OPTIMUM TURNOUT SPACING ON FOREST HAUL ROADS

by

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ABSTRACT

Mathematical models are developed to determine the optimum spacing of turnouts and to predict the time lost due to the acceleration and deceleration of the vehicle and the time the vehicle spends in the turnout. Previous articles have not completely defined a method of deriving or measuring the delay in the turnout attributable to turnout spacing. The concept of the expected F-factor, a measurement of the expected delay in the turnout, is introduced. The expected F-factor is the expected distance the loaded vehicle is from the empty vehicle, once the empty vehicle has come to a complete halt in the turnout, divided by the turnout spacing. Two forms of the expected F-factor equation were developed.

The results show that the total expected delay time attributable to turnout spacing may be a significant part of the travel empty time (i.e., 20 percent) but its significance is reduced when compared to the round trip time. The optimum turnout spacing model is concerned with minimizing the sum of the turnout construction and maintenance costs and the cost of delays attributable to turnout spacing. If the results of the optimum turnout spacing model are used in the initial design of the road network then the total potential savings can be important. Implementation of the optimum turnout spacing model can be achieved with the utilization of tables.
These tables can be utilized as a guide in the design and construction of forest haul roads.
# TABLE OF CONTENTS

| LIST OF TABLES | vii |
| LIST OF FIGURES | viii |
| ACKNOWLEDGEMENT | x |
| 1.0 INTRODUCTION | 1 |
| 2.0 DEVELOPMENT OF THE OPTIMUM SPACING MODEL | 5 |
| 2.1 FORMATION OF THE PROBLEM | 5 |
| 2.2 THE ASSUMPTIONS OF THE MODEL | 7 |
| 2.3 THE TURNOUT DELAY TIME | 10 |
| 2.4 THEORETICAL DEVELOPMENT OF THE EXPECTED F-FACTOR | 14 |
| 2.4.1 ONE LOADED VEHICLE MEETING ONE EMPTY VEHICLE | 16 |
| 2.4.2 ONE EMPTY VEHICLE MEETING A FLEET OF LOADED VEHICLES | 19 |
| 2.4.3 A FLEET OF LOADED VEHICLES MEETING A FLEET OF EMPTY VEHICLES | 24 |
| 2.4.4 HEADWAY PROBABILITY DISTRIBUTIONS | 30 |
| 2.4.4.1 UNIFORM ARRIVAL DISTRIBUTION | 32 |
| 2.4.4.2 EXponential HEADWAY DISTRIBUTION | 33 |
| 2.4.4.3 ERLANG HEADWAY DISTRIBUTION | 34 |
| 2.4.4.4 PEARSON TYPE III HEADWAY DISTRIBUTION | 35 |
| 2.5 DEVELOPMENT OF THE COST EQUATION | 37 |
| 2.6 DISCOUNTING OF THE COST EQUATION | 40 |
| 3.0 THE ARRIVAL DISTRIBUTION OF LOGGING TRUCKS | 42 |
| 3.1 ARRIVAL DATA FROM TWO OPERATIONS | 42 |
| 3.2 ANALYSIS OF THE ARRIVAL DATA | 44 |
LITERATURE CITED .................................................. 122
APPENDICES ............................................................. 124
1 ROAD STANDARDS SURVEY ........................................... 125
2 ABBREVIATIONS, SYMBOLS, AND UNITS ............................ 128
3 ACCELERATION AND DECELERATION OF A VEHICLE ................. 131
4 ANALYSIS OF HEADWAY DISTRIBUTIONS .............................. 138
5 SIMULATION OF THE F-FACTOR ....................................... 151
6 THE LENGTH OF THE VEHICLE ....................................... 175
7 DERIVATIVES OF THE EXPECTED F-FACTOR EQUATIONS ............. 182
8 OPTIMUM TURNOUT SPACING COMPUTER PROGRAM .................. 196
9 GRAPHICAL RESULTS OF THE SENSITIVITY ANALYSIS ............ 198
10 CONVERSION FACTORS ............................................... 222
## LIST OF TABLES

<table>
<thead>
<tr>
<th>TABLE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Goodness Of Fit Tests—Coastal Study</td>
</tr>
<tr>
<td>II</td>
<td>Goodness Of Fit Tests—Interior Study—Both Scales</td>
</tr>
<tr>
<td>III</td>
<td>F-factor Simulation — The Interaction Between A Single Loaded Vehicle And A Single Empty Vehicle</td>
</tr>
<tr>
<td>IV</td>
<td>F-factor Simulation — An Empty Vehicle Meeting Loaded Vehicles Based On Equations 2.18, 2.19, 2.30, And 2.31</td>
</tr>
<tr>
<td>V</td>
<td>F-factor Simulation — An Empty Vehicle Meeting Loaded Vehicles Based On Equations 2.18, 2.19, 2.33, And 2.34.</td>
</tr>
<tr>
<td>VI</td>
<td>F-factor Simulation — Interaction Between Two Fleets Of Vehicles Based On Equations 2.18, 2.19, 2.30, And 2.31</td>
</tr>
<tr>
<td>VII</td>
<td>Truck Hauling Costs</td>
</tr>
<tr>
<td>VIII</td>
<td>Excavation Costs Per Turnout</td>
</tr>
<tr>
<td>IX</td>
<td>Turnout Maintenance Costs</td>
</tr>
</tbody>
</table>
# LIST OF FIGURES

<table>
<thead>
<tr>
<th>FIGURE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Interaction Between An Empty Vehicle And A Loaded Vehicle</td>
<td>17</td>
</tr>
<tr>
<td>2 Diagram Showing The Physical Situation Leading To The Critical Headway</td>
<td>20</td>
</tr>
<tr>
<td>3 Case 3—An Empty Vehicle Stops At Turnout S₃ Since Next Turnout Is Occupied</td>
<td>25</td>
</tr>
<tr>
<td>4 Case 4—An Empty Vehicle Remains In Turnout S₃ Since Next Turnout Is Occupied</td>
<td>29</td>
</tr>
<tr>
<td>5 Frequency Histogram Of Headways For Coastal Study (5-minute Intervals)</td>
<td>46</td>
</tr>
<tr>
<td>6 Frequency Histogram Of Headways For Interior Study (1-minute Intervals)</td>
<td>48</td>
</tr>
<tr>
<td>7 Frequency Histogram Of Headways For Interior Study (5-minute Intervals)</td>
<td>49</td>
</tr>
<tr>
<td>8 Effect Of Traffic Flow Rate On The Total Expected Delay Time For Various Turnout Spacings</td>
<td>69</td>
</tr>
<tr>
<td>9 Effect Of The Turnout Spacing On The Total Expected Delay Time For Various Traffic Flow Rates</td>
<td>70</td>
</tr>
<tr>
<td>10 Effect Of The Velocity Of The Empty Vehicle On The Total Expected Delay Time For Various Velocities Of The Loaded Vehicle</td>
<td>71</td>
</tr>
<tr>
<td>11 Effect Of The Velocity Of The Loaded Vehicle On The Total Expected Delay Time For Various Velocities Of The</td>
<td></td>
</tr>
</tbody>
</table>
12 Effect Of The Velocity Of The Loaded Vehicle On The Optimum Turnout Spacing For Various Velocities Of The Empty Vehicle .............................................. 78
13 Effect Of The Velocity Of The Empty Vehicle On The Optimum Turnout Spacing For Various Velocities Of The Loaded Vehicle ............................................. 79
14 Effect Of The Traffic Flow Rate On The Optimum Turnout Spacing .................................................. 80
15 Effect Of The Turnout Construction Cost On The Optimum Turnout Spacing ............................................... 81
16 Effect Of The Adjusted Truck Hauling Cost On The Optimum Turnout Spacing ............................................... 82
17 Effect Of The Expected Useful Life Of The Road On The Optimum Turnout Spacing ............................................... 83
18 Effect Of The Turnout Spacing On The Cost Function Based On Equation 2.38 .................................................. 84
19 Effect Of Perturbations To The Velocity Of The Loaded Vehicle On The Total Expected Delay Time ...................... 90
20 Effect Of Perturbations To The Velocity Of The Loaded Vehicle On The Optimum Turnout Spacing ...................... 91
21 Effect Of Perturbations To The Velocity Of The Loaded Vehicle On The Cost Difference ................................. 92
22 Effect Of Deviations From The Optimum Turnout Spacing On The Maximum Cost Difference .......................... 113
23 Effect Of Deviations From The Optimum Turnout Spacing On The Average Cost Difference ............................ 114
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1.0 INTRODUCTION

Logging haul roads are an integral part of the harvesting system. Since hauling costs account for approximately 10 to 20 percent of the total logging cost, the design of the road networks should be analysed before and during the harvesting process. The hauling cost components are interdependent. As a company invests more capital into road maintenance and construction the log hauling costs should decrease. Consequently, there is a trade off between the initial investment in the road and the resulting hauling cost. This trade off is reflected in the road standards that are used for the various sections of the road network.

Forest companies and government agencies include various features and service conditions of the road in their assembling of the components of road standards. The road standards reflect the required physical and service characteristics of the haul roads. Some of the components that can be considered for inclusion in a specific road standard are: design speed, maximum adverse and favourable gradients, subgrade width, frequency of turnouts, and right-of-way width. The decision as to which components should be included in the road standards can be complex and there are no definite criteria for solving this problem. The potential design elements must be evaluated to determine which should be included in the road standards. Environmental conditions and user factors should be considered.
User factors include use of the road for future timber development, utilization of the road by others (e.g., mining and agriculture), and government regulations.

Single-lane roads or double-lane roads can be utilized in the extraction of timber. Single-lane roads are less expensive to construct but the cost of truck delays in turnouts may be greater than the added investment of constructing and maintaining a double-lane road. Furthermore, the potential increase in the speed of the vehicles on a double-lane road must be considered. Since road construction and maintenance costs increase as the turnout spacing is decreased, there is a theoretical optimum range of turnout spacing. It may be desirable to determine this optimum range prior to deciding on whether to build a single-lane or double-lane road.

There have been few publications concerned with the topic of optimum turnout spacing on forest haul roads. Il'in (1965) determined the critical traffic flow at which the number of lanes should be switched from one to two but he did not calculate or derive an expression to determine the expected turnout delay time. The United States Forest Service developed a manual to predict the cost of truck and trailer transportation (Byrne et al., 1947). The system involved a very simple method to estimate the lost time in minutes per mile. The method assumed there is a delay in the turnout equivalent to the time required for a loaded vehicle to travel one-half turnout spacing but it failed to justify this assumption. Porpaczy and Waelti (1976) produced an article with similar objectives as Il'in. This paper, as is the case for the previous articles, does not have a well defined method of
deriving or estimating turnout delay times.

Since Il'in (1965) and Porpaczy and Waelti (1976) have written papers concerned with determining the number of lanes required for log transportation, this study will concentrate on the problem of deciding on the spacing of turnouts on a single-lane road. The major objective of the study is to develop a method to determine the optimum turnout spacing as a function of traffic flow rate, speeds of the empty and loaded vehicles, "expected useful life of the road", turnout costs, and trucking costs. A second objective is to analyse the sensitivity of the solution and test whether turnout spacing has any economic significance with respect to the road design problem.

During the fall of 1976, the author conducted a mail and telephone survey of road standards (Appendix 1). Results of this survey revealed that there was extensive variation in the length of turnouts (30 to 150 feet) and the spacing of turnouts (2 to 11 per mile). This reinforces the need to investigate the spacing of turnouts.

The total time a logging truck spends in turnouts can be measured, but it is difficult to determine the portion of this time that is directly attributable to turnout spacing. For a number of reasons a logging truck may wait longer in a given turnout than is necessary. For example, if the logging truck operator expects there will be delays further along in the road network he may decide to wait longer in a turnout than is required. This paper develops a method to calculate the delay time specifically attributable to turnout spacing.

Once the total delay time attributable to turnout spacing has been determined, the cost of the delays must be evaluated.
The relevant costs are those that are functions of the turnout spacing: the turnout construction cost, the turnout maintenance cost, and the delay cost attributable to turnout spacing. Optimum turnout spacing can then be derived from the cost function. Simulation models are utilized to verify and test the sensitivity of the delay time, costing, and optimum turnout spacing models.

Throughout the text Imperial units are utilized. A table to convert Imperial units into the International System of Units is located in Appendix 10.
2.0 DEVELOPMENT OF THE OPTIMUM SPACING MODEL

2.1 FORMATION OF THE PROBLEM

The determination of the optimum number of turnouts per unit distance of road is concerned with minimizing the sum of turnout construction and maintenance costs and the cost of delays in turnouts. The construction and maintenance costs can be predicted from existing company records or manuals but the delay cost is not so easily obtained. The delay cost could be viewed as the truck hauling cost (dollars per unit time period) multiplied by the estimated time the vehicles must spend in the turnouts. This estimated time could be determined by time studies, simulation, or mathematical formulas. If the actual time spent in the turnouts was utilized in the calculation of the delay cost an over-estimation of the delay cost would result. This occurs since some of the time spent in the turnout includes delays not attributable to turnout spacing. If a driver realizes his vehicle must later queue at the landing or at the dump, then he may drive slower and wait longer in turnouts than if he did not expect a delay at these locations. Furthermore, the total delay time would include delay situations that would not influence the round trip time, since the empty vehicle would experience a delay at the landing regardless of the delay in the turnout. Consequently, this approach to determining delay costs would have to eliminate the delays not attributable to turnout spacing.

Another approach is to view delay costs in terms of the
number of round trips a vehicle can complete in a day. The initial step is to determine the number of complete round trips a given vehicle can achieve for a particular turnout spacing. The initial arrangement can be determined from previous experience or from an estimate of a good turnout spacing arrangement. A simulation, based on dispatching rules, traffic behaviour, loading times, and unloading times, can be utilized to determine the number of complete round trips per day. From this starting arrangement the turnout spacing can be increased or decreased to determine whether the variation will alter the number of complete round trips. This process can be repeated for a reasonable range of turnout spacings. A comparison of the total transportation cost per cunit is then required to determine the best turnout spacing arrangement. This is an adequate method provided one can accurately forecast the truck dispatching schedule. Since in reality this is not the case, the number of complete round trips per vehicle per day cannot be consistently determined.

The better of the two approaches appears to be the first method provided the delays in the system not attributable to turnout spacing are eliminated. Since a vehicle may have to queue at the landing regardless of the turnout spacing an adjusted hauling cost is utilized. The approach to this study will be to let the delay cost be equal to the adjusted hauling cost multiplied by the turnout spacing delay times that are attributable to turnout spacing.
2.2 THE ASSUMPTIONS OF THE MODEL

The nature of the log hauling process dictates that right-of-way priority belongs to the logging trucks. Therefore the only vehicles that will be considered in the delay process are the logging trucks themselves and it will therefore be assumed that the other vehicles will not hinder the progression of the logging trucks. Furthermore, delays at the landing and dump will not be considered as they are not solely related to turnout spacing but depend on the loading rate, the unloading rate, the dispatching procedure, the total number of vehicles, and the number of loaders. Depending on the starting up and shutting down mode of the company, there may be no turnout delays during the first few hours and last few hours of the "hauling" day. The various dispatching policies can be reflected in the model by limiting the number of hauling hours per day to the "conflict" hours. The "conflict" hours refer to that portion of the day where there is movement of both loaded and empty vehicles over a section of road. It shall be assumed that the empty vehicle will always stop at the turnout which will yield the least delay.

A constant turnout construction cost is assumed. Variation in turnout construction costs could be partially accounted for by dividing the road section to be analysed into subsections, each with a particular turnout construction cost. The discounting of costs is included in the model but amortization of road costs will not be considered. A fixed traffic flow rate will be assumed.
Further limitations will be added to the model by constraining some of the speed characteristics of the vehicles. The following assumptions shall be included:

1. All of the loaded vehicles travel at the same constant velocity.
2. All of the empty vehicles travel at the same constant velocity except when decelerating into a turnout.
3. The spacing of turnouts will be sufficient to always allow the empty vehicle to accelerate to its designated speed before having to decelerate into a turnout.
4. The empty vehicle will accelerate or decelerate at a constant rate.

The model will be developed in four parts:

1. determination of the total expected delay time per vehicle per unit distance of road
2. determination of the expected delay time in the turnout
3. determination of the cost equations
4. determination of the solutions to the cost equations.

The total expected delay time per vehicle per unit distance equation will consist of the delay caused by the acceleration and deceleration of the empty vehicle entering and leaving turnouts, plus an expression for the expected delay in the turnout. The model to describe the delay time in the turnout is developed in the following section. The general cost equation is:

\[
\text{Cost} = \text{turnout construction cost} + \text{delay cost attributable to turnout spacing} + \text{turnout maintenance cost}.
\]

The optimum turnout spacing is determined by taking the first
derivative of the cost function with respect to the turnout spacing and utilizing a search technique to find the values for which this derivative is equal to zero. Furthermore, these values must be checked to determine whether they calculate a maximum or minimum value of the cost function.
2.3 **THE TURNOUT DELAY TIME**

In the previous section the general formulation and basic assumptions of the model were outlined. The first phase is to determine the delay time per vehicle per unit distance of road. The time required for a vehicle, travelling at a constant speed, to traverse a unit distance of road is the inverse of the velocity of the vehicle. The empty vehicle will experience an expected "n" turnout delays while progressing along this unit length of road. The total time required to travel this section is the uninterrupted travel time plus the time spent in the turnouts plus the time lost in the acceleration and deceleration of the empty vehicle.

From the basic theory of the rectilinear motion of a particle (Meriam, 1971) it can be easily shown that the stopping distance required for an empty vehicle, travelling at velocity $v$, is:

$$D_s = \frac{v^2}{2a}$$

and its corresponding stopping time is:

$$T_s = \frac{v}{a}$$

A listing of the symbols utilized in the formulas is located in Appendix 2. Similarly, the acceleration distance ($D_a$) and acceleration time ($T_a$) are:

$$D_a = \frac{v^2}{2a_a}$$

and

$$T_a = \frac{v}{a_a}$$

The next step is to derive an expression for the delay time in the turnout. This delay time is not a constant but
depends on the distance the loaded vehicle is from the empty vehicle when the empty vehicle comes to a complete stop in the turnout. The delay in the turnout can be represented by the ratio of the distance between the two vehicles and the turnout spacing. This ratio will be referred to as the F-factor. Thus:

\[ F = \frac{D}{S} \]

where:
- \( F \) = F-factor
- \( D \) = distance between the two vehicles
- \( S \) = distance between turnouts.

The turnout spacing is assumed to be uniform. The F-factor is utilized rather than distance or time since it gives a better concept of the effect that turnout spacing has on delays. Furthermore, previous articles (Byrne et al., 1947) on the subject have utilized the same ratio to represent this delay. The theoretical development of an equation for calculating the expected F-factor will be discussed in the next section.

Utilizing the above concepts an expression can be derived to calculate the expected delay time per turnout incident. The expected delay time per turnout incident consists of the time lost due to vehicle acceleration and deceleration:

\[
\left[ \frac{V_2 - V_1}{a_A} \right] + \left[ \frac{V_2 - V_2}{a_0} \right] = \frac{V_2}{2} \left[ \frac{1}{a_A} + \frac{1}{a_0} \right]
\]

and the expected time the empty vehicle spends in the turnout:

\[ (SF)/V \]

The expected delay time per turnout incident \( \bar{t} \) is therefore:

\[
\bar{t} = \frac{V_2 (a_A + a_D)}{2a_A a_D} + \frac{(SF)}{V}
\]

where:
\( V_l \) = velocity of loaded vehicle

\( F \) = expected F-factor.

Since there are \( n \) expected delays per unit distance then the total expected delay time per unit distance per vehicle (\( \overline{T} \)) is:

\[
\overline{T} = n \left[ \frac{V_l}{2a_A a_D} + \frac{SF}{V_l} \right] \tag{2.6}
\]

The \( n \) delays consist of:

1. The conflicts with the initial number of loaded vehicles in the unit distance, which is equal to the unit distance of road times the traffic flow rate divided by the velocity of the loaded vehicle, plus

2. The number of loaded vehicles entering the unit distance while the empty vehicle traverses the unit distance of road, which is equal to the delay time plus the uninterrupted travel time multiplied by the traffic flow rate.

The expression for \( n \) thus becomes:

\[
n = \frac{H}{V_l} + H \left[ \frac{1}{V_l} + n \left( \frac{SF}{V_l} + \frac{V_l (a_A + a_D)}{2a_A a_D} \right) \right] \tag{2.6}
\]

or

\[
n = \left[ \frac{H}{V_l} + \frac{H}{V_2} \right] \left[ 1 - \frac{H \overline{T} (a_A + a_D)}{V_l} - \frac{H V_2 (a_A + a_D)}{2a_A a_D} \right]^{-1} \tag{2.7}
\]

where:

\( H \) = traffic flow rate of loaded vehicles (i.e., vehicles per hour (vph)).

Substituting equation 2.7 into equation 2.6 and simplifying yields an expression for the total expected delay time that is
not directly a function of \( n \).

\[
T = \left( \frac{H}{V_I} + \frac{H}{V_2} \right) \left[ \frac{V_2 (a_A + a_D)}{2a_A a_D} \right] \left[ \frac{H S F H V_2 (a_A + a_D)}{1 - \frac{H S F H V_2 (a_A + a_D)}{2a_A a_D}} \right]^{-1}
\]

Once the total delay time per unit distance per vehicle has been calculated, the total delay cost per unit distance can be calculated by multiplying equation 2.8 by the traffic flow rate and the hauling cost. The use of equation 2.8 in determining the optimum turnout spacing will be discussed in section 2.5.
2.4 THEORETICAL DEVELOPMENT OF THE EXPECTED F-FACTOR

The calculation of the expected F-factor involves an analysis of the interactions between two separate traffic streams on a single-lane road. The objective is to translate the real traffic situation into a workable mathematical model. Several approaches are available. Fluid-flow analogies involve the principle that the movement of traffic will behave like the flow of fluids. The result is the description of the total traffic network rather than the conflicts between individual vehicles. Wohl and Martin (1967) suggested that these methods are only applicable for high traffic densities. Since log hauling systems involve low traffic densities another approach must be found.

Car-following theories involve intervehicle relationships. In general, it is assumed that the speed characteristics of each vehicle depend to some extent on the speed characteristics of each of the preceding vehicles; thus, a driver's response is constrained by the surrounding vehicles and the characteristics of the road (Wohl and Martin, 1967). This approach is too detailed for use in this study.

Traffic engineers have utilized probability theory to analyse delay situations for merging traffic, intersection control, and left-turn storage areas (Wohl and Martin, 1967), Drew (1968)). The turnout delay problem is similar to the merging traffic delay situation where one traffic stream constrains the movement of another traffic stream.

The model developed herein will utilize probability
theory. This approach involves the interaction between the "average" loaded vehicle and the "average" empty vehicle. In this approach there are certain inherent assumptions. Namely, a uniform velocity and rate of acceleration of the vehicles is assumed. It will be further assumed that the length of the vehicles will be neglected as well as the length of the turnouts. The effect of these assumptions on the model will be discussed later.

The three situations that will be discussed are the meeting of one loaded and one empty vehicle, the interactions between a single empty and a fleet of loaded vehicles, and the conflicts between a group of empty and a group of loaded vehicles.
2.4.1 ONE LOADED VEHICLE MEETING ONE EMPTY VEHICLE

The situation pertaining to the interaction between one empty vehicle and one loaded vehicle is illustrated in Figure 1. A backwards approach is utilized for solving this problem. The first phase is to determine where the two vehicles will meet (point B) if the empty vehicle does not utilize a turnout. Once this has been accomplished the location where the empty vehicle must begin to decelerate in order to stop at the critical turnout \( S_c \) is determined. At this moment the empty vehicle is at point C and the loaded vehicle, having backtracked an equal time interval, is at point A. As illustrated in the diagram, the critical distance \( X_{cf} \) is defined as the distance from the critical turnout where the two vehicles would meet if the empty vehicle did not pull into the critical turnout. Furthermore, the loaded vehicle will be exactly the F-factor times the turnout spacing from the critical turnout when the empty vehicle stops at this turnout. The loaded vehicle must travel the distance \( A-S_c \) minus the F-factor times the turnout spacing in the time the empty vehicle decelerates. Thus

\[
\frac{v_i v_2}{a_o} = v_i \left(\frac{X_{cf} + D_3}{v_2} \right) + X_{cf} - FS
\]

or

\[
X_{cf} = \left[ \frac{2FSv_2a_o + v_i v_2}{2a_o(v_i + v_2)} \right] 2.9
\]
Projected meeting point

<table>
<thead>
<tr>
<th>Turnout locations</th>
<th>S₀</th>
<th>S₁</th>
<th>S₂</th>
<th>S₃</th>
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<tbody>
<tr>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
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Distances

\[
\begin{align*}
&= \frac{V₀(X_{cf} + Dₛ)}{V₂} \\
&= \frac{X_{cf}}{V₂} - Dₛ \\
\end{align*}
\]

Times

\[
\begin{align*}
&= \frac{V₀(X_{cf} + Dₛ)}{V₂} \\
&= \frac{X_{cf}}{V₂} - Tₛ \\
\end{align*}
\]

where:

- \(Dₛ\) = stopping distance
- \(Tₛ\) = stopping time
- \(S_j\) = turnout locations
- \(F\) = F-factor
- \(X_{cf}\) = critical distance
- \(V₀\) = velocity of the loaded vehicle
- \(V₂\) = velocity of the empty vehicle

**Figure 1**---Interaction between an empty vehicle and a loaded vehicle
It is reasonable to assume that the location of point B is described by an uniform distribution on the interval 0−S. We will now introduce \( F_{\text{max}} \) which is defined as the largest F-factor before the empty vehicle can travel to the next turnout. The critical distance corresponding to a F-factor equal to zero is denoted by \( x_{co} \). Since \( x_{cf,\text{max}} \) minus \( x_{co} \) equals the turnout spacing an expression for \( F_{\text{max}} \) can be derived.

\[
F_{\text{max}} = \frac{(V_1 + V_2)}{V_2}
\]

The density function of the F-factor can be determined since for each critical distance there corresponds an unique F-factor and the density of the critical distance formula is uniform over the interval 0−S. Thus

\[
Pr(F_{\text{c}} < F < F_{\text{c}} + \delta F) = \frac{(x_{c}(F_{\text{c}} + \delta F) - x_{c,F_{\text{c}}})}{S} = \frac{\delta FV_2}{(V_1 + V_2)}
\]

where:

- \( F \) = a given F-factor
- \( \delta F \) = a small change in the F-factor.

