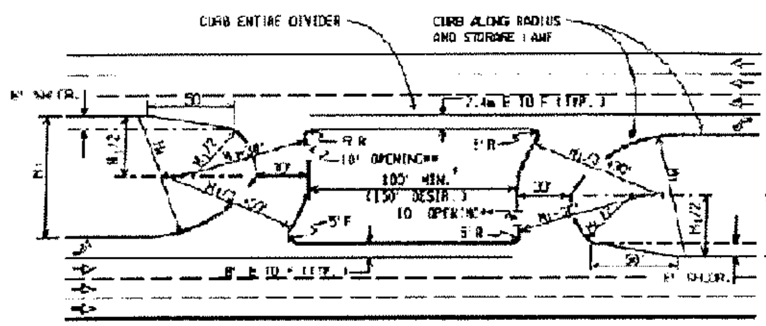


Figure 6b. Uncured section of directional median crossovers (Source: MDOT Geometric Design Guide 670).



Based on the studies cited, directional median crossovers likely provide better traffic operations and safety performance than bidirectional median crossovers. Figures 6a and 6b illustrate MDOT guidelines for designing directional median crossovers.

Location and Design of Loons

The design vehicle and the number of opposing lanes directly govern the required median width at the MUTIT median crossover junction. If the available median width is not sufficient, then agencies add additional pavement outside the travel lane to allow the design vehicle to complete the U-turn maneuver and merge back into the traffic stream. The additional pavements are typically referred to as "loons." Sisiopiku and Aylsworth-Bonzelet (7) defined loons as expanded paved aprons opposite a median crossover. Figure 7 shows a schematic diagram of a loon design, and figure 8 is a photo of an actual loon implementation in Wilmington, NC. The design width for loons will be the difference between the recommended median width in table 1 and the available median width.

Sisiopiku and Aylsworth-Bonzelet (7) evaluated the design and operation of loons and developed guidelines for loon design and placement. The important findings of the study were:

- Consistent placement of advance warning signs preceding the indirect median crossover and associated loon assisted driver expectancy when using MUTITs.
- Proper design of U turns for the appropriate design vehicle was essential to ensure safe traffic operation at the loons.
- At signalized median crossovers, the clearance intervals should account for the extra travel time required for drivers to travel through the loon.
- Suboptimal gap acceptance for U-turn maneuvers and driver confusion were two issues for loons either tapered into downstream right-turn lanes or for situations where right-turn lanes were located within approximately 45.7 m (150 ft) downstream of the loon. However, the placement of a loon and consecutive right-turn lane was recommended for major roads with MUTITs and high U-turning volumes at the median crossover
- Minimal differences were found between the travel times for commercial and passenger vehicles at MUTIT sites with signalized median crossovers. At unsignalized median crossovers, commercial vehicles were forced to wait for larger gaps in the conflicting traffic stream to complete their U-turn maneuvers.
- Several crashes involved commercial vehicles parked or backing within the median crossovers. Inadequate storage in the left lane preceding the median crossover due to the parked commercial vehicles caused spillback into through lanes. Commercial vehicles parked in the loon presented challenges for larger commercial vehicles executing U turns.

- A majority of the crashes at the loons were fixed-object crashes or sideswipe crashes. The objects most commonly hit were delineator posts, signposts (in the median and along the mainline), and spot locations of guardrail. A majority of the sideswipe crashes involved vehicles merging into traffic from the loon, or mainline traffic attempting to use the right turn lane.
- The study recommended a minimum 1.82-m (6-ft) auxiliary shoulder, with a 0.91-m (3-ft) paved area to provide the additional width necessary to ensure that the required pavement width will not be destroyed by U-turning vehicles that require the entire width of the loon. The study also recommended placement of short curves at both ends of the tapered section of the loon to assist the driver through the loon and U-turn maneuver.

Overall, loons are good design practice for facilities with narrow medians. With the use of loons, agencies can realize safety and operational benefits of a divided roadway (boulevard) with MUTITs, without incurring the significant cost of acquiring enough land along the entire corridor to provide sufficient median width.

Alternative Intersection Design

Michigan corridors with MUTIT typically have medians widths ranging from 18.3 to 30.5 m (60 to 100 ft). A wide median on the major road at the intersection of the major road and the cross street increases the pedestrian crossing distance along the sidestreet. Larger clearance intervals are required for the sidestreet signal phase with an increased possibility of vehicles and pedestrians getting "stranded" in the median space. Therefore, narrower medians with sufficient pedestrian refuge areas may be more efficient for the pedestrians and sidestreet traffic at the intersection of the major road and cross street. Figure 9 shows a possible reduction in median width at the intersection for a roadway with a median width of 18.3 m (60 ft) and a posted speed limit of 80.5 mi/h (50 mi/h). The reduction in median width was achieved by using reverse curves of sufficiently large radii on normal crowned sections of the roadway.

Capacity of Nonsignalized U-Turn Lanes

The Highway Capacity Manual 2000 (HCM) treats U turns as left turns for estimating saturation flow rate. However, the operational effects of U turns and left turns are different. U-turning vehicles have slower turning speeds than left-turning vehicles. Al-Masaeid (8) studied the capacity of U turns at unsignalized intersections as a function of the conflicting traffic flow on two opposing through lanes for median-divided roadways in Jordan . Figure 10 shows the field data collection results. He developed regression equations to predict the U-turn capacity based on the conflicting flows on two opposing through lanes.

