IMPLICATIONS OF ORIGIN-DESTINATION DISTRIBUTION IN FREEWAY SIMULATION

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ABSTRACT

Analyses with traffic simulation are common in transportation engineering and land development planning studies. The requirements for creating an accurate simulation include the geometry, traffic flow, and demand characteristics of the system. Demand is typically available in terms of volume (vehicles per hour) on specific links. However, most sophisticated simulators require the estimation of the origin-destination distribution of demand. This research examined the implications of origin-destination distribution derived from traffic volumes in freeway simulation. A complete methodology was developed including the development and implementation of five OD distribution techniques, the development of a traffic simulation, and the analysis of the resulting output. The five methods for generating OD distribution were: deterministic, proportionate, QueensOD (with default seed), QueensOD (with custom seed), and FREQ output. The origin-destination matrices by method were compared for similarities. An experimental analysis was performed on a segment of westbound H-1 freeway in Honolulu, HI from the airport to Waikele. The five origin-destination distribution methods were applied to the INTEGRATION traffic simulation model. The model output in terms of average vehicle spot speeds at five cross-sections and total travel time in the network were compared for each OD method. The OD distributions of the proportionate and FREQ output methods were similar to each other and they produced the best fit to the field data.
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LIST OF SYMBOLS

OD  Origin-Destination, usually a 2-dimensional matrix

$i$  Origin number

$j$  Destination number

$I$  Total number of origins

$J$  Total number of destinations

$T_{ij}$  Volume of vehicles traveling between an origin and destination

$F_i$  Volume of vehicles traveling from an origin

$F_j$  Volume of vehicles traveling to an origin

$R_j$  Volume of vehicles in nearest mainline link upstream of destination

$ML$  Mainline Node

$\mu_{yx}$  Predicted linear regression point

$\beta_0$  Regression intercept coefficient

$\beta_1$  Regression slope coefficient

$x$  Mean of variable, $x$

$y$  Mean of variable, $y$

$R$  Pearson’s correlation coefficient

$\mu$  Mean of the sample values

$v$  Number of degrees of freedom

$\alpha$  Confidence interval or level of significance

$\chi^2$  Chi-squared distribution test statistic

$\chi^2_{\alpha,v}$  Chi-squared distribution test upper (or lower) bound percentage point
\( H_0 \)  Null Hypothesis
\( H_1 \)  Alternative Hypothesis
\( s \)  Sample Variance
\( t_0 \)  t-test statistic
\( t_{\alpha/2,\nu} \)  t-test upper bound percentage point
\( \sigma \)  Square root of variance
\( W \)  Wilcoxon statistic
\( Z_+ \)  Sum of positive ranks
\( Z_- \)  Sum of negative ranks
CHAPTER 1
INTRODUCTION AND PROBLEM STATEMENT

1.1. Introduction

A simulation is as reliable as the information provided to the model. The realism of the simulation results may be sensitive to the model data inputs. In simulating traffic conditions there are many variables that an engineer must adjust to properly analyze a given system. Besides modeling the physical aspects of the freeway system the engineer must also determine parameters to account for driving behavior. Driving behavior pertains to car following, lane changing, passing, and other flow variables. Driving behavior variables are difficult to ascertain as they are an approximation of the actual conditions. The objective in traffic simulation modeling is estimating realistic system conditions to best produce a satisfactory outcome that is applicable to a real traffic network.

Traffic models have two purposes in this study: some models produce origin-destination (OD) distribution estimates and other models are used to simulate the OD estimation results. Simulation models have variable characteristics: static or dynamic, deterministic or stochastic, microscopic or macroscopic. Each simulation model has its own logic and use limitations [1]. A simulation model is applicable to specific components of a transportation system: single freeway or entire city network, queuing behavior or vehicle travel time. Depending on the complexity and the characteristics of the model the required resources and input information may be drastically different.
Modeling a transportation network is generally accomplished by a process called sequential demand-modeling. This procedure in transportation planning and analysis is also known as the four-step travel forecasting process. The purpose of travel forecasting is to predict travel demand in order to estimate the likely transportation consequences of several transportation alternatives being considered for implementation. The processes of each of these four components are: the decision to travel for a given purpose, the choice of destination, the choice of travel mode, and the choice of route or path on the actual or planned physical network.

The four sequential components which comprise this method are called trip generation, trip distribution, modal analysis, and network assignment [1]. Although this method seems like a simple four-step procedure, the travel forecasting method is complex and subject to great variability. The required driving behavior parameters are seldom exact and are often merely an approximation of real conditions. In particular, the trip distribution subset of the forecasting method is difficult to develop.

1.2. Problem Statement

Trip distribution is the allocation of vehicles between origin and destination points of travel on a given network. A common class of trip distribution models estimates an origin-destination distribution matrix for a network [2]. An OD matrix summarizes vehicle movement estimates. These tables contain the number of trips for each combination of origin and destination within a network for a given period of time [3]. Theoretically, the most accurate way to determine a network origin and destination distribution is to track the starting and ending points of all vehicle trips. This is infeasible due to the resulting data management and surveillance/privacy
issues [4]. However, measuring system characteristics such as traffic volumes and traffic speeds is feasible. Surveillance and control systems can collect traffic data without impeding traffic conditions [4]. Despite the collection of volume and speed data, the number of possible origin and destination distribution variations for a network is enormous. The variations in distributing the vehicles within a network likely result in a myriad of different simulation results. This study intends to determine the implications of origin-destination distribution in freeway simulation.

1.3. Objectives

The goal for this proposed research is to analyze the implications and effectiveness of different origin and destination distribution techniques for application in the simulation of a freeway system. To obtain these results the following objectives are defined:

- Select and implement several origin-destination distribution techniques.
- Derive origin-destination matrices by applying origin-destination distribution techniques and statistically analyze results for similarities and differences.
- Develop a base traffic simulation to accommodate the origin-destination distribution techniques. The base traffic simulation outputs (performance measures) must be representative of actual traffic performance.
- Simulate the origin-destination distribution variations using a base set of network parameters.
- Assess the implications of the trip distribution techniques based on statistical testing of the freeway simulation model results.
1.4. Thesis Structure

This document is organized as follows. Chapter 2 presents major past efforts on freeway origin-destination distribution estimation. This chapter is divided into many sections which discuss methods used to estimate OD distribution matrices for simulation purposes. Chapter 3 outlines the methodology applied to this study. The methodology includes the general components utilized for the generation of the origin-destination distribution matrices, the development of the simulation, and the analysis techniques. Chapter 4 presents the processes of the experiment development. This includes the application of the methodology to the real freeway network and collected data. The application of the origin-destination distribution and simulation development components are considered in the fourth chapter. Chapter 5 outlines the study results and corresponding analyses. The types and methods of analysis are discussed in the methodology section. Chapter 6 presents the study summary and conclusions. Future research is also considered in the sixth chapter. Finally, sample codes of software macros, data inputs for simulations, and simulation outputs are included in the Appendices. Critical data required for this study are included in the Appendices, as well.
CHAPTER 2
PAST EFFORTS ON FREEWAY ORIGIN-DESTINATION ESTIMATION

2.1. Introduction

The method used in developing an origin-destination distribution matrix is often subject to the available network information and data. Network information and data such as residential or commercial district locations may influence individual driver behavior. The trip-making component of driver behavior corresponds to the decisions required by the driver for if, and when, the driver will make a trip. As a function of driver behavior, trip-making must be quantified based on this information in the form of an OD distribution matrix. Pertinent data in trip distribution may range from network geometry and traffic speeds to travel surveys and household incomes [5]. The cost and availability of obtaining these data versus the simulation accuracy is a factor in the choice of OD analysis types. Many different problem formulation and numerical solution approaches may be considered based on these criteria [2].

2.2. Iterative Techniques

Iterative techniques are a common type of origin-destination estimation. Several such general distribution methods are used for trip distribution including the gravity, Fratar, growth factor, and intervening opportunities models [2]. Iterative techniques were first introduced to rudimentary transportation engineering applications as early as the 1920’s. However, these methods were not widely accepted until the late 1950’s [6]. Iterative models have gained popularity, especially
in past decades, due to their small requirements in computer processing power for large networks. Iterative techniques, such as the gravity model, have achieved virtually universal use because of the model simplicity, model accuracy, and due to the support from the U.S. Department of Transportation [7].

Another feature of iterative OD estimation models is that these types of models adhere to the theory of maximum entropy. The construction of spatial interaction models, such as the gravity model, is based on the entropy maximization principle of statistical mechanics. The method is based on an analogy with a branch of physics known as statistical mechanics which enables the physicist to explain and predict certain “macroproperties” of a system to a desired degree of accuracy without having to explain (at a “microlevel”) the behavior of each individual particle [8].

2.2.1. Gravity Model

Of all iterative origin-destination estimation techniques, the gravity model is the most popular. The gravity model is derived from an analogy to Newton’s law of gravitation. Newton’s law states that the force of attraction between two bodies is directly proportional to the product of the masses of the two bodies and inversely proportional to the square of the distance between them [1]. The form used in the transportation version of the gravity model is slightly different from the original version but the transportation gravity model still possesses many of the properties of Newton’s law.

The major variables required in the gravity model are trip productions, trip attractions, friction factor, and socioeconomic adjustment factors [9]. Trip productions and attractions are related to land use characteristics and the first step of
the sequential demand-modeling process: trip generation. Trip productions and attractions are not equivalent to origins and destinations. The friction factor, or impedance, component often requires data such as land use characteristics, average trip travel time, and trip distance [5]. Such variables are sensitive to time and location and may be empirical. These variables are often physically characterized by dividing a network into Traffic Analysis Zones (TAZ). These zones vary by location and are dynamic with respect to specific networks. Therefore, land use distribution is likely different for different networks. The socioeconomic adjustment factors are basically a scalar that is determined by iteration. This factor accounts for socioeconomic impacts on trip-making behavior that may not be accounted for by the other variables. The gravity model ensures flow conservation throughout a network.

2.2.2. Fratar Model

The Fratar model is often used to estimate external trips that are either produced and/or are attracted outside the boundaries of the region under study from outlying areas. Also, this model does not distinguish between productions and attractions and does not consider the direction of travel [1]. Unlike the gravity model, the Fratar model requires knowledge of origin-destination distribution in the form of a seed matrix. The main purpose of this model is to predict the origin-destination distribution at a future point in time. The variable used to aid in this prediction is called the growth factor. This variable is based on the anticipated land use changes that are expected to occur within the zone between the base year and target year [1]. Each iteration of the Fratar model is intended to improve the estimates of volumes
between zones. Similar to the gravity model, the Fratar model is frequently used as a method of origin-destination estimation.

2.3. Optimization Techniques

The majority of origin-destination distribution methods that employ an optimization technique have a similar procedure. Generally, there is an objective function that is either minimized or maximized. The variables that comprise the objective function are subject to a series of constraints. If the process is iterative, then constraints and/or the objective function frequently change in subsequent iterations. Despite similar general procedures, each optimization technique is based on different philosophies and each produces different origin-destination distribution matrices.

2.3.1 Predictive Estimation Minimization

In the past two decades a number of Predictive Estimation Minimization (PEM) methods have been developed including the gradient based optimization routine MINOS. The objective of this method is to minimize the difference between the actual traffic conditions and simulated traffic conditions for the network where many actual traffic conditions are known. The known traffic conditions may include traffic counts, vehicle speed, density, etc. The unknown traffic parameter is the vehicle OD distribution. The freeway network is reproduced in a simulation process where various OD traffic loadings are applied. The optimal OD distribution, which would reproduce identical traffic conditions in simulation, is determined in an iterative simulation process. The optimal OD distribution is determined by minimizing the difference in actual and simulated traffic conditions. Hence, the OD
matrix estimation process can be defined as the optimization problem that searches for the optimal OD matrix that minimizes the deviations of the simulated traffic conditions and actual traffic conditions [4].

One of the major concerns regarding predictive estimation minimization is the issue of identifiability. Similar to MINOS other PEM methods have been susceptible to identifiability issues. Identifiability issues pertain to the optimization model's ability to determine the goodness-of-fit or accuracy of the origin-destination distribution results. Often an optimization program produces local minimization results without converging onto the dominant global minimum. The sensitivity of optimal solution accuracy is usually influenced by the seed matrix [4]. A good initial "guess" at the actual distribution improves the accuracy in the optimization process. Also, conducting a PEM procedure with an accurate seed matrix, or good initial estimate of the network OD distribution, improves the identifiability of the origin-destination distribution results. That is, a global minimum is determined as the best defined of various local minima. The PEM tool is a broad term for a group of different optimization methods and is not specific to any one OD distribution technique.

2.3.2. QueensOD

One popular tool for the estimation of origin-destination tables is QueensOD [2]. This program is comprised of several different processes that aid in the OD distribution approximation. Initially, a static seed matrix is developed. The term "static" means that each cell with the matrix may be determined based on a single equation and does not require iteration or optimization. This matrix is the base data
for the iterative process. The seed matrix can be user defined or generated by the model based on the gravity model [2]. Once the seed matrix is developed, QueensOD assigns the objective function and begins the synthetic optimization process by performing multiple iterations. The term “synthetic” refers to any matrix generated by an iterative optimization procedure. The objective function minimizes link flow error by minimizing the difference between the actual link flow and predicted link flow. The method by which QueensOD predicts individual link flow is not available for user review.

The optimization is accomplished by minimizing an objective function. The initial objective function is comprised of seed matrix assumptions. Since the seed matrix may not adhere to flow conservation throughout the system, the solution may be infeasible. To alleviate this problem, Stirling’s approximation is applied to the system [2]. This technique avoids the introduction of flow conservation constraints by transforming the Lagrange constraints using partial differential calculus. Instead of eliminating the flow conservation error, Stirling’s approximation minimizes the conservation error within the constraints. The error introduced by this synthetic approximation is often less than 1% and results in a feasible solution [2]. The developers of QueensOD state that popular estimation methods that conduct the approximation process often are susceptible to large margins of error. These errors are much larger than those introduced by the QueensOD method.

Despite the error minimization and elimination techniques, QueensOD does have some shortcomings. The QueensOD component of the interaction between trip generation and trip distribution is made by assumption. The QueensOD program
develops the initial seed file assuming that the average trip distance is five kilometers for all trip types at any time of the day for any location regardless of network characteristics [2]. In real world conditions, the average trip distance may vary by time and location. Therefore, by introducing an initial seed file into the program the assumed travel distance does not influence the optimized solution. However, the implementation of an initial seed file does require knowledge of existing trip-making behavior, which may not be available.

2.3.3. Origin-Destination Distribution from FREQ

FREQ is a freeway queuing simulator that produces time-location queuing diagrams and other simulation related data and statistics. FREQ also features an internal OD distribution model. The model produces origin-destination tables as a complimentary output to the queuing diagrams. The OD distribution outcomes are also used by FREQ for the simulated queuing results. Inputted system data includes such features as the number of lanes, location, freeway curvature, and flow volumes.

In general, the OD matrices are computed in FREQ as the solution to a steady-state optimization problem [10]. However, according to U.C. Berkeley’s Dr. A. May who developed FREQ, this solution method is not always the steady-state optimization technique. May states that “Whenever everything else fails, the allocation between cells (origin-destination pairs) is done on a proportional basis with the constraint that the entries in the vertical columns (horizontal rows) add up exactly to the vertical column (horizontal row) sums”. Regardless of the method, in previous case studies the FREQ simulation results were found to provide reasonably good approximations when compared with actual origin-destination distribution data [11].
2.4. Travel Surveys

One of the oldest techniques used in the estimation of origin-destination distribution is the traditional survey method. This method is especially effective on a single freeway system as there is usually only one path between any origin and destination along the network. Survey data may also provide information other than the vehicle path. For example, a survey may be used to collect data regarding land use and socioeconomic information. This data may be applied to projection models and provide valuable trip generation information. The determination of the number of travelers to be sampled is a critical element in any origin-destination survey. The sample size affects the reliability of the survey results, the number of personnel needed, the cost of the survey, data processing requirements, quality control procedures, and the survey duration [12]. Therefore, minimizing the sample size, while obtaining accurate results, is a goal in any OD survey. Different survey types are frequently used and may require different sample sizes. In past surveys, the data on travel habits was obtained from interviewing a sample consisting of 4 to 5% of the total households in the region and about 20% of the truck and taxi companies [1].

2.4.1. Roadside Surveys

Roadside surveys provide a lot of information about travel demand patterns. The roadside survey is also known as a roadside interview. The direct survey method involves stopping travelers at interview stations located along the freeway and/or on the freeway entrance and exit ramps. Motorists are asked questions about their freeway entrance and exit points, the ultimate origin and destination of their trip, the trip purpose, and the trip frequency [12].
The standard procedure for estimating freeway origin-destination matrices directly from roadside interviews is to expand the survey sample results at each interview location. The sample results are expanded to represent the total volume of travelers passing the site. The same procedure can be used to estimate the origins of trips for survey stations located at freeway exits. If survey stations are located at every freeway entrance, it is helpful to have traffic volume counts at the freeway exits to act as accuracy checks on the demand estimation. If the estimated volume of vehicles exiting at any off-ramp, based on survey data, does not match the observed exiting volume the estimates are adjusted so that they match the observed volume [12].

2.4.2. Home Surveys

A direct survey method that collects comprehensive data with a lot less interference with traffic flow is the home survey. This survey method is also known as the postcard method. Instead of stopping vehicles and interviewing motorists on the freeway, travelers are handed postcards at survey stations and asked to fill them out and mail them to the agency sponsoring the survey. The postcards contain questions about the trip that the traveler is in the process of making, such as trip purpose, frequency, freeway entrance and exit, and ultimate trip origin and destination. To increase the accuracy of the survey the cards are usually coded so that the time and location at which they are distributed is known [12].