Once the density function of the F-factor has been determined the expected F-factor can be determined by integrating the density function times the F-factor between zero and \( F_{\text{max}} \).

\[
\overline{F} = \int_{0}^{F_{\text{max}}} \frac{V_2 F}{V_1 + V_2} \delta F = \frac{V_2 F^2}{V_1 + V_2} \bigg|_{0}^{F_{\text{max}}} = \frac{V_1 + V_2}{2V_2}
\]

This expected F-factor is referred to as the simple F-factor to distinguish it from the expected F-factor equations developed in the next section. Those equations are referred to by their corresponding headway probability functions. A simulation model developed in Chapter 4 is utilized to confirm the simple F-factor equation.
2.4.2 ONE EMPTY VEHICLE MEETING A FLEET OF LOADED VEHICLES

In the previous section it was assumed each turnout incident was independent of any other. In this section the turnout events will be partially dependent on each other. This situation involves the investigation of the meeting of an empty vehicle and a fleet of loaded vehicles.

It is not necessary to analyse the interaction between the first loaded vehicle and the empty vehicle since this has been accomplished in the previous section. Once a loaded vehicle has passed the empty vehicle, stopped in the turnout, two situations may arise:

1. the next loaded vehicle may be close enough to prohibit the empty vehicle from advancing to the next turnout or
2. the empty vehicle may proceed.

An analysis of the headway distribution of the loaded vehicles is essential to solve this problem. Drew (1968) defines headway as "the interval of time between successive vehicles moving in the same lane and measured from head to head as they pass a point on the road." There exists a critical headway ($h_c$) where the empty vehicle can just travel to the next turnout and experience no delay in that turnout. The critical headway must be determined prior to the calculation of the probability of each of the above situations occurring.

Initially, the first loaded vehicle and the empty vehicle are positioned at turnout $S_c$ (Figure 2). The second loaded vehicle is at point B.
Situation pertaining to the empty vehicle experiencing no delay in turnout $S_2$ since it comes to a complete halt at the turnout as the loaded vehicle passes the turnout.

Figure 2---Diagram showing the physical situation leading to the critical headway
The time required for the second loaded vehicle to travel distance \( D \) is equivalent to the time required for the empty vehicle to travel one turnout spacing. Consequently, 

\[
T_i = T_S + T_A + (S-D_S-D_A)/v_2 = (2S a_A + v_2^2 (a_A + a_D))/(2a_D v_2) \tag{2.13}
\]

and

\[
D_i = v_i (2S a_D + v_2^2 (a_A + a_D))/(2a_D v_2) \tag{2.14}
\]

Since vehicle arrangements are generally measured in time units the critical headway can be easily calculated as:

\[
h_c = T_i + S/v_i = \frac{[2S a_D (v_i + v_2) + v_i v_2^2 (a_A + a_D)]}{[2a_D v_i v_2]} \tag{2.15}
\]

and the probability that a headway is less than the critical headway is:

\[
Pr(h < h_c) = \int_0^{h_c} g(h) \, dh \tag{2.16}
\]

where:

\[
g(h) = \text{headway probability density function.}
\]

The expected \( F \)-factor (\( F_2 \)) for headways less than the critical headway varies with respect to the expected headway for headways less than the critical headway. Thus:

\[
F_2 = \left[ \int_0^{h_c} hg(h) \, dh \right] v_i / S \tag{2.17}
\]

The expected \( F \)-factor (\( F_1 \)) for headways greater than the critical headway is the weighted average of functions similar to \( F_2 \) except that the limits of the integrals vary. This is because an empty vehicle can proceed one, two, or more turnout spacings before being forced into a turnout. The result is that the limits of the integrals vary with respect to the turnout spacing and the number of turnouts the empty vehicle
passes before being forced into a turnout. The simple F-factor will be utilized to estimate $\bar{F}$, since the actual function is complex and its limits are difficult to determine. Thus:

$$\bar{F} = \frac{(V_1 + V_2)}{2V_2}$$ \hspace{1cm} 2.18

A simulation model, developed in Chapter 4, is utilized to verify the accuracy of this method.

The expected F-factor equation for the meeting of one empty vehicle and a fleet of loaded vehicles consists of two parts:

1. the component that represents the average F-factor attributable to the delay situation where the headway is less than the critical headway and,

2. the component that represents the delay situation where the headway is greater than the critical headway.

The probability of each of these situations occurring must be calculated, as well as the expected F-factor attributable to each of the situations. Consequently, the expected F-factor equation is:

$$\bar{F} = \Pr(h > h_c)\bar{F}_1 + \Pr(h < h_c)\bar{F}_2$$ \hspace{1cm} 2.19

where:

$h$ = headway

$\bar{F}_1$ = the expected F-factor for headways greater than the critical headway

$\bar{F}_2$ = the expected F-factor for headways less than the critical headway.

The first turnout incident of an empty vehicle's trip is independent of the headway probability distribution and the expected F-factor resulting from this incident is the simple F-factor. The expected F-factor representing the rest of the
turnout incidents the empty vehicle encounters on its trip, is given by equation 2.19. The total number of trips per day and the total number of turnout incidents per day must be determined prior to calculating the expected F-factor. The expected F-factor is composed of two parts:

1. The portion that represents the first turnout incident on a trip which is equal to the simple F-factor times the number of trips per vehicle divided by the total number of turnout incidents.

plus

2. The portion representing the remaining turnout incidents on the trip which is equal to equation 2.19 times the remaining number of turnout incidents divided by the total number of turnout incidents.

Thus:

$$F = \left[ \Pr(h > h_c) F_1 + \Pr(h < h_c) F_2 \right] \frac{Q_3}{Q_3 + I} + \left[ \frac{(V_1 + V_2)}{2V_2} \right] \frac{I}{Q_3 + I}$$

where:

- $Q_3 = H_d - I = \text{number of headways}$
- $d = \text{number of "conflict" hours per day}$
- $I = \text{number of trips per day}$.

Four headway probability distributions and their respective expected F-factors are discussed in section 2.4.4. The expected F-factor equations derived in this section will be referred to by their respective headway probability distribution. Chapters 4 and 6 will discuss the relative effect of these equations compared to the simple F-factor and the average F-factor resulting from the meeting of two fleets of vehicles.
In this section the turnout incidents between a fleet of loaded and a fleet of empty vehicles will be examined. The four cases that will cause an empty vehicle to utilize a particular turnout are:

Case 1. An empty vehicle may stop at a turnout because a loaded vehicle is approaching.

Case 2. An empty vehicle may remain in a given turnout since a loaded vehicle is so close as to prohibit the empty vehicle from advancing to the next turnout.

Case 3. An empty vehicle is required to halt at a particular turnout since the next turnout is occupied (Figure 3).

Case 4. An empty vehicle cannot proceed to the next turnout but remains at its present turnout since the next turnout is occupied (Figure 4).

In the first two cases, situation A, the empty vehicle is forced to use a particular turnout since a loaded vehicle is approaching while in the latter two cases, situation B, the empty vehicle is required to use a particular turnout since the next turnout is occupied and a loaded vehicle is approaching. Since situation A has been discussed in section 2.4.2, the following discussion will primarily involve situation B.
Turnout locations $S_0, S_1, S_C, S_3$

Distances

where:

$\bigcirc$ = empty vehicle

$\bullet$ = loaded vehicle

$\downarrow$ = potential location of empty vehicle

$\bigcirc$ = location of set 1 empty vehicles

$S_3$ = turnout locations

Figure 3—Case 3—An empty vehicle stops at turnout $S_3$ since next turnout is occupied
Figure 3 illustrates the case in which an empty vehicle, travelling at a constant velocity, is delayed at a turnout since one empty vehicle or a group of empty vehicles (set 1) is occupying the next turnout. Herein this case will be referred to as a Case 3 situation. If set 1 did not occupy turnout $S_C$ then the empty vehicle would be able to stop at turnout $S_C$ instead of being forced to utilize turnout $S_3$. Maintaining this situation then the empty vehicle would be located between points A and B when the loaded vehicle is between points C and E. The expected F-factor for the Case 3 situation is:

$$F_3 = F_1 + \frac{S + S + V}{S + V} = \frac{F_1 + (V_1 + V_2)}{V_2} \quad 2.21$$

where:

$F_3$ = expected F-factor for a Case 3 situation.

Equations 2.12 and 2.19 can be used to determine an expected F-factor representing the meeting of an empty vehicle and a fleet of loaded vehicles. Utilizing this expected F-factor the expected distance the empty vehicle, stopped at $S_3$, is from the loaded vehicle can be determined. The expected F-factor equation is the sum of equation 2.18 and the distance between turnouts $S$ and $S$ divided by the turnout spacing plus the distance the loaded vehicle can travel in the time the empty vehicle can travel one turnout spacing divided by the turnout spacing. Since $F_1$ has been approximated by equation 2.18 $F_3$ becomes:

$$F_3 = \frac{3(V_1 + V_2)}{2V_2} \quad 2.22$$

After the loaded vehicle has passed both groups of empty vehicles and each vehicle has accelerated to its original speed the distance between the empty vehicle and the group of empty vehicles is the interval distance ($D_I$). The interval distance
is the turnout spacing plus the distance an empty vehicle can travel, at a constant velocity, in the time a loaded vehicle can travel one turnout spacing.

$$D_t = S(1+V_2/V_1)$$ \[2.23\]

As illustrated in Figure 4, Case 4 is the situation that stems from an empty vehicle waiting in a given turnout since the adjacent turnout is occupied and a loaded vehicle is approaching. If the headway between any two loaded vehicles is less than the critical headway then the expected F-factor attributable to the second loaded vehicle is equivalent to the expected F-factor of equation 2.17.

The expected F-factor equation representing the meeting of two fleets of vehicles is composed of the probability that each of the above cases will occur, times their respective expected F-factor. The expected F-factor equation representing the meeting of two fleets of vehicles is therefore:

$$F = \Pr(h > h_c | A) F_a + \Pr(h < h_c | A) F_a + \Pr(h < h_c | B) F_a + \Pr(h > h_c | B) (F_a + 1 + V_2 / V_1)$$ \[2.24\]

where:

A = conditional event that an empty vehicle is required to utilize a turnout because a loaded vehicle is approaching

B = conditional event that an empty vehicle must utilize a turnout because an empty vehicle or a group of empty vehicles already occupy the adjacent turnout and a loaded vehicle is approaching.

The expected F-factor for each of the above cases can be easily determined but the probability of each of the events occurring is difficult to determine. It would not be necessary to
calculate these probabilities provided it can be shown that the probability of a conditional event B is relatively small. Consequently, it is desirable to determine an upper limit of the probability of event B. If the two fleets of vehicles are independent of each other, then the probability of the number of empty and loaded vehicles arriving during any given time interval, t, can be determined. Since for low traffic flow rates this probability is relatively small then the expected F-factor equation representing the meeting of a single empty vehicle and a fleet of loaded vehicles will adequately describe the expected F-factor resulting from the interactions between two fleets of vehicles. This statement is supported by a simulation model developed in Chapter 4.
Figure 4---Case 4--An empty vehicle remains in turnout $S_3$ since next turnout is occupied

where:

- $\bigcirc$ = empty vehicles
- $\bullet$ = loaded vehicles
- $\bigtriangleup$ = location of set 1 vehicles

$S_j = $ turnout locations
2.4.4 HEADWAY PROBABILITY DISTRIBUTIONS

The theoretical technique described in section 2.4.2 requires some knowledge of the probabilistic aspects of traffic behaviour. There have been no studies undertaken to determine the headway distribution of logging trucks; however, traffic engineers have conducted studies to determine the arrival distribution of low traffic flow along highways and rural roads. Various authors have attempted to describe the headway or arrival distribution using various probability distributions. These have included the negative binomial distribution, the Pearson Type III distribution, the Erlang distribution, and the Pearson Type I distribution. Descriptions of these distributions may be found in Wohl and Martin (1967), Haight (1963), and Drew (1968).

Gerlough (1955) tabulated the arrival frequency of field data for various rural and urban roads in the United States. The attempt to fit a Poisson distribution to four sets of data by performing goodness of fit tests proved to be unsuccessful at the five percent acceptance level. A cause for the failure of the tests may have been the fact that the Poisson distribution assumes there can be more than one arrival during a time interval equivalent to the minimum headway. The minimum headway is the theoretical minimum allowable time interval between the fronts of successive vehicles. At very low traffic flows this assumption may prove to be insignificant.

Wohl and Martin (1967) utilized a shifted exponential curve to examine the frequency distribution of vehicle headways. In
his empirical example (1000 vph) the shifted exponential curve fits the data better than the non-shifted curve for lower headways while the reverse held true for larger headways. The shifted exponential curve states that:

$$Pr(h>t) = e^{-h(t-\gamma)}$$ \hspace{1cm} (2.25)

where:

$$t = \text{time interval}$$

$$\gamma = \text{minimum headway}.$$ 

Schuhl (1955) developed a partially shifted exponential curve of the form:

$$Pr(h>t) = e^{-\left(\frac{t-\gamma}{h_2-\gamma}\right)} + e^{-t/h_2}$$ \hspace{1cm} (2.26)

where:

$$h_1 = \text{mean headway for constrained flow}$$

$$h_2 = \text{mean headway for free flow}.$$ 

This equation allows the speed characteristics of some of the vehicles to be influenced by the characteristics of other vehicles. This equation to some extent involves car-following theory. Wohl and Martin argued that since all vehicles are constrained at higher flows then equation 2.26 can be restated as:

$$Pr(h>t) = e^{-\left(\frac{t-\gamma}{h_2-\gamma}\right)}$$ \hspace{1cm} (2.27)

Wohl and Martin tested equation 2.27 on two sets of data and concluded that this equation was acceptable at the ten percent level of significance, for a data set with a flow rate of 500 vph, but was rejected for a data set with a flow rate of 1000 vph. Consequently, a general exponential equation does not adequately describe these sets of data.

Since these authors were not able to consistently fit data to a probability distribution several distributions will be...
analysed. In the analysis it will be assumed that the empty vehicle will wait in a turnout until a headway is larger than the critical headway. The probability that the time interval between two loaded vehicles is greater than the critical headway must be determined. This is equivalent to the probability of there being no arrivals during this time period. The expected F-factor formulas for headways less than the critical headway will be derived. The probability distributions that will be analysed are the uniform arrival distribution, the exponential headway distribution, the Erlang headway distribution, and the Pearson Type III headway distribution.

2.4.4.1 UNIFORM ARRIVAL DISTRIBUTION

The uniform probability distribution has a constant value of 1/(i-j) over its interval (i,j). Feller (1966) showed that for an uniform arrival distribution the probability that the headway is greater than the critical headway is:

\[ Pr(h > h_c) = (1-h_c)^m \quad \text{for } 0<h_c<1 \]

where:

\[ m = \text{number of headways}. \]

It was determined in Section 2.4.2 that the number of headways is given by Q_3. Therefore the above equation may be rewritten as:

\[ Pr(h > h_c) = (1-h_c)^{Q_3} \quad 2.28 \]

The density function of this distribution is the derivative of the probability that the headway is less than the critical
headway. Thus:

\[ g(h) = \frac{\partial [Pr(h<h_c)]}{\partial h} = -Q_3(1-h)^{Q_3-1} \]

The expected F-factor formula for headways less than the critical headway is derived as follows:

\[ SF_2 = \frac{\int_0^{h_c} h g(h) \delta h}{\int_0^{h_c} g(h) \delta h} \]

\[ F_2 = \frac{V_i}{S} \left[ 1 - \frac{Q_3 ([1-h_c]^{Q_3+1}-1)}{(Q_3+1) ([1-h_c]^{Q_3-1})} \right] \]

2.4.4.2 EXPONENTIAL HEADWAY DISTRIBUTION

The gap density function of the exponential headway distribution is:

\[ g(h) = \frac{\partial [Pr(h<h_c)]}{\partial h} = Q_3 e^{-Q_3 h} \]

This gap density function can be integrated between zero and the critical headway to determine the probability of a headway being less than the critical headway, yielding:

\[ Pr(h<h_c) = 1-e^{-Q_3 h_c} \]

Furthermore, the expected F-factor equation for headways less than the critical headway can be derived from the gap density function.

\[ SF_2 = \frac{\int_0^{h_c} h g(h) \delta h}{\int_0^{h_c} g(h) \delta h} = \frac{e^{-Q_3 h_c}/Q_3 (-Q_3 h_c - 1)}{e^{-Q_3 h - 1}} \]

\[ F_2 = V_i \left[ e^{-Q_3 h_c} (-Q_3 h_c - 1) + 1 \right] / \left[ S(-Q_3 (e^{-Q_3 h_c} - 1)) \right] \]
2.4.4.3 ERLANG HEADWAY DISTRIBUTION

The distinction between the gamma and Erlang probability functions is that the Erlang function is restricted to positive integer values for alpha. The gap density function of the Erlang distribution is:

\[ g(h) = (\alpha Q_3)^\alpha h^{\alpha-1} e^{-\alpha Q_3 h} / (\alpha - 1)! \]

Wohl and Martin (1967) showed that the probability of a headway being less than the critical headway is:

\[ Pr(h < h_c) = \int_0^{h_c} [\lambda (\lambda h)^{\alpha-1} e^{-\lambda h} \delta h] / [ (\alpha - 1)! ] \]
\[ = 1 - e^{-\lambda h_c} \sum_{i=0}^{\alpha} (\lambda h_c)^i / i! \]

2.32

The logging vehicle headway frequency distributions produced from data collected by Smith and Tse (1977) and Boyd and Young (1969) have been shown to be left skewed (Chapter 4). Consequently, the expected F-factor equations will be developed for functions that are left skewed. If alpha equals two then the probability of a headway being less than the critical headway is:

\[ Pr(h < h_c) = 1 - e^{-\lambda h_c} (1 + \lambda h_c) \]

2.33

where:

\[ \lambda = Q_3C = 2Q_3 \]
\[ g(h) = \lambda^2 h e^{-\lambda h} \]

and the expected F-factor is derived as follows:

\[ \overline{F_2} = \frac{\int_{h_c}^{h} h g(h) \delta h}{\int_{h_c}^{h} g(h) \delta h} \]
\[ V_1 = \int_{h_c}^{h} g(h) \delta h \]
\[ S = \left[ e^{-\lambda h_c} (-\lambda^2 h_c^2 - 2\lambda h_c - 2) / \lambda + 2 / \lambda \right] \]

2.34

Similarly, the expected F-factors and probability functions can
be calculated for other values of alpha.

2.4.4.4 PEARSON TYPE III HEADWAY DISTRIBUTION

The gap density function of the Pearson Type III distribution is:

\[ g(h) = \left( b^a [h-c]^a - 1 e^{-b(h-c)} \right) / \Gamma(a) \]

where:

- \( c \) = minimum value of \( h \)
- mean = \( a/b + c \)
- variance = \( a/b^2 \)
- \( \Gamma(a) = \text{gamma function} \)

\[ = \int_0^\infty z^{a-1} e^{-z} \, dz \]

\[ = (a-1)! \quad \text{for } a = 0, 1, 2, \ldots \]

For the same reasons as cited for the Erlang headway distribution the value of \( 'a' \) will be set to two. The gap density function becomes:

\[ g(h) = b^2 (h-c)e^{-b(h-c)} \quad c<h<\infty \]

The probability that the headway is less than the critical headway is:

\[ \Pr(h<h_c) = \int_{c}^{h_c} g(h) \, dh = e^{-b(h-c)} (-bh-1+cb) \]

\[ \Pr(h<h_c) = 1-e^{-b(h_c-c)} (bh_c+1-cb) \]

and the expected F-factor for headways less than the critical headway is derived as follows:

\[ \bar{F} = \frac{\int_{c}^{h_c} h g(h) \, dh}{V} = \frac{\left[ e^{-b(h-c)} (-bh^2-2h-2/b+cbh+c) \right]_{h_c}^{h_c}}{\left[ e^{-b(h-c)} (-bh+1+cb) \right]_{c}^{c}} \]
\[ P_2 = \frac{V}{S} \left[ \frac{[c + 2/b + e^{-b(h_c - c)} (-bh_c^2 - 2h_c - 2/b + cbh_c + c)]}{[1 - e^{-b(h_c - c)} (bh_c + 1 - cb)]} \right] \]
2.5 DEVELOPMENT OF THE COST EQUATION

The only costs that are needed in the cost equation are those that are functions of the turnout spacing: the turnout construction cost, the turnout maintenance cost, and the delay cost attributable to turnout spacing. The general cost equation is:

\[
C_T = \frac{C_c}{S} + \frac{C_m}{S} + Q_i H M T
\]

where:

- \(C_T\) = total cost attributable to turnout spacing per unit distance of road
- \(C_c\) = turnout construction cost
- \(C_m\) = turnout maintenance cost
- \(H\) = traffic flow rate
- \(M\) = adjusted truck hauling cost
- \(Q_i\) = expected useful life of the road
- \(S\) = distance between turnouts
- \(\bar{T}\) = delay time per vehicle per unit distance of road.

Since a vehicle may have to queue at the landing regardless of the turnout spacing an adjusted hauling cost is utilized. A necessary condition for determining the optimum turnout spacing is that the first derivative of the cost function with respect to the turnout spacing is equal to zero. Once the derivative has been determined then a search technique can be used to locate this zero. Furthermore, when the value of the second derivative of the cost function is positive, the direction of concavity of the function is upwards. Provided the value of the second derivative is positive this zero represents a
minimum value. Some of the numerical analysis techniques available are bisection, regula falsi, secant, and Newton's method. For further reference consult Conte and deBoor (1972) or Arden and Astill (1970).

When discounting of maintenance costs is not used the cost equation becomes:

\[ C_T = \frac{C}{S} + \frac{Q}{HMT} \]  \hspace{1cm} (2.38)

where:

\[ C = C_M + C_C \]

Three forms of the cost equation shall be utilized in determining the optimum turnout spacing. In the first case it shall be assumed that the turnout spacing is non-uniform and the expected F-factor is a function of the turnout spacing. In the second case the expected F-factor will be a function of the turnout spacing but the turnout spacing will be assumed to be uniform. The third case will assume uniform turnout spacing but will utilize the simple F-factor.

If the turnout spacing is non-uniform and the expected F-factor is a function of turnout spacing then the cost equation becomes:

\[
C_T = \frac{C}{y(S)} + \frac{Q}{HMT} \left[ \frac{\frac{y(S)P}{V_1} + \frac{V_2}{V_1}}{1 - \frac{Hy(S)P}{V_1}} \right] \left[ \frac{1}{K} \right]^{-1}
\]

\hspace{1cm} (2.39)

where:

\[ y(S) = \text{turnout spacing function} \]

\[ K = \frac{2a_n a_0}{(a_n + a_0)} \]

and the optimum turnout spacing occurs when:
0 = \left[ y(S) \right]^2 \left( \frac{\partial F}{\partial S} + \left[ \frac{\partial [y(S)]}{\partial S} \right] F \right) 
- H^2 \bar{F}^2 k^2 v_2 \left( \frac{\partial [y(S)]}{\partial S} \right) - y(S) \left( 2 \left[ \frac{\partial [y(S)]}{\partial S} \right] C v_i v_2 \bar{F} k h [h v_2 - k] \right) 
- v^2 \bar{F} v_2 \left( \frac{\partial [y(S)]}{\partial S} \right) C [k^2 - 2 h v_2 k + h^2 v_2^2] 
\[ 2.40 \]

By maintaining the expected F-factor equation to be a function of the turnout spacing and allowing the turnout spacing to be uniform then the cost equation becomes:

\[ C_T = -l - q_h \frac{H}{H} \left[ \frac{S \bar{F} v_2}{V} \right] \left[ \frac{H S \bar{F} h v_2}{V_i} \right] \left[ \frac{V}{V_i} \right] \left[ \frac{K}{K} \right] \left[ 1 - \frac{H}{V_i} - \frac{V}{V_i} \right] \] 
\[ 2.41 \]
and the optimum turnout spacing occurs when:

\[ 0 = S^2 \left[ M q_i, H^2 k^2 \left( \bar{F} + S \frac{\partial F}{\partial S} \right) - V + v_2 \right] - k^2 H^2 f^2 C v_i \] 
\[ + S \left[ 2 C v_i h h k v_i \left( k - h v_2 \right) \right] + C v_i v_2 \left( 2 h v_2 k - k^2 - H^2 v_2^2 \right) \] 
\[ 2.42 \]

The cost equation involving the simple F-factor equation and uniform turnout spacing is:

\[ C_T = -l - q_h \frac{H}{H} \left[ \frac{S \bar{F} v_2}{V} \right] \left[ \frac{H S \bar{F} h v_2}{V_i} \right] \left[ \frac{V}{V_i} \right] \left[ \frac{K}{K} \right] \left[ 1 - \frac{H}{V_i} - \frac{V}{V_i} \right] \] 
\[ 2.43 \]
and the optimum turnout spacing occurs when:

\[ 0 = S^2 \left[ M q_i, H^2 k^2 \bar{F} \left( V + v_2 \right) - k^2 H^2 f^2 C v_i \right] 
\[ + S \left[ 2 C v_i h h k v_i \left( k - h v_2 \right) \right] + C v_i v_2 \left( 2 h v_2 k - k^2 - H^2 v_2^2 \right) \] 
\[ 2.44 \]

Since the simple F-factor is utilized in this equation, the quadratic formula can be used to find the roots of the equation. The use and the sensitivity of the equations will be discussed in Chapter 6.
2.6 DISCOUNTING OF THE COST EQUATION

Turnout construction costs are expenses of the present while delay costs are incurred over the life of the road. In order to relate present costs to future costs some form of discounting is required. The present worth formula for equal payments made in the future states that:

\[ PW = \frac{Q_2 (1+i)^m - 1}{i(1+i)^m} \]

where:

- \( PW \) = present worth
- \( Q_2 \) = value of the constant cost
- \( i \) = interest rate over period \( m \)
- \( m \) = life of project.