$C = 799 - 0.31 * q_c$

$C = 1{,}545 - 790 * \text{exponential}\ (q_c/3{,}600)$

$C = 799 - 0.62 * q_{cp}$

Where:

C = capacity of U-turn movement in passenger car equivalent units per hour (PCU/hr).

q_c = conflicting traffic flow on two lanes (PCU/hr).

q_{cp} = conflicting traffic flow per lane (PCU/hr).

Yang et al. (9) studied the gap acceptance of U-turn maneuvers at median opening for 10 sites in Tampa, FL, and concluded that the critical gap ranged from 5.8 seconds to 7.4 seconds. Carter et al. (10) collected data at 14 signalized intersections with U turns in North Carolina. Based on a large database, they recommend a saturation flow adjustment factor of 0.82 for U-turn lanes at signalized intersections without conflicting right-turn overlap phase on the side street. Tsao and Ando (11) and Liu et al. (12) suggested saturation flow rate reduction factors of 0.8 and 0.76 for U-turn lanes at signalized intersections, respectively.

Provision of a Signal Phase to Serve U turns

The HCM suggests implementing a protected left-turn phase when the cross product of the hourly left-turning volumes and the corresponding hourly opposing through volumes exceeds the threshold value based on the number of opposing through lanes. Cross product thresholds of 50,000, 90,000, and 110,000 are applicable for one, two, and three lanes of opposing through traffic, respectively. The Traffic Control Devices (TCD) Handbook suggests the following criteria for where and when a left-turn phase should be provided:

1. Volume

a. Number for left turns multiplied by the opposing conflicting volumes in the peak hour exceeds 100,000 on a four-lane street or exceeds 50,000 on a two-lane street.
b. Left-turn peak-hour volume of more than 90 vehicles per hour, or 50 vehicles per hour on streets with through traffic at speeds over 72 km/h (45 mi/h).
c. At pretimed signal-controlled intersections, more than two vehicles per cycle per approach at the end of green during peak

hour.

2. Delay

Left-turn delay of more than 2.0 vehicle hours in the peak hour on a critical approach, provided there are at least two left turns per cycle during peak hour and the average delay per left-turning vehicle exceeds 35 seconds.

3. Crashes—number of left-turn crashes

a. One approach—4 crashes in 1 year or 6 crashes in 2 years.
b. Both approaches—6 crashes in 1 year or 10 crashes in 2 years.

The criteria above apply when determining whether a separate left-turn phase is needed at a signal-controlled intersection. The criteria can be applied equally, or in a more conservative way, applied to determine when signal control is needed at median crossovers to accommodate U turns. Signalized median crossovers can provide higher U-turn capacities compared to unsignalized median crossovers when the green time for the signalized median U-turn phase is adequate to satisfy the traffic demand. In addition, it is relatively easy to coordinate the signal at a median crossover with the signal at the main intersection without adding much extra delay to the high-volume mainline traffic.

Figure 7. Schematic of a loon implementation for a Michigan MUTIT.

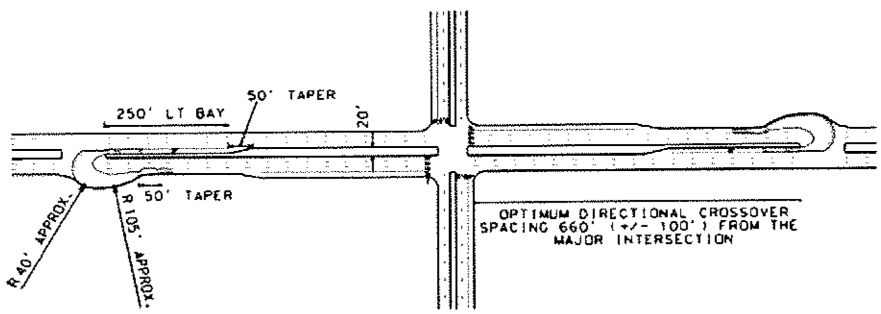
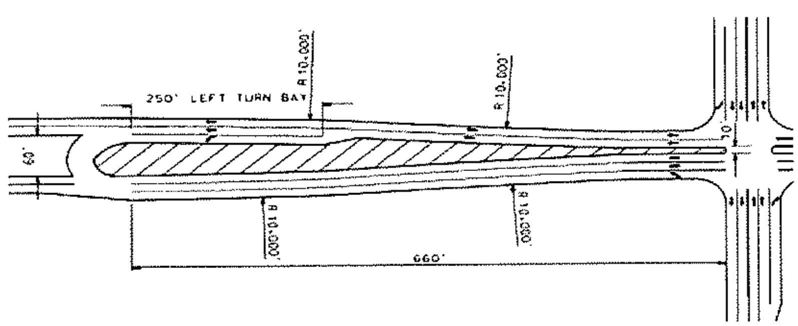


Figure 8. Example of loon implementation for a Michigan MUTIT.

Figure 9. Example of a transition from a wide median section to a narrow median section on MUTIT corridors.