The processes for which the freeway origin-destination matrices are estimated in the home survey method are similar to the processes used in roadside survey method. In the roadside survey method the number of trips between any origin and
destination are determined by implementing statistical techniques. The main
difference between the two statistical techniques is the difference in the fraction of
vehicles surveyed between any origin and destination. In a postcard survey this
fraction is based on the total number of cards returned for a particular station rather
than the number of cards distributed.

2.5. Summary

The origin-destination distribution is an essential element of many network
based traffic models. In general, it is impossible to identify a unique optimal OD
distribution matrix from volume counts alone [13]. As a result there are several
different methods that have been used in OD estimation. These methods vary in
complexity, cost, and analysis time. Some of the distribution methods may
incorporate more information into OD trip estimation than others. Due to variable
input requirements and variable output results there is no universal method for
estimating origin-destination distribution.
CHAPTER 3
METHODOLOGY

3.1. Introduction

Three major components are required to determine the implications of origin-destination distribution in freeway simulation. Firstly, a series of OD distribution matrices are generated using different distribution models. These distribution models all use the same set of input volume data to develop the appropriate origin-destination trip pairs. The resulting OD distribution matrices are used as the single variable input component in the traffic simulation. All other traffic simulation inputs are held constant throughout the experiment. Next, an accurate traffic simulation is developed. Traffic simulation is the basis for which the implications of the origin-destination distribution are determined. After the traffic simulation for each origin-destination distribution is completed the appropriate data are extracted from the simulation model. Finally, these data are used to analyze the implications of origin-destination distribution in freeway simulation. This analysis is completed by making comparisons among the simulation results. The complete methodology flow diagram is included as Figure 3.1.

3.2. Origin-Destination Distribution Methods

The first component required for this study is the origin-destination distribution estimation. For this study a total of five OD distribution methods were implemented and tested. Each technique considers a different approach to the OD
Figure 3.1 - Methodology Flow Diagram

- **Objectives**
  - Deterministic
  - Proportionate

- **Freeway O-D Methods**
  - Automate development of O-D tables and INTEGRATION input files using MS Excel

- **Geometry**
  - Driving Behavior

- **Traffic Loads**
  - QueensOD (w/ Default Seed)
  - QueensOD (w/ Custom Seed)

- **Traffic Loads**
  - FREQ12

- **S** = INTEGRATION Traffic Simulator

- **MOE**
  - Deterministic
  - Proportionate
  - QueensOD (w/ Default Seed)
  - QueensOD (w/ Custom Seed)

  - MOE FREQ

- **Statistical analysis of differences of OD matrices by each method**

- **Compare w/ contemporaneous travel time and speed field data**

- **Determine Accuracy of Each Method and make Recommendations**
distribution. The five techniques applied are the deterministic, proportionate, QueensOD (with default seed), QueensOD (with custom seed), and FREQ output methods. These methods of OD distribution estimation are utilized as the corresponding models were available for use in this study. Other methods were considered but disregarded due to financial limitations. The generation procedure and interaction between each of the methods is included in the methodology flow diagram.

To illustrate several of the following OD distribution techniques a sample network is included as Figure 3.2. The sample network includes four origins and six destinations with flow volumes traveling from the left to the right in a numerically ascending order. The mainline nodes are designated by the term “ML”. The corresponding flow volumes for origins, destinations, and the mainline are included on each link. The origin and destination flow volume values are arbitrary. The mainline flow volume values are calculated to ensure flow conservation throughout the system. Figure 3.2 is the source of data for the distribution technique examples in the subsequent sections in this Chapter. (Real data are used in subsequent Chapters.)

Figure 3.2 - Sample Network
3.2.1. Deterministic

The deterministic method is a variant of the common first-in-first-out (FIFO) technique used in many transportation processes. In the case of origin-destination matrix estimation, the vehicles that have been introduced to the system first are the vehicles that have priority to exit the system. The deterministic logical procedure is for all vehicle flow from the furthest upstream origin to travel to the nearest unfulfilled downstream destination. If the vehicle flow required to fulfill the destination requirement is achieved, then the remaining vehicle flow from the origin proceeds to the next downstream destination. If the vehicle flow from an origin is exhausted, then vehicle flow begins from the next downstream origin. Once the "supply" of traffic from a given origin is exhausted, no additional traffic is allocated to destinations from that origin. The same is true when the "demand" is exhausted for any destination. In the form of a trip table, or in symbolic form, the origins may be represented by "i" and destinations by "j". Similarly, the total number of origins may be represented by "I" and the total number of destinations by "J". The volume of trips between any origin and destination is represented by the term "T_{ij}". The term "F_i" represents the entire flow volume from an origin, "i". The term "F_j" represents the entire flow required by a destination, "j". The subscript notation used in the symbolic subscript refers to the sequential number of the origin or destination. For example, Node 4 is actually the second origin. Therefore, the symbolic form refers to the relative origin node, or relative destination node, rather than the overall node number. A computational representation of the four deterministic conditions, in logical order, is as follows:
1) IF $F_i = \sum_{j=2}^{J} T_{i,j-1}$ OR $F_j = \sum_{i=2}^{I} T_{i-1,j}$

THEN $T_{i,j} = 0$

ELSE Condition 2

2) IF $T_{i-1,j} = 0$

THEN Condition 3

ELSE Condition 4

3) IF $F_i > F_i - \sum_{j=2}^{J} T_{i,j-1}$

THEN $T_{i,j} = F_i - \sum_{j=2}^{J} T_{i,j-1}$

ELSE $T_{i,j} = F_j$

4) IF $F_i > F_j - \sum_{i=2}^{I} T_{i-1,j}$

THEN $T_{i,j} = F_j - \sum_{i=2}^{I} T_{i-1,j}$

ELSE $T_{i,j} = F_i$

This scheme is not all encompassing as the first origin and destination require some special conditions. However, this is the general procedure for all other origin-destination pairs and a close representation of the special cases. For a better understanding of the deterministic procedure a numerical example is provided below. For simplicity the following subscripts used for calculations refer to the absolute node number rather than the relative origin or relative destination number. The
calculations and flows to be determined are for origin-destination pairs $T_{7,9}$ and $T_{7,10}$ based on the data from Figure 3.2.

Example:

- $T_{7,9} \Rightarrow \text{Condition 1: } IF \quad F_7 = T_{7,2} + T_{7,3} + T_{7,5} + T_{7,6} \quad OR \quad F_9 = T_{1,9} + T_{4,9}$
  
  \[322 = 0 + 0 + 0 + 0 \quad 433 = 0 + 184\]

  \[\text{Condition 1} = FALSE \Rightarrow \text{Condition 2}\]

  \[\text{Condition 2: } IF \quad T_{4,9} = 0\]
  
  \[184 = 0\]

  \[\text{Condition 2} = FALSE \Rightarrow \text{Condition 4}\]

  \[\text{Condition 4: } IF \quad F_7 > F_9 - T_{1,9} - T_{4,9}\]
  
  \[322 > 433 - 0 - 184\]

  \[\text{Condition 4} = TRUE \Rightarrow T_{7,9} = F_9 - T_{1,9} - T_{4,9}\]
  
  \[249 = 433 - 0 - 184\]

- $T_{7,10} \Rightarrow \text{Condition 1: } IF \quad F_7 = T_{7,2} + T_{7,3} + T_{7,5} + T_{7,6} + T_{7,9} \quad OR \quad F_{10} = T_{1,10} + T_{4,10}$
  
  \[322 = 0 + 0 + 0 + 0 + 249 \quad 539 = 0 + 0\]

  \[\text{Condition 1} = FALSE \Rightarrow \text{Condition 2}\]

  \[\text{Condition 2: } IF \quad T_{4,10} = 0\]
  
  \[0 = 0\]

  \[\text{Condition 2} = TRUE \Rightarrow \text{Condition 3}\]

  \[\text{Condition 3: } IF \quad F_{10} > F_7 - T_{7,2} - T_{7,3} - T_{7,5} - T_{7,6} - T_{7,9}\]
  
  \[539 > 322 - 0 - 0 - 0 - 0 - 249\]

  \[\text{Condition 3} = TRUE \Rightarrow T_{7,10} = F_7 - T_{7,2} - T_{7,3} - T_{7,5} - T_{7,6} - T_{7,9}\]
  
  \[73 = 322 - 0 - 0 - 0 - 0 - 249\]

The resulting distribution may assign no vehicle flow to several downstream destinations resulting in zero "$T_{ij}$" values. The sum of all traffic flow from all origins is equal to the sum of all traffic flow to all destinations. The only system information
DOCUMENTS
CAPTURED AS RECEIVED
required in a deterministic analysis is the amount of flow volume for each origin and
destination. Flow conservation is maintained and the distribution can be assigned by
implementing an automated spreadsheet analysis. Table 3.1 shows the resultant
deterministic OD table for the entire ten node sample network.

Table 3.1 - Sample Deterministic Origin-Destination Matrix

<table>
<thead>
<tr>
<th>Destinations</th>
<th>2</th>
<th>3</th>
<th>5</th>
<th>6</th>
<th>9</th>
<th>10</th>
<th>Σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Origins</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>76</td>
<td>188</td>
<td>207</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>471</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>0</td>
<td>38</td>
<td>103</td>
<td>184</td>
<td>0</td>
<td>325</td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>249</td>
<td>73</td>
<td>322</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>466</td>
<td>466</td>
</tr>
<tr>
<td>Σ</td>
<td>76</td>
<td>188</td>
<td>245</td>
<td>103</td>
<td>433</td>
<td>539</td>
<td>1584</td>
</tr>
</tbody>
</table>

3.2.2. Proportionate

The proportionate method is dependent on the amount of mainline flow within
the system. The mainline flow is the volume of vehicles in the system between
consecutive origins and/or destinations. The mainline flow may be characterized
simply by the conservation of flow between any number of intersecting link segments
within a network. Figure 3.2 is an example of mainline flows between origins and/or
destinations where the mainline flow volume is designated between any two
consecutive “ML” nodes. The variable “R_j” represents the nearest upstream mainline
flow volume approaching a given destination. At any mainline link along the
network the mainline flow consists of a compilation of all upstream origin input and
destination output flows. The proportionate distribution technique is based on
proportionally distributing the mainline flow among downstream destinations. This
method ensures that all applicable origin-destination pairs will have some interaction.
Again, the symbolic subscript refers to the relative origin node, or relative destination
node, rather than the overall node number. A computational representation of the single condition is as follows:

1) \( IF \ i > j \)

\[
THEN \ T_{i,j} = 0
\]

\[
ELSE \ T_{i,j} = \left( F_i - \sum_{j=2}^{j} T_{i,j-1} \right) \times \left( \frac{F_j}{R_j} \right)
\]

Again, this scheme does require some slight deviation for the first column of the trip table because the first destination requires some special conditions. However, this is the general procedure for all other OD pairs and a close representation of the special cases. For a better understanding of the procedure a brief numerical example has been prepared. Again, the following subscripts used for calculations refer to the absolute node number rather than the relative origin or relative destination number.

The calculations and flows to be determined are for OD pairs \( T_{4,3} \) and \( T_{7,9} \) based on the data from Figure 3.2.

Example:

- \( T_{4,3} \Rightarrow \text{Condition 1: } IF \ i > j \)
  
  \[
  4 > 3
  \]

\[ Condition\ 1 = \text{TRUE} \Rightarrow T_{4,3} = 0 \]

- \( T_{7,9} \Rightarrow \text{Condition 1: } IF \ i > j \)
  
  \[
  7 > 9
  \]

\[
T_{7,9} = \left( F_7 - T_{7,2} - T_{7,3} - T_{7,5} - T_{7,6} \right) \times \left( \frac{F_9}{R_9} \right)
\]

\[ Condition\ 1 = \text{FALSE} \Rightarrow 143 = (322 - 0 - 0 - 0) \times \left( \frac{433}{972} \right) \]
The nearest origin will have the largest flow contribution, based on the percentage of origin flow to the nearest destination. Intuitively, the furthest origin from a destination will contribute the smallest percentage of the vehicles, based on overall flow from the origin. Despite contributing a smaller percentage of vehicles, an origin does not necessarily provide a fewer number of vehicles. The sum of all traffic from all origins is equal to the sum of all traffic to all destinations. The proportionate method is similar to the deterministic method in that both ensure flow conservation and require minimal system information. Also, both are static and the flow distribution may be assigned by implementing an automated spreadsheet. Table 3.2 shows the resultant deterministic OD table for the entire ten node sample network.

Table 3.2 - Sample Proportionate Origin-Destination Matrix

<table>
<thead>
<tr>
<th>Origin</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>Σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>76</td>
<td>188</td>
<td>95</td>
<td>40</td>
<td>32</td>
<td>40</td>
<td>471</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>0</td>
<td>150</td>
<td>63</td>
<td>50</td>
<td>62</td>
<td>325</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>143</td>
<td>179</td>
<td>322</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>208</td>
<td>258</td>
<td>466</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>76</td>
<td>188</td>
<td>245</td>
<td>103</td>
<td>433</td>
<td>539</td>
<td>1584</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.2.3. QueensOD (with default seed)

Unlike static methods, QueensOD implements a synthetic iterative technique. The program consists of linear programming, partial differential calculus, constraint transformation, and objective function variation methods [2]. The QueensOD procedure is much more complex than static procedures and requires more input information from the user. However, the default seed file is merely a function of traffic count data and an arbitrary vehicle travel time. The resulting OD distribution seed is a simple equal flow distribution among all applicable pairs. The seed file is
unbalanced and violates vehicle flow conservation. The QueensOD method does not ensure flow conservation as the number of vehicles in the system can vary.

3.2.4. **QueensOD (with custom seed)**

QueensOD offers the option to include a user-defined seed file. The seed file is an OD distribution matrix in a computer input format. This file is recommended if the user has some prior information on the system’s expected outcome. Unlike the default seed OD distribution matrix, the proportionate method distributes the trips among OD pairs and ensures flow conservation among origins and destinations. Therefore, the proportionate method is considered a good candidate for use as a seed file, in place of the default seed file. This static-synthetic method provides QueensOD with more information, yet requires no additional network data. Also, the proportionate method does not assign zero values to any of the origin-destination pairs. Since QueensOD eliminates all pairs with zero flow from consideration in future iterations, the proportionate method does not eliminate possible pairs prior to the initial iteration or optimization. The deterministic method was not considered as a seed file for this same reason of eliminating OD pairs prematurely.

3.2.5. **FREQ Output**

Many simulation models require OD distribution information as a fundamental data input. However, some models do not require this information and may independently generate OD distributions internally with volume count data alone. FREQ is a macroscopic deterministic simulation model that independently develops origin-destination matrices based on origin and destination flow volume data [14]. The OD matrices are used by FREQ to develop freeway queuing models
for a variety of network sizes. These matrices were obtained from another study that analyzed the same segment of freeway with the same origin and destination data [15]. The resulting OD distribution output performed very well in simulating freeway queuing conditions. The FREQ results were accurate when compared to contemporaneous freeway queuing field data. The generation of OD matrices with each of the five methods presented above is discussed in Section 4.3 and the corresponding subsections.

3.3. Simulation Model

The INTEGRATION model is a microscopic traffic simulator that analyzes transportation networks on a routing-based platform. That is, the origin-destination distribution is determined external to the model. The actual trip path and arrival times to be determined within the simulation are based on the modeled interactions with other vehicles [1]. Due to the stochastic nature of the program the results may vary slightly for each simulation. However, the variation is minimal. This is a realistic outcome as a driver’s path and behavior is unlikely to be exactly the same every time s/he is to repeat a trip.

3.3.1. Model Logic

The INTEGRATION model is designed to consider each vehicle trip as an interrelated six step process. The first three decisions are to be considered prior to the initiation of a trip. These three preliminary driver decisions are the trip destination, the mode utilized to complete the trip, and time period to begin the trip. All of these preliminary decisions are interpreted prior to simulation and are a portion of the required user inputs. Since INTEGRATION only models vehicular trips the only two
required preliminary considerations are number of trips between origin-destination pairs and the time period in which the applicable trips occur [17].

The remaining three parts of the six step process are all internal to the INTEGRATION model. These three types of trip decisions are made once a trip has started and usually need to be revisited several times as the actual trip progresses.

Firstly, the driver must decide which route to take in order to reach the destination. In this particular freeway simulation this option is limited as there is only one possible route between any origin and destination. Therefore, this step does not directly apply for this freeway simulation. The next decision the driver must consider is the speed at which to travel and which lane to utilize. In general, this decision is dictated based on a minimization procedure where the driver attempts to minimize his/her trip travel time. The objective for all drivers is to arrive at the required destination in the least amount of time possible. To optimize the travel time every driver considers the travel speed in the lanes to the left and right and the lane that the vehicle currently occupies. If the lane to either side of a vehicle is flowing at a higher speed the vehicle will change lanes, assuming that there is an acceptable gap and all other conditions permit a lane change. If the lane currently occupied by the vehicle is the best option, then the vehicle will remain in the current lane. The final step considered by the INTEGRATION model is the vehicle merging process. Once a vehicle arrives at the end of a link the driver may be required to merge into a parallel traffic stream and must then decide whether to accept or reject any available gaps and how to merge with a converging traffic stream. Since these aspects of driving behavior are subject
to the freeway geometry and traffic conditions the trip path must be adjusted frequently [17].

Many of the behavioral aspects implemented in the INTEGRATION model are pre-specified parameters. However, the user is required to input many parameters beyond the basic network geometry. Other required variables include origin-destination volume, lane capacity, link speed, and jam density, and others. Also, the user may choose to include optional components such as lane striping, which may alter lane changing behavior and lane usage. These parameters are often unique to a given freeway network and likely require tuning to accurately reflect realistic driving behavior. Therefore, a “trial and error” approach may be appropriate in determining the required variables if specific information for each parameter is not available. The INTEGRATION model provides both statistical outputs and a real-time display of the freeway system with the vehicles in motion.

