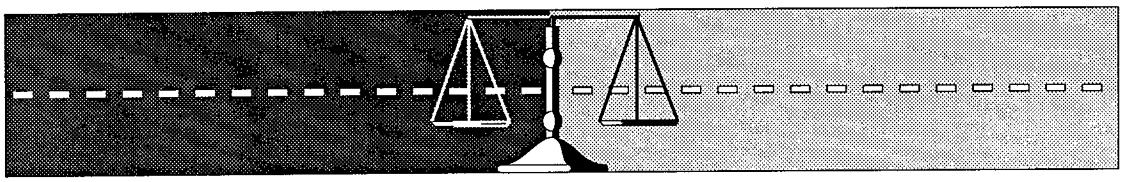
FINAL
CONTRACT REPORT

PB99-115420

A TOOL TO AID THE COMPARISON OF IMPROVEMENT PROJECTS FOR THE VIRGINIA DEPARTMENT OF TRANSPORTATION



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16. Abstract

The goal of this effort is to assist the Virginia Department of Transportation (VDOT) in improving the comparison in planning of potential primary and secondary roadway improvement projects. Historical projects that have been implemented or considered for implementation have been used as a case study data set. Methods are proposed for estimating cost, performance gain and crash risk reduction of future roadway projects, with the main focus being the presentation of tradeoffs among these criteria. If, in a particular case, more accurate and/or appropriate data is available for one or more of these criteria (e.g. from a simulation study that has been performed), then this information can easily be used to supplement or replace the estimations proposed here.

The project comparison instrument combines three major decisionmaking attributes in project selection: crash risk, performance, and project cost. By quantifying these attributes across a number of proposed highway improvement projects, projects can more readily be compared to one another, and a more holistic view of potential projects is achieved. This is an important step when choosing a portfolio of projects each year.

In order to compare projects, attributes are quantified in the following manner for planning level decisions. Crash risk reduction is calculated as the number of crashes avoided per year at the project site. Particular roadway improvements are typically assumed to decrease the expected number of crashes by a statistically determined and pretabulated percentage. Performance gain is quantified by the vehicle minutes of travel time avoided in the peak hour. Finally, cost is modeled as the sum of preliminary engineering, right of way and construction costs. Once the objectives are quantified, they can be graphically displayed in a Project Comparison Chart.

Examples for applying this approach are given in the text and in the accompanying workbook.

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NOTICE

The project that is the subject of this report was done under contract for the Virginia Department of Transportation, Virginia Transportation Research Council. The opinions and conclusions expressed or implied are those of the contractors, and although they have been accepted as appropriate by the project monitors, they are not necessarily those of the Virginia Transportation Research Council or the Virginia Department of Transportation.

Each contract report is peer reviewed and accepted for publication by Research Council staff with expertise in related technical areas. Final editing and proofreading of the report are performed by the contractor.

Foreword

This *final report* is supplemented by the accompanying *Workbook*. While the Workbook is more concerned with the applied side of the project results, the final report describes the project team's approach to the various issues of concern in comparing roadway improvement projects and gives the theoretical background for the methodologies used in the Workbook. The final report also provides literature reviews for the topics of relevance.

The final report comprises three chapters: chapter 1 gives an overview of the project, chapter 2 reviews existing assessment tools for crash risk, performance and cost, and chapter 3 describes the selection and adoption of those tools for the present project. The Reference section provides a bibliography for both the Workbook and the final report. The final section of the document is formed by appendices that contain the database which was used for the project.

Final Report

Table of Contents

CHAPTER 1
OVERVIEW OF PROJECT

1.1 Background

This document describes the results of the project, "A Tool to Aid the Comparison of Roadway Improvement Projects of the Virginia Department of Transportation." The project has been performed by the Center for Risk Management of Engineering Systems at the University of Virginia and has been supported by the Virginia Department of Transportation (VDOT) through its research agency, the Virginia Transportation Research Council (VTRC). Figure 1.1 gives an overview of the different project phases that were performed between December 1996 and October 1997.

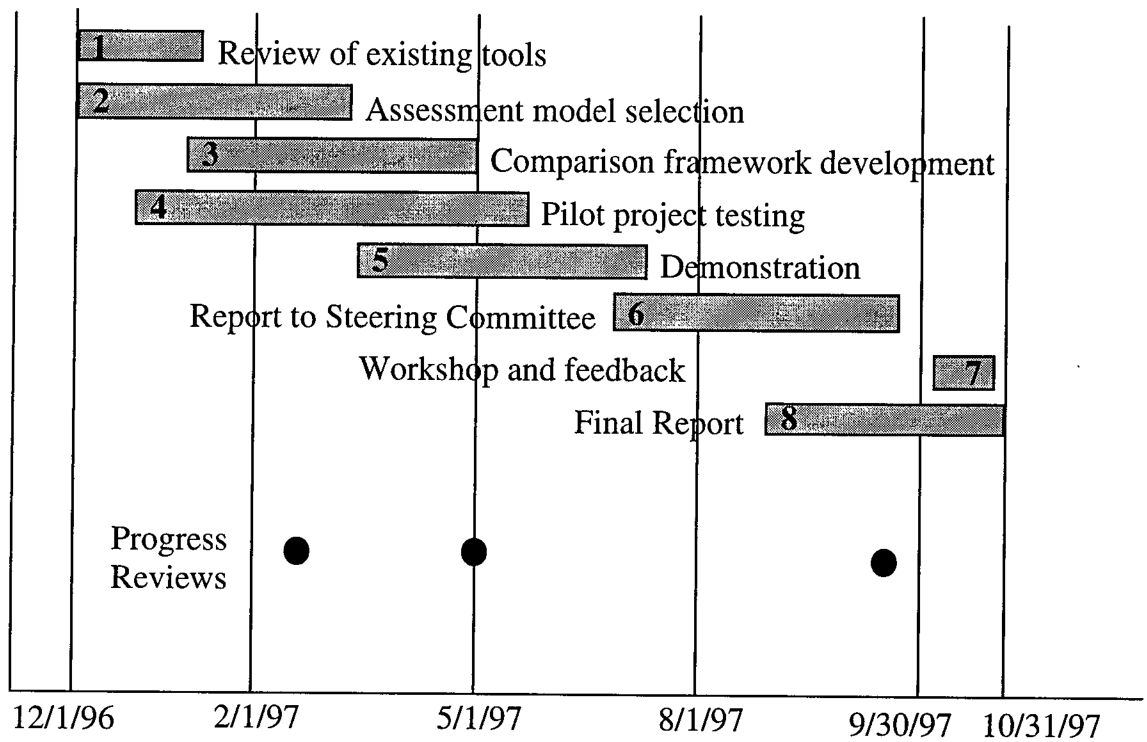

Figure 1.1: Gantt Chart of Project Tasks.

This "report," along with the "Workbook," constitutes the final project. While the Workbook is more concerned with the applied side of the project results, the report describes the project team's approach to the various issues of concern in comparing roadway improvement projects and gives the theoretical background for the methodologies used in the Workbook. The report also provides literature reviews for the topics of relevance.

For more information on the project mission, the reader is referred to the Executive Summary of the Workbook.

1.2 Task 1: Review of Existing Tools

Task 1 involved a comprehensive review of existing tools for the assessment of crash risk, road and highway performance, and improvement project cost. This review served as the groundwork for the subsequent tasks.

The review of crash risk assessment methodologies was developed and enhanced by the team's interaction with VDOT personnel, notably the traffic engineering personnel at the state level and in the pilot district. This interaction, along with the team's continued research, reflected the project's directive to adopt existing assessment tools rather than develop them.

The review of performance assessment methodologies uncovered a number of alternatives for measuring performance gain. For example, measuring delay at intersections, and speed and traffic density at road sections, can both lead to travel time as a metric for performance.

VDOT's design engineers base their cost estimates on experience with previous projects. The tool developed in this study will not change the way project costs are estimated, since these cost estimates are used for other purposes. Therefore, the emphasis of cost estimation tools was to build on the tools used in VDOT's current practice.

1.3 Task 2: Tool Selection

This task consisted of two parts. The first was the selection of ultimate measures for crash risk reduction, facility performance improvement, and project cost applicable to the comparison of the diverse improvement projects under consideration by VDOT. The second was the selection of models and processes that will use information that is readily available during the planning phase of each project, either in the form of observed data or expert estimates, to generate these measures.

For crash risk, the reduction in the mean number of accidents per year at a site is the recommended measure of an improvement project's effectiveness. The reduction in the average number of fatalities per year at a site could also be considered.

Some existing predictive models that generate a base rate of crashes from road structure, geometry, and other factors for crash risk assessment require too many resources to be used for a planning study. They may be more useful for design evaluation. Another approach is the Accident Reduction Factor (ARF) method, which uses an expectation of the accident rate reduction to predict the effectiveness of an improvement. Most techniques for the evaluation of crash risk fall into one of these two categories, and both have been considered.

The Accident Reduction Factor (ARF) approach currently in use in Virginia is recommended, provided that some modifications that account for uncertainty are made. This approach uses an estimate of the percentage reduction in the accident rate at a given site. This method provides an estimate for the expected reduction in the average number of accidents per year at the site. Some of the crash risk data needed for this approach can be obtained from HTRIS.

For performance, the selection of an effective measure was a challenging problem. Unlike crash risk, performance is generally not well defined, and may change meaning according to the situation in which it is being considered. Some measures of performance are available in VDOT's information system, but others may be impossible to obtain.

Given the planning-level orientation of this study, the team recommends the use of travel time saved, expressed in terms of total minutes saved during the peak hour. This can be calculated by multiplying the traffic volume during the peak hour by the estimated average travel time saved per vehicle, both "without" and "with" a project, although both may be hypothetical estimates. This measure has a number of advantages. First, it is equally applicable for intersection, segment, or any other type of facility improvement. Second, it can be used in conditions of both data adequacy and data scarcity, with only the levels of uncertainty (confidence intervals) being changed. Third, it accurately reflects the drivers' intuitive notion of facility performance.

There were several ways of computing the performance measure. Both the daily traffic and road inventory information (e.g., 4-lane divided highway) are readily available through HTRIS. Several techniques exist to estimate travel time saved. Additionally, there are many proven methods for the reliable incorporation of expert judgment into such estimates that are available. These alternatives are presented in the Workbook.

The recommended measure for cost of a project is the summation of preliminary engineering cost, right of way cost, and construction cost. It has been assumed that only the cost over the construction period will be considered in this demonstration; however, lifecycle costs can be integrated in the future. The best way to generate these cost estimates without project-specific information is cost per mile estimates. The cost per mile figures are commonly used for initial planning purposes at VDOT. Several cost estimation tables from the Transportation Planning Division will be included in the comparison tool. It is important to note that most estimates are revised as plans are prepared, changed, and carried out, and the new information affects both the estimates and their level of uncertainty.

1.4 Task 3: Decisionmaking Framework Development

The development of the overall decisionmaking framework within which proposed projects are considered involved both the specification of the project's role in the decisionmaking process, and the internal processes which will drive the tool.

For most smaller projects, the motivation for the project comes from the county level and is channeled through the resident engineer to the district office. Larger projects, conversely, are often motivated in a "top-down" manner by the state Traffic Engineering and Planning Divisions. The district engineer uses the cost estimates given by the motivating entity and develops an assessment of the potential effectiveness of each improvement; the projects recommended at this level are then submitted to the state Transportation Planning Division for potential inclusion in the six-year plan.

The methods incorporated in the tool can be used for two different types of comparison. The first is the comparison of different options in the same jurisdiction for preliminary planning purposes, which might be useful during the public hearings that are commonly held for local-level projects. The information these methods provide would be useful in illustrating to the public the costs, benefits, and tradeoffs associated with each proposed project or set of projects.

The second is the comparison of multiple projects and portfolios at the state planning level as described previously.

Several points of specification have been generated reflect, in part, the team's understanding of the role of this tool in the future decision-making processes involving VDOT's planning efforts.

1. The decision tool will be used by the district traffic engineers in the process of recommending projects for inclusion and funding in the six-year plan. The methodology will not necessarily be suitable for smaller-scale projects funded through maintenance or other discretionary budget items.

2. The tool is meant to help the district engineers visualize and evaluate the tradeoffs and uncertainties involved in the selection of projects, and is in no way meant to recommend certain projects, perform or eliminate the need for value judgments, or supplant the engineer's decisionmaking role in any way.

3. The tool will be used during the planning phase, before project design is underway.

4. The tool will use confidence intervals as the primary means of representing the uncertainty in estimates and measurements with which it is to be used.

Through the discussion and modification of these statements and others, the team has worked towards a consensus with VDOT regarding the purpose, use, and limitations of the tool.

1.5 Task 4: Development of Test Project Database

The team worked with the pilot district traffic engineer in the construction of a set of projects that accurately represents the number, types, and magnitudes that are proposed within a district in a given year.

The collection of these projects' effects on performance was particularly challenging, since this information is not stored and catalogued in database form.

While some data (such as number of accidents and daily traffic) were accessible through HTRIS, others (in particular, performance-related information) were obtained from VDOT Resident Engineers and the Culpeper District Traffic Engineer. The questionnaires reproduced below were developed and used by the project team for eliciting data from VDOT engineers in order to facilitate the communication between the team and the engineers.

The main purpose of the questionnaire was to gather performance-related information on particular projects, but data on countermeasures (i.e. project type), which is of relevance to the crash risk analysis, was also collected, as well as some background information on the project motivation.

Originally, approximately 70 projects from the Culpeper district had been identified with the help of the Six-Year-Plans of 1990 through 1996 and the District Traffic Engineer. This set comprised projects that were to be implemented between 1990 and 1997. As it was desired to work with projects that had already been implemented so that model predictions could be compared to observations, attention had generally to be limited to projects that had been realized in the years 1992 through 1994. (HTRIS provides crash data from 1990 on, and a two-year crash-sample both before and after project implementation was desired.) Due to the difficulties of collecting data on projects that are completed and no longer of primary concern, and the problems of eliciting data that was available *before* that project was implemented (in order to make a "prediction"), the feasible database shrank to 29 projects within the considered time-frame. However, from out of these 29 projects, a complete set of information (construction dates, location, crashes "without" project, daily traffic, cost estimation, project type, performance effects) was only available for 10 projects, and "with" project crash data was available for 9 out of these 10 projects. For the remaining 19 projects, data was missing, such as exact location or construction dates, but mostly performance-related information. An overview is given by the table of projects in the appendix which gives the reader an idea of what data is at hand or not.

In addition, roughly 120 hazardous locations from Critical Rate Listings (1994 and 1995) were analyzed (see Chapter 6 of Workbook and Appendix of the report).

↓ *Please verify information and check if accurate, or correct as necessary*

☐ **Project-ID**: **UVA #:**

☐ **Construction Start/End:**

☐ **Project Description:**

#	[Intersection]	**Before Project**	**After Project**	**Source of Data**
	Respond to ALL of the following rows			
1	Level of Service (A - F) during Peak-Hour			
2	v/c during Peak-Hour			
3	Percentage of Daily Traffic during Peak-Hour			
	Respond to AT LEAST ONE of the following rows			
4	Avg. Time in Queue per Vehicle (minutes) during Peak-Hour			
5a AND 5b	Avg. Time in Queue per Vehicle (minutes) during Peak-Hour % Reduction			
6	Avg. Time in Queue Saved per Vehicle (minutes) during Peak-Hour			

Countermeasure from Countermeasure Classification table (circle *all* that apply):

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

other(describe):

Project Motivation – Rank the following items according to their importance for implementing the project ("1" corresponding to "most important"):

__improve safety __public demand __aesthetics __maintenance

__improve performance (capacity) __economic development __environment

__other (describe):

↓ *Please verify information and check if accurate, or correct as necessary*

☐ **Project-ID**: **UVA #:**

☐ **Construction Start/End:**

☐ **Project Description:**

#	[Road Section]	**Before Project**	**After Project**	**Source of Data**
	Respond to ALL of the following rows			
1	Level of Service (A - F) During Peak-Hour			
2	v/c during Peak-Hour			
3	Percentage of Daily Traffic During Peak-Hour			
4	Length of Section (ft.)			
	Respond to AT LEAST ONE of the following rows			
5	Avg. Speed (mi/h) during Peak-Hour			
6	Avg. Travel Time per Vehicle (minutes) during Peak-Hour			
7a AND	Travel Time per Vehicle (minutes) During Peak-Hour			
7b	% Reduction			
8	Avg. Travel Time Saved per Vehicle (minutes) during Peak-Hour			

Countermeasure from Countermeasure Classification table (circle *all* that apply):

1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

other (describe):

Project Motivation – Rank the following items according to their importance for implementing the project ("1" corresponding to "most important"):

__improve safety __public demand __aesthetics __maintenance

__improve performance (capacity) __economic development __environment

__other–describe:

1.6 Task 5: Demonstration of the Comparison Tool

To contrast observed with predicted performance, Figure 1.2 compares the predicted number of crashes avoided per year to the observed number of crashes avoided for select projects (post-implementation analysis). Note that, unfortunately, only observed performance data (travel time saved) are available, as all projects have already been implemented, and estimates of the true time savings are not available (i.e. pre-implementation estimates of the expected post-implementation conditions were not available). Also, only cost information from 6-year-plans was available and no true, final cost. (Inquiries to the FMS and PPMS databases resulted in the same cost information that was available from the 6-year-plans.) One project (Project J in the database in the appendix) was not included in the graph, although data was available, because it would "dwarf" all other projects in terms of cost and the predicted number of crashes avoided per year.

