RELIABILITY ANALYSIS OF SIGHT DISTANCE FOR STOP-CONTROL INTERSECTIONS ON HORIZONTAL ALIGNMENTS

by

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ABSTRACT

Intersection sight distance (ISD) for stop-control intersections refers to the provision of adequate sight distance between a minor-road stopped vehicle and a major-road vehicle. The AASHTO policy for ISD for intersections on straight roadways is based on the extreme values of the component design variables, such as major-road design speed and time gap, and assumes that these variables are deterministic. This research presents a reliability method that considers the moments (mean and variance) of the probability distribution of each random variable instead of the extreme values. This reliability method also accounts for the correlations among the component random variables. A performance function in terms of a safety margin is defined as the difference between the expected available and expected required ISD. Relationships for the mean and standard deviation of the safety margin are developed using First-Order Second-Moment analysis. Design graphs for the obstruction location are established for different radii of horizontal curves, design speed, and probability of failure. The reliability method is very useful as it provides the reliability associated with ISD design values. For evaluation purposes, the method can be used to determine the probability of failure of a particular intersection for an existing obstruction and current traffic conditions. The method can also be used to design the obstruction location for a given probability of failure. It was found that the deterministic method generally provides a higher probability of failure when the obstruction is closer to the minor road.

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This thesis is dedicated to my loving family. They have both suffered during my studies and supported me throughout my studies. They have always encouraged me during difficult times and they granted me great devotion.

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Chapter 1: INTRODUCTION

1.1 Background

Traditionally, reliability (probabilistic) analysis has been used structural or geotechnical engineering, but not in transportation engineering Highway geometric design guides, such as the American Association of State Highway and Transportation Officials AASHTO (2001) and the Transportation Association of Canada TAC (1999), provide the minimum and desirable values of different highway geometric design elements, but do not quantify the reliability level. The design guides work on the assumption that if the recommended design values are correctly applied, the resulting road design would have an adequate margin of safety. This assumption is also accepted by the courts whenever there is a ruling on the designer's liability for vehicular collisions, where road geometry or operation is alleged.

Reliability analysis in transportation engineering uses highway design variables. These variables are mostly random variables. A performance function, in terms of the safety margin, is used to estimate the reliability level. The reliability level for existing conditions and potential improvements is evaluated in terms of the probability of failure. A small probability of failure reflects a high reliability level, and vice versa.

Intersection sight distance (ISD) is one of the most important design aspects of highway geometric design. The intersection sight distance is the distance that provides the driver with an unobstructed view of the entire intersection. It is generally considered that if the sight distance available for turning or crossing vehicle movements is equal to or more than the sight distance required on the major-road, drivers should have sufficient sight distance to avoid a collision., There is, however, no measure of reliability associated with sight distance evaluation for stop-control intersections at a major-road on a horizontal curve. To quantify the reliability associated with sight distance for stop-control intersections at a major-road on a horizontal curve; this research developed a reliability model. The following sections discuss various aspects involved in the modelling of intersection sight distance.

1.1.1 Intersection

An intersection is defined as the general area where two or more roadways intersect or cross. It is an integral and important part of the highway system as the efficiency, safety, cost of operation, maintenance, and the capacity of the highway network depends on the intersection design. From a highway geometric point of view, an intersection is the most sensitive part of the highway since many activities occur at the intersection in a very short period of time.

The selection of an intersection depends on several factors such as highway classification, traffic volume, safety, topography, and highway user benefits. There are three general types of intersections most commonly used in North America: 3-legged, 4-legged, and multi-legged intersections. Different types of intersections given in AASHTO (2001) and TAC (1999) are shown in Figure 1.1.

A 3-legged intersection consists of 3 entry/exits paths. Each path may be two-way or one-way. This type of intersection may be channelized or unchannelized and is generally used where the minor road is connected to the major road. A 4-legged intersection is the intersection of two major roads, two minor roads, or one major road and one minor road. This type of intersection consists of four entry/exit paths that may be one way or two ways and may be channelized or unchannelized. The configuration of 4-legged intersection depends on traffic volume, traffic type and topography of the area. A multi-legged intersection has more than 4 legs. It is a very complicated type of intersection and should be avoided if possible.

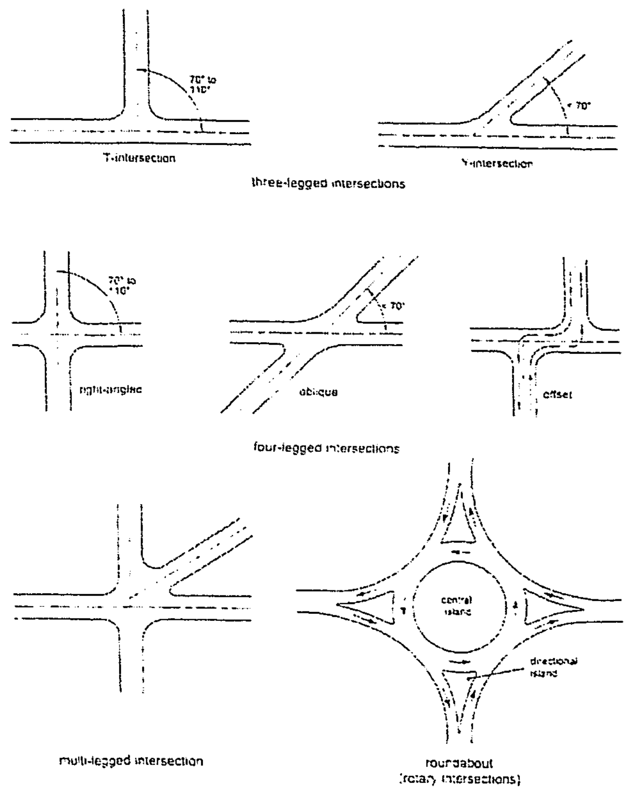


Figure 1.1 Different Types of Intersection (Source: TAC 1999)

1.1.2 Intersection Sight Distance

The concept of intersection sight distance can be explained from the sight triangle given in Figure 1.2. Each quadrant of an intersection should contain a triangular area free of obstruction that might block an approaching/departing driver's view of potentially conflicting vehicles (AASHTO 2001). One leg of this triangle is along the minor road while the other is along the major road. There are two types of sight triangles: approach sight triangle, and departure sight triangle.

For uncontrolled or yield-control intersections, AASHTO (2001) recommends that a clear approach triangle should be provided for an approaching vehicle in order to avoid any conflict. For stop-control intersections, a clear sight triangle

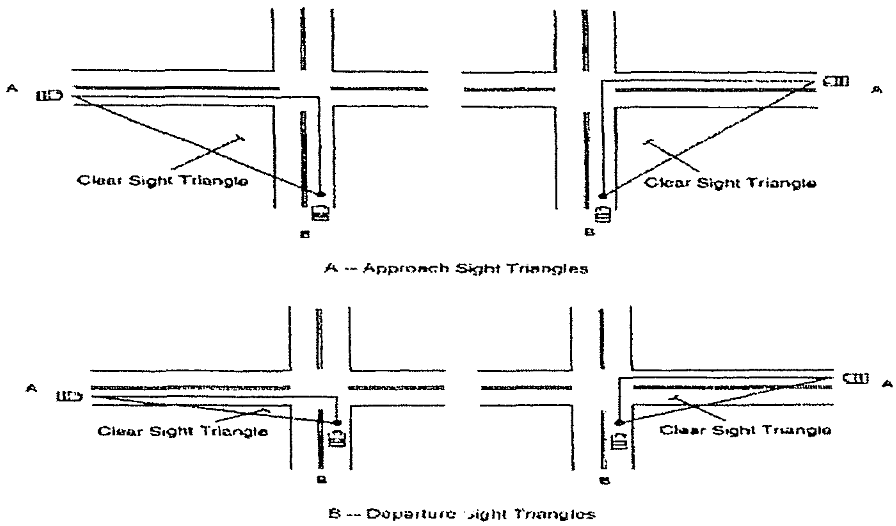


Figure 1.2 Intersection Sight Triangles (Source: AASHTO 2001)

called a departure triangle should be provided on both sides of the minor road. The intersection sight distance in both directions along the major road should be equal to the distance traveled by the major-road vehicle at the design speed, during the time gap required by the minor-road vehicle to manoeuvre safely.

The traffic control provided at any intersection depends on the type of traffic and the location of the intersection. AASHTO (2001) recommends that sight distance requirements at an intersection should be determined according to the type of traffic control used at that intersection. With respect to controls, intersections are generally categorized as:

- Case A. Intersection with no control
- Case B. Intersection with stop-control on the minor road
- Case C. Intersection with yield-control on the minor road
- Case D. Intersection with traffic signal control
- Case E. Intersection with all-way stop control
- Case F. Left turn from the major road

The details of the above categories can be found in AASHTO (2001). Since this research focuses on stop-control intersections (Case B), the procedure for the calculation of sight distance for only this case is described here. At a Case B intersection, a stopped vehicle on the minor road has the following three choices.

- Case B1. Left turn from the minor road
- Case B2. Right turn from the minor road
- Case B3. Crossing manoeuvre from the minor road

For all cases, length of departure sight triangle leg along the major road (left or right) is calculated by multiplying the major-road design speed by the time gap required for the minor-road vehicle. The time gap values depend on the type of minor-road vehicle, the type of manoeuvre, and the number of lanes on the major-road. Note that the time gap values for Case B2 or Case B3 are less than those for Case B1, for all types of vehicles.

1.1.3 Speed

Speed is one of the most important factors considered by travelers in selecting alternative routes (AASHTO 2001). The different types of speed used in highway geometric design include design speed, operating speed and posted speed. The design speed is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favourable that the design features of the highway govern (AASHTO 1994). Design speed is a selected speed used to determine the various geometric features of the roadway (AASHTO 2001). The facility should accommodate nearly all demands with reasonable adequacy and should not collapse under extreme traffic demands. The assumed design speed of any highway should be reasonable with respect to topography, functional classification of highway and the adjacent land use.

The operating speed is the speed at which road users are observed operating their vehicles during free-flow conditions. Operating speed is less than design speed. The 85th percentile of the distribution of observed speeds is the most frequently used measure of the operating speed associated with a particular location or highway geometric element. The posted speed is the speed limit, posted on the highway. This speed also entails legal considerations. The posted speed is generally less than the design speed, but operating speeds may exceed the posted speed. Recent studies of National Cooperative Highway Research Program (NCHRP) report 504 (Fitzpatrick et al. 2003) have shown that the 85th percentile of the operating speed exceeds the posted speed limits and that the 50th percentile of the operating speed is closer to or exceeds the posted speed limit.

1.1.4 Time Gap

A driver stopped on a minor-road approach must observe the gaps in the opposing traffic streams and determine whether a gap is adequate to complete a crossing or turning manoeuvre. After accepting a gap, the driver can complete the manoeuvre safely. Recent research work (Harwood et al, 1996) recommended time gap values for different type of vehicles. These values are given in AASHTO (2001). Details of the development of time gap models can be found in the NCHRP report 383 (Harwood et al. 1996).

1.2 Research Problem Statement

The design values for ISD analysis of stop-control intersections in AASHTO (2001) and TAC (1999) design guides are based on the extreme values at a certain percentile of the variables involved in the design (deterministic approach). There is no particular criterion to reflect the measure of reliability of ISD especially when the major road has a horizontal curve. It is possible that an existing/proposed obstruction does not satisfy the deterministic design values, but may have a reliability value that is deemed acceptable to the designer.

Hence, it is important to estimate the reliability level in terms of the probability of failure associated with any existing or proposed intersection design associated with any existing or proposed intersection design.

1.3 Research Objectives

The objectives of this research are as follows:

- To develop a modified deterministic mathematical model for the evaluation of available sight distance at existing/proposed stop-control intersections with a horizontal curve on the major road.
- To develop a reliability model to quantify the reliability level of sight distance at stop-control intersections with a horizontal curve on the major road.
- To develop design aids to help designers evaluate ISD easily.
- To apply the models developed to actual and hypothetical intersections to illustrate the application of the models.

1.4 Thesis Organization

The thesis is organized in 6 chapters. A brief description of each chapter is given Figure 1.3:

- Chapter 1 addressed general concepts including intersection types, intersection sight distance, speed, and time gap. The research problem statement and objectives were also described.
- Chapter 2 contains a comprehensive literature review of intersection sight distance based on the deterministic and reliability methods. A description of previous research conducted using reliability analysis in the area of highway geometric design is also presented.
- Chapter 3 provides information about reliability analysis, parameters of reliability analysis (such as mean, variance, variation coefficient, reliability index), and probability of failure. Some important concepts used in reliability analysis are also presented.
- Chapter 4 presents the development of modified deterministic and reliability

models, guideline for data preparation for the reliability analysis and model verification, sensitivity analysis, comparison of both models developed, and the design aids.

- Chapter 5 presents the practical application of the models developed to an actual and hypothetical intersection for illustration. Some suggestions for the improvement of existing ISD are also presented.
- Chapter 6 contains conclusions and recommendations for future research. The conclusions are related to features of the model and to the applicability of the models. The proposed future research includes the extension of proposed reliability model to other highway geometric elements.

In addition, the thesis includes four appendices. Appendix A includes the notation used in the thesis and Appendix B includes the first derivative of the reliability model with respect to one of the random variables, as an example. Appendix C and Appendix D present the design graphs developed for the deterministic and the reliability models, respectively.

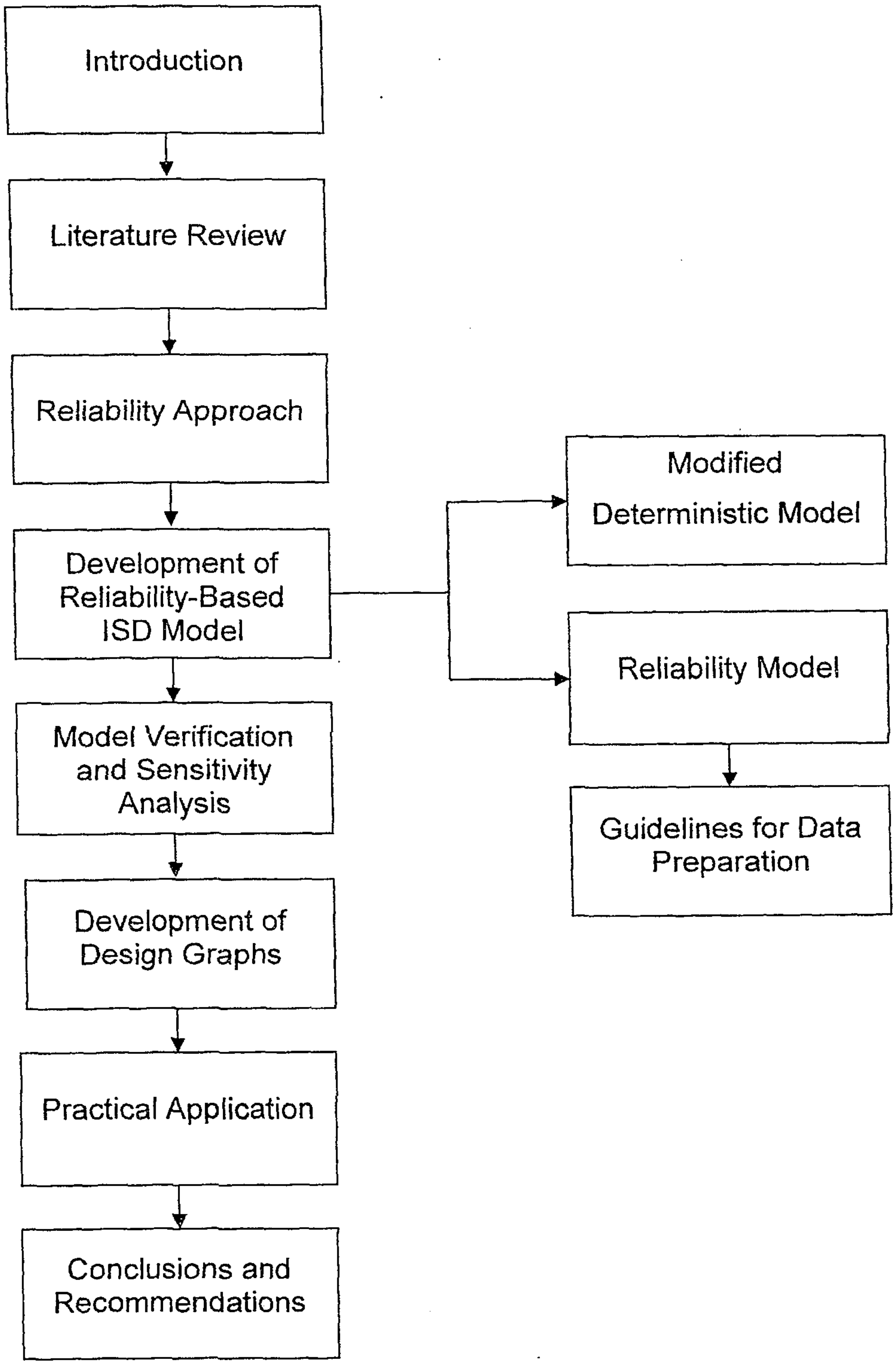


Figure 1.3 Thesis Organization

Chapter 2: LITERATURE REVIEW

2.1 Deterministic ISD Models

Intersection sight distance at intersections between minor and major roads is the adequate sight distance that should be provided for vehicles on the minor road. Several ISD deterministic models have been developed for stop-control and signalized intersections. Most models are based on the minimum sight distance required by AASHTO policy. This sight distance is a function of the major-road design speed and the gap-acceptance time for the minor-road vehicle.

2.1.1 ISD for Stop-Control Intersections

Fitzpatrick et al. (1990, 1998) and Harwood et al. (2000) found that the gap-acceptance time for the minor-road vehicle depends on the type of minor- road vehicle and the number of lanes of the major road to be crossed. Gattis et al. (1998) found that if the intersection has an acute angle on the right side of the minor road that can improve the intersection sight distance. But acute angle, on the left side of the minor road may obstruct the minor-road driver's line of sight. Gattis (1992) introduced analytical geometry to determine the intersection sight distance for horizontally curved roadways with tangential intersections and found that this type of intersection may have inadequate ISD. Easa et al. (2004) presented a three-dimensional model for stop-control intersection sight distance that provides a new idea for the analysis of sight distance for intersections on three-dimensional alignments. The study also considered the surface of the major-road and off-road obstructions.

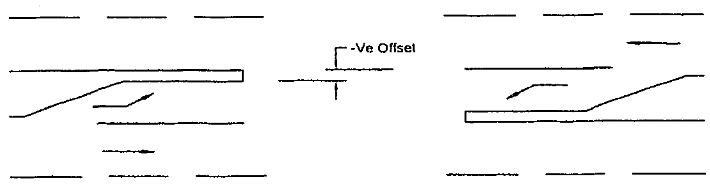
2.1.2 ISD for Signalized Intersections

In a study conducted by McCoy et al. (1992, 1997), the authors developed guidelines for offsetting opposing left-turn lanes to eliminate left-turn sight distance problems. The minimum offsets needed between opposing left-turn lanes to provide adequate sight distance were determined by setting the available sight distance equal to the required sight distance of AASHTO and then solving for the offset. The concept of critical gap across all design speeds depends on the type of vehicle and the number of lanes to be crossed by the left-turn vehicle. It was found that the offset of the opposing left-turn lanes is a function of the available sight distance.

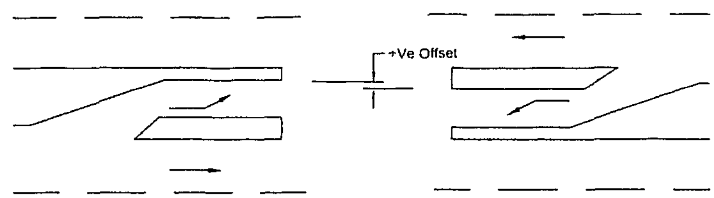
The offset is the distance from the right-edge of the left-turn lane median to the left-edge of the opposite left-turn vehicle lane line. When the two left-turn lanes are exactly aligned, the offset distance has a value of zero. A negative offset describes the situation where the opposing left-turn lane line is shifted to the left of the inner side left-turn lane median. A positive offset describes the situation where the opposing left-turn lane line is shifted to the right of the inner side of the left-turn lane median as in shown Figure 2.1. The left-turn lanes that are aligned or that have a positive offset provide greater sight distances than those that have a negative offset. A positive offset provides greater sight distance than the aligned left-turn lanes. Easa and Ali (2004) developed modified guidelines for intersection offsets using the proper location of the point of conflict of the left-turn and opposing through-lane vehicles. Easa et al. (2004) have also extended left-turn sight distance analysis to intersections located on horizontal curves.

2.2 Reliability Analysis in Highway Geometric Design

Reliability analysis is most commonly used in areas of civil engineering such as geotechnical and structural engineering. A performance function is defined by the difference of the available and the required values of the design variables. The available and the required components can be explained easily by defining a highway system where the driver-vehicle component requires (demands) a specific dimension of the geometric element and the highway geometry provides (supplies) a different dimension of that component. In the context of sight



(a)



(b)

Figure 2.1 (a) Negative and (b) Positive Offsetting between Left-turn Medians

distance, it is always desirable that the "supplied" sight distance should be at least equal to or greater than the required sight distance. Whenever the required sight distance exceeds the available sight distance, failure occurs. The probability associated with this failure can be estimated.

The probability of failure in highway geometric design does not necessarily mean that a collision will occur, but it clearly indicates a potential for a collision. The probability of failure corresponds to the area where the function is negative Figure 2.2. Reliability analysis provides a direct way of measuring safety rather than designing a component using the extreme value of the design variables. The analysis can provide the designers with an estimate of the reliability level that may be useful for safety conscious design procedures. Reliability analysis has been applied to some areas of transportation engineering. The following sections provide a brief literature review of existing applications of reliability analysis in traffic operation and highway geometric design.

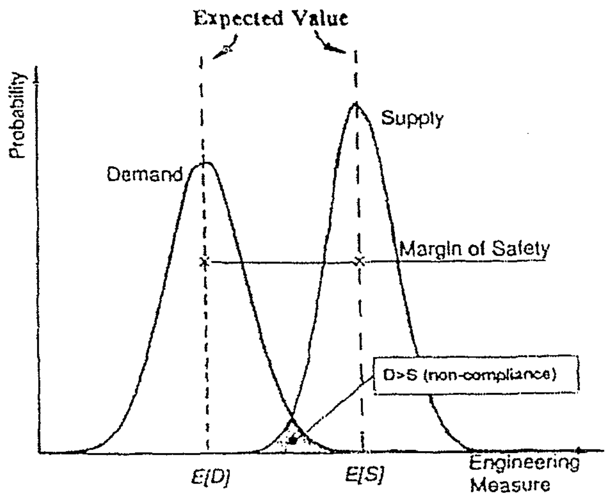


Figure 2.2 Demand vs. Supply for Reliability Level (Source: Navin et al. 1998)

2.2.1 Intergreen Interval

A reliability-based approach to the design of intergreen interval at traffic signals was introduced by Easa (1993). His applied probabilistic method considered approach speed, reaction time, deceleration rate, and vehicle length as random variables. A dilemma (failure) zone was explained within which a driver, faced with yellow, could neither stop nor clear the intersection. The research concluded that the intergreen interval based on the probabilistic method may be considerably greater than the deterministic interval. The author also suggested that, unlike the deterministic method, the probabilistic method provides information on the percentage of drivers with a probability of failure associated with the dilemma zone.

2.2.2 ISD at Intersections

Easa (2000) used a reliability approach to evaluate the stop-control intersection sight distance models of AASHTO (1990). Three AASHTO cases were addressed in: (1) No control, (2) Yield Control, and (3) Stop Control on the minor road. Design graphs were developed for the stop-control intersections. As the author recognized that the reliability procedure for the three manoeuvres (crossing, turning right, and turning left) of the stopped vehicles is similar, he presented only the procedure for the crossing manoeuvre. AASHTO WB-15 was used as the design vehicle. It was concluded that the current AASHTO design values of ISD appear to have high reliability levels. The author also suggested that the reliability method has the advantage of providing the designer with alternative design values that have a range of reliability levels. Thus, a designer can design a new intersection based on a specific reliability level or can estimate the reliability for an existing obstruction and evaluate necessary improvements.