This formula can be incorporated into the cost functions developed in section 2.5 by allowing:

\[ Z_2 = \frac{(1+i)^m - 1}{i(1+i)^m} \]

and incorporating this equation into the delay component of the cost functions. Since equation 2.46 is independent of the turnout spacing, the inclusion of discounting into the derivative of the cost function will result in an additional constant. This process executed on equations 2.43 and 2.44 results in:

\[ C = \frac{C}{S} + Q_2 HMZ_2 \left[ \frac{H}{V_1} \right] + \frac{1}{\left( \frac{H}{V_1} - \frac{1}{K} \right)} \left[ 1 - \frac{HSF}{V_1} \right] \]

and the optimum turnout spacing occurs when:

\[ 0 = S_2 \left[ M_0 H^2 K^2 Z_2 (V_1 + V_2) - K^2 H^2 F_2 C V_2 \right] + S_2 \left[ 2C V_2 H F K (V_1 - H V_2) \right] + C V_2^2 (2H V_2 K - K^2 - H^2 V_2) \]

In the development of this transformation process it was
assumed that the maintenance cost is not discounted and there is a constant traffic flow volume per year. In reality the traffic flow volume per year is not a constant but is variable from year to year. Chapter 6 will discuss the sensitivity of this assumption.
3.0 THE ARRIVAL DISTRIBUTION OF LOGGING TRUCKS

3.1 ARRIVAL DATA FROM TWO OPERATIONS

There have been no studies to determine the headway distribution of logging trucks, but there have been two studies that produced basic data that may be suitable in the analysis of interarrival times. The two study areas were located in coastal British Columbia (B.C.) and in north central B.C. The coastal study, utilizing five-minute time intervals, measured the time the loaded logging trucks arrived at the log dump. The sample consisted of 201 arrivals over an eight-day period with an average flow rate of approximately two-and-one-half vehicles per hour. These data are quite restrictive for determining the headway distribution of vehicles since there is only one frequency class for vehicle headways of between zero and five minutes.

The interior operation consists of vehicles travelling on a company haul road and utilizing one of two government weigh scales. The first scale was open between the hours of 6 A.M. and 5 P.M. while the second scale was open until all of the hauling operations had ceased for the day. The loaded vehicles would stop at the first weigh scale provided it was open. Since the arrival time at the various scales was recorded to the nearest minute these data appear to be more useful for headway analysis than do the data from the coastal operation. Observations were recorded for the month of February 1976 with a total sample size of 813 observations and
an average flow rate of approximately three vehicles per hour. For a more detailed description of the two studies consult Boyd and Young (1969) and Smith and Tse (1977). The interarrival times of the two studies will be analysed in section 3.2.
3.2 ANALYSIS OF THE ARRIVAL DATA

The primary purpose of the following analysis was to determine whether the arrival frequency of logging trucks fits a known probability function. Since the expected F-factor equations involve headways instead of arrival times the data were grouped into headway frequency classes rather than arrival frequency classes. A $\chi^2$ goodness of fit test was utilized to examine the closeness of fit of the data to various hypothesized probability distributions. This technique is used to compare the expected frequency, for a given distribution, to the observed frequency. The chi-square value is determined as:

$$\chi^2 = \sum_{i=1}^{k} \frac{(o_i - e_i)^2}{e_i}$$

where:

- $o_i$ = observed frequency of the $i$ cell
- $e_i$ = expected frequency of the $i$ cell
- $k$ = total number of cells.

The initial step is to determine the cell widths and the upper limit of the first cell. Once this has been accomplished a frequency histogram of the observed data can be determined. The type of distributions most likely to fit the observed data can be estimated from these graphs. A computer program written in the programming language BASIC for the Hewlett Packard (HP) 9830A calculator is used in this analysis.

The headway frequency histogram of the coastal study is illustrated in Figure 5. This graph shows that the frequency of the headways is left skewed. Goodness of fit tests were performed for the exponential, shifted exponential, and Erlang
probability distributions. The goodness of fit calculations are located in Appendix 4 while Table I summarizes the results. The results indicate that none of the tested distributions fit the observed data.

Figure 6 illustrates the headway frequency distribution for the north central interior study. This figure groups the headways into one-minute intervals while Figure 7 uses class widths of five minutes. Both of these graphs indicate that a skewed distribution may fit the observed data. Table II summarizes the results of the goodness of fit tests. As was the case for the coastal study, none of the tested probability functions, with class widths of one minute, fit the observed data. When the class widths were increased to five minutes the exponential and shifted exponential functions were not rejected but these functions do not adequately describe the data for headways less than the critical headway. The goodness of fit tests failed for the one-minute class intervals because there were too few observations for the lower headways (i.e., headways less than three minutes).

If the values of the expected F-factor formulas do not change radically with respect to various probability distributions then one or more of the tested probability distributions may be suitable for use in the expected F-factor formulas. The sensitivity of the expected F-factor models will be discussed in Chapters 4 and 6.
Figure 5—Frequency histogram of headways for coastal study (5-minute intervals)
<table>
<thead>
<tr>
<th>Probability Dist.</th>
<th>Reject Hypoth</th>
<th>Number of Classes</th>
<th>Class Width (min)</th>
<th>LCL (min)</th>
<th>Computed $\chi^2$</th>
<th>Table $\chi^2$</th>
<th>$\nu$</th>
<th>$\chi^2$ $\nu$ .005</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exponential</td>
<td>Yes</td>
<td>25(14)</td>
<td>5</td>
<td>2.5</td>
<td>76.3</td>
<td>12</td>
<td>28.3</td>
<td></td>
</tr>
<tr>
<td>Erlang ($\lambda=2$)</td>
<td>Yes</td>
<td>25(12)</td>
<td>5</td>
<td>2.5</td>
<td>62.7</td>
<td>10</td>
<td>25.2</td>
<td></td>
</tr>
<tr>
<td>Shift expon</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>:0.1 min</td>
<td>Yes</td>
<td>25(14)</td>
<td>5</td>
<td>2.5</td>
<td>75.2</td>
<td>12</td>
<td>28.3</td>
<td></td>
</tr>
<tr>
<td>:0.5 min</td>
<td>Yes</td>
<td>25(14)</td>
<td>5</td>
<td>2.5</td>
<td>71.0</td>
<td>12</td>
<td>28.3</td>
<td></td>
</tr>
<tr>
<td>:1.0 min</td>
<td>Yes</td>
<td>25(14)</td>
<td>5</td>
<td>2.5</td>
<td>65.8</td>
<td>12</td>
<td>28.3</td>
<td></td>
</tr>
</tbody>
</table>

- Rejection of hypothesis

* The parenthesized number is the final number of frequency classes while the non-parenthesized number is the original number of frequency classes

- Lower class limit

* The number of degrees of freedom
Figure 6—Frequency histogram of headways for interior study (1-minute intervals)
Figure 7—Frequency histogram of headways for interior study (5-minute intervals)
Table II—Goodness Of Fit Tests—Interior Study—Both Scales

<table>
<thead>
<tr>
<th>Probability Dist.</th>
<th>Reject Hypoth</th>
<th>Number of Classes</th>
<th>Class Width (min)</th>
<th>LCL (min)</th>
<th>Computed $\chi^2$</th>
<th>Table $\chi^2$</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exponential</td>
<td>Yes</td>
<td>80 (53)</td>
<td>1</td>
<td>0.5</td>
<td>51</td>
<td>51</td>
<td>53.7</td>
</tr>
<tr>
<td>Erlang ($\lambda = 2$)</td>
<td>Yes</td>
<td>80 (48)</td>
<td>1</td>
<td>0.5</td>
<td>46</td>
<td>46</td>
<td>53.7</td>
</tr>
<tr>
<td>Shift expcn :0.1 min</td>
<td>Yes</td>
<td>80 (53)</td>
<td>1</td>
<td>0.5</td>
<td>51</td>
<td>51</td>
<td>53.7</td>
</tr>
<tr>
<td>Shift expcn :0.5 min</td>
<td>Yes</td>
<td>80 (52)</td>
<td>1</td>
<td>0.5</td>
<td>50</td>
<td>50</td>
<td>53.7</td>
</tr>
<tr>
<td>Exponential</td>
<td>Yes</td>
<td>21 (17)</td>
<td>5</td>
<td>0.5</td>
<td>15</td>
<td>15</td>
<td>32.8</td>
</tr>
<tr>
<td>Erlang ($\lambda = 2$)</td>
<td>Yes</td>
<td>21 (13)</td>
<td>5</td>
<td>0.5</td>
<td>11</td>
<td>11</td>
<td>26.8</td>
</tr>
<tr>
<td>Shift expcn :0.1 min</td>
<td>No</td>
<td>21 (17)</td>
<td>5</td>
<td>0.5</td>
<td>15</td>
<td>15</td>
<td>32.8</td>
</tr>
<tr>
<td>Shift expcn :0.5 min</td>
<td>No</td>
<td>21 (16)</td>
<td>5</td>
<td>0.5</td>
<td>14</td>
<td>14</td>
<td>31.3</td>
</tr>
<tr>
<td>Exponential</td>
<td>No</td>
<td>20 (16)</td>
<td>5</td>
<td>5.5</td>
<td>14</td>
<td>14</td>
<td>31.3</td>
</tr>
</tbody>
</table>

- Rejection of hypothesis

* The parenthesized value is the final number of frequency classes while the non-parenthesized number is the original number of frequency classes

- Lower class limit

+ The number of degrees of freedom
4.0 SIMULATION FOR THE VERIFICATION OF THE EXPECTED F-FACTOR EQUATIONS

Simulation can be an valuable tool in the verification of the accuracy of mathematical equations. In this instance it will be used to determine the accuracy and limitations of the expected F-factor equations. The technical description of the three simulation models is discussed in Appendix 5 while the features and results of the models will be analysed in the following section.
4.1 THE SIMULATION MODELS

A discrete, critical event stochastic simulation model was written in the general purpose programming language FORTRAN for the IBM 370 Model 168 computer to confirm or reject the simple F-factor equation. The program simulates the meeting of one empty vehicle and one loaded vehicle. Initially, the empty and loaded vehicles are randomly separated by a distance of between five and ten miles. The position where the two vehicles would meet if the vehicles travel unhindered is determined. The final phase was to move the empty vehicle to the first turnout it can safely utilize. Simultaneously, the loaded vehicle was backtracked for an equal time interval. The F-factor can be easily determined by calculating the distance the loaded vehicle is from the empty vehicle, stopped in the turnout, and dividing this value by the turnout spacing. This procedure is repeated for the desired number of repetitions and the sample mean is determined. The program is executed either nine or twenty times and the mean and standard deviation of the samples are determined. A t-test is performed with the null hypothesis being that the mean of the average F-factors is equal to the simple F-factor. The speed and turnout spacing combinations were arbitrarily chosen. The results of the simulations are documented in Table III. The results of the simulations, with a one percent level of significance, were not rejected seventy-three percent of the time.
Table III—F-factor Simulation — The Interaction Between A Single Loaded Vehicle And A Single Empty Vehicle Based On Equation 2.12

<table>
<thead>
<tr>
<th>Sample</th>
<th>Vehicle Speed</th>
<th>Turn</th>
<th>Ave.</th>
<th>S. D.</th>
<th>Eq.</th>
<th>Rejection of Hypothesis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded</td>
<td>Empty</td>
<td>Spac</td>
<td>F</td>
<td>x10^{-3}</td>
<td>F</td>
<td>( \alpha = 0.05 )</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>10</td>
<td>20</td>
<td>0.1</td>
<td>0.7517</td>
<td>5.9</td>
<td>0.7500</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>10</td>
<td>20</td>
<td>0.5</td>
<td>0.7524</td>
<td>4.5</td>
<td>0.7500</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>30</td>
<td>30</td>
<td>0.05</td>
<td>0.9995</td>
<td>3.0</td>
<td>1.0000</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>30</td>
<td>30</td>
<td>0.15</td>
<td>1.0025</td>
<td>4.4</td>
<td>1.0000</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>30</td>
<td>40</td>
<td>0.3</td>
<td>0.8923</td>
<td>5.2</td>
<td>0.8750</td>
</tr>
<tr>
<td>10000 (9)</td>
<td>10</td>
<td>35</td>
<td>0.15</td>
<td>0.6391</td>
<td>3.8</td>
<td>0.6429</td>
</tr>
<tr>
<td>10000 (20)</td>
<td>10</td>
<td>35</td>
<td>0.2</td>
<td>0.6453</td>
<td>3.4</td>
<td>0.6429</td>
</tr>
<tr>
<td>10000 (20)</td>
<td>30</td>
<td>40</td>
<td>0.2</td>
<td>0.8684</td>
<td>14.5</td>
<td>0.8750</td>
</tr>
<tr>
<td>10000 (20)</td>
<td>30</td>
<td>40</td>
<td>0.3</td>
<td>0.9026</td>
<td>14.1</td>
<td>0.8750</td>
</tr>
<tr>
<td>10000 (20)</td>
<td>10</td>
<td>35</td>
<td>0.15</td>
<td>0.6386</td>
<td>7.8</td>
<td>0.6429</td>
</tr>
<tr>
<td>10000 (20)</td>
<td>10</td>
<td>35</td>
<td>0.3</td>
<td>0.6382</td>
<td>10.3</td>
<td>0.6429</td>
</tr>
</tbody>
</table>

1 The sample specifications: where the non-parenthesized number is the sample size and the parenthesized number is the number of samples.

2 The distance between turnouts in miles.

3 Standard deviation.

4 The equation value of the expected F-factor.

5 Hypothesis was not rejected for a 10% level of significance.

6 Hypothesis was not rejected for a 1% level of significance.
A similar simulation model was used to test the validity of the expected F-factor equation resulting from a single empty vehicle interacting with a fleet of loaded vehicles. This model includes the probabilistic aspects of flow behaviour by having the headway distribution of the loaded vehicles conform to a predetermined probability distribution. The model simulates the meeting of a single empty vehicle and a fleet of loaded vehicles. The empty vehicle progresses along an endless road until it meets a loaded vehicle. It is then backtracked to the first turnout it can safely utilize. The model allows the empty vehicle to wait in the turnout until one or more loaded vehicles have passed. The experience of the model is collected by the same method and the expected F-factor equations tested in the same manner as the previous model. Simulations were conducted for the exponential and Erlang (alpha=2) headway frequency distributions. The results are tabulated in Tables IV and V. With a one percent level of significance the exponential F-factor equation was within acceptable limits ninety percent of the time while the estimate of the Erlang (alpha=2) F-factor equation was always within acceptable limits.

The range of traffic flow for which the probability form of the expected F-factor equations adequately describe the meeting of two fleets of vehicles must yet be determined. Consequently, a simulation model was developed to predict the expected F-factor resulting from the meeting of two fleets of vehicles. Since this model is more complex than the previous models a General Purpose Simulation System V (GPPSV) program was used to model this situation. The characteristics of the
model are outlined in Appendix 5. The results of the simulation runs are tabulated in Table VI. The equations were tested by the same method as the previous models. The results indicate that the exponential F-factor equation is sufficiently accurate for a traffic flow rate of 60 vph or less and therefore equation 2.19 is sufficiently accurate for all practical levels of traffic flow involved in log hauling. A reason for the failure of the t-test, with a flow rate of 100 vph, is that the number of conditional event B delay situations has increased to the point where they significantly affect the expected F-factor.
Table IV—F-factor Simulation -- An Empty Vehicle Meeting Loaded Vehicles Based On Equations 2.18, 2.19, 2.30, And 2.31

Sample size = 10000
Number of samples = 9
Acceleration = 19759 mph²
"Conflict" hours = 11
Exponential headway distribution

<table>
<thead>
<tr>
<th>Vehicle Speed (mph)</th>
<th>Distance (miles)</th>
<th>Traffic Flow (vph)</th>
<th>Ave. S.D.</th>
<th>Expect</th>
<th>Rejection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded Loaded</td>
<td>Empty Empty</td>
<td>Rate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Between</td>
<td>F X10⁻³</td>
<td>F</td>
<td>of Hypothesis</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Flow</td>
<td></td>
<td></td>
<td>α = 0.05</td>
</tr>
<tr>
<td>10 20 0.1</td>
<td>1</td>
<td>0.7476 3.06 0.7507</td>
<td>Yes³</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 0.7492 3.14 0.7514</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 0.7523 4.46 0.7526</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 0.7521 4.18 0.7543</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 0.7023 5.02 0.7034</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 0.6247 3.16 0.6269</td>
<td>No</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 30 0.05</td>
<td>1</td>
<td>0.9980 5.42 1.0020</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 1.0056 4.23 1.0042</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 1.0131 3.23 1.0085</td>
<td>Yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 1.0215 6.07 1.0208</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 1.0922 6.03 1.0971</td>
<td>Yes³</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 1.1317 5.38 1.1300</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>40 30 0.2</td>
<td>1</td>
<td>1.1697 10.66 1.1684</td>
<td>No²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 1.1694 6.03 1.1703</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 1.1690 3.24 1.1737</td>
<td>Yes</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 1.1804 5.26 1.1817</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 1.1503 5.59 1.1558</td>
<td>Yes³</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 1.0690 6.03 1.0718</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30 40 0.3</td>
<td>1</td>
<td>0.8737 4.76 0.8767</td>
<td>No²</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 0.8799 5.85 0.8785</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 0.8811 4.07 0.8815</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10 0.8841 5.18 0.8873</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60 0.8138 7.80 0.8165</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100 0.7062 4.86 0.7054</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1 Standard deviation
2 Hypothesis was not rejected for a 10% level of significance
3 Hypothesis was not rejected for a 1% level of significance
Table V--F-factor Simulation -- An Empty Vehicle Meeting
Loaded Vehicles Based On Equations 2.18, 2.19, 2.33, And 2.34
Sample size = 10000
Number of samples = 9
"Conflict" hours = 11
Erlang(alpha=2) headway distribution

<table>
<thead>
<tr>
<th>Vehicle Speed</th>
<th>Distance</th>
<th>Traffic Ave.</th>
<th>S.D.</th>
<th>Expect</th>
<th>Rejection</th>
</tr>
</thead>
<tbody>
<tr>
<td>(mph)</td>
<td>(mph)</td>
<td>(miles)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td>40</td>
<td>0.3</td>
<td>1</td>
<td>0.8740</td>
<td>4.46</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>0.8756</td>
<td>5.44</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>0.8784</td>
<td>6.71</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td>0.8974</td>
<td>4.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>0.9762</td>
<td>5.67</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>0.8317</td>
<td>6.96</td>
</tr>
<tr>
<td>40</td>
<td>30</td>
<td>0.2</td>
<td>1</td>
<td>1.1679</td>
<td>6.13</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2</td>
<td>1.1647</td>
<td>6.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>1.1692</td>
<td>5.14</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td>1.1829</td>
<td>6.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>60</td>
<td>1.3243</td>
<td>9.39</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>100</td>
<td>1.2811</td>
<td>7.94</td>
</tr>
</tbody>
</table>

¹ Standard deviation

² Hypothesis was not rejected for a 10% level of significance
Table VI---F-factor Simulation -- Interaction Between Two Fleets Of Vehicles Based On Equations 2.18, 2.19, 2.30, And 2.31

Empty vehicle speed = 40 mph
Loaded vehicle speed = 30 mph
Acceleration = 19759 mph^2
Exponential headway distribution

<table>
<thead>
<tr>
<th>Ave. Sample Size</th>
<th>Distance Of Samples</th>
<th>Traffic Turnouts</th>
<th>Ave. Traffic Flow Rate</th>
<th>Ave. S.D. ( F \times 10^{-3} )</th>
<th>Expect Rejection Hypothesis</th>
<th>Hypothesis</th>
<th>οL = 0.05</th>
</tr>
</thead>
<tbody>
<tr>
<td>9718</td>
<td>4</td>
<td>0.3</td>
<td>10</td>
<td>0.8842</td>
<td>1.03</td>
<td>0.8873</td>
<td>No^2</td>
</tr>
<tr>
<td>24150</td>
<td>5</td>
<td>0.3</td>
<td>20</td>
<td>0.8757</td>
<td>1.24</td>
<td>0.8873</td>
<td>No^2</td>
</tr>
<tr>
<td>37174</td>
<td>5</td>
<td>0.3</td>
<td>60</td>
<td>0.8483</td>
<td>1.70</td>
<td>0.8165</td>
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<td>35390</td>
<td>5</td>
<td>0.3</td>
<td>100</td>
<td>0.9098</td>
<td>4.57</td>
<td>0.7054</td>
<td>Yes</td>
</tr>
</tbody>
</table>

1 Standard deviation

2 Hypothesis was not rejected for a 10% level of significance

3 Hypothesis was not rejected for a 1% level of significance
5.0 THE COST VARIABLES AND THE MODIFICATION OF THE EXPECTED F-FACTOR

5.1 THE COST OF TRUCK TRANSPORTATION

The cost equations developed in Chapter 2 require an estimate of trucking costs. This cost involves such cost elements as the wages of the operator, vehicle licences, permits, insurance, depreciation, repair, maintenance, fuel, and oil. An estimate of these cost elements can be obtained from past records, production manuals, appraisal manuals, equipment rental manuals, and technical reports.

The Journal of Logging Management periodically publishes equipment rental rates prepared by the Truck Loggers Association, Vancouver, B.C. (Journal Of Logging Management, 1978). Their calculations include depreciation (straight line), interest (simple 15 percent), insurance ($1.30 per $100.00 of investment), repair and maintenance (55 percent of depreciation), tires, lubrication, wages of the operator (International Woodworkers of America wage plus a cost of living provision plus 30 percent payroll loading), overhead (10 percent of total), and profit (10 percent of total). The rental rate is based on a ten-hour day but there are no provisions for fuel costs. Rental rates are calculated for two truck classes (twelve and fifteen foot bunks) and three coastal zones.

The United States Bureau of Land Management produces a manual to predict the truck hauling costs for Oregon and
Washington (United States Bureau of Land Management, 1977). The cost predicted for a White truck (Model 4964) and Peerless trailer includes a fixed cost ($6.59 per hour), an operating cost ($9.32 per hour), a driver's wage ($11.62 per hour) and an overhead cost ($2.75 per hour).

Smith and Tse (1977) subdivided trucking costs into in-use costs and travelling costs. In-use costs include fixed costs (depreciation, interest, opportunity charges, and insurance) and the wage of the driver while the travelling costs include such items as fuel, oil, tires, repairs, and maintenance. Smith tabulated the hauling costs for three truck classes (eight, ten, and twelve foot bunks).

Table VII summarizes the predicted hauling costs from these three sources. The results of these articles cannot be directly compared since the costs apply to three distinct operating areas (north central interior of B.C., coastal B.C., and northwestern United States). Consequently, a cost element from one area may be higher or lower than that cost element for the other two areas. These costs are given not as a hard and fast rule but as a guideline for the cost elements that should be included in the calculation of the adjusted hauling rate.
### Table VII—Truck Hauling Costs

<table>
<thead>
<tr>
<th>Source</th>
<th>Zone</th>
<th>Bunk Size (feet)</th>
<th>Straight Time ($)</th>
<th>Overtime Time ($)</th>
<th>Delay Time ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Smith and Tse (1977)</td>
<td>8</td>
<td>24.19</td>
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</tr>
<tr>
<td></td>
<td>10</td>
<td>27.12</td>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td>12</td>
<td>32.24</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Journal of Logging Management (1978)</td>
<td>12</td>
<td>34.55</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A²</td>
<td>37.05</td>
<td></td>
<td></td>
<td></td>
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<td>B²</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>Basic</td>
<td>45.45</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A</td>
<td>48.64</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>52.05</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>United States Bureau of Land Management (1977)</td>
<td></td>
<td>30.28</td>
<td>32.26</td>
<td>20.03</td>
<td></td>
</tr>
</tbody>
</table>

1. Based on average hourly cost between 'small' log and 'large' log loads. 'Large' sawlogs had a minimum butt diameter of fourteen inches.

2. The zones refer to isolation areas where:
   - A = up coast areas with poor access
   - B = isolated parts of Dean and Rivers PSYU's and Queen Charlottes etc.

3. These are the rental rates without including the ten percent profit margin.
5.2 THE TURNOUT CONSTRUCTION AND MAINTENANCE COSTS

Road construction costs can be obtained from company records, manuals, and various publications but seldom do they categorize turnout construction costs. Since turnouts are usually located where the road can be relatively easily widened the actual turnout construction cost is somewhat less than that anticipated for a comparable stretch of road.

Sauder and Nagy (1977) estimated the road construction costs for three road classes (B.C. Forest Service - 3, 4a, 4b). The calculations included the cost estimation of the subgrade construction (cable shovel, bulldozer and drill), the ballasting (from rock quarries and gravel pits), and the surfacing.

The B.C. Forest Service Districts have developed appraisal manuals (B.C. Forest Service (1975), B.C. Forest Service (1977)) to estimate road construction costs under a variety of conditions (terrain, soils, road classes). These manuals can be used to calculate the cost of a station of road and then apply a portion of this figure to the turnout construction cost.

The United States Bureau of Land Management (1977) determined the turnout excavation costs for two classes of turnouts (a fifty-foot turnout plus two twenty-five-foot approaches and a one hundred-foot turnout plus two fifty-foot approaches) under a variety of soil conditions and side slope classes. The total turnout construction cost included the excavation cost plus the grading and surfacing costs. The
results of their costing study are shown in Table VIII.