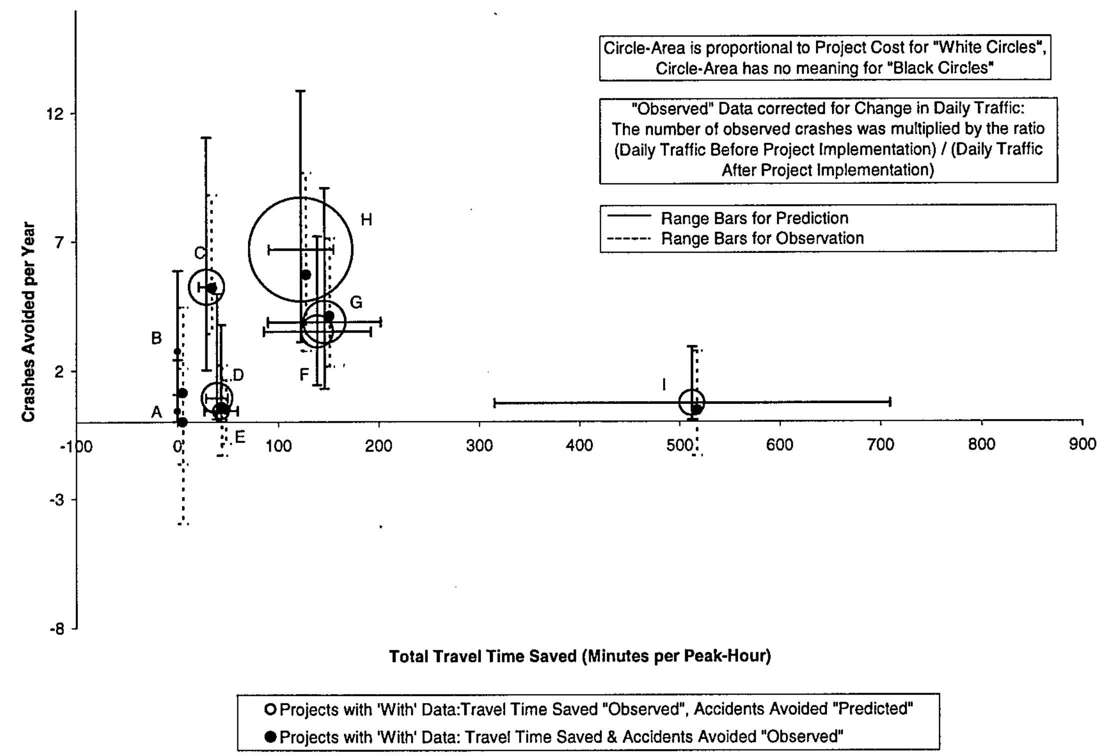

Figure 1.2: Predicted and observed crashes avoided per year vs. total travel time saved

Figure 1.2 is a graph that has been introduced in the Workbook. However, the small, black circles represent the average number of crashes avoided per year once the projects had been implemented. At most sites, the Daily Traffic increases over time – since "with" project occurs in time *after* "without" project, the underlying Daily Traffic will have changed. In order to approximately compensate the effects of higher Daily Traffic after project implementation, the observed number of crashes per year under "with" project conditions was corrected by multiplying it by the ratio (Daily Traffic in the 2 years preceding project implementation) / (Daily Traffic in the 2 years following project implementation). The adjusted number was then used to compute the observed number of crashes avoided. While the relationship between Daily

Traffic and Crashes per Year will usually not be strictly linear, it is felt that this adjustment procedure can well serve as a first approximation.

Table 1.1: Sample Projects depicted in Figure 1.2

Index	Number	Description	Location	Construct Start	Construct End	Total Travel Time Saved	Accidents Saved per Year Predicted (Total)	Accidents Saved per Year Observed	Total Cost (in $1,000)
A	0029-056-112, pe101, n501	29- Madison Construct Right Turn Lane Northbound	@ Madison Cty. High School (Rt. 9731), 0.3 mi S of Intersection Rt. 29 Bus./Rt. 231	11/02/92	06/22/93	0	0.4	0.0	16
B	0029-023-v10, pe101, m501	29 - Culpeper Construct Right Turn Lane	@ Rt. 666	10/27/94	11/02/94	0	2.8	1.1	15
C	0020-002-s21, pe101, rw201, c501	20- Albemarle Improve Horizontal and Vertical Alignment	3.4 mi S of Rt. 53 to 3.8 mi S of Rt. 53	11/01/93	01/26/95	29	5.2	5.2	595
D	0015-056-701, pe101, rw201, m600, 0015-023-705, rw201, m400	15 - Culpeper and Madison Bridge Replacement	Crooked Run: Culpeper/Madison CL	03/22/93	12/04/93	39	0.9	0.6	440
E	0033-039-107, pe101, rw201, c501	33- Greene Improve Turning Radius	Stanardsville, Intersection Rt. 230	04/27/92	06/25/92	43	0.4	0.5	125
F	0020-002-123, pe101, rw201, m501	20 - Albemarle Extend Acceleration Lane	@ Intersection I-64	11/11/96	7/97	139	3.5		475
G	0020-002-s17, pe101, rw201, c501	At route 20 at Intersection Route 742 (Avon St. extended) Realigned Route 20 and improved intersection	2.9 mi S of Corporate City Limits C-ville	07/20/92	11/03/93	140	3.9	4.1	826
H	0003-023-104, Pe103, RW203, C503	Route 3 4-lane widening from Orange County line west to east of Lignum - Culpeper Residency	2.5 mi W of Culpeper/Orange CL to 0.3 mi W of C/O CL	02/17/93	09/28/94	121	6.7	5.7	5000
I	0033-054-106, pe101, rw201, n501	33- Louisa Install Right turn lane	Intersection Rt. 628	01/20/92	04/17/92	512	0.7	0.5	300

The underlying calculations for the confidence interval for the observed number of crashes avoided take into account the fact that both "without" and "with" data are samples, i.e. that neither the true parameters for the "without" nor for the "with" project conditions are known. The upper bound for the observed number of crashes *avoided* was computed in the following way: After the number of accidents per year "with" project had been adjusted for changes in daily traffic (as compared with the "without" project situation), the lower bound for the observed number of crashes with project was subtracted from the upper bound for the number of crashes without project. This resulted in an optimistic estimate, i.e. the upper bound for the number of crashes avoided. On the other hand, subtracting the upper bound for the observed number of crashes with project from the lower bound of crashes without project resulted in a pessimistic estimate of the number of crashes avoided, yielding the lower bound. In order to arrive at a 95% confidence interval (i.e. 2.5% interval on each tail of the distribution) for the observed number of crashes *avoided*, a 68% confidence interval was used to compute the bounds for the number of crashes with and without project. (A 68% confidence interval results in 16% intervals on either tail of the distribution. $0.16 \times 0.16 = 0.256 \approx 2.5\%$, yielding an approximate 95% confidence interval for the observed number of crashes avoided, as this number is a result of 2 random variables [number of crashes without and with project]. The approach is not analytically exact, but was used as a first approximation.)

Figure 1.2 shows that the predicted and actual average numbers of crashes avoided per year lie close together for the represented projects (which constitute all projects for which complete information was available at the time of editing). Note that for one of the projects, no "with" project data was available, as was completed in July 1997.

1.7 Task 6: Report to Steering Committee

The project team has met with the Steering Committee on three occasions to refine objectives and report progress. The last report to the Steering Committee on September 19, 1997 focused on the Workbook, which is a result of the project team's past research and aims to make the proposed approach to project comparison understandable and accessible to VDOT analysts who may otherwise not be familiar with this new procedure. At the same time, results have been presented that have been obtained by the application of the methodology. As far as the limited availability of data permitted, predictions (on the effects of a project) have been compared to observed results (see Section 1.6).

1.8 Task 7: Preparation of Workshop Materials

Notes for a one-and-a-half-day workshop have been prepared by the project team for interested VDOT personnel. The workshop is intended to introduce the proposed approach to project comparison to practitioners in order to arouse their interest in the methodology and, even more important, to get their feedback as to the appropriateness and feasibility of the suggested procedures. If VDOT wishes to implement this methodology, then this step is of critical importance as it has been the project teams intent from the beginning to develop a tool that is of practical use to VDOT. The feedback will hopefully allow the project team to streamline the methodology and enhance the practicability of the approach.

1.9 Task 8: Final Project

The final project comprises

(a) a Workbook which presents the computations underlying the proposed methodology in a structured, easy-to-follow way, including some supporting material, and

(b) the final report which includes theoretical background for the proposed computations, literature review etc.

CHAPTER 2
REVIEW OF EXISTING ASSESSMENT TOOLS

The assessment tools that were reviewed for crash risk, performance, and cost are summarized in this section.

2.1 Review of Existing Crash Risk Assessment Tools

Crash risk is a measure of the probability and severity of traffic accidents on the roadway. A precise and meaningful definition of the crash risk is needed. For the purpose of this study, the crash risk will be defined as the mean number of accidents per year at a given location. This number can be found by multiplying the accident rate (i.e. the number of accidents per vehicle for spot locations, or number of accidents per vehicle miles for segment locations) by the number of vehicles entering a spot location, or the number of entering vehicles times segment length for segment locations. This measure can be divided into severity categories, such as fatal crashes, injury crashes, and property-damage-only crashes, or accident type, such as rear-end, angle, etc.

Various studies that estimate crash risk through data analysis, regression tools, and expert judgment have been identified. The following discussion concerns issues underlying the crash-risk modeling. It considers the two potential paradigms: the *base-rate paradigm* and the *reduction factor* paradigm. Specific examples of the two approaches will be presented in the following sections.

2.1.1 Base-rate Models

Base-rate analysis uses regression or other statistical analysis to estimate a function that relates crash risk to roadway inventory variables (i.e., lane width and accident rate). From these studies, it is possible to predict the base crash rate at a particular site using crash and inventory data from other sites. However, they would be ineffective to predict the impact of an improvement project. This is because (i) any single design variable change is overshadowed by the random term in the regression equation, therefore the conclusion for a single project has large uncertainty and (ii) the supporting data is not before-after based, thus the impact of a roadway project could not show up in the base-rate approach if the sites in the database never experienced such an improvement project.

A number of models have attempted to estimate crash risk based solely on roadway-inventory analysis. Miaou and Lum (1993) investigated four different regression models for relationships between vehicle accidents and design variables. These included an additive regression model, a multiplicative regression model, and two multiplicative Poisson regression models. Miaou and Lum (1993) discussed advantages and drawbacks of each model. The underlying assumption of a normally distributed accident rate is found to be inaccurate in the first two models because it is more likely that the actual distribution of accidents on a given roadway is rightly-skewed. That is, the likelihood of no accidents occurring along a particular stretch of roadway may be higher than the likelihood of one accident occurring (see Figure 2.1). It will be important that the similar assumptions for accident distributions be used when calculating crash risk

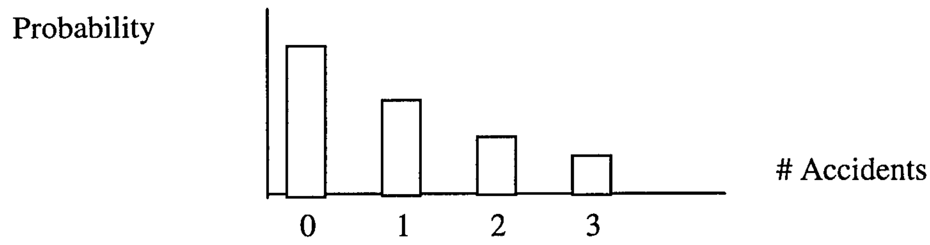

Figure 2.1: Rightly-skewed Distribution of Accidents.

Many risk studies become curtailed or statistically unmeaningful due to lack of data. The collection of data is one of the most resource-intensive activities of a project and is particularly cumbersome on a tight time budget. Furthermore, it is sometimes impossible to predict accident rates for a new road without historical data records.

2.1.2 Maher and Summersgill's Model

The base-rate method, as demonstrated by Maher and Summersgill (1996), uses a mathematical model with the Poisson distribution. Its high level of data inclusion allows the development of models to predict accident rates at intersections based on their flows (volume), geometries, signaling, and other roadway-inventory features.

$$\mu = \lambda T = \exp(\eta) = \exp(\beta\chi)$$

μ=mean number of accidents
T=observation period (year)
λ=the expected number of accidents per year
η=linear predictor
β=contains parameters which are to be estimated by the fitting process
χ=explanatory variables such as type of intersection

Maher and Summersgill's (1996) studies are based on mathematical models using the Poisson distribution to make accident predictions for certain sections of roadway and junctions (intersections) dependent upon roadway geometries and flow volumes. The study uses generalized linear models (GLMs) in order to analyze the accident and roadway data.

Thirteen different types of intersections are included. For the gathering of data, specific sites were randomly chosen for observation among a larger sample. The accident data is then composed of accident records reporting injuries at sites within a 20 meter radius of an intersection. Pedestrian flows and accidents are also taken into account.

Models were developed at several levels of detail:

Level 1: Coarse models relate total accidents and also vehicle-only and vehicle-pedestrian accidents to a simple flow function.

Level 2: Accidents are further subdivided into the nature of the accident (different vehicle and pedestrian types.)

Level 3: Same flow functions as Level 2, but also take into account geometries and signals of the roadway level of statistical significance, stability of the model, and comprehensibility of the effect, and the size of the effect and ease of measurement.

In this view, information about a roadway or intersection is entered into detailed models, including such information as design speed, lane width, roadway width, etc. Using these design factors, a measurement of "risk level" is determined based on these regression models. Any potential road situation can be entered into the system and given some estimate of risk level. It therefore has the advantage of being applicable to roads that do not yet exist; a kind of "advance" estimate of safety.

However, Maher and Summersgill's (1996) models are extremely data intensive and concentrate only on intersections. When data are collected for a short period of time and extrapolated to estimate mean flows, the model loses further validity.

Specific problems found with the models:

1. Low mean value problem - there was a large discrepancy between the observed value and predicted value at low accident levels. Using expected values as opposed to putting so much reliance on scale deviations is suggested to account for this problem.

2. Overdispersion - there were a large variation in accidents among sites; i.e., the variables taken into account in the study could not completely account for the causation of the accidents. Several possible reasons for this are given by the researchers. There are other unobserved explanatory variables, errors in the explanatory variables (the flow estimates taken from "snapshot" observation periods), or the model may be mis-specified.

3. Random error in the flow estimates - "one of the major costs is that of carrying out these flow counts and therefore it is important to have an appreciation for the effect of the length of the flow counts on the accuracy of the models which will be developed" (Maher and Summersgill 1996)

4. Aggregation of predictions - there appeared to be some "commonality between the missing variables". For example , whether the pavement was wet or dry was not taken into account.

Maher and Summersgill proposed several modifications to the Poisson distribution to address these problems. However, they do cite cases of inaccurate or incomplete measurement of variables.

2.1.3 Single-Factor Base-Rate Models

Specific studies exist that detail a more specific relationship between accident rates and roadway features. Knuiman et al. (1993) developed a detailed study associating median width and highway accident rates. This study examined the effects of median width on homogeneous highway sections. An accident database, road inventory database, and a traffic volume file were incorporated from the Highway Safety Information System (HSIS).

This particular study found that the safety benefits of a median were lost once its width was below 20 to 30 ft. This study is important because it outlines a boundary for improvement. Thus, simply adding a median will not decrease a roadway's accident rate by x percent, rather the dimensions of such a feature are taken into consideration.

Analysis of accident patterns at and safety features of intersections is particularly well documented. In 1986 Zegeer et al. examined the safety effects of cross-section design features for two lane roads. Lau and May developed an accident prediction model for unsignalized intersections in 1988 and for signalized intersections in 1989. Hauer et al. (1988) attempt to estimate the safety only at signalized intersections whereas King and Goldblatt (1986) draw relationships to accident patterns under different types of intersection control. More specifically, the attribute of clear vision at signalized intersections was studied by the Michigan Department of State Highways (1973).

General studies of geometric design were undertaken by Cirillo et al. (1969) on the interstate system. Luyanda et al. (1983) completed statistical analysis of highway accident conditions. Miaou at al. (1991) more specifically examine the relationship between truck accidents and geometric design. Studies of other specific design attributes are numerous. For example, the effect on safety from bridge width is examined by the Transportation Research Board (1987) and the effectiveness of clear recovery zones by Graham and Harwood (1982).

2.1.4 Interactive Highway Design Model

The Federal Highway Administration has undertaken an initiative in response to concern for the relationships between safety and geometric design from the Transportation Research Board (TRB) in the late 1980s. The Interactive Highway Safety Design Model (IHSDM) is currently under development. IHSDM is broken down into two levels: 1) incorporating HSIS for two-lane rural highways, and 2) evaluating and finalizing geometric design details with computer aided design (CAD).

The first level of development concerns the safety evaluation for preliminary design planning of two-lane rural highways. The relationships between accidents and highway features are formulated through regression analysis. Bared and Vogt (1996) present two stages for prediction. Stage 1 is aimed at predicting the total number of accidents from the given highway characteristics and stage 2 attempts to forecast the relative frequency of different accident severities once accidents have taken place. Below is the general model used to estimate the number of accidents at a particular intersection.

$$\mu_i = e^{\beta_0 + \sum_{j=1}^{k} (x_{ij}\beta_j)} = F(x_i)$$

$$P(Y_i) = \frac{\mu Y_i e^{-\mu i}}{Y_i}$$

where

$P(Y_i)$ = Probability of having Y_i accidents on highway segment or intersection i
μ_i = Expected or mean accident count for segment or intersection i
β_j = Regression coefficient of the j-th independent variable
(j=0 is the index of the intercept variable, j = 1, ..., k the index of the other independent variables)
x_{ij} = Highway characteristic variable j for segment or intersection i

This model is useful in that it attempts to predict not only crash risk rates but also severities. However, the model is currently unable to predict crash rates on new projects without historical records.

2.1.5 Conflict Opportunity Approach

An alternative approach to accident prediction was developed by Alan Kaub (VDOT) in cooperation with the Florida Department Of Transportation. This model is called *Statistically Probable Conflict Opportunities (SPCO).* It aims to overcome the problems with conventional exposure based models (which are usually non transferable and which are subject to skewed responses due to outliers in sample data) and models that try to link real conflicts (e.g. brakes applied) and actual accident events (which are usually unsuccessful in establishing that link).

In the SPCO model, probable conflicts per year are determined as:

Probable Conflicts per Year = Conflicts per Year [P(Angle) + P(Rear-End) + ...
...+ P(Sideswipe) + P(Fixed Object)].

Also, it is assumed that:

P(Conflict Opportunity) = P(Vehicle Arrival) x P(Opposition to Arrival) , where:

P(Vehicle Arrival): probability that any vehicle arriving will desire to make a particular movement,

P(Opposition to Arrival): probable arrival of one or more opposing conflicts (from angle, rear-end, side or fixed object) such that the opposing vehicle may not permit the completion of the intended maneuver during the time the arriving vehicle is exposed to conflict.

It is assumed that there is no relationship between conflicts and accident *types*. Furthermore, it is assumed that all accidents can be classified as angle, rear-end, side-swipe or fixed-object.

"The fundamental mechanism of the Probable Conflict Opportunity/Accident Model is the development of a calibrated relationship of the ratio of annual statistical conflicts to annual accidents which is stable over all geometries, volumes, speeds and traffic control types from one site to the next regardless of the human decisionmaking relationship between accidents and probable conflict opportunities, and which with relative accuracy predicts annual accidents at any individual site." The SPCO model does not require any accident history data for the sites.

The number of accidents per year (at an intersection) can be determined as:

Intersection Accidents per Year =
(Annual Sum Probable Conflicts)/([Model] Conflicts/Accident)

where:

Annual Sum Probable Conflicts: Linear combination of probability for angle, rear-end, sideswipe and fixed object accidents. The *speed-based* coefficients have been calibrated to numerous national accident studies and are intended to remain consistent nationally from one intersection or driveway to the next regardless of geometry, traffic volumes, traffic control types, or locations.

[Model] Conflicts/Accident: Multiple linear, marginally decreasing relationship between annual accidents and annual probable conflict opportunities for intersections, calibrated with numerous national exposure or rate-based models. The expression is a function of both volume on the major approach(es) and volume on the minor approach(es).