Easa and Hussain (2004) have also developed a new probabilistic approach to evaluate offset requirements for left-turn vehicles at signalized intersections with four-lane divided major roads. It was found that the deterministic method provides higher reliability at lower design speeds, but that the reliability level decreases with an increase in speed along the major road. It was recommended that the reliability index should be carefully examined for high-speed major roads.

2.2.3 Design Consistency

Hirsh Moshe (1987) applied the reliability approach to evaluate design consistency in geometric design for horizontal curves. The researcher showed that the design consistency method based on comparing two speed distributions (for tangent and horizontal curve sections) does not give the full picture of all the speed changes that the drivers incur. In the extreme case, it is argued that theoretically, even when the two distributions are identical, it is still possible that each driver might experience a speed change. Hence it was proposed to obtain and analyze the distribution of the speed differences within the same section. A simulation method of reliability analysis was compared with the desired speed distribution at the tangent section, using a dataset of mean speeds corresponding to various curve radii and, then, the distributions were plotted for the simulated data to determine the mean speed of the different radii of horizontal curve. It was shown that the existing approach was likely to underestimate the amount of speed changes, and hence, was prone to accept inconsistent designs as consistent. The proposed approach was theoretical due to lack of data and hence, there were no practical applications presented in this research.

2.2.4 ISD at Railroad Crossings

Easa (1994) introduced reliability analysis for sight distance studies at railroad crossings. In this research, two cases were considered: (1) sight distance required along the highway and the railroad for an approaching vehicle and (2) sight distance required along the railroad for a stopped vehicle. The probabilistic method used was based on the first-order second moment (FOSM) of reliability analysis. Case 1 was modeled as a system with two parallel components. The probabilities of failures of each component and of the system were developed. Case 2 was modeled as a single-component system, and a design graph for the sight distance required along the railroad was presented. The normality assumption of the safety margin used in the reliability analysis was confirmed using Monte Carlo simulation.

2.2.5 Geometric Elements

Navin (1990) calculated the safety factor of isolated highway geometric design elements such as stopping sight distance, horizontal curves, decision sight distance, passing sight distance, and vertical curves. The reliability levels were calculated using the FOSM method at low and high values of the component design variables recommend by various design guides. It was concluded that the results based on preliminary data indicate that the safety index is the most meaningful safety measure of road design. Navin et al. (1998) applied the reliability approach to highway geometric design in relation to vehicle dynamics and driver expectation. The approach requires that designers explicitly input the mean and variance of the design variables and incorporate the dynamic features of the driver, vehicle, and roadway. The authors indicated that the methodology encourages designers to be more aware of the link between driver-vehicle-road and that the methodology should lead to a more safety conscious design. The additional burden on the designer is the need for more information and more analysis.

Chapter 3: RELIABILITY APPROACH

3.1 Introduction

In general, any engineering design consists of a balanced system of elements that satisfies various criteria of serviceability, durability, performance, and safety based on the demands made on the design. For example, a structure should be strong enough to resist all types of expected loadings. There are always uncertainties in the design depending on the variables involved. Reliability analysis is used to identify and quantify the uncertainties in a system design. .

The reliability level of any designed system cannot be established using a deterministic approach even though the design must involve a known and agreed safety margin. Instead, reliability analysis is based on a probabilistic approach that defines reliability in terms of the probability of failure. In the deterministic approach, the system is designed using extreme values of the system's components design variables but, in the probabilistic approach, the system is designed using probability distributions of the system's component variables. The following sections present information about the parameters of reliability analysis, existing reliability methods, and several important concepts used in reliability approach.

3.2 Parameters of Reliability Analysis

Any physical quantity that does not have a fixed numerical value is called a variable. A variable that can have any of a range of values that are equally likely to occur but can be described probabilistically is known as a random variable. In analytical models, variables are accounted for and it is assumed that the processes are uniformly distributed. Any random variable Y can be expressed as a function which assumes the values in the interval $(-\infty, \infty)$. The assumed value of a random variable is unpredictable and dependent on some chance system. Random variables are used in simulation techniques and their interactions also influence the results of the system. The parameters of a random variable include the mean, variance, correlation coefficient, and coefficient of variation.

The mean is the arithmetic average of any dataset. It can be obtained by summing up all dataset values and dividing the sum by the number of values. In statistics, the mean is a measure of the center of any distribution. The variance is the arithmetic average of the squared differences between the values and the mean value. The variance is a measure of the degree of dispersion of the dataset values around the mean. The standard deviation is the square root of the variance.

The interrelation between random variables is called the correlation of random variables. Let X and Y be two random variables. Mathematically, the correlation between the two random variables is measured by the coefficient of correlation as given below:

$$\rho = \frac{Cov[X,Y]}{\sigma_X \sigma_Y} \tag{3.1}$$

where σ_X and σ_Y are the standard deviations of the random variables X and Y. The value of the correlation coefficient ranges from – 1 to + 1. A relationship in which the values of two variables increase or decrease together is called a positive correlation and vice versa when the value of one variable increases as the value of the other variable decreases. The two random variables can be considered to be statistically independent if the correlation coefficient is less than ±0.3; they can be considered to be perfectly correlated if the correlation coefficient is greater than ±0.9 (Haldar et al. 2000). An intermediate value of 0.5 is normally used for analysis purposes.

The variation coefficient (CV) is a relative measure of data dispersion compared to mean. This can be explained by taking the ratio of the standard deviation and the mean and this can be expressed as

$$CV = \frac{\sigma}{\mu} \tag{3.2}$$

where σ and μ are the standard deviation and mean respectively of the dataset selected. CV has no units. It may be reported as decimal values or as a percentage. If the standard deviation of a dataset is very small, the values are closely bunched with a very small variation coefficient. If the standard deviation of a dataset is high, the values are scattered and give a high value for the coefficient of variation. For example, for μ = 20 and σ = 10, CV will be 0.5 or 50% which means that there is a huge dispersion of data compared to the mean values. If μ = 100 and σ = 2, CV will be 0.02 or 2% which means that there is very little dispersion of the data compared to the mean values. In many engineering problems, a CV of 0.1 to 0.3 is commonly used for a random variable (Haldar et al. 2000).

3.3 Existing Reliability Methods

Various methods have been proposed in order to measure the reliability of any design. Current reliability methods are classified into three groups: the exact reliability method, the point-estimate method, and the first-order second-moment method.

3.3.1 Exact Reliability Method

The exact reliability method requires full probability distributions for all the variables involved. This method may use analytical, numerical, or simulation techniques and is used where the probability of failure is of critical importance., The method is very difficult to use due to the nonlinear behaviour of performance functions in engineering systems, but it is widely used in structural and geotechnical engineering.

3.3.2 Point-Estimate Method

The point-estimate method is used when the performance functions are available in the form of charts or as finite-element solutions. This method can account for up to three central moments (mean value, standard deviation, and skewness). The method is used when the reliability analysis concerns unbiasedness, efficiency, sufficiency, and consistency of the random variable. Rosenblueth (1975) developed a point-estimate method which is still in practice for approximating the low-order moments of a function of random variables.

3.3.3 First-Order Second-Moment Method (FOSM)

The first-order second-moment method (FOSM) is very simple and consists of straightforward mathematical techniques. It is based on the truncated Taylor's expansion series. FOSM requires the mean and variance of the random variables. It requires only an approximation of the first two moments (expected value and variance) of a random variable as a non-linear function of other random variables.

The FOSM method is widely used in almost all types of engineering fields and now appears to be recognized as an important tool in transportation engineering. Failure in branches of civil engineering, such as structural and geotechnical engineering, may have catastrophic consequences in which case the FOSM approach may be inappropriate. In highway geometric design, where the probability of failure does not necessarily mean a collision will occur, the relatively higher values of the probability of failure may be acceptable. As the FOSM method was used in the current research, it is useful to describe the method before presenting the ISD reliability analysis.

Suppose that Z is a non-linear function of several random variables:

$$Z = f(Y_1, Y_2, \ldots, Y_n) \tag{3.3}$$

Then, $f(Y_1, Y_2, \ldots, Y_n)$ can be expanded in a Taylor series about the mean values μ_{Y1} to μ_{Yn}. In this case, we have;

$$Z = f(\mu_{Y1}, \mu_{Y2}, \ldots, \mu_{Yn}) + \sum_{i=1}^{n} (Y_i - \mu_{Y_i})\left(\frac{\partial f}{\partial Y_i}\right) + \in \tag{3.4}$$

where the partial derivatives are evaluated at $\mu_{Y1}, \mu_{Y2} \ldots \mu_{Yn}$ and $\in$ represents the higher order form. Truncating the series of Equation (3.4) at linear terms, we can obtain the first-order approximate mean and variance of Z, E[Z] and Var[Z] respectively, as below:

$$E[Z] \cong f(\mu_{Y1}, \mu_{Y2}, \ldots, \mu_{Yn}) \tag{3.5}$$

which shows that the mean of the function is approximately equal to the function of the means, and

$$\mathrm{Var}[Z] \cong \sum_{i=1}^{n} \left(\frac{\partial f}{\partial Y_i}\right)^2 \sigma_{Y_i}^2 + \sum_{i \neq j}^{n} \sum^{n} \left(\frac{\partial f}{\partial Y_i}\right)\left(\frac{\partial f}{\partial Y_j}\right) \mathrm{Cov}(Y_i, Y_j) \tag{3.6}$$

where the partial derivatives are evaluated at the mean values of random variables. σ_{yi} is the standard deviation of the Y_i, and σ^2_{Yi} is its variance, and $\mathrm{Cov}(Y_i, Y_j)$ is the covariance of the random variables Y_i and Y_j, which is given by

$$\mathrm{Cov}(Y_i, Y_j) = \rho_{Yi,Yj}\, \sigma_{Yi}\, \sigma_{Yj} \tag{3.7}$$

where $\rho_{Yi,Yj}$ is the co-efficient of correlation between random variables Y_i and Y_j (which ranges from –1 to +1). It is noted that if Y_i and Y_j are uncorrelated (or statistically independent) for all i and j, then Equation (3.6) can be simplified to

$$\mathrm{Var}[Z] \cong \sum_{i=1}^{n} \left(\frac{\partial f}{\partial Y_i}\right)^2 \sigma_{Y_i}^2 \qquad (3.7a)$$

The first-order approximation of E[Z] can be improved by including the second-order terms of a Taylor series expansion of $f(Y_1, Y_2 \ldots Y_n)$. The second-order approximation mean of Z would be

$$E[Z] = f(\mu_{Y1}, \mu_{Y2}, \ldots, \mu_{Yn}) + \frac{1}{2} \sum_{i=1}^{n} \sum_{j=1}^{n} \left(\frac{\partial^2 f}{\partial Y_i \partial Y_j}\right) \mathrm{Cov}[Y_i, Y_j] \qquad (3.8)$$

A useful measure of dispersion of a random variable Y_i, is the variation co-efficient of random variable Y_i, CV_{Yi}, is defined as

$$CV_{Y_i} = \frac{\sigma_{Yi}}{\mu_{Yi}} \qquad (3.9)$$

The preceding analysis does not require any assumptions about the form of the probability distributions of the variables. In addition, the analysis does not rely on any specific percentile values of the component variables, but only on the moments of their probability of distributions (mean and standard deviation).

3.4 Important Concepts

3.4.1 Safety Margin

The idea of the safety margin can be best explained by a simple example of supply and demand for any entity. Supply is that which is provided and demand is that which is required. The difference between the supply and demand is called the safety margin. If the supply is more than the demand, the safety margin will be positive but if the supply is less than the demand, then the safety margin will be negative. Let D_a be the supply and D_r be the demand. The difference between D_a and D_r, is the safety margin F, which is given by

$$F = D_a - D_r \tag{3.9a}$$

If D_a and D_r are statistically independent random variables, i.e. there is no correlation between D_a and D_r. The expected value of F is given by

$$E[F] = E[D_a] - E[D_r] \tag{3.10}$$

and the standard deviation of F is given by

$$\sigma_F = \sqrt{Var[D_a] + Var[D_r]} \tag{3.11}$$

3.4.2 Reliability Index

The ratio between the expected value of the safety margin, E[F] and the standard deviation of the safety margin σ_F is commonly known as the safety index or reliability index, denoted by β. Since the safety margin is a function of many random variables, its distribution tends to be normal even if the component variables are not normal (Ang et al. 1975 and Haldar et al. 2000). The distribution of the safety margin is shown in Figure 3.1.The reliability index is given by

$$\beta = \frac{E[F]}{\sigma_F} \tag{3.12}$$

Substituting the value of E[F] and σ_F into Equation (3.12), then

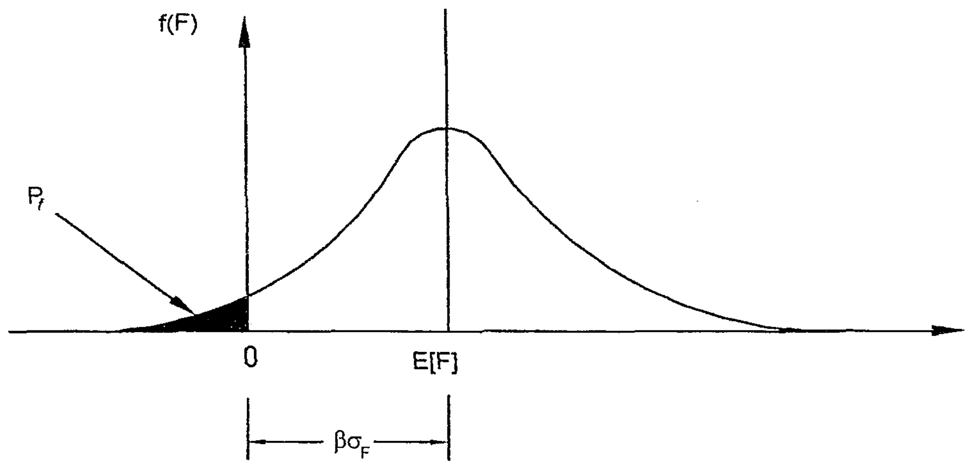


Figure 3.1 Probability Distribution of the Safety Margin

$$\beta = \frac{E[D_a] - E[D_r]}{\sqrt{Var[D_a] + Var[D_r]}} \tag{3.13}$$

From the Equation (3.13) above, it can be noted that the reliability index value is directly proportional to the expected value of the safety margin and inversely proportional to the square root of its variance. In other words, we can say that the reliability index of the safety margin is inversely proportional to its variation coefficient.

3.4.3 Probability of Failure

When the expected demand is greater than the expected supply, then a relatively high probability of failure exists. In Figure 3.1, the shaded area represents the probability of failure where F < 0. A large value of β indicates that the probability of failure is small. The estimate of the probability of failure for a normal random variable F, is as follows

$$P_f = \phi(-\beta) = 1 - \phi(\beta) \tag{3.14}$$

where $\phi(\beta)$ is the area under the probability density function of the standard normal variate from - ∞ to - β. This area can be obtained from tables of the standard normal variate. Substituting for β in Equation (3.14) from Equation (3.13), the probability of failure can be expressed as:

$$P_f = 1 - \phi\left(\frac{E[D_a] - E[D_r]}{\sqrt{Var[D_a] + Var[D_r]}} \right) \tag{3.15}$$

For the positive values of β, the probability of failure will be less than 50% and vice versa for negative values of β. It is to be noted that the probability of failure using Equation (3.15) assumes that the random variables D_a and D_r are statically independent. This assumption is valid since D_a and D_r do not have correlated random variables.

Chapter 4: DEVELOPMENT OF RELIABILITY-BASED ISD MODEL

4.1 Introduction

Two models were developed: (1) a modified deterministic model for calculating available sight distance, and (2) a probabilistic model for quantifying the reliability level of ISD. Both models assume that the major road has a horizontal curve and that the intersection angle is 90^0. The models are directly applicable to existing or proposed intersections.

The modified deterministic method was developed using analytical geometry and extreme values for all variables. Available sight distance was calculated and compared with the required sight distance to determine the lateral clearance needed for the obstruction. The FOSM method was then applied to develop a reliability model. The dataset used for the modified deterministic and the reliability models was taken from existing research. Using the modified deterministic and reliability models developed, design aids were established to facilitate the design and evaluation of ISD at proposed and existing intersections. The following sections present the model development, verification and sensitivity analysis.

4.2 Component Design Variables

Two categories of variables are used in this research. The first type is the deterministic variables that will remain same for the modelled intersection. They include the lane widths for the major and minor road, the distance from the obstruction to the edge of the minor and major road, the radius of curvature, and the widths of the minor and major roadways. The calculation for the minimum radius of the major road horizontal curve is directly proportional to the square of the design speed and inversely proportional to the maximum rate of superelevation and to the maximum side friction factor. Note that as the minimum radius is likely to be some value between 105 to 150 m for a major road design speed of 60 km/h (depending on the superelevation and coefficient of friction factor) the radius of 100 m was not included for speeds greater and equal to 60 km/h. Table 4.1 shows the values of minimum radii at different design speeds. Note that this table was extracted from AASHTO (2001).

The second type of variables is the random variables. These variables are expected to change randomly and include: the speed of the major-road vehicle, the time gap for the minor-road stopped vehicle, vehicle width, the positioning of the approaching vehicle within the lane, the distance from the driver's eye to the front of vehicle, the distance from the driver's eye to the left-side of the vehicle, and the distance from the front of a minor-road stopped vehicle to the edge of the major road.

Table 4.1 Minimum radius requirements of horizontal curves based on maximum superelevation and limiting values of coefficient of friction (AASHTO 2001)

Design Speed (km/h)	Minimum Radius (m)					
	e_{max}[a] = 4	e_{max} = 6	e_{max} = 8	e_{max} = 10	e_{max} = 12	Range
20	15	15	10	10	10	10-15
30	35	30	30	25	25	25-35
40	60	55	50	45	45	45-60
50	100	90	80	75	70	70-100
60	150	135	125	115	105	105-150
70	215	195	175	160	150	150-215
80	280	250	230	210	195	195-280
90	375	335	305	275	255	255-375
100	490	435	395	360	330	330-490
110	-[b]	560	500	455	415	415-560
120	-	755	665	595	540	540-755
130	-	950	830	740	665	665-950

[a] Maximum Superelevation, [b] Not Applicable

4.3 Modified Deterministic Method

The proposed modified model assumes: (1) the intersection is on a horizontal curve of a major road, (2) the angle of intersection is 90^0, (3) the major and the minor road have no grade, (4) the minor road has one lane in each direction, and (5) the major-road approaching vehicle is in the nearest lane to the minor-road vehicle. The design vehicle used in the model's development is a passenger car. The geometry is shown in Figure 4.1. The main variables involved in the model's development are:

- Distance from the front of minor-road vehicle to the edge of major road (D)
- Major-road lane width (L_{wmaj})
- Minor-road lane width (L_{wmin})
- Distance between the obstruction and the curved path of the approaching vehicle from the right/left side (M_1)
- Distance between the obstruction corner and edge of the major road (m_1)
- Distance between the obstruction and the driver's eye of the minor-road vehicle (M_2)
- Distance between the obstruction corner and the edge of the minor road (m_2)
- Number of major-road lanes (n)
- Horizontal curve radius (R)
- Radius of the horizontal curved-path of the approaching vehicle (R_n)
- Central angle for the arc with length S_a (ϕ)
- Central angle between the observer and the obstruction (ϕ_1)
- Central angle between the obstruction and the object (ϕ_2)
- Distance between the centre of the horizontal curve and the edge of the obstruction (q)
- Available sight distance (S_a)
- Median width of the major road (U)
- Vehicle width (V_w)
- Design speed on the major road (V_{maj})
- Major-road width (W_{maj})

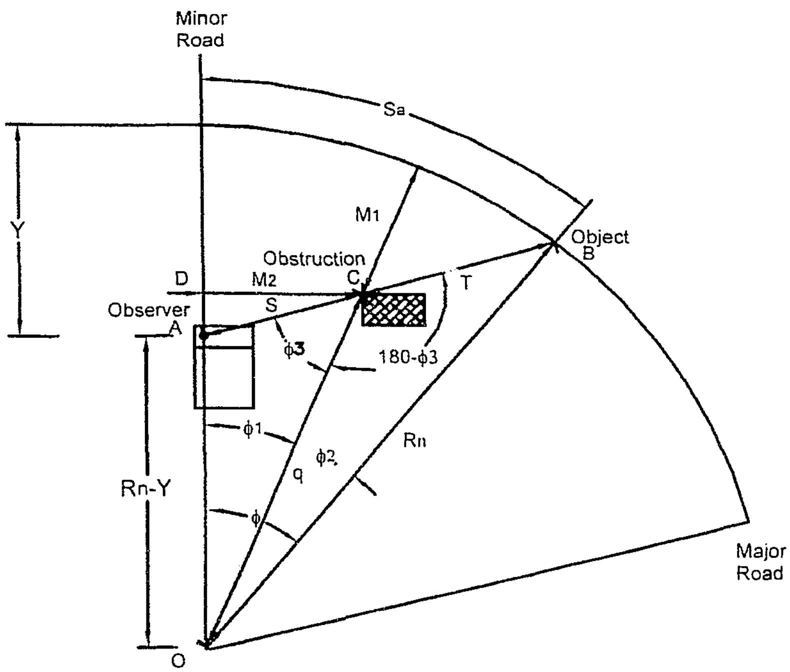


Figure 4.1 Major-Road Vehicle Approaching from the Right

- Minor-road width (W_{min})
- Distance from the minor-road driver's eye to the side/top of the approaching major-road vehicle (Y)
- Lateral distance between the left-side of the minor-road vehicle and the driver's eye (Y_i)
- Lateral distance between the left-side of the vehicle and the right-side of the lane line (Y_L)
- Distance from the minor-road driver's eye to the front of vehicle (Y_p).