Signal Phasing

The signal control at the intersection of the major road and minor crossroad operates with two signal phases because all left turns are prohibited at this junction. Figures 11a and 11b show the typical signal phasing diagram for the 2-phase signal. In some cases, the green signal indication at the median crossover junction for phase 2 can be delayed slightly relative to the green signal indication for the through/right-turning vehicles on the crossroad. This facilitates uninterrupted movement for the left-turning vehicles from the crossroad. If the median crossover is unsignalized, the signal phasing would only apply at the major road/minor road junction. Typical signal cycle lengths for the MUTIT range from 60 to 120 seconds. If the left-turn volumes are heavy, shorter cycle lengths will reduce spillback into the intersection. The pedestrians move in the direction of traffic with signalized pedestrian phases. Signalized

pedestrian phases across the major road with wide medians might reduce the operational efficiency of the MUTIT when cross-street traffic is minimal but pedestrian presence is significant during the peak hour periods.

Signing Plan

Figure 12 shows the typical signing plan for MUTIT in Michigan. Figures 13a to 13e show several examples of "innovative" signing treatments for MUTITs executed in Michigan. Sisoupiku and Aylsworth-Bonzelet (7) observed several motorists violating the turn prohibition and executing direct left-turns from the crossroad at rural sites. At intersections where violations were observed, there existed standard indirect left-turn signs and overhead signing prohibiting left-turns. Positive guidance communicated through additional signs may be beneficial in reducing driver confusion and ensuring higher rates of driver compliance.

Figure 10. Scatter plot of U-turn capacity versus conflicting traffic flow for unsignalized median openings.
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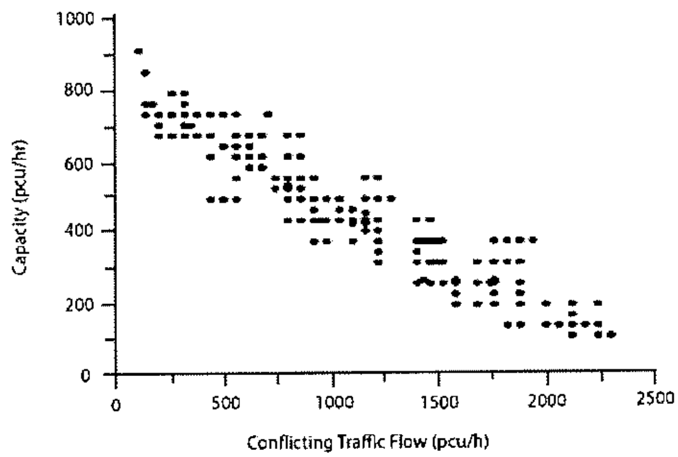


Traffic Operational Performance

Reid and Hummer (13) compared traffic operations along a typical arterial highway with MUTITs versus conventional designs with two-way left-turn lanes (TWLTL). The analysis corridor was a 4.02-km (2.5-mi) section of the Northwestern Highway Corridor in Detroit, MI. The section consisted of five major signalized intersections with varied spacing from 0.5 to 1.1 km (1,600 to 3,500 ft) and annual average daily traffic (AADT) ranging from 52,000 to 60,000 vehicles per day. Researchers used CORSIM to simulate traffic performance and used SYNCHRO to develop optimized signal timings. Four time periods were considered in the analysis, including peak periods in the morning, noon, midday (2:00 p.m. to 3:00 p.m.), and evening. Average measures of effectiveness (MOEs) were developed for a total of 48 CORSIM runs. The MUTIT showed a 17 percent decrease in total travel time within the study area network compared to TWLTL.

Figure 11. Example of typical signal phasing for the MUTIT.

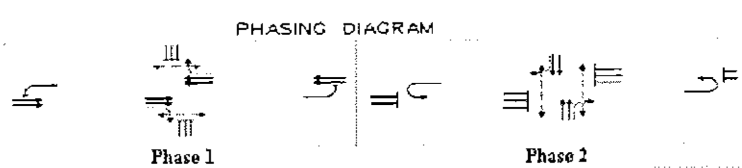


Figure 12. Example of typical signing plan for the MUTIT in Michigan.

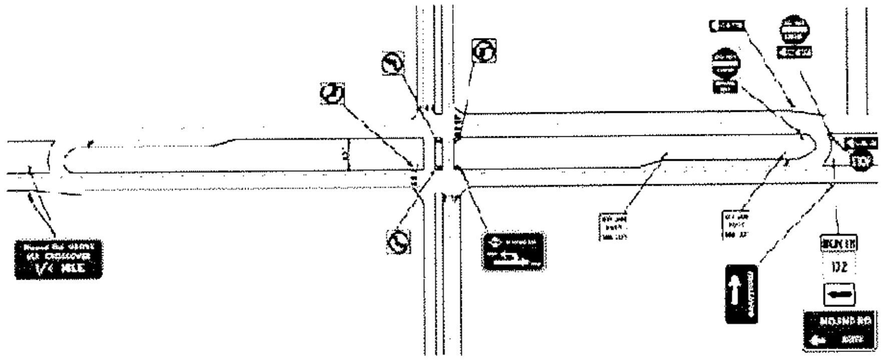


FIGURE 13. EXAMPLES OF "INNOVATIVE" SIGNING PLANS FOR THE MUTIT IN MICHIGAN.