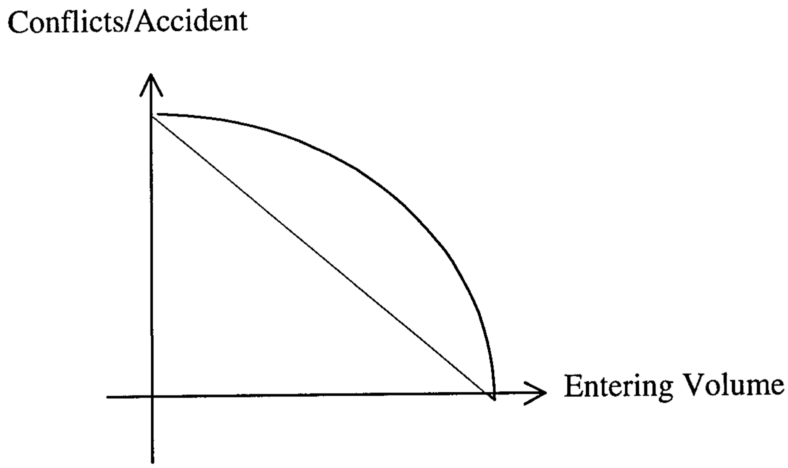

Figure 2.2: Conflicts per Accident versus Entering Volume at an Intersection

The ratio of annual probable conflict opportunities to annual accidents varies from approximately 500,000:1 to 4,000,000:1.

It was found that the TRAF-SAFE software which is based on the SPCO approach produced annual accident estimates that were within 3 standard deviations of the true mean for 99% of the sites, within 2 standard deviations for 91% of the sites, within 1 standard deviation for 72% of the sites, and within 0.5 standard deviations for 54% of the sites.

The mathematics that are underlying the SPCO are adopted from the HCM, which means that they are highly detailed. For example, required input variables are: volumes, approach geometry & bays, approach speed, right turn radii, perception/reaction time, stop sign setback, vehicle length, merge headway, saturation flows, cycles/phases/splits at pretimed signals, protected/permitted movements, right-on-red etc.

2.1.6 Reduction Factor Approach

Reduction-factor tools, unlike regression analysis on roadway-inventory variables in the base-rate approach, are based on "Before & After" studies. It is possible to calculate figures of crash risk reduction and magnitude based upon specific improvements, but these studies assume that the crash base rate is available. In the reduction-factor approach, current values of design factors and current risk levels are entered into the system. The user specifies which design factors will be changed during improvement, and the system estimates the subsequent expected improvement. In much of the literature, this expectation of improvement takes the form of an *accident reduction factor (ARF)*, or a percentage reduction in the number of accidents per vehicle at a given site. These factors are often further divided by types of accident (fatality, injury, property-damage-only).

ARFs will be presented in more detail in section 3.1.

2.2 Review of Existing Performance Assessment Tools

The term performance is used in this work, rather than capacity or level of service. Both capacity and level of service are precisely defined terms, neither of which is adequate for the comparison of diverse roadway improvements, such as of an intersection with a section of roadway. Rather, a generalization of the concepts of capacity and level of service is needed to better represent the varying relation of the potential throughput to the traffic demands, driver comfort, minimization of delays, and connectivity advantages of the improvement.

In this direction, the study team has considered extensions of the concepts of capacity and level of service, the use of demand and capacity models in reliability engineering (where the reliability is defined as the probability that the varying demand does not exceed the varying capacity), the conceptualization of the "performance" of a roadway in addition to its capacities and level of service, and the potential for use of a hierarchy of attributes for the definition of roadway performance.

The objective for the performance modeling module is to provide a computationally feasible and accurate means of evaluating the performance of a facility before and after a suggested improvement. Examples of several different approaches exist in the literature, ranging from queuing theory and car-following models to sophisticated traffic simulation software packages. Some of these use specific aspects of the physical layout and traffic characteristics to predict performance measure values; others use the *change* in certain characteristics to find the change overall performance. It is also important to note that each type of model yields different types of output. The appropriate *measure* of performance must be considered along with to the process for its estimation.

2.2.1 Statistical Traffic Flow Models

Several traffic engineering and analysis textbooks and surveys discuss simple idealized models that ascribe simple mathematical relationships to traffic characteristics such as speed, flow, and density. Two examples are Garber and Hoel (1997) and Mannering and Kilareski (1990). The most comprehensive source in the literature (Transportation Research Board 1975) also begins with these models.

The flow models are based on the Poisson distribution, a probability distribution frequently used to model "random" arrivals at service systems. In this distribution, vehicle arrivals per unit time at a point in the system are distributed randomly with mean l. Mathematically, they take the general form:

$$P(x) = \frac{(\lambda t)^x e^{-\lambda t}}{x!}$$

where P(x) = probability that x vehicles will arrive during counting period of t; l = average rate of arrival (vehicles per sec); t = duration of each counting period (sec); and e=natural base of logarithms (TRB 1975).

These equations can be generalized to find the probability of greater than x or less than x arrivals per unit time; therefore, a distribution of arrivals at a facility can be created.

Greenshields proposed the Linear Speed-Concentration Model, which assumes a linear relationship between speed and density, as shown in Figure 2.3.

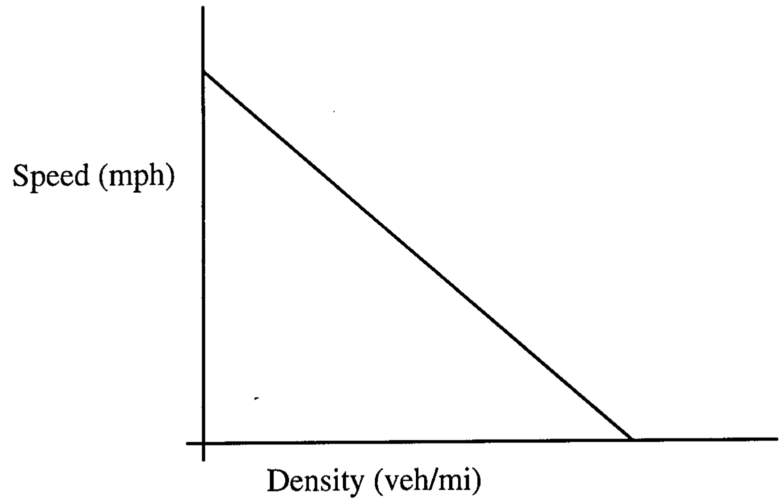

Figure 2.3: Greenshields' Speed-Density Relationship.

The relationship between flow and density (and thus that between flow and speed) is parabolic, as shown in Figures 2.4 and 2.5, below.

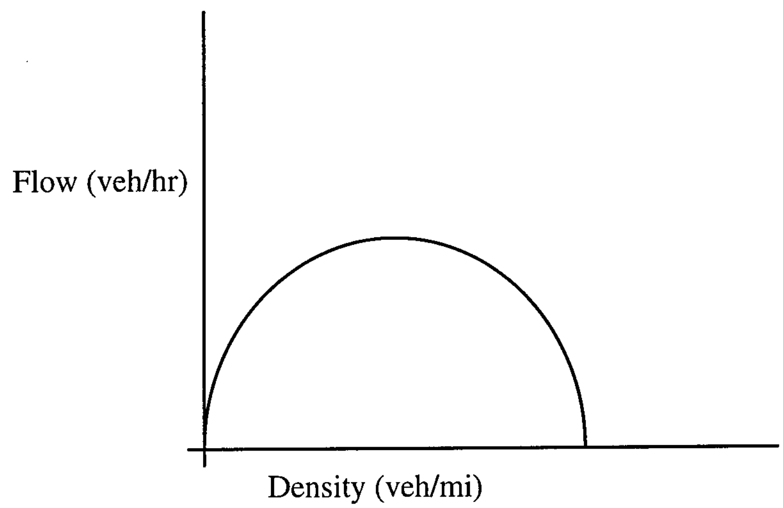

Figure 2.4: Greenshields' Flow-Density Relationship.

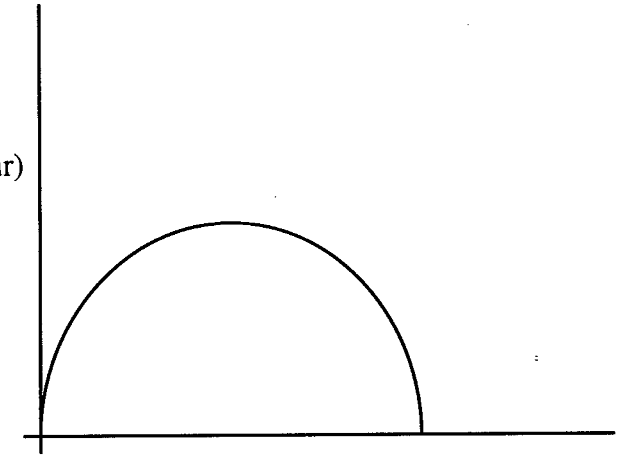

Figure 2.5: Greenshields' Flow-Speed Relationship

The optimum flow occurs, in this model, at a speed halfway between zero and free speed, and at a density halfway between zero and the jam density of the facility.

It is possible to analyze the performance of a facility judging by past traffic counts, using the average as the l-parameter in a Poisson arrival distribution. Then, the facility's size, width, and design speed could be used to find how close the actual flow is to the maximum flow.

This is an extremely simple approach to determining a highway facility's performance, and in the case of uninterrupted flow is quite effective. The data needed for such an approach, traffic counts and peak hour factors, are typically maintained by the state DOT.

But despite the attractiveness of the model's simplicity, it is incapable of dealing with interrupted flow, which occurs both in the case of jammed traffic and signalized or signed intersections. Finally, bottlenecks or other situations in which the amount of traffic entering a facility is greater than that leaving it (i.e., in cases of increasing density), the results of this model are inaccurate and not useful.

2.2.2 Highway Capacity Manual (HCM) Section Analysis Models

The Transportation Research Board (TRB) publishes the Highway Capacity Manual (TRB 1993), which is meant to provide practical and useful traffic models for varying transportation scenarios. These models have been developed by the TRB for the express purpose of capacity analysis, and are easily applied to this work. Highway Capacity Manual (HCM) models exist for virtually every road facility, with varying degrees of complexity. Additionally, there exists a software version of the HCM models, the Highway Capacity Software (HCS).

One model in the HCM is the *basic freeway segment.* This will be discussed as an illustration of the methods used in these models generally.

Mannering and Kilareski (1990) provide the following definitions for use with the HCM models:

Hourly Volume is the actual hourly demand volume for the highway in vehicles per hour, given the symbol V. Generally, the highest 24-hour volume (i.e., peak-hour volume) is used for V in traffic analysis computations.

The *Peak-Hour Factor* accounts for the nonuniformity of traffic flow over the peak hour. It is denoted PHF and is typically defined as the ratio of the hourly volume (V) to the maximum 15-min rate of flow (V_{15}) expanded to an hourly volume. Therefore,

$$PHF = \frac{V}{V_{15} \times 4}$$

This equation indicates that the further the PHF is from unity, the more *peaked* or nonuniform the flow.

Service Flow is the actual rate of flow for the peak 15-min period expanded to an hourly volume and expressed in vehicles per hour. Service flow is denoted SF and as defined as

$$SF = \frac{V}{PHF}$$

Within a certain level of service, there exists a *maximum service flow* for a given facility, defined as

$$MSF_i = c_j \times \left(\frac{v}{c}\right)_i$$

where MSF_i is the maximum service flow rate per lane for level of service i under ideal conditions in passenger cars per hour per lane (pcphpl), $(v/c)_i$ is the maximum volume-to-capacity ratio associated with level-of-service i (given in tabular form in the Highway Capacity Manual), and c_j is the capacity under ideal conditions for a freeway with design speed j (Mannering 1990).

This maximum service flow represents highway behavior under ideal conditions. However, some allowances must be made for roadway geometry, traffic composition, and unfamiliar drivers. Thus, for a basic one-directional freeway segment, the actual *service flow rate* can be estimated using the equation

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_p$$

where

SF_i is the service flow rate for level of service i under prevailing conditions
N is the number of lanes
f_w is a factor to adjust for nonideal lane widths and/or lateral clearances
f_{HV} is a factor to adjust for the effect of nonpassenger cars in the traffic
f_p is a factor to adjust for the effect of nonideal driver populations

A "nonideal driver population" is a segment of the driving population that is unfamiliar with the freeway system. Generally, an ideal driver population is made up of commuters and frequent local drivers that are familiar with the system and make mistake-free decisions; however, this is not always the case, and must be allowed for (TRB 1985).

Use of the HCM for segment analysis is advantageous for several reasons. First, its models are well accepted by the district engineers, and are not likely to yield counterintuitive results. The only data required are usually limited road inventory information, user estimates of traffic composition, and traffic counts for the subject areas. Level of service can be examined without being used as a form of measurement itself, and many of the adjustment factors can be assigned by the user based on his or her opinion; this "expert judgment" aspect of these models may be particularly attractive, in that the user will feel more like he or she is using a tool, and less like the computer is telling him or her what to do.

There are also a few disadvantages associated with these models. The incorporation of the HCM models into a larger software tool is potentially a very tedious task, and it is possible that the results between different types of facilities will vary greatly. Some models, such as those for signalized intersections, may require data that are not easily available, for example average peak hour delay time per vehicle. Furthermore, these models may be relatively insensitive to the performance ramifications of safety-motivated decisions and improvements, since the parameters involved are somewhat coarse.

2.2.3 HCM Intersection Analysis Models

The HCM also provides models for intersections, and these focus primarily on level of service. Level of service, when applied to signalized or signed intersections, is based in the delay per vehicle at each approach to the intersection.

Use of the HCM intersection models retains many of the advantages of the HCM segment models, including its general acceptance and treatment of level of service. However, these models require quite a bit more data to use than the segment models, and might rely more heavily on estimation of parameters.

2.2.4 Simulation Models

Many computer simulation packages are available that are either specifically designed or easily manipulated to deal with traffic analysis. Nearly all of these packages are based to some extent on the Highway Capacity Manual's techniques for capacity analysis.

Arnold and McGhee (1996) surveyed the existing packages as of January 1996, and assessed the usefulness and effectiveness of the most popular ones, with the intent of developing policies for VDOT's use and acceptance of the various packages. Among those that were found to be useful were HCS (the software version of the HCM procedures), SIGNAL94, and HCM/Cinema.

The best package, however, was TRAF-NETSIM. This package is quite a bit more complex, both in input requirements and output detail, than the others. Although its performance was found to be close to the others in isolated, smooth-flowing intersection analysis, it was the only package useful for analyzing congested intersections or arterial corridors involving multiple intersections.

This package is used frequently in VDOT's current practice when a large-scale pre-proposal capacity analysis is called for. It tends to be used for larger projects in which a detailed analysis represents only a small portion of the project's total planning and design cost. An example of this is the recent 236 Corridor Study (Northern Virginia District, 1996).

TRAF-NETSIM applies interval-based simulation to describe traffic operations. It deals with each vehicle in the system separately, and updates the status of every vehicle and variable control device (such as a traffic signal) every second. Among those variables that can be specified are physical components of the facility (such as number and direction of lanes at each approach), logical components of the facility (such as the presence and timing of various traffic signal stages), and the behavioral and physical characteristics of the traffic itself. Several discussions of the model's capabilities are available: A brief one at the manufacturer's World Wide Web homepage (Viggen Corporation 1996), and a more detailed VTRC report (Sulzberg and Demetsky 1991) are easily available.

TRAF-NETSIM's major advantage is the fact that it allows the user to estimate, at a very high level of accuracy, delay times at intersections, level of service at intersections, and other measures of effectiveness for intersections and corridors. Since capacity improvements on non-urban highways are generally confined to these two types of facilities, it would be the most general and comprehensive for the purposes of this study.

As mentioned before, TRAF-NETSIM requires extremely detailed input information, which is both costly and time-consuming to obtain and submit. The possibility of actually creating a software tool which could incorporate the TRAF-NETSIM models in the time period budgeted for Phase I of this project was minimal. The other simulation models listed, however, are less useful and would offer no substantial advantages over other methodologies considered, besides the ease of computation.

2.2.5 Other Models

Many other approaches, both traditional and innovative, are available to the study team for the modeling of performance. For example, a reliability-based approach has been proposed, where the probability of the demand for a traffic facility exceeding the capacity of the facility is equated to the probability of failure of that subsystem. Gnedenko and Ushakov (1995) describe probabilistic reliability models of this sort.

Various traffic demand forecasting models are described in the literature (see, e.g., Faulkner and Velichansky 1993). Viewing both the demand and supply of capacity as stochastic variables leads to a diagram such as that in Figure 2.6.

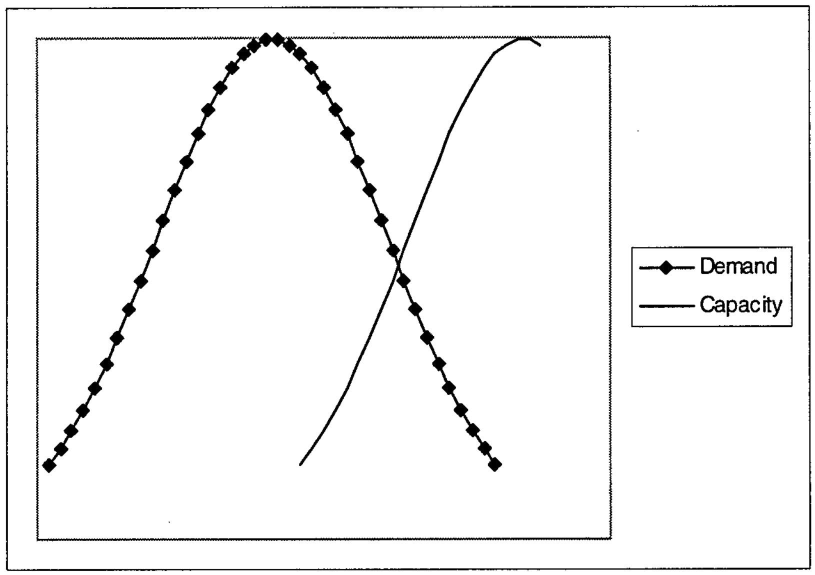

Figure 2.6: Probability densities of demand versus capacity in engineering reliability assessment.

2.3 Review of Existing Cost Assessment Tools

The project team first took a broad view of cost issues in VDOT's allocation process. Since this review, the framework of cost analysis has been refined. However, this section provides a valuable perspective on background cost issues.

In this section, some of the complexities in characterizing the costs associated with the stewardship of the highway transportation system are introduced. The study team's prototype recommendation is to consider either or both of the total capital cost of the improvement project and the annualized capital cost of the project for planning comparison purposes.

2.3.1 Burden of Ownership

Dell'Isola (1991) has introduced the concept of the "Burden of Ownership", defined as the costs and risks incurred by the Virginia Department of Transportation regarding any roadway in the state. Dell'Isola (1991) claims that Value Engineering (VE) should be applied to complex highway improvement projects the same way it is applied to other types of construction endeavors. A graphical representation of the structure he proposes for the burden of ownership is shown in Figure 2.7, below.