Turnout maintenance costs can be estimated from road maintenance costs. The cost per unit surface area is not equivalent since the turnouts would experience less wear than the main road surface. The B.C. Forest Service (1977) estimated the machine rates, production rates, and maintenance schedule for mainline and spur roads. Table IX illustrates the projected turnout maintenance costs based on these figures. These figures must be altered to reflect the different deterioration rates between the turnout surface and the roadway surface.
# Table VIII---Excavation Costs Per Turnout*

## 14-foot Subgrade (10-foot Usable Width)

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
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<tbody>
<tr>
<td>Slope</td>
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<td></td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>$ 7.10</td>
<td>1.3</td>
<td>28</td>
<td>$ 50.70</td>
<td>1.3</td>
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<td>10</td>
<td>7.10</td>
<td>1.3</td>
<td>28</td>
<td>50.70</td>
<td>1.3</td>
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<tr>
<td></td>
<td>20</td>
<td>8.15</td>
<td>2.0</td>
<td>32</td>
<td>103.25</td>
<td>2.0</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>12.45</td>
<td>2.7</td>
<td>49</td>
<td>197.00</td>
<td>2.8</td>
<td>101</td>
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<tr>
<td></td>
<td>40</td>
<td>13.45</td>
<td>3.5</td>
<td>53</td>
<td>138.45</td>
<td>3.5</td>
<td>71</td>
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<td></td>
<td>50</td>
<td>21.85</td>
<td>4.7</td>
<td>86</td>
<td>206.70</td>
<td>4.7</td>
<td>106</td>
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<tr>
<td></td>
<td>60</td>
<td>79.00</td>
<td>8.0</td>
<td>311</td>
<td>461.70</td>
<td>8.0</td>
<td>255</td>
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<tr>
<td></td>
<td>70</td>
<td>171.20</td>
<td>12.0</td>
<td>674</td>
<td>970.70</td>
<td>12.0</td>
<td>509</td>
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<tr>
<td></td>
<td>80</td>
<td>208.80</td>
<td>13.2</td>
<td>822</td>
<td>1146.60</td>
<td>13.8</td>
<td>588</td>
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<tr>
<td></td>
<td>90</td>
<td>262.15</td>
<td>14.8</td>
<td>1032</td>
<td>1368.90</td>
<td>15.0</td>
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<td></td>
<td>100</td>
<td>316.25</td>
<td>17.0</td>
<td>1245</td>
<td>1618.50</td>
<td>17.0</td>
<td>830</td>
</tr>
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</table>

1 Standard lengths: 50-foot turnout plus two 25-foot approaches

## 20-foot Subgrade (12-foot Usable Width)

<table>
<thead>
<tr>
<th></th>
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<tr>
<td>Slope</td>
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</tr>
<tr>
<td></td>
<td>0</td>
<td>$ 19.55</td>
<td>1.7</td>
<td>77</td>
<td>$ 243.75</td>
<td>1.0</td>
<td>125</td>
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<td>10</td>
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<td>243.75</td>
<td>1.0</td>
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<td>40</td>
<td>52.60</td>
<td>4.0</td>
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<td>466.05</td>
<td>4.0</td>
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<td>50</td>
<td>54.10</td>
<td>5.7</td>
<td>213</td>
<td>407.55</td>
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<td>10.1</td>
<td>1059</td>
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<td>10.1</td>
<td>872</td>
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<tr>
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<td>70</td>
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<td>2560.35</td>
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<td>3120.00</td>
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<td>90</td>
<td>674.35</td>
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<td>2655</td>
<td>3502.20</td>
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<td>100</td>
<td>794.50</td>
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<td>3128</td>
<td>4093.05</td>
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</tr>
</tbody>
</table>

2 Standard lengths: 100-foot turnout plus two 50-foot approaches

* From United States Bureau of Land Management (1977)
Table IX--Turnout Maintenance Costs

<table>
<thead>
<tr>
<th>Machine (Grader)</th>
<th>Rate per Shift</th>
<th>Road Class</th>
<th>Maintenance Schedule in days³</th>
<th>Annual Maintenance Cost per Turnout 2 miles/shift</th>
<th>3 miles/shift</th>
<th>4 miles/shift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cat 12</td>
<td>$227</td>
<td>Mainline</td>
<td>20</td>
<td>$11,448</td>
<td>$7,632</td>
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<td></td>
<td></td>
<td>Spur</td>
<td>40</td>
<td>3,816</td>
<td>2,862</td>
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<tr>
<td>Cat 14</td>
<td>277</td>
<td>Mainline</td>
<td>20</td>
<td>13,970</td>
<td>9,313</td>
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<td></td>
<td></td>
<td>Spur</td>
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<td>4,657</td>
<td>3,492</td>
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<td>Mainline</td>
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<td>16,844</td>
<td>11,229</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Spur</td>
<td>40</td>
<td>5,615</td>
<td>4,211</td>
<td></td>
</tr>
</tbody>
</table>

¹ Assume a road width of sixteen feet and a turnout area of seven hundred fifty square feet. Calculations based on four turnouts per mile, two hundred operating days per year and eighty-five percent availability.

² Based on B.C. Forest Service (1977)

³ These costs can be readily adjusted to determine the costs for various maintenance periods.
5.3 MODIFICATION OF THE EXPECTED F-FACTOR EQUATIONS

The expected F-factor equations, developed in Chapter 2, assume there is no minimum waiting time in a turnout but in reality there is a minimum delay period. In traffic analysis the merging of traffic from a ramp into the main traffic stream may be classified as an ideal or forced merge. An ideal merge does not cause the main traffic stream to reduce speed or change lanes to allow the merging vehicle to enter the main traffic stream but a forced merge results in one of these phenomena to occur. A parallel can be drawn with regards to a logging truck utilizing a turnout where an ideal pull-in will not cause the loaded vehicle to reduce speed to allow the empty vehicle to utilize the turnout but a forced pull-in will cause the loaded vehicle to reduce speed. In the calculation of the F-factor it has been assumed the vehicles travel at a constant speed except when the empty vehicle decelerates into a turnout and accelerates from a turnout. The operator of the empty vehicle may be able to increase his speed over a section of road so as to change the classification of a pull-in from being a forced pull-in to an ideal pull-in. Provided the operator of the empty vehicle knows the location of the loaded vehicles a modification of the expected F-factor should only apply to a forced pull-in.

The model does not account for the length of the turnout. If the length of the turnout is accounted for then there is a section of road where there are two lanes such that the empty vehicle and loaded vehicle can pass each other. This though
would not significantly alter the classification of a pull-in.

In concluding, if the operator does not know the location of the other vehicles then a modification of the expected $F$-factor equation may be required. However, in this study it will be assumed the operator knows the location of the other vehicles.
6.0 THE USE AND TESTING OF THE MODEL

6.1 THE USE OF THE MODEL

The solution of the optimum turnout spacing model results from the evaluation of a complex set of equations and the utilization of a search technique. This solution can be used as a guideline for determining turnout spacing and can be used as an aid in determining when the road should be switched from being single-lane to double-lane. Furthermore, a section of the model, equation 2.8, can be utilized to determine the total expected delay time per vehicle per unit distance of road.

Figures 8, 9, 10, and 11 illustrate the effect the independent variables, traffic flow, turnout spacing, velocity of the loaded vehicles, and velocity of the empty vehicles, have on the total expected delay time per vehicle per unit distance of road. It is assumed the rate of acceleration is 11855 mph², the rate of deceleration is 19759 mph², and the headway frequency distribution is exponential. The formulas to determine the rates of acceleration and deceleration are developed in Appendix 3. Unless it is stated otherwise, it is assumed the velocity of the loaded vehicle is 20 mph, the velocity of the empty vehicle is 25 mph, the turnout spacing is 0.1 miles, and the traffic flow rate is 4 vph. Furthermore, the total expected delay time (T) is expressed as a percentage of the total travel empty time.
Figure 8--Effect of traffic flow rate on the total expected delay time for various turnout spacings.
Figure 9--Effect of the turnout spacing on the total expected delay time for various traffic flow rates
Figure 10—Effect of the velocity of the empty vehicle on the total expected delay time for various velocities of the loaded vehicle
Figure 11---Effect of the velocity of the loaded vehicle on the total expected delay time for various velocities of the empty vehicle
Figure 8 illustrates the effect of the traffic flow rate (1 to 20 vph) on the total expected delay time for various turnout spacings (0.05 to 0.35 miles). The total expected delay time (%) increases as the traffic flow rate and the turnout spacing increase, though the marginal effect decreases as the traffic flow rate increases. For most log hauling situations, where the traffic flow rate is less than 5 vph, the total expected delay time is less than 20 percent of the travel empty time. Figure 9 illustrates the effect that the turnout spacing has on the total expected delay time for various traffic flow rates. The total expected delay time (%) increases and the marginal effect decreases as the turnout spacing increases. The total expected delay time is less than 15 percent of the travel empty time provided the traffic flow rate is less than 5 vph and there is a minimum of 5 turnouts per mile.

Figure 10 illustrates the effect that the velocity of the empty vehicle (15 to 45 mph) has on the total expected delay time for various velocities of the loaded vehicle (10 to 40 mph). The total expected delay time (%) and the marginal effect increase as the velocity of the empty vehicle increases. The total expected delay time is less than 20 percent of the travel empty time. Figure 11 depicts the effect that the velocity of the loaded vehicle (10 to 35 mph) has on the total expected delay time for various velocities of the empty vehicle (20 to 40 mph). The total expected delay time and the absolute value of the first derivative of the function decrease as the velocity of the loaded vehicle increases. Based on these four figures, the general trends that an increase or decrease in one
of the independent variables have on the total expected delay time can be predicted.

Since the cost function is complex and a search technique is required to locate the optimum turnout spacing, it is advantageous to develop a computer program to determine the optimum turnout spacing. Appendix 8 documents a computer program that can be utilized to determine the optimum turnout spacing. Besides determining the optimum solution, the program calculates the costs, based on equation 2.38, for the optimum turnout spacing, 50 percent of the optimum turnout spacing, 100 percent of the optimum turnout spacing, and 200 percent of the optimum turnout spacing. A further discussion on the sensitivity of the optimum solution to fluctuations in the equation's variables is located in section 6.2.

Figures 12, 13, 14, 15, 16, and 17 illustrate the effect that the independent variables, the velocity of the loaded vehicle, the velocity of the empty vehicle, the traffic flow rate, the turnout construction cost, the adjusted truck hauling cost, and the expected useful life of the road, have on the optimum turnout spacing. In the calculation of the optimum turnout spacing it is assumed the rate of acceleration is $19759 \text{ mph}^2$, the rate of deceleration is $11855 \text{ mph}^2$, the number of "conflict" hours per day is 5, the number of operating days per year is 200, and the headway frequency distribution is exponential. Unless it is stated otherwise, it is assumed the velocity of the loaded vehicle is 25 mph, the velocity of the empty vehicle is 40 mph, the traffic flow rate is 4 vph, the turnout construction cost is $100, the adjusted truck hauling cost is $15 per hour, and the expected useful life of the road
is 20 years.

Figure 12 depicts the effect that the velocity of the loaded vehicle (10 to 45 mph) has on the optimum turnout spacing for various velocities of the empty vehicle (25 to 40 mph). The optimum turnout spacing increases but the marginal effect decreases as the velocity of the loaded vehicle increases. For any given velocity of the loaded vehicle, the optimum turnout spacing increases as the velocity of the empty vehicle increases.

Figure 13 illustrates the effect that the velocity of the empty vehicle (14 to 45 mph) has on the optimum turnout spacing for various values of the velocity of the loaded vehicle (10 to 35 mph). The same general relationships between the optimum turnout spacing and the independent variables, as shown in Figure 12, are evident in Figure 13.

The effect that the traffic flow rate (1 to 20 vph) has on the optimum turnout spacing is illustrated in Figure 14. The optimum turnout spacing decreases as the traffic flow rate increases. The general shape of the curve appears to be hyperbolic and asymptotic to the x-axis and y-axis.

Figure 15 depicts the effect the turnout construction cost ($50 to $1000) has on the optimum turnout spacing. The optimum turnout spacing increases and the marginal effect decreases as the turnout construction cost increases.

The effect that the adjusted hauling cost ($1 to $45 per hour) has on the optimum turnout spacing is illustrated in Figure 16. The general shape of the curve appears to be hyperbolic. The optimum turnout spacing decreases as the adjusted hauling cost increases.
Figure 17 illustrates the effect that the expected useful life of the road (1 to 25 years) has on the optimum turnout spacing. The optimum turnout spacing decreases as the expected useful life of the road increases. The general shape of the curve can be depicted by a hyperbolic function. Based on the six figures, the general effect that changes to one of the independent variables has on the optimum turnout spacing can be predicted.

Figure 18 illustrates the cost function, based on equation 2.38. In this diagram it is assumed:

1. $a_A = 19759 \text{ mph}^2$
2. $a_D = 19759 \text{ mph}^2$
3. 5 "conflict" hours per day
4. 200 operating days per year
5. $L = 50$ feet
6. the driver's reaction time is 2 seconds
7. $H = 4$ vph
8. $V_1 = 20$ mph
9. $V_2 = 25$ mph
10. $C = $100
11. $M = $15 per hour
12. $Q_1 = 20$ years
13. exponential headway frequency distribution.

The cost function curve begins relatively steeply but flattens as the optimum turnout spacing is approached. The portion to the right of the optimum turnout spacing is not as steep as the portion to the left of the optimum. A further discussion of the effect that perturbations to the turnout spacing has on
the cost equation is discussed in section 6.2.13.
Figure 12---Effect of the velocity of the loaded vehicle on the optimum turnout spacing for various velocities of the empty vehicle.
Figure 13---Effect of the velocity of the empty vehicle on the optimum turnout spacing for various velocities of the loaded vehicle
Figure 14---Effect of the traffic flow rate on the optimum turnout spacing
Figure 15---Effect of the turnout construction cost on the optimum turnout spacing
Figure 16---Effect of the adjusted truck hauling cost on the optimum turnout spacing
Figure 17---Effect of the expected useful life of the road on the optimum turnout spacing
Figure 18---Effect of the turnout spacing on the cost function based on equation 2.38
6.2 Sensitivity Analysis

The previous chapters have outlined a method to calculate the optimum turnout spacing but the formulas utilized assume that there are no errors in the independent variables of the formulas. Generally, the so-called "true" values of the variables are unknown and estimates of these values are required. A measure of the stability of the functions with respect to variations in the value of the variables is desirable. This section will include a sensitivity analysis of the optimum turnout spacing function, expected F-factor function, total expected delay time function, and cost function with respect to the independent variables, some of the assumptions, and headway distributions.

The general format of this section is to discuss the effect that the independent variables, the assumptions of the model, and the headway distributions have on the functions. Simulation is the basic modelling tool used in the sensitivity analysis since the equations are generally too complex to allow for a direct comparison method. Discrete, deterministic simulation models were written in BASIC for the HP9830A desktop calculator to solve the expected F-factor, the total expected delay time, the optimum turnout spacing, and the cost function for a wide range of values of the independent variables. The basic method is to loop the independent variables between a lower and an upper limit in constant or variable increments. The independent variable being tested is looped in a similar manner but the function is also evaluated for perturbed values.
of the independent variable. The difference between the actual and the perturbed values of the function are calculated. Once the experience of the simulation is assembled into groups, the average, the standard deviation, and the maximum value of each group are determined. Graphs of some of the results of the simulations are located in Appendix 9.
6.2.1 VELOCITY OF THE LOADED VEHICLE

There is a potential error between the estimated and true value of the average velocity of the loaded vehicle. It will be assumed that the estimated value will be within 10 mph of the actual value. Therefore the sensitivity analysis will be based on a measurement error of 10 mph or less.

It can be shown that for the simple F-factor the rate of change in the expected F-factor ($\delta \bar{F}$) for a given perturbation in the velocity of the loaded vehicle ($\delta V_L$) is:

$$\delta \bar{F} = \frac{1}{2V_L^2}$$

This formula can be utilized as an estimation of the rate of change for the expected F-factor involving a single empty vehicle meeting a fleet of loaded vehicles. This formula, though, does not account for the total expected delay time error. Since the total expected delay time function is complex a simulation model is utilized to determine the percentage difference in the total expected delay time ($\delta \bar{T}$) for various perturbations in the velocity of the loaded vehicle. The results of the simulation, as illustrated in Figure 19, show that the error may be large but it coincides with a large error in the value of the velocity of the loaded vehicle. The assumptions, the independent variables, and the range of values of these variables are:

1. The headway frequency distribution is exponential
2. $a_A = a_D = 19759 \text{ mph}^2$
3. $S = 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, \text{ or } 0.35 \text{ miles}$
4. $V_L = 10, 20, 30, \text{ or } 40 \text{ mph}$
5. \( V_2 = 10, 20, 30, \) or 40 mph
6. \( H = 2, 6, \) or 10 vph.

The velocities of the vehicles are further constrained by:

\[
V_1 \leq V_2 + 10 \text{ mph} \\
V_2 \leq V_1 + 20 \text{ mph}.
\]

As illustrated in Figure 20, there is a smaller percent difference in the optimum turnout spacing \((\delta S^*)\) than in the initial percent perturbation in \(V_1\). The difference between the percentage values, \(\delta V_1 - \delta S^*\), decreases as the perturbation of \(V_1\) is decreased. As was the case in the previous simulation the same independent variables and their range of values are utilized in this simulation. The extra variables required are:

1. \( Q_1 = 2 \) or 10 years
2. \( C = $100, \$600, \) or \$1000
3. \( M = $25 \) or \$45 per hour
4. 200 operating days per year
5. 5 "conflict" hours per day
6. The stopping distance is less than one-half of the turnout spacing.

By themselves large perturbations in \(V_1\) appear to be significant, but the effect that these perturbations have on the cost equations is the real question. This relationship is illustrated in Figure 21. The values of the independent variables are the same as for the optimum turnout spacing simulation. The cost difference is less than $50 per vehicle per year per mile for values of \(V_1\) greater than 20 mph. Velocity perturbations based on a true velocity of 10 mph generally result in a cost difference of less than $100 per vehicle per year per mile. This value transformed into dollars
per cunit, based on 25 cunits per trip, results in a cost difference of less than $0.01 per cunit-mile.
Figure 19---Effect of perturbations to the velocity of the loaded vehicle on the total expected delay time
Figure 20.—Effect of perturbations to the velocity of the loaded vehicle on the optimum turnout spacing.
Figure 21---Effect of perturbations to the velocity of the loaded vehicle on the cost difference
6.2.2 VELOCITY OF THE EMPTY VEHICLE

It will be assumed that the potential maximum error in the measurement of the velocity of the empty vehicle will be the same as the maximum error for the velocity of the loaded vehicle, 10 mph. The rate of change in the simple F-factor for a given perturbation in the velocity of the empty vehicle ($\delta V_2$) is:

$$\delta \bar{F} = -V_1/(2V_2^2)$$

This rate of change formula can be utilized to predict the potential rate of change for the expected F-factor involving the meeting of an empty vehicle and a fleet of loaded vehicles. Simulations to predict the error in the calculation of the total expected delay time ($\delta \bar{T}$), the optimum turnout spacing ($\delta S^*$), and the cost equation ($\delta C_r$) were conducted in a similar manner as in the case of the velocity of the loaded vehicle. The results of these simulations were similar to the case involving perturbations to the velocity of the loaded vehicle:

1. the percentage difference in the total expected delay time and the percentage difference in the optimum turnout spacing were slightly lower than in the case involving the velocity of the loaded vehicle
2. the cost difference was slightly higher than in the case involving the velocity of the loaded vehicle (Appendix 9).
In the sensitivity analysis of the traffic flow rate \( (H) \) the perturbations that were used are 1, 2, and 4 vph. The functions \( F, T, S^*, \) and \( C_T \) are tested for actual traffic flow rates of 1, 2, 4, 6, 8, and 10 vph. The rest of the independent variables and their range of values are the same as for the simulations involving the sensitivity analysis of the velocity of the loaded vehicle. The expected F-factor resulting from the meeting of one empty vehicle and a fleet of loaded vehicles experienced a change of less than 0.01, for any of the tested flow rates. This difference is insignificant. The percent error in the calculation of the total expected delay time \( (\delta \bar{T}) \) can be approximated by:

\[
\delta \bar{T} = (100) \frac{\delta H}{H}
\]

As can be shown from this approximation formula the error created can be large. The results of the simulation to determine the percent difference in the optimum turnout spacing have small standard deviations. If the perturbation is less than 100 percent of the actual flow rate then the maximum value of \( \delta S^* \) will be approximately 70 percent of the actual optimum turnout spacing. Maintaining this as a maximum perturbation then the cost difference function \( (\delta C_T) \) has a larger maximum value than that experienced in the case of the sensitivity analysis of either \( V_1 \) or \( V_2 \) (based on a maximum perturbation of 100 percent). The maximum value of \( \delta C_T \), based on a 100-percent perturbation of the traffic flow rate, is approximately $400 per vehicle per mile per year but the average value is
approximately $85.
6.2.4 EXPECTED USEFUL LIFE OF THE ROAD

The expected useful life of the road \( (Q_i) \) is analysed for 5, 10, 15, 20, and 25 years with perturbations \( (\delta Q_i) \) of 2.5, 5, and 10 years. The remaining independent variables in the simulation models utilize the same range of values as the simulations to test the sensitivity of the velocity of the loaded vehicle. The results of the simulation involving the percent difference in the optimum turnout spacing have small standard deviations for the various groupings of \( Q_i \) and \( \delta Q_i \). As \( \delta Q_i \) increases \( \delta s^* \) increases. Provided \( Q_i \) can be predicted to within 100 percent of the actual value then the maximum value of \( \delta s^* \) is approximately 35 percent of the actual optimum turnout spacing. This maximum value decreases to 21 percent for a perturbation of 50 percent and 12 percent for a perturbation of 25 percent. The results of the cost difference simulation had a larger coefficient of variation than the results of the \( \delta s^* \) simulation. By maintaining the assumption that the maximum error in the estimation of \( Q_i \) is 100 percent then the maximum cost difference is approximately $35 per year per vehicle per mile. If the perturbation is 50 percent then the maximum value of \( \delta C_T \) is $20. The accuracy of the estimation of \( Q_i \) is not as critical as in the situations involving the velocity of the loaded vehicle, the velocity of the empty vehicle, nor the traffic flow rate, since for a given percent perturbation in the expected useful life of the road the maximum value of \( \delta C_T \) is less.
6.2.5 TURNOUT CONSTRUCTION COST

The sensitivity analysis of the turnout construction cost included costs of $100, $250, $500, and $1000 with perturbations of $50, $100, and $200. The \( S^* \) simulation produced similar results as the model involving the sensitivity analysis of the expected useful life of the road. A 100-percent perturbation in C resulted in \( S^* \) being approximately 34 percent of the actual optimum turnout spacing while a 40-percent perturbation resulted in \( S^* \) being approximately 17 percent. The simulation to determine \( C' \) produced results that are not as significant as the previous \( C' \) simulations; i.e., velocity of the loaded vehicle, velocity of the empty vehicle, traffic flow rate, and expected useful life of the road. A 100-percent perturbation of C results in a cost difference of less than $30 per year per vehicle per mile.
The simulations to determine the difference in the optimum turnout spacing and the cost difference were conducted with adjusted truck hauling costs of $10, $15, $25, $35, and $45 per hour and with perturbations of $2.5, $5, and $10 per hour. Both of these simulations utilized the same basic assumptions and range of values of the variables as the simulations involving the analysis of the velocity of the loaded vehicle. There was a small standard deviation within the various groups, M and SM, of the simulation to determine $\delta S^*$. For a given percent perturbation of the independent variable, the resulting $\delta S^* (\%)$ value is less in the case involving the adjusted truck hauling cost than in the case involving any of the previously analysed variables. A 100-percent perturbation in the adjusted hauling cost results in $\delta S^*$ being approximately 34 percent of the actual optimum turnout spacing while a 50-percent perturbation results in $\delta S^*$ being approximately 20 percent. The results of the simulations to determine the cost difference had a larger coefficient of variation than the simulation to determine the percent difference in the optimum turnout spacing. In the case involving an adjusted hauling cost of $10 per hour and a 100-percent perturbation, the maximum cost difference was approximately $40 per vehicle per mile per year while the average difference was approximately $14. The effect of a perturbation to the adjusted hauling cost on the cost is not as significant as in the case involving the perturbation of the turnout construction cost.
6.2.7 THE ACCELERATION AND DECELERATION OF THE EMPTY VEHICLE

Perturbations to the rate of deceleration of the empty vehicle will not significantly affect the expected F-factor, provided the perturbations can be predicted to within 50 percent of the actual rate of deceleration. A 50-percent perturbation will cause the maximum error in the expected F-factor to be less than 0.01 or 20 percent of the actual F-factor. This summary is based on a simulation model that utilized the same range of values of the variables as was utilized in the analysis of the velocity of the loaded vehicle.

A simulation model was utilized to determine the effect that perturbations to either the rate of acceleration or deceleration have on the total expected delay time. The assumptions, constraints, and range of values of the independent variables utilized in the program were:

1. \( V_1 = 10, 15, 20, 25, 30, 35, \) or 40 mph
2. \( V_2 = 10, 15, 20, 25, 30, 35, \) or 40 mph
3. \( V_1 \leq V_2 + 10 \) mph
4. \( V_2 \leq V_1 + 20 \) mph
5. \( S = 0.05, 0.10, 0.15, 0.20, 0.25, 0.30, \) or 0.35 miles
6. \( H = 2, 4, 6, 8, \) or 10 vph
7. \( a_A \) or \( a_D = 14555, 24259, \) or 33963 mph\(^2\)
8. exponential headway distribution.

The rate of acceleration or deceleration being tested was assumed to have an actual rate of 14555, 19407, 24259, 29111, or 33963 mph\(^2\) while the perturbations were 2426, 4852, and 9704 mph\(^2\). The development of formulas to determine the rate of
acceleration and deceleration are outlined in Appendix 3. The simulation results were grouped according to their respective rates of acceleration or deceleration and their perturbations. A 66-percent perturbation will cause the maximum value of $\delta T$ to be less than 20 percent of the travel empty time. The simulation indicated that the percent change in the total expected delay time will be less than one-half of the perturbation, expressed in percent.

A simulation model was developed to examine the effect that perturbations to the rate of acceleration or deceleration have on the optimum turnout spacing and cost functions. The range of the values of the variables and the assumptions that were utilized in the model were essentially the same as those utilized in the sensitivity analysis of the velocity of the loaded vehicle (Section 6.2.1). It was assumed that the rate of acceleration equals the rate of deceleration, 14555 or 24259 mph$^2$. The perturbations were 2426, 4852, and 9704 mph$^2$ and the maximum perturbation was 80 percent. Since the maximum value of $\delta S^*$ was less than 7 percent of the actual optimum turnout spacing the effect that perturbations to the rate of acceleration or deceleration have on the optimum turnout spacing function is insignificant.
6.2.8 THE DISCOUNT RATE

A simulation model was developed to determine the effect that the discount rate has on the optimum turnout spacing function and the cost function. The simulation results were grouped with respect to their corresponding discount rate (2.5, 5.0, 7.5, 10.0, or 12.5 percent per annum) and expected useful life of the road (2, 5, 10, or 20 years). The range of values of the variables and the assumptions utilized in the simulation were:

1. \( V_1 = 10, 20, 30, \) or 40 mph
2. \( V_2 = 10, 20, 30, \) or 40 mph
3. \( V_2 \leq V_1 + 20 \) mph
4. \( V_1 \leq V_2 + 10 \) mph
5. \( H = 3 \) or 10 vph
6. \( C = $100, $600, \) or $1100
7. \( M = $25 \) or $45 per hour
8. \( a_A = a_0 = 19759 \text{ mph}^2 \)
9. exponential headway frequency distribution
10. 200 operating days per year
11. 5 "conflict" hours per day.

The marginal effect of \( \delta S^* \) (%) decreased as the expected useful life of the road increased or as the discount rate increased (Appendix 9). The maximum value of \( \delta S^* \) was approximately 50 percent of the optimum turnout spacing which corresponded to a situation involving a discount rate of 12.5 percent and an expected useful life of the road of 20 years. When the discount rate was decreased to 7.5 percent the maximum value of
\( S^* \) became approximately 34 percent.