Figure 13a. Example 1 of innovative signing. (Credit: Lee Rodegerdts)

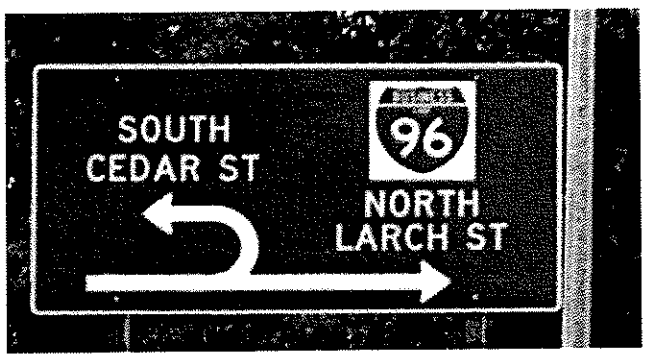


Figure 13b. Example 2 of innovative signing.
(Credit: Chris J. Bessert, www.michiganhighways.org)

Figure 13c. Example 3 of innovative signing.
(Credit: Warren Hughes)

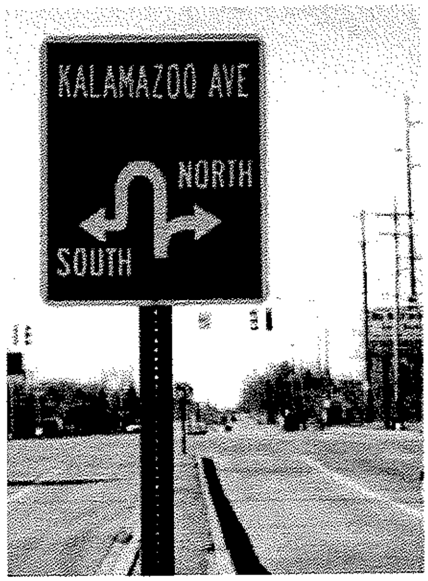


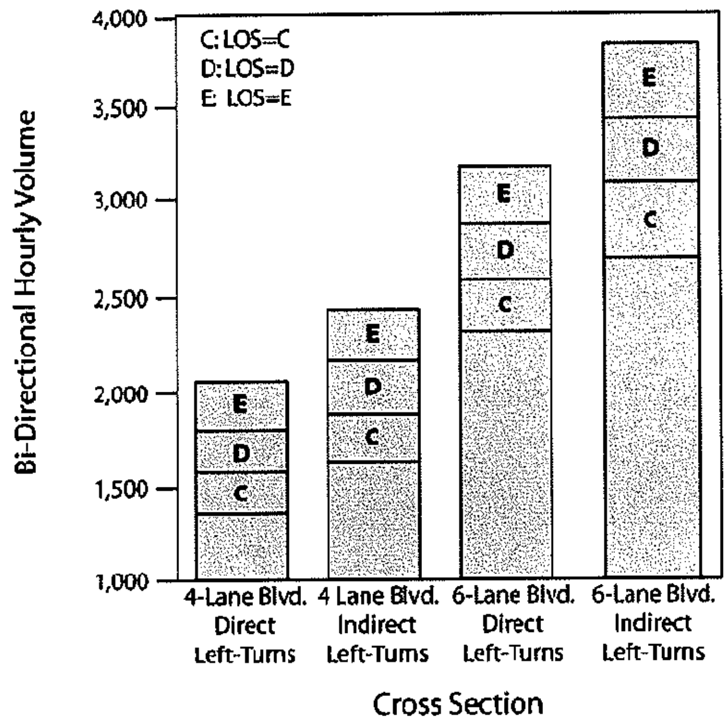
Figure 13d. Example 4 of innovative signing.
(Credit: Shawn Glynn)

Average speeds increased by 25 percent for MUTIT compared to the TWLTL. The average number of stops increased for the MUTIT compared to the TWLTL. The analysis indicated that the MUTIT had the potential to significantly improve system travel times and speeds in the corridor during the busiest hours of the day to not compromise system travel times during off-peak periods. Reid and Hummer (14) later used CORSIM to compare the traffic performance of seven unconventional arterial intersection designs, including the quadrant, median U-turn, superstreet, bowtie, jughandle, split intersection, and continuous flow intersections. The study used turning movement volumes from existing isolated intersections in Virginia and North Carolina. Off-peak, peak, and volumes corresponding to 15 percent higher than the peak volumes were examined. A total of 36 to 42 CORSIM simulation runs of 30-minute durations were analyzed for each intersection. For MUTITs, the CORSIM models used unsignalized U-turn crossovers for two-lane collector roads and signalized U-turn crossovers for four-lane collector roads. Entering volumes for the simulated intersections ranged from 4,500 vehicles per hour (vph) to 7,500 vph.

The MUTIT produced significantly lower average total travel times in comparison to the conventional intersection. The change in overall travel times for all movements through the intersection, when compared to a conventional intersection, was −21 to +6 percent during peak conditions. The overall change in the number of stops when compared to a conventional intersection was −2 to +30 percent during peak conditions.

Maki (15) compared the MUTIT and the conventional TWLTL on 4-lane and 6-lane boulevards and found a 20 to 50 percent increase in capacity (throughput) for the MUTIT. Figure 14 shows the level of service (LOS) comparison between corridors with MUTITs and conventional intersections.