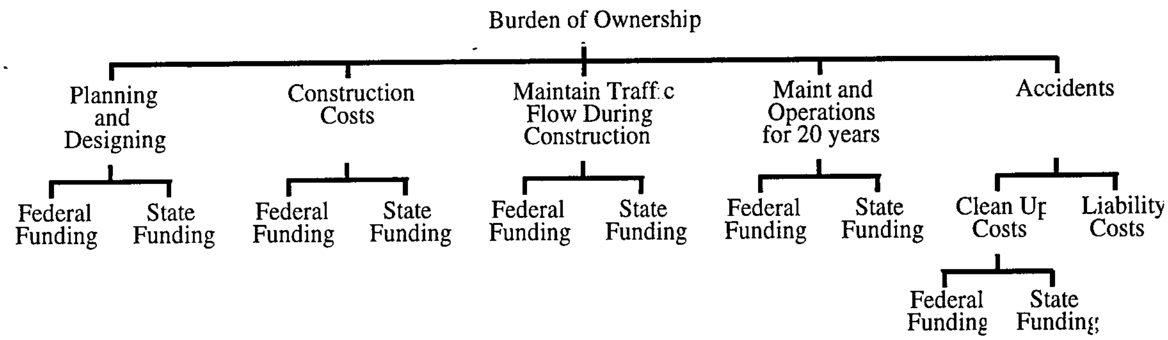

Figure 2.7: Burden of Ownership (Dell'Isola 1991).

The Burden of Ownership, shown in Figure 2.7, includes the costs Maintenance and Operation (30%), Planning and Design Costs (15%), Construction Costs (30%), and Cost of Maintaining Traffic During Construction (25%). The percentages represent the breakdown of each of the above costs over a typical 20 year life span of a highway project (Dell'Isola 1991).

Surprisingly, the Construction Costs are only 30% of what the department of transportation can expect to spend on a project over the project's 20 year lifetime. This confirms the necessity to look beyond the construction costs when considering competing projects.

Other important considerations in decisionmaking are the proportion of each of the four costs above that will be met by Federal Funding as opposed to State Funding, and the amount of funds allocated for each type of cost.

The above costs are largely deterministic. Most of the state's risk lies in the non-deterministic Risk of Accidents. Unlike the costs above which are deterministic or nearly so, this cost is merely an expected value. The Risk of Accidents is composed of Liability Arising from Accidents and Cost of Cleanup.

Liability Arising from Accidents includes court costs should the state be sued and any settlement if the state is found responsible for the accident or negligent in preventing an accident. In today's litigious society, this cost can be substantial, but estimating it is difficult at best. Culkin et al. (1988) and Kilareski (1991) deal explicitly with this problem, and come to the same conclusion: One can only assume that a project that lowers the probability of an accident also lowers the expected cost of liability.

The Cost of Cleanup refers the site of an accident and can include everything from removing and the actual vehicle(s) to the extreme event of containing and detoxifying a hazardous waste spill.

Although the carrier and his insurer would be responsible for most of the cost of the cleanup, the state's expense would probably also be considerable in the event of a hazardous waste spill.

2.3.2 User Burden

The User Burden, shown in Figure 2.8, includes Cost of Vehicle Operation, Travel Time, and Risk of Accidents. It is within these risks that the benefits of a highway project lie. Presumably, the state would pursue a project only if it resulted in the reduction of one or more of the User Risks (Kragh et al. 1986).

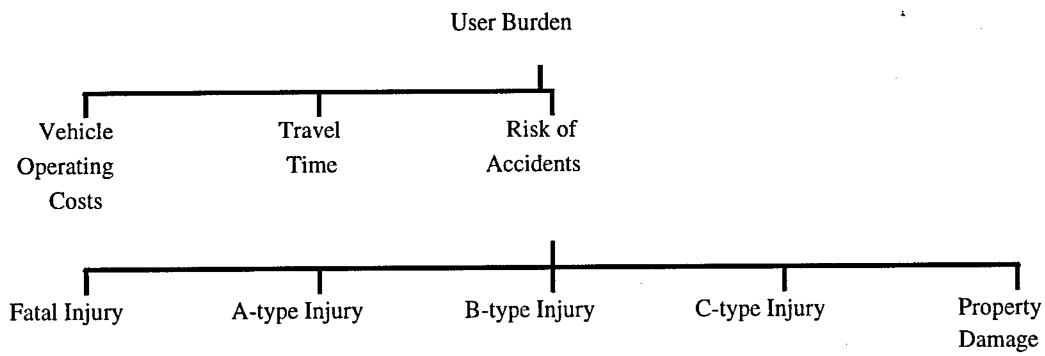

Figure 2.8: User Burden.

Costs of Vehicle Operation include, for example, fuel and maintenance costs. A road improvement that eliminates grades and curves, shortens the route, or repairs and prevents surface defects decreases the users' cost of operating their vehicles. This indirectly affects the government as well, as pointed out by Grenzeback and Woodle (1992), in the sense that less time on the road reduces the number of accidents and the environmental impact caused by traffic.

One of the most common reasons for making improvements is to increase speed or to reduce Travel Time. A typical stream of traffic contains both personnel and commercial traffic, and the personal traffic is composed of several different types as well. For each type of traffic, the importance of travel time differs. Travel time is critical for a truck driver but much less important for a tourist or shopper.

The Risk of Accidents can be further defined by the type of accident. Motor vehicle accidents have traditionally been classified as Fatal, A, B, or C injury, or Property Damage only. A fatal injury is one that results in death within 90 days of the accidents. An A-type injury is incapacitating such that a person cannot walk or leave the scene without assistance. A B-type injury is a nonincapacitating injury that is evident to anyone at the scene, but the victim can function unassisted. A C-type injury is the least severe, and includes symptoms such as pain or nausea. Notice that these definitions do not include any reference to the permanence of the injury. A person suffering from a complex fracture of his femur is considered the same category (A-type) as a person who has actually lost a limb. Property damage includes accidents that reduce the monetary value of some property to include property other than the vehicles such as the roadway, buildings, or animals. Obviously, a single accident can result in property damage and injuries of each of the four classifications.

Alternately, the American Association for Automotive Medicine has developed a more definitive way to classify accidents, the Maximum Abbreviated Injury Scale (MAIS). This scale is used by the US Department of Transportation. The MAIS includes minor injury, moderate injury, serious injury, severe injury, critical injury, and maximum injury (death).

2.4 Comprehensive Approach to Roadway Improvement Project Selection

The Ohio Department of Transportation's (ODOT) "Draft Major/New Construction Program 1998 – 2005" (1997) contains a methodology for selecting roadway improvement projects based on multiple objectives. More exactly, the approach, which applies to capacity projects of more than $2 million only, employs a scoring method: For each objective, a project can receive up to a maximum number of point, based on its performance with regard to that objective. The points received for each objective are then added to give an overall score. The overall score serves as a ranking criterion for implementation priority, i.e. the project with the highest overall score is the "first priority" for implementation, and so on. Of course, these rankings are subsequently subject to review and possibly change. The ODOT approach uses the following objectives (called goals) (The goals are broken down into criteria, here indicated in parantheses, followed by the maximum obtainable score.):

- Transportation Efficiency (Average Daily Traffic [20], Volume to Capacity Ratio [20], Roadway Classification [5], Macro Corridor Completion [10])
- Safety (Accident Rate [10])
- Economic Development (Job Creation [10], Job Retention [5], Economic Distress [5], Cost Effectiveness of Investment [5], Level of Investment [5])
- Funding (Bonus Category; Public/Private/Local Participation [15])
- Unique Multi-Modal or Regional Impacts (Bonus Category; [10])

Following is an example of the scoring scheme, in this case for the Volume to Capacity Ratio and the Highway Classification.

Table 2.1: ODOT Scoring Scheme

V/C Ratio	Points	V/C Ratio	Points
> 1.50	20	1.00-1.04	10
1.45 – 1.50	19	0.95 – 0.99	9
1.40 – 1.44	18	0.90 – 0.94	8
…	…	…	…

Highway Classification	Points
Interstate	5
Macro-Corridor	5
National Highway System	2
Freeway/Expressway	2
Principal Arterial	2
Minor Arterial	1

It should be noted that this methodology appears very "objective" at first glance, because tables are provided for each criterion that attribute points to a given performance level. However, it should not be forgotten that this is only a seeming objectiveness, since both maximum scores per criterion (which in fact corresponds to a weighting of the different criteria) and the tables that link the performance level to the points attributed are quite subjective! Also, the safety aspect, which is of primary concern in the VDOT study, contributes only 15 points (maximum) to a total maximum of 125 points. At the same time, ODOT attempts to incorporate many of the more "political" aspects (job creation etc.) of the decision-making process into the formal assessment procedure.

The benefit of the ODOT approach is the communication of ODOT's agenda to the public, because any interested person could, in theory, replicate the Departments process of prioritization. However, since the main focus of the ODOT procedure is the final score that a project receives, the methodology is not very helpful in uncovering trade-offs between different objectives (cost, safety). As different aspects are collapsed into one score, the individual decision-maker has less of a chance to make decisions based on his judgments and priorities with regards to the various objectives. Hence, the ODOT methodology tends to be much more "prescriptive" than the approach that is presented here (VDOT study).

CHAPTER 3
SELECTION AND ADOPTION OF ASSESSMENT TOOLS

This section will detail the recommended tools for the assessment of crash risk, performance gain, and cost.

3.1 Selection and Adoption of Crash Risk Assessment Tool

In this chapter, ARFs will be discussed in more detail since it is suggested that they be used for the assessment of crash-risk reduction. While not the most detailed approach, it is felt that the use of ARFs is the most feasible tool, considering the (limited) information and time at hand for planning-level decisions.

ARFs can be used to estimate the reduction of crashes at a particular road-site/segment of road following the implementation of a specific accident countermeasure.

The ARF-approach is "generic" in that it requires that a projected/implemented accident countermeasure be categorized as one of the predefined countermeasures and that it assumes that a particular countermeasure will always result in a reduction of the number of accidents of x% per vehicle that is passing the particular road section, regardless of the specific circumstances of the individual implementation (i.e. the current design parameters). The ARF-approach is based on "Before & After" studies which serve to determine the anticipated reduction of the number of accidents for a future project: It is assumed that, if a certain countermeasure, say "add left-turn lane (to intersection)", has resulted in an average reduction the number of accidents of x% for past implementations, it will result in a reduction of x% for future projects as well.

From a scientific point of view, it would be preferable to use a "control group" (i.e. sites where the countermeasure is not implemented) to determine the effects of a countermeasure. However, this strategy is usually deemed impractical, because an identified high-risk site should not remain unchanged for the sake of research alone. Hence the use of "Before & After" studies at the same site. It should be noted that problems arise from the fact that the "x%" is an averaged figure, which may not hold for a specific (future or past) project, and that sufficient data may not be available to calculate a meaningful ARF for a specific countermeasure.

Rather than just using one overall ARF per countermeasure, differentiated ARF can be established, e.g. differentiated by accident type (rear-end etc.) or consequence (fatality, injury, property-damage only).

3.1.1 Use of Accident Reduction Factors

ARFs are typically used to assess benefit cost (BC) ratios. BC ratios allow the ranking of different projects based on the "dollars of benefit" that result from the "dollars of cost" for a given alternative. Wattleworth et al. (1988) used ARFs in their work for the Florida Department of Transportation (FDOT) in order to better assess benefit-cost ratios. The ARFs were determined to be the most cost-effective and least data-intensive method for calculating a risk improvement after a poll of 48 state highway transportation departments. The B/C ratio will be further discussed in Section 3.1.2.3. MacFarland (1979) employs accident reduction factors with several other techniques to perform an economic analysis on the loss of life.

3.1.2 Estimation of Accident Reduction Factors

The accident reduction factor survey performed by Wattleworth et al. (1988) for the Florida Department of Transportation provides reduction factor models that are able to project accident rates after improvements. The Florida researchers compared accident and other data three years before and three years after an improvement was made. The goal of the study was to improve estimates of benefit/cost ratios for safety improvements for the state of Florida.

Several accident databases were integrated to constitute the FDOT accident information. Fields included: district, milepost, section, date, time, fatalities, and injuries. The data were categorized into periods three years before the improvement and three years "after". These accidents were then linked to records of project information which included project number, improvement type, location, and construction period.

The accident rates were computed as

$$r = \frac{1{,}000{,}000(n)}{(t)(ADT)} \quad \text{for intersections, and}$$

$$r = \frac{1{,}000{,}000(n)}{(t)(ADT)(l)} \quad \text{for sections,}$$

where

r	= accident rate
n	= number of accidents during observation period
t	= observation period (days)
l	= section length (miles)
ADT	= Average Daily Traffic (vehicles per day)

These rates were expressed in accidents per million vehicles at intersections, and accidents per million vehicle miles for sections.

An accident reduction percentage was then calculated for each specific roadway improvement. It should be pointed out that the study did not include specific roadway characteristics at the time of the accident such as: pavement (wet or dry), degree of turns, angle of slope, roadway width, etc. The roadway geometries were taken into account with the improvement project itself, e.g., "widening of lanes". Therefore, as mentioned above, it is not possible to exactly predict the crash risk reduction for a given intersection or roadway because the accident reduction factors are averaged over a variety of different pre-existing situations. The ARFs can only serve as a very general tool for the characterization of planning-level decisions. Table 3.1. gives an example of typical before and after conditions at an improvement site.

The percentage of accident reduction, i.e. the Accident Reduction Factor, is calculated as the difference in rates over the previous rate:

$$ARF = \frac{100(r_{before} - r_{after})}{r_{before}}$$

The following calculation provides an example of estimation of the ARF for a particular class of projects. The example applies to a project involving "New Roadway Segment Lighting," improvement type 65, following the classification of Wattleworth et al. (1988).

Table 3.1a: Before Project.

Project	1	2	Total
Total Accidents.	332	160	492
Project Length (mi.)	2.3	1.9	
Mean ADT (veh./day)	15836	13523	
Study Period (years)	3	3	

Table 3.1b: After Project.

Project	1	2	Total
Total Accidents	174	113	287
Project. Length (mi.)	2.3	1.9	
Mean ADT (veh./day)	15638	15630	
Study Period (years)	3	3	

The accident rate before an improvement is calculated as the total number of accidents at a particular location divided by the Average Daily Traffic, number of days in a year, and length of the section in miles.

$$r_{before} = \frac{n}{(t)(ADT)(l)} = \frac{492}{39.822} = 7.233 \text{ accidents / year}$$

The accident rate after an improvement is calculated in the same manner.

$$r_{after} = \frac{n}{(t)(ADT)(l)} = \frac{287}{71.092} = 3.922 \text{ accidents / year}$$

These rates are the number of accidents per vehicle-mile. In the given example, the ARF is found to be:

$$ARF = 100\%\left(\frac{r_{before} - r_{after}}{r_{before}}\right) = 100\%\left(\frac{7.223 - 3.992}{7.223}\right) = 45\%$$

Thus, the improvement caused a 45% decrease in the number of accidents for the exposure of the roadway.

3.1.3 Hypothesis Testing and Confidence Intervals for Accident Reduction Factors

In order to assess the results of a countermeasure, a statistical hypothesis test is performed. The null hypothesis H_0 to be tested is: "The accident rate *before equals* the accident rate *after* improvements have been made." (Wattleworth et al. 1988) It is hoped that this hypothesis can be rejected in favor of the alternative hypothesis H_1. In the given situation, H_1 is: The accident rate *after* is *less* than the accident rate *before* improvements have been made. Following a commonly accepted idea (cf. Wattleworth et al. 1988), the occurrence of accidents can be assumed to follow a Poisson distribution. A Poisson Distribution is described by the equation:

$$P(x; \lambda) = \frac{e^{-\lambda} \lambda^{x}}{x!}$$

where

λ = mean rate of occurrence of accidents (accidents per year)
x = number of accidents in a year of exposure

Then, the appropriate test statistic for a Poisson Distribution Test will be (cf. Montgomery 1991):

$$Z_0 = \frac{\overline{x} - \lambda_0}{\sqrt{\lambda_0 / n}}$$

where

$\overline{x}$ = number of accidents per year observed after improvement (accidents/year)
λ_0 = base rate (before improvement) of accident occurrence (accidents/year)
n = number of observation periods (years)

It is assumed that λ_0 is the "true" mean of the accident occurrence rate at the project site (before any improvement is made). H_0 is rejected in favor of H_1 if $Z_0 < Z_\alpha$, where $(1 - \alpha)$ is the "level of confidence" of the test. Assuming that n is large, Z_α is the lower α percentage point of the standard normal distribution. However, it should be noted that in Wattleworth et al. (1988), the above formula is used for n = 1. Also, constant average daily traffic is implied in this calculation. For, say, a 95% confidence level, the following calculations and transformations can be made:

$$Z_{0.05} = -1.645 > \frac{\overline{x} - \lambda_0}{\sqrt{\lambda_0}}$$

$$\Rightarrow \quad -1.645 * \sqrt{\lambda_0} > \overline{x} - \lambda_0$$

$$\Rightarrow \quad \frac{-1.645}{\sqrt{\lambda_0}} > \frac{\overline{x} - \lambda_0}{\lambda_0}$$

$$\Rightarrow \quad \frac{1.645}{\sqrt{\lambda_0}} < \frac{\lambda_0 - \overline{x}}{\lambda_0}$$ = Relative Reduction of the Number of Accidents per Vehicle (as decimal number)

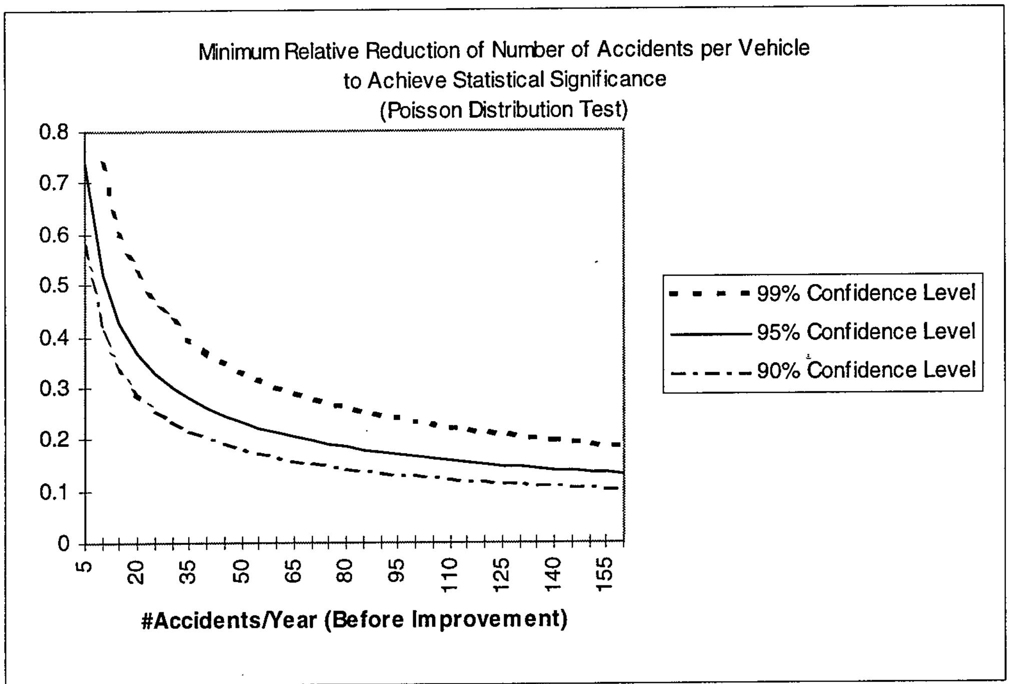

Figure 3.1: Test of Significance for ARFs.