The results of the cost difference (\( \delta C_T \)) simulation had a larger coefficient of variation than the \( S^* \) simulation. The marginal effect of \( \delta C_T \), for maximum and average values, decreased as the discount rate or the expected useful life of the road increased. The maximum value of \( \delta C_T \) was only \$35 per vehicle per mile per year which was based on a discount rate of 12.5 percent and an expected useful life of the road of 20 years. For lower discount rates (i.e., 5 percent) the effect of the discount rate on the cost function is not very significant.
6.2.9 THE MAINTENANCE COST

A simulation model was developed to determine the effect that maintenance costs have on the cost function and optimum turnout spacing function. The results of the simulation were grouped with respect to their corresponding maintenance costs ($0.05, $1.00, $2.50, $5.00, $7.50, $10.00, or $12.50 per annum) and expected useful life of the road (2, 5, 10, 15, 20, or 25 years) before the maximum and average values were determined. The values of the independent variables and the assumptions of the model were the same as those utilized in the analysis of the velocity of the loaded vehicle (Section 6.2.1). The marginal effect of \( \delta S^* \) (\%), for various maintenance costs, decreased as the expected useful life of the road increased. Since the coefficient of variation for \( \delta S^* \) was approximately one, for each group, there was a large spread in the values of \( \delta S^* \). To have a maximum value of \( \delta S^* \) equal to approximately 50 percent of the actual optimum turnout spacing, a maintenance cost of $7.50 per annum is required over a 25-year period. This same maximum value is achieved for a maintenance cost of $10 over 20 years or $12.50 over 15 years (Appendix 9).

The grouped results of the cost difference had a larger coefficient of variation than the \( \delta S^* \) simulation. When \( \delta S^* \) is less than 50 percent then the value of \( \delta C_T \) is less than $300 per year per mile or approximately $0.02 per cunit (based on 25 cunits per load over a 10-mile haul). Based on a maintenance cost of $10 per annum the value of \( \delta C_T \) increases to approximately $0.03 per cunit when \( \delta S^* \) is increased to
approximately 60 percent. In the case of a maintenance cost of $5 per annum over a 20 year period the maximum value of $\delta C_r$ would be approximately $0.01 per cunit.
6.2.10 The Derivative of the Expected F-Factor

In the determination of the optimum turnout spacing, the first derivative of the cost function with respect to the turnout spacing must be equal to zero. Since the expected F-factor is a variable in the cost function, the first derivative of the expected F-factor is included in the optimum turnout spacing function. A simulation model is used to determine the effect that the first derivative of the expected F-factor equation has on the optimum turnout spacing function and the resulting cost function. The same values of the independent variables were used in this simulation, as were utilized in section 6.2.1 except that:

1. $h = 1, 2, 4, 6, 8, \text{ or } 10 \text{ vph}$
2. The shifted exponential and Pearson Type III ($a=2$) headway distributions were used.

For a traffic flow rate of less than 4 vph, the value of $\delta s^*$ was 20 percent of the optimum turnout spacing. The maximum value became approximately 51 percent when the traffic flow rate was increased to 10 vph. The corresponding maximum cost difference was only $27 per year per mile per vehicle. Generally, the inclusion of the first derivative of the expected F-factor in the calculation of the optimum turnout spacing is insignificant.
6.2.11 THE LENGTH OF THE LOADED VEHICLE

Throughout the development of the model it has been assumed that the loaded vehicle has no length. Some of the equations developed in Chapter 2 can be modified to include the effect of the length of the loaded vehicles (Appendix 6). Furthermore, the appendix includes a summary of the results of a simulation program written to verify the accuracy of the expected F-factor equations which account for the length of the loaded vehicle. This simulation determined the average F-factor for delay situations involving the shifted exponential and the Pearson Type III(a=2) headway distributions.

A simulation model was developed to compare the exponential to the shifted exponential F-factor equation and the Erlang(alpha=2) to the Pearson Type III(a=2) F-factor equation. The simulation assumed:

1. \( L = 60 \) feet
2. \( a_0 = 19759 \) mph²
3. \( V_1 = 10, 20, 30, \) or 40 mph
4. \( V_2 = 10, 20, 30, \) or 40 mph
5. \( V_2 \leq V_1 + 20 \) mph
6. \( V_1 \leq V_2 + 10 \) mph
7. \( S \leq V_2^2 / a_0 \)
8. \( S = 0.10, 0.15, 0.20, 0.25, 0.30, \) or 0.35 miles
9. \( H = 1, 2, 4, 6, 8, \) or 10 vph

The results of the simulation indicated that the effect of the length of the vehicle on the expected F-factor was approximately:
\[ \delta \overline{F} = \frac{L}{S} \]

where:

\( L \) = effective length of the vehicle.

For a turnout spacing of 0.1 miles the difference in the expected F-factor is approximately 0.11.

The effect of the length of the vehicle on the optimum turnout spacing proved to be inconsequential. In the simulation utilized to determine this conclusion it was assumed:

1. \( Q_1 = 2 \) or 10 years
2. \( M = $25 \) or $45 per hour
3. \( C = $100, $600, \) or $1100
4. 200 operating days per year
5. 5 "conflict" hours per day.

The maximum value of \( \delta S^* \), for the case involving the shifted exponential headway distribution, was less than 2 percent of the actual optimum turnout spacing while for the case involving the Pearson Type III \((a=2)\) headway distribution the maximum value of \( \delta S^* \) was less than 5 percent. Consequently, the length of the vehicle will not significantly affect the optimum turnout spacing nor the cost equation.
6.2.12 The Headway Probability Distributions

A simulation model was developed to compare the expected F-factors developed from various headway probability distributions. The range of values of the independent variables were essentially the same as for the simulation involving the effect of perturbations to the velocity of the loaded vehicles on the expected F-factors (Section 6.2.1). The exceptions were that the length of the loaded vehicle was 60 feet and the traffic flow rate was 1, 2, 3, 4, 6, or 10 vph. The simulation compared the simple, exponential, Erlang (alpha=2), shifted exponential, and Pearson Type III (a=2) F-factor equations. The maximum difference between the simple F-factor and the exponential F-factor was 0.0527 at a traffic flow rate of 10 vph. This value dropped to 0.0054 when the rate was decrease to 4 vph. A larger maximum difference occurred between the simple F-factor and the Erlang (alpha=2) F-factor, the values being 0.091 for a rate of 10 vph and 0.027 for a rate of 4 vph. The maximum difference between the exponential and Erlang (alpha=2) F-factors was the smallest of those tested. This difference was of the order of magnitude of $10^{-3}$. By incorporating the effect of the length of the vehicle into the expected F-factor equations there was essentially no difference in the results.

A similar simulation model was developed to determine the effect that various headway distributions have on the optimum turnout spacing function and the cost function. The values of the variables utilized in the simulation were essentially the
same as those used in the analysis of the velocity of the loaded vehicles except that the traffic flow rate was 1, 2, 4, 6, or 10 vph. The results were grouped with respect to their F-factors, simple, exponential, and Erlang(alpha=2), before being compared. The maximum value of $\delta S^*$ was less than 3 percent of the actual optimum turnout spacing and the maximum value of $\delta C_r$ was approximately $11 per vehicle per mile per year. Consequently, the type of headway distribution utilized will not significantly alter the optimum turnout spacing nor cost equation.
6.2.13 THE TURNOUT SPACING AND THE OPTIMUM TURNOUT SPACING

There is potentially a small error in the measurement of the average turnout spacing. A simulation model was developed to determine the effect that perturbations to the turnout spacing (0.05, 0.10, and 0.20 miles) have on the total expected delay time. The perturbations were added to the true values of the turnout spacing (0.05, 0.10, 0.15, 0.20, 0.25, 0.30, and 0.35 miles). The range of values of the independent variables utilized in the simulation was essentially the same as those utilized in section 6.2.1. Graphs were constructed to illustrate the effect that perturbations to the turnout spacing have on the total expected delay time (Appendix 9). The total expected delay time and the absolute value of the first derivative of the function decrease as the turnout spacing increases. Furthermore, the total expected delay time increases as the size of the perturbation increases. If it is assumed that the maximum perturbation of the turnout spacing is 20 percent then the maximum difference in the total delay time is approximately 30 percent of the travel empty time.

A simulation model was developed to illustrate the effect that deviations from the optimum turnout spacing (25% to 300%) have on the cost equation. The range of values of the independent variables was similar to those utilized in section 6.2.1 except:

1. headway frequency distribution was the shifted exponential
2. \( L = 60 \) feet
3. 25 cunits per trip
4. M = $5, $10, $15, or $20 per hour
5. Q = 5, 15, or 25 years
6. H = 1, 4, or 8 vph.

The cost difference was measured in dollars per cunit-mile. The experience of the model, averages, standard deviations, and maximum values, was recorded with respect to the traffic flow rate, the adjusted truck hauling cost, and the expected useful life of the road. The values of the cost difference function, in dollars per cunit-mile, did not vary significantly with respect to the traffic flow rate.

Figures 22 and 23 depict the effect that deviations from the optimum turnout spacing have on the average and maximum values of the cost equation with respect to the expected useful life of the road and the adjusted truck hauling cost. The traffic flow rate was set equal to 1 vph. The cost difference and the marginal effect increased as the perturbations to the optimum turnout spacing increased. Furthermore, the cost difference increased as the adjusted hauling cost increased but decreased as the expected useful life of the road increased. The coefficient of variations of the grouped results are stable. The average coefficient of variation was 0.68 while the maximum value was 0.72. For each of the groups, the maximum cost difference was approximately three times the size of the average cost difference while the maximum cost difference is approximately four and one-half times the value of the standard deviation of the cost difference. The maximum value of the cost difference, given an adjusted truck hauling cost of $20 per hour, an expected useful life of the road of 5 years, and a 100-percent perturbation, is approximately $0.007
per cunit-mile while the average value is approximately $0.003 per cunit-mile.
Figure 22: Effect of deviations from the optimum turnout spacing on the maximum cost difference

TRUCK HAULING RATES

- 3.00/HR
- 10.00/HR
- 15.00/HR
- 20.00/HR

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)

COST DIFFERENCE IN DOLLARS PER CUNIT-MILE TIMES 0.0001

OPTIMUM SPACING PLUS % OF THE OPTIMUM SPACING

0 50 100 150 200 250 300
0 42 84 126 168 210 252 336 378 420
Figure 23—Effect of deviations from the optimum turnout spacing on the average cost difference.
7.0 DISCUSSION AND CONCLUSIONS

7.1 DISCUSSION

A model has been developed to determine the optimum turnout spacing which is a function of velocities of the vehicles, traffic flow rate, expected useful life of the road, acceleration rate, deceleration rate, turnout construction cost, turnout maintenance cost, and adjusted hauling cost. The results of the model should be utilized as a guide and not as the absolute spacing of the turnouts. The road network, company policy, and terrain will tend to dictate the actual location of the turnouts.

Figures 12, 13, 14, 15, 16, and 17 illustrate the relationships between the independent variables and the optimum turnout spacing. The optimum turnout spacing increased with increasing turnout construction cost, velocity of the empty vehicle, and velocity of the loaded vehicle. Optimum turnout spacing decreased as the expected useful life of the road, traffic flow rate, and adjusted hauling cost increased. Based on these figures, an overestimation of the independent variables will not cause as dramatic a change in the optimum turnout spacing as an underestimation of the independent variables. The absolute value of the slope of the curves decrease as the value of the independent variable increases. Consequently, there is not as dramatic an affect to the optimum turnout spacing, provided the independent variable is overestimated rather than underestimated. This though, does
not imply that the values of the independent variables should be overestimated.

A sensitivity analysis showed that a given percent perturbation to each of the independent variables will affect the optimum turnout spacing differently. Generally, the traffic flow rate was the most sensitive variable followed by velocity of the loaded vehicle and velocity of the empty vehicle. The turnout construction cost, expected useful life of the road, and adjusted hauling cost caused about the same change to the optimum turnout spacing. These variables were not as sensitive as the traffic flow rate, velocity of the empty vehicle, and velocity of the loaded vehicle. These sensitivity relationships cannot be compared unless the potential accuracy of the estimation of the variables is considered. Generally the traffic flow rate, speed of the vehicles, and expected useful life of the road are easier to predict than the turnout construction cost and adjusted hauling cost. Consequently, the confidence in the initial estimation of the variables must be considered. Generally, the optimum turnout spacing was not significantly affected by the discount rate, headway frequency distribution assumptions, rate of acceleration, rate of deceleration, and the length of the vehicle. The inclusion of the maintenance cost in the determination of the optimum turnout spacing can be significant (i.e., a 50 percent change in the optimum turnout spacing for a maintenance cost of $7.50 per annum over a 25-year span).

The total expected delay time can be a significant part of the travel empty time (Figures 8, 9, 10, and 11). This significance is dramatically reduced when the total cycle time
is considered. If the travel empty time is one-quarter of the total time then for a given $T$ of 20 percent of the travel empty time the corresponding effect of $T$ on the total round trip time is 5 percent. By comparing the total expected delay time to the total cycle time rather than to the travel empty time the significance of the delay time is reduced fourfold.

Figures 22 and 23 depict the effect that deviations from the optimum turnout spacing have on the cost (dollars per cunit-mile). Road construction and log hauling costs are highly variable within and between different regions. Sauder and Nagy (1977) have shown that the cost of road construction and hauling is of the order of magnitude of $10 per cunit, based on a haul distance of approximately 13 miles. This cost estimation is in the order of $1 per cunit-mile. Based on a maximum deviation from the optimum turnout spacing of 100 percent (Figure 22), the maximum potential savings are about $0.01 per cunit-mile, $0.13 per cunit, or about 1 percent of the total road construction and hauling cost. These calculations, though, can be misleading. A small percentage of a large number may be significant. The total potential savings must be considered rather than the potential savings per cunit-mile. Based on a traffic flow rate of 4 vph, a 10-mile haul, 25 cunits per trip, 5-year time period, 200 working days per year, and 5 "conflict" hours per day, the total potential savings are $50000. Consequently, the potential savings become important. For a situation with a traffic flow rate of 4 vph, an 100-mile haul, 15-year time period, 200 working days per year, 5 "conflict" hours per day, velocity of the loaded vehicle of 20 mph, velocity of the empty vehicle of 40 mph, an
adjusted hauling cost of $25 per hour, and a turnout construction and maintenance cost of $100 per turnout the optimum turnout spacing is 510 feet. The total potential savings, based on a 100 percent deviation from the optimum turnout spacing is $68,190.

The implementing of the optimum turnout spacing model can be achieved with the utilization of tables. A table approach neglects the necessity of the repeated evaluation of a complex set of equations. Consequently, the tables can be incorporated into the road standards.

Future areas of research that can be investigated are:

1. Develop a method to predict the adjusted hauling cost, turnout construction cost, and turnout maintenance cost.

2. Incorporate the effect that loaded logging trucks have on the travel time of vehicles other than logging trucks.

3. Develop a model that involves the interaction between fleets of vehicles, non-uniform turnout spacing, daily variation in traffic flow rates, and variation in velocities of vehicles.
7.2 CONCLUSIONS

A model was developed to allow an engineer to determine optimum turnout spacing for specific conditions and determine effect of turnout spacing on hauling cost. Simulation models were used to test the sensitivity of optimum turnout spacing to perturbations of the independent variables. Generally, optimum turnout spacing was most sensitive to the traffic flow rate followed by the speeds of the loaded and empty vehicles. Optimum turnout spacing was less sensitive to turnout construction cost, expected useful life of the road, and adjusted hauling cost. The results from a simulation revealed that a 100 percent increase of the traffic flow rate will cause a maximum decrease of 70 percent of the optimum turnout spacing. Much of this relationship is due to the effect of increasing traffic flow rate on total expected delay time. The simulations determined that for realistic conditions (e.g., 5 vph and 0.3 miles between turnouts) the expected delay time was 20 percent of the travel empty time. The percent deviation in the total expected delay time was also found to vary significantly (approximately proportionally) with the percent deviation in the traffic flow rate.

Based on the concept of uniform spacing of turnouts, the potential savings of utilizing the model as a guide in the design of the road network can be determined. For example, if an engineer uses the optimum turnout spacing rather than a 100 percent deviation from the optimum, then the resulting potential savings can be as high as 1 percent of the
transportation cost. In the situation cited in Section 7.1 this 1 percent potential savings represents approximately $10000 per year over a 10-mile haul at 4 vph. Since off-road haul distances can easily approach 30 miles and wood can be hauled over several such routes simultaneously, potential savings through optimum turnout spacing could be in the order of $90000 per year.

The concept of the expected F-factor was developed and utilized to estimate the expected delay of a truck in a turnout. This is a measure of the expected separation distance between the loaded vehicle and the empty vehicle as a proportion of the turnout spacing when the empty vehicle has come to a complete halt in the turnout. Previous articles have not completely defined a method of deriving or measuring this delay. Two forms of the expected F-factor equation were developed:

1. \( \bar{F} = \frac{(V_1 + V_2)}{(2V_2)} \) : for one empty vehicle meeting one loaded vehicle

2. \( \bar{F} = \Pr(h>h_c)\bar{F}_1 + \Pr(h<h_c)\bar{F}_2 \) : for one empty vehicle meeting a fleet

These equations were tested by simulation and found to be realistic. The simulations showed that the interaction between a fleet of empty vehicles and a fleet of loaded vehicles was adequately described by the expected F-factor equation representing the meeting of an empty vehicle and a fleet of loaded vehicles.

The expected F-factor equation for one vehicle meeting a fleet requires the use of a headway distribution function. Two sets of interarrival time data were analysed to determine if
the headway distribution of logging trucks fits a known probability distribution. The frequency histogram of the headways was left skewed but was found to follow neither an exponential nor Erlang distribution. A simulation model showed that the type of headway distribution utilized, exponential or Erlang \( (\kappa = 2) \), did not significantly alter the expected F-factor. Since the data appeared to follow a distribution similar to the Erlang, and the F-factor is relatively insensitive to the type of headway distribution, it is concluded that the derived F-factor will adequately model a realistic hauling situation.

Previous authors have written papers concerned with determining the number of lanes required for log transportation. They did not calculate or derive an expression to determine the expected turnout delay time. Since this is now available, the topic of determining the number of lanes required for log transportation should be reinvestigated. A method should be developed to predict the adjusted hauling cost, turnout construction cost, and turnout maintenance cost. The effect that loaded logging trucks have on the travel time of vehicles other than logging trucks should also be investigated.


APPENDICES
APPENDIX 1  ROAD STANDARDS SURVEY
Dear Sir:

I am a graduate student at the University of British Columbia and currently doing a study on road standards. The objective of this project is to evaluate the design elements used in the determination of road standards and road classification. It would be appreciated if you could forward the road specifications your company employs in the construction of its forest roads. Some of the design elements under consideration could be:

1. Road types (i.e. Main, secondary, branch and spur)
2. Design speed
3. Minimum sight distance (horizontal, vertical)
4. Curve radius
5. Adverse and favourable grade
6. Subgrade width
7. Surface material
8. Ditch width and depth (rock or soil)
9. Maximum surfacing depth
10. Right-of-way width
11. Turnouts (length, width and number per mile)
12. Culverts (type)
13. Compaction (equipment utilized and degree of compaction)
14. Use of roads (winter, summer or year-round)

Thank you for your co-operation.

Yours truly,

Dennis I. Anderson

DIA/Graduate Studies
<table>
<thead>
<tr>
<th>Specifications</th>
<th>British Columbia (Total=16)</th>
<th>Rest of Canada (Total=30)</th>
<th>United States (Total=15)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Speed</td>
<td>12 (75)</td>
<td>23 (77)</td>
<td>8 (53)</td>
</tr>
<tr>
<td>Right-of-way</td>
<td>10 (63)</td>
<td>30 (100)</td>
<td>9 (60)</td>
</tr>
<tr>
<td>Subgrade width</td>
<td>11 (69)</td>
<td>21 (70)</td>
<td>11 (73)</td>
</tr>
<tr>
<td>Surface width</td>
<td>14 (87)</td>
<td>29 (97)</td>
<td>14 (93)</td>
</tr>
<tr>
<td>Surface depth</td>
<td>2 (13)</td>
<td>21 (70)</td>
<td>11 (73)</td>
</tr>
<tr>
<td>Road Gradient</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- Favourable</td>
<td>13 (81)</td>
<td>23 (77)</td>
<td>13 (87)</td>
</tr>
<tr>
<td>- Adverse</td>
<td>13 (81)</td>
<td>23 (77)</td>
<td>13 (87)</td>
</tr>
<tr>
<td>Curve radius</td>
<td>13 (81)</td>
<td>19 (63)</td>
<td>11 (73)</td>
</tr>
<tr>
<td>Sight distance</td>
<td>4 (25)</td>
<td>6 (20)</td>
<td>4 (27)</td>
</tr>
<tr>
<td>- Horizontal</td>
<td>3 (19)</td>
<td>16 (53)</td>
<td>3 (20)</td>
</tr>
<tr>
<td>- Vertical</td>
<td>1 (6)</td>
<td>1 (3)</td>
<td>1 (7)</td>
</tr>
<tr>
<td>Ditch depth</td>
<td>12 (75)</td>
<td>18 (60)</td>
<td>12 (80)</td>
</tr>
<tr>
<td>width</td>
<td>6 (37)</td>
<td>10 (33)</td>
<td>10 (67)</td>
</tr>
<tr>
<td>Turnouts length</td>
<td>8 (50)</td>
<td>3 (10)</td>
<td>9 (60)</td>
</tr>
<tr>
<td>width</td>
<td>6 (37)</td>
<td>3 (10)</td>
<td>8 (53)</td>
</tr>
<tr>
<td>frequency</td>
<td>9 (56)</td>
<td>5 (17)</td>
<td>9 (60)</td>
</tr>
<tr>
<td>Back slope</td>
<td>3 (19)</td>
<td>12 (40)</td>
<td>6 (40)</td>
</tr>
<tr>
<td>Fill slope</td>
<td>3 (19)</td>
<td>9 (30)</td>
<td>6 (40)</td>
</tr>
<tr>
<td>Cross slope</td>
<td>2 (13)</td>
<td>4 (13)</td>
<td>3 (20)</td>
</tr>
<tr>
<td>Load capacity</td>
<td>5 (31)</td>
<td>10 (33)</td>
<td>1 (7)</td>
</tr>
<tr>
<td>Survey response</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>- sample size</td>
<td>27</td>
<td>53</td>
<td>66</td>
</tr>
<tr>
<td>- response</td>
<td>19 (70)</td>
<td>30 (57)</td>
<td>28 (42)</td>
</tr>
</tbody>
</table>

* Parenthesized values represent percentages of:
1. the affirmative response, or
2. the sample size in the cases involving the "survey response"
APPENDIX 2 ABBREVIATIONS, SYMBOLS, AND UNITS

I ROMAN SYMBOLS AND ABBREVIATIONS

\[ a, a_A, a_D \] acceleration and deceleration where subscripts \( A \) and \( D \) refer to acceleration and deceleration

\[ C_C \] turnout construction cost

\[ C_M \] turnout maintenance cost

\[ C_T \] total cost

\[ D, D_A, D_S \] distance where subscripts \( A \) and \( S \) refer to stopping distance and acceleration distance

\[ D_I \] interval distance, the distance between two sets of empty vehicles after the loaded vehicle has passed both groups of empty vehicles and each vehicle has accelerated to its original speed

\[ d \] number of "conflict" hours per day

\[ F, F_L \] the F-factor, which is the distance the loaded vehicle is from the empty vehicle, once the empty vehicle has come to a complete halt in the turnout, divided by the turnout spacing. The subscript \( L \) refers to the F-factor when one accounts for the length of the vehicle

\[ \bar{F} \] the expected F-factor

\[ \bar{F}_1 \] the expected F-factor for headways greater than the critical headway

\[ \bar{F}_2 \] the expected F-factor for headways less than the critical headway

\[ \bar{F}_3 \] the expected F-factor for a Case 3 turnout delay situation
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{\text{MAX}}$</td>
<td>the maximum $F$-factor resulting from the interaction between one empty vehicle and one loaded vehicle</td>
</tr>
<tr>
<td>$f$</td>
<td>coefficient of friction</td>
</tr>
<tr>
<td>$G$</td>
<td>road gradient, incline, or grade</td>
</tr>
<tr>
<td>$g$</td>
<td>acceleration of gravity</td>
</tr>
<tr>
<td>GPSSV</td>
<td>General Purpose Simulation System V</td>
</tr>
<tr>
<td>$H$</td>
<td>traffic flow rate, the number of vehicles passing a point per time unit</td>
</tr>
<tr>
<td>$h$</td>
<td>headway, which is the time interval between successive vehicles measured from front to front</td>
</tr>
<tr>
<td>$h_1, h_2$</td>
<td>vehicle headway where the subscripts 1 and 2 refer to constrained and free flow</td>
</tr>
<tr>
<td>$h_{c}, h_{cL}$</td>
<td>critical headway where the subscript CL refers to the inclusion of vehicle length in the calculation of the critical headway</td>
</tr>
<tr>
<td>$K$</td>
<td>$2a_A a_D / (a_A + a_D)$</td>
</tr>
<tr>
<td>$L$</td>
<td>the length of the loaded vehicle</td>
</tr>
<tr>
<td>$M$</td>
<td>adjusted hauling cost (i.e., dollars per hour)</td>
</tr>
<tr>
<td>$n$</td>
<td>the number of expected delays while the empty vehicle travels a unit distance of road</td>
</tr>
<tr>
<td>$P_W$</td>
<td>present worth</td>
</tr>
<tr>
<td>$Q_1$</td>
<td>&quot;the expected useful life of the road&quot;</td>
</tr>
<tr>
<td>$Q_3$</td>
<td>the number of headways during the &quot;conflict&quot; hours</td>
</tr>
<tr>
<td>$R_T$</td>
<td>the minimum time gap between the back of the first vehicle and the front of the second vehicle</td>
</tr>
<tr>
<td>$S, y(S)$</td>
<td>distance between turnouts</td>
</tr>
<tr>
<td>$S_c$</td>
<td>the critical turnout</td>
</tr>
<tr>
<td>$S_j$</td>
<td>turnout locations</td>
</tr>
</tbody>
</table>
$T, T_A, T_S$  time where the subscripts $A$ and $S$ refer to the acceleration time and the stopping time

$ar{T}$  total expected delay time per unit distance per empty vehicle

t  time or headway

$V_1$  velocity of loaded vehicle

$V_2$  velocity of empty vehicle

$W$  gross weight of the vehicle

$X_{CF}$  critical distance

$Z_2$  a present worth function

II Other Symbols

$\alpha, \lambda$  parameters of the Erlang headway distribution

$\tau$  minimum headway

$\chi^2$  chi-square value

$e$  base of the natural logarithms
APPENDIX 3  ACCELERATION AND DECELERATION OF A VEHICLE

The braking force required to stop a vehicle involves the weight of the vehicle \( W \), the road gradient or incline \( \Theta \), and the coefficient of friction \( f \) between the tires and the road surface. In this instance it will be assumed there is a constant road gradient, a constant coefficient of friction, and a braking force measured along the plane of the incline. By equating these conditions along the plane of the incline the braking force equation can be obtained:

\[
\text{Braking force} = Wa_i/g = Wf\cos\Theta - W\sin\Theta
\]

where:

\( a_i \) = deceleration (feet per second per second)
\( g \) = acceleration of gravity.