4.3.1 Available Sight Distance

Available sight distance is the safe distance required to complete the manoeuvre safely. A sightline is a straight line that is between the driver's eye in the minor-road vehicle and the side/top of the approaching major-road vehicle. The available sight distance is the length of arc from the front of the major-road approaching vehicle to the point at which it may collide with the minor-road vehicle, Figures 4.1 and 4.2. This distance is given by

$$S_a = R_n \phi \tag{4.1}$$

From ΔAOC,

$$\sin\phi_1 = \frac{M_2}{q}$$

$$\phi_1 = \sin^{-1}\left(\frac{M_2}{q}\right) \tag{4.2}$$

Using the Law of Cosines for the ΔAOC, then

$$S^2 = q^2 + (R_n - Y)^2 - 2q(R_n - Y)\cos\phi_1$$

Substituting for the Φ_1 in the equation above, gives

$$S^2 = q^2 + (R_n - Y)^2 - 2q(R_n - Y)\cos\left[\sin^{-1}\left(\frac{M_2}{q}\right)\right]$$

$$S^2 = q^2 + (R_n - Y)^2 - 2q(R_n - Y)\cos\left[\sin^{-1}\left(\frac{M_2}{q}\right)\right]$$

$$S^2 = q^2 + (R_n - Y)^2 - 2q(R_n - Y)\left[\sqrt{1 - \left(\frac{M_2}{q}\right)^2}\right]$$

$$S = \sqrt{q^2 + (R_n - Y)^2 - 2(R_n - Y)\sqrt{(q^2 - M_2{}^2)}} \tag{4.3}$$

Similarly, ϕ_3 can be determined from ΔACO, as

$$(R_n - Y)^2 = S^2 + q^2 - 2Sq\cos\phi_3$$

$$\phi_3 = \cos^{-1}\left[\frac{S^2 + q^2 - (R_n - Y)^2}{2Sq}\right] \tag{4.4}$$

From ΔOCB, T can be determined as

$$R_n{}^2 = q^2 + T^2 - 2qT\cos(180 - \phi_3)$$

$$T^2 - 2qT\cos(180 - \phi_3) + q^2 - R_n{}^2 = 0$$

Solving the above quadratic equation for T, then

$$T = q\cos(180 - \phi_3) + \sqrt{R_n{}^2 - q^2 + q^2\cos^2(180 - \phi_3)}$$

$$T = -q\cos\phi_3 + \sqrt{R_n{}^2 - q^2 + q^2\cos^2\phi_3}$$

$$T = -q\cos\phi_3 + \sqrt{R_n{}^2 - q^2\sin^2\phi_3} \tag{4.5}$$

Using the Law of Cosines for ΔOCB, ϕ can be determined as

$$(S + T)^2 = (R_n - Y)^2 + R_n{}^2 - 2(R_n - Y)R_n\cos\phi$$

$$\phi = \cos^{-1}\left[\frac{(R_n - Y)^2 + R_n{}^2 - (S + T)^2}{2(R_n - Y)R_n}\right]$$

Substituting for S and T in the above Equation

$$\phi = \cos^{-1}\left[\frac{(R_n - Y)^2 + R_n{}^2 - \left[\sqrt{q^2 + (R_n - Y)^2 - 2(R_n - Y)\sqrt{(q^2 - M_2{}^2)}} - \frac{\left[S^2 + q^2 - (R_n - Y)^2\right]}{2S} + \sqrt{R_n{}^2 - q^2\left(1 - \left[\frac{S^2 + q^2 - (R_n - Y)^2}{2Sq}\right]^2\right)}\right]^2}{2(R_n - Y)R_n}\right] \tag{4.6}$$

Substituting for ϕ in Equation (4.1), then

$$S_n = R_n \cos^{-1}\left[\frac{(R_n - Y)^2 + R_n^2 - \left[X - \frac{X^2 + q^2 - (R_n - Y)^2}{2X} + \sqrt{R_n^2 - q^2\left(1 - \left[\frac{X^2 + q^2 - (R_n - Y)^2}{2Xq}\right]^2\right)}\right]^2}{2(R_n - Y)R_n}\right] \quad (4.7$$

where, $X = \sqrt{q^2 + (R_n - Y)^2 - 2(R_n - Y)\sqrt{(q^2 - M_2^{\ 2})}}$

The radius of the horizontal curved-path of the approaching major-road vehicle, R_n, is given by

$R_n = R + U/2 + Y_L$ (Vehicle approaching from right) (4.8)

$R_n = R - 0.5W_{maj} + L_{wmaj} - Y_L - V_w$ (Vehicle approaching from left) (4.9)

The obstruction distances, M_1 and M_2, are given by

$M_1 = m_1 + nL_{wmaj} + U + Y_L$ (Vehicle approaching from right) (4.10)

$M_1 = m_1 + L_{wmaj} - Y_L - V_w$ (Vehicle approaching from left) (4.11)

$M_2 = m_2 + L_{wmin} - Y_L - Y_i$ (Vehicle approaching from right) (4.12)

$M_2 = m_2 + 0.5W_{min} + Y_L + Y_i$ (Vehicle approaching from left) (4.13)

The distance from the minor-road driver's eye to the side/top of the approaching major-road vehicle, Y, is given by

$Y = Yp + D + nL_{maj} + U + Y_L$ (Vehicle approaching from right) (4.14)

$Y = Yp + D + L_{wmaj} - Y_L - V_w$ (Vehicle approaching from left) (4.15)

The distance between the center of the horizontal curve and the edge of obstruction, q, is given by

$$q = R_n - M_1 \tag{4.16}$$

It is expected that a critical case occurs when the major-road vehicle approaches from the left, on the nearest lane to the minor road vehicle (inside lane for a vehicle approaching from right and outside lane for a vehicle approaching left). Based on this assumption, Equations (4.9), (4.11), (4.13), and (4.15), are for a vehicle approaching from the left side of the major road. Substituting the values of R_n and M_1 in Equation (4.16)

$$q = R - 0.5W_{maj} - m_1 \tag{4.17}$$

The distance between the driver's eye of the minor-road vehicle and the centre of horizontal curve is given by

$$R_n - Y = R - 0.5W_{maj} - Y_p - D \tag{4.18}$$

Hence substituting for R_n, M_1, M_2, Y and q in Equation (4.7) for the critical case, then

$$S_a = R_n \cos^{-1}\left[\frac{\left(R - 0.5W_{maj} - Y_p - D\right)^2 + \left(R - 0.5W_{maj} + L_{wmaj} - Y_L - V_w\right)^2 - \left[X - \frac{X^2 + \left(R - 0.5W_{maj} - m_1\right)^2 - \left(R - 0.5W_{maj} - Y_p - D\right)^2}{2X} + \sqrt{\left(\left(R - 0.5W_{maj} + L_{wmaj} - Y_L - V_w\right)^2 - \left(R - 0.5W_{maj} - m_1\right)^2\right)\left(1 - \left[\frac{X^2 + \left(R - 0.5W_{maj} - m_1\right)^2 - \left(R - 0.5W_{maj} - Y_p - D\right)^2}{2X\left(R - 0.5W_{maj} - m_1\right)}\right]^2\right)}\right]^2}{2\left(R - 0.5W_{maj} - Y_p - D\right)\left(R - 0.5W_{maj} + L_{wmaj} - Y_L - V_w\right)}\right] \tag{4.19}$$

where, $X = \sqrt{\begin{matrix}(R - 0.5W_{maj} - m_1)^2 + (R - 0.5W_{maj} - Y_p - D)^2 - \\ 2(R - 0.5W_{maj} - Y_p - D)\sqrt{(R - 0.5W_{maj} - m_1)^2 - (m_2 + 0.5W_{min} + Y_L + Y_l)^2}\end{matrix}}$

4.3.2 Required Sight Distance

The required sight distance along the major road is given by (AASHTO 2001),

$$S_r = 0.278\, V_{maj} T_g \tag{4.20}$$

where

S_r = required sight distance for a vehicle approaching from the left or right (m),

V_{maj} = major-road design speed (km/h), and

T_g = time gap required for the minor-road stopped vehicle to manoeuvre safely (sec).

Note that the time gap depends on the type of design vehicle and the number of lanes to be crossed. Table 4.2 shows the values of the time gap for different design vehicles (AASHTO 2001).

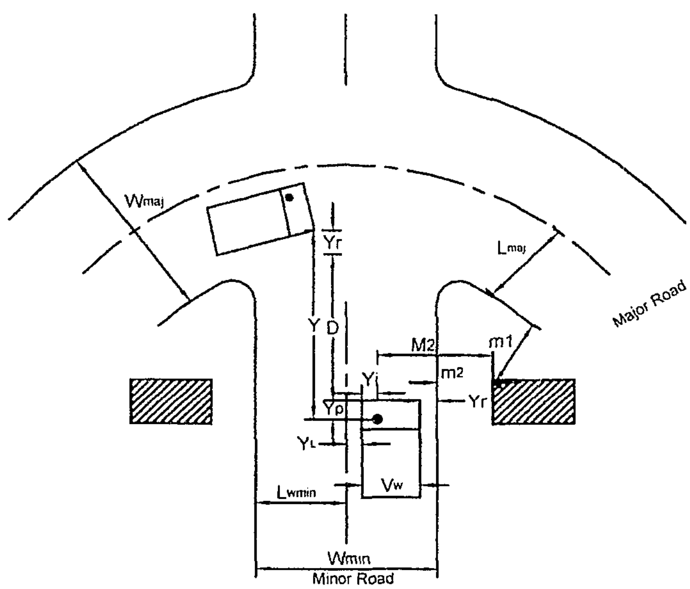


Figure 4.2 Variables of Minor and Major Roads

Table 4.2 Recommended time gap for a minor-road stopped vehicle to turn-left for at stop-control intersection (AASHTO 2001)

Design Vehicle	Time gap (sec) at major-road design speed
Passenger Car	7.50
Single-Unit Truck	9.50
Combination Truck	11.50

Note: Time gaps are for a stopped vehicle to turn left onto a 2-lane highway with no median and grades 3 % or less. For multilane highways and grades on minor road approaches, an adjustment is required.

4.4 Modelling Using Reliability Analysis

The FOSM method requires two moments (mean and variance) as described already in Chapter 3. In the reliability analysis, it is important to determine the random variables. The proposed model involves the following random variables:

- Vehicle width (V_w)
- Distance between the driver's eye and the front of the minor-road vehicle (Y_p)
- Distance between the driver's eye and the left side of the minor-road vehicle (Y_l)
- Distance between left side of the vehicle and the right side of the lane line (Y_L)
- Distance between the front of minor-road vehicle and the edge of the major road pavement (D)
- Speed of the major-road vehicle (V_{maj})
- Time gap for the stopped minor-road vehicle (T_g)

4.4.1 Mean of Available Sight Distance

The mean of the available sight distance was determined using Equation (3.5). By replacing the extreme values of the random variables with mean values for Equation (4.19), the expected value of S_a, $E[S_a]$, is given by

$$E[S_a] = R_n Cos^{-1}\left[\frac{\left(R - 0.5W_{maj} - \mu_{Y_p} - \mu_D\right)^2 + \left(R - 0.5W_{maj} + L_{wmaj} - \mu_{Y_L} - \mu_{V_w}\right)^2 - \left[X - \frac{X^2 + \left(R - 0.5W_{maj} - m_1\right)^2 - \left(R - 0.5W_{maj} - \mu_{Y_p} - \mu_D\right)^2}{2X} + \sqrt{\frac{\left(R - 0.5W_{maj} + L_{wmaj} - \mu_{Y_L} - \mu_{V_w}\right)^2 - \left(R - 0.5W_{maj} - m_1\right)^2}{\left(1 - \left[\frac{X^2 + \left(R - 0.5W_{maj} - m_1\right)^2 - \left(R - 0.5W_{maj} - \mu_{Y_t} - \mu_D\right)^2}{2X\left(R - 0.5W_{maj} - m_1\right)}\right]^2\right)}}\right]^2}{2\left(R - 0.5W_{maj} - \mu_{Y_p} - \mu_D\right)\left(R - 0.5W_{maj} + L_{wmaj} - \mu_{Y_L} - \mu_{V_w}\right)}\right] \tag{4.21}$$

where the µ's are the mean of the random variables and X is given by

$$X = \sqrt{\left(R - 0.5W_{maj} - m_1\right)^2 + (R - 0.5W_{maj} - \mu_{Y_p} - \mu_D)^2 - 2(R - 0.5W_{maj} - \mu_{Y_p} - \mu_D)\sqrt{\left(R - 0.5W_{maj} - m_1\right)^2 - \left(m_2 + 0.5W_{min} + \mu_{Y_L} + \mu_{Y_t}\right)^2}}$$

The higher-order terms of the Taylor's series expansion of Equation (3.3), were added in the expected value formula and found to be negligible in previous studies (Easa 1993 and Easa and Hussain 2004) as well as in the current study.

4.4.2 Variance of Available Sight Distance

The variance is the second moment used in the FOSM reliability method. Using Equation (4.19), the variance of S_a, Var[S_a] from Equation (3.6), is given by the following:

$$Var[S_a] = \left(\frac{\partial S_a}{\partial V_w}\right)^2 \sigma^2_{V_w} + \left(\frac{\partial S_a}{\partial Y_L}\right)^2 \sigma^2_{Y_L} + \left(\frac{\partial SD_a}{\partial Y_i}\right)^2 \sigma^2_{Y_i} + \left(\frac{\partial S_a}{\partial Y_p}\right)^2 \sigma^2_{Y_p} + \left(\frac{\partial S_a}{\partial D}\right)^2 \sigma^2_D +$$
$$2\left(\frac{\partial S_a}{\partial V_w}\right)\left(\frac{\partial S_a}{\partial Y_L}\right)\mathrm{Cov}(V_w, Y_L) + 2\left(\frac{\partial S_a}{\partial V_w}\right)\left(\frac{\partial S_a}{\partial Y_i}\right)\mathrm{Cov}(V_w, Y_i) \tag{4.22}$$

where σ's are the standard deviations of the random variables. In Equation (4.22), the first derivatives are evaluated at the mean values of random variables. The first derivatives of $\frac{\partial S_a}{\partial V_w}, \frac{\partial S_a}{\partial Y_L}, \frac{\partial S_a}{\partial Y_i}, \frac{\partial S_a}{\partial Y_p}$ and $\frac{\partial S_a}{\partial D}$ were obtained by using a mathematical software package, called Mathematica (Wolfram Research Inc.). As the results of all the derivatives are too long, the derivative of $\frac{\partial S_a}{\partial V_w}$ is included in the thesis for the purpose of illustration. It is given in the Appendix B. The covariances are given by

$$\mathrm{cov}(V_w, Y_L) = \sigma_{Vw}\sigma_{YL}\rho_{VwYL} \tag{4.23}$$
$$\mathrm{cov}(V_w, Y_i) = \sigma_{Vw}\sigma_{Yi}\rho_{VwYi} \tag{4.24}$$

where ρ_{VwYL} = co-efficient of correlation between the random variables V_w and Y_L and ρ_{VwYi} = co-efficient of correlation between the random variables V_w and Y_i. Note that there is a negative covariance between the random variables V_w and Y_L and positive covariance between the random variables V_w and Y_i.

4.4.3 Mean of Required Sight Distance

The mean of the required sight distance was obtained by replacing the extreme values of the random variables with the mean values. From Equation (4.20), the expected value of S_r, $E[S_r]$, is given by

$$E[S_r] = 0.278\ \mu_{vmaj}\ \mu_{Tg} \tag{4.25}$$

where μ_{Vmaj} and μ_{Tg} are the mean values of V_{maj} and T_g respectively.

4.4.4 Variance of Required Sight Distance

The variance of S_r, $Var[S_r]$ based on Equation (3.7a), is given by

$$Var[S_r] = \left(\frac{\partial S_r}{\partial Vmaj}\right)^2 \sigma^2_{V_{maj}} + \left(\frac{\partial S_r}{\partial T_g}\right)^2 \sigma^2_{T_g} \tag{4.26}$$

where the partial derivatives are given by

$$\frac{\partial S_r}{\partial V} = 0.278\,\mu_{T_g} \tag{4.27}$$

$$\frac{\partial S_r}{\partial T_g} = 0.278\,\mu_{V_{maj}} \tag{4.28}$$

4.4.5 Probability of Failure

As S_a and S_r are statistically independent and the probability of failure for the ISD can be determined by substituting the related parameters in Equation (3.13),

$$\beta = \frac{E[S_a] - E[S_r]}{\sqrt{Var[S_a] + Var[S_r]}} \tag{4.29}$$

The values of $E[S_a]$ and $E[S_r]$ were determined using Equations (4.21) and (4.25) and the variance of S_a and S_r were determined using Equations (4.22), and (4.26), respectively. The reliability index was computed using Equation (4.29).

Using Equation (3.15), the probability of failure becomes

$$P_f = 1 - \phi\left[\frac{E[S_a] - E[S_r]}{\sqrt{Var[S_a] + Var[S_r]}}\right] \tag{4.30}$$

where Φ is the CDF of the standard normal variate.

4.4.6 Guideline for Data Preparation

The FOSM method used for reliability analysis in this research requires data based on the two moments (mean and variance). Some guidelines for data preparation were developed for use in the proposed analysis. Suppose that there are 'n' types of vehicles using a particular intersection. Let a given random variable (e.g. vehicle width) be denoted by R. The percentage frequencies of all types of vehicles are f_1, f_2, ..., f_n where $\sum_{i=1}^{n} f_i$ = 100 and the corresponding values of the vehicle widths are R_1, R_2, ..., R_n. Then the mean and the standard deviation of R, ($\overline{R}$ and σ_R) are given by

$$\overline{R} = \frac{\sum_{i=1}^{n} f_i R_i}{100} \tag{4.31}$$

$$\sigma_R = \sqrt{\frac{\sum_{i=1}^{n} \left(R_i - \overline{R}\right)^2}{n - 1}} \tag{4.32}$$

The reliability method requires data on the means, variances, and correlations of various random variables. The means and standard deviations can be determined through observations at the intersection being analyzed. To determine the mean of other random variables, extreme values with respect to the percentile values can be used. Assuming that the random variables are normally distributed, the relationship between the mean and extreme values is given by

$$\mu_{xi} = \frac{E_{xi}}{(1 + ZCV_{xi})} \tag{4.33}$$

where μ_{Xi} = mean value of random variable Xi, E_{Xi} = extreme value of random variable corresponding to a certain percentile value, Z = number of standard deviations of the normal distribution corresponding to a certain percentile value, and CV_{Xi} = co-efficient of variation of random variable X_i.

Note that Z is positive (negative) for variables for which the extreme values are based on a high (low) percentile value. For example, the Z value of any random variable with respect to the 95th percentile value will be 1.64 and the 5th percentile value for the same random variable will be –1.64. For large values of the standard normal variate, tables of the normal distribution do not provide the area under the distribution for the fine values of the variate. The following least-squares approximation can be used (Easa 1992).

$$\beta = -0.615 + [0.378 - 2.199(0.841 + \ln P_f)]^{0.5} \qquad P_f \leq 0.1 \tag{4.34}$$

$$P_f = \exp(-0.841 - 0.558\beta - 0.455\beta^2) \qquad \beta \geq 1.0 \tag{4.35}$$

4.5 Model Verification

4.5.1 Verification of modified Deterministic Model

The modified deterministic model developed in this research was verified graphically using AUTOCAD, and mathematically using ISD values of AASHTO (2001), for the straight intersections. The values of R_n and Y were computed by using Equation (4.9) and (4.15), as 197.29 m and 6.29 m respectively. For the graphical verification, an on scale arc of radius 197.29 m and a line crossing through the center of the circle were drawn. The arc showed the moving path of a major-road vehicle approaching from the left side. The vertical line represented the minor-road vehicle path. The observer was placed on the minor road 6.29 m from the moving path of the major-road vehicle. The object location was found from the following relationship:

$$\phi = \frac{S_r}{R_n} \tag{4.36}$$

where S_r = 125.1 m using Equation (4.20) for V_{maj} = 60 km/h and T_g = 7.5 sec (passenger car).

The central angle between the minor-road driver's eye and the object was calculated using Equation (4.36) as 0.6341 radians. Other geometric data were used from Table 4.3. A line of sight was established between the object and the observer.

An obstruction was placed anywhere on the line of sight. The values of m_2 and m_1 were measured graphically. The value of m_2 was inputted to the software developed in Microsoft Excel and the value of m_1 was determined by iteration for the condition S_r - S_a = 0. The graphical values were compared with those obtained analytically. The graphical and analytical results were identical.
For the mathematical verification, the required sight distance was calculated using Equation (4.20) as 166.8m, for major-road design speed of 80 km/h and a minor-road stopped vehicle (passenger car). A departure triangle was made for a vehicle approaching from the left whose leg length along the major road was 166.8 m and whose leg length along the minor road was 6.29 m.

An obstruction was placed at m_2 = 20 m. The value of M_2 = 24.743 m was computed using Equation (4.13). By interpolating, the value of M_1 was calculated as 5.357 m and from Equation (4.11), the value of m_1 was determined to be 4.667 m.

In the modified deterministic model, a very large radius of $1x10^7$ m was used. This made the major-road horizontal curve virtually flat. The value of $m_2 = 20$ m was input, and $m_1 = 4.671$ m was calculated using the "Solver" tool of Excel for the condition of $S_r - S_a = 0$. The results showed that the model added 0.08% to the value of m_1 which is negligible. This shows that the mathematical model was working well.

4.5.2 Verification of Reliability Model

The reliability model developed in this research is based on the modified deterministic method which was verified in the preceding section. In the reliability analysis, the variation coefficient is an important parameter for expressing the relative measure of dispersion of the data around the mean value. The details of the variation coefficient were discussed in Chapter 3.

For the specific dataset of Tables 4.3 and 4.5, design graphs illustrated a specific design speed of 60 km/h and probability of failure 5%, CV = 5% and 1%, Figures 4.3-4.4. The graphs show that by decreasing the CV of all random variables, the results calculated by the reliability model are, as expected, getting closer to the results calculated by the deterministic model. The results also show that for the case when CV = 0, the values of all variables approach the extreme values and correspond to the modified deterministic model, Figure 4.5.

4.6 Sensitivity Analysis and Comparison of Models

4.6.1 Sensitivity Analysis of Random Variables

A sensitivity analysis was performed to check the effect of the variations in the random variables on the obstruction location, m_1 and m_2. This analysis also provides information about the importance of the random variables. The base case used for the sensitivity analysis with m_2 = 8 m, CV = 10% for all random variables, ρ_{VwYL} = 0.5, ρ_{VwYi} = 0.5, and P_f = 5% (Table 4.5).

For the sensitivity analysis, three groups of design speeds were selected: 40 km/h, 60 km/h, and 100 km/h. The effects of a 20% increase in the mean value of each random variable were computed and are shown in Tables 4.6 - 4.8. These tables show that as the mean values of the random variables increase, the m_1 increases. The most sensitive random variables are major-road speed and the time gap, distance between the front of minor-road vehicle and the edge of the major road, distance between the minor-road driver eye and the front of vehicle, vehicle width, and distance between left side of the vehicle and the right side of the lane line. The most sensitive random variable is distance between the minor-road driver eye and the left side of the vehicle. At lower major-road speeds, the effect of the increase in the mean values is greater as compared with higher speeds.

The results of the sensitivity analysis of the variation coefficient of all random variables to m_1 are shown in Table 4.9. The effect of the variation coefficient of the major-road speed only (keeping CV constant for all other random variables) were also examined (Table 4.10). The results indicate that by decreasing the variation coefficient of all random variables, the corresponding value of m_1 increases, and vice versa. Similar effects were found by varying the variation coefficient of speed only. The effect of the variation coefficient on m_1 decreases as the radius increases. The effect of the correlation coefficients ρ_{VwYL} and ρ_{VwYi} on m_1 is shown in Table 4.11. As the sensitivity analysis for the correlation coefficient indicated that m_1 is quite insensitive to the correlation coefficients, intermediate values of the correlation coefficient may be used.

4.6.2 Comparison between Modified Deterministic and Reliability Models

A hypothetical example was used for the comparison of the modified deterministic and reliability models. The data used were: R = 400 m, V_{maj} = 60 km/h, T_g = 7.5 sec (passenger car), and CV = 10%. The values of the other variables and random variables were those shown in Tables 4.3 and 4.5. Probability of failures of 0.001%, 0.01%, 0.1%, 1%, 5%, 10% and 20% were used for illustration.

The values of the modified deterministic model were very close to P_f = 0.1% at all radii. It was interesting, however, that for values of m_2 of less than 10 m, the deterministic curve was closer to P_f = 1% for almost all radii,. This implies that the values obtained from the modified deterministic model exhibit a higher probability of failure when the obstruction is closer to the observer. Figure 4.6 compares the modified deterministic and reliability models at different probabilities of failure.

4.7 Establishing Design Graphs

4.7.1 Design graphs of Modified Deterministic Model

Since stop-control intersections are likely to exist on 2-lane rural and urban highways, both the major and minor roads are assumed to be 2-lane roadways. It is also assumed that the minor road has no skew or grade, Figure 4.2. The vehicle width (passenger car) was obtained from AASHTO (2001) and the values of the other variables were obtained from the literature. The database used to illustrate the design graphs is presented in Table 4.3 for a major road with a horizontal curve and for a minor road with level grade. The radii of curvature were selected using the criteria provided in AASHTO (2001), as shown in Table 4.1. The variables used for a typical intersection are:

Major-road lane width, $L_{wmaj} = 3.6$ m

Minor-road lane width, $L_{wmin} = 3.6$ m

Major-road width, $W_{maj} = 7.2$ m

Minor-road width, $W_{min} = 7.2$ m

Number of lanes on the major road, n = 2

Distance from edge of the major road to the front of the minor-road vehicle, D = 3 m.