Figure 3.1 can be used to determine if there is statistically significant evidence that an improvement has reduced the number of accidents: If the actual relative reduction lies above the curve in Figure 3.1, then the improvement can be considered statistically significant (at the chosen confidence level). Wattleworth et al. (1988) proposes the use of a Poisson Distribution Test (as opposed to a more conservative Poisson Comparison of Means Test) because the Poisson Distribution Test is used in most references as well as the Florida HSIP Manual. The Highway Safety Evaluation Procedural Guide (1981) describes the same approach as Wattleworth et al. (1988).

3.1.4 Countermeasures and Accident Reduction Factors

Virginia's ARFs (as of September 94) are listed in Table 3.2, where ARF_I refers to accidents involving injuries and/or fatalities, ARF_P to accidents with property-damage only. The table includes the expected Service Life for the different countermeasures.

Table 3.2: ARFs for Virginia (VDOT, September 94).

Countermeasure	**ARF_I**	**ARF_P**	**Service Life (Years)**
Widen Pavement	0.5	0.5	20
Widen Pavement (Additional Lane)	0.5	0.5	20
Widen Shoulders	0.5	0.5	20
Widen Pavement and Improve Alignment	0.87	0.73	20
Grooving	0.68	0.61	10
Widen Bridge	0.92	0.95	20
Eliminate Substandard Bridge	0.5	0.5	30
Improve Horizontal Alignment	0.87	0.73	20
Improve Vertical Alignment	0.87	0.73	20
Install Railroad Protective Devices	0.5	0.5	10
Signing	0.5	0.5	6
Install Guardrail	0.5	0.5	10
Median Barrier	0.5	0.5	15
Install Roadside Delineators	0.5	0.5	2
Impact Attenuators	0.5	0.5	10
Channelization	0.29	0.58	10
Left/Right-Turn-Lane (LTL/RTL):			
LTL 2-Lane Highway	0.29	0.58	10
LTL 4-Lane Divided Highway	0.29	0.58	10
Extend LTL 4-Lane Divided Highway	0.29	0.58	10
RTL 2-Lane Highway	0.29	0.58	10
RTL 4-Lane Divided Highway	0.29	0.58	10
Install Traffic Control Signals	0.5	0.5	10
Modify Existing Traffic Control Signals	0.5	0.5	10
Install Flashing Caution Signal	0.5	0.5	10
Install Flashing Lights on Signs	0.5	0.5	10
Improve Sight Distance	0.57	0.79	10
Raised/Recessed Pavement Markers	0.5	0.5	2
Illumination	0.22	0.5	15
Bridge Approach Guardrail Transition	0.92	0.5	10
Roadside Object	0.24	0.5	10

It should be noted that the origin of these ARFs is unclear (at this point); more recently (1996), ARFs from New York have been adopted in Virginia. New York has an extensive program which logs all improvement projects and is able to update ARFs on an annual basis with new information. The Post-Implementation Evaluation System (PIES) quantitatively estimates the expected crash reduction from a specific improvement type. Note that PIES is also able to elicit differentiated ARFs for different types of accidents (rear-end etc.). In their study for the Florida Department of Transportation, Wattleworth et al. (1988) have identified 103 countermeasures. In this study, sufficient data were available only for 58 out of these 103 possible actions; out of these 58, 24 were found to have significant ARFs on "all accidents" for the state of Florida. A very comprehensive overview of ARFs used across the United States and proposed in the literature can be found in a study from 1996 which was conducted for the Kentucky Transportation Cabinet (Agent et al. 1996). Tables with countermeasures and associated ARFs from this study, as well as from Wattleworth et al. (1988) and ARFs used in the state of New York can be found in Appendix B.

3.1.5 Working with Accident Reduction Factors

The following is adapted from Highway Safety Improvement Program: How to Propose a Highway Safety Project. The first step in working with ARFs, once they have been estimated, is the identification of road-sites with significant accident histories. To that end, accidents can simply be plotted on a map. As a more sophisticated method, VDOT has adopted the Rate Quality Control Method, which was developed in Evaluation of Criteria for Safety Improvements on the Highway by Roy Jorgensen and Associates.

Rate Quality Control Method

The following description of the RQCM follows Wattleworth et al. (1988). In order to identify hazardous locations, the RQCM calculates a safety ratio, a ratio of greater than 1 indicating a high accident location. To perform the calculations, accident locations are separated into spot and segment locations. In the case of spot locations, the accident rate is correlated to an exposure measurement of the number of entering vehicles. For segment locations, the accident rate is correlated to the number of vehicle miles traveled. The safety ratio is calculated as:

$$\text{Safety Ratio} = \frac{\text{Actual Accident Rate}}{\text{Critical Accident Rate}}$$

where

$$\text{Actual Accident Rate} = \frac{\#\text{Accidents / year} \;*\; 1{,}000{,}000}{\text{ADT} \;*\; 365 \;*\; \text{length}} \quad \text{for segment locations}$$

and

$$\text{Actual Accident Rate} = \frac{\#\text{Accidents / year} \;*\; 1{,}000{,}000}{\text{ADT} \;*\; 365} \quad \text{for spot locations .}$$

For Segment locations (with a length of 0.11 to 3 miles), the critical accident rate is defined as:

$$L_p = L_c + K\sqrt{L_c/M} - 1/2M$$

where

L_p = Critical Accident Rate

L_c	= Average Accident Rate	
M	= Average Vehicle Exposure for one year at the location (million vehicle miles)	
K	= statistical constant at 95% and 99.95% confidence levels, where	
	K = 1.645 for rural areas (95%)	
	K = 3.291 for urban areas (99.95%)	

For Spot locations (with a length of no more than 0.10 miles), the critical accident rate is defined as:

$$A_p = A_c + K\sqrt{A_c/V} - 1/2V$$

where

A_p	= Critical Accident Rate
A_c	= Average Accident Rate
V	= Average Vehicle Exposure for one year at the spot (million vehicles)
K	= statistical constant at 95% and 99.95% confidence levels, where
	K = 1.645 for rural areas (95%)
	K = 3.291 for urban areas (99.95%)

For use with the RQCM, VDOT's Traffic Engineering Division computes accident rates on roadway segments and at intersections each year.

Note that VDOT uses the RQCM in such a way as to identify the highest 10% of accident rates to be dangerous (cf. Highway Safety Improvement Program: Accident Rates/Critical Rates).

ARFs can be used in the following way to estimate the number of accidents avoided per year:

1. Estimating the Current, Before-Improvement Accident Occurrence per Year

If the distribution of the number of accidents per year at a given location can be assumed to be Poisson, then (interarrival-) times between accidents follow an exponential distribution:

$$f(t) = \lambda * e^{-\lambda t}, \qquad t \geq 0$$

where

t	= time until (next) accident
λ	= mean of accident occurrence (per year)
f(t)	= probability density function for interarrival-times

An unbiased estimator for the accident occurrence per year λ is (Hoyland/Rausand 1994, p.373):

$$\bar{\lambda} = x / n$$

where

x	= total number of accidents
n	= number of observation periods (years)

If, for a given location, the number of accidents per year is unknown, but the accident rate r and the Average Daily Traffic ADT are at hand, the following calculation can be made:

$$x = \frac{r * 365 * ADT}{1{,}000{,}000} = \bar{\lambda} \qquad \text{(since n = 1)}$$

where

$$r = \frac{1{,}000{,}000(x)}{(365)(ADT)}$$

for intersections, and

$$x = \frac{r * 365 * ADT * 1}{1{,}000{,}000} = \bar{\lambda} \qquad \text{(since n = 1)}$$

where

$$r = \frac{1{,}000{,}000(x)}{(365)(ADT)(l)}$$

for sections (with l being the length of the section).

2. Estimating a Confidence Interval for Accident Occurrence per Year

The two-sided confidence interval for a (1-ε) confidence level for $\bar{\lambda}$ is (Bourne/Green 1972, pp. 340-346):

$$\frac{\bar{\lambda} * \chi^2_{11}}{2\,x} \leq \lambda \leq \frac{\bar{\lambda} * \chi^2_{22}}{2\,x}$$

$$\Rightarrow \quad \lambda_L = \frac{\chi^2_{11}}{2\,n} \leq \lambda \leq \frac{\chi^2_{22}}{2\,n} = \lambda_U$$

where

> χ^2_{11} is that value which is exceeded by 100[1 - ε/2]% of values generated by a chi-square distribution with (2x) degrees of freedom,
> χ^2_{22} is that value which is exceeded by 100[ε/2]% of values generated by a chi-square distribution with (2[x+1]) degrees of freedom.

It should be noted that different degrees of freedom are used for the lower and upper bounds. The motivation for this procedure is as follows: It is assumed that the observation of a location does not stop at the exact moment of an accident occurring, but rather at the end of a certain, predefined time period (e.g. end of calendar year). An "optimistic assumption" can be made, relating the x observed accidents to the total observation time; or a "pessimistic assumption" can be made by assuming that an additional accident was just about to occur when the observation ended. Rather than choosing either the optimistic or the pessimistic assumption, both are often combined to generate a confidence value with a maximum spread, as done above.

Figure 3.2 demonstrates how the confidence interval for the yearly number of accidents shrinks as the observation period becomes longer. For this example, it is assumed that the actual data is such that, no matter how long the observation period, $\bar{\lambda}$ is estimated to be 30 accidents per year (i.e. 30 accidents observed over 1 year, 90 observed over 3 years, etc.). Also, it is assumed that a 95% confidence level is desired.

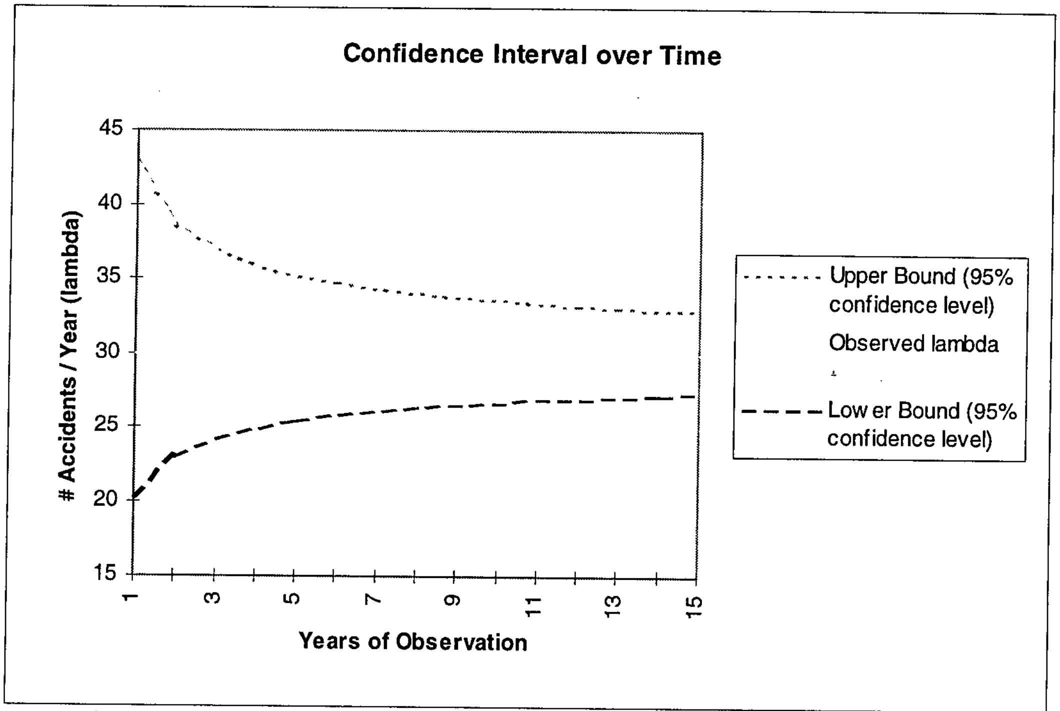

Figure 3.2: Influence of Increased Information on Confidence Interval.

3. Select Appropriate ARF

Depending on the chosen countermeasure which is to be implemented, an appropriate ARF has to be selected from the table of ARFs. (For tables of ARFs, including those currently used by Virginia, see Section 3.1.4.)

4. Estimating Crashes Avoided per Year

Once λ is known (more specifically, λ_L, λ, and λ_U), the number of crashes avoided per year can be estimated using the formula

$$\text{Crashes Avoided / Year} = \lambda * \text{ARF}$$

(To avoid confusion between the effects of the project and a change in Daily Traffic, the daily traffic is assumed to be constant.)

By substituting either λ_L, λ, or λ_U into the above formula, a range and mean for crashes avoided per year can be found. Also, if a more specific figure than ATGR is available for the given project, it should be used instead of the average number.

5. Break-Down by Accident-Types

The above description assumes that only one overall ARF is available for a certain countermeasure. However, if more detail is desired, the described calculations can be performed several times, for different types of accidents (fatal, injury, property-damage-only; or rear-end, angle, etc.). This decision will depend on the availability of both differentiated ARFs and accident occurrence data.

ARFs are also used to calculate Benefit/Cost (BC) ratios for the different projects in order to prioritize them according to "dollars of benefit" per "dollar of cost." Virginia uses the following BC ratio:

$$BC = \frac{\sum((NFI * QDollars * ARF_I) + (NPD * AAPD * ARF_P))_{improvement} * ATGR}{(PECost + RWCost + UtilCost + ConstCost) * CRF}$$

where

NFI	= # related fatal and injury accidents per year
QDollars	= Weighted average cost of fatal and injury accidents at all similar locations
ARF_I	= Percent reduction in fatal and injury accidents
NPD	= # related property-damage only accidents per year
AAPD	= Annual average cost of property-damage only accidents
ARF_p	= Percent reduction in property-damage only accidents
ATGR	= Projected district annual traffic growth rate

The numerator of the BC ratio is the sum of the estimated reduction in accident costs due to each improvement and represents the annual safety benefits of the project.

PECost	= Estimated preliminary engineering cost
RWCost	= Estimated right of way costs
UtilCost	= Estimated utility relocation cost
ConstCost	= Estimated construction cost
CRF	= Capital recovery factor

3.2 Selection and Adoption of Performance Assessment Tool

The selection of an effective measure for performance is a challenging problem. Unlike crash risk, performance is generally not well-defined, and may change meaning according to the situation in which it is being considered. Some measures are available in VDOT's information system, but others may be impossible to obtain.

Any discussion of performance must start with a discussion of terminology. The Highway Capacity Manual (HCM) subdivides traffic into two categories: uninterrupted flow and interrupted flow. These classifications break down further: uninterrupted flow includes freeways and long stretches between traffic signals on roadways. Interrupted flow can include (but is not limited to) signalized intersections, unsignalized intersections, and RR crossings.
The following definitions are found in the 1994 HCM:

Capacity - The maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of lane or roadway during a given time period under prevailing roadway, traffic, and control conditions. Can be broken down into vehicle capacity or person capacity. Most often, capacity is measured over a specified 15-min peak period.

Levels of Service (LOS) - Qualitative measures that characterize operational conditions within a traffic stream and their perception by motorists and passengers - 6 levels

Table 3.3: Measures of Effectiveness (MOE) for Level of Service Definition

	MOE
Basic freeway segments	Density (pc/mi/ln)
Signalized intersections	Average stopped delay (sec/veh)
Unsignalized intersections	Average total delay (sec/veh)

Service flow rate- Maximum hourly rate at which persons or vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given period given set conditions remaining in given LOS

3.2.1 Model Selection

Given the planning-level orientation of this study, the team recommends the use of travel time saved, expressed in terms of *minutes saved during the peak hour*. This can be calculated by multiplying the traffic volume during the peak hour by the estimated average travel time saved per vehicle, both "before" and "after" a project, although both may be estimates rather than observations. This measure has numerous advantages over the others considered. First, it is equally applicable for intersection, segment, or any other type of facility improvement. Second, it can be used in conditions of both data adequacy and data scarcity, with only the levels of uncertainty (confidence intervals) being changed. Third, it reflects the drivers' intuitive notion of facility performance. The travel time saved per vehicle can either be estimated directly (as the average time required to traverse a road section, or average time spent in queue at an intersection), or inferred from anticipated changes in the average speed (for road sections), along with the length of the project location.

3.2.2 Calculation Method

Many possibilities exist for the calculation of travel time savings. (The team consciously avoids the use of the term "delay" here, since it has a much more specific meaning for intersection analyses in the literature.) As mentioned above, the separation of facility types in the Highway Capacity Manual and elsewhere is indicative of the necessity for several different ways to approach travel time.

Roadway segments have the following attributes:

- Average speed
- Length
- Number of lanes
- Density
- ADT
- LOS

In theory, the calculation of travel time saved is simple. For a roadway, it can be calculated as

$$T_s = \frac{(ADT_{before})(l)}{s_{before}} - \frac{(ADT_{after})(l)}{s_{after}}$$

Signalized intersections require a higher level of detail to analyze at the design level. The 1994 HCM states that unlike other facility types (such as roadway segments), signalized intersections show less of a correlation between capacity and LOS, and thus require different calculations. "It is critical to note at the outset that both capacity and LOS must be fully considered to evaluate the overall operation of a signalized intersection." (HCM 1994) Capacity is calculated for each lane group (lanes that have a common stop line). Volume/Capacity ratios of a designated lane group during a peak 15-minute interval are then representative of the intersection. LOS is more strongly affected by quality of progression, length of green phases, cycle length, and other factors.

For an intersection, the calculation is somewhat more complex than it is for a segment. The HCM specifically addresses the issue of delay as pertaining to intersection analysis. Subtracting the total delay before a project from the total delay after the project will yield total travel time saved. "For planning purposes, it may be more appropriate to consider the provision of adequate future capacity as related to geometric design features. Delay must be less of a concern, because it may be improved significantly through coordination of signals and improved signal design." However, the HCM goes on to state that "in the analysis of existing problem locations, delay may be a more significant consideration when improved controls are concerned." (HCM 1994)

Estimates for total delay can be made through observation or detailed analysis at the design level. For example, one may estimate delay from a sample distribution at a site and project with expert opinion. An average car at a two-lane intersection may experience an average total delay of two minutes to get through the light. However, 40% of the cars are waiting to turn left and backing up traffic as opposing traffic approaches. An engineer could speculate that the addition of a left turn lane would decrease the total delay of 50-60% of the cars by 1 minute.