3.3.2. Geometry

Every INTEGRATION simulation is developed using a link-node system. A freeway system may be idealized into a series of nodes, which act as connection points, and links, which act as the roadway between any two nodes. Nodes are assigned using user specified x-y coordinates that approximate the real layout of the modeled network and are required wherever a change in the roadway or roadway characteristics may occur. Such locations may include freeway interchanges, lane drops, posted speed changes, etc. Links may be defined simply by assigning the upstream and downstream node numbers to a given link. The link requires the system information which pertains to a specific roadway segment. Every link requires the
number of lanes, the actual length of the link, and the two nodes that join the link to other features.

The placement of the nodes and links only has an effect on the simulation results if assigned inappropriately. The type of movement between any two links is dictated by the angle between the two links. If the angle between two connecting links is greater than 135 degrees and less than 225 degrees the downstream link is considered to be a straightaway connection and will accommodate straight movements. For example, two links at 180 degrees form one straight segment. However, if the angle between two connecting links is less than 135 degrees or more than 225 degrees the downstream link will only accommodate right or left turns, respectively [18]. This condition is valuable when assigning intersection junctions or ramps. These turning features are complimented by an optional lane striping file which directs vehicles to go straight or turn off of the mainline freeway onto a ramp. The introduction of ramps and striping influences vehicles by altering the lane changing behavior and results in more realistic traffic flow.

The placement of nodes and links may also affect the visual component of the simulation. The length of the links or placement of nodes may dictate the appearance of vehicles within the network. The length of a link is defined as an input variable and is not affected by the x-y coordinates. Similarly, the speed of a vehicle is a function of input parameters and the internal logic of the model. The visual output is only an approximate measure of performance or geometry. The level of visual accuracy depends on the quality of user input.
3.3.3. Driving Behavior

Typically driving behavior variables are difficult to estimate as such parameters may be unique among different freeway networks and different simulation models. Also, the parameters may change depending on the level of congestion and modifications to the freeway system. The INTEGRATION model requires several dynamic variables. In general, these variables are defined for each link. Each link includes parameters such as free speed, speed at capacity, jam density, saturation flow rate, etc. [19]. To best estimate the travel behavior an initial set of common behavioral data are used and adjustments are made by fine tuning the parameters. This is an iterative process that incorporates knowledge of the network, understanding of the simulation model, and evaluation of the model output. Driving behavior is sensitive to all of these conditions.

3.3.4. Traffic Loads

In the previous section the topic of unique traffic conditions was covered. To determine these traffic variables an iterative tuning process must be implemented. A set of base conditions must be used to initiate the tuning process. Similarly, the base traffic loading component of simulation must also be an arbitrary estimate of the real origin-destination distribution. That is, a base OD distribution matrix must be used throughout the tuning and simulation processes in order to make the proper adjustments to the driving behavior parameters. However, unlike the driving behavior parameters the initial origin-destination matrix must not be altered. This same matrix must be used in all forms of simulation whether adjusting parameters, simulating existing conditions, or making proposed alterations to an existing freeway

29
network. Otherwise the simulation may yield inconsistent results. Therefore, determining a good base OD distribution matrix for the traffic loading is critical for obtaining accurate simulation results since the matrix cannot be changed.

For a large freeway network, where actual origin-destination distribution information is unknown, there are an unlimited number of possible OD distributions. Arbitrary decisions on the base traffic loading will likely result in poor simulation performance. Hence, another approach must be pursued. The most logical approach is to use information provided by similar methods. In this particular case this information is available as the same freeway network has been simulated using other traffic models. The FREQ simulation model results are especially comprehensive as the program produces both a queuing simulation and an OD distribution matrix from the given input volumes. Also, in previous experiments the FREQ model has produced accurate results when compared to contemporaneous field data. FREQ is an appropriate source of the base OD distribution matrix for other simulation models. Therefore, the FREQ origin-destination distribution output was used as the base traffic data set for the INTEGRATION simulations. The FREQ simulation outputs are presented in Section 4.3.3.

3.3.5. Fine Tuning

To determine the accuracy of a simulation, when developed to replicate an existing system, comparisons must be made between the real freeway system and the simulated freeway system. This task may be accomplished using two methods: visual inspection and statistical comparison. The INTEGRATION model offers a real-time visual component. This output is a two-dimensional plan view display of the
simulated freeway network where every vehicle is represented with respect to the relative location in the network. Vehicle speed is represented by vehicle color. The color of each vehicle is dictated by the current speed with respect to the various inputted speed characteristics. For example, vehicles that are stopped or moving very slowly are red whereas vehicles traveling at nearly free flow speed are green [17]. A total of four colors are used to represent different speeds. Blue and yellow are used to represent the speed of vehicles when just above and just below the speed-at-capacity, respectively. Based on the color and density of vehicles on the display the user may be able to observe any queuing propagation and any congested areas. If a prior knowledge of the vehicle speeds and congested locations is known the visual component may aid in the simulation tuning process. A sample of the INTEGRATION visual output is included in Section 5.4.

The other method used to determine the accuracy of a simulation is the method of making statistical comparisons with a known data set. To gather important system data, detectors can be included anywhere within the network. Detectors collect simulation data such as vehicle speeds, traffic volume, vehicle travel time, etc. These detectors may be located anywhere within the system and may collect data on a per lane basis or for the entire link for a predetermined time span [17]. These comparisons are critical in determining the overall accuracy of a model and in the simulation tuning process. By comparing simulation data with contemporaneous field data driving behavior parameters may be altered in order to produce a better simulation outcome. The contemporaneous data collected for this study is discussed
in Section 3.4.2. With the aid of the visual component the driver parameters, and in some cases the geometry, may be adjusted in a quick and efficient manner.

In some cases the model may require some “tricks” to best model driver behavior. Simulation “tricks” may be introduced when other real driving characteristics do not successfully mimic real driving behavior. These “tricks” may include the alteration of driver behavior variables or modifications of the freeway geometry. For example, if the model is not representing accurate lane changing behavior, some lane striping that does not really exist may be introduced. However, this pseudo-geometry change may better represent the actual driver behavior for the network. This tuning method is not the first alternative as the alterations deviate from the actual conditions. However, in some cases “tricks” are the best way to simulate realistic conditions and obtain acceptable results.

3.4. Data Analysis

Once the simulation runs are complete for all five of the origin-destination distribution methods the corresponding results must be analyzed. However, before any analysis occurs the types of simulation outputs and the types of statistical analyses must be considered. The INTEGRATION model provides a wide variety of outputs. These statistics may pertain to the entire simulation network, such as the pollution created during the simulation period. Or the statistics may be specific to a certain location within the simulation, such as the vehicle volume passing a detector on a link. Either type of statistic may be useful in analyzing the implications of the OD distribution in simulation. For the purposes of this study contemporaneous data,
average vehicle spot speeds, and vehicle travel time were used in the evaluation of the alternative OD matrices.

3.4.1. Data Preparation

To determine if there is any correlation or similarities among any of the results the data sets may require some manipulation to be analyzed in the appropriate form. One data manipulation is the transformation of the results from a numeric format into a plot. A plot of the results provides a visual component to the analysis. This type of data representation does not allow for any statistical conclusions and is strictly for comparative purposes. However, a plot immediately provides a data set with a degree of validity when used for comparison. Also, a graphical format may provide a better understanding of any statistical testing procedures. The methods of analyzing plotted data are detailed in Section 3.5 and the corresponding subsections.

A data set that does require a transformation is the data provided in matrix form. For each of the origin-destination matrices there are an equal number of OD trip pairs. Despite any zero values, each matrix has the same origin-destination configuration. Therefore, each matrix may be assembled in an array format with each OD pair representing a unique location within the array. The configuration of the data within the array is arbitrary. However, the configuration must be consistent for any data sets subject to comparison. Each origin-destination pair represents a single data class and each vehicle represents a single data sample. This analysis is detailed in Section 5.3.

Similarly, the time-dependent results must also be properly formatted for testing. These simulation results must be transformed into an array form, as well.
Therefore, any data are placed into a unique format where each location within the array is represented by a specific time span. Each time span represents a single data class and each of the corresponding results represents a data sample value. For example, if the average vehicle speeds for a time span is 47 mph then that time span, or class, contains a value of 47. Or, if the collective vehicle travel time from 4:15 PM to 4:30 PM is 648 vehicle-hours then the class 6 contains a value of 648 data samples. For this testing the array is in a sequential order. Organizing the array in this fashion is arbitrary. However, a similar format is required for each data set as similar classes must be paired accordingly. This analysis is detailed in Section 5.5 and the corresponding subsections.

3.4.2. Contemporaneous Data

The purpose of the simulations is to compare the effects of different OD distribution inputs on an existing freeway network. Therefore, real network data may be collected and used as a basis for comparison. In this case the contemporaneous data that has been gathered is cross-sectional average vehicle spot speeds across all lanes at five locations. These data was collected using three different methods: Autoscope, moving observer (MO), and internet data from real-time traffic data acquisition stations. The specific details of each of these three methods are as follows:

- **Autoscope** - Autoscope is a video-based vehicle detection system for traffic data collection. The Autoscope system uses temporal and spatial machine-vision to recognize vehicles on a color image [20]. Traffic flow is recorded to
a VHS tape and then analyzed, via the Autoscope system, to collect traffic data.

- **Moving Observer** - The moving observer method is a technique that actually places a real driver within the network. The MO collects contemporaneous travel data throughout the network at predetermined times and locations. A moving observer experiences the real traffic conditions and records speed and elapsed time data. The MO provides the best data for use in statistical comparisons between real-world data and simulation generated data.

- **Internet Data** - The internet data sets are collected using a third data collection method. This method was developed by the TrafficWerks Company, which has developed the Econolite Traffic Data Collection and Management Service (ETDCMS). This system works in conjunction with the state Department of Transportation (DOT) to collect traffic data using the existing wire induction loops placed on the freeway. The purpose of the ETDCMS is to retrieve the traffic data and store it in an open architecture database. The database is accessed via a secure internet site and is available to all users [21].

Additionally, other simulation models have also been used to simulate the same freeway network for the same time period and have provided vehicle speeds and travel time statistics. These other base models include two macroscopic simulators: KRONOS and FREQ. These models are available from the University of Minnesota and U.C. Berkeley, respectively, and can simulate freeway conditions to produce network statistics.
3.4.3. Distribution Results

The distribution results are the initial measure of similarity among the five distribution methods. That is, an analysis must be performed on the distribution results to determine if any similarities exist, and to what degree, among the five methods. The distribution analysis is the first of the two required analysis components: distribution analysis and simulation analysis. The corresponding analysis of the INTEGRATION outcomes will determine if any similarity exists, and to what degree, between the five origin-destination distribution results and freeway simulation outcomes.

Unlike the simulation analyses, no other distribution data are provided for comparative purposes. Therefore, the distribution analysis does not incorporate contemporaneous data or the outcomes from other simulation models. Also, a total of 544 classes are considered in the distribution analysis versus 16 classes for any time dependent outcomes. The distribution classes are arbitrarily chosen and transformed into an array format. As a result, no continuity or relation exists between any two distribution classes. For time dependent data consecutive data points are related and exhibit continuity. This limits the effective analysis techniques for distribution comparison as plots and some statistical techniques provide little useful information. All distribution analysis is strictly statistical as graphical techniques do not apply.

3.4.4. Vehicle Speeds

One measure of effectiveness that directly applies to the INTEGRATION simulation model, the contemporaneous field data, and the other simulation models is vehicle speeds. All of these contemporaneous data sources provide the average
vehicle spot speeds over a time period as well as average vehicle spot speeds for specific segments. Contemporaneous field data may be collected at arbitrary locations within a network. These locations are subject to appropriate resources such as data collection equipment and field personnel. These data must be captured in real-time as data acquired on a different day may not reflect the same travel conditions. For example, if seven different locations are analyzed on seven different days the vehicle speeds data may not be universal for a single day because traffic patterns usually change daily. This may be the result of many factors such as different driving conditions, variable trip distribution, etc. Therefore, collecting all speed data at the same time is critical for an accurate comparison. Although the speeds data from other days may not be applicable, this data are included strictly for the purpose of comparison. Vehicle speeds collection in traffic simulation is much easier and does not require complicated resources. Average vehicle spot speeds over a predefined time period may be collected by applying traffic data detectors within the simulated system.

The data collected from all of these sources are compared in two ways: graphically and statistically. The graphic component of this comparison is a plot of the average vehicle spot speeds for each time period at each of the different cross-section locations. These average vehicle spot speeds plots incorporate the results from all of the available data sources, contemporaneous and simulated, for a given location. The intention of each plot is to illustrate how similar each of the data sources is to one another. The plots are displayed and discussed but no statistical analysis is performed.
The statistical component of the comparison incorporates the INTEGRATION simulation results. The statistical component does not include the data sets provided by the KRONOS and FREQ models. For certain statistical method the vehicle speeds analysis incorporates comparisons of the contemporaneous data and simulation speed data. The analysis methods are applied strictly to the five distribution method outcomes and selected contemporaneous data to determine the corresponding implications. The statistical analysis methods applied to the average vehicle spot speeds data are discussed in Section 3.5 and the corresponding subsections.

3.4.5. Vehicle Travel Time

Another output provided by the INTEGRATION model is the total vehicle travel time for a given period. This MOE is more abstract than the vehicle speeds at a certain location. The vehicle travel time may be described as the total amount of time all vehicles spent within the network. This results in the quantified measure of vehicle-hours per period. Unfortunately, this MOE can only be quantified in simulation. This value is unrealistic to obtain as contemporaneous data because it would require that each vehicle within the system is individually monitored. However, this statistic is easy to track and calculate when using simulation models. Vehicle travel time is an applicable MOE for comparison among the INTEGRATION, KRONOS, and FREQ simulations.

Similar to the vehicle speeds comparison, the vehicle travel time analysis includes a graphical and statistical analysis. The graphical component consists of a single plot for the entire network with all of the data from each of the simulation models and the corresponding variants. The statistical component only analyzes the
INTEGRATION simulation variants. All other conditions are the same as the conditions used in the vehicle speeds analysis. The statistical results are provided in Section 5.5.6.

3.5. Analysis Methods

Several forms of data will be analyzed to assess the implications of origin-destination distribution in freeway simulation. Several analysis methods are considered as different analysis methods characterize different aspects of the results. Both input data and output statistics from traffic simulation must be tested to determine if any of the methods exhibit similarities. Also, the analysis will determine the degree of correlation between OD distribution methods and the resulting simulation outcomes. The analysis types and applications are discussed in the subsequent sections.

3.5.1. Observation Analysis

One method for analysis is to assess the data by visual inspection. Such a technique is best accomplished by plotting the data in a distinguishable form. Since contemporaneous data are available the moving observer results will be used as the primary basis for comparison. The secondary comparison data will be considered on a case by case basis. A plot is developed for vehicle speeds at each of the five cross-section locations and for the collective network travel time. The observation analysis is strictly for comparative purposes and is not a statistical measure of performance. These analyses are provided in Section 5.5 and the appropriate subsequent sections.
3.5.2. Criterion Analysis

The H-1 simulation is developed to perform similarly to the provided contemporaneous data. Therefore, the INTEGRATION simulation outcome and the contemporaneous data are used to assess the relationship between the five distribution techniques in simulation. In general, one set of contemporaneous data must be used as the basis of comparison. For this study the moving observer data best replicates the real driving conditions. Therefore, the MO data are used for the criterion analysis. However, where moving observer data are not available an average of the three internet data sets is used as the basis for comparison.

The criterion analysis consists of counting the number of time periods at a cross-section in which the contemporaneous and simulation data are within an acceptable range of speeds. That is, all simulated average vehicle spot speeds data sets are compared with the moving observer data. This comparison is based on the absolute difference in the speed data. For this study a range of ±10 mph is used. This range was chosen because a 20 mph range can adequately characterize the level of congestion. For example, any cross-sectional average spot speed between 40 and 60 mph likely represents free flow conditions. Conversely, cross-sectional average vehicle speeds between 10 and 30 mph represent heavy congestion. If the MO speed for a time period is 25 mph and the simulation speed is 40 mph the network conditions are likely dissimilar for that cross-section during that period.

The purpose of this statistical application is to determine if any OD distribution method provided similar results to any other OD distribution method in simulation based on comparison to contemporaneous data. This analysis method is only applied to the vehicle speeds data as contemporaneous data are not provided for
any other MOE. This analysis technique may also be referred to as the 10/75 criterion analysis throughout this study. An explanation of this title is included in Section 5.5. The statistical results are provided in Section 5.5 and the corresponding subsections.

3.5.3. Regression Analysis

In many situations many variables are inherently related. Regression analysis is a statistical technique for modeling and investigating the relationship between variables [16]. If a linear relationship exists between two variables, where one variable is plotted on the x-axis and the other variable on the y-axis, a straight line can be used to summarize the data [22]. The equation of this line is:

$$\mu_{Y|x} = \beta_0 + \beta_1 x$$

where the slope and the intercept of the line are called regression coefficients. To estimate the regression coefficients the method of least squares is applied. This method results in a line that minimizes the sum of squared vertical distances from the data points to the line in terms of the y-axis. The least squares estimates of the intercept and slope regression coefficients in the simple linear regression model are:

$$\beta_0 = \bar{y} - \beta_1 \bar{x}$$

$$\beta_1 = \frac{\sum_{i=1}^{n} y_i x_i - \left( \sum_{i=1}^{n} y_i \right) \left( \sum_{i=1}^{n} x_i \right)}{\sum_{i=1}^{n} x_i^2 - \frac{\left( \sum_{i=1}^{n} x_i \right)^2}{n}}$$

$$\sum_{i=1}^{n} y_i x_i = \frac{\sum_{i=1}^{n} x_i^2 \left( \sum_{i=1}^{n} y_i \right)}{n}$$
where

\[ \bar{y} = \left( \frac{1}{n} \right) \sum_{i=1}^{n} y_i \]

\[ \bar{x} = \left( \frac{1}{n} \right) \sum_{i=1}^{n} x_i \]

The slope of the simple linear regression line is an excellent measure of similarity between two data sets, where the intercept regression coefficient is near zero. Where two OD distribution data sets are compared the slope of the line is indicative of the relationship between two sets of data. If the regression line has a slope of 1.00, and intercept near zero, the two data sets subject to comparison have similar values. If the slope of the regression line deviates from 1.00, greater than or less than, the two data sets are less similar. For example, if the travel time results for one set of data (plotted on the x-axis) are consistently lower than another set of data (plotted on the y-axis), and the intercept is near zero, the slope will be much greater than 1.00.