If angle \( \Theta \) is relatively small then \( \sin\Theta \) is approximately equal to \( \tan\Theta \) and \( \cos\Theta \) approaches one. The equation can be rewritten as:

\[
a_i = g(f-\tan\Theta)
\]

or

\[
a_D = 79036 (f-G/100)
\]

where:

\( a_D \) = deceleration (mph\(^2\)).

To account for adverse and favourable gradients the equation can be adjusted to:

\[
a_D = 79036 (f\pm G/100)
\]

If \( \Theta \) is relatively large then:

\[
a_D = 79036 (f\cos\Theta \pm \sin\Theta)
\]

\[
a_D = 79036 (100f\pm G)/(G^2+100^2)^{0.5}
\]
where:

$D^h_s$ = horizontal stopping distance

$D^r_s$ = incline stopping distance

$F_p$ = frictional force

$N$ = normal force

$W$ = weight of the vehicle

$\Theta$ = angle of the incline

The next step is to determine the error between equations A.1 and A.2. A variable epsilon ($\epsilon$) can be introduced such that:

$$\frac{(100f\pm G)}{(G^2+100^2)^{0.5}} = (f\pm G/100)(1+\epsilon)$$

or

$$0 = \epsilon^2 (G^2+100^2) + 2\epsilon(G^2+100^2) + G^2$$

Solutions for epsilon are illustrated in the table below.

In reality the coefficient of friction is proportional to speed. Therefore, a coefficient of friction should be chosen such that the stopping distance or time is correct (See table below).
If the braking distance is defined as being along the horizontal plane then:

\[ D_s^H = D_s^I \cos \theta \]

where:

\[ D_s^H = \text{horizontal braking distance} \]
\[ D_s^I = \text{incline braking distance} \]

the deceleration equation becomes:

\[ a_D = 79036 (f \pm g/100) \]

and epsilon equals zero.

The acceleration of a vehicle is limited by the horsepower of the vehicle and the forms of resistance to vehicle movement: rolling, air, gradient, engine, and inertia (Matson et al. (1955) and Byrne et al. (1947)). Matson et al. (1955) showed that the residual horsepower available for acceleration is:

\[ HP_A = HP_T - HP_{ROLL} - HP_{AIR} - HP_{GRADE} \]
\[ = HP_T - (WV \{R_R \pm 20G\} + 0.0026K_i AV^2) / 375 \]

where:

\[ A \quad = \text{frontal area of vehicle in square feet} \]
\[ HP_A \quad = \text{residual horsepower} \]
\[ HP_{AIR} \quad = \text{power to overcome air resistance} \]
\[ HP_{GRADE} \quad = \text{power to overcome gradient resistance} \]
\[ HP_{ROLL} \quad = \text{power to overcome rolling resistance} \]
\[ HP_T \quad = \text{total horsepower available} \]
\[ K_i \quad = \text{streamlining factor} \]
\[ R_R \quad = \text{rolling resistance in pounds/ton} \]
\[ W \quad = \text{vehicle weight in tons} \]

and the potential acceleration is:
\[ a_A = \frac{(14819 \ H P_A)}{(W \ V_g)} \]

The above formula calculates the potential acceleration but seldom is the potential acceleration equivalent to the actual acceleration. Consequently, the acceleration formula should be modified to account for this deviation. If the acceleration time or distance is known then the acceleration can be easily determined from basic dynamics, equations 2.3 or 2.4.

Similarly, the stopping distance or time tables developed in this appendix can be utilized, provided the coefficient of friction becomes a coefficient of traction. In the development of the model the table approach will be utilized in favour of the potential acceleration formula approach. Consequently, the tables are used in a similar manner as for the case involving deceleration.
<table>
<thead>
<tr>
<th>Road Gradient (%)</th>
<th>Epsilon</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.000049604</td>
</tr>
<tr>
<td>2</td>
<td>0.000199725</td>
</tr>
<tr>
<td>3</td>
<td>0.000449010</td>
</tr>
<tr>
<td>4</td>
<td>0.000798332</td>
</tr>
<tr>
<td>5</td>
<td>0.001245908</td>
</tr>
<tr>
<td>6</td>
<td>0.001791597</td>
</tr>
<tr>
<td>7</td>
<td>0.002434749</td>
</tr>
<tr>
<td>8</td>
<td>0.003174604</td>
</tr>
<tr>
<td>9</td>
<td>0.004009128</td>
</tr>
<tr>
<td>10</td>
<td>0.004938118</td>
</tr>
<tr>
<td>11</td>
<td>0.005959723</td>
</tr>
<tr>
<td>12</td>
<td>0.007072758</td>
</tr>
<tr>
<td>13</td>
<td>0.008274801</td>
</tr>
<tr>
<td>14</td>
<td>0.009565637</td>
</tr>
<tr>
<td>15</td>
<td>0.010942270</td>
</tr>
<tr>
<td>16</td>
<td>0.012403550</td>
</tr>
<tr>
<td>17</td>
<td>0.013946550</td>
</tr>
<tr>
<td>18</td>
<td>0.015570130</td>
</tr>
<tr>
<td>19</td>
<td>0.017271420</td>
</tr>
<tr>
<td>20</td>
<td>0.019049160</td>
</tr>
<tr>
<td>21</td>
<td>0.020899980</td>
</tr>
<tr>
<td>22</td>
<td>0.022822160</td>
</tr>
<tr>
<td>23</td>
<td>0.024812970</td>
</tr>
<tr>
<td>24</td>
<td>0.026870420</td>
</tr>
<tr>
<td>25</td>
<td>0.028991170</td>
</tr>
</tbody>
</table>
### Table Of Stopping Times--Seconds

<table>
<thead>
<tr>
<th>Velocity (mph)</th>
<th>Coefficient Of Friction Plus Or Minus Road Gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>4.6</td>
</tr>
<tr>
<td>10</td>
<td>9.1</td>
</tr>
<tr>
<td>15</td>
<td>13.7</td>
</tr>
<tr>
<td>20</td>
<td>18.2</td>
</tr>
<tr>
<td>25</td>
<td>22.8</td>
</tr>
<tr>
<td>30</td>
<td>27.3</td>
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<td>35</td>
<td>31.9</td>
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<tr>
<td>40</td>
<td>36.4</td>
</tr>
<tr>
<td>45</td>
<td>41.0</td>
</tr>
<tr>
<td>50</td>
<td>45.5</td>
</tr>
<tr>
<td>55</td>
<td>50.1</td>
</tr>
<tr>
<td>60</td>
<td>54.7</td>
</tr>
</tbody>
</table>
Table Of Stopping Distances—Feet

<table>
<thead>
<tr>
<th>Velocity (mph)</th>
<th>Coefficient Of Friction Plus Or Minus Road Gradient</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.05</td>
</tr>
<tr>
<td>5</td>
<td>17.</td>
</tr>
<tr>
<td>10</td>
<td>67.</td>
</tr>
<tr>
<td>15</td>
<td>150.</td>
</tr>
<tr>
<td>20</td>
<td>267.</td>
</tr>
<tr>
<td>25</td>
<td>418.</td>
</tr>
<tr>
<td>30</td>
<td>601.</td>
</tr>
<tr>
<td>35</td>
<td>818.</td>
</tr>
<tr>
<td>50</td>
<td>1670.</td>
</tr>
<tr>
<td>55</td>
<td>2021.</td>
</tr>
<tr>
<td>60</td>
<td>2405.</td>
</tr>
</tbody>
</table>
APPENDIX 4  ANALYSIS OF HEADWAY DISTRIBUTIONS

The analysis of headway distributions involved data from two studies, a coastal study (Canadian Forest Products Limited) and a northern interior study (Northwood Pulp and Timber Limited). The analysis involved the determination of the observed frequency, the calculation of the traffic flow rate, and $\chi^2$ goodness of fit tests. The coastal study used frequency classes of five minutes while the interior study used frequency classes of one and five minutes. The goodness of fit tests employed the use of two computer programs, U.B.C. FREQ and a program written in BASIC for the HP9830A calculator.

A brief documentation of the BASIC program follows. With the use of "REM" statements in the program the model is easily explained. The basic sections of the program are:

1. Input of the lower cell limit, the cell interval, and the number of cells
2. Calculation of the cell boundaries
3. Input of the parameters of the distribution to be tested
4. Determining observed frequency
5. Determining expected frequency
6. Regrouping of the cells such that there is a minimum of five expected values in any cell
7. Calculation of the chi-square value.
**Program Variables**

<table>
<thead>
<tr>
<th>Variable</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Chi-square value</td>
</tr>
<tr>
<td>C</td>
<td>frequency of the observed and expected data</td>
</tr>
<tr>
<td>D</td>
<td>frequency of observed data</td>
</tr>
<tr>
<td>E</td>
<td>frequency of expected values</td>
</tr>
<tr>
<td>F</td>
<td>cell boundaries</td>
</tr>
<tr>
<td>N</td>
<td>initial number of cells</td>
</tr>
<tr>
<td>N1</td>
<td>cell width</td>
</tr>
<tr>
<td>N2</td>
<td>initial value</td>
</tr>
<tr>
<td>N3</td>
<td>number of cells after regrouping</td>
</tr>
<tr>
<td>V1</td>
<td>total number of vehicles</td>
</tr>
<tr>
<td>V2</td>
<td>vehicles per hour</td>
</tr>
<tr>
<td>V3</td>
<td>number of hauling hours per day</td>
</tr>
</tbody>
</table>
Program Listing

5 REM          GOODNESS OF FIT TEST
10 DIM F(80),C(2,80),Q$(40),X(2,80),D(80),B$(1),E(80)
15 REM          FREQUENCY OF OBSERVED DATA
20 DATA
30 DATA
60 DISP "INPUT DATE";
70 INPUT Q$
80 PRINT LIN2,Q$,
90 DISP "INPUT PROBABILITY DISTRIBUTION";
100 INPUT Q$
110 A=0
120 MAT C=ZER
130 DISP "INPUT NUMBER OF CLASSES & INTERVAL";
140 INPUT N,N1
150 DISP "INPUT INITIAL VALUE";
160 INPUT N2
165 REM          CALCULATION OF CELL BOUNDARIES
170 FOR I=0 TO N-1
180 F(I+1)=I*N1+N2
190 NEXT I
200 F(N)=200
210 DISP "INPUT # VEHICLES, VEH./HR., DAY LENGTH";
220 INPUT V1,V2,V3
230 DISP "IS DATA TO BE ENTERED";
240 INPUT B$
245 REM DETERMINING THE FREQUENCY OF THE OBSERVED DATA
250 FOR J=1 TO N
260 IF B$="Y" THEN 300
270 C(1,J)=D(J)
280 GO TO 310
290 DISP "INPUT OBSERVED VALUE" J;
300 INPUT C(1,J)
310 NEXT J
320 C1=C3=0
325 REM DETERMINING THE FREQUENCY OF THE EXPECTED VALUES
330 FOR J=1 TO N
335 REM THE PROBABILITY FUNCTION
340 C(2,J)=V1*(1-(1+2*F(J)*V2/60)*EXP(-2*F(J)*V2/60))
350 C2=C(2,J)
360 C(2,J)=C(2,J)-C1
370 C1=C2
380 C3=C3+C(2,J)
390 NEXT J
400 PRINT LIN3,"THE DISTRIBUTION IS "Q$,LIN3
410 PRINT "NUMBER OF CLASSES = "N
420 PRINT "CLASS INTERVAL = "N1
430 PRINT "INITIAL VALUE = "N2
440 PRINT "TOTAL NUMBER OF VEHICLES = "V1
450 PRINT "VEHICLES PER HOUR = "V2
460 PRINT "LENGTH OF DAY = "V3
470 PRINT LIN2
480 FOR I=1 TO N
490 PRINT J;C(1,J);C(2,J)
500 NEXT J
510 PRINT LIN2
515 REM GOODNESS OF FIT TEST
560 MAT D=ZER
565 MAT E=ZER
570 N3=1
575 REM REGROUPING OF THE CELLS SUCH THAT THERE IS
576 REM A MINIMUM FREQUENCY OF FIVE EXPECTED VALUES
577 REM IN EACH CELL
580 FOR I=1 TO N
590 D(N3)=D(N3)+C(2,I)
595 E(N3)=E(N3)+C(1,I)
600 IF D(N3)<5 THEN 620
610 N3=N3+1
620 NEXT I
630 IF D(N3) >= 5 THEN 660
640 N3=N3-1
645 E(N3)=E(N3)+E(N3+1)
650 D(N3)=D(N3)+D(N3+1)
660 FOR I=1 TO N3
670 C(2,I)=D(I)
675 C(1,I)=E(I)
680 NEXT I
690 A=0
695 REM CALCULATION OF THE CHI-SQUARE VALUE
700 FOR I=1 TO N3
710 A=A+(C(1,I)-C(2,I))^2/C(2,I)
720 NEXT I
730 PRINT LIN1
740 PRINT"CELL NUMBER	OBSERVED	EXPECTED"LIN1
750 FOR J=1 TO N3
760 PRINT J;C(1,J);C(2,J)
770 NEXT J
780 PRINT LIN1
790 PRINT "THE CHI-SQUARE VALUE IS "A
800 PRINT LIN2
810 END
### Coastal Study

**Observed Headway Frequency Distribution**

<table>
<thead>
<tr>
<th>Class Mark (minutes)</th>
<th>Frequency</th>
<th>Class Mark (minutes)</th>
<th>Frequency</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.25</td>
<td>2</td>
<td>65</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>16</td>
<td>70</td>
<td>1</td>
</tr>
<tr>
<td>10</td>
<td>60</td>
<td>75</td>
<td>0</td>
</tr>
<tr>
<td>15</td>
<td>31</td>
<td>80</td>
<td>0</td>
</tr>
<tr>
<td>20</td>
<td>17</td>
<td>85</td>
<td>2</td>
</tr>
<tr>
<td>25</td>
<td>9</td>
<td>90</td>
<td>0</td>
</tr>
<tr>
<td>30</td>
<td>14</td>
<td>95</td>
<td>0</td>
</tr>
<tr>
<td>35</td>
<td>7</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
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<td>5</td>
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<td><strong>193</strong></td>
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Traffic Flow Rate

flow* : 201 vehicles/83.4 hours = 2.41 vehicles/hour
or 193 vehicles/83.4 hours = 2.32 vehicles/hour

* There was a total of 201 vehicles observed over the 8-day period but there were only 193 headway observations. The actual flow rate lies between the two calculated values.
### Goodness Of Fit Tests—Results

<table>
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<tr>
<th>Probability Dist.</th>
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<th>LCL</th>
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<th>$\gamma$</th>
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<tr>
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<td>28.3</td>
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</tbody>
</table>

* The parenthesized number is the final number of frequency classes while the non-parenthesized number is the number of frequency classes.

+ The number of degrees of freedom

• Lower class limit
**Interior Study - First Weigh Scale**

**Observed Headway Frequency Distribution**

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<th>Class Mark (min)</th>
<th>Freq.</th>
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<tr>
<td>21</td>
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<td>43</td>
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<td>2</td>
<td>Total 661</td>
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</tr>
</tbody>
</table>

**Traffic Flow Rate**

\[
\text{flow}^* = 681 \text{ vehicles} / 183.5 \text{ hours} = 3.71 \text{ vehicles/hour}
\]

or

\[
661 \text{ vehicles} / 183.5 \text{ hours} = 3.60 \text{ vehicles/hour}
\]

*There was a total of 681 vehicles observed but there were only 661 headway observations. The actual flow rate lies between the calculated values.*
# Goodness Of Fit Tests—Results

<table>
<thead>
<tr>
<th>Probability Dist.</th>
<th>Program</th>
<th>Number*</th>
<th>Class Width (min)</th>
<th>LCL - Value (min)</th>
<th>Computed $\chi^2$</th>
<th>$\nu$</th>
<th>$\chi^2$</th>
<th>Table $\chi^2$</th>
<th>$\alpha&lt;0.005$</th>
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<td>+500</td>
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<td>FREQ</td>
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<td>+500</td>
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</tbody>
</table>

* The parenthesized value is the final number of frequency classes while the non-parenthesized number is the initial number of classes

+ The number of degrees of freedom

• Lower class limit
Interior Study - Both Weigh Scales

Observed Headway Frequency Distribution

<table>
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<th>Class Mark</th>
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<td>65</td>
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<td>Total 775</td>
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</tr>
</tbody>
</table>

Traffic Flow Rate

flow*: 814 vehicles/252.2 hours = 3.23 vehicles/hour

or 775 vehicles/252.2 hours = 3.07 vehicles/hour

* There was a total of 814 vehicles observed but there were only 775 headway observations. The actual flow rate lies between the calculated values.
### Goodness Of Fit Tests--Results

<table>
<thead>
<tr>
<th>Probability Dist.</th>
<th>Program</th>
<th>Number</th>
<th>Class LCL</th>
<th>Value (min)</th>
<th>( \chi^2 )</th>
<th>( \nu )</th>
<th>( \chi^2 ) for ( \nu = 0.005 )</th>
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<td>1</td>
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<tr>
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<td>33.7</td>
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<td>32.8</td>
</tr>
<tr>
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<td>29.4</td>
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<td>5.5</td>
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<td>14</td>
<td>31.3</td>
</tr>
</tbody>
</table>

* The parenthesized value is the final number of frequency classes while the non-parenthesized number is the initial number of classes

+ The number of degrees of freedom

• Lower class limit
APPENDIX 5  SIMULATION OF THE F-FACTOR

The purpose of the simulation models was to test the validity and determine the limitations of the expected F-factor equations, equations 2.12, 2.18, and 2.19. The programs that were developed consisted of the interaction between two single vehicles, the meeting of a single empty vehicle and a fleet of loaded vehicles, and the interaction between two fleets of vehicles.

The models do not reflect all of an operation's turnout delay times since many of the inherent delays in the system are excluded. The models assume that the empty vehicle will always utilize the turnout which will yield the least delay. Furthermore the models assume the empty vehicle has a constant rate of acceleration, the turnout spacing is uniform, and the turnouts have no length. The specific characteristics of each of the models will be discussed in their corresponding sections.

Interaction Between A Loaded Vehicle And An Empty Vehicle

The simple F-factor equation to be tested is:

$$F = \frac{(V_1 + V_2)}{2V_2}$$

Initially, the empty vehicle is positioned opposite turnout $S_0$ and the loaded vehicle is randomly located (uniform distribution over the interval $(5,10)$) at a distance $D_1$ away. If both vehicles were to proceed at their corresponding velocities they would meet at a point that is at a distance
distance $X$ from turnout $S$. Thus:

$$X = \frac{(V_2 D_1)}{(V_1 + V_2)}$$

The first turnout (INTE) that the empty vehicle could potentially use can be readily determined as:

$$\text{INTE} = \frac{X}{S}$$

where:

\text{INTE} \text{ is an integer number.}

Referring to the figure below, the model tests whether or not the empty vehicle can safely utilize turnout INTE. If the vehicle cannot safely use this turnout, then the model will check prior turnouts until it finds the first turnout that the empty vehicle can safely use. Now the F-factor can be easily determined.
Distance (miles) | 0 | X | 5 | D1 | 10

Turnout locations | S | INTE

where:

X = potential meeting point

D1 = initial distance vehicles are apart

where:

● = location of loaded vehicle

○ = location of empty vehicle

The variable names refer to the variable names used in the computer program.
## Program Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>rate of acceleration</td>
</tr>
<tr>
<td>D1</td>
<td>initial distance vehicles are apart</td>
</tr>
<tr>
<td>D3</td>
<td>stopping distance</td>
</tr>
<tr>
<td>F1</td>
<td>coefficient of friction</td>
</tr>
<tr>
<td>F9</td>
<td>F-factor for a single meeting</td>
</tr>
<tr>
<td>INTE</td>
<td>first turnout that empty vehicle tries to use</td>
</tr>
<tr>
<td>N</td>
<td>the sample size</td>
</tr>
<tr>
<td>S</td>
<td>distance between turnouts</td>
</tr>
<tr>
<td>S3</td>
<td>the average F-factor for the sample</td>
</tr>
<tr>
<td>TOTAL</td>
<td>the sum of the F-factors</td>
</tr>
<tr>
<td>T3</td>
<td>the stopping time</td>
</tr>
<tr>
<td>T4</td>
<td>time for empty vehicle to travel distance (Y_1)</td>
</tr>
<tr>
<td>V1</td>
<td>speed of loaded vehicle</td>
</tr>
<tr>
<td>V2</td>
<td>speed of empty vehicle</td>
</tr>
<tr>
<td>X</td>
<td>potential meeting point</td>
</tr>
<tr>
<td>y1</td>
<td>distance from (X) to beginning of deceleration zone</td>
</tr>
<tr>
<td>Y2</td>
<td>distance loaded vehicle is from turnout when empty vehicle begins to decelerate</td>
</tr>
<tr>
<td>Y6</td>
<td>distance loaded vehicle is from turnout when empty vehicle has stopped in turnout</td>
</tr>
</tbody>
</table>
C

C SIMULATION OF ONE EMPTY VEHICLE MEETING ONE LOADED VEHICLE
C

F1 = 0.25
A1 = 79036.3636 * F1
Q1 = RAND(SCLCCK(0.0))
N = 10000
Z1 = N

C

C THE VEHICLE'S CHARACTERISTICS
C

V1 = 10.
V2 = 35.
S = 0.2

C

C THE STOPPING FUNCTIONS
C

D3 = V2**2 / (2. * A1)
T3 = V2 / A1
WRITE (6, 101)
101 FORMAT ('I')

C

C THE NUMBER OF SAMPLES
C

DO 1 I1 = 1, 20
TOTAL=0.
C
C THE SAMPLE SIZE
C
DO 2 I=1,N
C
C INITIAL DISTANCE THE VEHICLES ARE APART
C
D1=FRAND (0.0) *5. +5.
C
C DETERMINE TURNOUT THAT EMPTY VEHICLE CAN SAFELY PULL INTO
C
X=V2*D1/(V1+V2)
INTE=X/S
Y1=X-S*INTE+D3
T4=Y1/V2
Y2=T4*V1+Y1-D3
T5=Y2/V1

C
C CAN EMPTY VEHICLE SAFELY PULL INTO THIS TURNOUT
C
4 IF(T5.GT.T3) GO TO 3
C
C TRY TO STOP EMPTY VEHICLE AT PREVIOUS TURNOUT
C
Y1=Y1+S
Y2=Y2+V1*S/V2+S
T5=Y2/V1
GO TO 4
C
C CALCULATION OF THE F-FACTOR
C
3  Y6 = (T5 - T3) * V1

   F9 = Y6 / S

C
C SUMATION OF THE F-FACTOR
C
2  TOTAL = TOTAL + F9

C
C THE AVERAGE F-FACTOR
C

   S3 = TOTAL / Z1

1  WRITE (6, 100) S3

100 FORMAT ( 1', 5X, F8.4)

RETURN

END
Interaction Between An Empty Vehicle And A Fleet Of Loaded Vehicles

The $F$-factor equations to be tested are equations 2.18 and 2.19. The four headway distributions that are required are the exponential, shifted exponential, Erlang($\alpha=2$), and Pearson Type III($a=2$). The equations required for these distributions are:

1. Exponential:
   \[
   \Pr(h<h_c) = 1-e^{-Q_3 h_c}
   \]
   \[
   \bar{F}_2 = \frac{V_i \left[ e^{-Q_3 h_c} (-Q_3 h_c - 1) + 1 \right]}{-S Q_3 (e^{-Q_3 h_c} - 1)}
   \]
   \[
   \bar{F}_1 = \frac{(V_i + V_2)}{2 V_2}
   \]

2. Shifted exponential:
   \[
   \Pr(h<h_{cL}) = 1-e^{-Q_3 (h_c-R_T)}
   \]
   \[
   \bar{F}_2 = \frac{V_i \left[ \frac{V}{V_i} + R_T + \frac{1}{Q_3} - e^{-Q_3 (h_c-R_T)} (h_c + \frac{V}{V_i} + \frac{1}{Q_3}) \right]}{S \left( 1-e^{-Q_3 (h_c-R_T)} \right)}
   \]

3. Erlang ($\alpha=2$):
   \[
   \Pr(h<h_c) = 1-e^{-2 Q_3 h_c (1+2 Q_3 h_c)}
   \]
   \[
   \bar{F} = \frac{V_i \left[ e^{-2 Q_3 h_c (-2 Q_3 h_c^2 - 2 h_c - 1/Q_3) + 1/Q_3} \right]}{S \left( 1-e^{-2 Q_3 h_c (1+2 Q_3 h_c)} \right)}
   \]

4. Pearson Type III($a=2$):
   \[
   \Pr(h<h_c) = 1-e^{-b(h_c-R_T) \left[ b \left( h_c-R_T \right) + 1 \right]}
   \]
   \[
   \bar{F}_2 = \frac{V_i}{S} \left[ \frac{e^{-b(h_c-R_T) \left[ b \left( -h_c^2 - \frac{h_c}{V_i} + R_T h_c + \frac{R_T}{V_i} \right) - 2 h_c^2 - \frac{L}{V_i} + R_T \right] + \left[ R_T + \frac{L}{V_i} + \frac{2}{b} \right]}{1-e^{-b(h_c-R_T) \left[ b \left( h_c-R_T \right) + 1 \right]}} \right]
   \]

This model is similar to the one that has been developed in the previous section. In this model the functions to determine vehicle headways are:
1. Exponential:

\[ Y = [-\log(\mu)]/\lambda + \zeta \]

where:

\( \mu \) = uniform \((0, 1)\) variable
\( \lambda \) = mean of the exponential distribution
\( \zeta \) = minimum headway

and

2. Erlang (alpha=2) and Pearson Type III (a=2):

\[ Y = \left[ \sum_{i=1}^{\alpha} (-\log(\mu)/\lambda) \right] + \zeta \]