Alternatively, the following calculations from the HCM could be made in order to estimate actual delay figures:

Estimate for Average Total Delay

$$D = \frac{3600}{c_{m,x}} + 900T\left[\frac{v_x}{c_{m,x}} - 1 + \sqrt{\left(\frac{v_x}{c_{m,x}} - 1\right)^2} + \frac{\left(\frac{3600}{c_{m,x}}\right)\left(\frac{v_x}{c_{m,x}}\right)}{450T}\right]$$

D = average total delay (sec per vehicle)
v_x = volume for movement x, expressed as hourly flow rate
$c_{m,x}$ = capacity of movement x, expressed as an hourly flow rate
T = analysis period (hr) (for a 15-min period, use T=0.25)

$$D_A = \frac{D_r V_r + D_t V_t + D_l V_l}{V_r + V_t + V_l}$$

D_A = average approach delay (sec per vehicle)
D_r, D_t, D_l = computed average total delay for right-turn, through, and left-turn movements
V_r, V_t, V_l = volume or flow rate of right-turn, through, and left-turn traffic on the approach

Total average delay for intersection can be expressed as

$$D_I = \frac{D_{A,1}V_{A,1} + D_{A,2}V_{A,2} + D_{A,3}V_{A,3} + D_{A,4}V_{A,4}}{V_{A,1} + V_{A,2} + V_{A,3} + V_{A,4}}$$

$D_{a,x}$ = average approach total delay on approach
$V_{a,x}$ = volume or flow rate on approach x

Estimates can also be generated for intersection and road segments using models not included in the Highway Capacity Manual, for instance. Additionally, there are many proven methods for the reliable incorporation of expert judgment into such estimates that are available. These alternatives are presented below.

Expert Estimation
It is feasible for expert estimation of travel time reduction to be incorporated into the decision framework in a way that yields meaningful results while accounting for the necessarily subjective nature of the estimate. This can be accomplished through several methods, most notably the use of the triangular distribution and the fractile method. These methods are discussed in detail in section 3.3. Essentially, an expert familiar with the site provides estimates of the minimum, maximum, and mean travel time differential that will result from a project, and this is used to construct a probability distribution. In the fractile method, the 50% confidence interval is also obtained. In the absence of useful data, this is a good way to fit the qualitative intuition of the engineer into a quantitative decision analysis framework.

Indirect Observation
In some cases, it may be desirable or necessary to observe travel time indirectly. Provided that the estimates obtained for certain nonobservable parameters are chosen properly, this observation can lead to a more rational and mathematically consistent formulation of travel time saved than direct estimation.

It is possible to say

$$T = kV\frac{D}{C}$$

where T is the travel time added due to congestion, C is the physical capacity of the facility, V is the actual of volume of traffic during the peak hour, and D is the demand or number of vehicles that are vying for use of the facility during the peak hour. Here, k is an unknown constant.

This is simply a mathematical statement implying that travel time added is directly related to the ratio of demand over capacity; when this is greater than one (and hence demand is greater than capacity), the travel time added increases, and when it is less than one, the travel time decreases. The actual volume is also directly related to the travel time added, meaning that as the amount of traffic on a facility increases, the travel time added due to congestion for each vehicle increases as well.

Capacity and volume can both be observed, and are recorded in VDOT's databases. Demand can be estimated using factors which account for the size, growth, economic character, and type of the district within which the potential project site is located. Once the constant k is calibrated effectively using known examples, this method could be very useful. In fact, in situations where the district's planners have reason to believe their demand estimates are more reliable than the engineer's travel time estimates, this method would be preferred.

Simulation
As mentioned in section 2.2.5, simulation models are relied upon by VDOT to provide the most reliable and cost-effective estimates of performance criteria for projects that are currently available. Although the formulation of a distribution requires some observation, once this is done different scenarios can be explored at very little cost.

Unfortunately, simulation analyses are still too costly and time-intensive to be used for any but the largest capacity-based projects. It is the recommendation of the study team that if simulation results are available, they should be used, and the higher confidence placed in the output in such simulations should be taken into account.

As indicated in section 3.2.1, it is proposed that, when detailed simulation data (or similar) is not available, travel time saved is derived from basic expert estimates of average changes in travel time per car, or changes in average speeds through the site, along with information on daily traffic. The appropriate calculations have been implemented in the performance gain worksheet in the Workbook. If a direct estimate of the time saved per vehicle is available, then that estimate needs only to be multiplied by the appropriate average number of vehicles that enter the site per peak-hour. If estimates of "without-" and "with-project"-speeds are available, then an average travel time (without and with project) can be found by dividing the length of the site by the appropriate speed, which yields the travel time. The travel time saved per vehicle is the difference of the computed travel times without and with project.

3.3 Selection and Adoption of Cost Assessment Tool

Cost analysis presented in this section can be applied at several levels of project comparison: design alternatives for a specific project, potential projects within a district, and a portfolio of projects at the state level. It was chosen to measure project cost as the summation of preliminary engineering, right of way, and construction costs

3.3.1 Cost as a Constraint

In the context of the Six year Plan, cost is a constraint rather than an measure endpoint. Cost provides a bound on the number and types of projects that VDOT is able to approve. It can even be argued that cost is often a dominating constraint over others such as materials, manpower, and weather conditions.

Cost cannot be considered outside the realm of other project objectives. For example, it is often presented as an objective to "utilize all available federal funds" (Programming and Scheduling Manual, 1988). The Six year Plan also states that it is an objective to "maximize the use of state and federal construction funds" (1990/91 Six Year Plan). However, what does this contribute to the overall goal of making roadways safer and more efficient? Objectives included in this model (crash as reduction in crash risk and performance increase) as well as others not addressed, such as political, environmental, and public relations will always enter into the decision. It should be kept in mind that the methodology presented in this report does not intend to and cannot produce final decisions. It can only provide an aid in the decision process. Other factors, such as funding issues, will have to be considered additionally.

3.3.2 Current Practice

The Six Year Plan is updated annually with two distributions: one in May after the Commonwealth Transportation Board approves the tentative program, and the second in July after the Board approves the final program. In this plan, three cost estimates are documented for each project: Preliminary Engineering (PE), Right of Way (RW), and Construction (CN). A total cost estimation is the summation of each of these expected costs.

Cost estimations can be made at the residency level or the state planning level. They are derived from three major sources: similar projects done in the past, normalized cost estimates for a particular improvement, and expert judgment of engineers. For example, if a project is to widen a length of three miles from two lanes to four, the cost may be calculated from averaged historical data:

$$3 \text{ miles} \times 2 \text{ lanes} \times \frac{\$8{,}000{,}000}{\text{lane mile}} = \$48{,}000{,}000$$

After a project has completed most of its preliminary engineering, the Right of Way and Construction estimations are often revised with greater accuracy. However, all cost estimates are expected values and do not directly reflect the increased confidence in that estimate. Many feel that the unpredictability of cost overrun makes the initial estimates a shot in the dark, but the refined estimates can be relied on much more heavily.

Cost should be estimated as closely as possible to maximize benefit to VDOT. Accurately estimating costs and its probability of overrun will:

- Assist in the maximization of the use of federal and special program funding
- Lessen the burden of project cost overrun on districts

3.3.3 Confidence in Estimates

Confidence in cost estimates is indirectly taken into consideration in the Six Year Plan. For example, the Program Development Unit works to "identify projects to utilize approximately 50% more federal funds than are anticipated." This practice allows for delays in project development and other causes necessitating a change from federal to state project funding while enabling VDOT to continue utilizing all available federal funds. (Programming and Scheduling Manual) A risk of cost overrun and inaccurate measurements is therefore damage to district budgets.

There is a definite stigma to cost overrun, because it is often viewed as preventable. However, large cost overrun has come to be expected in project funding processes. Cost overrun can be related to a variety of causes: unknown natural conditions (e.g., discovered slab of rock in land being developed for a highway), time delay costs, contractor shortcomings, or poor management. New development projects will likely have much higher costs overruns than those repeated on a regular basis. It is important for VDOT to not only recognize but analyze the potential for cost overrun. Confidence intervals in an estimate are able to relate not only the expected cost, but also the probabilities of cost overrun. Bridge projects, for example, may have estimates that are much closer to actual cost than a roadway improvement because they have fewer design variables. This difference in uncertainty can be relayed by assigning a percentage uncertainty to the estimated cost. (i.e., the project is estimated to cost $20,000 ± 20%)

Project decisionmakers must be able to analytically assess the likelihood of a project going over (or under) budget. Statistical methods presented here need not be used in every project assessment, however they can be a helpful additional perspective to projects facing high probabilities of cost overrun.

Triangular Distribution

The triangular distribution can give the decisionmakers a general picture of the potential for cost overrun. In the triangular distribution assessment method, the engineer is asked the following questions:

1. What is the most likely total cost of the project?

2. What is the best case cost for the project?

3. What is the worst case cost for the project?

Note that these questions address the total cost of the project. In the Six Year Plan, total cost is comprised of Primary Engineering, Right of Way, and Construction.

$$\text{Total Project Cost} = PE + RW + CN$$

It is possible to perform separate distributions for each category of cost and then combine for a cumulative distribution.

Example
Project Proposal for a two and a half mile lane widening project
1. Most likely total cost: $8,500

2. Best case total cost: $7,000

3. Worst case cost: $10,000

The worst case cost takes into consideration the possibility of time delay and unknown hazards. The worst case cost can be estimated from similar projects done in the past. VDOT has an adequate database of planned and actual spending outcomes for standard projects. The engineer could examine a sampling of these to determine the likely best and worst case scenarios.

The height of the triangular distribution is calculated using the equation of area of a triangle:

$$A = \frac{1}{2}\,\text{base1} * \text{height} + \frac{1}{2} * \text{base}_2 * \text{height} = 1$$

$$\text{height} = 2/\text{base} = 2 / (10{,}000 - 7{,}000) = 0.00067$$

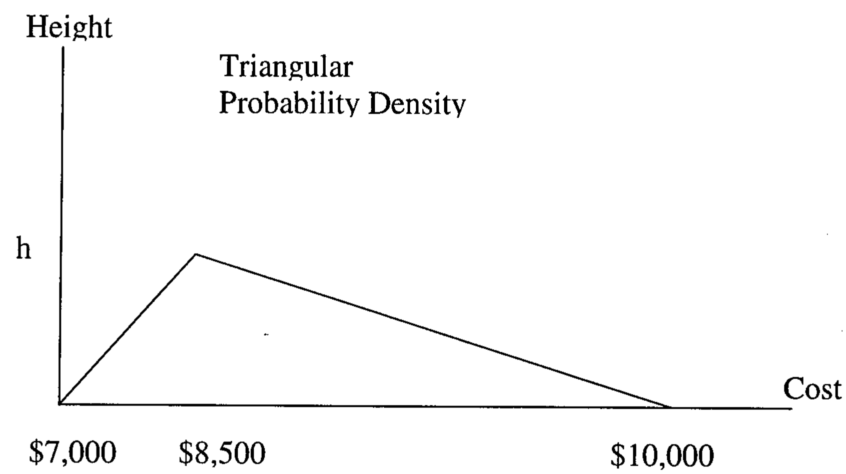

Figure 3.3: Triangular Distribution Method.

A project with a greater risk of cost overrun will have a longer tail. This tail represents the risk of extreme events: those situations which are probabilistically low, but have grave consequences. Note that Project B has a greater probability of more cost overrun.

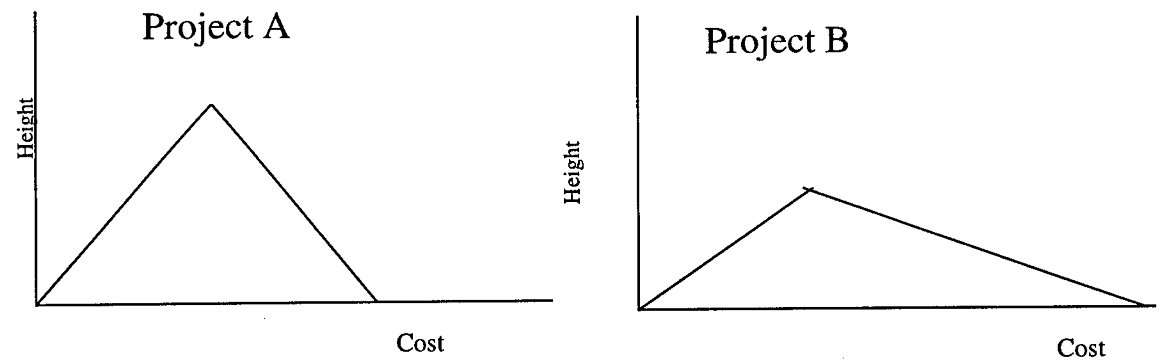

Figure 3.4: Project Triangular Distribution Comparisons.

Fractile Method
The fractile method adds more expert opinion into the model which reflects a more accurate depiction of the chance of cost overrun.
There are two additional questions in the fractile method:
1. What is the most likely cost of the project? (50% fractile)
2. What is the best case cost of the project? (0% fractile)
3. What is the worst case cost of the project? (100% fractile)
4. What is the median value of project cost increase? (25% fractile)
5. What is the cost with a 25% probability above the median? (75% fractile)

In the same example:
1. Most likely total cost: $8,500
2. Best case total cost: $7,000
3. Worst case cost: $10,000
4. There is a 25% probability that the cost will be below $7,500
5. There is a 75% probability that the cost will be below $9,500

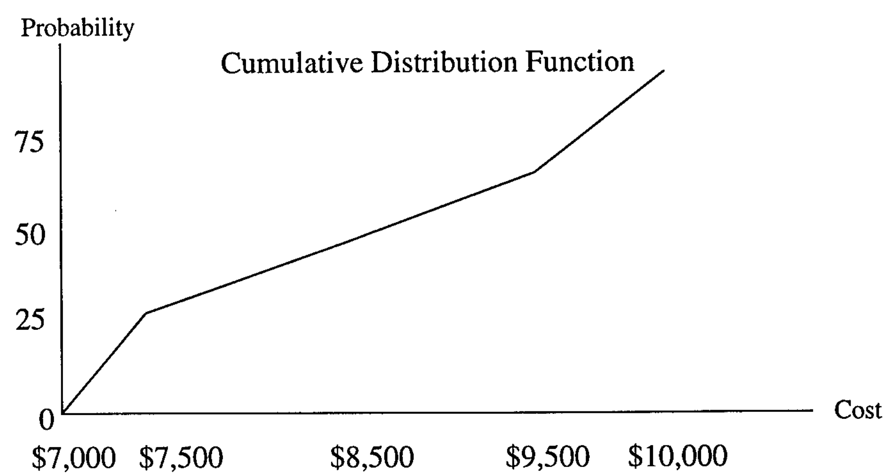

Figure 3.5: Cumulative distribution Function with the Fractile Method.

Note that this information gathered from the fractile method can be easily integrated into the Comparison Tool. The 25^{th} fractile is \$1000 less than the expected cost. Likewise, the 75th percentile is \$1000 greater than the expected cost. One can then represent the uncertainty with these two fractiles. The cost estimate becomes \$8,500 ± \$1000.

Note that before preliminary engineering is performed, the project will have a total cost estimate. However, after PE, the cost estimate will have a much smaller confidence interval.

3.3.4 Post Evaluation

A secondary benefit of this method is that over time, VDOT engineers will likely be making better cost estimations. Sources of cost overrun may also be better identified and understood through the process of confidence assignment. If the sources of cost overrun are identified, they may perhaps be better avoided in the future. Post evaluation of cost assessment is encouraged by new VDOT initiatives. For example, the VDOT Strategic Outcome Area document prepared by the MIS 2000 committee specifically designates planned vs. actual outcomes for usage of federal and state funds as a performance measure for financial measurement.

It is not necessary to discriminate between the first and second estimate because these accuracy difference swill be displayed in the confidence intervals. Most likely, the second estimate will display a closer interval. The Six Year Plan acknowledges the difference in these estimates by distinguishing the planning and engineering estimate in separate columns. A planning estimate is often developed with a great deal of uncertainty as to the final project specifications, scope, and design. However, as planning progresses, project plans become more detailed and explicit. The estimate is then refined with this new information. By the time of the final field inspection, the engineering estimate is prepared and the plans are approximately 50% complete. "The engineering estimate is far more reliable and is not expected to vary significantly from actual contract process." (1990/91 Six Year Plan)

3.3.5 Additional Expenditures

Previous funding will not be an element in this decision tool. These figures are standard in the Six Year Plan. The review of the Composite of Allocations and the Expenditures Report will flag "project activities that have been opened to charges and are exceeding authorized expenditures." (Programming and Scheduling Manual, 1988) These unanticipated allocations will certainly be a factor in the project selections, but like other political considerations, the tool will not specifically target these problems.

Maintenance costs will also not be taken into consideration because they do not appear within the framework of the Six Year plan. However, these may be entered as additional information in a project Comparison Table.

In the review of existing cost assessment tools, cost savings measures are discussed. For example, in a project that reduces crash risk, there will be a monetary savings to the drivers as well as any litigation costs that VDOT could face. In capacity projects, there is often a savings in fuel consumption and the time value of the drivers. Calculations for these savings are valuable tools for future projects in VDOT. However, the Comparison Tool will not present these cost savings directly because they are addressed in a multiobjective manner. When there is a crash risk reduction, that value is denoted on the graphical analysis and accompanying table. A capacity improvement is related in time saved. There is an implicit "cost savings" with both of these occurrences, but the improvement has already been accounted for. Therefore, an additional display of these benefits would be double counting. The only costs associated directly in the model will be capital costs.