The extreme values of the random variables used in the analysis are, $V_w = 2.1$ m, $Y_P = 2.4$ m, $Y_i = 0.533$ m, $Y_L = 0.61$ m, Tg = 7.5 sec (passenger car) and D = 3 m. Substituting the above values in Equation (4.19), a simple form of this equation is obtained as

$$S_a = (R-2.71)Cos^{-1}\left[\frac{(R-9)^2 + (R-2.71)^2 - \left[X - \frac{X^2 + (R-3.6-m_1)^2 - (R-9)^2}{2X} + \sqrt{\left((R-2.71)^2 - (R-3.6-m_1)^2\right)\left(1 - \left[\frac{X^2 + (R-3.6-m_1)^2 - (R-9)^2}{2X(R-3.6-m_1)}\right]^2\right)}\right]^2}{2(R-9)(R-2.71)}\right] \tag{4.37}$$

where X is given by

$$X = \sqrt{(R-3.6-m_1)^2 + (R-9)^2 - 2(R-9)\sqrt{(R-3.6-m_1)^2 - (m_2+4.743)^2}}$$

By comparing Equations (4.37) and (4.20), we obtained the following:

$$0.278V_{maj}T_g = (R-2.71)Cos^{-1}\left[\frac{(R-9)^2 + (R-2.71)^2 - \left[X - \frac{X^2 + (R-3.6-m_1)^2 - (R-9)^2}{2X} + \sqrt{\left((R-2.71)^2 - (R-3.6-m_1)^2\right)\left(1 - \left[\frac{X^2 + (R-3.6-m_1)^2 - (R-9)^2}{2X(R-3.6-m_1)}\right]^2\right)}\right]^2}{2(R-9)(R-2.71)}\right] \tag{4.38}$$

where X is given by

$$X = \sqrt{(R-3.6-m_1)^2 + (R-9)^2 - 2(R-9)\sqrt{(R-3.6-m_1)^2 - (m_2+4.743)^2}}$$

Here V_{maj}, T_g, R, are the input variables to determine the m_1 or m_2. For a given design speed and radius of the curvature, m_1 or m_2 were calculated by inputting one of them in Equation (4.38). A working sheet was prepared in Excel and tool "Solver" was used for iterations to determine the values of m_1 for respective values of m_2 for the condition $S_a - S_r = 0$. For a specific radius and design speed, by inputting m_2 values, the corresponding m_1 values were computed and the design graphs were developed.

The design graphs based on the modified deterministic method are shown in Appendix C, Figures C4.1-C4.4. The graphs provide the m_1 on (y-axis) corresponding to m_2 on (x-axis). Each graph represents the m_1 and m_2 for a set of R = 200 m, 400 m, 600 m and 800 m, at a given design speed. It should be noted that the graphs are shown for major-road design speeds of 40 km/h, 60 km/h, 80 km/h and 100 km/h. For a particular design speed and radius of horizontal curve, if m_1 (m_2) is known, it may be plotted on the corresponding graph so that the minimum value of m_2 (m_1) can be determined.

Note that for flat horizontal curves with radii greater than 200 m, the rate of increase in m_1 is likely to decrease with the increase in m_2 especially at lower speeds. A negative slope of the curve in the design graph indicates that the closer corner of obstruction to the minor-road vehicle controls, and vice versa when the farther corner of obstruction controls.

4.7.2 Data used for Reliability Analysis

The means and standard deviations of the random variables can be determined through observations at the intersection being analyzed. For analysis purposes, extreme values with respect to the percentile values of the random variables were used to determine the mean values, assuming that all random variables were normally distributed. The mean values were determined from Equation (4.33). The standard deviations were calculated by using Equation (3.9).

The extreme values of the random variables are V_w = 2.1 m (99th percentile, Z =2.32), Y_p = 2.4 m (85th percentile, Z = 1.013), Y_i = 0.533 m (99th percentile, Z = 2.32), Y_L = 0.61 m (95th percentile, Z = 1.64), D = 3.0 m (85th percentile, Z = 1.013), T_g = 7.5 sec. (85th percentile, Z = 1.013). $\rho_{VwYL} = \rho_{VwYi}$ = 0.5. The mean and the standard deviation of the major-road speed were computed assuming that the extreme values represent the 99.87th percentile (Z = 3). Substituting the mean values in Equation (4.21), and using CV = 10% for all variables, a simple form of the expected available sight distance is obtained

$$E[S_a] = (R-2.233)Cos^{-1}\left[\frac{(R-8.503)^2+(R-2.233)^2-\left[X-\frac{X^2+(R-3.6-m_1)^2-(R-8.503)^2}{2X}+\sqrt{\frac{(R-2.233)^2-(R-3.6-m_1)^2}{\left(1-\left[\frac{X^2+(R-3.6-m_1)^2-(R-8.503)^2}{2X(R-3.6-m_1)}\right]^2\right)}}\right]^2}{2(R-8.503)(R-2.233)}\right] \tag{4.39}$$

where X is given by

$$X = \sqrt{(R-3.6-m_1)^2+(R-8.503)^2-2(R-8.503)\sqrt{(R-3.6-m_1)^2-(m_2+4.561)^2}}$$

4.7.3 Design Graphs of Reliability Analysis

Table 4.4 shows the values of the reliability index corresponding to the probability of failure used in the analysis. Intermediate correlation coefficient values of 0.5 were used. Design graphs were developed for the probability of failures of 0.1%, 1%, 5%, and 10%, and for CV = 5% and 10%, using the same range of major-road design speeds and radii.

The model could not be shown in closed form due to the very long equations. A computer program was developed in Excel. For a specified design speed, radius, variation coefficient of all variables, probability of failure and m_2, the corresponding value of m_1 was computed by iterations for a specified value of the reliability index. Only m_2 was changed and the respective m_1 was computed by iterations. Design graphs were developed for m_2 on the x-axis and m_1 on the y-axis for different design speeds, CV, and probabilities of failure.

Figures D4.1 - D4.8 correspond to P_f = 0.1%, Figures D4.9 - D4.16 correspond to P_f = 1%, Figures D4.17 - D4.24 correspond to P_f = 5%, and Figures D4.25-D 4.32 correspond to P_f = 10%. CV = 5% and CV = 10% were used for each set of design speeds and probability of failure. The radius of 100 m was included only in the design graphs with design speed of 40 km/h.

Note that for the lowest probability of failures, m_1 increases with the increase in the variation coefficient of all random variables, but that for the higher probability of failure (e.g. 5% or greater), m_1 decreases with the increase in the variation coefficient of all random variables. The design graphs are very easy to use and can be used to analyze an existing or proposed intersection.

Table 4.3 Extreme Values of data used for modified deterministic method

Variables	Extreme Values		Reference
	Value	Percentile	
V_w	2.1 m	99^{th}	AASHTO (2001)
Y_p	2.4 m	85^{th}	NCHRP-383 (1996)
Y_i	0.533 m	99^{th}	McCoy (1997)
Y_L	0.61 m	95^{th}	AASHTO (2001)
T_g	7.5 sec	85^{th}	NCHRP-383 (1996)
D	3 m	85^{th}	NCHRP-383 (1996)

Table 4.4 Probability of failure and reliability index

Probability of Failure	Reliability Index (β)
0.001	4.75
0.01	3.72
0.1	3.10
1	2.327
5	1.645
10	1.286

Table 4.5 Input data used for the base case

Mean and Standard Deviation of Random Variables			
Variables	Mean	Standard Deviations	CV
V_w	1.705	0.170	0.10
Y_P	2.179	0.218	0.10
Y_i	0.433	0.043	0.10
Y_L	0.524	0.052	0.10
T_g	6.810	0.681	0.10
D	2.724	0.272	0.10

Table 4.6 Sensitivity of m_1 to the mean values of random variables (Design speed = 40 km/h)

Variables	Effect on m_1 values due to 20 % increase in the mean value of each Random Variable[a]							
	R = 100 M (Base case m_1 = 7.94 m)		R = 200 M (Base case m_1 = 5.89 m)		R = 400 M (Base case m_1 = 4.99 m)		R = 800 M (Base case m_1 = 4.62 m)	
	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
V_w	8.0	+ 1.1	6.0	+ 1.1	5.1	+ 1.4	4.7	+1.5
Y_p	8.3	+ 4.9	6.3	+ 6.5	5.4	+ 7.9	5.0	+ 8.6
Y_i	8.0	+ 0.1	5.9	- 0.1	5.0	- 0.1	4.6	- 0.3
Y_L	8.0	+ 0.5	5.9	+ 0.3	5.0	+ 0.3	4.6	+ 0.2
T_g	9.2	+ 16.2	6.6	+ 11.1	5.4	+ 8.5	4.9	+ 6.7
D	8.4	+ 6.3	6.4	+ 8.3	5.5	+ 10.1	5.1	+ 11.2
V	9.2	+16.2	6.6	+11.1	5.4	+ 8.5	4.9	+ 6.7

[a] Base case Table 4.5, m_2 = 8 m and P_f = 5%
[b] Required m_1 due to an increase of 20% in the mean values of each random variable (the mean value of other variables remain unchanged)

Table 4.7 Sensitivity of m_1 to the mean values of random variables (Design speed = 60 km/h)

Variables	Effect on m_1 values due to 20 % increase in the mean value of each Random Variable[a]					
	R = 200 M (Base case m_1 = 7.45 m)		R = 400 M (Base case m_1 = 5.94 m)		R = 800 M (Base case m_1 = 5.28 m)	
	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
V_w	7.5	+ 0.7	6.0	+ 0.7	5.3	+ 0.9
Y_p	7.9	+ 5.6	6.6	+ 7.1	5.7	+ 8.2
Y_i	7.5	+ 0.4	5.9	0	5.3	0
Y_L	7.5	+ 0.4	6.0	+ 0.3	5.3	0
T_g	8.3	+ 11.5	6.4	+ 7.7	5.6	+ 5.4
D	8.0	+ 7.1	6.5	+ 9.0	5.8	+ 10.6
V	8.3	+ 11.5	6.4	+ 7.7	5.6	+ 5.4

[a] Base case Table 4.5, m_2 = 8 m and P_f = 5%
[b] Required m_1 due to an increase of 20% in the mean values of each random variable (the mean value of other variables remain unchanged)

Table 4.8 Sensitivity of m_1 to the mean values of random variables (Design speed = 100 km/h)

Variables	Effect on m_1 values due to 20 % increase in the mean value of each Random Variable[a]					
	R = 200 M (Base case m_1 =10.3 m)		R = 400 M (Base case m_1 = 7.4 m)		R = 800 M (Base case m_1 =6.1 m)	
	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
V_w	10.7	+0.4	7.4	0	6.1	0
Y_p	10.8	+ 4.2	7.8	+ 6.0	6.6	+ 7.4
Y_i	10.4	+ 0.3	7.4	0	6.1	0
Y_L	10.4	+ 0.5	7.4	0	6.1	0
T_g	11.9	+ 15.0	8.0	+ 9.0	6.4	+ 5.6
D	10.9	+ 5.4	7.9	+ 7.620	6.7	+ 9.4
V	11.9	+ 15.0	8.0	+ 9.0	6.4	+ 5.6

[a] Base case Table 4.5, m_2 = 8 m and P_f = 5%
[b] Required m_1 due to an increase of 20% in the mean values of each random variable (the mean value of other variables remain unchanged)

Table 4.9 Sensitivity of m_1 to variation coefficient of all random variables

CV	Effect to m_1 due to variation in coefficient of variation[a]					
	R = 200 M (Base case m_1 =7.45 m)		R = 400 M (Base case m_1 = 5.94 m)		R = 800 M (Base case m_1 = 5.28 m)	
	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
1%	8.3	+ 11.96	6.5	+ 9.21	5.6	+ 6.15
5%	7.9	+ 6.10	6.2	+ 4.63	5.4	+ 2.89
20%	6.8	- 9.32	5.6	- 6.55	5.2	- 2.05

[a] Base case Table 4.5, m_2 = 8 m, design speed= 60 km/h and P_f = 5%
[b] Required m_1 due to change in the CV of each random variable (the CV of other variables remain unchanged)

Table 4.10 Sensitivity of m_1 due to variation in the variation coefficient of design speed

CV	Effect to m_1 due to variation in coefficient of variation of speed[a]					
	R = 200 M (Base case m_1 = 7.45 m)		R = 400 M (Base case m_1 = 5.94 m)		R = 800 M (Base case m_1 = 5.28 m)	
	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
1%	8.3	+ 11.9	6.4	+ 8.3	5.6	+ 6.2
5%	7.9	+ 5.5	6.2	+ 4.0	5.4	+ 3.1
20%	7.0	- 6.7	5.6	- 5.3	5.1	- 4.4

[a] Base case Table 4.5, m_2 = 8 m, design speed= 60 km/h and P_f = 5%

[b] Required m_1 due to change in CV of design speed only (the CV of other variables remain unchanged)

Table 4.11 Sensitivity to m_1 to the correlation coefficient of correlated random variables

Correlation Coefficients		Effect to m_1 due to variation in correlation coefficient[a]					
		R = 200 M (Base case m_1 = 7.45 m)		R = 400 M (Base case m_1 = 5.94 m)		R = 800 M (Base case m_1 = 5.28 m)	
ρ_{VwYL}	ρ_{VwYi}	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)	m_1[b]	Diff. (%)
1.0	0.0	7.5	0	5.9	0	5.3	0
0.0	1.0	7.5	0	5.9	0	5.3	0
1.0	1.0	7.5	0	5.9	0	5.3	0
0.0	0.0	7.5	0	5.9	0	5.3	0

[a] Base case Table 4.5, m_2 = 8 m, design speed= 60 km/h and P_f = 5%

[b] Required m_1 due to change in correlation coefficient of correlated random variables

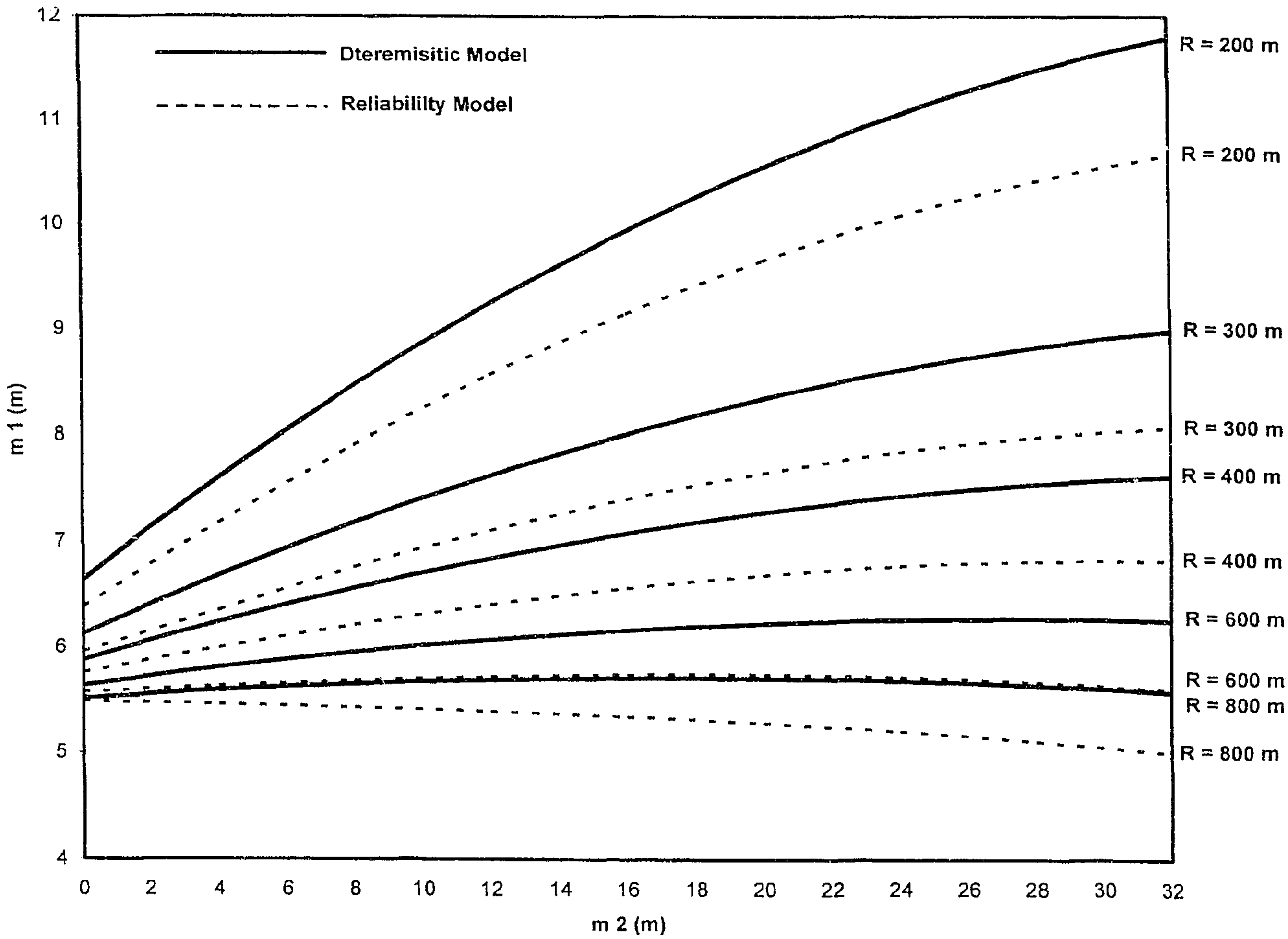


Figure 4.3 Comparison of Modified Deterministic and Reliability Models at P_f = 5%, V_{maj} = 60 km/h, CV = 5% for all random variables.

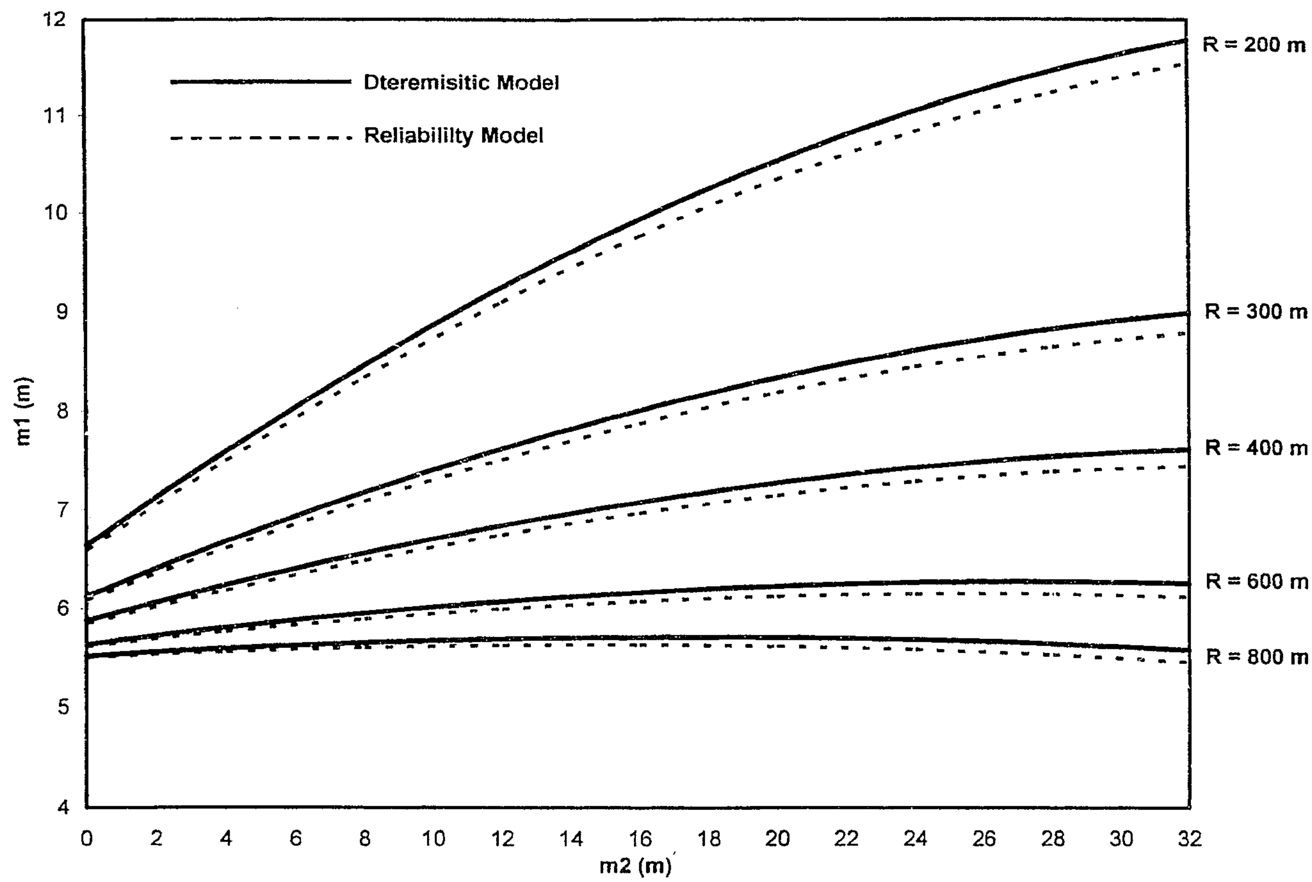


Figure 4.4 Comparison of Modified Deterministic and Reliability Models at P_f = 5%, V_{maj} = 60 km/h, CV = 1% for all random variables.