Alone, this regression is not an exact measure of similarity or dissimilarity among two data types. Regression is a means of calibrating the unknown coefficients of a prediction equation and is a method for model calibration [24]. The caveat of this analysis method is that the slope of the regression line does not account for the scatter of the plotted data and is largely a measure of the weight of the data. That is, if two points are plotted that are equally unrelated, than the regression slope may still be equal to 1.00. For example, if the points (3,5) and (5,3) are plotted the resulting regression line will be 1.00 (if the intercept coefficient is constrained to cross at }
0). However, the minimized error may still be very large. Henceforth, if the plotted data points are scattered the regression line may still be at, or near, 1.00.

Additionally, a measure of the error is desirable to better determine the level of similarity between two sets of data. This analysis is applied to the distribution results and the vehicle travel time. For this study the criterion variable is always the vertical variable within the regression analysis results matrix. The predictor variable is in a horizontal format within the results matrix. That is, the vertical column method is regressed on the horizontal row method. A plotted example of this analysis is provided for the origin-destination distribution in Section 5.3.

3.5.4. Pearson’s Correlation

A variant of the linear regression analysis method is Pearson’s correlation. This method may be considered to represent the strength of the association between two variables. The absolute value of the Pearson’s correlation coefficient indicates the strength of the linear relationship [22]. That is, the correlation between two variables reflects the degree to which the two variables are related. The Pearson’s correlation coefficient ranges from -1.00 to 1.00. A coefficient value of 1.00 represents a perfect positive linear relationship between two variables. A lesser value indicates that there is a scatter in the linear regression and all of the data points do not fit directly on the linear regression line. A larger regression error results in a Pearson’s correlation value closer to zero. A correlation of zero means that there is no linear relationship between the two variables. A correlation value of -1.00 indicates that there is a perfect negative relationship between the two variables [23].

The correlation value equation is:
This equation is used most often because this form does not require prior 
computation of the means. Pearson's correlation coefficient is an index of the degree 
of linear association between two random variables. The magnitude of Pearson's 
correlation coefficient, \( R \), indicates whether the regression provides accurate 
predictions of the criterion available. This is an index of the goodness-of-fit. If the 
Pearson's correlation value is squared, \( R^2 \), the result may be described in terms of a 
percentage. For example, if \( R \) is equal to 0.90 then 81\% of the total variation in \( Y \) 
is explained by variation in \( X \) [24]. Both of the variables \( Y \) and \( X \) are subject to 
comparison.

As discussed in the precedent section, a linear regression is not an exact 
measure of similarity or dissimilarity among two data types. The same is true for 
Pearson’s correlation. A correlation coefficient of 1.00 indicates that there is no 
regression estimation error as all data points are on the regression line. Although, this 
relationship may not be perfectly similar as the relationship between the two data 
types may be skewed. For example, if the data points (2,1) and (4,2) are plotted the 
resulting Pearson’s correlation coefficient will be 1.00 as both points lie directly on 
the linear regression line. However, the slope of this line is less than 1.00. Therefore, 
there is a relation between the two data sets but each set has a different linear 
magnitude.
Alone the Pearson’s correlation does not necessarily measure the level of similarity between two data sets accurately. However, if coupled with the linear regression slope coefficient, \( \beta_i \), the Pearson’s correlation coefficient, \( R \), does provide a good statistical representation of the variables subject to comparison. Where these two coefficients behave dissimilarly a linear relationship is likely nonexistent. This analysis is applied to the distribution results, vehicle speeds, and vehicle travel time. Unlike linear regression, when using Pearson’s correlation the two variables subject to analysis need not be distinct [24]. A plotted example of this analysis is provided for the origin-destination distribution in Section 5.3.

3.5.5. Independent t-Test Analysis

The t-test is used to test hypothesis and confidence intervals for two independent data sets. For the purposes of this study the hypothesis test for a difference in means, where the variances are unknown, are considered. This method determines whether two data sets are similar or dissimilar based on a normal distribution. The parameter of interest, the mean, is used to develop the testing hypotheses [16]. These hypotheses are:

\[
H_0 : \mu_1 = \mu_2 \\
H_1 : \mu_1 \neq \mu_2
\]

The test statistic, \( t_0 \), is:

\[
t_0 = \frac{\bar{x}_1 - \bar{x}_2}{s_p \sqrt{\frac{1}{n_1} + \frac{1}{n_2}}}
\]
where

\[ s_p = \sqrt{\frac{(n_1 - 1)s_1^2 + (n_2 - 1)s_2^2}{n_1 + n_2 - 2}} \]

From these statistics the P-value may also be determined. The P-value is the upper, or lower, percentage point as a measure of randomness and is available in tabular form. The P-value is subject to two variables: the number of degrees of freedom, \( v \), and the confidence interval (or level of significance), \( \alpha \). The number of degrees of freedom is two less than the total number of data samples, subject to some special conditions, used in the experiment. The level of significance, also known as the confidence interval, is a parameter that dictates the probability of error in the analysis. For example, if \( \alpha = 0.05 \) there is a 5% probability that the outcome will result in an error. The testing requires a two-tailed significance. Therefore, the value for the confidence interval must be divided by two. Accordingly, the hypothesis is rejected if either of the two following criteria is correct:

\[
\text{IF } t_0 > t_{\alpha/2,v} \text{ THEN hypothesis is rejected}
\]

\[
\text{IF } t_0 < -t_{\alpha/2,v} \text{ THEN hypothesis is rejected}
\]

\[
\text{IF } -t_{\alpha/2,v} < t_0 < t_{\alpha/2,v} \text{ THEN hypothesis is not rejected}
\]

Otherwise, the two data sets may be considered to be similar. For the purposes of this study the significance value is tabulated. If the significance value is greater than 0.05 the two data sets are considered to be similar and the hypothesis is not rejected. If the significance value is less than or equal to 0.05 the hypothesis is rejected and the two data types are considered to be similar. This analysis is applied
to the simulation vehicle travel time results. This analysis is presented in Section 5.5.6.

3.5.6. Other Analysis Methods

The number of statistical analysis methods is enormous. These methods may include simple techniques such as the average or standard deviation of each of the data sets. More complex methods may apply, as well. However, many of these methods were omitted for one of two reasons: 1) The method is already incorporated in the application of another method, or 2) the method does not directly apply.

One method that may have been considered is chi-squared distribution testing. The chi-squared distribution is a special case of the gamma distribution. This distribution is used extensively in interval estimation and tests of hypotheses [16].

The chi-squared distribution can assess the randomness of an array of numbers by fitting them to a known probability distribution [9]. That is, if an array of values is compared to another array, this method will determine whether the arrays are similar or random with respect to one another. A hypothesis method, similar to the t-testing method, is used to reject data set comparisons. To determine whether or not an array is random the observed distribution statistic must be compared to the $P$-value. If the observed distribution statistic is less than the $P$-value the hypothesis is true and the arrays are similar. Otherwise the array hypothesis is false and the arrays are dissimilar. That is, in symbolic form:

\[
\begin{align*}
\text{IF } \chi^2 &\leq \chi^2_{\alpha,\nu} \text{ THEN } \text{array is similar} \\
\text{IF } \chi^2 &> \chi^2_{\alpha,\nu} \text{ THEN } \text{array is dissimilar}
\end{align*}
\]
However, the chi-squared distribution method is not directly applicable to this study. Generally, this analysis method is applied to discontinuous data sets such as subjective survey data that can be organized into a histogram format. Or, the chi-squared distribution testing may be applied to a small data set to find if the samples are representative of a larger known data source [16]. Since a direct application to simulation analysis is not generally considered the chi-squared distribution testing method is omitted from this study.

Another broad range of testing that was considered for analysis purposes is the sign test. The sign test is a nonparametric procedure used with two related samples to test the hypothesis that the distributions of two variables are the same. This test makes no assumptions about the shape of these distributions [22]. Another test, which is a variant of the sign test, is the Wilcoxon signed-rank test. This test incorporates information about the magnitude of the differences and is therefore more powerful than the sign test. For this test the difference between any two “before” and “after” data points are ranked based on magnitude. The sum of the positive, “$Z_+$”, and negative, “$Z_-$”, ranks is calculated [25]. The Wilcoxon statistic is calculated based on this ranking system. The positive Wilcoxon statistic equation is:

$$W = \frac{Z_+ - \mu}{\sigma}$$

where “$\mu$” is the mean and the square root of the variance is “$\sigma$”. Next, the same procedure for the chi-squared distribution and independent samples t-test is performed: the null hypothesis is examined [26].

The Wilcoxon signed-rank test is the nonparametric version of the independent samples t-test. This test is appropriate when the independent samples t-
test is to be conducted but the dependent variable is not normally distributed [27]. When the number of data samples is greater than or equal to eight the sampling distribution of \( W \) is a reasonably close approximation of the normal distribution [25]. Therefore, this test is very similar to the independent samples t-test as there is a minimum of 16 data samples for any analysis in this study. The Wilcoxon signed-rank test is appropriate for testing in this study; however, in this case, this test is similar to parametric distribution tests. Therefore, this test is repetitive and unnecessary.

3.6. Summary

The methodology may be divided into several broad components to determine the implications of origin-destination distribution in freeway simulation. Firstly, a set of varying origin-destination distributions is developed based on a given data set. Next, a base set of these data is chosen to use for the development of a network simulation. The network parameters are adjusted in an iterative tuning process to accurately simulate a given freeway system. The tuning process may require adjusting simulation parameters such as the freeway geometry and parameters that affect driving behavior. Once the OD distribution matrices are created and the simulations are complete the results are analyzed. There are many forms of analysis which are applicable to the many forms of data. The outputs are then subjected to several statistical testing methods and graphical comparisons.
CHAPTER 4
EXPERIMENT DEVELOPMENT

4.1. Introduction
To analyze the implications of origin-destination distribution in freeway simulation the methodology was applied to a real freeway network. Every freeway network has similarities and differences compared to other freeway systems. Therefore, every simulation is unique and requires special consideration. For this case several components are required in the experiment development. These components include the location of the experiment, the development and implementation of the five OD distribution methods, and the simulation development.

4.2. Research Location
The freeway system to be analyzed is the Honolulu H-1 freeway in the westbound direction between the Honolulu International Airport (East of Exit 15B to Hickam AFB/Pearl Harbor) and Waiekele (West of Exit 7 to Waiekele) forming an 11 mile (15 km) length of mainline freeway. This segment of the H-1 freeway includes a total of 17 origins and destinations. There are eight origins and nine destinations consisting of a mainline origin, a mainline destination, seven on-ramps, and eight off-ramps.

Nearly all of the traffic counts of vehicles entering and exiting the freeway are from Tuesday, February 5, 2002. The only exceptions are the data collected at the Kamehameha Highway on-ramp and the data collected as vehicles entered the system at the airport viaduct. These traffic volumes were counted on Wednesday, November
19, 2003 and Tuesday, November 18, 2003, respectively. These data were utilized because the February 5, 2002 data were flawed and were discarded. The time span of the analysis is between 3:00pm and 7:00pm in the afternoon during the peak volume and congestion period. The volume count data are partitioned into consecutive 15 minute increments. There are a total of 16 time periods that occur during this 4 hour span. The freeway vehicle count data are collected and prepared in a tabular format. The vehicle count data for this study are exhibited in Appendix A.

4.3. Distribution Development

One of the objectives of this study is the estimation of the origin-destination distribution matrices. There are five OD methods that are subject to testing. Therefore, five distribution techniques were developed. Each of these techniques requires varying levels of data input. The deterministic and proportionate distributions are generated using the WatsonOD module in Microsoft Excel. The QueensOD application is used to develop the QueensOD (with default seed) distribution. The QueensOD (with custom seed) origin-destination distribution utilizes both the WatsonOD module and QueensOD application as this distribution incorporates both proportionate and synthetic optimization methods. The FREQ output distribution is independent of other models and only requires the simulation of the FREQ model to generate the appropriate distribution.

4.3.1. WatsonOD Module

The WatsonOD module was developed in order to generate freeway origin-destination matrices and INTEGRATION input files. The focus in the development process was to make a dynamic module that is easy to use and understand. For this
purpose the module was developed to replicate and interact with a Microsoft Excel spreadsheet. WatsonOD was developed using the Visual Basic for Applications (VBA) programming language. The VBA language is a trimmed down version of the Visual Basic programming language. VBA is designed to work with Microsoft Office products [28]. Visual Basic for Applications was a suitable tool for this application because VBA incorporates event-driven programming. Also known as event procedures, event-driven programming allows the user to execute subroutines by imposing an event, such as a mouse click or pressing the “Enter” key [29]. This feature makes the program user-friendly because running subroutines is as simple as clicking on the “OK” or “Cancel” buttons on a user form. The WatsonOD module uses this feature to aid in the development of OD distribution matrices.

Adapting the WatsonOD module to Excel is accomplished by installing the module as a Microsoft Excel “Add-in” feature. This feature allows the user to access the module by introducing a WatsonOD icon to the Excel toolbars. This icon is displayed in Figure 4.1. Once the user clicks on the icon the initial subroutine is activated and the “Origin & Destination Wizard” user form appears. The user form is shown in Figure 4.2. The user form requires that the user makes or chooses six different inputs or options. These inputs and options are:

- **Origins** – The number of origins in the freeway network. The user may choose between 1 and 200 origins in the form of a numerical input.
- **Destinations** – The number of destinations in the freeway network. The user may choose between 1 and 200 destinations in the form of a numerical input. The maximum total of origins and destinations combined is 240.
• **Time Periods** – The total number of time periods to be analyzed. The user may choose between 1 and 1000 time periods in the form of a numerical input.

• **Distribution Type** – The type of distribution. The choice is made by choosing one of the two option buttons. The options are deterministic or proportionate. The distribution type is unique and cannot be changed once the subroutine is executed.

• **Input Type** – The type of input file desired. The choice is made by choosing one of the two option buttons. The options are INTEGRATION or VISSIM\(^1\). The input type is unique and cannot be changed once the subroutine is executed.

• **Generate Chart** – A checkbox that allows the user to generate a chart. The chart is a graphical display of the user inputs in the form of a linear freeway system. This feature is only sensitive to the type and number of each ramp and flow volumes.

**Figure 4.1 - WatsonOD Excel Toolbar Icon**

\(^1\) The VISSIM option is not explained herein. It was developed to assist in the use of the VISSIM model for a freeway simulation project for Hawaii DOT.
For all subsequent WatsonOD examples and figures the data from the sample network, Figure 3.2, are applied in a proportionate format. Also, for the purposes of this study the WatsonOD module will only be used for the INTEGRATION model.

Each of these categories features a tool tip to assist the user. After all of the inputs are entered into the user form the subroutine may be executed. Once the user clicks on the "OK" button a new Excel workbook is opened and three new worksheets are generated. Also, depending on the user form entries, a chart may be developed, as well. The worksheets are titled “Assumptions”, “Input Data”, and “Results”. The optional chart is titled “Visual”. The fill color of each of the cells within each worksheet is indicative of the cell function. This information is provided on the “Assumptions” sheet so that confusion may be avoided. Cells are protected so that the user has limited modification access. The user may only change cells that are designated as adjustable cells and do not have a fill color. Other cells may only be highlighted to allow for copying and pasting purposes. Otherwise, cell access and modification is prohibited.
Once the subroutine is executed the "Assumptions" worksheet is the first sheet the user encounters. The "Assumptions" sheet is shown in Figure 4.3. This sheet requires that the user enter information about the simulation parameters. This information is required for both the WatsonOD module and the INTEGRATION model. Several of these input parameters require knowledge of the simulation conditions, such as start time, and the INTEGRATION model, such as driver classes.

**Figure 4.3 - WatsonOD "Assumptions" Worksheet**

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<td>0</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>9</td>
<td></td>
<td>Last O&amp;D Pair Loading</td>
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</tr>
<tr>
<td>10</td>
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</tr>
<tr>
<td>11</td>
<td></td>
<td>Global Scaling Factor</td>
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<td></td>
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</tr>
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<td>12</td>
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<td>Random Vehicle Headway</td>
<td>100%</td>
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<td></td>
<td></td>
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</tr>
<tr>
<td>13</td>
<td></td>
<td>Driver Class 1</td>
<td>100%</td>
<td></td>
<td></td>
<td></td>
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<td>14</td>
<td></td>
<td>Driver Class 2</td>
<td>0%</td>
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<td></td>
<td></td>
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<td></td>
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<tr>
<td>15</td>
<td></td>
<td>Driver Class 3</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
<td>16</td>
<td></td>
<td>Driver Class 4</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
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<td>17</td>
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<td>0%</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>18</td>
<td></td>
<td>Total Demand Fraction</td>
<td>0%</td>
<td></td>
<td></td>
<td></td>
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<td></td>
</tr>
<tr>
<td>19</td>
<td></td>
<td>Passenger Car Equivalency</td>
<td>1.0</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Project Name</td>
<td>H1 O&amp;D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

After the user has completed the "Assumptions" sheet requirements the "Input Data" sheet may be accessed. The "Input Data" sheet is similar to a standard vehicle count data table. The "Input Data" sheet is shown in Figure 4.4. If using a spreadsheet program, the user may copy and paste the count data directly into the WatsonOD "Input Data" worksheet. This makes the input process fast and eliminates sources of error. The periods and corresponding time spans are entered automatically by the module. The user is also required to define whether each of the columns of data is an origin or a destination. To eliminate error in designating the ramp type
drop-boxes have been included so the user only has two options; “origin” or “destination”. Once the data column has been assigned a ramp type the fill color will change to show the input is a heading designation rather than an adjustable cell.