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APPENDICES

List of Projects for Comparison Tool

	Count	Label	Index	Number	Description	Location	First Appearance in 6 year plan (From 1990)	PE	RW	CN	TOT		Completion year		Construct Start	Construct End
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
6	C1	A	1	0003-023-104, Pe103, RW203, C503	Route 3 4-lane widening from Orange County line west to east of Lignum - Culpeper Residency	2.5 mi W of Culpeper/Orange CL to 0.3 mi W of C/O CL	1990	200	650	4150	5000	9000	1994		02/17/93	09/28/94
7	C2		4	0015-068-106, Pe101	15-Orange Drainage (PE only)	Town of Orange	1990	50	0	0	50	90			03/22/93	12/04/93
8	C3	B	5	0015-056-701, pe101, rw201, m600, 0015-023-705, rw201, m400	15 - Culpeper and Madison Bridge, Replacement	Crooked Run: Culpeper/Madison CL	1993	76	39	325	440	792			03/22/93	12/04/93
9	C4		8	0015-030-114, pe101, n501	15 & 29 Fauquier close Crossover	1.7 mi S of Prince William CL	1993	2	0	13	15	27	1994		10/26/93	11/19/93
10	C5		9	0015-030-114, pe102, n502	15 & 29 Fauquier Construct Left Turn Lane - Southbound	1.6 mi S of Prince William CL	1993	4	0	31	35	63	1994		10/25/93	11/19/93
11	C6		10	p015-030-15, n501	15/29 - Fauquier Improve Park and Ride Lot	1.6 mi S of Prince William CL	1993	0	0	3	3	5.4	1993		04/07/93	04/22/93
12	C7		11	0015-030-116, pe101, m501	15 & 29 Faquier Construct Left Turn Lanes at 4 Locations	Rt. 17 (Opal), Prince William CL	1994	3	0	77	80	144			03/27/95	on hold
13	C8		15	0015-032-108, pe101, rw201, m501	15 - Fluvanna Realignment of intersection	@ Rt. 6 (Dixie)	1993	87	65	283	435	783	1994		06/24/94	06/27/94
14	C9	C	21	0020-002-s17, pe101, rw201, c501	At route 20 at Intersection Route 742 (Avon St. extended) Realigned Route 20 and improved intersection	2.9 mi S of Corporate City Limits C-ville	1990	105	186	535	826	1487	1993		07/20/92	11/03/93
15	C10	D	22	0020-002-s21, pe101, rw201, c501	20- Albemarle Improve Horizontal and Vertical Alignment	3.4 mi S of Rt. 53 to 3.8 mi S of Rt. 53	1990	70	110	415	595	1071	1994	s to v	11/01/93	01/26/95
16	C11		23	0020-068-s13, pe101, n501	20 - Orange Channelization in Northeast Quadrant to Improve sight Disptance	Intersection Rt. 522	1990	5	5	60	70	126	1992		07/23/92	11/03/93
17	C12		26	0020-068-112, pe101, rw201, c501,b604	20 - Orange Bridge and Approaches	Blue Run: 0.7 mi E of Rt. 231	1990	90	145	985	1220	2196			10/25/94	10/26/95
18	C13		27	0020-068-v12, pe101, rw201, c501, b604	20 - Orange Bridge and Approaches	Blue Run: length: 0.8 mi	1993	208	100	1042	1350	2430	1995		10/25/94	10/26/95
19	C14		38	0029-002-125, pe101, n501	29 - Albemarle Close Crossover	3.5 mi S of Greene CL	1993	2	0	11	13	23.4	1994		06/09/93	06/25/93
20	C15		39	0029-002-125, pe101, n502	29 - Albemarle Construct Left Turn Lane Southbound	3.7 mi S of Greene CL	1993	4	0	28	32	57.6	1994		05/03/93	06/25/93

	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
1																				
2																				
3																				
4	**Before (2 years)**													**After (2 years)**						
5	**Fatality**	**Injury**	**PDO**	**Persons Injured**	**$ Damage**	**Accident Rate**	**Accident Rate corrected**	**Accidents per Entering Vehicle**	**DVMT or DEV**	**Daily Traffic**	**Accidents/ Year**	**lower Bound**	**upper Bound**	**Fatality**	**Injury**	**PDO**	**Persons Injured**	**$ Damage**	**Accident Rate**	**Accident Rate corrected**
6	0	12	4	18	60800	176	1.76	3.9	12431	5650.455	8	4.5727	12.9915	0	2	2	4	10350	43	0.43
7	maintenance only							#DIV/0!												
8	0	2	0	2	4700	69	0.69	0.3	3968	7936	1	0.1211	3.61234	0	0	1	0	3000	40	0.4
9	not done?							#DIV/0!												
10	not done?							#DIV/0!												
11	maintenance only							#DIV/0!												
12	on hold							#DIV/0!												
13	0	1	1	1	10600	0.41	0.41	0.4	6650	6650	1	0.1211	3.61234	0	0	2	0	10000	0.45	0.45
14	0	3	7	6	46270	n/a	2.44221931	2.4	5609.091	5609.091	5	2.39769	9.19517	0	0	1	0	1500	n/a	0.22
15	0	7	6	13	59820	777	7.77	3.1	2291	5727.5	6.5	3.46097	11.1152	0	1	0	1	10000	51	0.51
16	0	6	7	11	51550	2.15	2.15	2.2	8269	8269	6.5	3.46097	11.1152	0	0	2	0	10800	0.3	0.3
17	same thing as C13							#DIV/0!												
18	0	2	1	3	60000	87	0.87	0.7	4680	5850	1.5	0.30934	4.38364	no HTRIS data						
19	no HTRIS data							#DIV/0!						1	0	0	3	15000	n/a	n/a
20	0	0	0	0	0	0	0	#DIV/0!			0	0	1.84444	0	0	0	0	0	0	0

	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC	BD	BE
1																					
2																					
3																					
4																					
5	DVMT or DEV	Length	Speed Without	Speed With	lower Bound Speed Without	upper Bound Speed Without	lower Bound Speed With	upper Bound Speed With	lower Bound Daily Traffic (Before)	upper Bound Daily Traffic (Before)	lower Bound Total Travel Time Without	upper Bound Total Travel Time Without	lower Bound Total Travel Time With	upper Bound Total Travel Time With	Difference of upper Bounds	Difference of lower Bounds	Average Travel Time Saved	Error Band	ARF (Fatality, Injury)	ARF (PDO)	Project Type (ARF)
6	13086	2.2	58	64	56	60	62	66	4520.364	6780.545	994.48	1598.271	904.073	1443.6	154.6714286	90.40727273	123	32	0.87	0.73	4
7																					
8	3372	0.5	48	52	46	50	50	54	6348.8	9523.2	380.928	621.0783	352.711	571.392	49.68626087	28.21688889	39	11	0.935	0.865	4,7
9																					
10																					
11																					
12																					
13	6050																		0.87	0.73	Improve Horizontal Alignment ??
14	6170								4487.273	6730.909	358.9818	807.7091	269.236	605.782	201.9272727	89.74545455	146	56	0.87	0.73	4
15	2653	0.4	47	52	45	49	50	54	4582	6873	224.4245	366.56	203.644	329.904	36.656	20.78004535	29	8	0.87	0.73	4
16	9000																		0.57	0.79	Improve Sight Distance
17																					
18		0.8																	0.5	0.5	Eliminate Substandard Bridge ??
19	n/a	0.2																	0.5	0.5	Median Barrier ??
20	31072																		0.29	0.58	Left Turn Lane

	BF	BG	BH	BI	BJ	BK	BL	BM	BN	BO	BP	BQ	BR	BS	BT	BU	BV	BW	BX	BZ
1																				
2																				
3																				
4																				
5	Accidents Saved/ Year Predicted (Fatality, Injury)	lower Bound	upper Bound	Accidents Saved/ Year Predicted (PDO)	lower Bound	upper Bound	lower Error Band	upper Error Band	Accidents Saved/ Year Predicted (Total)	Accidents Saved/ Year Observed	Cost	Count	Additional Info (Nodes)				Daily Traffic After / Daily Traffic Before	Time Saved +10 (to create offset in chart)	Acc./Saved Observed per Year	lower Bound Accidents 'With' per Year
6	5.22	2.697249	9.118282	1.46	0.3978	3.7382	3.585	6.1765	6.68	6	5000	C1	233254 & 2993ft. E to 50102 &1600ft. W				1.052691	128	5.699679046	0.863675
7				0	0	0	0	0	0	0	50	C2								0
8	0.935	0.113233	3.377537	0	0	1.5954	0.8218	4.038	0.935	0.5	440	C3	50270 & 1313ft. S to 720320 & 1208ft. E				0.849798	44	0.588374852	0.087177
9									0			C4								0
10									0			C5								0
11									0			C6								0
12									0			C7								0
13	0.435	0.011013	2.423659	0.365	0.0092	2.0336	0.7797	3.6573	0.8	0	435	C8	323301							#DIV/0!
14	1.305	0.269122	3.813763	2.555	1.0272	5.2643	2.5636	5.218	3.86	4.5	826	C9	720323				1.1	151	4.090909091	0.01019
15	3.045	1.224247	6.273858	2.19	0.8037	4.7667	3.2071	5.8056	5.235	6	595	C10	111072 & 267ft. N to 716174 & 1493ft. N				1.15801	34	5.181304184	0.01019
16	1.71	0.627538	3.721948	2.765	1.1117	5.697	2.7358	4.9439	4.475	5.5	70	C11	503471							#DIV/0!
17									0	0	1220	C12								0
18	0.5	0.060552	1.806169	0.25	0.0063	1.3929	0.6831	2.4491	0.75	1.5	1350	C13	503475 + 1050ft N to 503140 + 3780ft S (corresponds to .2mi to 1.0 mi E of Rte. 231)							0
19	0	0	0.922222	0	0	0.9222	0	1.8444	0			C14	111332 + 630ft N to 110946 + 2678ft S (corresponds to 3.4 to 3.6 mi S of Greene CL)							#DIV/0!
20	0	0	0.534889	0	0	1.0698	0	1.6047	0	0	32	C15	Using node 111332, looking from 57 ft. to 157 ft N, got 0 acc before, 0 acc after.							0

	CA	CB	CC	CD	CE	CF	CG	CH	CI	CJ	CK	CL	CM	CN	CO
1															
2			Note: In order to achieve (approx.) 95% conf. Intervals for the # of crashes avoided,												
3			68% conf. Intervals were used for the 'with' and 'without' data. ((0.025)^0..5 = 0.16)												
4			Note: All "observed" accident data has been corrected for change in Daily Traffic, I.e. has been normalized to 'before' traffic.												
5	**upper Bound Accidents 'With' per Year**	**lower Bound Accidents 'Without' per Year**	**upper Bound Accidents 'Without' per Year**	**lower error 'without'**	**upper error 'without'**	**lower Bound Accidents Avoided per Year**	**upper Bound Accidents Avoided per Year**	**lower error**	**upper error**	**Project Motivation**					
6	3.264496	6.03	10.53	3.4273023	4.9915054	2.77	9.66	2.933906	3.964989	safety, performance, pub. Demand					
7	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
8	1.644263	0.36	2.31	0.8788953	2.6123387	-1.29	2.23	1.876619	1.636941	safety					
9	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
10	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
11	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
12	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
13	#DIV/0!	0.36	2.31	0.8788953	2.6123387	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	safety, performance, pub. Demand					
14	1.291636	3.45	7.12	2.6023069	4.1951695	2.16	7.11	1.930061	3.021442	safety, pub. Demand					
15	1.291636	4.73	8.84	3.0390297	4.6151976	3.44	8.83	1.744202	3.645172						
16	#DIV/0!	4.73	8.84	3.0390297	4.6151976	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!						
17	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						
18	#DIV/0!	0.69	2.95	1.1906645	2.8836361	#DIV/0!	2.95	#DIV/0!	2.951882						
19	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!						
20	#DIV/0!	0.00	0.92	0	1.8444448	#DIV/0!	0.92	#DIV/0!	0.91629						

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P
21	Count		Index	Number	Description	Location	First Appearance in 6 year plan (From 1990)	PE	RW	CN	TOT		Completion year		Construct Start	Construct End
22	C16	E	42	0029-023-v10, pe101, m501	29 - Culpeper Construct Right Turn Lane	@ Rt. 666	1994	1	0	14	15	27	1995		10/27/94	11/02/94
23	C17		47	0029-056-111, pe101, n501	29 - Madison Construct Right Turn Lane Southbound	Intersection Rt. 662 (2.4 mi S of Rt. 230)	1993	4	0	28	32	57.6			11/09/92	06/11/93
24	C18	F	48	0029-056-112, pe101, n501	29- Madison Construct Right Turn Lane Northbound	@ Madison Cty. High School (Rt. 9731), 0.3 mi S of Intersection Rt. 29 Bus./Rt. 231	1993	1	0	15	16	28.8			11/02/92	06/22/93
25	C19	G	53	0033-039-107, pe101, rw201, c501	33- Greene Improve Turning Radius	Stanardsville, Intersection Rt. 230	1990	20	80	25	125	225			04/27/92	06/25/92
26	C20	H	56	0033-054-106, pe101, rw201, n501	33- Louisa Install Right turn lane	Intersection Rt. 628	1990	40	100	160	300	540	1992		01/20/92	04/17/92
27	C21		57	0033-054-107, pe101, rw201, n501	33/208 Louisa Increase Turning Radius		1990	7	8	35	50	90	1991		01/20/92	04/17/92
28	C22		59	6211-078-105, pe104, rw204, c504, b604, d606	2.006 mi (EBL) grade, drain str., asp. conc. pave. from 0.470 mi E int. rt. 522 to 0.259 miles E. Covington River (Rappahannock, Develop 4 lanes)	0.3 mi E of Covington River, 0.5 mi E of Rt. 522	1990	450	480	5420	6350	####	1994		05/04/92	05/06/94
29	C23		16	0015-068-107, pe101, c501	15 - Orange Improve Vertical Clearance	Town of Gordonsville CSX RR Underpass	1994	50	0	130	180	324			04/95	07/05/95
30	C24		18	6017-030-108, pe101,rw201,c501	17- Fauquier 2 Lanes on New Location on 4 Lane Right of Way, 2.6 mi	Warrenton Bypass Extension: Rte. 29 Bypass North of Warrenton - Rte. 17	1990	850	1410	####	####	####			03/06/95	10/97
31	C25	I	24	0020-002-123, pe101, rw201, m501	20 - Albemarle Extend Acceleration Lane	@ Intersection I-64	1993	55	70	350	475	855	1997		11/11/96	7/97
32	C26		32	6029-002-f19, pe102, rw202, c502	29 - Albemarle widen to 6 Lanes w/Cont. RTL's Each direction, 1.3 mi	0.2 mi North Rio Rd. (Rte. 631) - South Fork Rivanna River	1993	591	1063	3916	5570	####	1996		06/02/95	08/01/97
33	C27		49	0029-056-114, pe101, n501	29 - Madison Construct Right Turn Lane	@ Rte. 230	1994	1	0	14	15	27			10/24/94	
34	C28	J	68	0631-002-185, C502	Route 631 from Route 29 to Route 1403 (Berkmar Dr.) Widen from 2-3 lanes to 5 lanes	length 0.25 mi	?				0	0			10/7/1996 ??	9/25/1996 ?? 8 months duration (Hores)
35	C29		69	6033-039-V05-C501	Widening of Route 33 from Route 29 west towards Quinque - Charlottesville Residency	length 2.5 mi					0	0			12/28/92	04/19/96

	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
21	Fatality	Injury	PDO	Persons Injured	$ Damage	Accident Rate	Accident Rate corrected	Accidents per Entering Vehicle	DVMT or DEV	Daily Traffic	Accidents/ Year	lower Bound	upper Bound	Fatality	Injury	PDO	Persons Injured	$ Damage	Accident Rate	Accident Rate corrected
22	0	9	5	21	120475	1.22	1.22	1.2	15694	15694	7	3.82696	11.7448	0	8	4	10	70950	1.19	1.19
23	0	3	2	7	42300	0.2	0.2	0.2	34113	34113	2.5	0.81174	5.83417	0	1	2	2	23150	0.11	0.11
24	0	1	1	2	7425	0.1	0.1	0.1	27198	27198	1	0.1211	3.61234	0	1	1	2	13525	0.09	0.09
25	0	1	0	3	11050	0.28	0.28	0.3	4950	4950	0.5	0.01266	2.78582	0	0	0	0	0	0	0
26	0	1	2	1	8550	n/a	0.34773446	0.3	11818.18	11818.18	1.5	0.30934	4.38364	0	0	2	0	3100	n/a	0.21
27	0	0	1	0	4500	0.09	0.09	0.1	15251	15251	0.5	0.01266	2.78582	0	1	1	1	4150	0.17	0.17
28	0	6	3	9	46225	104	1.04	2.1	11759	5879.5	4.5	2.05768	8.5424	0	1	1	1	5500	26	0.26
29	0	0	1	0	1500	46	0.46	0.3	2970	4714.286	0.5	0.01266	2.78582	XXX	XXX	XXX	XXX	XXX	XXX	
30	0	14	20	18	131250	n/a	n/a	#DIV/0!	n/a		17	11.773	23.7558	XXX	XXX	XXX	XXX	XXX	XXX	
31	0	5	9	7	46450	180	1.8	1.0	10600	20000	7	3.82696	11.7448	XXX	XXX	XXX	XXX	XXX	XXX	
32	0	43	61	67	413845	263	2.63	3.6	54080	40059.26	52	42.4878	63.0067	XXX	XXX	XXX	XXX	XXX	XXX	
33	0	2	1	2	16750	0.13	0.13	0.1	31650	31650	1.5	0.30934	4.38364	XXX	XXX	XXX	XXX	XXX	XXX	
34	0	21	24	34	161620	555	5.55	1.4	11091	44364	22.5	16.4116	30.1068	XXX	XXX	XXX	XXX	XXX	XXX	
35	1	22	20	33	163825	683	6.83	17.1	8618	3447.2	21.5	15.5597	28.9604	XXX	XXX	XXX	XXX	XXX	XXX	

	AK	AL	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC	BD	BE
21	DVMT or DEV	Length	Speed Without	Speed With	lower Bound Speed Without	upper Bound Speed Without	lower Bound Speed With	upper Bound Speed With	lower Bound Daily Traffic (Before)	upper Bound Daily Traffic (Before)	lower Bound Total Travel Time Without	upper Bound Total Travel Time Without	lower Bound Total Travel Time With	upper Bound Total Travel Time With	Difference of upper Bounds	Difference of lower Bounds	Average Travel Time Saved	Error Band	ARF (Fatality, Injury)	ARF (PDO)	Project Type (ARF)
22	13851																0	0	0.29	0.58	22
23	38770																		0.29	0.58	Right Turn Lane
24	30615																0	0	0.29	0.58	22
25	4950								3960	5940	79.2	178.2	52.8	118.8	59.4	26.4	43	17	0.87	0.73	8
26	13000								9454.545	14181.82	756.3636	1701.818	441.212	992.727	709.0909091	315.1515152	512	197	0.29	0.58	21
27	15950																		0.5	0.5	??
28	10521	2																	0.5	0.5	??
29	XXX	0.63																	0.5	0.5	??
30	XXX	1.14																	0.5	0.5	Widen Pavement (additional Lane)
31	XXX	0.53							16000	24000	170.6667	384	85.3333	192	192	85.33333333	139	53	0.5	0.5	2
32	XXX	1.35	38	48	36	40	46	50	32047.41	48071.11	6489.6	10816	5191.68	8464.7	2351.304348	1297.92	1825	527	0.976925	0.97165	2,9,19,24
33	XXX																		0.29	0.58	Right Turn Lane
34	XXX	0.25	28	32	26	30	30	34	35491.2	53236.8	1774.56	3071.354	1565.79	2661.84	409.5138462	208.7717647	309	100	0.5	0.5	2
35	XXX	2.5	52	62	50	54	60	64	2757.76	4136.64	766.0444	1240.992	646.35	1034.16	206.832	119.6944444	163	44	0.5	0.5	2