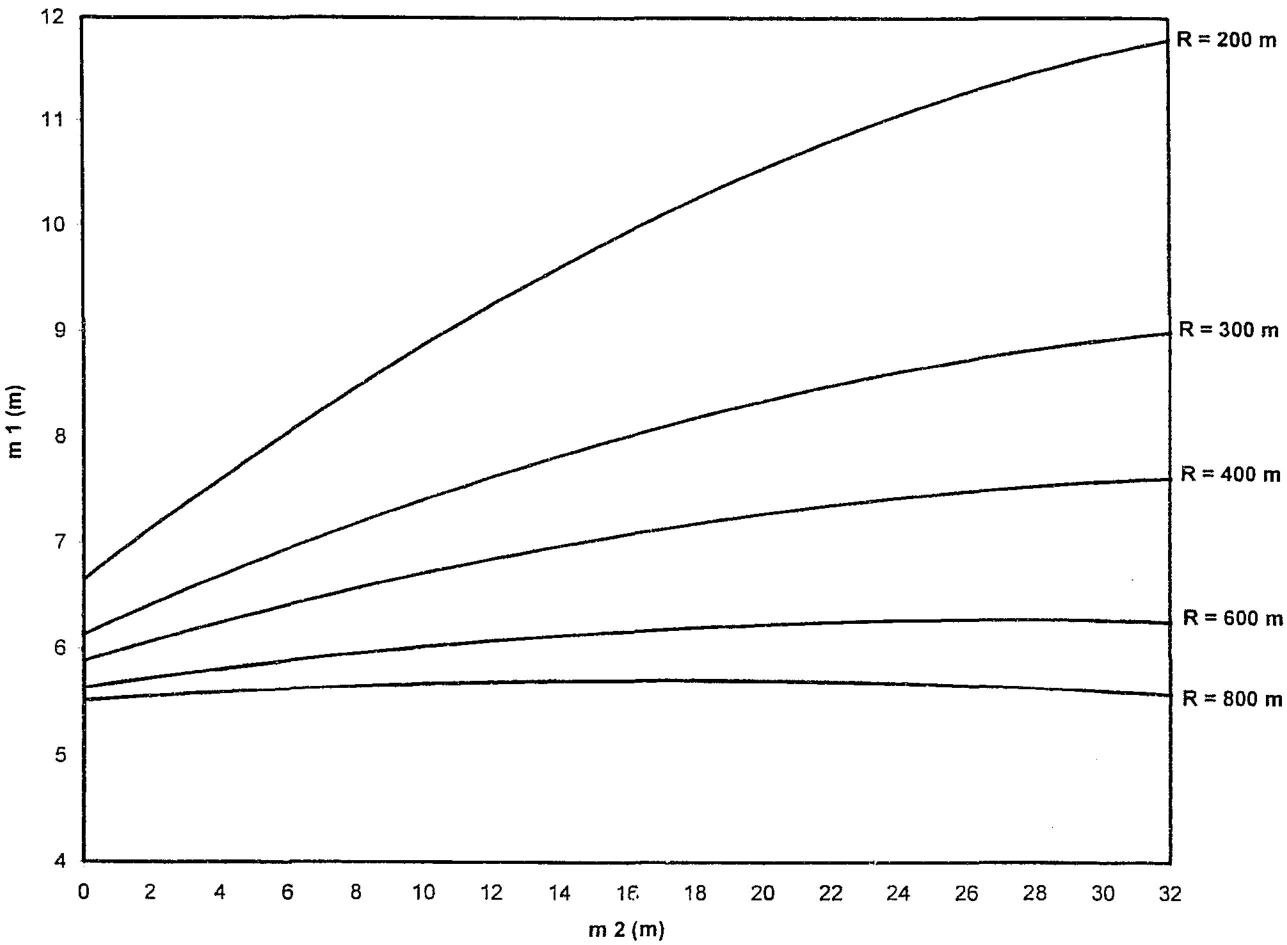


Figure 4.5 Comparison of Modified Deterministic and Reliability Models at V_{maj} = 60 km/h, CV = 0% for all random variables

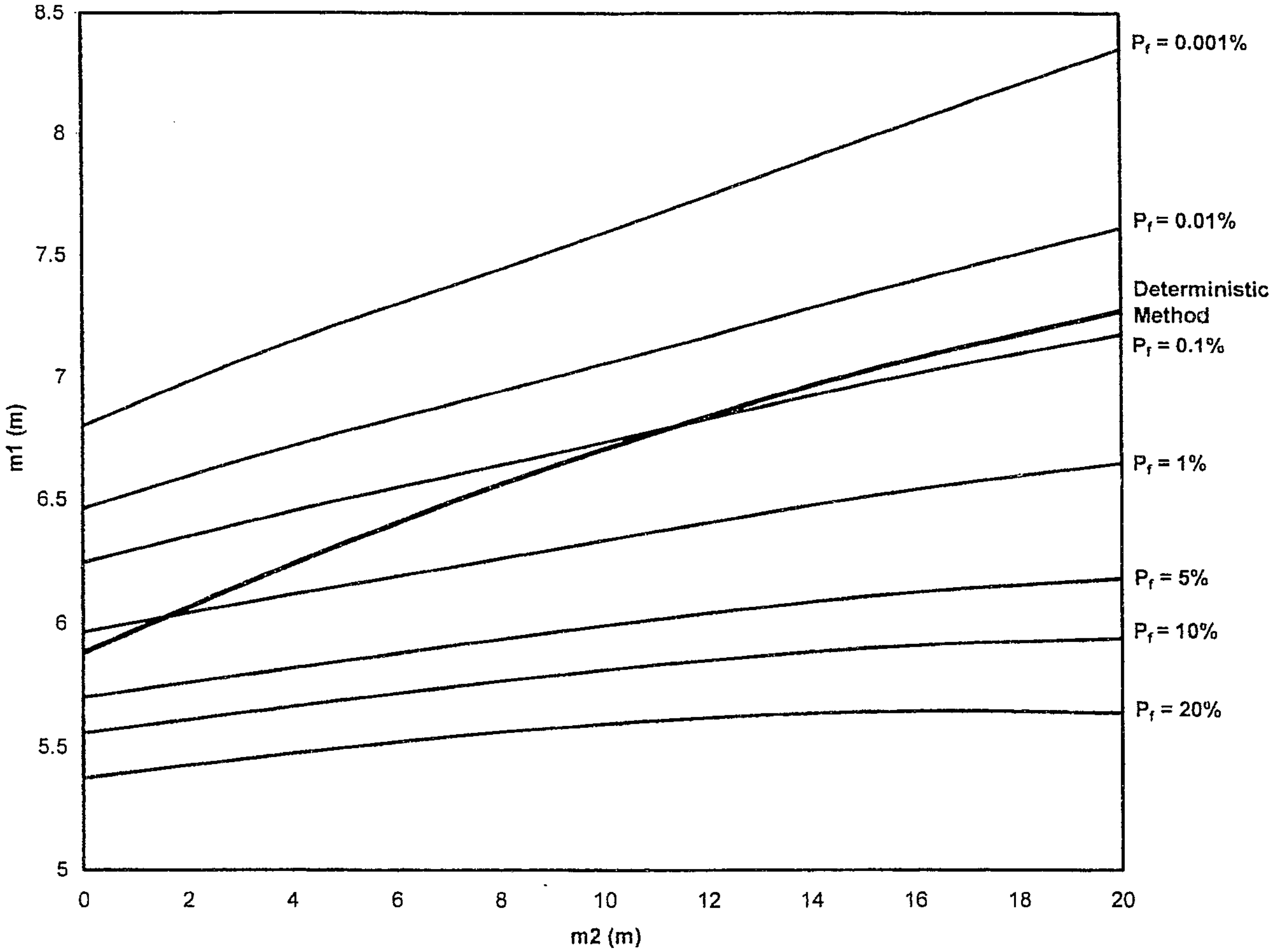


Figure 4.6 Comparison of Modified Deterministic and Reliability Model at various probabilities of failure for R = 400 m, V_{maj} = 60 km/h, CV = 10% for all variables

Chapter 5: PRACTICAL APPLICATION

The purpose of this chapter is to illustrate the application of the modified deterministic and reliability models. An actual intersection, located in the City of Toronto (Dundas Street and Pembroke Street), was used for this purpose. In addition, a hypothetical example was used to illustrate the use of the developed design aids.

5.1 Actual Intersection

It is important to mention here that the intersection of Dundas Street (major-road) and Pembroke Street (minor-road) is not directly applicable to the developed models as the Pembroke Street operates one-way only (southbound), which means that the vehicles on the minor-road travel towards the center of the horizontal curve and therefore, the obstruction is critical for the minor-road vehicle on the outside of the curve. For the purpose of illustration, it was assumed that the traffic on the Pembroke Street is not restricted to one-way. The reliability of the intersection was evaluated for the vehicle on the inside of the horizontal curve

5.1.1 Intersection Geometry

At the intersection of Dundas and Pembroke, Dundas, the major road, is a four-lane undivided highway (East-West bound) and Pembroke, the minor road, is a two-lane undivided residential street (North-South bound). There is a pedestrian crossing with mounted flashers at a mid block location on the major road near the intersection. It should be noted that street cars run in the central two lanes of the major road. As shown in Figure 5.1, the major-road has a horizontal curve right on the intersection. Although a little skew is present at the location, it was assumed for simplicity that the skew is zero. It was also assumed that there was no slope on the major or minor road. Some geometric variables were measured physically at the site and others were taken from the geometric drawing obtained from the Toronto Works Department. A scanned image of the geometric drawing is shown in Figure 5.1. Details of the geometric input data are given in Table 5.1. Note that this intersection has typical road widths (3.6 m for the minor and major road). The following sections present the procedure followed by the analysis in detail.

5.1.2 Model Application

5.1.2.1 Modified Deterministic Model

The radius of the horizontal curve (centerline of the major road) was taken as the average of the radii of curves along internal and external edges of the major road (i.e. (152.40 + 132.26)/2 = 142.33 m). To determine the available sight distance, the calculations are shown for two cases: vehicle approach from the left side and vehicle approach from the right side.

For a vehicle approaching from the left, using Equations (4.9), (4.11), (4.13), and (4.15), R_n, M_1, M_2, and Y are calculated as:

$$R_n = 142.33 - 0.5 \times 14.40 + 3.6 - 0.61 - 2.1 = 136.02 \text{ m}$$

$$m_1 = (20.14 - 14.4)/2 = 2.87 \text{ m}$$

$$M_1 = 2.87 + 3.6 - 0.61 - 2.1 = 3.76 \text{ m}$$

$$m_2 = (20.10 - 7.2)/2 = 6.45 \text{ m}$$

$$M_2 = 6.45 + 0.5 \times 7.2 + 0.61 + .533 = 11.193 \text{ m}$$

Table 5.1 Input data for Application

Variables	Values
R	142.33 m
V_{maj}	40 km/h
L_{wmaj}	3.6 m
L_{wmin}	3.6 m
W_{maj}	14.4 m
W_{min}	7.2 m
n	2
u	0
D	3. 0 m
Y_p	2.4 m
Y_L	0.61 m
Y_i	0.533 m

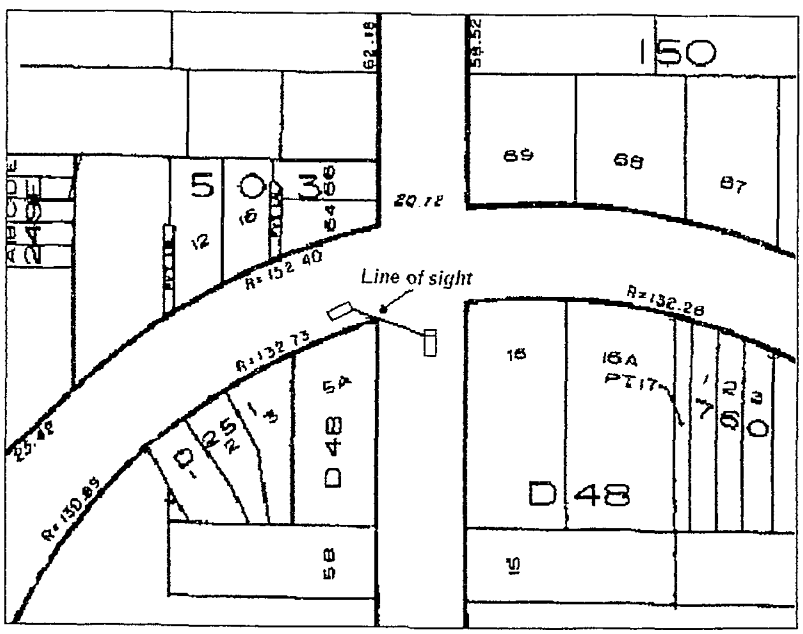


Figure 5.1 A sketch of site selected for application of the model

$Y = 2.4 + 3 + 3.6 -.61 - 2.1 = 6.29$ m

$T_g = 7.5$ sec (passenger car)

Using Equation (4.20), the required sight distance is

$S_r = 0.278 \times 40 \times 7.5 = 83.4$ m

Using Equation (4.7), the available sight distance is

$S_a = 23.42$ m

For a vehicle approaching from the right, using Equations (4.8), (4.10), (4.12), and (4.14), R_n, M_1, M_2, and Y are calculated as:

$R_n = 142.33 + 0 + 0.61 = 142.94$ m

$m_1 = (20.14 - 14.4)/2 = 2.87$ m

$M_1 = 2.87 + 2 \times 3.6 + 0 +.61 = 10.68$ m

$m_2 = (20.10 - 7.2)/2 = 6.45$ m

$M_2 = 6.45 + 3.6 - 0.61 - 0.533 = 8.907$ m

$Y = 2.4 + 3 + 3.6 + 0 + 0.61 = 9.61$ m

$T_g = 7.5$ sec (passenger car)

Using Equation (4.20), the required sight distance is

$S_r = 0.278 \times 40 \times 7.5 = 83.4$ m

Using Equation (4.7), the available sight distance is

$S_a = 78.68$ m

The above calculations show that the available sight distance is less than the required sight distance in both cases, i.e. the sightline to the object is obstructed. The results also confirmed our expectation that the critical case occurs when the major-road vehicle is approaching from the left. The big difference between the S_a from the left and the S_a from the right, might be due to skew on the left of the intersection. It should be noted that the above calculations are based on the extreme values. It might be possible that the above design has a reliability value that is acceptable to the designer.

The following section describes the procedure for quantifying the reliability level of the existing design of the intersection. The reliability model is applied only to the critical case, the approaching vehicle from the left.

5.1.2.2 Reliability Model

Assuming 10% coefficient of variation for all random variables, the mean and standard deviation values in Table 4.5 were used at this particular intersection. The mean and standard deviation of major road speed at the intersection are 30.77 km/h and 3.077 respectively. Now,

Using Equation (4.21),

$E[S_a] = 26.33$ m

Using Equation (4.25),

$E[S_r] = 58.25$ m

the expected safety margin can be found by calculating the difference of the expected available sight distance and expected required sight distance. It is given as:

$E[S_a] - E[S_r] = -31.92$

Note that the above value is negative indicating a very low value for the reliability index or, in other words, a high probability of failure. Using Equation (4.22), the variance of the available sight distance is,

$Var[S_a] = 0.32$

Similarly, using Equation (4.26), the variance of required sight distance is

$Var[S_r] = 67.87$

Using Equation (4.29), the reliability index, $\beta = -3.88$

From the standard normal variate table, the value of $(\phi)\beta$ corresponding to the reliability index (β), is 0.00006. Using Equation (4.30), the probability of failure,

$P_f = 99.99$ %

All data was input into the computer software developed. We found that the results were identical to the above calculation.

5.2 Results and Discussion

The results obtained from the modified deterministic method show that the available sight distance is not adequate at the modelled intersection for the minor-road vehicle on the inside of the curve. As the probability of failure for the vehicle approaching from the left is very large, it would be appropriate to conclude that the intersection does not fulfill minimum sight distance requirements for vehicles approaching from the left or right. The results also indicate that two-way operation of Pembroke Street at Dundas Street is not advisable due to the existing ISD constraint.

To improve the sight distance at the intersection, using the modified deterministic model, for the existing value of m_1 = 2.87 m, m_2 = 62.62 m is required but the existing m_2 is only 6.45 m. As the intersection is located in a developed area and it is not possible to increase $m_{2,}$. Another way to improve the sight distance is to keep the m_2 fixed and change m_1. For the existing m_2 = 6.45 m, the minimum required m_1 is 7.55 m, an increase of 4.68 m. Therefore changing the m_1 might be possible at this location.

Using the reliability model Figure D4.29, for a radius of 142.33 m, the values of m_1 are in the range of 5.84 - 6.74 m and in the range of 0 - 20 m for $m_{2,}$. Assuming that m_2 will be fixed at 6.45 m, the required value of m_1 will be approximately to 6.40 m for P_f = 10%. For a higher probability of failure, for example P_f = 20%, m_1 are in the range of 5.65 - 6.18 m, for m_2 in the range of 0 - 20 m and, for the fixed value of m_2 = 6.45 m, the required m_1 is approximately to 6.11 m. A design with a higher probability of failure would require less value of m_1.

5.3 Hypothetical Example

A hypothetical example was used to illustrate the application of the model using the design aids. A hypothetical intersection is located in a rural setting. The major and minor roads are two-lane roadways with lane widths of 3.6 m each. There is a horizontal curve of radius 400 m on the major road at the intersection. The major-road design speed is 60 km/h. An obstruction is located on the inside of the horizontal curve to the left of the minor road. The far corner of the obstruction is at a distance of m_2 = 8.1 m from the edge of the minor road and at a distance of m_1 = 6.05 m from the inner edge of the major road. The location of the obstruction is being questioned by the authorities due to the increasing number of collisions at the intersection. The designer argues that he used graphical techniques to establish sightlines and insists that the sightlines are unobstructed, but he does not have a quantifiable measure to substantiate his argument. A senior designer suggests that a reliability analysis should be carried out at the location to estimate the reliability level of the intersection sight distance at the intersection.

Using the modified deterministic model, from Figure C4.2, for m_2 = 8.1 m, required m_1 is 6.57 m. The required m_1 is less than the current m_1 at the location, but the difference is small (0.52 m). Using the reliability model and assuming CV = 10%, from Figure D4.25 the probability of failure of this intersection is estimated to be approximately 5% or less. As a probability of failure of 5% is deemed acceptable in geometric design, it might not be appropriate to suggest that the designer placed the obstruction incorrectly.

Chapter 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This thesis has presented a new reliability model for the analysis of sight distance at stop-control intersections with horizontal curves on major roads. It was assumed that the minor road has no skew and zero grades. Graphical design aids, which are easy to use, were developed to determine the obstruction location for different probabilities of failure, design speeds, and curve radii. Based on the research, the following comments are offered:

1. The reliability model developed in this research is simple and straightforward. It requires only the means and standard deviations of the random variables. The reliability level is defined in terms of a reliability index and the probability of failure is related to the reliability level. A small probability of failure reflects a high reliability level, and vice versa.
2. The reliability model presented in this thesis can be used to determine the reliability level (probability of failure) of sight distance for stop-control intersections. The results show that the deterministic model generally produces higher probabilities of failure when the obstruction is closer to the minor road.
3. The reliability model may also be useful in defining different levels of safety for intersections with respect to the reliability level. For example, high-speed intersections may be designed with higher reliability levels, and vice versa for low-speed intersections.
4. Using the design aids established in this thesis, the probability of failure of any stop-control intersection (existing or proposed) can be easily determined using the major-road design speed, horizontal curve radius, and obstruction location. In addition, for a desired probability of failure, the obstruction location can be determined. If an existing intersection does not satisfy sight distance requirements, the proposed reliability model can be

used to estimate the existing reliability level and to recommend necessary improvements.

5. The design aids in this research were established for the case of a left-turn of a minor-road stopped vehicle. For the case of a right-turn and for crossing manoeuvres, the model presented will provide conservative obstruction clearance values with less probabilities of failure.
6. The sensitivity analysis of the reliability model shows that the most sensitive random variables are major-road speed, vehicle characteristics, and time gap of the minor-road vehicle. The sensitivity to the obstruction clearances decreases with the increase in radius of curvature. The sensitivity analysis for mean values of the random variables also shows that mean of vehicle width is less sensitive to the obstruction clearance values, so the model is also applicable to the ISD of truck traffic.

6.2 Recommendations

Areas proposed for future research include the following:

1. The reliability model presented in this research covers only one situation (stop-control intersection. where the obstruction is inside the horizontal curve).The model should be extended to include situations where the obstruction is outside the horizontal alignment.
2. The model considers only horizontal alignments on the major road. It was also assumed that the minor road has a zero grade and no skew. This model should be extended to include three-dimensional intersections that have both horizontal and vertical alignments on the major road and to include cases where the minor road has a skew and a grade.
3. This research should be extended to include other cases of intersection sight distance presented in AASHTO. A comprehensive research is also required to establish a database that can provide information about the statistical nature and distributions of the various variables used in

geometric design, such as operating speed, time gap, operational characteristics, and vehicle characteristics.

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APPENDIX A

Notation

APPENDIX A: Notation

$Cov(V_w, Y_L)$ = Covariance between the random variables V_w and Y_L

$Cov(V_w, Y_i)$ = Covariance between the random variables V_w and Y_i

CV = Variation Co-efficient

D = Distance from the front of minor-road vehicle to the edge of major-road pavement

L_{wmaj} = Major-road lane width

L_{wmin} = Minor-road lane width

M_1 = Distance between the obstruction and curved path of the approaching vehicle from right/left side

m_1 = Distance between the obstruction corner and edge of the major road

M_2 = Distance between the obstruction and the minor-road driver's eye

m_2 = Distance between the obstruction corner and edge of the minor road

n = Number of lanes of the major road

R = Horizontal curve radius

R_n = Radius of the horizontal curved-path of the approaching vehicle

ϕ = Central angle for the arc with length S_a

ϕ_1 = Central angle between the observer and the obstruction

ϕ_2 = Central angle between the obstruction and the object

P_f = Probability of failure

q = Distance between the centre of horizontal curve and the edge of obstruction

S_a = Available sight distance

S_r = Required sight distance

T_g = Time gap required for the minor-road stopped vehicle

U = Median width of the major road

V_w = Vehicle width

V_{maj}	= Major-road design speed
W_{maj}	= Major-road Width
W_{min}	= Minor-road Width
Y	= Distance from the minor-road driver's eye to the approaching vehicle side/top
Y_L	= Lateral distance between the left side of vehicle and the right side of the lane line
Y_p	= Distance from the minor-road driver's eye to the front of vehicle
Y_i	= Lateral distance between the left side of vehicle and the driver's eye
Z	= Number of standard deviations of the normal distribution corresponding to a certain percentile value
β	= Reliability index
∂	= First derivative of a function
μ	= Mean of a random variable
ρ	= Co-efficient of correlation between two random variables
σ	= Standard deviation of a random variable

APPENDIX B

First Derivative of The Random Variables

APPENDIX B: First Derivative of The Random Variables

The first derivatives of $E[S_a]$, Equation (4.21) with respect to the random variables , V_w, Y_L, Y_i, Y_P and D, were computed using Mathematica, a powerful mathematical software package. The results involved in very long equations and all could not be included in this thesis. Only one is being placed here as a sample. Software can not accept all types of alpha and numeric values due to some limitations. Original variables were replaced by some single word, and are shown in Table below:

Original Random Variable	Replaced by	Original Variable	Replaced by
Vw	k	L_{wmaj}	s
Y_L	w	W_{maj}	a
Y_i	g	W_{min}	t
Y_p	p	m_1	x
D	y	m_2	j

The input Equation to the software for derivative of E[Sa] with respect to Vw is:

```
D[(R - 0.5*a+ s - w - k)*ArcCos[((R - 0.5*a - p - y)^2 + (R - 0.5*a+ s - w - k)^2 -
    (Sqrt[(R - 0.5*a- p - y)^2 + (R - 0.5*a - x)^2 - 2*(R - 0.5*a - p - y)*Sqrt[(R - 0.5*a -
x)^2 - (j + 0.5*t + w + g)^2]] -
     (Sqrt[(R - 0.5*a - p - y)^2 + (R - 0.5*a - x)^2 - 2*(R - 0.5*a - p - y)*Sqrt[(R - 0.5*a
- x)^2 - (j + 0.5*t + w + g)^2]]^2 + (R - 0.5*a - x)^2 -
      (R - 0.5*a- p - y)^2)/(2*Sqrt[(R - 0.5*a - p - y)^2 + (R - 0.5*a - x)^2 - 2*(R -
0.5*a - p - y)*Sqrt[(R - 0.5*a - x)^2 - (j + 0.5*t + w+ g)^2]]) +
     Sqrt[(R - 0.5*a + s - w - k)^2 - (R - 0.5*a- x)^2*(1- ((Sqrt[(R - 0.5*a - p - y)^2 +
(R - 0.5*a - x)^2 - 2*(R - 0.5*a - p - y)*
           Sqrt[(R - 0.5*a - x)^2 - (j + 0.5*t + w + g)^2]]^2 + (R - 0.5*a - x)^2 - (R -
0.5*a - p - y)^2)/(2*Sqrt[(R - 0.5*a - p - y)^2 + (R - 0.5*a - x)^2 -
          2*(R - 0.5*a- p - y)*Sqrt[(R - 0.5*a- x)^2 - (j + 0.5*t + w + g)^2]]*(R - 0.5*a
- x)))^2)])^2)/(2*(R - 0.5*a - p - y)*(R - 0.5*a + s - w - k))], k]
```