Figure 4.4 - WatsonOD “Input Data” Worksheet

Once all of the required “Input Data” sheet requirements have been met the user may proceed to the “Results” worksheet. This sheet is unique because it requires no input information. The only option available for the user is the choice of which time period to analyze. The “Results” worksheet displays the OD matrix and the corresponding INTEGRATION origin-destination input. The parameters box and the OD matrix are shown in Figure 4.5. The parameters box allows the user to choose a specific time period for analysis. The user may choose from all of the time periods included for the vehicle count data analysis. This information dictates which time period data are analyzed in the OD matrix. The matrix includes the number of trips between any two pairs and the sum of all trips for each origin and destination. This matrix is strictly visual and may not be altered by the user.
The second component included in the "Results" worksheet is the INTEGRATION origin-destination input section. An example is shown in Figure 4.6. This INTEGRATION input component incorporates the inputs and results from all three of the other worksheets and converts them into a format that is applicable to the INTEGRATION model (e.g. the requisite text input file). Ultimately, the WatsonOD module converts raw traffic count data into traffic loading information suitable for a traffic simulator. This process is dynamic as the INTEGRATION input automatically adjusts, in terms of values and size, to account for a variable OD matrix. That is, the number of rows required by the INTEGRATION input adjusts dynamically to accommodate each of the trip pair entries in the OD matrix. This range of cells is accessible to the user and may be copied and pasted into a file format suitable for the INTEGRATION model.
The “Visual” chart component is included as an optional feature. An example of the chart is shown in Figure 4.7. The chart is a basic linear representation of the freeway network that is being analyzed. The origins and destinations are numbered and represented by converging and diverging links, respectively. The incoming and outgoing flow volumes are represented on the ramps. The mainline volumes are included on the mainline links. All of these volumes are produced directly from the vehicle count data and have not been converted into an hourly flow rate format. The chart is included as a visual feature and does not have any other functionality. However, the chart does allow the user to view the data in a graphical format.

Figure 4.7 - WatsonOD “Visual” Chart
The WatsonOD module expedites the origin-destination distribution process while requiring a minimal amount of network information. The module can manipulate data into two different formats for two different simulation models. The WatsonOD module was developed in VBA and is the culmination of many subroutines. The entire “IntAssumptions” subroutine code has been included in Appendix B as a sample of the program code. The “IntAssumptions” subroutine is used to generate the “Assumptions” worksheet for the INTEGRATION input type.

4.3.2. QueensOD Simulation

QueensOD is an origin-destination distribution model that was developed by Professor Michel Van Aerde in conjunction with the INTEGRATION model [17]. QueensOD Release 2.10 is used to model all origin-destination estimation for this study. QueensOD is a DOS-based program that does not offer a user interface. Input data are introduced into the model via “.dat” file extension text input files. These input files may be developed in a simple word processing program, such as Microsoft Notepad. The data are tab separated and organized in a row and column structure. A sample Notepad “.dat” input file is shown in Figure 4.8. The model is initialized by running a line of code in a command prompt window. Once the model has finished several output files are generated. These files are similar in format to the input files and contain the estimation results which may be directly used as INTEGRATION input files.
QueensOD was created to support the INTEGRATION traffic network simulation model. The two models have the same data file structures and file formats [18]. Therefore, several of the same input files that are used in the INTEGRATION model may be used in the QueensOD model. Specifically, the “Node Characteristics” and “Link Characteristics” files are compatible. These two files are discussed in the subsequent INTEGRATION simulation section. However, QueensOD does require some unique input files that are not required for the INTEGRATION model. The three unique files are the “QueensOD Master Control” file, the “Observed Link Traffic Flows” file, and the “Seed O-D Demands” file. The seed file is only required for the QueensOD (with custom seed) distribution method. The other two input files are required for both the QueensOD (with custom seed) and QueensOD (with default seed) simulations.

In general, the QueensOD input is based partially on collected data and partially on assumption. Data input may include parameters such as flow volumes or link lengths. These data are provided and may be entered directly without any
assumption. However, the QueensOD input that does involve user assumptions must be considered on a case by case basis. That is, the analysis assumptions may be specific to any unique freeway network. The QueensOD input file that does require user assumptions is the master file. The most important assumption is the estimation methodology. For the purposes of this study a Least Poisson Error (LPE) model for link flow errors and seed matrix correlation is implemented. The LPE method is a compromise between the other two error methods that are available: the Least Squared Error (LSE) model and the Least Relative Error (LRE) method. The LPE method uses a combined technique to determine the error in terms of percentage and absolute magnitude. The LSE and LRE methods measure error directly in terms of percentage and absolute magnitude, respectively.

Other QueensOD input parameters include variables that can alter accuracy by minimizing computation time. Since computation time is not critical the accuracy is maximized by not restricting the number of iterations performed by QueensOD. The other QueensOD user input assumptions are not critical to the outcome of the model or are unknown. Therefore, the provided default variables are used in those situations. All of the required input files that are unique to the QueensOD model are included in Appendix C. This includes the "master control" file and the "observed link traffic flows" file. Both of these files apply to the first time period. The "Node Characteristics", "Link Characteristics", and optional "Seed O-D Demands" files are omitted from this appendix because they are covered in the INTEGRATION simulation section. The INTEGRATION simulation components are detailed in Section 4.4 and the corresponding subsections.
4.3.3. **FREQ Simulation**

The FREQ origin-destination distribution matrices applied to this study were developed by the FREQ12 software. The FREQ model provides a user interface for input such as traffic count data and geometry features. The output is generated in a text format, similar to QueensOD. A sample of the FREQ origin-destination distribution output is provided in Figure 4.9. The output requires manipulation to be converted from a text matrix format to an INTEGRATION input format.

**Figure 4.9 - Sample FREQ Output File**

4.4. **Simulation Development**

For this study the INTEGRATION version Release 2.30 model is utilized. The INTEGRATION model is similar to the QueensOD model in several ways. Although the two models are designed for different purposes they share many principles. For example, the model data structure and file format are identical. However, one of the major differences is in the development of the input files. The
INTEGRATION model requires a representation of a physical system whereas the QueensOD model is strictly based on data collection and manipulation.

4.4.1. Idealized Freeway

Whenever an object is analyzed, in an engineering perspective, some assumptions and simplifications must be made. In the case of freeway simulation a real network must be idealized into a format that can be interpreted by a computer. The INTEGRATION model requires that the H-1 freeway system be discretized into a network of links and nodes. These links and nodes must be introduced wherever the freeway has a parameter discontinuity. For example, a different link may be required at a freeway interchange or where a lane drop or lane transition may occur.

Therefore, the first task in simulating a network is gathering information about the freeway geometry. Detectors must be added to the simulation network, as well. These detectors, which collect traffic data, are located at five general freeway locations: the Airport, Radford Drive, Halawa Stream, Aiea Heights Road, and the Waimalu Street on-ramp. These locations are numbered one through five, respectively, and match locations where data were collected in the field.

Geometric characteristics for the H-1 freeway were collected prior to the onset of this study. This information was provided and used to model the freeway network. Figure 4.10 displays a detailed drawing of the idealized freeway network in a linear format. The circled red numbers represent node locations and blue numbers within squares represent links. The yellow numbers within hexagons are the locations of detectors. All link lengths are in terms of meters. The idealized H-1 freeway is a 16 origin-destination system (7 origins, 9 destinations).
The majority of physical network features are addressed in Figure 4.10. The number of lanes, link lengths, detector locations, etc. is characterized by this drawing. However, this network is in a linear format and does not represent the actual freeway geometry. The network was digitized onto a Honolulu road map to determine the required Cartesian coordinates. The units of measurement are in terms of x-y coordinates and the scale must be consistent. A graphic representation of the digitized network map is shown in Figure 4.11. Other system information is included on this drawing, as well. An index is provided with the figure to aid in interpreting the freeway system. The names of the ramps and corresponding nodes/links are included in Table 4.1. This information is directly applicable to the INTEGRATION model input files.

4.4.2. Fine Tuning

Tuning the simulation input parameters requires prior knowledge of prevailing traffic conditions of the H-1 freeway. In general, the most severe queuing is known to occur on the H-1 freeway upstream of the Pearl City/Waimalu exit (Node 47). This queuing begins to form quickly and propagate upstream for several kilometers. This congestion is responsible for the variable vehicle speeds at the detector locations. Despite the severe queuing and rapid propagation, the congestion is concentrated to the H-1 freeway and does not result in much queuing on any of the ramps.

When the H-1 freeway was initially simulated with base conditions the queuing behavior did not emulate the real traffic conditions. Severe congestion quickly propagated onto several on-ramps and the mainline freeway remained at free flow speed. As the tuning process progressed changes were made to select
Figure 4.10 - Idealized Linear Freeway Network
Table 4.1 - Ramp Names & Corresponding Node Numbers

<table>
<thead>
<tr>
<th>Ramp/Node No.</th>
<th>Type</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Origin</td>
<td>H-1 Mainline @ Airport</td>
</tr>
<tr>
<td>2</td>
<td>Destination</td>
<td>Hickam/Pearl Harbor</td>
</tr>
<tr>
<td>3</td>
<td>Destination</td>
<td>USS Arizona/Kamehameha Hwy.</td>
</tr>
<tr>
<td>4</td>
<td>Origin</td>
<td>Kamehameha Hwy.</td>
</tr>
<tr>
<td>5</td>
<td>Destination</td>
<td>H-3 Fwy./EB Moanalua Fwy.</td>
</tr>
<tr>
<td>6</td>
<td>Destination</td>
<td>Aiea/WB Moanalua Fwy.</td>
</tr>
<tr>
<td>7</td>
<td>Origin</td>
<td>H-3 Fwy.</td>
</tr>
<tr>
<td>8</td>
<td>Origin</td>
<td>Moanalua Fwy. and Aiea/Ulune St.</td>
</tr>
<tr>
<td>9</td>
<td>Destination</td>
<td>Pearl City/Waimalu</td>
</tr>
<tr>
<td>10</td>
<td>Origin</td>
<td>Waimalu</td>
</tr>
<tr>
<td>11</td>
<td>Destination</td>
<td>Waipahu</td>
</tr>
<tr>
<td>12</td>
<td>Destination</td>
<td>NB H-2 Hwy./Mililani</td>
</tr>
<tr>
<td>13</td>
<td>Origin</td>
<td>Kamehameha Hwy.</td>
</tr>
<tr>
<td>14</td>
<td>Origin</td>
<td>H-2 Fwy.</td>
</tr>
<tr>
<td>15</td>
<td>Destination</td>
<td>Waikele</td>
</tr>
<tr>
<td>16</td>
<td>Destination</td>
<td>H-1 Mainline @ Waikele</td>
</tr>
</tbody>
</table>
parameters such as free flow speed, speed at capacity, jam density, and lane capacity. INTEGRATION still did not simulate the H-1 conditions properly. Since parameter tuning alone was not sufficiently effective, two geometric "tricks" were introduced to improve the simulation.

When the H-1 freeway was initially simulated with base conditions the queuing behavior did not emulate the real traffic conditions. Severe congestion quickly propagated onto several on-ramps and the mainline freeway remained at free flow speed. As the tuning process progressed changes were made to select parameters such as free flow speed, speed at capacity, jam density, and lane capacity. INTEGRATION still did not simulate the H-1 conditions properly. Since parameter tuning alone was not sufficiently effective, two geometric "tricks" were introduced to improve the simulation.

The first alteration was the introduction of a very short (0.060 km) six-lane "dummy" link (Link 29) into the system. This link does not exist in the real system. The purpose of this "dummy" link is to force incoming on-ramp traffic into the mainline H-1 freeway and reduce ramp congestion which does not exist in reality. This gives the incoming on-ramp traffic higher priority to merge into the system which is consistent of the habit of Oahu drivers to yield to on-ramp traffic.

The second "trick" occurred just upstream of this merge (Link 28). The purpose of this second alteration is to bias traffic to the leftmost lanes prior to the merge. This was accomplished by introducing lane striping to a certain link. For this
scenario striping is added to the network to alert drivers for the upcoming merge. A computer image of the INTEGRATION simulation "tricks" is shown in Figure 4.12.

The model required extremely high lane capacities for proper simulation. The lane capacity on some links was as high as 3600 vphpl (vehicles per hour per lane). The common value used for lane capacity in freeway simulation is between 1900 and 2300 vphpl. These tuning modifications greatly improve the traffic simulation. As a result, the simulation accurately replicated the real traffic conditions for the given conditions as presented in Section 5.5 and the corresponding subsections.

**Figure 4.12 - Modified INTEGRATION Simulation**

The INTEGRATION model requires six input files to run properly. The six files required for this simulation were the "INTEGRATION Master Control", "Node Characteristics", "Link Characteristics", "Signal Timing", "Origin-Destination Traffic Demands", and "Incidents" files. Two optional files were used in this simulation: "Lane Striping" and "Detector Location". All eight of the INTEGRATION model input files are included in Appendix D. These are the input files that were used to simulate the first time period using the OD distribution data of the proportionate method.
4.5. Summary

Several preparatory measures were required to develop this experiment. A physical freeway system and raw traffic data were interpreted into a computer format for two existing models: QueensOD and INTEGRATION. This process required the discretization of a segment of the H-1 freeway. The network was idealized and an iterative tuning process was applied to produce an accurate freeway simulation. Another dynamic program, the WatsonOD module, was developed specifically to be applied to the development of this experiment. The application of this program greatly reduced various sources of origin-destination distribution errors. Also, the WatsonOD module made the development of the INTEGRATION origin-destination distribution input file quick and easy. The FREQ model data were obtained from a previous experiment which modeled the same freeway system. These preparatory measures used for this experiment development were essential for determining the implications of OD distribution in freeway simulation.
CHAPTER 5
RESULTS AND ANALYSIS

5.1. Introduction
To determine the implications of origin-destination distribution in freeway simulation two sets of results must be analyzed: the distribution results and the simulation results. The distribution results are used as input information to generate simulation results. The analysis consists of stochastic testing and visual inspection. The statistical portion of the analysis is applied to both the distribution results and the simulation results. This analysis is performed using a variety of analysis methods. The visual inspection pertains to the INTEGRATION simulation screen display and comparative data plots of the simulation results. The observation analysis is performed strictly for discussion purposes. This visual component is not intended for any statistical purposes and is not used to form any conclusions.

5.2. Distribution Results
The complete OD distribution results for all 16 time periods are included in Appendix E. The results are displayed in an Excel spreadsheet format. All traffic volumes are in vehicles per hour. A volume of zero is applied to any origin-destination pair that does not have any flow interaction for a given time period. The number of significant figures applied to each trip pair varies. For example, all proportionate volumes produced by the WatsonOD module are rounded to the nearest whole vehicle. Meanwhile, the QueensOD model produces flow volumes to the
nearest tenth of a vehicle. These exact values, including significant figures, are entered as provided by each distribution program.

5.3. Distribution Analysis

The purpose of the distribution analysis is to determine if the origin-destination distribution methods produce similar results. Each of the 16 time periods are analyzed as a single array of data. Each of the five distribution arrays are a 544 class string of data. The two forms of analysis applied to the origin-destination distribution results are the regression analysis and the Pearson's correlation analysis. A plot of a data comparison is provided in Figure 5.1. The data points in this plot are the proportionate OD distribution and the QueensOD (with default seed) origin-destination distribution. The corresponding linear regression line is included, as well. The dashed diagonal line on this graphical display represents the ideal line where all data points would follow if both methods had generated identical OD inputs.

Nevertheless, the agreement between these two origin-destination distributions is relatively good as the regression slope coefficient, \( \beta \), is 0.880 and Pearson's correlation coefficient, \( R \), is 0.930.

Tables 5.1 and 5.2 are in the form of results matrices. The top and leftmost columns and rows contain the name of each distribution technique. The location where any two methods intersect contains a value. Table 5.1 represents the slope coefficients of the linear regression line for every combination of the OD distribution techniques. A slope and correlation equal to 1.00 represents a perfect match where the intercept is zero, i.e., regression line and data point configuration at 45 degrees
through the origin. Table 5.2 represents the corresponding Pearson’s correlation coefficients.

**Figure 5.1 – Linear Distribution Regression Plot**

![OD Distribution Plot](image)

**Table 5.1 - OD Distribution Regression Analysis**

<table>
<thead>
<tr>
<th>Distribution Regression</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.270</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.257</td>
<td>0.880</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.267</td>
<td>0.953</td>
<td>0.997</td>
<td></td>
</tr>
<tr>
<td>FREQ</td>
<td>0.270</td>
<td>1.000</td>
<td>0.984</td>
<td>1.023</td>
</tr>
</tbody>
</table>

**Table 5.2 - OD Distribution Correlation Analysis**

The results from the distribution regression analysis indicate that the deterministic method produced much different OD inputs than the other four methods. Conversely, the proportionate and FREQ output methods exhibit a perfect relationship in terms of the slope of the linear regression line. The remaining five OD distribution combinations have slope values near 1.00. The same relationships are prevalent in the distribution correlations. The deterministic distribution appears to be
much different than the other four methods. The origin-destination data of the
deterministic method are scattered while the other six distribution combinations are
much closer to 1.00. In fact, the FREQ output and proportionate data types produce a
Pearson's correlation that is exactly 1.00.