The critical headway must be determined to check if an empty vehicle can proceed from its current turnout. The empty vehicle progresses along a continuous road.
## Program Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>rate of acceleration</td>
</tr>
<tr>
<td>D1</td>
<td>initial distance vehicles are apart</td>
</tr>
<tr>
<td>D2</td>
<td>a distance function similar to D1</td>
</tr>
<tr>
<td>D3</td>
<td>stopping distance</td>
</tr>
<tr>
<td>F1</td>
<td>coefficient of friction</td>
</tr>
<tr>
<td>F9</td>
<td>F-factor</td>
</tr>
<tr>
<td>H</td>
<td>traffic flow rate</td>
</tr>
<tr>
<td>INTE</td>
<td>first turnout empty vehicle tries to utilize</td>
</tr>
<tr>
<td>LENG</td>
<td>the length of the loaded vehicle</td>
</tr>
<tr>
<td>N</td>
<td>sample size</td>
</tr>
<tr>
<td>Q</td>
<td>total number of vehicles during the day</td>
</tr>
<tr>
<td>R</td>
<td>alpha variable of Erlang function</td>
</tr>
<tr>
<td>S</td>
<td>distance between turnouts</td>
</tr>
<tr>
<td>S3</td>
<td>average F-factor of the sample</td>
</tr>
<tr>
<td>TOTAL</td>
<td>sum of the F-factors of the sample</td>
</tr>
<tr>
<td>T1</td>
<td>critical headway</td>
</tr>
<tr>
<td>T2</td>
<td>loaded vehicle headway</td>
</tr>
<tr>
<td>T3</td>
<td>stopping time</td>
</tr>
<tr>
<td>T4</td>
<td>time of empty vehicle to travel distance Y1</td>
</tr>
<tr>
<td>T5</td>
<td>time for loaded vehicle to reach turnout</td>
</tr>
<tr>
<td>V1</td>
<td>speed of loaded vehicle</td>
</tr>
<tr>
<td>V2</td>
<td>speed of empty vehicle</td>
</tr>
<tr>
<td>X</td>
<td>potential meeting point</td>
</tr>
<tr>
<td>XLAM</td>
<td>a parameter in the gamma function</td>
</tr>
<tr>
<td>Y1</td>
<td>distance from X where deceleration zone begins</td>
</tr>
<tr>
<td>Y2</td>
<td>distance loaded vehicle is from turnout when</td>
</tr>
</tbody>
</table>
empty vehicle begins to decelerate

\[ Y_6 \text{ the } F\text{-factor times the turnout spacing} \]
Program Listing

C
C
C SIMULATE AN EMPTY VEHICLE AND A FLEET OF LOADED VEHICLES
C

REAL H(18), LENG, MIN
DATA H/1., 2., 4., 10., 60., 100., 1., 2., 4., 10., 60., 100., 1.,
12., 4., 10., 60., 100./

C
C THE VEHICLE'S CHARACTERISTICS
C

S=0.3
V1=30.
V2=40.
D0=11.
F1=0.25
A1=79036.3636*F1
LENG=60./5280.
MIN=LENG/V1/D0+0.000050505051

C
C THE CRITICAL HEADWAY
C

T1= (S*A1*(V1+V2)+V1*V2**2)/(A1*V1*V2*D0)+LENG/(V1*D0)
Q1=RAND(SCLOCK(0.0))
N=10000

C
C THE STOPPING FUNCTIONS
C
D3 = V2 ** 2 / (2. * A1)
T3 = V2 / A1
WRITE (6, 101)
101 FORMAT ('1')
C
C THE NUMBER OF SAMPLES
C
DO 1 I1 = 1, 18
TOTAL = 0.
Q3 = D0 * H(I1)
C
C THE SAMPLE SIZE
C
DO 2 I = 1, N
IF (I.GT. 1.5) GO TO 3
C
C THE INITIAL DISTANCE THE FIRST TWO VEHICLES ARE APART
C
D1 = FRAND (0.0) * 5. + 5.
D2 = D1
GO TO 4
C
C CALCULATION OF THE VEHICLE HEADWAY
C
3 D4 = FRAND (0.0)
T2 = -1. * ALOG (D4) / Q3
C
C IS VEHICLE SPACE HEADWAY LESS THAN THE VEHICLE'S LENGTH

C

IF(T2.LT.MIN) GO TO 3
D2=D1+T2*V1*D0

C

C IS THE VEHICLE HEADWAY LESS THAN THE CRITICAL HEADWAY

C

IF(T2.GT.T1) GO TO 5

C

C CALCULATE F-FACTOR FOR HEADWAYS LESS THAN CRITICAL HEADWAY

C

Y6=T2*V1*D0+LENG
F9=Y6/S
D1=D2
GO TO 6

5 D2=D2-(X-Y1+D3)

4 IF(I.GT.1.5) GO TO 7

C

C DETERMINING MEETING POINT OF THE FIRST ENCOUNTER

C

X=V2*D2/(V1+V2)
GO TO 8

C

C DETERMINING MEETING POINT FOR THE OTHER ENCOUNTERS

C

7 X=(V1*D2-V1*V2**2/(2.*A1))/(V1+V2)

C

C FIND FIRST TURNOUT EMPTY VEHICLE WILL TRY TO PULL INTO

C
8 \ INTE = X/S  
   Y1 = X - S * INTE + D3  
   T4 = Y1 / V2  
   Y2 = T4 * V1 + Y1 - D3  
   T5 = Y2 / V1  
   
C
C CAN EMPTY VEHICLE SAFELY PULL INTO THIS TURNOUT
C
10 IF (T5 .GT. T3) GO TO 9  
C
C TRY TO STOP EMPTY VEHICLE AT THE PREVIOUS TURNOUT
C
       Y1 = Y1 + S  
       Y2 = Y2 + V1 * S / V2 + S  
       T5 = Y2 / V1  
       GO TO 10  
C
C CALCULATE THE F-FACTOR
C
9 \ Y6 = (T5 - T3) * V1 + LENG  
   F9 = Y6 / S  
   IF (I .LT. 1.5) GO TO 6  
   D1 = D2  
C
C SUMATION OF THE F-FACTORS
C
6 \ TOTAL = TOTAL + F9  
2 CONTINUE  
   Z1 = N
C
C CALCULATION OF THE AVERAGE F-FACTOR
C
S3 = TOTAL/Z1
1 WRITE(6,100) H(I1),S3
100 FORMAT(' ',5X,F6.1,5X,F8.4)
WRITE(6,101)
RETURN
END
Interaction Between Two Fleets Of Vehicles

The purpose of this model is to determine the range of traffic flow rates for which the expected F-factor equations are valid. The basic equation to be tested is:

$$F = Pr(h > h_c)F_1 + Pr(h < h_c)F_2$$

Most of the testing of the model will involve the exponential F-factor equation.

The simulation model, written in GPSSV, has been formulated such that it is restricted to certain combinations of vehicle speeds, turnout spacings, and acceleration rates. Generally, the empty vehicle must travel faster than the loaded vehicle. The figures on the following pages illustrate the formulation of the simulation model.
Time Parameters Of The Empty Vehicles

Turnout locations: $S_1$, $S_{i+1}$, $S_{j+2}$, $S_{j+3}$

Decision Point

Times:

- $A = \frac{(2aS-V_2^2)}{(2aV_2^2)} \times 60$
- $B = \frac{V_2}{a} \times 60$
- $C = \frac{(Sa-V_2^2)}{(aV_2^2)} \times 60$
- $D = \frac{S}{V_2} \times 60$

$S_j$ = turnout locations

Distances:

- $E = \left[ \frac{(2aS+V_2^2)}{(2aV_2^2)} \right] \times 60 + \frac{(3S/V_2)}{60}$
- $F = \left[ \frac{(2aS+V_2^2)}{(2aV_2^2)} \right] \times 60 + \frac{(3S/V_2)}{60}$

*the times are in minutes
Time Parameters Of The Loaded Vehicles

R = \( \frac{S}{V_1} \) \times 60

\( S_j \) = turnout locations

\[ Q = \left[ \frac{(aS+V_2^2)}{(aV_2)} \bigg] \bigg(\frac{S}{V_1} \bigg) \times 60 \]

\[ X = \left[ \frac{(4S/V_1)}{(aS+V_2^2)} \bigg] \bigg(\frac{S}{V_1} \bigg) \times 60 \]

\[ Y = \left[ \frac{(aS+V_2^2)}{(aV_2)} \bigg] \bigg(\frac{S}{V_1} \bigg) \times 60 \]

\[ Z = \left[ \frac{(2S/V_1)}{(aS+V_2^2)} \bigg] \bigg(\frac{S}{V_1} \bigg) \times 60 \]

*times are in minutes
1. Time parameters for the empty vehicles

   A = time to travel from dump to first decision point
   B = time to decelerate into turnout from decision point
   C = time to accelerate and travel at constant velocity to next decision point
   D = travel time between decision points at constant velocity
   E = travel time to accelerate and travel at constant velocity from first turnout to landing
   F = time to travel to landing (at constant velocity) from decision point

2. Time parameters for loaded vehicles

   R = time to travel from last turnout to dump
   Q = time from decision point to turnout
   X = time from landing to first decision point travelling at constant velocity
   Y = same as Q
   Z = time from turnout to decision point
Program Listing

$RUN *GPSSV SPRINT=-OUT PAR=SIZE=B

SIMULATE

* 

* 

* 

* TURNOUT SIMULATION

*  MODEL BY LARRY A. HENKELMAN

* GRADUATE STUDIES

* FORESTRY, UBC

* NOVEMBER, 1977

* 

* 

* TIME UNITS=1/1000 MINUTES

* 

* STORAGE DEFINITIONS # TURNOUTS & MAX # VEHICLES-2

STORAGE S1-S40,2

* 

* VARIABLE DEFINITIONS

1 VARIABLE PH1-1 V1 TAKES 1 FROM PRESENT TURNOUT

* 

* FUNCTION DEFINITIONS VEHICLE HEADWAY DISTRIBUTIONS

EXPON FUNCTION RN1,C24

0,0/.1,.104,.2,.222,.3,.335,.4,.509,.5,.69,.6,.915

.7,1.2/.75,1.38,.8,1.6/.84,1.83/.88,2.12/.9,2.3
* EXPO FUNCTION RN2,C24

0.0/.1.104/.2.222/.3.335/.4.509/.5.69/.6.915
.7.1.2/.75.1.38/.8.1.6/.84.1.83/.88.2.12/.9.2.3
.92.2.52/.94.2.81/.95.2.99/.96.3.2/.97.3.5
.98.3.9/.99.4.6/.995.5.3/.998.6.2/.999.7/1.8

* THE LETTERS IN COL 62 OF THE ADVANCE BLOCKS REFER TO THE
* THE TIME PARAMETERS ON THE ACCOMPANYING FIGURE
*

******************************************************************************
*
* MODEL SEGMENT 1 - EMPTY TRUCKS
*

RMULT 31,743  RANDOM NUMBER GENERATOR SEED
GENERATE 1000,FN$EXPON  GENERATE AN EMPTY TRUCK
ASSIGN 1,40,PH  STARTING AT TURNOUT # 40
ADVANCE 389  GO TO 1ST. DECISION PT. A
DDD GATE LR PH1,AAA  IF SHUT, USE TURNOUT, AAA
TEST NE PH1,1,EEE  IF LAST TURNOUT GO TO EEE
GATE SNF V1,AAA  NEXT FULL? PULL IN NOW.
BBB ADVANCE 450  DOESN'T USE TURNOUT D
ASSIGN 1-,1,PH  CONSIDER NEXT TURNOUT
TRANSFER ,DDD LOOP
*

* TRUCKS CANNOT PROCEED, MUST TURNOUT

AAA ADVANCE 121  DECELERATE INTO TURNOUT B
ENTER PH1,1  TRUCK ENTERS TURNOUT
LINK PH1,FIFO TRUCK QUEUES IN TURNOUT
CCC LEAVE PH1,1 EMPTY TRUCK LEAVES TURNOUT
ADVANCE 329 GOES TO NEXT DECISION PT. C
ASSIGN 1-,1,PH CONSIDER NEXT TURNOUT
TEST NE PH1,0,FFF IF LAST TURNOUT GO TO FFF
TRANSFER ,DDD LOOP

* LAST TURNOUT, PROCEED TO LANDING.

FFF ADVANCE 1861 THE LAST TURNOUT E
TRANSFER ,GGG
EEE ADVANCE 1861 THE LAST TURNOUT F
GGG TERMINATE

* MODEL SEGMENT 2 - LOADED TRUCKS

GENERATE 1000,PN$EXPO GENERATE A LOADED VEHICLE
ASSIGN 1,3,PH REMEMBER GATE 3
LOGIC S 1 SHUT GATE 1 TO EMPTY TRUCKS
ADVANCE 1229 GO TO 1ST DECISION POINT X
LOGIC S 2 SHUT GATE 2 TO EMPTY TRUCKS
ADVANCE 571 TRAVEL TO 1ST TURNOUT Y
LOGIC R 1 PASS TURNOUT 1, SO OPEN GATE
UNLINK 1,CCC,ALL RELEASE ALL TRUCKS AT 1

* GENERALIZED TURNOUT

HHH ADVANCE 29 ADVANCE TO THE NEXT DECISION PT. Z
TEST NE PH1,41,III IS THIS THE LAST TURNOUT ?
LOGIC S PH1 CLOSE GATE REMEMBERED IN PH1
ADVANCE 571 GO TO THE NEXT TURNOUT
LOGIC R V1 OPEN GATE AS PASSING TURNOUT
UNLINK V1,CCC,ALL RELEASE LINEUP AT TURNOUT
ASSIGN 1+,1,PH THINK ABOUT NEXT TURNOUT
TRANSFER ,HHH LOOP

* LAST TURNOUT BEFORE DUMP

III ADVANCE 571 GO TO LAST TURNOUT (#40)
LOGIC R 40 OPEN GATE 40 AS PASS
UNLINK 40,CCC,ALL RELEASE LINEUP AT GATE 40
ADVANCE 600 GO TO DUMP
TERMINATE R

*******************************************************************************
* MODEL SEGMENT 3 - TIMING
*******************************************************************************

GENERATE 2000000
TERMINATE 1

*******************************************************************************
* CONTROL CARDS
*******************************************************************************

START 1
END
Some of the equations developed in this thesis can be modified to illustrate the effect of the length of the vehicle. This length will not affect the time lost due to vehicle acceleration and deceleration but it will alter the expected $F$-factor equations. The simple $F$-factor equation is adjusted to allow for the loaded vehicle to travel one length of the loaded vehicle. Consequently, the simple $F$-factor equation will become:

$$F_L = \left[ \int_0^{r_{\text{max}}} (V_2 i \delta i) / (V_1 + V_2) \right] + L / S = (V_1 + V_2) / (2V_2) + L / S$$

where:

$L =$ the length of the loaded vehicle.

In the case of a single empty vehicle meeting a fleet of loaded vehicles the critical headway is increased, since the empty vehicle must wait the extra delay time created by the length of the loaded vehicle. Consequently, the critical headway will become:

$$h_{cL} = \left[ 2S a_n a_o (V_1 + V_2) + V_1 V_2 (a_n + a_o) + 2L a_n a_o V_2 \right] / \left[ 2a_n a_o V_1 V_2 \right]$$

The expected $F$-factor and the probability equations are accordingly modified. The formulas can be further modified if it is assumed there is a minimum gap between vehicles.

With a shifted exponential headway distribution the modified gap density function becomes:
where:

\( R_T \) = the minimum time gap between the back of the first loaded vehicle and the front of the next vehicle

\( R_T + L/V_i \) = minimum headway between vehicles.

The probability that a headway is less than the critical headway is:

\[
\Pr(h < h_{CL}) = 1 - e^{-Q_3 \left( h_{CL} - R_T - \frac{L}{V_i} \right)}
\]

and the expected F-factor for headways less than the critical headway is:

\[
\bar{F}_2 = \frac{V_i}{S} \left[ \frac{L/V_i + R_T + 1/Q_3 - e^{-Q_3 \left( h_c - R_T \right)} \left( h_c + L/V_i + 1/Q_3 \right)}{1 - e^{-Q_3 \left( h_c - R_T \right)}} \right]
\]

A shifted Erlang (alpha=2) headway distribution is developed from a Pearson Type III density function. If the parameter "a" is set to two then the gap density function becomes:

\[
g(t) = b^2 (t-c) e^{-b(t-c)} \quad c < t < \infty
\]

where:

\( c = R_T + L/V_i \)

\( b = 2 / (1/Q_3 - R_T - L/V_i) \)

The probability that the headway is less than the critical headway is:

\[
\Pr(h < h_{CL}) = \int_c^{h_{CL}} b^2 (h-c) e^{-b(h-c)} \, dh
\]

\[
= 1 - e^{-b(h_c - R_T)} \left[ b(h_c - R_T) + 1 \right]
\]

and the expected F-factor for headways less than the critical headway is:
\[ F_2 = \frac{V_i}{S} \left[ \frac{e^{-b(h_c R_T)}}{1-e^{-b(h_c R_T)}} \left[ b \left( -h_c \frac{h_c L}{V_i} + R_T h_c + \frac{R_T L}{V_i} \right) - 2h_c \frac{2}{b} - \frac{L}{V_i} + R_T \right] \right] \]
Simulation Of The F-factor For The Shifted Exponential And Pearson Type III Headway Distributions

The effective length is that portion of the length of the loaded vehicle that will cause the empty vehicle to remain in a turnout while the loaded vehicle is passing the empty vehicle. A simulation model was used to confirm or reject the shifted exponential and Pearson Type III (a=2) F-factor equations.

The computer program written to simulate the meeting of an empty vehicle and a fleet of loaded vehicles was modified to include the effect of the length of the loaded vehicle on the expected F-factor (Chapter 4). This version of the program required a minimum headway of the length of the loaded vehicle divided by its velocity plus a reaction time. The length of the loaded vehicle was assumed to be 60 feet while the driver's reaction time was approximately 2 seconds.

The tables below tabulate the average F-factor and expected F-factor of the simulations with a sample size of 10000 repeated 9 times. A t-test was utilized to test the null hypothesis that the expected F-factor is equivalent to the average F-factor of the samples. The F-factor equation developed for the shifted exponential headway case was not rejected at the one percent level of significance and was only rejected, at a five percent level of significance, 16.7 percent of the time. The F-factor equation for the Pearson Type III (a=2) headway case was rejected slightly more often than the F-factor equation for the shifted exponential headway case. It can be concluded that these two expected F-factor
equations adequately describe the interaction between a single empty vehicle and a group of loaded vehicles, for their respective headway distributions.
Table: For The F-factor Simulation Of The Interaction Between A Single Empty Vehicle And A Fleet Of Loaded Vehicles Based On Equations 2.18, 2.19, A.5, And A.6

Sample size = 10000
Number of samples = 9
Acceleration = 19759 mph²
"Conflict" hours = 11
Shift = (L/V, + 2 seconds)
L = 60 feet
Shifted exponential headway distribution

<table>
<thead>
<tr>
<th>Vehicle Speed (mph)</th>
<th>Distance (miles)</th>
<th>Traffic (vph)</th>
<th>Ave.</th>
<th>S.D.</th>
<th>Expect</th>
<th>Rejection</th>
<th>Hypothesis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded</td>
<td>Empty</td>
<td>Turnouts</td>
<td>Rate</td>
<td>Flow</td>
<td>F</td>
<td>x10⁻³</td>
<td>F</td>
</tr>
<tr>
<td>40</td>
<td>30</td>
<td>0.2</td>
<td>1</td>
<td>1.2226</td>
<td>4.68</td>
<td>1.2258</td>
<td>No</td>
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<tr>
<td>2</td>
<td>1.2272</td>
<td>5.69</td>
<td>1.2283</td>
<td>No²</td>
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<td></td>
<td></td>
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<tr>
<td>4</td>
<td>1.2330</td>
<td>8.64</td>
<td>1.2330</td>
<td>No²</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>1.2453</td>
<td>4.45</td>
<td>1.2498</td>
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<td>8.68</td>
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<td>No²</td>
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<tr>
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<td>5.58</td>
<td>0.9299</td>
<td>No²</td>
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<tr>
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<td>5.65</td>
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<td></td>
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<td>0.1</td>
<td>1</td>
<td>0.8624</td>
<td>2.72</td>
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<tr>
<td>2</td>
<td>0.8635</td>
<td>2.84</td>
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<tr>
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<tr>
<td>60</td>
<td>0.8378</td>
<td>3.23</td>
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<td>0.7747</td>
<td>No²</td>
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<td></td>
</tr>
</tbody>
</table>

1 Standard deviation

2 Hypothesis was not rejected for a 10% level of significance

3 Hypothesis was not rejected for a 1% level of significance
Table For The $F$-factor Simulation Of The Interaction Between A Single Empty Vehicle And A Fleet Of Loaded Vehicles Based On Equations 2.18, 2.19, A.7, And A.8

<table>
<thead>
<tr>
<th>Vehicle Speed (mph)</th>
<th>Distance (miles)</th>
<th>Traffic Turnouts Rate (vph)</th>
<th>Ave.</th>
<th>S.D.</th>
<th>Expect</th>
<th>Rejection</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loaded (mph)</td>
<td>Empty (mph)</td>
<td>Between Flow F $\times 10^{-3}$ F Hypothesis</td>
<td>(\alpha = 0.05)</td>
<td></td>
<td></td>
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<tr>
<td>40</td>
<td>30</td>
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<td>1</td>
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<td>3.84</td>
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<td>10.05</td>
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<td>4.79</td>
<td>0.8727</td>
<td>Yes</td>
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<td></td>
</tr>
</tbody>
</table>

1 Standard deviation
2 Hypothesis was not rejected for a 10% level of significance
There are several derivatives of the expected F-factor equations and components of the equation that are required in the optimization routines. Many of these derivatives are listed on the following pages.

1. Critical headway

\[ h_c = \left[ 2S a_a a_0 (V_1 + V_2) + V_1 V_2 (a_a + a_0) \right] / (2a_a a_0 V_1 V_2) \]

\[ \frac{\partial h_c}{\partial S} = \frac{V_1 + V_2}{V_1 V_2} \]

2. Exponential F-factor

\[ F_z = \bar{D}/S \]

\[ \bar{F} = \left[ \frac{V_1 + V_2}{2V_2} e^{-Q_3 h_c} + \frac{\left(1-e^{-Q_3 h_c}\right) \bar{D}}{S} \right] \frac{Q_3}{(Q_3 + 1)} + \left[ \frac{V_1 + V_2}{2V_2 (Q_3 + 1)} \right] \]

\[ \frac{\partial \bar{F}}{\partial S} = \left\{ -Q_3 e^{-Q_3 h_c} \left[ \frac{V_1 + V_2}{2V_2} \right] \frac{\partial h_c}{\partial S} + \left[ \frac{Q_3 e^{-Q_3 h_c} \bar{D}}{S} \right] \frac{\partial h_c}{\partial S} - \left[ \frac{\bar{D} (1-e^{-Q_3 h_c})}{S^2} \right] \right\} \]

\[ \bar{D} = V_1 \left[ e^{-Q_3 h_c} (-Q_3 h_c - 1) + 1 \right] / \left[ -Q_3 (e^{-Q_3 h_c} - 1) \right] \]

\[ \frac{\partial \bar{D}}{\partial S} = \frac{V_1 Q_3^2 (\frac{\partial h_c}{\partial S}) e^{-Q_3 h_c} [e^{-Q_3 h_c} (-Q_3 h_c - 1) + 1]}{[-Q_3 (e^{-Q_3 h_c} - 1)]^2} + \frac{V_1 Q_3^2 (\frac{\partial h_c}{\partial S}) e^{-Q_3 h_c} h_c}{-Q_3 (e^{-Q_3 h_c} - 1)} \]
3. Erlang (alpha=2) F-factor

\[ \overline{F} = \left( \frac{V_1 + V_2}{2V_2} \right) e^{-\lambda h_c} \left( 1 + \lambda h_c \right) + \left( 1 - e^{-\lambda h_c} \right) \left( \frac{D}{S} \right) \frac{Q_3}{(Q_3 + 1)} \]

\[ \lambda = 2Q_3 \]

\[ \frac{\partial \overline{F}}{\partial S} = \left\{ \left( \frac{V_1 + V_2}{2V_2} \right) e^{-\lambda h_c} \left( -\lambda^2 h_c \frac{\partial h_c}{\partial S} \right) + \left( \frac{\partial D}{\partial S} \right) \frac{1 - e^{-\lambda h_c} \left( 1 + \lambda h_c \right)}{S} \right\} \left\{ \frac{Q_3}{(Q_3 + 1)} \right\} \]

\[ D = V_1 \left[ e^{-\lambda h_c} \left( -\lambda h_c^2 - 2h_c - 2/\lambda \right) + 2/\lambda \right] / \left[ 1 - e^{-\lambda h_c} \left( 1 + \lambda h_c \right) \right] \]

\[ \frac{\partial D}{\partial S} = \left( V_1 e^{-\lambda h_c} \lambda^2 h_c \frac{\partial h_c}{\partial S} \right) \left( e^{-\lambda h_c} \left( \lambda h_c^2 + 2h_c + 2/\lambda \right) - 2/\lambda \right) \left[ 1 - e^{-\lambda h_c} \left( 1 + \lambda h_c \right) \right]^2 \]

\[ + \frac{h_c}{1 - e^{-\lambda h_c} \left( 1 - \lambda h_c \right)} \]

4. Simple F-factor including the vehicle's length

\[ \overline{F} = \left( V_1 + V_2 \right) / \left( 2V_2 \right) + L/S \]

\[ \left( \frac{\partial \overline{F}}{\partial S} \right) = -L/S^2 \]
5. Shifted exponential F-factor

\[
\mathcal{F} = \left\{ \left[ \frac{L}{V_i} + R_f + \frac{1}{Q_3} \right] e^{-Q_3(h_c R_f)} \left[ h_c + \frac{L}{V_i} + \frac{1}{Q_3} \right] \left( \frac{V_i}{S} \right) + e^{-Q_3(h_c R_f)} \left[ \frac{V_i + V_2}{2V_2} + \frac{L}{S} \right] \right\}
\]

\[
\frac{\partial \mathcal{F}}{\partial S} = \left( \frac{-V_i Q_3}{(Q_3 + 1) S^2} \right) \left( \frac{L}{V_i} + R_f + \frac{1}{Q_3} \right) e^{-Q_3(h_c R_f)} \left[ h_c + \frac{L}{V_i} + \frac{1}{Q_3} \right] - \left( \frac{L}{(Q_3 + 1) S^2} \right)
\]

\[
+ \left( \frac{V_i Q_3 e^{-Q_3(h_c R_f)}}{(Q_3 + 1) S} \left| \frac{\partial h_c}{\partial S} \right| \frac{L}{V_i} \right)
\]

\[
- \left( \frac{Q_3 e^{-Q_3(h_c R_f)}}{(Q_3 + 1)} \right) \left( \frac{V_i + V_2}{2V_2} + \frac{L}{S} \right) + \frac{L}{S^2} \right\}
\]

\[\text{A15}\]
6. Pearson Type III (a=2) F-factor

Let

\[ a = 2 \]
\[ b = \frac{2}{(1/Q_3 - R_f - L/V)} \]
\[ c = R_f + L/V \]