The output is as under:

$$
\begin{aligned}
&\Big((-0.5`\ a-k+R+s-w)\Big(-\Big(-2\,(-0.5`\ a-k+R+s-w)+\\
&\quad\Big(2\,(-0.5`\ a-k+R+s-w)\Big(\surd\Big((-0.5`\ a-k+R+s-w)^2-(-0.5`\ a+R-x)^2\\
&\qquad\Big(1-\Big(2\,(-0.5`\ a+R-x)^2-2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\\
&\qquad\quad(-0.5`\ a-p+R-y)\Big)^2\Big/\Big(4\,(-0.5`\ a+R-x)^2\Big((-0.5`\ a+R-x)^2-\\
&\qquad\quad 2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\,(-0.5`\ a-p+R-y)+\\
&\qquad\quad(-0.5`\ a-p+R-y)^2\Big)\Big)\Big)\Big)-\Big(2\,(-0.5`\ a+R-x)^2-2\\
&\qquad\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\,(-0.5`\ a-p+R-y)\Big)\Big/\\
&\qquad\Big(2\surd\Big((-0.5`\ a+R-x)^2-2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\\
&\qquad\quad(-0.5`\ a-p+R-y)+(-0.5`\ a-p+R-y)^2\Big)\Big)+\\
&\qquad\surd\Big((-0.5`\ a+R-x)^2-2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\\
&\qquad\quad(-0.5`\ a-p+R-y)+(-0.5`\ a-p+R-y)^2\Big)\Big)\Big)\Big/\\
&\quad\Big(\surd\Big((-0.5`\ a-k+R+s-w)^2-(-0.5`\ a+R-x)^2\Big(1-\Big(2\,(-0.5`\ a+R-x)^2-\\
&\qquad 2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\\
&\qquad(-0.5`\ a-p+R-y)\Big)^2\Big/\Big(4\,(-0.5`\ a+R-x)^2\\
&\qquad\Big((-0.5`\ a+R-x)^2-2\sqrt{-(g+j+0.5`\ t+w)^2+(-0.5`\ a+R-x)^2}\\
&\qquad\quad(-0.5`\ a-p+R-y)+(-0.5`\ a-p+R-y)^2\Big)\Big)\Big)\Big)\Big)\Big)\Big/
\end{aligned}
$$

(Continued on next page)

$$
\begin{aligned}
&(2\,(-0.5`\,a-k+R+s-w)\,(-0.5`\,a-p+R-y))-\\
&\quad\Big((-0.5`\,a-k+R+s-w)^2-\Big(\surd\Big((-0.5`\,a-k+R+s-w)^2-(-0.5`\,a+R-x)^2\\
&\qquad\Big(1-\Big(2\,(-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)\Big)^2\Big/\Big(4\,(-0.5`\,a+R-x)^2\Big((-0.5`\,a+R-x)^2-\\
&\qquad\quad 2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\;(-0.5`\,a-p+R-y)+\\
&\qquad\quad(-0.5`\,a-p+R-y)^2\Big)\Big)\Big)\Big)-\Big(2\,(-0.5`\,a+R-x)^2-2\\
&\qquad\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\;(-0.5`\,a-p+R-y)\Big)\Big/\\
&\qquad\Big(2\surd\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)\Big)+\\
&\qquad\surd\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)\Big)^2\\
&\quad+(-0.5`\,a-p+R-y)^2\Big)\Big/(2\,(-0.5`\,a-k+R+s-w)^2\,(-0.5`\,a-p+R-y))\Big)\Big)\Big/\\
&\Big(\surd\Big(1-\Big((-0.5`\,a-k+R+s-w)^2-\Big(\surd\Big((-0.5`\,a-k+R+s-w)^2-(-0.5`\,a+R-x)^2\\
&\qquad\Big(1-\Big(2\,(-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)\Big)^2\Big/\Big(4\,(-0.5`\,a+R-x)^2\Big((-0.5`\,a+R-x)^2-\\
&\qquad\quad 2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\;(-0.5`\,a-p+R-y)+\\
&\qquad\quad(-0.5`\,a-p+R-y)^2\Big)\Big)\Big)\Big)-\Big(2\,(-0.5`\,a+R-x)^2-\\
&\qquad 2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\;(-0.5`\,a-p+R-y)\Big)\Big/\\
&\qquad\Big(2\surd\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)\Big)+\\
&\qquad\surd\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\\
&\qquad\quad(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)\Big)^2+\\
&\quad(-0.5`\,a-p+R-y)^2\Big)^2\Big/(4\,(-0.5`\,a-k+R+s-w)^2\,(-0.5`\,a-p+R-y)^2)\Big)\Big)-
\end{aligned}
$$

(Continued on next page)

$$
\begin{aligned}
&\mathrm{ArcCos}\Big[\Big((-0.5`\,a-k+R+s-w)^2-\Big(\sqrt{\Big((-0.5`\,a-k+R+s-w)^2-(-0.5`\,a+R-x)^2\Big(1-\Big(2\,(-0.5`\,a+R-}\\
&\qquad\quad \overline{x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\,(-0.5`\,a-p+}\\
&\qquad\quad \overline{R-y)\Big)^2\Big/\Big(4\,(-0.5`\,a+R-x)^2\Big((-0.5`\,a+R-x)^2-}\\
&\qquad\quad \overline{2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\,(-0.5`\,a-p+R-y)+}\\
&\qquad\quad \overline{(-0.5`\,a-p+R-y)^2\Big)\Big)\Big)\Big)}-\Big(2\,(-0.5`\,a+R-x)^2-\\
&\qquad 2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}\,(-0.5`\,a-p+R-y)\Big)\Big/\\
&\quad\Big(2\sqrt{\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}}\\
&\qquad \overline{(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)}\Big)+\\
&\quad\sqrt{\Big((-0.5`\,a+R-x)^2-2\sqrt{-(g+j+0.5`\,t+w)^2+(-0.5`\,a+R-x)^2}}\\
&\qquad \overline{(-0.5`\,a-p+R-y)+(-0.5`\,a-p+R-y)^2\Big)}\Big)^2+(-0.5`\,a-p+R-y)^2\Big)\Big/\\
&\quad(2\,(-0.5`\,a-k+R+s-w)\,(-0.5`\,a-p+R-y))\Big]
\end{aligned}
$$

APPENDIX C

Design Graphs of The Modified Deterministic Model

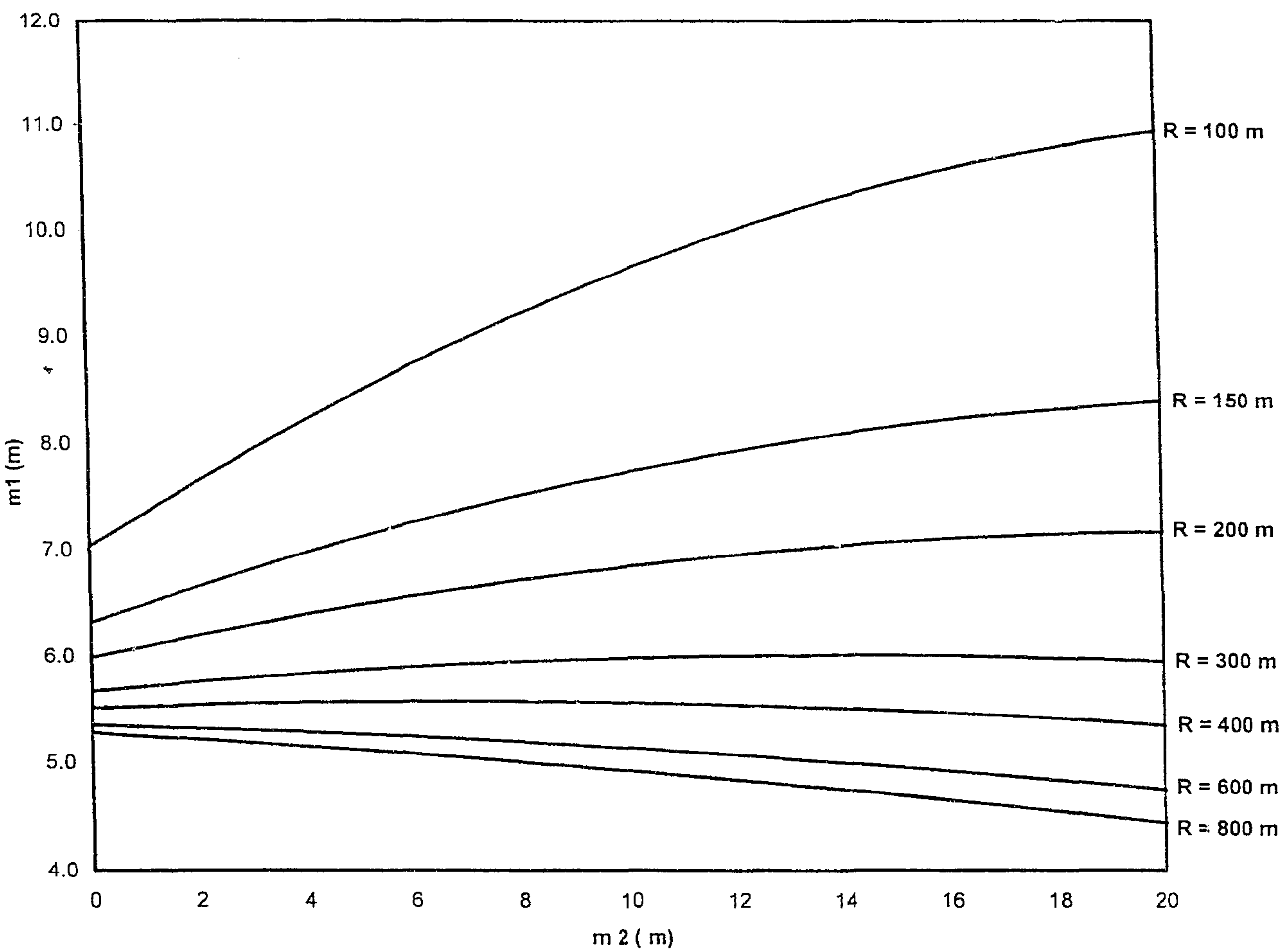

Figure C4.1 Design graph for m_1 and m_2 based a modified deterministic model for design speed of 40 km/h (2-lane minor road intersecting with 2-lane major road and lane width = 3.6 m)

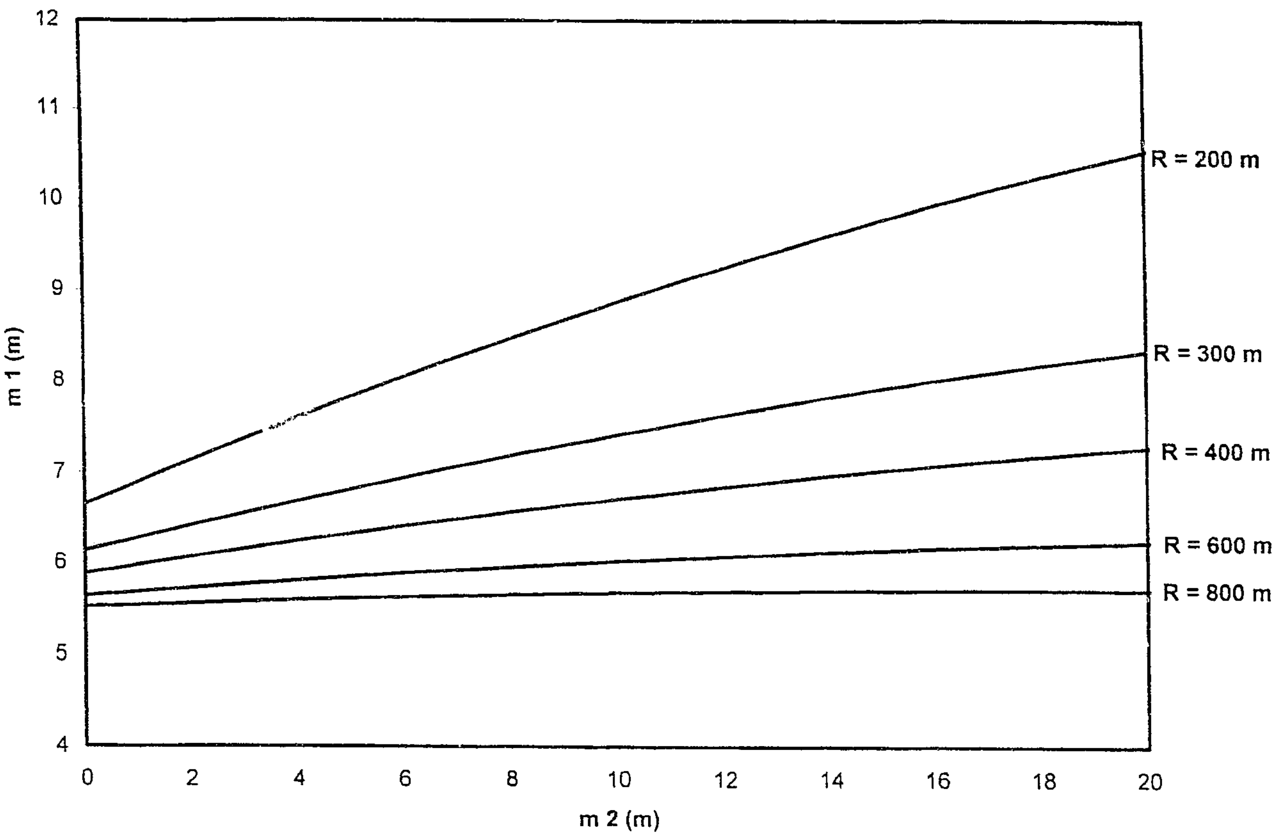

Figure C4.2 Design graph for m_1 and m_2 based a modified deterministic model for design speed of 60 km/h (2-lane minor road intersecting with 2-lane major road and lane width = 3.6 m)

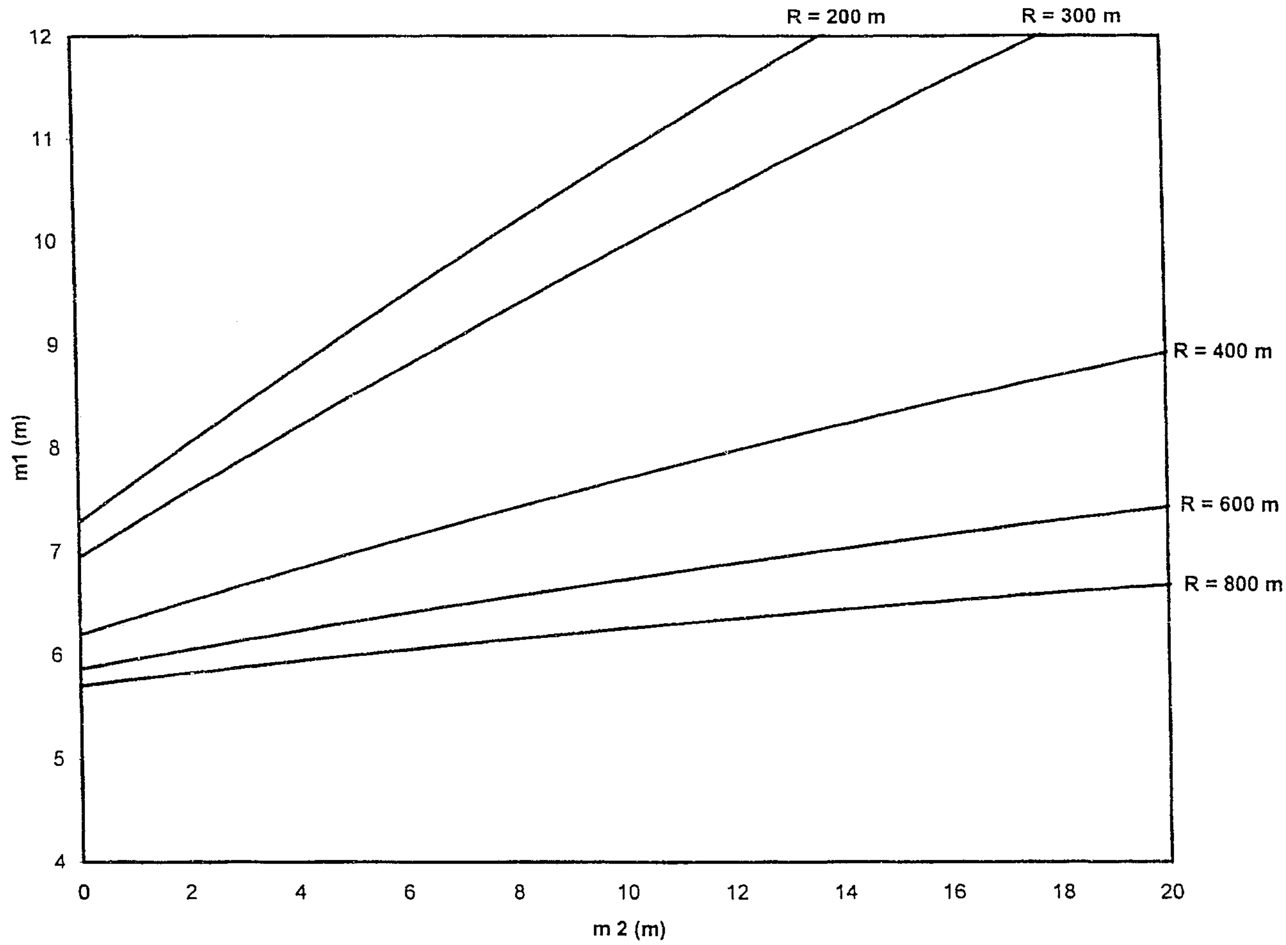

Figure C4.3 Design graph for m_1 and m_2 based a modified deterministic model for design speed of 80 km/h (2-lane minor road intersecting with 2-lane major road and lane width = 3.6 m)

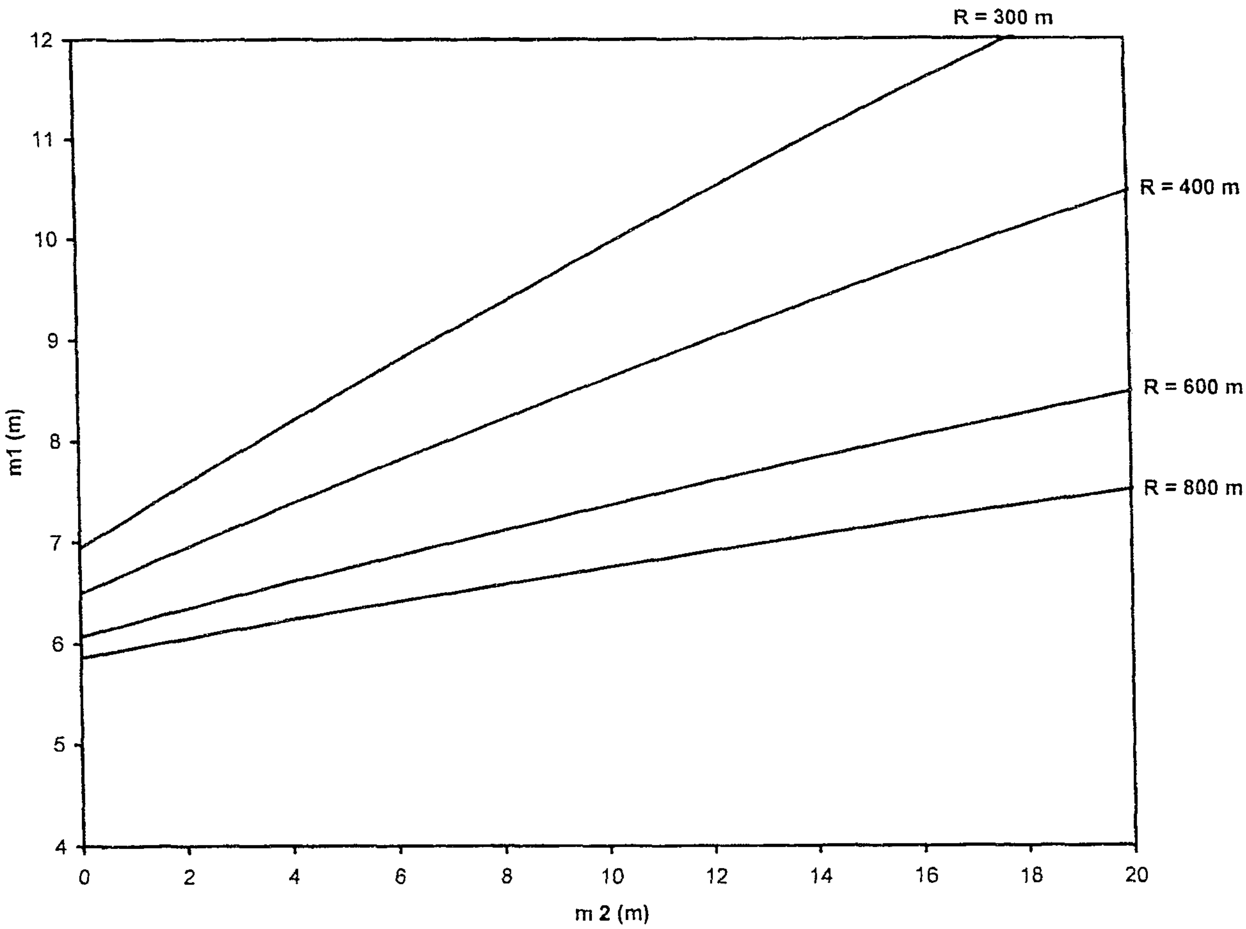

Figure C4.4 Design graph for m_1 and m_2 based a modified deterministic model for design speed of 100 km/h (2-lane minor road intersecting with 2-lane major road and lane width = 3.6 m)

APPENDIX D

Design Graphs of The Reliability Model

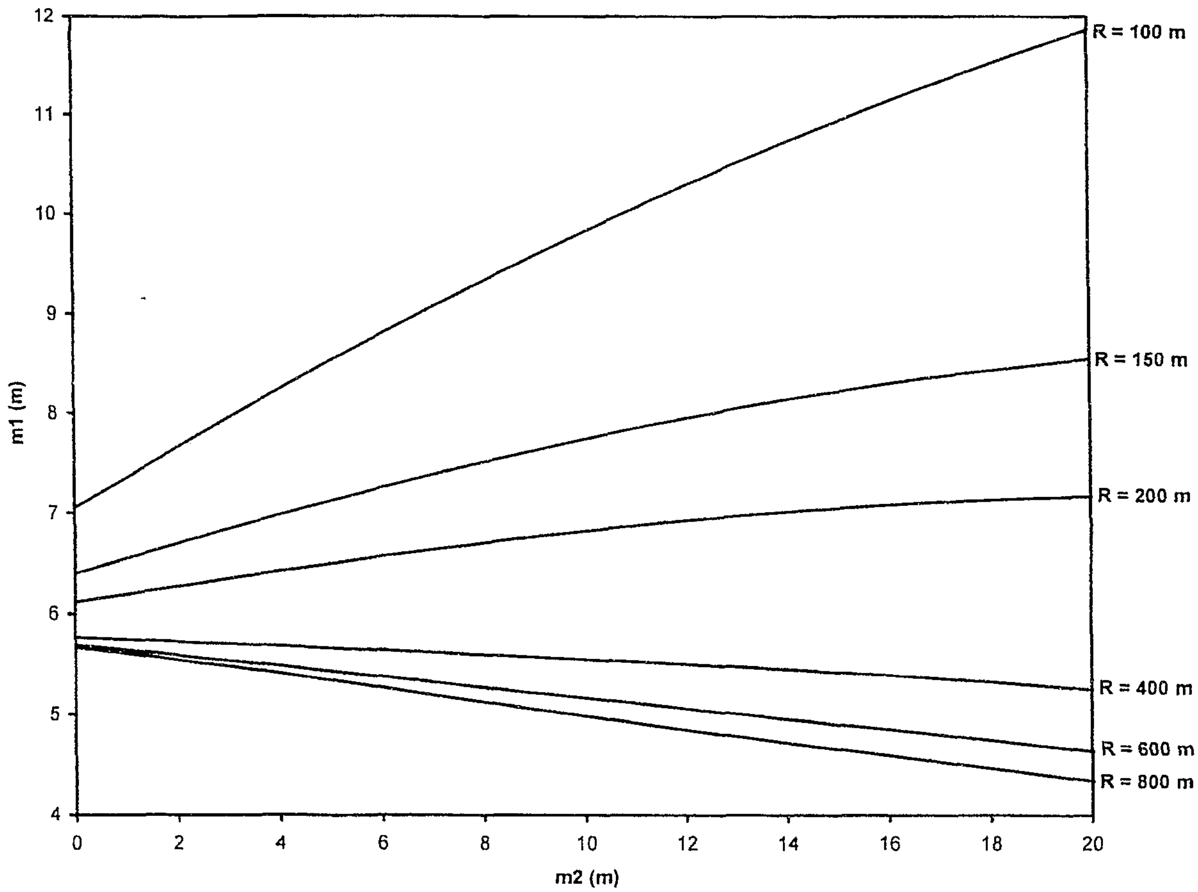

Figure D4.1 Design graph for P_f = 0.1% and CV = 5% for all variables (design speed = 40 km/h)