Based on regression slope and Pearson's correlation the relationships between
the five distribution techniques are determined. The origin-destination data produced
by the deterministic method are clearly different from the other four methods. The
proportionate and FREQ origin-destination distribution output is statistically
identical, therefore, these distribution outcomes are likely the exact same or closely
similar. The QueensOD (with default seed) and proportionate combination appear to
be slightly dissimilar in relation to the other combinations.

5.4. Simulation Results

The simulation results used for analysis were two MOE: total vehicle travel
time in the network and average vehicle spot speeds at five cross-sections for which
field data are available. These results are available for analysis once the simulation is
complete. The visual output generated by INTEGRATION during the simulation is
also a resource for comparison. An example of an INTEGRATION screen image is
shown in Figure 5.2. This image is from the proportionate distribution simulation.
The image is from 1.50 hours (5:30 PM) into the simulation. The red lines along the
freeway network indicate the queuing lengths. The green lines indicate that vehicles
are moving at a rate near the free flow speed.
5.4.1. Vehicle Speeds

Vehicle spot speeds were detected at five locations. The placement of the detectors at each location is included in Figure 4.10. Each location has varying amounts of contemporaneous field data, as well. The five data sets are available in two formats: tabular and graphical. Both formats provide all existing data, contemporaneous or simulated, for comparison. The QueensOD model is referred to by the abbreviation “QOD”. The three forms of plotted data are subdivided into three distinct formats. All of the INTEGRATION simulation results are defined by a heavy line weight. The other simulation data, produced using FREQ and KRONOS, are distinguishable by a dashed line type. All contemporaneous data may be distinguished by a standard line type and weight. Each data set is plotted with a unique line color that is consistent for all plots.

Vehicle speed plots are available for five detector locations on the H-1 Freeway: the Airport, Radford Drive, Halawa Stream, Aiea Heights Road, and the
Waimalu Street on-ramp. These locations consecutively progress downstream from the Airport detector to the Waimalu detector. The tabular simulation results and collected contemporaneous data are included as Appendix F. The five corresponding data plots are included in the subsequent analysis sections.

5.4.2. Vehicle Travel Time
The only tool available for collecting vehicle travel time data is traffic simulation. Simulation is the primary source for producing a quantifiable value. Therefore, no contemporaneous data are available for comparison. All five of the INTEGRATION simulations and the FREQ and KRONOS simulations produce vehicle travel time data in units of vehicle-hours. All of the tabular simulation results are included with the vehicle speeds results as Appendix F. The corresponding plot is included in the subsequent analysis section. The vehicle travel time data are subdivided and plotted in the same format as the vehicle speeds data.

5.5. Simulation Analysis
The analysis of each of the five simulation techniques consists of two components: visual inspection and statistical comparison. The visual inspection is for comparison but does not facilitate conclusions. During preliminary model calibration visual inspection is critical because it reveals where the model generates congestion (queues) and how far it propagates them, which, in turn can be compared with general field conditions.

The statistical analysis consists of several different methods. This analysis is intended to determine the implications of each of the five OD techniques in traffic
simulation. The tests pertain to the simulated results of the five OD distribution methods in conjunction with a single contemporaneous field data set.

The linear regression and Pearson's correlation results are provided in the form of results matrices. The results are presented in the same format as in previous sections. The criterion analysis results for the five techniques are in the form of a table. The values within each table represent the difference in vehicle speeds between the contemporaneous moving observer (MO) data and the simulation data. These results are in an absolute form. For the Airport, Radford Drive, Aiea Heights Road, and Waimalu Street on-ramp data collection locations the speed difference is in comparison to the corresponding moving observer data. The only exception is for the Halawa Stream cross-section location. There are no moving observer data at the Halawa Stream location. Internet data from in-pavement loops were used for comparison. None of the three Internet data sets correspond to the simulation. Freeway data were collected in February 2002, and the Halawa Stream Internet/loop data station was developed in May 2002. An average of three days of data is used for the Halawa Stream contemporaneous data set.

Included in each table is the number of occurrences where the vehicle speeds for each method is within a given range. For this study the default speed range is ±10 mph. Generally, this value is a good measure of similar behavior in a freeway network. If the difference between the contemporaneous and simulation data is within the ±10 mph range at least 75% of the time the results are considered to be similar. This rule is arbitrary but given the length, congestion level, and capacity of the test network this rule was deemed appropriate for differentiating among the five
distribution methods. In the subsequent discussion this rule is referred to as the 10/75 criterion.

5.5.1. Vehicle Speeds - Airport

The furthest upstream point for data collection is the Airport location. The Airport vehicle speeds plot in shown in Figure 5.3. The moving observer and Autoscope data suggest that the Airport section is free of any congestion throughout the entire simulation. The Autoscope data remain at a nearly constant value. The MO data experience some deviations from free flow speed. However, these deviations are minor and the moving observer remains above 50 mph for the majority of the time periods.

The majority of the INTEGRATION results are near free flow speed for all of the time periods. These results are visibly similar to the Autoscope and moving observer data. One set of simulation data, the deterministic simulation, deviate from the other vehicle speeds data. Approximately 1.75 hours (5:45 PM) into simulation the deterministic distribution exhibits considerable queuing. This behavior continues through the end of the simulation. These findings would suggest that the queuing at the Airport detector does not alleviate prior to 7:00 PM. The other four distribution methods are all similar to one another.

The analysis results for the Airport cross-section are shown in Table 5.3. These statistical results correspond to the visual observations. Every distribution method, except for the deterministic method, fulfills the 10/75 criterion. All four of the occurrence results are much higher than the criterion of 75%. However, the deterministic results fail with only 6 out of a possible 16 occurrences. The four
successful methods still exceed the 75% criterion if the speed range is narrowed to ±8 mph.

Figure 5.3 – Vehicle Speeds at Airport Cross-Section

![Graph showing vehicle speeds at an airport cross-section with various data points and lines indicating different analysis methods.]

Table 5.3 - Airport Vehicle Speeds 10/75 Criterion Analysis

<table>
<thead>
<tr>
<th>Time Slice</th>
<th>Time</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>#</td>
<td>Time</td>
<td></td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>1</td>
<td>15:15</td>
<td>6</td>
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<td>5</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>15:30</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>15:45</td>
<td>6</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>16:00</td>
<td>6</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>16:15</td>
<td>6</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>16:30</td>
<td>7</td>
<td>8</td>
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<td>8</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>16:45</td>
<td>39</td>
<td>5</td>
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<td></td>
</tr>
<tr>
<td>8</td>
<td>17:00</td>
<td>43</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>17:15</td>
<td>43</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>17:30</td>
<td>42</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>11</td>
<td>17:45</td>
<td>43</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>12</td>
<td>18:00</td>
<td>43</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>13</td>
<td>18:15</td>
<td>43</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>14</td>
<td>18:30</td>
<td>32</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>18:45</td>
<td>38</td>
<td>9</td>
<td>9</td>
<td>9</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>19:00</td>
<td>38</td>
<td>10</td>
<td>10</td>
<td>11</td>
<td>10</td>
</tr>
<tr>
<td>No. of Occurrences</td>
<td>6</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>% within Range</td>
<td>38%</td>
<td>88%</td>
<td>88%</td>
<td>88%</td>
<td>88%</td>
<td></td>
</tr>
</tbody>
</table>

The Pearson’s correlation results are shown in Table 5.4. Similarly to the other forms of analysis, the comparisons that include the deterministic data are different. In fact, the resulting coefficient is a negative value for all four instances.
The other six method comparisons result in similar correlation in comparison with minor deviations from 1.00.

Table 5.4 - Airport Vehicle Speeds Correlation

<table>
<thead>
<tr>
<th>Airport Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>-0.408</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Default</td>
<td>-0.322</td>
<td>0.906</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>-0.351</td>
<td>0.933</td>
<td>0.917</td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>-0.141</td>
<td>0.836</td>
<td>0.918</td>
<td>0.888</td>
</tr>
</tbody>
</table>

5.5.2. Vehicle Speeds – Radford Drive

The next downstream detector collects data at the Radford Drive overpass location. The Radford Drive average vehicle speeds plot is shown in Figure 5.4. The moving observer and Autoscope data plots both exhibit a similar behavior. At approximately 16:15 (4:15 PM) both contemporaneous data sets begin to queue. The sudden reduction in vehicle speeds is indicative of queuing at the Radford Drive cross-section. In both cases, the queuing begins to relieve at 18:15 (6:15 PM). The Autoscope data maintain levels of congestion to a degree until 18:30 (6:30 PM) whereas the MO congestion dissipates immediately at 18:15 (6:15 PM).

Most of the Radford Drive simulation results observe a similar dip in average vehicle spot speeds. This is comparable to the contemporaneous data. Such a speed reduction suggests that each of the five INTEGRATION simulations, and the real freeway network, experiences queuing at this location. However, the duration of this congestion is variable. For example, the proportionate distribution results in congestion for approximately 1.50 hours from 16:30 to 18:00 (4:30 PM to 6:00 PM). Meanwhile, the QueensOD (with default seed) distribution merely results in a brief
spike in vehicle speeds at 17:15 (5:15 PM). All of the distribution methods result in a varying queuing duration. The most extreme case is the deterministic method. The deterministic simulation results in the earliest congestion which begins at approximately 16:00 (4:00 PM). Similar to the Airport results, this queuing does not alleviate at any point throughout the simulation.

**Figure 5.4 – Vehicle Speeds at Radford Drive Cross-Section**

![Radford Speeds](image)

The analysis results for the Radford cross-section are shown in Table 5.5. The proportionate, QueensOD (with custom seed), and the FREQ output methods are all within ±10 mph of the moving observer data for 14 of the 16 time periods. Unlike the Airport cross-section, the QueensOD (with default seed) does not meet the 10/75 criterion at the Radford cross-section. The deterministic method does not meet this criterion either. The three passing methods all maintain similar results for 81% of the time periods when the range is limited to only ±3 mph.

The Pearson’s correlation results for the Radford cross-section are shown in Table 5.6. The correlation between the deterministic technique average spot speeds
data and the other four corresponding methods is very poor. Every technique correlates poorly with the QueensOD (with default seed) data, as well. The correlation of the other three methods is very high when compared to one another. The other two data sets, deterministic and QueensOD (with default seed) appear to be the outliers but do not correlate with each other.

Table 5.5 - Radford Drive Vehicle Speeds 10/75 Criterion Analysis

<table>
<thead>
<tr>
<th>Time Slice</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>#</td>
<td>Time</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>18:16</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>18:30</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>18:45</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>18:00</td>
<td>37</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>5</td>
<td>18:15</td>
<td>16</td>
<td>20</td>
<td>31</td>
<td>30</td>
</tr>
<tr>
<td>6</td>
<td>18:30</td>
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<td>2</td>
<td>43</td>
<td>6</td>
</tr>
<tr>
<td>7</td>
<td>18:45</td>
<td>1</td>
<td>3</td>
<td>46</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>18:00</td>
<td>3</td>
<td>0</td>
<td>25</td>
<td>1</td>
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<tr>
<td>9</td>
<td>17:15</td>
<td>3</td>
<td>0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>17:30</td>
<td>3</td>
<td>1</td>
<td>11</td>
<td>0</td>
</tr>
<tr>
<td>11</td>
<td>17:45</td>
<td>3</td>
<td>1</td>
<td>46</td>
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<tr>
<td>12</td>
<td>18:00</td>
<td>9</td>
<td>6</td>
<td>41</td>
<td>30</td>
</tr>
<tr>
<td>13</td>
<td>18:15</td>
<td>49</td>
<td>16</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>14</td>
<td>18:30</td>
<td>49</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>15</td>
<td>18:45</td>
<td>49</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>16</td>
<td>19:00</td>
<td>50</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

| No. of Occurrences | 10 | 14 | 9 | 14 | 14 |

| % within Range    | 63% | 88% | 66% | 88% | 88% |

Table 5.6 - Radford Drive Vehicle Speeds Correlation

<table>
<thead>
<tr>
<th>Radford Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.486</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.178</td>
<td>0.501</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.388</td>
<td>0.905</td>
<td>0.816</td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>0.410</td>
<td>0.960</td>
<td>0.594</td>
<td>0.950</td>
</tr>
</tbody>
</table>

5.5.3. Vehicle Speeds – Halawa Stream

The next data collection point in the freeway network is the location by Halawa stream detector. The Halawa Stream vehicle speeds plot is shown in Figure 5.5. The Halawa stream cross-section does not have moving observer or Autoscope
contemporaneous data. However, sets of Internet data are available and were averaged into a single speed profile. With the exception of the very beginning and very end of the analysis period, the Internet speed data reveal slow speeds as the network is congested. Between 15:45 (3:45 PM) and 18:15 (6:15 PM) the speed is stable between 17 mph and 21 mph.

All five of the distribution methods quickly result in congestion. This behavior is similar to the pattern of the contemporaneous data. The deterministic simulation is the first to become congested. Again, this method remains congested through the end of the simulation. The other four methods begin to simulate queuing at approximately 16:15 (4:15 PM), about 30 minutes after the deterministic simulation. The congestion begins to clear between 18:00 and 18:30 (6:00 PM and 6:30 PM). Both QueensOD simulations spike upward in speeds at 17:30 (5:30 PM). They both immediately return to slower speeds at the following time period. Both simulations with QueensOD data are volatile and exhibit varying levels of congestion.

The analysis results for the Halawa stream cross-section are shown in Table 5.7. These results correlate with the plotted data as the contemporaneous data differs from the simulation data. At the Halawa stream cross-section only the proportionate method fulfills the 10/75 criterion. The proportionate results still meet the 75% criteria even when the speed range is narrowed to ±9 mph. None of the other four methods exceed 11 occurrences for the ±10 mph range. The source for these poor results may be partly attributed to the data used for comparison. The Internet data are an average of several data sets which were not collected at the same time as the moving observer data. Notably, moving observer and freeway volume data were
collected only six months after 9/11/2001. As shown in Figure 5.5, speed data collected in 2003 display extensive (over time) and severe (low speed) congestion, which was not the case in February, 2002.

**Figure 5.5 – Vehicle Speeds at Halawa Stream Cross-Section**

![Halawa Speed Graph](image)

**Table 5.7 - Halawa Stream Vehicle Speeds 10/75 Criterion Analysis**

<table>
<thead>
<tr>
<th>Time Slice</th>
<th>Halawa - Compared to Internet (Avg) Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>#</td>
<td>Time</td>
</tr>
<tr>
<td>1</td>
<td>15:15</td>
</tr>
<tr>
<td>2</td>
<td>15:30</td>
</tr>
<tr>
<td>3</td>
<td>15:45</td>
</tr>
<tr>
<td>4</td>
<td>16:00</td>
</tr>
<tr>
<td>5</td>
<td>16:15</td>
</tr>
<tr>
<td>6</td>
<td>16:30</td>
</tr>
<tr>
<td>7</td>
<td>16:45</td>
</tr>
<tr>
<td>8</td>
<td>17:00</td>
</tr>
<tr>
<td>9</td>
<td>17:15</td>
</tr>
<tr>
<td>10</td>
<td>17:30</td>
</tr>
<tr>
<td>11</td>
<td>17:45</td>
</tr>
<tr>
<td>12</td>
<td>18:00</td>
</tr>
<tr>
<td>13</td>
<td>18:15</td>
</tr>
<tr>
<td>14</td>
<td>18:30</td>
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<tr>
<td>15</td>
<td>18:45</td>
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<tr>
<td>16</td>
<td>19:00</td>
</tr>
<tr>
<td>No. of Occurrences</td>
<td>11</td>
</tr>
<tr>
<td>% within Range</td>
<td>69%</td>
</tr>
</tbody>
</table>

The Pearson’s correlation results for the Halawa stream cross-section are shown in Table 5.8. The correlation between the deterministic method average
vehicle spot speeds data and the other four corresponding methods are weak. The correlation among the other four methods is relatively strong. Among those the lowest coefficient value is 0.812 for proportionate and QueensOD (with default seed) combination. The highest coefficient value is 0.966 between the QueensOD variants.

### Table 5.8 - Halawa Stream Vehicle Speeds Correlation

<table>
<thead>
<tr>
<th>Halawa Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>OOD Default</th>
<th>OOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
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<td></td>
<td>0.394</td>
<td>0.812</td>
</tr>
<tr>
<td>OOD Default</td>
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<td>0.812</td>
<td>0.411</td>
<td>0.865</td>
</tr>
<tr>
<td>OOD Custom</td>
<td>0.411</td>
<td>0.865</td>
<td>0.966</td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>0.452</td>
<td>0.934</td>
<td>0.924</td>
<td>0.944</td>
</tr>
</tbody>
</table>

#### 5.5.4. Vehicle Speeds – Aiea Heights Road

The Aiea Heights Road overpass detector station is located just downstream of the convergence of two high volume freeways, the H-1 and the Moanalua freeway. The Aiea Heights Road average vehicle spot speeds plot is shown in Figure 5.6. The congestion responds accordingly as the freeway segment becomes congested soon after the simulation is initiated. This network behavior is reflected by the contemporaneous data. The moving observer and both Autoscope data sets become congested quickly and relieve late. The MO speed begins to queue at 15:30 (3:30 PM) and doesn’t relieve until 18:30 (6:30 PM). Both Autoscope speed profiles behave similarly with some slight variation from the moving observer data.