Figure 14. LOS comparison of divided highways.
(Source: Robert Maki, City of Surprise, AZ)



Bared and Kaiser (16) studied the traffic operational benefits of signalized median U turns on a typical 4-lane road intersecting a 4-lane road using CORSIM. The cross-street left turn movement was allowed at the major road/cross street intersection resulting in a three-phase signal. An acceleration lane was provided for the right-turning vehicles from the major road to the cross street. These two features used in the study are different from the typical MUTIT implementations in Michigan. Entering volumes at the intersections used in the simulations ranged from 2,000 vph to 7,000 vph. The key findings of the study were:

- Considerable savings of travel time were observed for the U-turn design at higher entering flows (greater than 6,000 vph) compared to conventional intersections with 10 percent and 20 percent left-turning volumes.
- On average, the proportion of vehicles stopping on the network was lower for the U-turn design. For 10 percent left-turning volumes, differences ranged from 20 percent to 40 percent. For 20 percent left turns, a noticeable reduction in percent stops started at about 4,500 vph.
- Providing an acceleration lane on the crossroad was recommended to improve traffic operational efficiency.
- Longer offsets for the U-turn crossovers resulted in increased travel time but benefited the network at higher traffic volumes by providing adequate storage for the U-turning vehicles and preventing spillback into the intersection.

Dorothy et al. (17) evaluated traffic operational measures to study the differences in the performance of MUTITs compared to the conventional TWLTLs. The TRAF-NETSIM model was used to simulate these situations for 1-hour periods. The simulated network had signals every 0.8 km (0.5 mi) with the directional crossovers every 0.4 km (0.25 mi). A 60/40 split between the entering volumes on major road and cross street was assumed. When turning percentages were low, the crossovers were modeled as STOP-controlled; with higher volumes, signal control was assumed in the model. The signal cycle was 80 seconds with a 60/40 distribution of green time for the major road phase and cross-street phase, respectively. The median width varied from 12.2 to 30.5 m (40 to 100 ft). The key findings were:

- When the left-turning traffic percentage was 10 percent, MUTITs with signalized directional crossovers had lower left-turn total travel times than conventional intersections. The differences were 20 seconds/vehicle, 40 seconds/vehicle and 150 seconds/vehicle at 30 percent, 50 percent and 70 percent mainline saturation, respectively. Similarly, MUTITs with signalized directional crossovers had lower left-turn total travel times than conventional intersections when the left-turning traffic percentage was 25 percent. The differences were 20, 30, and 70 seconds/vehicle at 30 percent, 70 percent, and 90 percent mainline saturation, respectively.
- The MUTITs provided consistently lower network travel times compared to the five-lane TWLTL design.
- For low left-turning percentages, the directional median crossovers with stop control had approximately the same left turn total

time and network total time, as compared to directional medians with signalized crossovers.

Topp and Hummer (18) compared median crossovers on the cross street with median crossovers on the arterial highway for MUTITs using CORSIM. The left-turning volumes on the major road varied from 100 vph to 400 vph, the through volumes on the major road varied from 1,000 vph to 2,000 vph, the left turns on the cross street varied from 50 vph to 200 vph, and the through volumes on the cross street varied from 500 vph to 1,000 vph. The median crossovers were signalized wherever warranted. Results showed that the MUTIT design with the U-turn movement located along the cross street reduced percent stops, total travel time, and delay for most of the volume combinations analyzed in comparison to the crossover on the arterial.

Savage (19) studied the conversion of five-lane roadway with a TWLTL to a MUTIT in Michigan and found a 20 to 50 percent increase in the corridor capacity. Koepke et al. (20) found that the directional crossover design provides about 14 to 18 percent more capacity than the conventional dual left-turn lane designs. The results of critical lane volume analyses, after taking into account overlapping traffic movements, revealed reductions of about 7 to 17 percent in critical lane volumes, depending upon the number of arterial lanes (six or eight) and the traffic mix. Lower critical lane volumes translate into higher traffic flow capacity at the intersection. A study by Stover (21) computed critical lane volumes for the intersection of two six-lane, arterial roads. The effects of redirecting left turns were computed using these volumes. The provision of dual left-turn lanes on all approaches reduces critical lane volumes by 12 percent compared to providing single left-turn lanes but still requires multiphase traffic signal controls. The rerouting of left turns via directional crossovers and their prohibition at the main intersection reduces critical lane volumes by 17 percent.

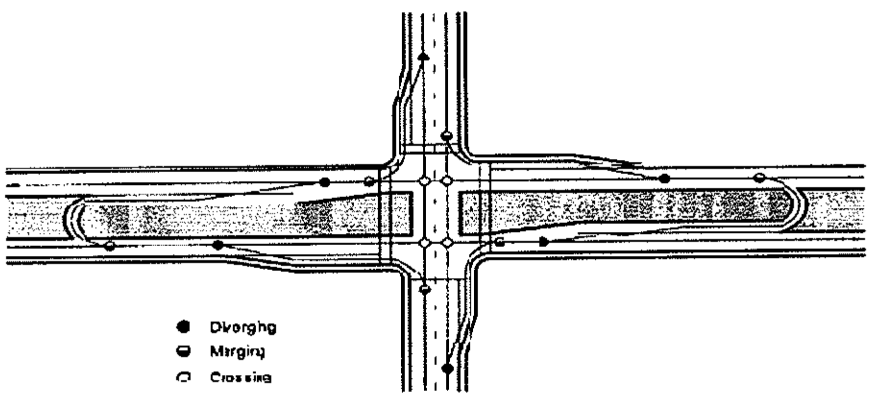
Overall, the literature shows that reducing signal phases and redirecting the left-turning movement at the intersection for the MUTIT provided significant benefits in terms of increased roadway capacity and reductions in travel time and vehicular delay when compared to conventional intersections.