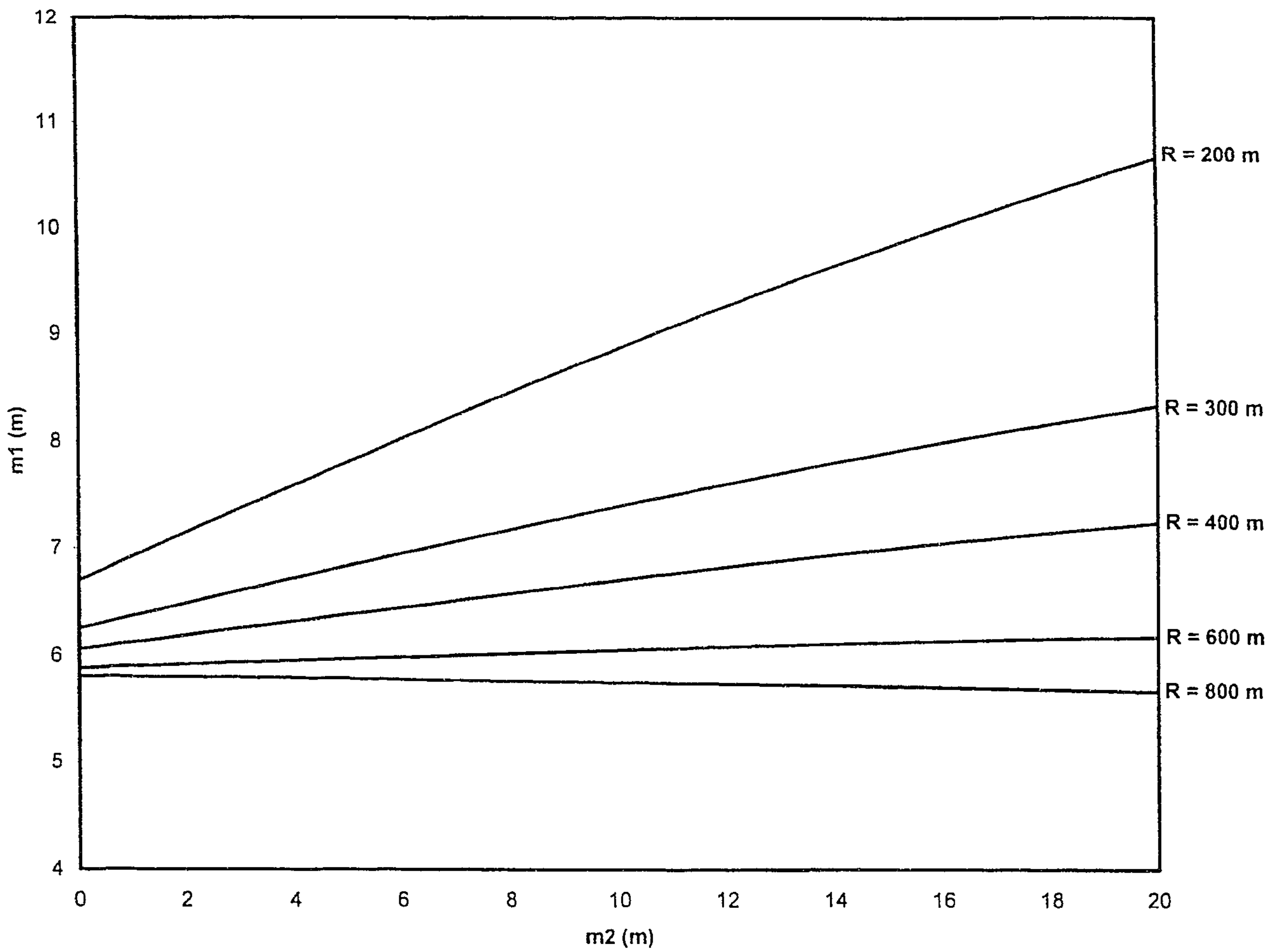

Figure D4.2 Design graph for P_f = 0.1% and CV = 5% for all variables (design speed = 60 km/h)

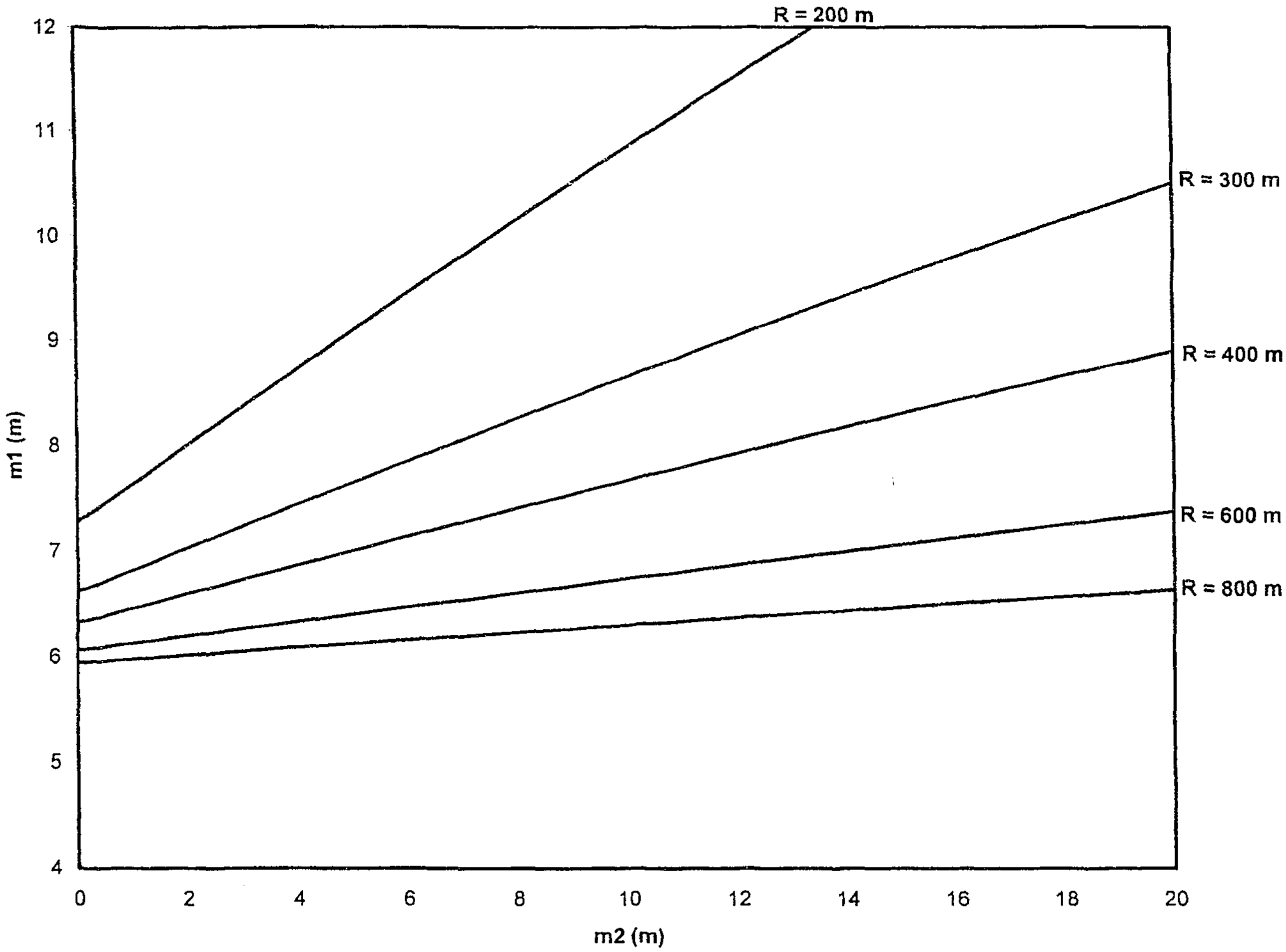

Figure D4.3 Design graph for P_f = 0.1% and CV = 5% for all variables (design speed = 80 km/h)

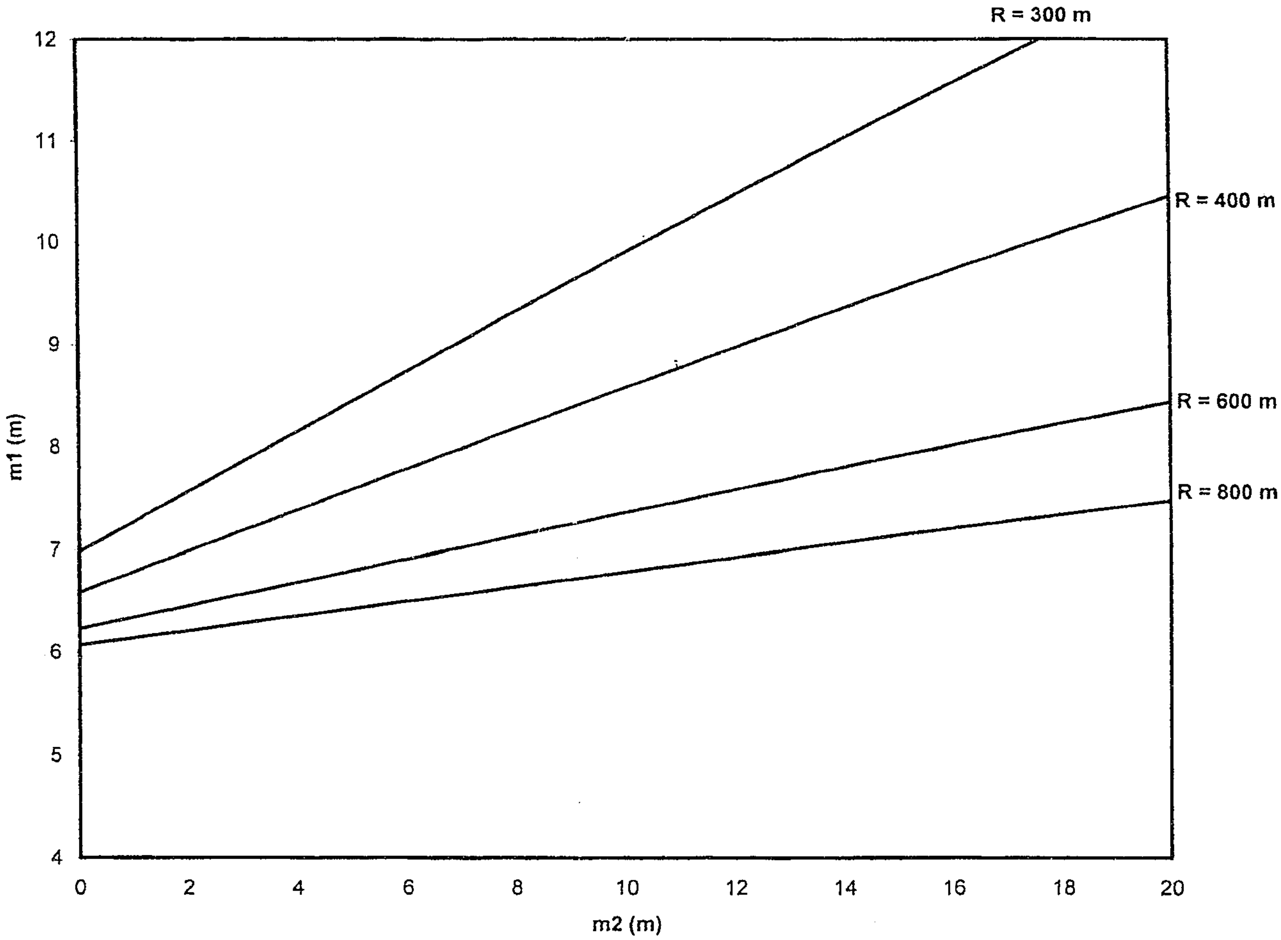

Figure D4.4 Design graph for P_f = 0.1% and CV = 5% for all variables (design speed = 100 km/h)

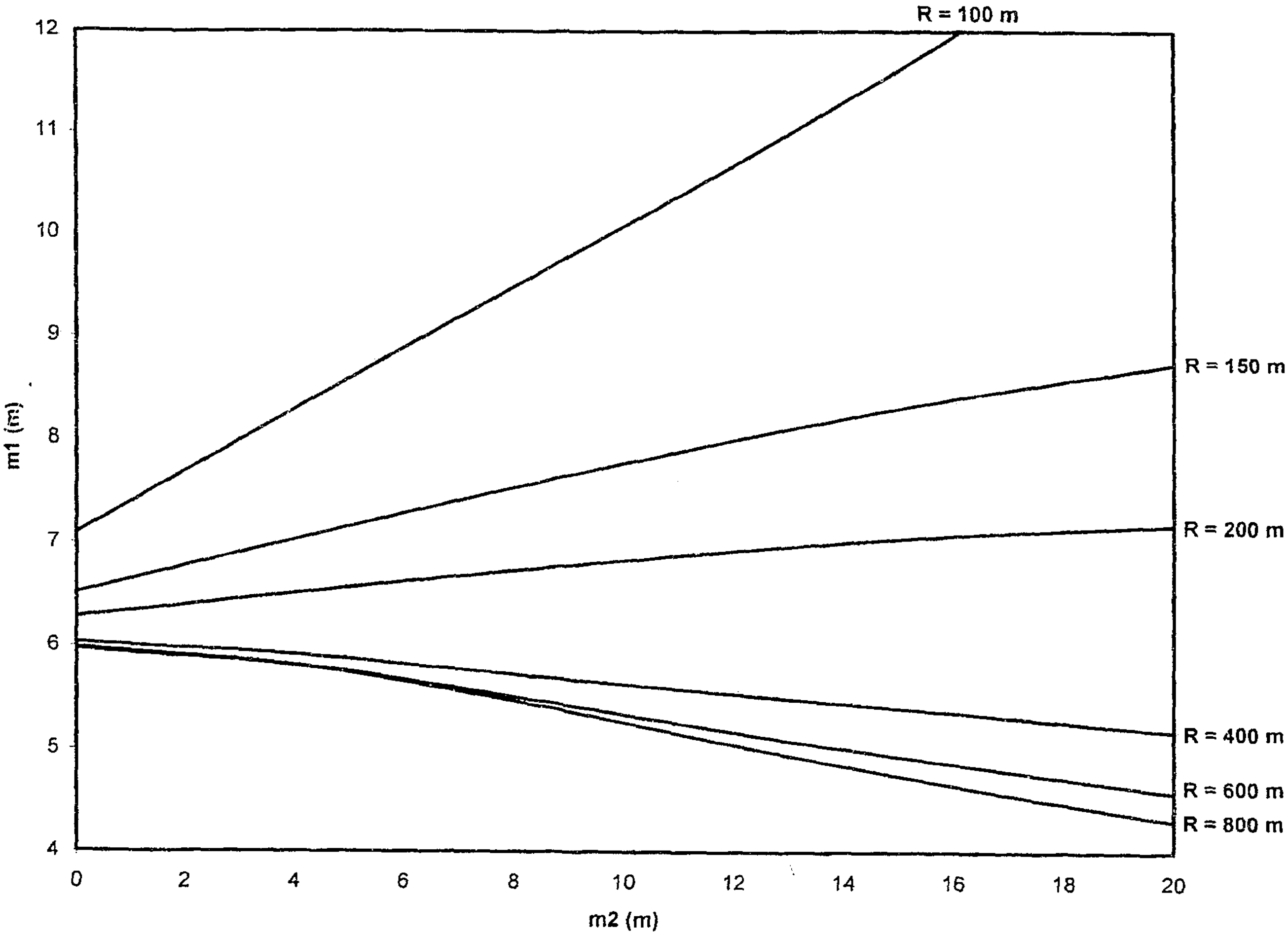

Figure D4.5 Design graph for P_f = 0.1% and CV = 10% for all variables (design speed = 40 km/h)

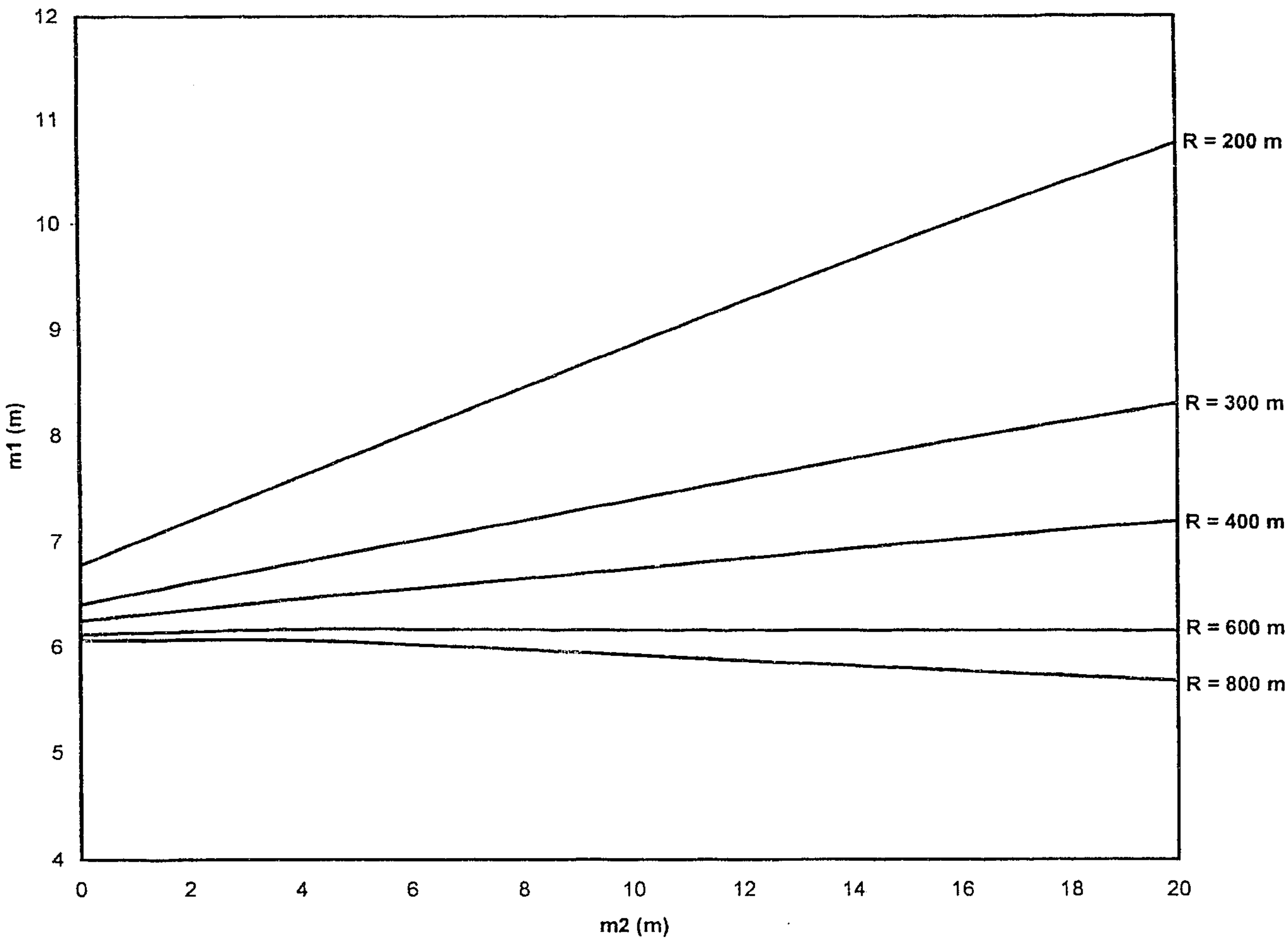

Figure D4.6 Design graph for P_f = 0.1% and CV = 10% for all variables (design speed = 60 km/h

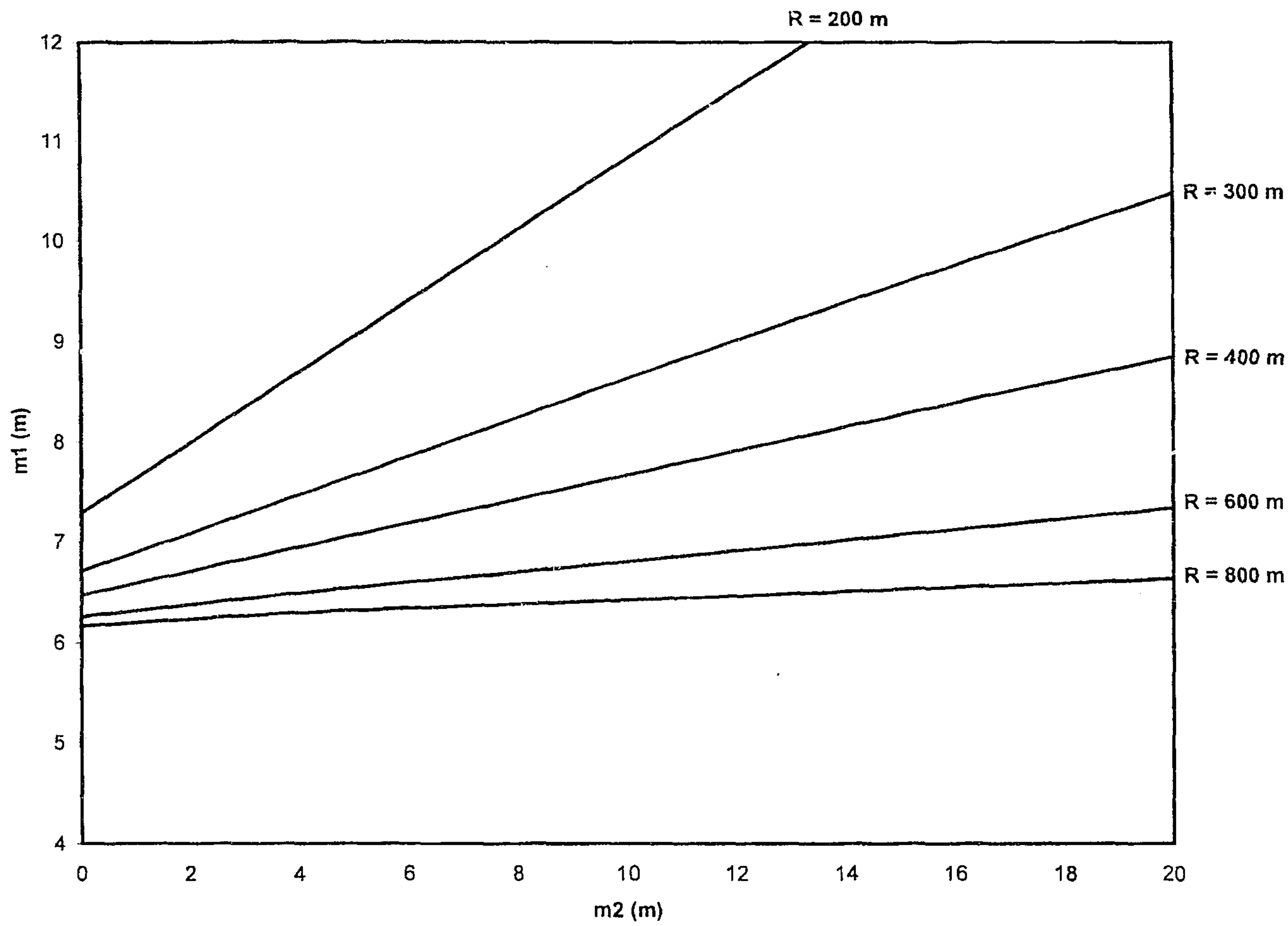

Figure D4.7 Design graph for P_f = 0.1% and CV = 10% for all variables (design speed = 80 km/h