The deterministic simulation becomes congested first and the QueensOD (with default seed) begins to queue last. The queuing behavior of all five of the distribution techniques is similar to the contemporaneous data. As is the case for the three upstream detector locations, the deterministic simulated queuing fails to
alleviate by 19:00 (7:00 PM). For the remaining four simulation methods the queuing begins to dissipate between 18:15 and 18:30 (6:15 PM and 6:30 PM). Despite differences in the length of queue time spans all five methods perform similarly while in a state of congestion. While vehicles are in a queue the speeds remain between approximately 16 mph and 20 mph. These speeds are consistent and are within the range of the contemporaneous speeds.

Figure 5.6 – Vehicle Speeds at Aiea Heights Road Cross-Section

The analysis results for the Aiea Height Road cross-section are shown in Table 5.9. All five of the distribution techniques exhibit similarities to the moving observer data in simulation. They all meet or exceed the 10/75 criterion for similarity to contemporaneous data. The QueensOD (with custom seed) data set has the greatest number of occurrences (14). The deterministic and proportionate data sets have the fewest number of occurrences (12). However, unlike the three preceding speed cross-section locations the five methods all are similar to the moving observer data. The deterministic, QueensOD (with custom seed), and FREQ output techniques
have speeds similar to field speeds for 12 time periods when the speed criterion is reduced to ±5 mph. The proportionate and QueensOD (with default seed) do not meet the 75% criterion at the reduced speed range.

Table 5.9 - Aiea Heights Road Vehicle Speeds 10/75 Criterion Analysis

<table>
<thead>
<tr>
<th>Time Slice</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>116:15</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
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<tr>
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<tr>
<td>316:45</td>
<td>1</td>
<td>36</td>
<td>41</td>
<td>33</td>
<td>36</td>
</tr>
<tr>
<td>416:00</td>
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<td>4</td>
<td>24</td>
<td>4</td>
<td>4</td>
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<td>5</td>
</tr>
<tr>
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<td>4</td>
<td>5</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>817:00</td>
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<td>5</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
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<td>5</td>
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<td>4</td>
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<td>4</td>
<td>7</td>
<td>8</td>
<td>8</td>
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<tr>
<td>1117:45</td>
<td>4</td>
<td>5</td>
<td>4</td>
<td>5</td>
<td>4</td>
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<td>3</td>
<td>3</td>
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<tr>
<td>1619:00</td>
<td>44</td>
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<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

The Pearson’s correlation results for the Aiea Heights Road cross-section are shown in Table 5.10. Again, the correlation between the deterministic technique speeds data and the other four corresponding methods is low. The correlation among the other four methods is relatively high. The correlations between the proportionate and the two QueensOD methods are slightly lower at 0.791 and 0.816.

Table 5.10 - Aiea Heights Road Vehicle Speeds Correlation

<table>
<thead>
<tr>
<th>Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.631</td>
<td>0.434</td>
<td>0.977</td>
<td>0.903</td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.543</td>
<td>0.946</td>
<td>0.901</td>
<td>0.933</td>
</tr>
</tbody>
</table>

86
5.5.5. Vehicle Speeds – Waimalu Street on-ramp

The final, and furthest downstream, data collection point is a location 500 feet upstream of the Waimalu Street on-ramp. The Waimalu Street on-ramp average vehicle spot speeds plot is shown in Figure 5.7. The contemporaneous data collected at the Waimalu Street on-ramp cross-section are similar to the contemporaneous data collected at the Airport cross-section. The moving observer data are free of congestion throughout the entire analysis. As a result the vehicle speeds remain above 50 mph for all 16 time periods.

Note that at the previous location, Aiea Heights Road, the freeway is 8 lanes wide and within one mile the freeway drops to 5 lanes (by Kaonohi Street overpass). This is a major bottleneck in this direction of the freeway and currently the Hawaii State Department of Transportation is conducting the “Waimalu widening project” to add a lane between the Kaonohi bottleneck and the Waimalu Street off-ramp. This station is at the midpoint between the Waimalu Street off-ramp and on-ramps.

The vehicle speeds produced at this location are near free flow speed for all five distribution simulations and correspond closely with the contemporaneous data. No queuing exists on this portion of the freeway throughout the entire simulation. This is the only location where the deterministic simulation does not produce any queuing.

The analysis results for the Waimalu Street on-ramp cross-section are shown in Table 5.11. All five of the distribution techniques fulfill the 10/75 criterion of similarity. In fact, all five of the comparative data sets are within the speed range for every time period. This is the only cross-section where any, let alone all, of the five
distribution methods have similar occurrences for all 16 time periods. If the speed range is narrowed to ±8 mph the deterministic data no longer meet the 75% criteria. If the speed range is slightly lowered, to ±7 mph, the proportionate and both QueensOD methods exceed the 12 occurrences threshold. Once below this speed range, none of the methods meet the minimum 75% criteria.

The Pearson’s correlation results for the Waimalu Street on-ramp cross-section are shown in Table 5.12. This cross-section has the lowest correlation coefficients for the deterministic data. However, unlike the other four cross-section locations at Waimalu the deterministic data are only slightly lower than the other results. In fact, the correlation among all of the methods is relatively low. This may be due to the heavy concentration of data points at one value and likely makes the regression and corresponding errors sensitive to outlying data points.
Table 5.11 – Waimalu Street on-ramp Vehicle Speeds 10/75 Criterion Analysis

<table>
<thead>
<tr>
<th>Time</th>
<th>Waimalu - Compared to Moving Observer Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>#</td>
<td>Time</td>
</tr>
<tr>
<td>1</td>
<td>16:15</td>
</tr>
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<td>2</td>
<td>16:30</td>
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<tr>
<td>3</td>
<td>16:45</td>
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<td>4</td>
<td>16:00</td>
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<tr>
<td>5</td>
<td>16:15</td>
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<tr>
<td>6</td>
<td>16:30</td>
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<tr>
<td>7</td>
<td>16:45</td>
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<tr>
<td>8</td>
<td>17:00</td>
</tr>
<tr>
<td>9</td>
<td>17:15</td>
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<tr>
<td>10</td>
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<tr>
<td>11</td>
<td>17:45</td>
</tr>
<tr>
<td>12</td>
<td>18:00</td>
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<tr>
<td>13</td>
<td>18:15</td>
</tr>
<tr>
<td>14</td>
<td>18:30</td>
</tr>
<tr>
<td>15</td>
<td>18:45</td>
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<td>19:00</td>
</tr>
<tr>
<td>No. of Occurrences</td>
<td>16</td>
</tr>
<tr>
<td>% within Range</td>
<td>100%</td>
</tr>
</tbody>
</table>

Table 5.12 – Waimalu Street on-ramp Vehicle Speeds Correlation

<table>
<thead>
<tr>
<th>Waimalu Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.776</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.668</td>
<td>0.825</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.728</td>
<td>0.847</td>
<td>0.928</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>0.604</td>
<td>0.815</td>
<td>0.787</td>
<td>0.717</td>
<td></td>
</tr>
</tbody>
</table>

5.5.6. Vehicle Travel Time

The vehicle travel time outcomes characterize the entire network. The complete vehicle travel time plot is shown in Figure 5.8. The travel time data plots for all five INTEGRATION simulations produce a similar concave shape which is similar to those produced by the FREQ and KRONOS models. This suggests that travel times within the network peak at approximately 17:00 (5:00 PM). This is consistent with the vehicle speeds as they are often at their lowest at this time. For this simulation the average vehicle spot speeds and network vehicle travel time
Statistics are interrelated. Towards the beginning and end times of the simulations the congestion alleviates and the network vehicle travel time is reduced.

As is prevalent in the vehicle speeds data, visibly the deterministic simulation is the most removed from the other four methods for vehicle travel time, as well. Unlike the other distribution simulations, the deterministic method queues failed to alleviate between 5:00 PM and 7:00 PM.

Among the four INTEGRA NON methods, the data plots follow a similar curvature. However, they are all distinct from one another. Differences are especially visible towards the end of the simulation. Of these four data sets the proportionate method appears to have the largest amount of total network travel time. The smallest amount of total network travel time is consistently achieved by the QueensOD (with default seed) simulation.

Figure 5.8 – Network Travel Time Plot

The slope of the linear regression line and the Pearson’s correlation coefficient are shown in Tables 5.13 and 5.14, respectively. The regression
coefficient for each of the ten method paired comparisons is a reflection of the plotted data points. Where the deterministic method is compared with any of the other four techniques the regression line slope is very low. In one case the regression coefficient is below 0.500. However, the other four methods produce similar results in comparison. Every travel time combination of the four is within a 10% range of the ideal 1.000 correlation coefficient. The proportionate and FREQ output methods produce a slope of 1.009. This value is the closest to perfect similarity.

The Pearson's correlation outcomes are very similar to the regression slope outcomes. That is, a very strong relationship is observed for all of the combinations with the exception of the deterministic method. These six combinations all have a correlation coefficient of greater than 0.960. In fact, three combinations are above 0.990. The slope and correlation data are very close to 1.000 for the proportionate/FREQ output combination and the QueensOD (with default seed)/FREQ output data.

**Table 5.13 – Network Travel Time Regression Analysis**

<table>
<thead>
<tr>
<th>Travel Time Regression</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.581</td>
<td>0.514</td>
<td>0.514</td>
<td>0.546</td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.465</td>
<td>1.038</td>
<td>1.085</td>
<td>1.022</td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.514</td>
<td>1.038</td>
<td>1.085</td>
<td>0.944</td>
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<td>FREQ Output</td>
<td>0.546</td>
<td>1.009</td>
<td>1.022</td>
<td>0.944</td>
</tr>
</tbody>
</table>

**Table 5.14 – Network Travel Time Correlation**

<table>
<thead>
<tr>
<th>Travel Time Correlation</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
<td>0.792</td>
<td>0.647</td>
<td>0.971</td>
<td>0.993</td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.647</td>
<td>0.961</td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.655</td>
<td>0.971</td>
<td>0.993</td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>0.730</td>
<td>0.992</td>
<td>0.982</td>
<td>0.992</td>
</tr>
</tbody>
</table>

The results from the Independent t-Test Analysis are included as Table 5.15. In general, the null hypothesis that the difference in total network travel time between two methods is significant is rejected if the test result value is greater than or equal to...
0.05. That is, the data sets are similar if the significance values are above 0.05. The deterministic travel time results are not similar to any of the other methods based on this requirement. However, all of the other method combinations are statistically similar to one another. They all are well above the required 0.05 significance level.

Table 5.15 – Network Travel Time t-Test Significance Analysis

<table>
<thead>
<tr>
<th>t-Test</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>QOD Default</th>
<th>QOD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proportionate</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Default</td>
<td>0.000</td>
<td>0.267</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>QOD Custom</td>
<td>0.000</td>
<td>0.571</td>
<td>0.611</td>
<td></td>
<td></td>
</tr>
<tr>
<td>FREQ Output</td>
<td>0.000</td>
<td>0.719</td>
<td>0.457</td>
<td>0.830</td>
<td></td>
</tr>
</tbody>
</table>

5.6. Summary

The results from the deterministic method for generating origin-destination distributions are much different than the other four methods. The remaining methods are similar based on the applied statistical testing methods. The regression and correlation results are available in the Tables 5.1 and 5.2, respectively.

The simulation analysis is a comparison of INTEGRATION model simulation outputs based on OD distribution inputs by five methods. A visual comparison of the technique is performed based on plotted data sets. These data sets include both simulation data and contemporaneous field data. A realistic criterion was used for relating contemporaneous data to simulation data. A summary of the outcomes for each of the five cross-section locations is shown in Table 5.16. The proportionate method met the 10/75 criterion at all five of the cross-section locations. The deterministic method only met this criterion at two locations: Aiea Heights Road and Waimalu Street on-ramp, both of which have free flow conditions.
The Pearson’s correlation was applied to the vehicle speeds data. Generally, the correlation between the deterministic results and each of the other four methods is very low. Conversely, the other four methods have a strong correlation between each other. Overall, the highest levels of correlation in average vehicle spot speeds occurred between the proportionate, QueensOD (with custom seed), and FREQ output origin-destination methods.

### Table 5.16 – Vehicle Speeds 10/75 Criterion Analysis Summary

<table>
<thead>
<tr>
<th>#</th>
<th>Location</th>
<th>Deterministic</th>
<th>Proportionate</th>
<th>OD Default</th>
<th>OD Custom</th>
<th>FREQ Output</th>
</tr>
</thead>
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<td>88%</td>
<td>88%</td>
<td>88%</td>
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<tr>
<td>2</td>
<td>Radford</td>
<td>63%</td>
<td>88%</td>
<td>50%</td>
<td>88%</td>
<td>88%</td>
</tr>
<tr>
<td>3</td>
<td>Halawa</td>
<td>69%</td>
<td>75%</td>
<td>44%</td>
<td>63%</td>
<td>69%</td>
</tr>
<tr>
<td>4</td>
<td>Aiea</td>
<td>75%</td>
<td>75%</td>
<td>81%</td>
<td>88%</td>
<td>81%</td>
</tr>
<tr>
<td>5</td>
<td>Waimalu</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
<td>100%</td>
</tr>
</tbody>
</table>

No. of Locations ±75%: 2 5 3 4 4
% of Locations ±75%: 40% 100% 60% 80% 80%

The travel time statistics reflected similar outcomes. Generally, the deterministic method results are not similar to the other four methods. The lack of similarity is prevalent in both the graphical and statistical analyses. The deterministic simulation results appear to be an outlying data set based on both visual and statistical comparisons. The other four methods all vary, graphically and statistically, to a lesser degree but they provided statistically similar (not significantly different) total network travel time outputs.
CHAPTER 6
SUMMARY AND CONCLUSIONS

6.1. Study Summary

Typically data collection for freeway traffic analyses includes the collection of volume data at each entry and exit point for a pre-specified time interval, usually 15 minutes. However, several sophisticated traffic simulation models require traffic demand inputs in the form of origin-destination tables. The objective of this research was to examine the implications of using different methods for synthesizing OD table data from the customary entry/exit volume data.

The study of the implications of origin-destination distribution in freeway simulation may be subdivided into three major components: the development and implementation of five OD distribution techniques, the development of a traffic simulation, and the analysis of the resulting output. The complete methodology diagram is included as Figure 3.1. The study was performed using a segment of the H-1 freeway in Honolulu, HI. All data for this segment of freeway was obtained from vehicle traffic counts. Contemporaneous average spot speeds data were also obtained from other sources.

The development and implementation of five origin-destination techniques utilized three programs: QueensOD, FREQ, and WatsonOD. The QueensOD program was used to develop the QueensOD (with default seed) and QueensOD (with custom seed) distributions. The FREQ program was used in a separate study to generate the FREQ output distributions [21]. The WatsonOD module was developed
for use in this study to produce the deterministic and proportionate distributions. These five resulting distributions were all prepared in a standardized format, ready for use in simulation models such as INTEGRATION and VISSIM.

The traffic simulation was prepared using the INTEGRATION simulation model. The INTEGRATION model is a microscopic traffic simulator. This model requires that the user specifies various vehicle and geometric characteristics. An accurate simulation requires an iterative tuning process to replicate field conditions. The contemporaneous data were used as a basis for comparison in the tuning of the simulation. Once the simulation was adequately tuned, the OD distribution was the only variable input file introduced for performing simulations. All of the other input parameters were held constant.

The first analysis was completed to determine if any of the distribution methods generate similar origin-destination distributions. These five distribution outcomes were analyzed by statistical testing methods. The second analysis was performed after the traffic simulations for the five distribution methods were completed. The analysis was comprised of a visual inspection of plotted outputs and statistical testing of average vehicle spot speeds and network travel time outcomes.

6.2. Conclusions

The intent of this study was to determine the implications of origin-destination distribution in freeway simulation. These implications are best characterized by the relationship between the five origin-destination distribution techniques and the resulting simulation outcomes. The specific performance of each of the five OD distribution methods was as follows:
• **Deterministic** – The deterministic OD distribution results were much different than the other four methods. In comparison to the other distribution techniques the Pearson's correlation coefficients were always less than 0.520 and the linear regression slope coefficients were always less than 0.270 which indicate a poor match between the OD values of this method compared to the OD values of every other method. The average vehicle spot speeds and network travel time were also dissimilar in comparison to the other distribution techniques. The Pearson's correlation coefficients were frequently less than 0.600 for each of the five cross-section locations and less than 0.800 for the network travel time. This method produced results that met the 10/75 criterion\textsuperscript{2} only at two out of five detector locations in the examined network. Generally, the deterministic distribution method produced both OD estimates and simulation results that were dissimilar to the other methods examined. It is not recommended for use in any application other than possibly as a seed matrix.

• **Proportionate** – The proportionate OD distribution results were similar to several of the other four methods. In comparison to the other distribution techniques, with the exception of the deterministic method, the Pearson's correlation coefficients were always greater than 0.930 and the linear regression slope coefficient was always greater than 0.880. The proportionate and FREQ output methods produced values of 1.000 in comparison, which

\textsuperscript{2} The 10/75 criterion is an analysis method used for characterizing average vehicle spot speeds in comparison with contemporaneous moving observer data. The number of occurrences where the two data types are within 10 mph of each another are tabulated. If the two data sets are within this range for 75% of the time spans the two methods are considered to be similar.
indicates a perfect match. The average vehicle spot speeds and network travel
time were also similar, except for the deterministic method, in comparison to
the other distribution techniques. The Pearson’s correlation coefficient was
frequently greater than 0.800, with only two exceptions, for each of the five
cross-section locations and greater than 0.960 for the network travel time
analysis. With the exception of the deterministic method the travel time
regression analysis slope coefficient was greater than 0.940 for all
proportionate analyses and the t-test proved there was significant similarity for
all comparisons. This method produced results that met the 10/75 criterion for
all five detector locations in the examined network.