Traffic Safety Performance

Table 2 from the FHWA Signalized Intersections: Informational Guide (22) shows the number of conflict points at a four-leg signalized intersection as compared to the MUTIT. The MUTIT eliminates all crossing (left turn) conflict points and reduces the number of merge/diverge conflict points as compared to a four-leg signalized intersection. Figure 15 shows the conflict point diagram for a MUTIT.

Observations indicated a 60 percent reduction in total crash frequencies and 75 percent reduction in total injuries. Reductions of 17 percent, 96 percent, and 61 percent were observed for rear-end crashes, angle crashes, and side-swipe crashes, respectively.

Figure 15. Conflict point diagram for the MUTIT.



Kach (23) compared the safety performance of conventional signalized intersections to MUTIT locations in the State of Michigan. The final comparison study subset consisted of 15 MUTIT locations and 30 conventional intersections.

Table 2. Number of conflict points at a four-leg signalized intersection compared to the MUTIT.

Conflict Type	Four-Leg Signalized Intersection	MUTIT
Merging/diverging	16	12
Crossing (left turn)	12	0
Crossing (angle)	4	4
Total	32	16

Maki (15) evaluated the safety benefits of replacing existing conventional signalized intersections with the MUTITs on Grand River

Avenue in Wayne County, MI. The 0.7-km (0.43-mi) study segment on Grand River Avenue was from the east of Poinciana to west of Delaware Street. The analysis period for the before-after study was 1990 to 1995.

The crossroads in all cases were undivided with crossroads intersecting at either 90 degrees or on a skew. Crash data for the years 1986-1990 were obtained for each site. Table 3 shows the safety performance of the MUTITs in comparison to conventional intersections. "Alpha" in Table 3 denotes the confidence level that the two rates are statistically different. Table 4 shows the estimated reduction in the expected number of crashes by crash type for all crashes, injury crashes, and property damage only (PDO) crashes for a road with 60,000 AADT.

Castronovo et al. (24) analyzed the MUTIT safety benefits versus conventional intersections as a function of traffic signal density using data from 123 segments of boulevards totaling 363.7 km (226 mi). The results indicated that as traffic signal density increased, the MUTIT had increasingly lower crash rates (measured in crashes per 161 million vehicle kilometers (100 million vehicle miles). For typical suburban conditions, with signal densities of one or more signals per 1.61 km (1 mi), the crash rate for MUTITs was about one half of the rate for conventional intersections. For typical rural conditions, with signal densities of one or less signal per 1.61 km (1 mi), the reduction in crashes for MUTITs was 36 percent when compared to conventional intersections.

In NCHRP Report 524 (25), researchers studied the safety performance of unsignalized median openings. The research results indicated that access management strategies that increase U-turn volumes at unsignalized median openings can be used safely and effectively. Analyses of collision data found that collisions related to U-turn and left-turn maneuvers at unsignalized median openings occur infrequently. In urban arterial corridors, unsignalized median openings had an average of 0.41 U-turn-plus-left-turn accidents per median opening per year. In rural arterial corridors, unsignalized median openings experienced an average of 0.20 U-turn-plus-left-turn accidents per median opening per year. On the basis of these limited collision frequencies, the authors concluded that there is no indication that U turns at unsignalized median openings are a general safety concern.

CONCLUSIONS

Based on the literature review conducted, the following summarizes the major conclusions:

- Michigan and other States have successfully used the MUTIT for over four decades without major problems related to traffic operational failures or safety hazards.
- Positive guidance communicated through additional signs and pavement markings at MUTIT sites may be beneficial in reducing driver confusion and enhancing traffic safety.
- With respect to driver expectancy, the MUTIT should not be mixed with other indirect and direct left-turn strategies on corridor level implementations.
- Though the MUTIT is typically a corridor treatment, the concept has been used successfully for isolated intersections to improve traffic operations and safety.
- Loons can be installed to accommodate larger U-tuming vehicles, so the MUTIT can be a feasible treatment for corridors with narrow medians.
- Directional median crossovers provide better operational and safety benefits compared to bidirectional median crossovers.
- Reducing signal phases at the intersection provides increased capacity for the MUTIT in comparison to the conventional intersections. The capacity increases are typically in the range of 20 percent to 50 percent.
- The total network travel time savings can and usually does outweigh the additional travel time required for left-turning vehicles from the major road and cross street for corridors with the MUTIT compared to conventional intersections.
- The safety performance of MUTIT is better than conventional intersections because they have fewer vehicle-vehicle conflict points. Typical total crash reductions range from 20 percent to 50 percent.
- Head-on and angle crashes that have high probabilities of injury are significantly reduced for the MUTIT compared to conventional intersections.

Table 3. Safety comparison of MUTITs and conventional intersections.