Then

\[
\bar{F} = \left\{ \frac{V_i}{S} \right\} \left[ e^{-b(h_c^2-R_f)} \left( b \left[ -h_c^2 + R_f \frac{h_c}{V} \right] + \frac{2}{b} \right) \right]
\]
\[
+ \left[ e^{-b(h_c^2-R_f)} \left( b \left[ h_c - R_f \right] + 1 \right) \right] \left\{ \frac{Q_3}{Q_3 + 1} \right\} + \left\{ \frac{V_i + V_2}{2V_2} \right\} \left\{ 1 \right\}
\]
\[
\frac{\partial \bar{F}}{\partial S} = \left\{ \frac{-Q_3 V_i}{(Q_3 + 1) S^2} \right\} \left[ e^{-b(h_c^2-R_f)} \left( b \left[ -h_c^2 + R_f \frac{h_c}{V} \right] + \frac{2}{b} \right) \right]
\]
\[
+ \left\{ \frac{Q_3 V_i}{(Q_3 + 1) S} \right\} \left[ e^{-b(h_c^2-R_f)} \left( \frac{\partial h_c}{\partial S} \right) b^2 \left( h_c^2 + R_f \frac{h_c}{V} \right) \right] + \left\{ \frac{-L}{(Q_3 + 1) S^2} \right\}
\]
\[
+ \left\{ \left\{ \frac{Q_3 V_i}{(Q_3 + 1)} \right\} \left( \frac{V_i + V_2}{2V_2} \right) \frac{\partial h_c}{\partial S} \right\} e^{-b(h_c^2-R_f)} b^2 \left( -h_c + R_f \right)
\]
APPENDIX 8 OPTIMUM TURNOUT SPACING COMPUTER PROGRAM

Program Variables

<table>
<thead>
<tr>
<th>Variable</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>deceleration</td>
</tr>
<tr>
<td>A2</td>
<td>acceleration</td>
</tr>
<tr>
<td>B</td>
<td>b parameter of Pearson Type III distribution</td>
</tr>
<tr>
<td>C</td>
<td>&quot;cost functions&quot;</td>
</tr>
<tr>
<td>D1</td>
<td>number of &quot;conflict&quot; hours per day</td>
</tr>
<tr>
<td>D2</td>
<td>number of operating days per year</td>
</tr>
<tr>
<td>F1</td>
<td>coefficient of friction</td>
</tr>
<tr>
<td>F2</td>
<td>simple F-factor including the vehicle's length</td>
</tr>
<tr>
<td>F3</td>
<td>expected F-factor</td>
</tr>
<tr>
<td>F4</td>
<td>prime of the expected F-factor</td>
</tr>
<tr>
<td>F7</td>
<td>coefficient of acceleration</td>
</tr>
<tr>
<td>H</td>
<td>traffic flow rate in vph</td>
</tr>
<tr>
<td>K</td>
<td>acceleration in mph²</td>
</tr>
<tr>
<td>L</td>
<td>the length of the vehicle in feet</td>
</tr>
<tr>
<td>L1</td>
<td>the length of the vehicle in miles</td>
</tr>
<tr>
<td>L2</td>
<td>minimum gap between vehicles in seconds</td>
</tr>
<tr>
<td>L3</td>
<td>minimum gap between vehicles in days</td>
</tr>
<tr>
<td>M</td>
<td>adjusted hauling cost in dollars per hour</td>
</tr>
<tr>
<td>P1, P2, P3</td>
<td>components of the cost function</td>
</tr>
<tr>
<td>Q1</td>
<td>expected useful life of the road in years</td>
</tr>
<tr>
<td>Q3</td>
<td>number of headways during the &quot;conflict&quot; hours</td>
</tr>
</tbody>
</table>
R1-R0 components of expected F-factor and prime of expected F-factor

S turnout spacing

V1 speed of loaded vehicle in mph

V2 speed of empty vehicle in mph

Y turnout spacing

Z prime of "cost function"
Program Listing

50 REM PROGRAM FOR THE CALCULATION OF OPTIMUM SPACING
100 REM PROGRAM DEVELOPED SEPT. 1978 BY D.I. ANDERSON
150 DIM Y(4), Z(4), C(4,3), A$(82), H(6), S(4,3)
200 A$(1,40) = "DISTRIBUTION OPT. SPACING  S* - 50% "
250 A$(41,80) = "  S* + 50%  S* + 200%"
300 MAT Z = ZER
350 DISP "INPUT THE VEHICLE'S LENGTH- FEET";
400 INPUT L
450 L1 = L / 5280
500 DISP "MINIMUM GAP BETWEEN TRUCKS- SEC. ";
550 INPUT L3
600 DISP "INPUT NUMBER OF CONFLICT HOURS PER DAY";
650 INPUT D1
700 L2 = L3 / 3600 / D1
750 DISP "NUMBER OF WORKING DAYS PER YEAR";
800 INPUT D2
850 DISP "INPUT COEFFICIENT OF FRICTION";
900 INPUT F1
925 DISP "INPUT COEFFICIENT OF ACCELERATION";
930 INPUT F7
950 A1 = 79036.363636 * F1
960 A2 = 79036.363636 * F7
1000 DISP "INPUT TRAFFIC FLOW IN VPH";
1050 INPUT H
1100 DISP "LOADED VEHICLE'S SPEED IN MPH";
1150 INPUT V1
1200 DISP "EMPTY VEHICLE'S SPEED IN MPH";
1250 INPUT V2
1300 DISP "ENTER COST PER TURNOUT (DOLLARS)";
1350 INPUT C
1400 DISP "TRUCK RENTAL RATE IN $/HR";
1450 INPUT M
1500 DISP "EXPECTED LIFE OF THE ROAD IN YRS";
1550 INPUT Q1
1600 PRINT LIN3"CALCULATION OF OPTIMUM SPACING AND COST FUNCTION
1650 PRINT LIN1"NUMBER OF HOURS PER DAY = "D1
1700 PRINT "NUMBER OF WORKING DAYS PER YEAR = "D2
1750 PRINT "COEFFICIENT OF FRICTION = "F1
1775 PRINT "COEFFICIENT OF ACCELERATION = "F7
1800 PRINT "THE LENGTH OF THE VEHICLE = "L" FEET"
1850 PRINT "VEHICLES PER HOUR = "H
1900 PRINT "MINIMUM GAP BETWEEN VEHICLES = "L3" SECONDS"
1950 PRINT "VELOCITY OF LOADED VEHICLE = "V1" MPH"
2000 PRINT "VELOCITY OF EMPTY VEHICLE = "V2" MPH"
2050 PRINT "COST PER TURNOUT = "C" DOLLARS"
2100 PRINT "TRUCK RENTAL RATE = "M" DOLLARS/HOUR"
2150 PRINT "EXPECTED LIFE OF THE ROAD = "Q1" YEARS"
2200 Q3=H*D1-1
2250 FOR S1=1 TO 3
2300 S3=V1+V2
2350 K=(2*A1*A2)/(A1+A2)
2400 Y (1) =0.00001
2450 Y (3) =10.5
2500 Y (2) =(Y (1)+Y (3))/2
2550 FOR I1=1 TO 3
2600 GOSUB S1 OF 7450,7450,6100
2650 NEXT I1
2700 IF Z(1)*Z(3)<0 THEN 2900
2750 Y(1)=F6*0.7
2800 Y(3)=F6*1.3
2850 Y(2)=(Y(1)+Y(3))/2
2900 Y4=Y(1)
2950 Y5=Y(3)
3000 Y6=2
3050 I1=2
3100 IF ABS(Y(1)-Y(3))<0.0001 THEN 3700
3150 IF Z(1)*Z(2)>0 THEN 3450
3200 Y(3)=Y(2)
3250 Y(2)=(Y(3)+Y(1))/2
3300 Z(3)=Z(2)
3350 GOSUB S1 OF 7450,7450,6100
3400 GOTO 3100
3450 Y(1)=Y(2)
3500 Y(2)=(Y(3)+Y(1))/2
3550 Z(1)=Z(2)
3600 GOSUB S1 OF 7450,7450,6100
3650 GOTO 3100
3700 IF S1=3 THEN 4000
3750 IF S1=2 THEN 3900
3800 GOSUB 4950
3850 GOTO 3950
3900 GOSUB 4950
3950 NEXT S1
4000 GOSUB 4950
4050 STANDARD
4100 PRINT LIN3
4150 PRINT A$
4200 PRINT LIN1
4250 PRINT "SIMPLE"
4300 PRINT " SPACING (FEET) S(1, 1); S(2, 1); S(3, 1); S(4, 1)
4350 PRINT " COST ($/M/VEH) C(1, 1); C(2, 1); C(3, 1); C(4, 1)
4400 PRINT " "
4450 PRINT "SHIFTED EXPONENTIAL"
4500 PRINT " SPACING (FEET) S(1, 2); S(2, 2); S(3, 2); S(4, 2)
4550 PRINT " COST ($/M/VEH) C(1, 2); C(2, 2); C(3, 2); C(4, 2)
4600 PRINT " "
4650 PRINT "PEARSON TYPE III"
4700 PRINT " SPACING (FEET) S(1, 3); S(2, 3); S(3, 3); S(4, 3)
4750 PRINT " COST ($/M/VEH) C(1, 3); C(2, 3); C(3, 3); C(4, 3)
4800 PRINT LIN2
4850 END
4900 REM S*
4950 J=1
5000 S(J, S1)=Y(2)*5280
5050 C(J, S1)=C/Y(2)+Q1*H*M*D2*D1*P3/P1*(H/V1+H/V2)
5100 J=2
5150 Y(2)=Y(2)*0.5
5200 REM 50% OF S*
5250 S(J, S1)=Y(2)*5280
5300 GOSUB S1 OF 7450,7450,6100
5350 C(J, S1)=C/Y(2)+Q1*H*M*D2*D1*P3/P1*(H/V1+H/V2)
5400 J=3
5450 Y(2)=Y(2)*3
5500 REM 150% OF S*
5550 S(J, S1) = Y(2) * 5280
5600 GOSUB S1 OF 7450, 7450, 6100
5650 C(J, S1) = C / Y(2) + Q1 * H * M * D2 * D1 * P3 / P1 * (H / V1 + H / V2)
5700 J = 4
5750 Y(2) = Y(2) * 4 / 3
5800 REM 200% OF S*
5850 S(J, S1) = Y(2) * 5280
5900 GOSUB S1 OF 7450, 7450, 6100
5950 C(J, S1) = C / Y(2) + Q1 * H * M * D2 * D1 * P3 / P1 * (H / V1 + H / V2)
6000 RETURN
6050 STOP
6100 REM SOLUTION OF VSX FOR THE PEARSON TYPE III DISTRIBUTION
6150 REM CALCULATION OF THE EXPECTED F-FACTOR
6250 F2 = S3 / (2 * V2) + L1 / Y(I1)
6300 B = 2 / (1 / Q3 - L1 - L2)
6350 R2 = EXP(-B * (R1 - L2))
6375 Q6 = 2 * R1 - 2 / B - L1 / V1 / D1 + L2
6400 R5 = R2 * (B * (-R1 + 2 - R1 * L1 / V1 / D1 + L2 * R1 + L1 * L2 / V1 / D1) + Q6)
6450 R6 = R2 * (E * (R1 - L2) + 1)
6500 R3 = V1 * D1 / Y(I1) * (R5 + L2 + L1 / V1 / D1 + 2 / B)
6550 F3 = (R3 + R6 * F2) * Q3 / (H * D1) + F2 / (H * D1)
6600 GOSUB 7000
6650 REM THE FIRST DERIVATIVE OF THE COST EQUATION
6700 P1 = 1 - H * Y(I1) * F3 / V1 - H * V2 / K
6750 P2 = F3 / V1 + Y(I1) * F4 / V1
6800 P3 = V2 / K + Y(I1) * F3 / V1
6825 Q7 = H / V1 + H / V2
6850 Z(I1) = -C / Y(I1) + 2 + (M * H * Q1 * D2 * D1) * (P2 / P1 + H * P2 * P3 / P1 * 2) * Q7
CALCULATION OF THE PRIME OF THE F-FACTOR

\[ R4 = \frac{S3}{(V1*V2*D1)} \]

\[ Q8 = \frac{Q3*V1}{(H*Y(II))} + 2 \]

\[ R7 = -\left( R2*(B*(-R1+L2*R1) - 2*R1-2/B+L2) + L2+L1/V1/D1+2/B \right)*Q8 \]

\[ R8 = \frac{Q3*V1}{(H*Y(II))} \times R2*R4*Bt2* (Rlt2-L2*R1) \]

\[ R0 = -L1/\left( H*D1*Y(II) \right)^2 \]

\[ F4 = R7 + R8 + R9 + R0 \]

EXPECTED F-FACTOR FOR SIMPLE AND SHIFTED EXPONENTIAL

\[ F2 = \frac{S3}{(2*V2)} + L1/Y(II) \]


\[ R2 = \text{EXP}(-Q3*(R1-L2)) \]

\[ R3 = ( (L1/V1/D1+L2+1/Q3) - R2*(R1+L1/V1/D1+1/Q3) ) \times V1*D1/Y(II) \]

\[ F3 = (R2*F2+R3) \times Q3/(H*D1) + F2/(H*D1) \]

CALCULATION OF THE FIRST DERIVATIVE OF THE Cost Equatio

\[ Z(II) = -C/Y(II) + 2*\left( M*H*Q1*D2*D1 \right) * (P2/P1+H*P2*P3/P1+2) * Q6 \]
REM CALCULATION OF THE PRIME OF THE F-FACTOR

R4 = S3 / (V1*V2*D1)
R5 = R3 / V1 / D1 * Y(I1)
R6 = -V1 * Q3 * R5 / H / Y(I1) ^ 2
R7 = V1 * Q3 ^ 2 * R4 * R2 / H / Y(I1) * (R1 + L1 / V1 / D1)
R8 = Q3 * R2 / H / D1 * (Q3 * R4 * F2 + L1 / Y(I1) ^ 2)
R9 = L1 / (H * D1 * Y(I1) ^ 2)
F4 = R6 + R7 - R8 - R9
RETURN
STOP
Operating Instructions

Once the computer program has been loaded into the memory of the calculator the program is started by pressing "RUN" then "EXECUTE". Once this has been accomplished the screen will display various questions. See the following pages for examples.
RUN
INPUT THE VEHICLE'S LENGTH- FEET? 50
MINIMUM GAP BETWEEN TRUCKS- SEC? 2
INPUT NUMBER OF CONFLICT HOURS PER DAY? 5
NUMBER OF WORKING DAYS PER YEAR? 200
INPUT COEFFICIENT OF FRICTION? .25
INPUT COEFFICIENT OF ACCELERATION? .25
INPUT TRAFFIC FLOW IN VPH? 1
LOADED VEHICLE'S SPEED IN MPH? 15
EMPTY VEHICLE'S SPEED IN MPH? 30
ENTER COST PER TURNOUT (DOLLARS)? 250
TRUCK RENTAL RATE IN $/HR? 15
EXPECTED LIFE OF THE ROAD IN YEARS? 10

CALCULATION OF THE OPTIMUM SPACING AND 'COST FUNCTION'

NUMBER OF HOURS PER DAY = 5
NUMBER OF WORKING DAYS PER YEAR = 200
COEFFICIENT OF FRICTION = 0.25
COEFFICIENT OF ACCELERATION = 0.25
THE LENGTH OF THE VEHICLE = 50 FEET
VEHICLE'S PER HOUR = 1
MINIMUM GAP BETWEEN VEHICLE'S = 2 SECONDS
VELOCITY OF LOADED VEHICLE = 15 MPH
VELOCITY OF EMPTY VEHICLE = 30 MPH
COST PER TURNOUT = 250 DOLLARS
TRUCK RENTAL RATE = 15 DOLLARS PER HOUR
EXPECTED LIFE OF THE ROAD = 10 YEARS

DISTRIBUTION OPT. SPACING S* - 50% S* + 50% S* + 200%

SIMPLE
SPACING(FeET) 2956.42 1478.21 4434.64 5912.85
COST ($/M/VEH) 912.73 1139.16 990.40 1149.22

SHIFTED EXPONENTIAL
SPACING(FeET) 2955.58 1477.79 4433.37 5911.16
COST ($/M/VEH) 913.20 1139.64 990.77 1149.23

PEARSON TYPE III
SPACING(FeET) 2951.35 1475.67 4427.02 5902.70
COST ($/M/VEH) 913.28 1140.41 991.46 1151.90
RUN
INPUT THE VEHICLE'S LENGTH-FEET? 50
MINIMUM GAP BETWEEN TRUCKS-SEC.? 2
INPUT NUMBER OF CONFLICT HOURS PER DAY? 5
NUMBER OF WORKING DAYS PER YEAR? 200
INPUT COEFFICIENT OF FRICTION? 0.25
INPUT COEFFICIENT OF ACCELERATION? 0.25
INPUT TRAFFIC FLOW IN VPH? 4
LOADED VEHICLE'S SPEED IN MPH? 20
EMPTY VEHICLE'S SPEED IN MPH? 25
ENTER COST PER TURNOUT (DOLLARS)? 100
TRUCK RENTAL RATE IN $/HR? 15
EXPECTED LIFE OF THE ROAD IN YEARS? 20

CALCULATION OF THE OPTIMUM SPACING AND 'COST FUNCTION'

<table>
<thead>
<tr>
<th>DISTRIBUTION</th>
<th>OPT. SPACING</th>
<th>S* - 50%</th>
<th>S* + 50%</th>
<th>S* + 200%</th>
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<tbody>
<tr>
<td>SIMPLE</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>SPACING (FEET)</td>
<td>371.21</td>
<td>185.61</td>
<td>556.82</td>
<td>742.42</td>
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<tr>
<td>COST ($/M/VEH)</td>
<td>3582.59</td>
<td>4298.54</td>
<td>3824.07</td>
<td>4311.96</td>
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<tr>
<td>SHIFTED EXPONENTIAL</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPACING (FEET)</td>
<td>369.94</td>
<td>184.97</td>
<td>554.91</td>
<td>739.89</td>
</tr>
<tr>
<td>COST ($/M/VEH)</td>
<td>3592.66</td>
<td>4311.46</td>
<td>3834.32</td>
<td>4322.90</td>
</tr>
<tr>
<td>PEARSON TYPE III</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SPACING (FEET)</td>
<td>370.79</td>
<td>185.39</td>
<td>556.18</td>
<td>741.58</td>
</tr>
<tr>
<td>COST ($/M/VEH)</td>
<td>3584.39</td>
<td>4301.40</td>
<td>3827.34</td>
<td>4318.89</td>
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</table>
Simulation models were developed to determine the effect perturbations to the independent variables, headway frequency distributions, and some on the assumptions have on the expected F-factor, total expected delay time, optimum turnout spacing, and cost. The difference between the actual and the perturbed values of the functions were calculated. These differences were grouped according to the true value of the independent variable and the perturbed values or according to the assumptions. The average, standard deviation, and the maximum value of each group were determined. Graphs of some of the results of the simulations are located on the following pages.
EFFECT THAT PERTURBATIONS TO THE VELOCITY OF THE EMPTY VEHICLE HAVE ON THE TOTAL EXPECTED DELAY TIME

DIFFERENCE IN TOTAL EXPECTED DELAY TIME (%)

PERTURBATION (MPH)

10
5 10
2.5 5 2.5

VELOCITY OF EMPTY VEHICLE (MPH)
THE EFFECT THAT PERTURBATIONS TO THE VELOCITY OF THE EMPTY VEHICLE HAVE ON THE COST FUNCTION

PERTURBATION (MPH)

AVERAGE

MAXIMUM

COST DIFFERENCE IN DOLLARS PER YEAR PER MILE PER VEHICLE

270
243
216
189
162
135
108
81
54
27
0
0
10
20
30
40

VELOCITY OF EMPTY VEHICLE (MPH)
EFFECT THAT PERTURBATIONS TO THE TRAFFIC FLOW RATE HAVE ON THE TOTAL EXPECTED DELAY TIME

DIFFERENCE IN TOTAL EXPECTED DELAY TIME (S)

PERTURBATION (VPH)

TRAFFIC FLOW RATE (VPH)
EFFECT THAT PERTURBATIONS TO THE TRAFFIC FLOW RATE HAVE ON THE OPTIMUM TURNOUT SPACING

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

TRAFFIC FLOW RATE (VPH)

AVERAGE
MAXIMUM

PERTURBATION (VPH)
EFFECT THAT PERTURBATIONS TO THE TRAFFIC FLOW RATE HAVE ON THE COST FUNCTION

COST DIFFERENCE IN DOLLARS PER YEAR PER MILE PER VEHICLE

PERTURBATION (VPH)

TRAFFIC FLOW RATE (VPH)
EFFECT THAT PERTURBATIONS TO THE EXPECTED USEFUL LIFE OF THE ROAD HAVE ON THE OPTIMUM TURNOUT SPACING

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)

AVERAGE

MAXIMUM

PERTURBATION (YEARS)

10

5

2.5
The effect that perturbations to the expected useful life of the road have on the cost function.
EFFECT THAT PERTURBATIONS TO THE TURNOUT CONSTRUCTION COST HAVE ON THE OPTIMUM TURNOUT SPACING

AVERAGE
MAXIMUM

PERTURBATION ($)

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

TURNOUT CONSTRUCTION COST ($)
THE EFFECT THAT PERTURBATIONS TO THE TURNOUT CONSTRUCTION COST HAVE ON THE COST FUNCTION
EFFECT THAT PERTURBATIONS TO THE TRUCK HAULING COST HAVE ON THE OPTIMUM TURNOUT SPACING

DIFERENCES IN OPTIMUM TURNOUT SPACING (%)

PERTURBATION ($/HR)

AVERAGE

MAXIMUM

TRUCK HAULING COST ($/HR)
The effect that perturbations to the truck hauling cost have on the cost function.
EFFECT THAT PERTURBATIONS TO THE COEFFICIENT OF FRICTION HAVE ON THE TOTAL EXPECTED DELAY TIME

PERTURBATION

0.1 +
0.075 +
0.05 +
0.025 +
0.1
0.075
0.05
0.025

DISSERENCES IN TOTAL EXPECTED DELAY TIME (X)

24
22
20
18
16
14
12
10
8
6
4
2
0

0.1
0.15
0.2
0.25
0.3
0.35

COEFFICIENT OF FRICTION

AVERAGE
MAXIMUM

------
-+++

211
EFFECT THAT DISCOUNT RATES FOR VARIOUS TIME PERIODS HAVE ON THE OPTIMUM TURNOUT SPACING

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

DISCOUNT RATE (%)

+ 12.5
+ 10
+ 7.5
+ 5
+ 2.5

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)
THE EFFECT THAT THE DISCOUNT RATE FOR VARIOUS
TIME PERIODS HAS ON THE COST FUNCTION

COST DIFFERENCE IN DOLLARS PER YEAR PER MILE PER VEHICLE

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)

DISCOUNT RATE (%)

AVERAGE ------
MAXIMUM -+-+-+-

+12.5
+10
+7.5
+5
12.5
10
7.5
5
2.5
2.5
EFFECT THAT MAINTENANCE COSTS FOR VARIOUS TIME PERIODS HAVE ON THE OPTIMUM TURNOUT SPACING

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)

TURNOUT MAINTENANCE COST ($/YEAR)

AVERAGE

MAXIMUM

0 5 10 15 20 25

80 72 64 56 48 40 32 24 16 8 0

12.5

10

7.5

5

2.5

1

0.5
EFFECT THAT MAINTENANCE COSTS FOR VARIOUS TIME PERIODS HAVE ON THE OPTIMUM TURNOUT SPACING

TURNOUT MAINTENANCE COST ($/YEAR)

AVERAGE
MAXIMUM

DIFERENCE IN OPTIMUM TURNOUT SPACING (%)

EXPECTED USEFUL LIFE OF THE ROAD (YEARS)
The effect that the maintenance cost for various time periods has on the cost function.
The effect that the maintenance cost for various time periods have on the cost function.

Average: ------
Maximum: +++++

Cost difference in dollars per year per mile per vehicle.

Expected useful life of the road (years).

Turnout maintenance cost ($/year): 2.5
2.5
0.5
0.5
EFFECT THAT THE PRIME OF THE F-FACTOR FOR VARIOUS TRAFFIC FLOW RATES HAS ON THE OPTIMUM TURNOUT SPACING

AVERAGE
MAXIMUM
E = SHIFTED EXPONENTIAL HEADWAY DISTRIBUTION
P = PEARSON TYPE III (α=2) HEADWAY DISTRIBUTION

DIFFERENCE IN OPTIMUM TURNOUT SPACING (%)

TRAFFIC FLOW RATE (VPH)
EFFECT THAT THE PRIME OF THE F-FACTOR FOR VARIOUS TRAFFIC FLOW RATES HAS ON THE COST FUNCTION

AVERAGE ------
MAXIMUM ++++++
E = SHIFTED EXPONENTIAL HEADWAY DISTRIBUTION
P = PEARSON TYPE III (a=2) HEADWAY DISTRIBUTION

COST DIFFERENCE IN DOLLARS PER YEAR PER MILE PER VEHICLE

TRAFFIC FLOW RATE (VPH)
EFFECT HEADWAY DISTRIBUTIONS FOR VARIOUS TRAFFIC FLOW RATES HAVE ON THE OPTIMUM TURNOUT SPACING

1 = SIMPLE VS EXPONENTIAL HEADWAY DISTRIBUTION
2 = SIMPLE VS ERLANG(\(\alpha=2\)) HEADWAY DISTRIBUTION
3 = EXPONENTIAL VS ERLANG(\(\alpha=2\)) HEADWAY DISTRIBUTION

DIFFERENCE IN THE EXPECTED F-FACTOR TIMES 0.0001
EFFECT THAT PERTURBATIONS TO THE TURNOUT SPACING HAVE ON THE TOTAL EXPECTED DELAY TIME

DIFFERENCE IN TOTAL EXPECTED DELAY TIME (%)

TURNOUT SPACING (MILES)

AVERAGE

MAXIMUM

PERTURBATION (MILES)

0.2

0.1

0.05
## APPENDIX 10 CONVERSION FACTORS

<table>
<thead>
<tr>
<th>Imperial Units</th>
<th>SI (Metric) Units¹</th>
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<tbody>
<tr>
<td>1 foot</td>
<td>= 0.3048 metres</td>
</tr>
<tr>
<td>1 mile</td>
<td>= 1.609344 kilometres</td>
</tr>
<tr>
<td>1 cunit</td>
<td>= 2.831685 metres³</td>
</tr>
<tr>
<td>1 pound</td>
<td>= 0.45359237 kilograms</td>
</tr>
<tr>
<td>1 ton</td>
<td>= 1.016047 tonne</td>
</tr>
<tr>
<td>1 foot²</td>
<td>= 0.09290304 metres²</td>
</tr>
<tr>
<td>1 horsepower</td>
<td>= 745.7 watts</td>
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</tbody>
</table>

¹ International System of Units