- QueensOD (with default seed) – The QueensOD (with default seed) OD
distribution results were similar to several of the other four methods. In
comparison to the other distribution techniques, with the exception of the
deterministic method, the Pearson’s correlation coefficients were always
greater than 0.930 and the linear regression slope coefficient was always
greater than 0.880. The average vehicle spot speeds and network travel time
were also similar, except for the deterministic method, in comparison to the
other distribution techniques. The Pearson’s correlation coefficient was
frequently greater than 0.780, with the exception of the Radford St. detector
location, for each of the five cross-section locations and greater than 0.960 for
the network travel time analysis. At the Radford St. detector location the
correlation coefficient was less than 0.620 for all four of the average vehicle
spot speed comparisons. With the exception of the deterministic method the
travel time regression analysis slope coefficient was greater than 0.940 for all QueensOD (with default seed) analyses and the t-test proved there was significant similarity for all comparisons. This method produced results that met the 10/75 criterion only at three out of five detector locations in the examined network.

- **QueensOD (with custom seed)** – The QueensOD (with custom seed) OD distribution results were similar to several of the other four methods. In comparison to the other distribution techniques, with the exception of the deterministic method, the Pearson’s correlation coefficients were always greater than 0.977 and the linear regression slope coefficient was always greater than 0.953. The average vehicle spot speeds and network travel time were also similar, except for the deterministic method, in comparison to the other distribution techniques. The Pearson’s correlation coefficient was frequently greater than 0.787, with only one exception, for each of the five cross-section locations and greater than 0.971 for the network travel time analysis. With the exception of the deterministic method the travel time regression analysis slope coefficient was between 1.022 and 1.085 for all QueensOD (with custom seed) analyses and the t-test proved there was significant similarity for all comparisons. This method produced results that met the 10/75 criterion four out of five detector locations in the examined network.

- **FREQ Output** – The FREQ Output OD distribution results were similar to several of the other four methods. In comparison to the other distribution
techniques, with the exception of the deterministic method, the Pearson’s correlation coefficients were always greater than 0.930 and the linear regression slope coefficient was always greater than 0.984. The proportionate and FREQ output methods produced values of 1.000 in comparison, which indicates a perfect match. The average vehicle spot speeds and network travel time were also similar, except for the deterministic method, in comparison to the other distribution techniques. The Pearson’s correlation coefficient was frequently greater than 0.787, with only two exceptions, for each of the five cross-section locations and greater than 0.982 for the network travel time analysis. With the exception of the deterministic method the travel time regression analysis slope coefficient was between 0.944 and 1.022 for all FREQ Output analyses and the t-test proved there was significant similarity for all comparisons. This method produced results that met the 10/75 criterion for four of the five detector locations in the examined network.

Based on these conclusions, the following recommendations are developed for the origin-destination component for freeway simulation:

- Four of the five OD distribution methods produced relatively similar results in terms of INTEGRATION simulation output. Depending on the nature of the analysis the user may or may not be inclined to determine and apply the most accurate OD distribution. For four of the five methods the simulation results were adequate for common simulation purposes, thus, a more comprehensive determination of the OD distribution matrices may not be necessary. However, if minor deviations between simulations are critical, then a
comprehensive determination of the OD distribution matrices is recommended. Typically, errors in the origin-destination distribution will be magnified in simulation.

- Based on the limited outcomes from this single base study, the proportionate (WatsonOD) and FREQ output distributions were similar to each other and produced simulation measures of effectiveness that were adequately close to field collected MOEs.

6.3. Future Research

The number of origin-destination distribution combinations for most freeways is large and dependent on the size of the network being analyzed. There are several traffic models and many planning models that require origin-destination information of travel demand.

This research analyzed five origin-destination distribution methods and the resulting simulation outcomes using the INTEGRATION model. The research was conducted on a single freeway network. Future research must be conducted on multiple networks to determine whether these outcomes are universal or if they are unique to this study. This research should also be expanded with the use of other simulation models such as AIMSUN, PARAMICS and VISSIM. Such a study could determine whether the distribution methods are generally sensitive in simulation, or if different traffic simulation models respond with varying degrees of impact.
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<th>7</th>
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APPENDIX B: WatsonOD “IntAssumptions” Subroutine

Sub IntAssumptions()
    Windows(NewWB).Activate
    Sheets(“Assumptions”).Select
    Range(“B2”).Select
    ActiveCell.FormulaR1C1 = ”Assumptions"
    Range(“B2”).Select
    With Selection.Font
        .Name = ”Arial”
        .FontStyle = ”Bold”
        .Size = 14
        .Strikethrough = False
        .Superscript = False
        .Subscript = False
        .OutlineFont = False
        .Shadow = False
        .Underline = xlUnderlineStyleNone
        .ColorIndex = xlAutomatic
    End With
    Range(“C3”).Select
    ActiveCell.FormulaR1C1 = ”Value”
    Range(“C3”).Select
    Selection.Font.Bold = True
    Range(“B4”).Select
    ActiveCell.FormulaR1C1 = ”Start Time”
    Range(“B5”).Select
    ActiveCell.FormulaR1C1 = ”Period Length”
    Range(“B6”).Select
    ActiveCell.FormulaR1C1 = ”First O&D Pair Loading”
    Range(“B7”).Select
    ActiveCell.FormulaR1C1 = ”Last O&D Pair Loading”
    Range(“B8”).Select
    ActiveCell.FormulaR1C1 = ”Global Scaling Factor”
    Range(“B9”).Select
    ActiveCell.FormulaR1C1 = ”Random Vehicle Headway”
    Range(“B10”).Select
    ActiveCell.FormulaR1C1 = ”Driver Class 1”
    Range(“B11”).Select
    ActiveCell.FormulaR1C1 = ”Driver Class 2”
    Range(“B12”).Select
    ActiveCell.FormulaR1C1 = ”Driver Class 3”
    Range(“B13”).Select
    ActiveCell.FormulaR1C1 = ”Driver Class 4”
    Range(“B14”).Select
    ActiveCell.FormulaR1C1 = ”Driver Class 5”
    Range(“B15”).Select
    ActiveCell.FormulaR1C1 = ”Total Demand Fraction”
    Range(“B16”).Select
    ActiveCell.FormulaR1C1 = ”Passenger Car Equivalency”
    Range(“B18”).Select
    ActiveCell.FormulaR1C1 = ”Project Name”
    Range(“B4”).Select
    ActiveCell.FormulaR1C1 = ”Start Time”
    Range(“B4:B18”).Select
    Selection.Font.Bold = True
    Columns(“B:B”).EntireColumn.AutoFit
    Range(“C3”).Select
    With Selection.Interior

102
.ColorIndex = 34  
.Pattern = xlSolid  
End With  
Range("B4:B16").Select  
With Selection.Interior  
.ColorIndex = 34  
.Pattern = xlSolid  
End With  
Range("B18").Select  
With Selection.Interior  
.ColorIndex = 34  
.Pattern = xlSolid  
End With  
Range("C4").Select  
ActiveCell.FormulaR1C1 = "15:00"  
Range("C5").Select  
ActiveCell.FormulaR1C1 = "15"  
Range("C6").Select  
ActiveCell.FormulaR1C1 = "0"  
Range("C7").Select  
ActiveCell.FormulaR1C1 = "0"  
Range("C8").Select  
ActiveCell.FormulaR1C1 = "1"  
Range("C9").Select  
ActiveCell.FormulaR1C1 = "100%"  
Range("C10").Select  
ActiveCell.FormulaR1C1 = "100%"  
Range("C11").Select  
ActiveCell.FormulaR1C1 = "0%"  
Range("C12").Select  
ActiveCell.FormulaR1C1 = "0%"  
Range("C13").Select  
ActiveCell.FormulaR1C1 = "0%"  
Range("C14").Select  
ActiveCell.FormulaR1C1 = "0%"  
Range("C15").Select  
ActiveCell.FormulaR1C1 = "0%"  
Range("C16").Select  
Selection.NumberFormat = "0%"  
ActiveCell.FormulaR1C1 = "1%"  
Range("C16").Select  
ActiveCell.FormulaR1C1 = "1%"  
Range("C5:C7").Select  
Selection.NumberFormat = "0"  
Range("C8").Select  
Selection.NumberFormat = "0.0"  
Range("C16").Select  
Selection.NumberFormat = "0.0"  
ActiveCell.FormulaR1C1 = "1"  
Range("C5:C16").Select  
With Selection  
.HorizontalAlignment = xlCenter  
.VerticalAlignment = xlBottom  
.WrapText = False  
.Orientati0n = 0  
.AddIndent = False  
.IndentLevel = 0  
.ShrinkToFit = False  
.ReadingOrder = xlContext  
.MergeCells = False  
End With  
Range("C18:E18").Select  
With Selection
.HorizontalAlignment = xlCenter
.VerticalAlignment = xlBottom
.WrapText = False
.Orientation = 0
.AddIndent = False
.IndentLevel = 0
.ShrinkToFit = False
.ReadingOrder = xlContext
.MergeCells = False
End With
Selection.Merge
ActiveCell.FormulaR1C1 = "H1 O&D"
Range("B4:C16").Select
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlInsideVertical)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlInsideHorizontal)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
Range("B18:E18").Select
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
 .LineStyle = xlContinuous
 .Weight = xlThin
 .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlInsideVertical)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
Range("C3").Select
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
Range("E4").Select
With Selection.Interior
  .ColorIndex = 34
  .Pattern = xlSolid
End With
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
Range("G4").Select
ActiveCell.FormulaR1C1 = "Heading Designation"
Range("E6").Select
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Interior
    .ColorIndex = 36
    .Pattern = xlSolid
End With
Range("G6").Select
ActiveCell.FormulaR1C1 = "Hard Wired Cells - Not Adjustable"
Range("E8").Select
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeTop)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
    .LineStyle = xlContinuous
    .Weight = xlThin
    .ColorIndex = xlAutomatic
End With
With Selection.Interior
    .ColorIndex = 35
    .Pattern = xlSolid
End With
Range("G8").Select
ActiveCell.FormulaR1C1 = "Adjustable Cells - Input Required"
Range("E10").Select
With Selection.Interior
    .ColorIndex = 35
    .Pattern = xlSolid
End With
Selection.Borders(xlDiagonalDown).LineStyle = xlNone
Selection.Borders(xlDiagonalUp).LineStyle = xlNone
With Selection.Borders(xlEdgeLeft)
    .LineStyle = xlContinuous
    .Weight = xlThin
End With

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With Selection.Borders(xlEdgeTop)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeBottom)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
With Selection.Borders(xlEdgeRight)
  .LineStyle = xlContinuous
  .Weight = xlThin
  .ColorIndex = xlAutomatic
End With
Range("G10").Select
ActiveCell.FormulaR1C1 = "O&D Input File Cells"
Range("E12").Select
Range("C1").Select
Columns("C:C").ColumnWidth = 9.86
Columns("D:D").ColumnWidth = 4.57
Columns("E:E").ColumnWidth = 4.57
Columns("F:F").ColumnWidth = 0.67
Rows("3:18").Select
Selection.RowHeight = 14.25
Columns("A:A").ColumnWidth = 3.57
Range("B2").Select
End Sub
APPENDIX C: QueensOD Input Files

“QueensOD Master Control” File

Master
22 100 0
1 10 0.1
1 0 0
TP01\TP01\output\test01.dat
test02.dat
none
none
test06.dat
none
none
test10.out
test11.out
test12.out
test13.out
test14.out
test15.out
test16.out

“Observed Link Traffic Flow” File

Flow File 1
1 900 43 44
900 1
1 3884 14000 21 21 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
2 304 3800 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
3 352 3800 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
4 3700 3800 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
5 580 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
6 812 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
7 488 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
8 4664 8400 7 7 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
9 2132 3800 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
10 1208 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
11 1552 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
12 2820 3800 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
13 968 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
14 376 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
15 876 1900 11 11 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
16 5860 11200 21 21 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
17 1580 10800 5 5 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
18 3580 13500 10 10 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
19 3580 13500 10 10 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
20 3228 10800 28 28 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1
21 6928 13500 34 34 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1

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APPENDIX D: INTEGRATION Input Files

"INTEGRATION Master Control" File

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testproportionate\output\node.dat
link.dat
signal.dat
proportionate.dat
incidents.dat
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none
none
none
none
none
none
none
none
none
none
nonestriping.dat
detector.dat
none

"Node Characteristics" File

Node File
44 1 1
1 13.20 0.70 3 0 0
2 12.50 1.50 2 -1 0
3 12.50 2.30 2 -2 0
4 12.40 2.60 3 0 0
5 13.30 4.90 2 -3 0
6 13.30 6.15 2 -4 0
7 13.20 6.50 3 0 0
8 14.10 10.50 3 0 0
9 8.90 9.80 2 -5 0
10 8.60 9.50 3 0 0
11 5.90 9.70 2 -6 0
12 5.20 10.10 2 -7 0
13 4.80 10.30 3 0 0
14 3.80 10.50 3 0 0
15 1.40 10.30 2 -8 0
16 1.10 9.60 2 -9 0
31 12.50 1.10 4 0 0
"Link Characteristics" File

Link File 43 1 1 1 1 1
1 1 31 0.649 110 2800 5 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
2 31 2 0.200 65 1900 2 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
3 33 3 0.200 65 1900 2 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
4 4 34 0.200 65 1900 2 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
5 37 5 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
6 39 6 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
7 7 40 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
8 58 42 0.200 110 2800 3 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
9 47 9 0.200 65 1900 2 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
10 10 48 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
11 50 11 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
12 51 12 0.200 65 1900 2 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
13 13 52 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
14 14 54 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
15 15 56 15 0.200 65 1900 1 0 40 95 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
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18 19 32 0.297 110 2700 5 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
19 20 33 0.864 110 2700 4 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
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22 35 36 0.919 110 2700 4 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
23 36 37 0.071 110 2700 5 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
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25 38 39 0.119 110 2700 5 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
26 39 40 0.377 110 2700 4 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
27 40 41 0.141 110 2700 5 0 65 80 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 11111
### "Signal Timing" File

**Comment:** Freeway simulation without ramp meters includes no traffic signals

#### Signal Timing Plan

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### "Origin-Destination Traffic Demands" File

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26  8  16  1794  1  0  900  1  0  0  0  0  0  1
27 10  11  192  1  0  900  1  0  0  0  0  0  1
28 10  12  349  1  0  900  1  0  0  0  0  0  1
29 10  15  87  1  0  900  1  0  0  0  0  0  1
30 10  16  580  1  0  900  1  0  0  0  0  0  1
31 13  15  126  1  0  900  1  0  0  0  0  0  1
32 13  16  842  1  0  900  1  0  0  0  0  0  1
33 14  15  49  1  0  900  1  0  0  0  0  0  1
34 14  16  327  1  0  900  1  0  0  0  0  0  1

"Incidents" File

Incidents
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"Lane Striping" File

Striping File
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2 19  5  010  010  010  011  001  0000  0000  0000  0000  0000  0000
3 23  5  010  010  010  010  001  0000  0000  0000  0000  0000  0000
4 25  5  010  010  010  010  001  0000  0000  0000  0000  0000  0000
5 33  6  010  010  010  010  011  001  0000  0000  0000  0000  0000  0000
6 36  5  010  010  010  010  011  0000  0000  0000  0000  0000  0000
7 37  5  010  010  010  010  001  0000  0000  0000  0000  0000  0000
8 42  5  010  010  010  010  001  0000  0000  0000  0000  0000  0000
9 28  4  010  010  010  001  0000  0000  0000  0000  0000  0000

"Detector Location" File

Detectors
5  0
1  1  1  0.1  0.005  60  Airport
2 1  21  0.3  0.005  60  Radford
3 1  24  0.1  0.005  60  Halawa
4 1  30  0.2  0.005  60  Alea
5 1  34  0.15  0.005  60  Waimalu
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### APPENDIX E: Origin-Destination Distribution Results

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APPENDIX F: Tabular Simulation Results

(Gray cells denote speeds below 50mph)

### Airport Speeds Results

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Network Travel Time Results

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| #         | FREQ | KRONOS | Determi
| Time     |      |        | nistic | Propor
| 1         | 15:15 | 517   | 278     | 271 | 272 | 288 | 271 |
| 2         | 15:30 | 516   | 460     | 412 | 404 | 401 | 406 |
| 3         | 15:45 | 449   | 599     | 407 | 499 | 498 | 491 |
| 4         | 16:00 | 544   | 570     | 773 | 581 | 580 | 579 |
| 5         | 16:15 | 632   | 638     | 850 | 675 | 601 | 634 | 642 |
| 6         | 16:30 | 958   | 988     | 902 | 725 | 663 | 908 | 909 |
| 7         | 16:45 | 728   | 734     | 931 | 719 | 852 | 720 | 710 |
| 8         | 17:00 | 785   | 777     | 939 | 720 | 688 | 726 | 724 |
| 9         | 17:15 | 789   | 784     | 937 | 718 | 714 | 714 | 713 |
| 10        | 17:30 | 754   | 778     | 927 | 708 | 687 | 708 | 708 |
| 11        | 17:45 | 681   | 758     | 900 | 692 | 622 | 664 | 669 |
| 12        | 18:00 | 577   | 700     | 903 | 680 | 595 | 624 | 661 |
| 13        | 18:15 | 494   | 650     | 908 | 628 | 512 | 533 | 575 |
| 14        | 18:30 | 363   | 576     | 850 | 541 | 453 | 466 | 495 |
| 15        | 18:45 | 310   | 492     | 895 | 472 | 370 | 393 | 437 |
| 16        | 19:00 | 225   | 494     | 860 | 384 | 311 | 368 | 350 |
| Average   |       | 539   | 613     | 812 | 696 | 633 | 668 | 871 |
REFERENCES


7. Transportation Engineering Online Lab Manual. *Travel Demand Forecasting: Trip Distribution.* Oregon State University, Portland State University, University of Idaho. 