Dataset	Rate Type	Group	Mean Crash Rates (Crashes/MVE)	Standard Deviation	Alpha
Corridor	All	MUTIT (*Reduction*)	1.554 *(14%)*	0.784	73
		Conventional	1.806	0.679	
Intersection Related	All	MUTIT (*Reduction*)	1.388 *(16%)*	0.593	80
		Conventional	1.644	0.643	
	PDO	MUTIT (*Reduction*)	0.982 *(9%)*	0.392	49
		Conventional	1.077	0.467	
	Injury	MUTIT (*Reduction*)	0.407 *(30%)*	0.266	97
		Conventional	0.58	0.252	

Table 4. Expected crashes for MUTITs and conventional intersections for a 5-year period [WH12].

Crash Type	Injury Crashes				PDO Crashes				All Crashes			
	Conventional		MUTTIT		Conventional		MUTTIT		Conventional		MUTTIT	
	%	Expected Crashes	%	Expected Crashes	%	Expected Crashes	%	Expected Crashes	%	Expected Crashes	%	Expected Crashes
Overturn	1.53	0.97	0.92	0.41	0.64	0.75	0.27	0.29	0.95	1.71	1.03	1.57
Fixed Object	3.56	2.26	4.25	1.89	4.77	5.62	6.97	7.5	4.36	7.85	6.13	9.38
Head-on	0.80	0.51	0.27	0.12	0.43	0.51	0.33	0.35	0.56	1.01	0.35	0.53
Angle St	36.87	23.4	19.77	8.8	18.35	21.63	9.06	9.75	24.73	44.53	12.12	18.54
Rear End	37.99	24.11	65.93	29.35	51.67	60.9	69.85	75.14	46.94	84.51	68.29	104.44
Angle Turn	3.56	2.26	4.76	2.12	6.71	7.91	7.74	8.33	5.62	10.12	6.84	10.46
Rear End Lt	1.53	0.97	0.81	0.36	4.18	4.93	0.93	1	3.27	5.89	0.88	1.35
Rear End Rt	0.20	0.13	0.65	0.29	1.45	1.71	1.43	1.54	1.02	1.84	1.19	1.82
Sdswipe Opp	0.20	0.13	0.13	0.06	0.27	0.32	0.22	0.24	0.25	0.45	0.20	0.3
Head-on Lt	13.75	8.73	2.52	1.12	10.89	12.84	2.75	2.96	11.87	21.37	2.66	4.07
Sdswpipe same	0.00	0	0.00	0	0.64	0.75	0.44	0.47	0.42	0.76	0.31	0.47
Σ	100.00	63.47	100.00	44.52	100.00	117.87	100.00	107.57	100.00	180.04	100.00	152.93

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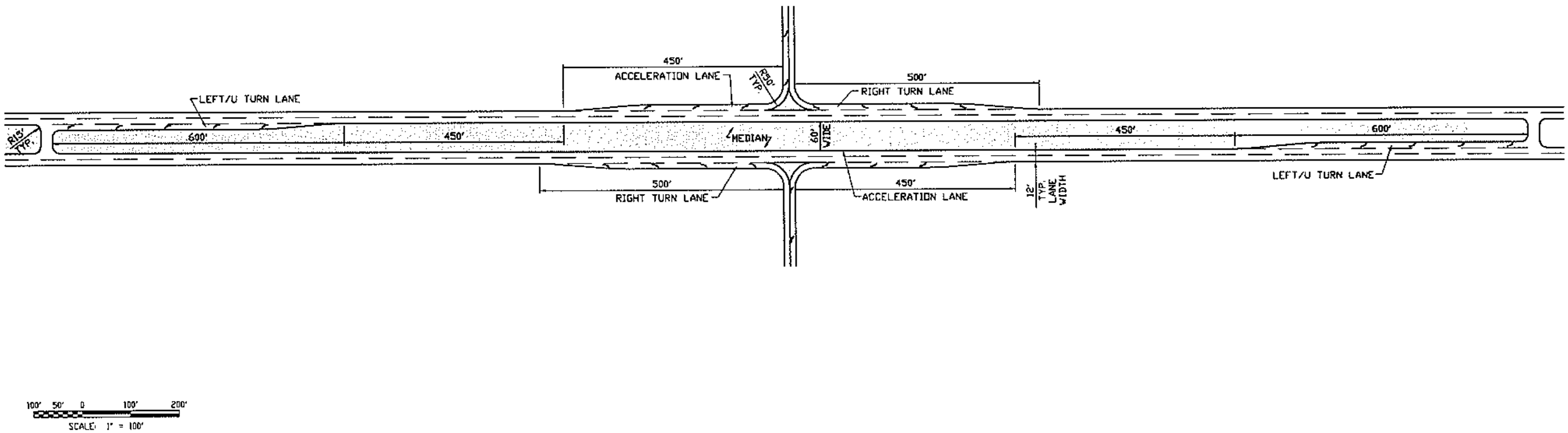
450'
ACCELERATION LANE
R50' TYP.
500'
RIGHT TURN LANE
LEFT/U TURN LANE
R15' TYP.
600'
450'
MEDIAN
60' WIDE
450'
600'
500'
RIGHT TURN LANE
450'
ACCELERATION LANE
12' TYP. LANE WIDTH
LEFT/U TURN LANE
100' 50' 0 100' 200'
SCALE: 1" = 100'