	BF	BG	BH	BI	BJ	BK	BL	BM	BN	BO	BP	BQ	BR	BS	BT	BU	BV	BW	BX	BZ
21	Accidents Saved/ Year Predicted (Fatality, Injury)	lower Bound	upper Bound	Accidents Saved/ Year Predicted (PDO)	lower Bound	upper Bound	lower Error Band	upper Error Band	Accidents Saved/ Year Predicted (Total)	Accidents Saved/ Year Observed	Cost	Count	Additional Info (Nodes)				Daily Traffic After / Daily Traffic Before	Time Saved +10 (to create offset in chart)	Acc./Saved Observed per Year	lower Bound Accidents 'With' per Year
22	1.305	0.596728	2.477295	1.45	0.4708	3.3838	1.6875	3.1061	2.755	1	15	C16	233223				0.882567	5	1.133058985	4.944144
23	0.435	0.089707	1.271254	0.58	0.0702	2.0952	0.8551	2.3514	1.015	1	32	C17	443190							#DIV/0!
24	0.145	0.003671	0.807886	0.29	0.0073	1.6158	0.424	1.9887	0.435	0	16	C18	443304				1.125634	5	0	0.209872
25	0.435	0.011013	2.423659	0	0	1.3464	0.424	3.3351	0.435	0.5	125	C19	358232				1	48	0.5	0
26	0.145	0.003671	0.807886	0.58	0.0702	2.0952	0.6511	2.178	0.725	0.5	300	C20	433142				1.1	517	0.454545455	0.209872
27	0	0	0.922222	0.25	0.0063	1.3929	0.2437	2.0651	0.25	-0.5	50	C21	433604							#DIV/0!
28	1.5	0.550472	3.264867	0.75	0.1547	2.1918	1.5449	3.2067	2.25	3.5	6350	C22	561177 to 561195 & 1867ft. E							#DIV/0!
29	0	0	0.922222	0.25	0.0063	1.3929	0.2437	2.0651	0.25		180	C23	Intersect. 231/15 @ 132.63 mi, checked from node 503328 (132.27mi) to 503336 (132.00mi)							0
30	3.5	1.913482	5.872402	5	3.0541	7.7221	3.5324	5.0945	8.5		14940	C24	checked accidents on BUS29, north of Warenton, between Rte. 17 and 15/29, nodes 314131 (3.51mi) to 314099 (4.65mi)							0
31	1.25	0.40587	2.917083	2.25	1.0288	4.2712	2.0653	3.6883	3.5		475	C25	checked from node 111295 (37.06mi) (one node off of ramp) to 111286 (37.59mi) (one node off ramp on other side)							0
32	21.0038875	15.20062	28.29211	29.635325	22.669	38.068	12.77	15.721	50.6392125		5570	C26	719777 + 263ft N to 110194 + 525ft S (1.35 mi length)							0
33	0.29	0.03512	1.047578	0.29	0.0073	1.6158	0.5375	2.0834	0.58		15	C27	869004							0
34	5.25	3.249832	8.025177	6	3.8443	8.9275	4.1559	5.7027	11.25		0	C28								0
35	5.75	3.645003	8.627821	5	3.0541	7.7221	4.0509	5.5999	10.75		0	C29	358218 & 1470 ft. W to 358229							0

	CA	CB	CC	CD	CE	CF	CG	CH	CI	CJ	CK	CL	CM	CN	CO
21	upper Bound Accidents 'With' per Year	lower Bound Accidents 'Without' per Year	upper Bound Accidents 'Without' per Year	lower error 'without'	upper error 'without'	lower Bound Accidents Avoided per Year	upper Bound Accidents Avoided per Year	lower error	upper error	Project Motivation					
22	9.119974	5.16	9.40	3.1730364	4.7448044	-3.96	4.46	5.092834	3.325496	safety, performance					
23	#DIV/0!	1.42	4.18	1.6882591	3.334165	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!						
24	1.983102	0.36	2.31	0.8788953	2.6123387	-1.63	2.10	1.627084	2.10262	safety, performance					
25	0.91629	0.09	1.64	0.4873411	2.2858155	-0.83	1.64	1.329114	1.144263	safety, performance					
26	1.983102	0.69	2.95	1.1906645	2.8836361	-1.30	2.74	1.751181	2.287465	performance, pub. Demand					
27	#DIV/0!	0.09	1.64	0.4873411	2.2858155	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!						
28	#DIV/0!	3.03	6.54	2.4423157	4.0423954	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!						
29	#DIV/0!	0.09	1.64	0.4873411	2.2858155	#DIV/0!	1.64	#DIV/0!	1.644263						
30	#DIV/0!	14.11	20.43	5.2270143	6.7557871	#DIV/0!	20.43	#DIV/0!	20.42587						
31	#DIV/0!	5.16	9.40	3.1730364	4.7448044	#DIV/0!	9.40	#DIV/0!	9.402699						
32	#DIV/0!	46.94	57.59	9.5122488	11.006678	#DIV/0!	57.59	#DIV/0!	57.58517	safety, performance, pub. Demand, econ. Developm., aesthetics					
33	#DIV/0!	0.69	2.95	1.1906645	2.8836361	#DIV/0!	2.95	#DIV/0!	2.951882						
34	#DIV/0!	19.18	26.36	6.0883519	7.6067585	#DIV/0!	26.36	#DIV/0!	26.35836	safety, performance, pub. Demand, econ. Developm.					
35	#DIV/0!	18.25	25.28	5.9403418	7.4603682	#DIV/0!	25.28	#DIV/0!	25.28396						

	A	B	C	D	E	F	G	H	I	J	K	L	M
1	**Critical Rate Listing**												
2	Label	DVMT	Length (miles)	Daily Traffic (vehicles)	no. of crashes per year	accident rate	crashes per million vehicle-miles	lower bound crashes per year	upper bound crashes per year	lower error crashes per year	upper error crashes per year	lower bound of crashes per million vehicle-miles	upper bound of crashes per million vehicle miles
3	E1	1014	0.39	2600	3	810	8.1	0.62	8.77	2.38	5.77	1.67	23.69
4	E2	810	0.3	2700	4	1352	13.52	1.09	10.24	2.91	6.24	3.69	34.64
5	E3	1680	0.3	5600	7	1141	11.41	2.81	14.42	4.19	7.42	4.59	23.52
6	E4	2598	0.46	5648	9	949	9.49	4.12	17.08	4.88	8.08	4.34	18.02
7	E5	3969	0.81	4900	13	897	8.97	6.92	22.23	6.08	9.23	4.78	15.35
8	E6	1680	0.3	5600	5	815	8.15	1.62	11.67	3.38	6.67	2.65	19.03
9	E7	1073	0.37	2900	3	766	7.66	0.62	8.77	2.38	5.77	1.58	22.39
10	E8	2300	0.4	5750	6	714	7.14	2.20	13.06	3.80	7.06	2.62	15.56
11	E9	1680	0.3	5600	4	652	6.52	1.09	10.24	2.91	6.24	1.78	16.70
12	E10	4290	0.33	13000	9	574	5.74	4.12	17.08	4.88	8.08	2.63	10.91
13	E11	1500	0.3	5000	3	547	5.47	0.62	8.77	2.38	5.77	1.13	16.01
14	E12	1512	0.36	4200	3	543	5.43	0.62	8.77	2.38	5.77	1.12	15.89
15	E13	3360	0.6	5600	6	489	4.89	2.20	13.06	3.80	7.06	1.80	10.65
16	E14	2964	0.52	5700	5	462	4.62	1.62	11.67	3.38	6.67	1.50	10.79
17	E15	2732	0.48	5692	4	401	4.01	1.09	10.24	2.91	6.24	1.09	10.27
18	E16	3010	0.35	8600	3	273	2.73	0.62	8.77	2.38	5.77	0.56	7.98
19	E17	6370	0.49	13000	5	215	2.15	1.62	11.67	3.38	6.67	0.70	5.02
20	E18	1064	0.3	3547	4	1029	10.29	1.09	10.24	2.91	6.24	2.81	26.37
21	E19	1020	0.3	3400	3	805	8.05	0.62	8.77	2.38	5.77	1.66	23.55
22	E20	1054	0.31	3400	3	779	7.79	0.62	8.77	2.38	5.77	1.61	22.79
23	E21	2040	0.34	6000	5	671	6.71	1.62	11.67	3.38	6.67	2.18	15.67
24	E22	1290	0.3	4300	3	637	6.37	0.62	8.77	2.38	5.77	1.31	18.62
25	E23	1915	0.34	5632	3	429	4.29	0.62	8.77	2.38	5.77	0.89	12.54
26	E24	5200	0.4	13000	6	316	3.16	2.20	13.06	3.80	7.06	1.16	6.88
27	E25	152080	3.64	41780	205	369	3.69	177.90	235.07	27.10	30.07	3.20	4.23
28	E26	22530	0.75	30040	22	267	2.67	13.79	33.31	8.21	11.31	1.68	4.05
29	E28	3140	0.31	10129	5	436	4.36	1.62	11.67	3.38	6.67	1.42	10.18
30	E29	9660	0.69	14000	14	397	3.97	7.65	23.49	6.35	9.49	2.17	6.66
31	E30	8330	0.59	14119	11	361	3.61	5.49	19.68	5.51	8.68	1.81	6.47
32	E31	3420	0.36	9500	4	320	3.2	1.09	10.24	2.91	6.24	0.87	8.20
33	E32	5040	0.6	8400	5	271	2.71	1.62	11.67	3.38	6.67	0.88	6.34
34	E33	40500	1.12	36161	38	257	2.57	26.89	52.16	11.11	14.16	1.82	3.53
35	E34	10140	0.39	26000	9	243	2.43	4.12	17.08	4.88	8.08	1.11	4.62
36	E35	7000	0.5	14000	6	234	2.34	2.20	13.06	3.80	7.06	0.86	5.11
37	E36	5040	0.36	14000	4	217	2.17	1.09	10.24	2.91	6.24	0.59	5.57
38	E37	4200	0.3	14000	3	195	1.95	0.62	8.77	2.38	5.77	0.40	5.72
39	E38	22680	0.55	41236	16	193	1.93	9.15	25.98	6.85	9.98	1.10	3.14
40	E39	14000	1	14000	8	156	1.56	3.45	15.76	4.55	7.76	0.68	3.08
41	E40	11298	0.31	36445	6	145	1.45	2.20	13.06	3.80	7.06	0.53	3.17
42	E41	12240	0.34	36000	6	134	1.34	2.20	13.06	3.80	7.06	0.49	2.92
43	E42	26280	0.73	36000	11	114	1.14	5.49	19.68	5.51	8.68	0.57	2.05
44	E44	9240	0.42	22000	10	296	2.96	4.80	18.39	5.20	8.39	1.42	5.45
45	E47	4680	0.39	12000	3	175	1.75	0.62	8.77	2.38	5.77	0.36	5.13
46	E49	435	0.3	1450	4	2519	25.19	1.09	10.24	2.91	6.24	6.86	64.50
47	E50	23127	1.06	21818	66	781	7.81	51.04	83.97	14.96	6.24	6.05	9.95
48	E51	1101	0.34	3238	3	746	7.46	0.62	8.77	2.38	17.97	1.54	21.82
49	E52	17599	1.14	15438	42	653	6.53	30.27	56.77	11.73	5.77	4.71	8.84
50	E53	2058	0.3	6860	4	532	5.32	1.09	10.24	2.91	14.77	1.45	13.63
51	E54	7046	0.32	22019	13	505	5.05	6.92	22.23	6.08	6.24	2.69	8.64
52	E55	3801	0.45	8447	7	504	5.04	2.81	14.42	4.19	9.23	2.03	10.40
53	E56	4316	0.52	8300	7	444	4.44	2.81	14.42	4.19	7.42	1.79	9.16
54	E57	16255	0.97	16758	22	370	3.7	13.79	33.31	8.21	7.42	2.32	5.61
55	E58	6099	0.35	17426	8	359	3.59	3.45	15.76	4.55	11.31	1.55	7.08
56	E59	8163	0.39	20931	7	234	2.34	2.81	14.42	4.19	7.76	0.94	4.84
57	E60	13918	0.63	22092	7	137	1.37	2.81	14.42	4.19	7.42	0.55	2.84
58	E61	543	0.48	1131	5	2522	25.22	1.62	11.67	3.38	7.42	8.19	58.87

	A	B	C	D	E	F	G	H	I	J	K	L	M
59	**Critical Rate Listing**												
60	Label	DVMT	Length: (miles)	Daily Traffic (vehicles)	no. of crashes per year	accident rate	crashes per million vehicle-miles	lower bound crashes per year	upper bound crashes per year	lower error crashes per year	upper error crashes per year	lower bound of crashes per million vehicle-miles	upper bound of crashes per million vehicle miles
61	D1	3895	0.52	7490	12	844	8.44	6.20	20.96	5.80	6.67	4.36	14.74
62	D2	1441	0.4	3603	3	570	5.7	0.62	8.77	2.38	8.96	1.18	16.67
63	D3	2508	0.44	5700	13	1420	14.2	6.92	22.23	6.08	5.77	7.56	24.28
64	D4	890	0.36	2472	4	1231	12.31	1.09	10.24	2.91	9.23	3.35	31.53
65	D5	1741	0.6	2902	6	944	9.44	2.20	13.06	3.80	6.24	3.47	20.55
66	D6	3565	0.81	4401	12	922	9.22	6.20	20.96	5.80	7.06	4.77	16.11
67	D7	1819	0.32	5684	5	753	7.53	1.62	11.67	3.38	8.96	2.45	17.57
68	D8	2002	0.4	5005	5	684	6.84	1.62	11.67	3.38	6.67	2.22	15.97
69	D9	2964	0.52	5700	7	647	6.47	2.81	14.42	4.19	6.67	2.60	13.33
70	D10	1280	0.4	3200	3	642	6.42	0.62	8.77	2.38	7.42	1.32	18.77
71	D11	1712	0.3	5707	4	640	6.4	1.09	10.24	2.91	5.77	1.74	16.39
72	D12	1712	0.3	5707	4	640	6.4	1.09	10.24	2.91	6.24	1.74	16.39
73	D13	5648	0.99	5705	12	582	5.82	6.20	20.96	5.80	6.24	3.01	10.17
74	D14	3005	0.3	10017	6	547	5.47	2.20	13.06	3.80	8.96	2.01	11.91
75	D15	6274	1.1	5704	12	524	5.24	6.20	20.96	5.80	7.06	2.71	9.15
76	D16	7239	1.27	5700	13	492	4.92	6.92	22.23	6.08	8.96	2.62	8.41
77	D17	1712	0.3	5707	3	480	4.8	0.62	8.77	2.38	9.23	0.99	14.03
78	D18	3535	0.62	5702	6	465	4.65	2.20	13.06	3.80	5.77	1.71	10.12
79	D19	2356	0.49	4808	4	465	4.65	1.09	10.24	2.91	7.06	1.27	11.91
80	D20	4859	0.86	5650	8	451	4.51	3.45	15.76	4.55	6.24	1.95	8.89
81	D21	2525	0.3	8417	4	434	4.34	1.09	10.24	2.91	7.76	1.18	11.11
82	D22	2914	0.51	5714	4	376	3.76	1.09	10.24	2.91	6.24	1.02	9.63
83	D23	4101	0.32	12816	5	334	3.34	1.62	11.67	3.38	6.24	1.08	7.80
84	D24	1020	0.3	3400	5	1343	13.43	1.62	11.67	3.38	6.67	4.36	31.34
85	D25	1710	0.3	5700	8	1281	12.81	3.45	15.76	4.55	6.67	5.53	25.26
86	D26	1280	0.4	3200	4	856	8.56	1.09	10.24	2.91	7.76	2.33	21.92
87	D27	1292	0.38	3400	4	848	8.48	1.09	10.24	2.91	6.24	2.31	21.72
88	D28	1361	0.4	3403	4	805	8.05	1.09	10.24	2.91	6.24	2.19	20.62
89	D29	1712	0.3	5707	4	640	6.4	1.09	10.24	2.91	6.24	1.74	16.39
90	D30	2111	0.37	5705	4	519	5.19	1.09	10.24	2.91	6.24	1.41	13.29
91	D31	8407	0.3	28023	17	554	5.54	9.90	27.22	7.10	6.24	3.23	8.87
92	D32	85479	1.68	50880	117	375	3.75	96.76	140.22	20.24	10.22	3.10	4.49
93	D33	10072	0.4	25180	5	136	1.36	1.62	11.67	3.38	23.22	0.44	3.17
94	D34	34555	1.05	32910	14	111	1.11	7.65	23.49	6.35	6.67	0.61	1.86
95	D35	4429	0.34	13026	7	433	4.33	2.81	14.42	4.19	9.49	1.74	8.92
96	D36	3902	0.3	13007	5	351	3.51	1.62	11.67	3.38	7.42	1.14	8.19
97	D37	2490	0.3	8300	3	330	3.3	0.62	8.77	2.38	6.67	0.68	9.65
98	D38	9753	0.39	25008	11	309	3.09	5.49	19.68	5.51	5.77	1.54	5.53
99	D39	6246	0.48	13013	7	307	3.07	2.81	14.42	4.19	8.68	1.23	6.33
100	D40	13272	1.02	13012	14	289	2.89	7.65	23.49	6.35	7.42	1.58	4.85
101	D41	35914	1.02	35210	35	267	2.67	24.38	48.68	10.62	9.49	1.86	3.71
102	D42	17945	0.69	26007	15	229	2.29	8.40	24.74	6.60	13.68	1.28	3.78
103	D43	24032	1.15	20897	20	228	2.28	12.22	30.89	7.78	9.74	1.39	3.52
104	D44	7172	0.65	11034	5	191	1.91	1.62	11.67	3.38	10.89	0.62	4.46
105	D45	12958	0.37	35022	7	148	1.48	2.81	14.42	4.19	6.67	0.60	3.05
106	D46	39770	1.13	35195	18	124	1.24	10.67	28.45	7.33	7.42	0.73	1.96
107	D47	13046	0.37	35259	5	105	1.05	1.62	11.67	3.38	10.45	0.34	2.45
108	D48	17562	0.5	35124	5	78	0.78	1.62	11.67	3.38	6.67	0.25	1.82
109	D49	3903	0.3	13010	11	772	7.72	5.49	19.68	5.51	6.67	3.85	13.82
110	D50	1712	0.3	5707	3	480	4.8	0.62	8.77	2.38	8.68	0.99	14.03
111	D51	7172	0.51	14063	5	191	1.91	1.62	11.67	3.38	5.77	0.62	4.46
112	D52	3604	0.3	12013	9	684	6.84	4.12	17.08	4.88	6.67	3.13	12.99
113	D53	6011	0.5	12022	7	319	3.19	2.81	14.42	4.19	8.08	1.28	6.57
114	D54	5218	0.4	13045	4	210	2.1	1.09	10.24	2.91	7.42	0.57	5.38
115	D55	5073	0.39	13008	3	162	1.62	0.62	8.77	2.38	6.24	0.33	4.73
116	D56	662	0.3	2207	3	1241	12.41	0.62	8.77	2.38	5.77	2.56	36.28
117	D57	18663	1.03	18119	53	778	7.78	39.70	69.33	13.30	5.77	5.83	10.18
118	D58	10590	0.67	15806	23	595	5.95	14.58	34.51	8.42	16.33	3.77	8.93
119	D59	2842	0.38	7479	5	482	4.82	1.62	11.67	3.38	11.51	1.57	11.25
120	D60	3727	0.5	7454	6	441	4.41	2.20	13.06	3.80	6.67	1.62	9.60
121	D61	8861	0.78	11360	12	371	3.71	6.20	20.96	5.80	7.06	1.92	6.48
122	D62	10506	0.54	19456	13	339	3.39	6.92	22.23	6.08	8.96	1.81	5.80
123	D63	586	0.5	1172	5	2337	23.37	1.62	11.67	3.38	9.23	7.59	54.55
124	D64	807	0.36	2242	4	1357	13.57	1.09	10.24	2.91	6.67	3.70	34.77
125	D65	358	0.3	1193	3	2295	22.95	0.62	8.77	2.38	6.24	4.73	67.09
126	D66	73	0.3	243	3	11259	112.59	0.62	8.77	2.38	5.77	23.22	329.04