Travel Run Summary for Honoapiilani Highway

From	To	Distance	A.M. Peak Hour		P.M. Peak Hour		Comments
		Miles	Eastbound	Westbound	Eastbound	Westbound	
Bridge	Transfer Station	0.3	49.9	41.3	44.5	42.9	Bridge or culvert at dry stream at approximately mile marker 16.5
Transfer Station	Access west of Mgrs. House	0.8	48.8	44.8	45.1	41.7	Unmarked access about 0.4 miles from general store
Access west of Mgrs. House	General Store/Mgrs. House	0.4	40.4	36.5	36.7	34.7	
General Store/Mgrs. House	Luawai St.	0.5	43.9	46.4	45.5	44.9	
Luawai St.	Mile Marker 14	0.6	47.7	51.9	49.9	50.3	
Mile Marker 14	New Access Intersection 1	0.8	50.5	50.6	51.0	53.7	Newly constructed intersection, no street name shown
New Access Intersection 1	New Access Intersection 2	0.7	50.6	53.0	50.9	58.3	End at about mile marker 12.5. Newly constructed intersection, no name
Eastbound is to Maalea							
Westbound is towards Lahaina							

Travel runs made 10/27 - 10/28

6

TABLE 2

Generalized **Annual Average Daily** Volumes for Florida's **Areas Transitioning into Urbanized Areas** OR **Areas Over 5,000 Not In Urbanized Areas**[1]

10/4/10

STATE SIGNALIZED ARTERIALS

Class I (>0.00 to 1.99 signalized intersections per mile)

Lanes	Median	B	C	D	E
2	Undivided	8,900	14,100	15,200	***
4	Divided	26,900	32,100	33,800	***
6	Divided	41,500	48,600	51,000	***

Class II (2.00 to 4.50 signalized intersections per mile)

Lanes	Median	B	C	D	E
2	Undivided	**	9,400	13,700	14,700
4	Divided	**	22,700	30,000	31,700
6	Divided	**	35,700	45,400	47,800

Class III (more than 4.5 signalized intersections per mile)

Lanes	Median	B	C	D	E
2	Undivided	**	4,700	10,700	13,400
4	Divided	**	11,500	25,500	28,900
6	Divided	**	18,000	39,800	43,900

Non-State Signalized Roadway Adjustments

(Alter corresponding state volumes by the indicated percent.)

Major City/County Roadways - 10%

Other Signalized Roadways - 35%

State & Non-State Signalized Roadway Adjustments

(Alter corresponding volume by the indicated percent.)

Divided/Undivided & Turn Lane Adjustments

Lanes	Median	Exclusive Left Lanes	Exclusive Right Lanes	Adjustment Factors
2	Divided	Yes	No	+5%
2	Undivided	No	No	-20%
Multi	Undivided	Yes	No	-5%
Multi	Undivided	No	No	-25%
–	–	–	Yes	+ 5%

One-Way Facility Adjustment

Multiply the corresponding two-directional volumes in this table by 0.6.

FREEWAYS

Lanes	B	C	D	E
4	42,600	57,600	68,700	73,600
6	63,900	86,600	103,300	113,700
8	85,200	115,600	137,600	153,700
10	106,400	145,600	172,400	192,800

Freeway Adjustments

Auxiliary Lanes	Ramp Metering
+ 20,000	+5%

UNINTERRUPTED FLOW HIGHWAYS

Lanes	Median	B	C	D	E
2	Undivided	8,000	15,100	21,100	26,800
4	Divided	31,400	45,400	58,800	66,600
6	Divided	47,200	68,100	88,200	100,000

Uninterrupted Flow Highway Adjustments

Lanes	Median	Exclusive left lanes	Adjustment factors
2	Divided	Yes	+5%
Multi	Undivided	Yes	-5%
Multi	Undivided	No	-25%

26,800 X 1.05 = 28,140

BICYCLE MODE[2]

(Multiply motorized vehicle volumes shown below by number of directional roadway lanes to determine two-way maximum service volumes.)

Paved Shoulder/ Bicycle Lane Coverage	B	C	D	E
0-49%	**	2,800	7,300	>7,300
50-84%	2,200	3,400	13,100	>13,100
85-100%	4,100	>4,100	***	***

PEDESTRIAN MODE[2]

(Multiply motorized vehicle volumes shown below by number of directional roadway lanes to determine two-way maximum service volumes.)

Sidewalk Coverage	B	C	D	E
0-49%	**	**	5,000	14,400
50-84%	**	**	11,300	18,800
85-100%	**	11,400	18,800	>18,800

[1] Values shown are presented as two-way annual average daily volumes for levels of service and are for the automobile/truck modes unless specifically stated. Although presented as daily volumes, they actually represent peak hour direction conditions with applicable K and D factors applied. This table does not constitute a standard and should be used only for general planning applications. The computer models from which this table is derived should be used for more specific planning applications. The table and deriving computer models should not be used for corridor or intersection design, where more refined techniques exist. Calculations are based on planning applications of the Highway Capacity Manual, Bicycle LOS Model, Pedestrian LOS Model and Transit Capacity and Quality of Service Manual, respectively for the automobile/truck, bicycle, pedestrian and bus modes.

[2] Level of service for the bicycle and pedestrian modes in this table is based on number of motorized vehicles, not number of bicyclists or pedestrians using the facility.

** Cannot be achieved using table input value defaults.

*** Not applicable for that level of service letter grade. For the automobile mode, volumes greater than level of service D become F because intersection capacities have been reached. For the bicycle mode, the level of service letter grade (including F) is not achievable because there is no maximum vehicle volume threshold using table input value defaults.

Source:
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END