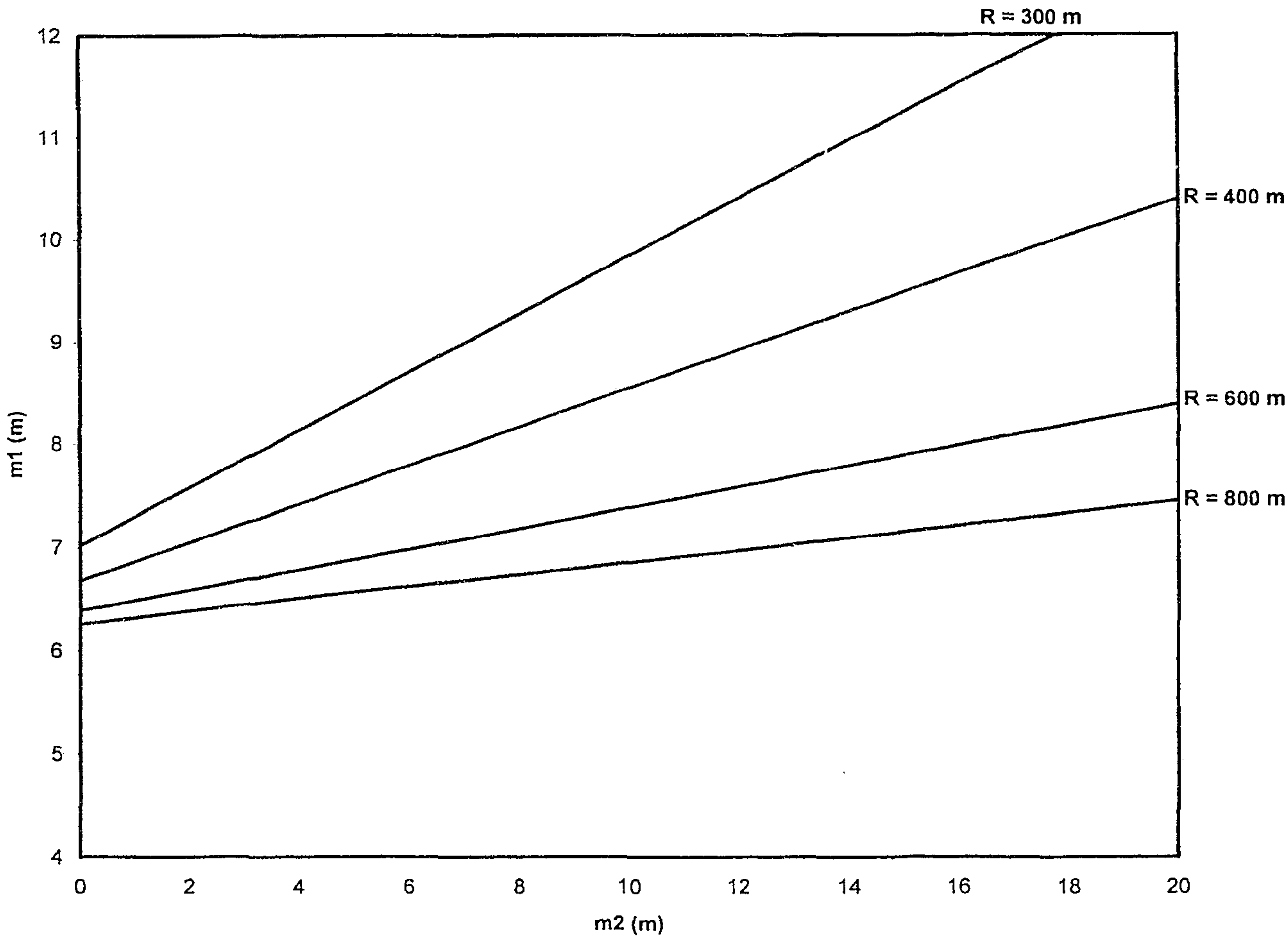

Figure D4.8 Design graph for P_f = 0.1% and CV = 10% for all variables (design speed = 100 km/h

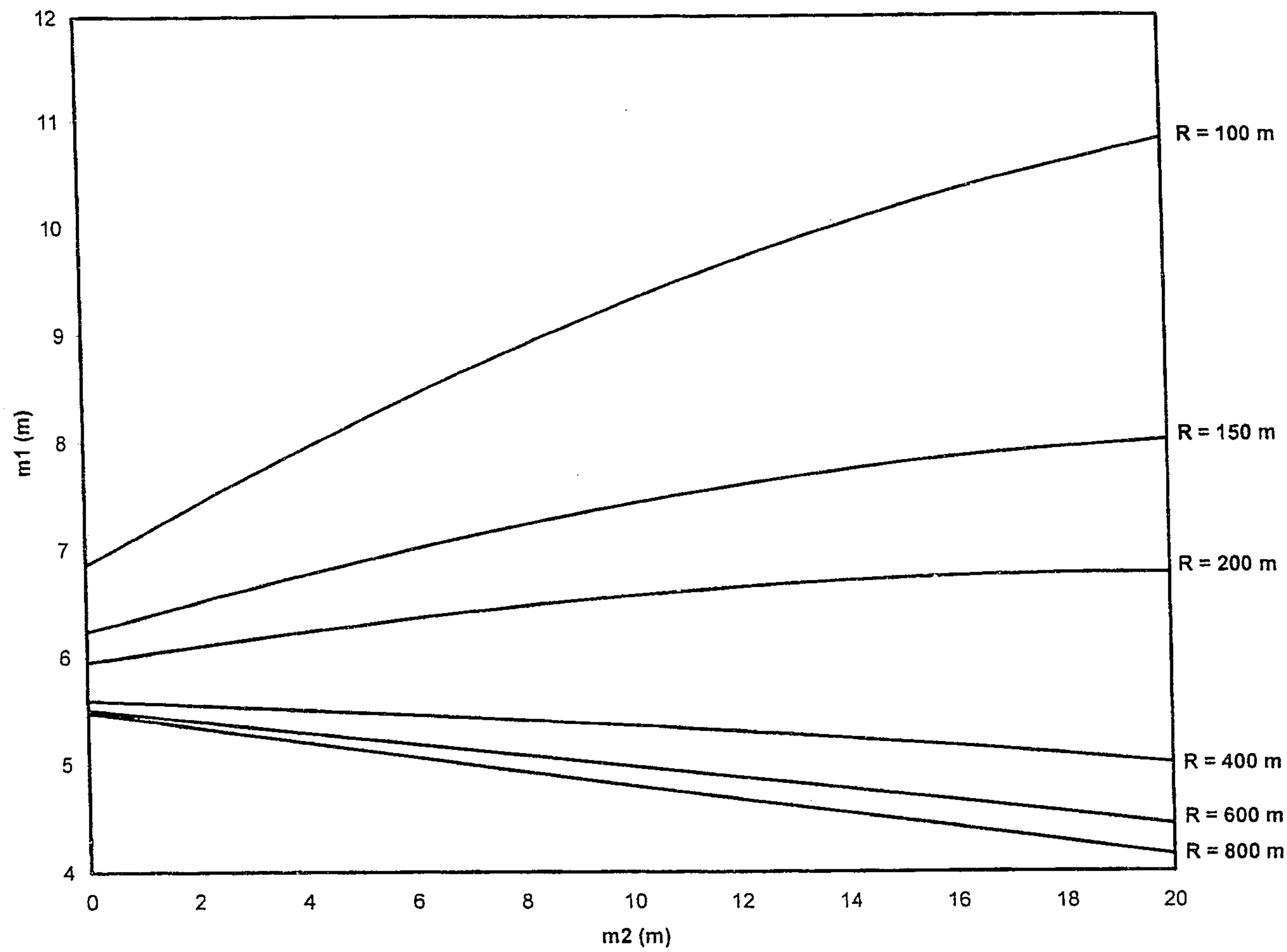

Figure D4.9 Design graph for P_f = 1% and CV = 5% for all variables (design speed = 40 km/h)

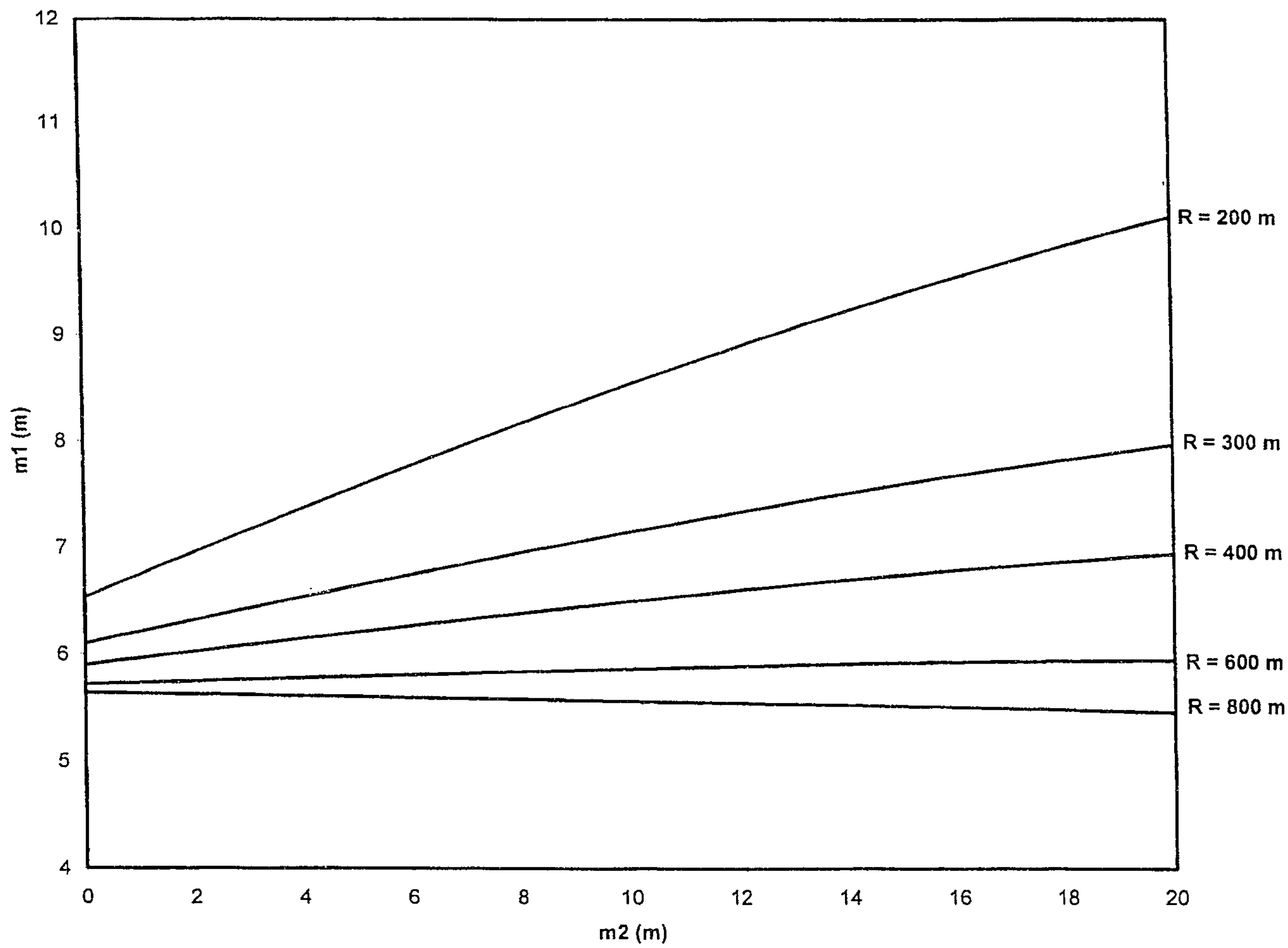

Figure D4.10 Design graph for P_f = 1% and CV = 5% for all variables (design speed = 60 km/h)

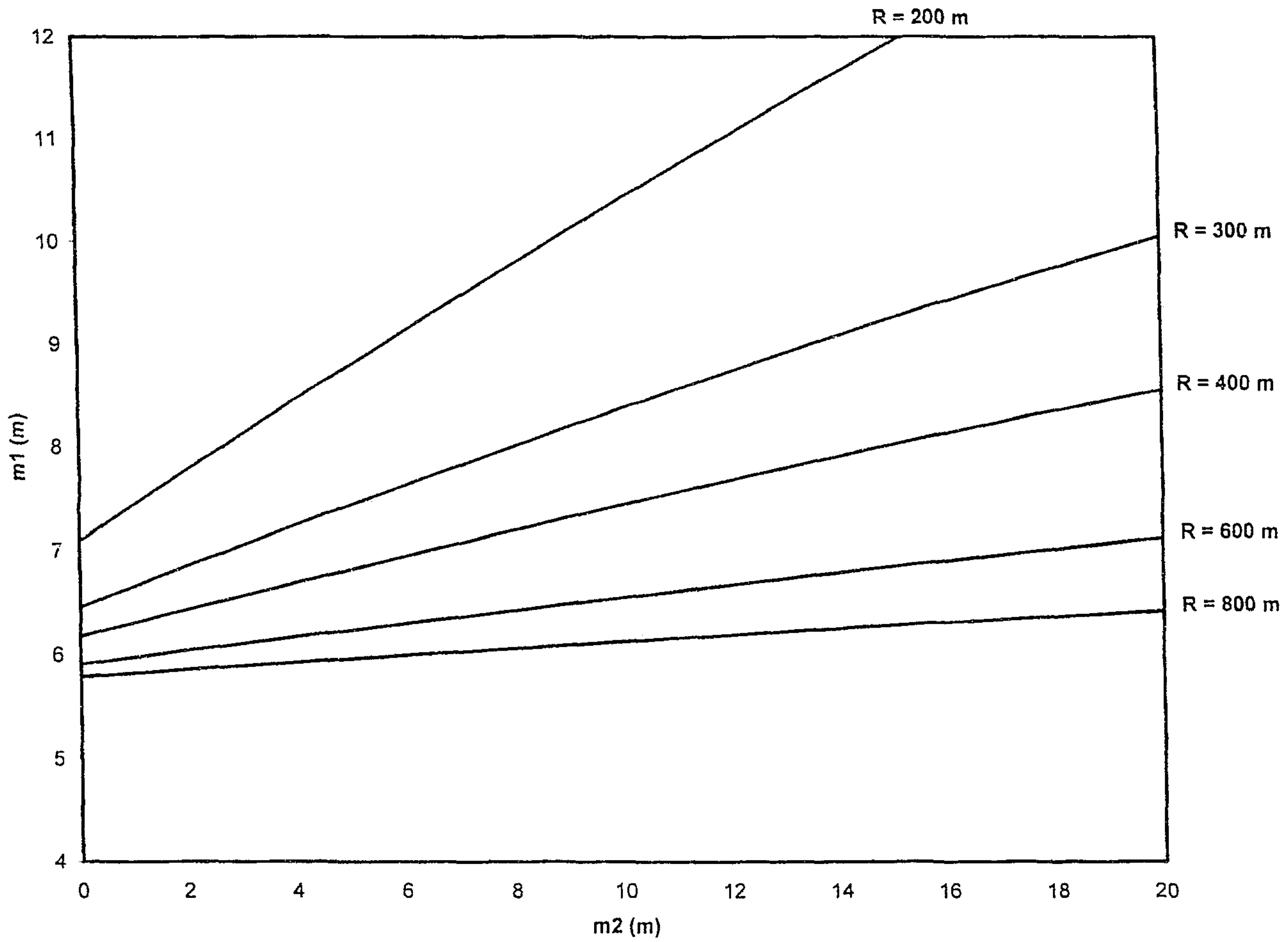

Figure D4.11 Design graph for P_f = 1% and CV = 5% for all variables (design speed = 80 km/h)

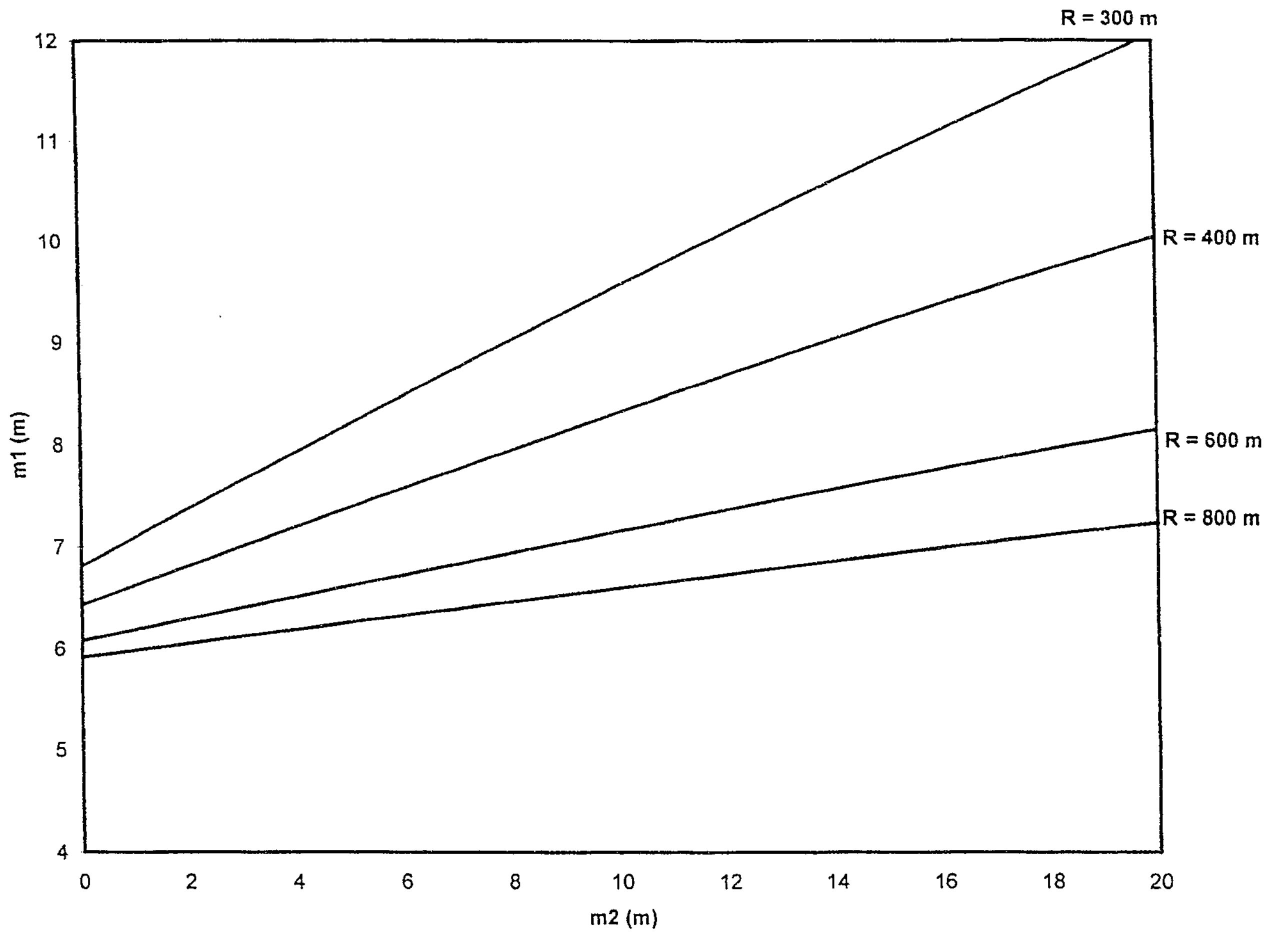

Figure D4.12 Design graph for P_f = 1% and CV = 5% for all variables (design speed = 100 km/h)

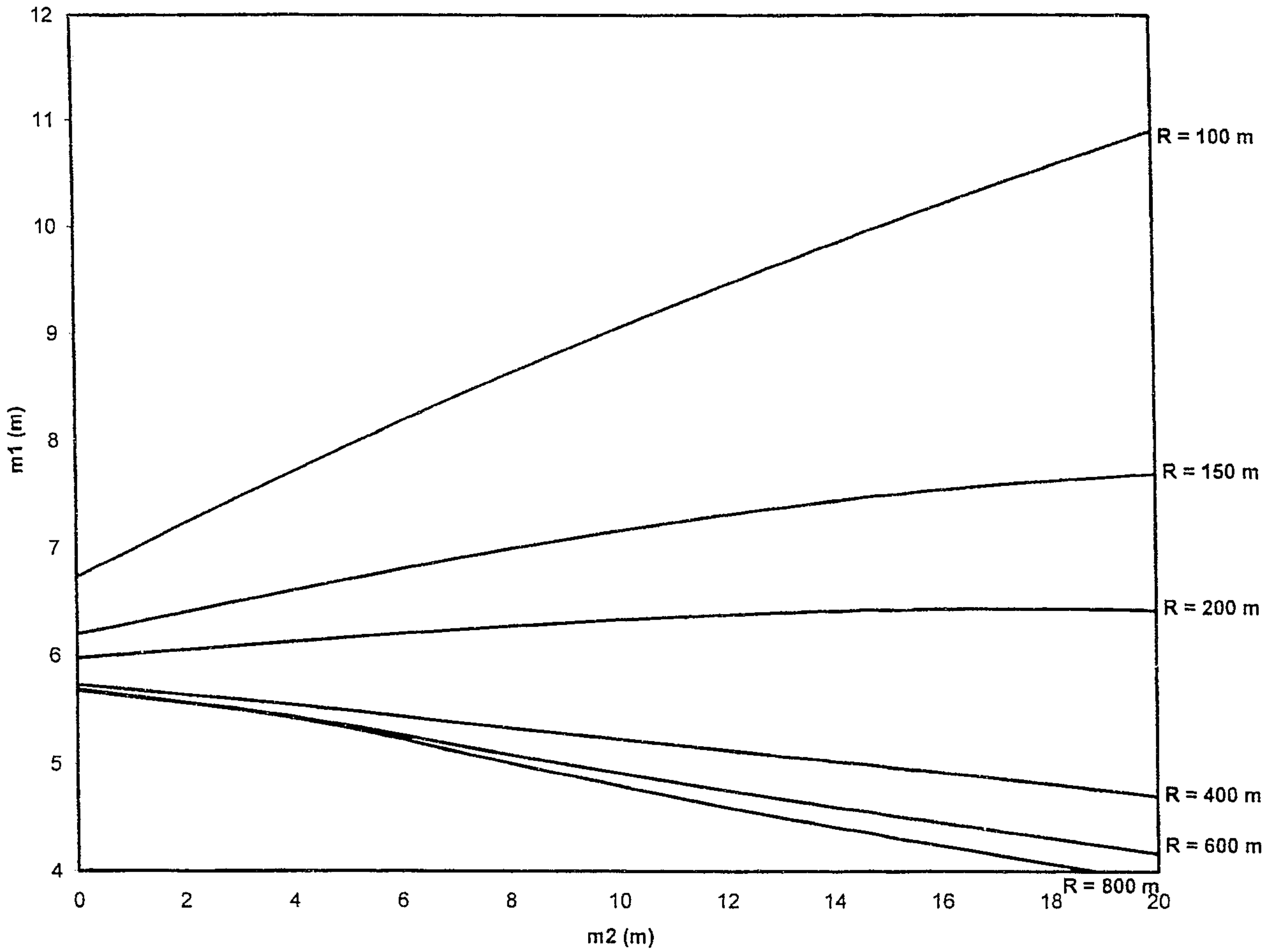

Figure D4.13 Design graph for P_f = 1% and CV = 10% for all variables (design speed = 40 km/h)

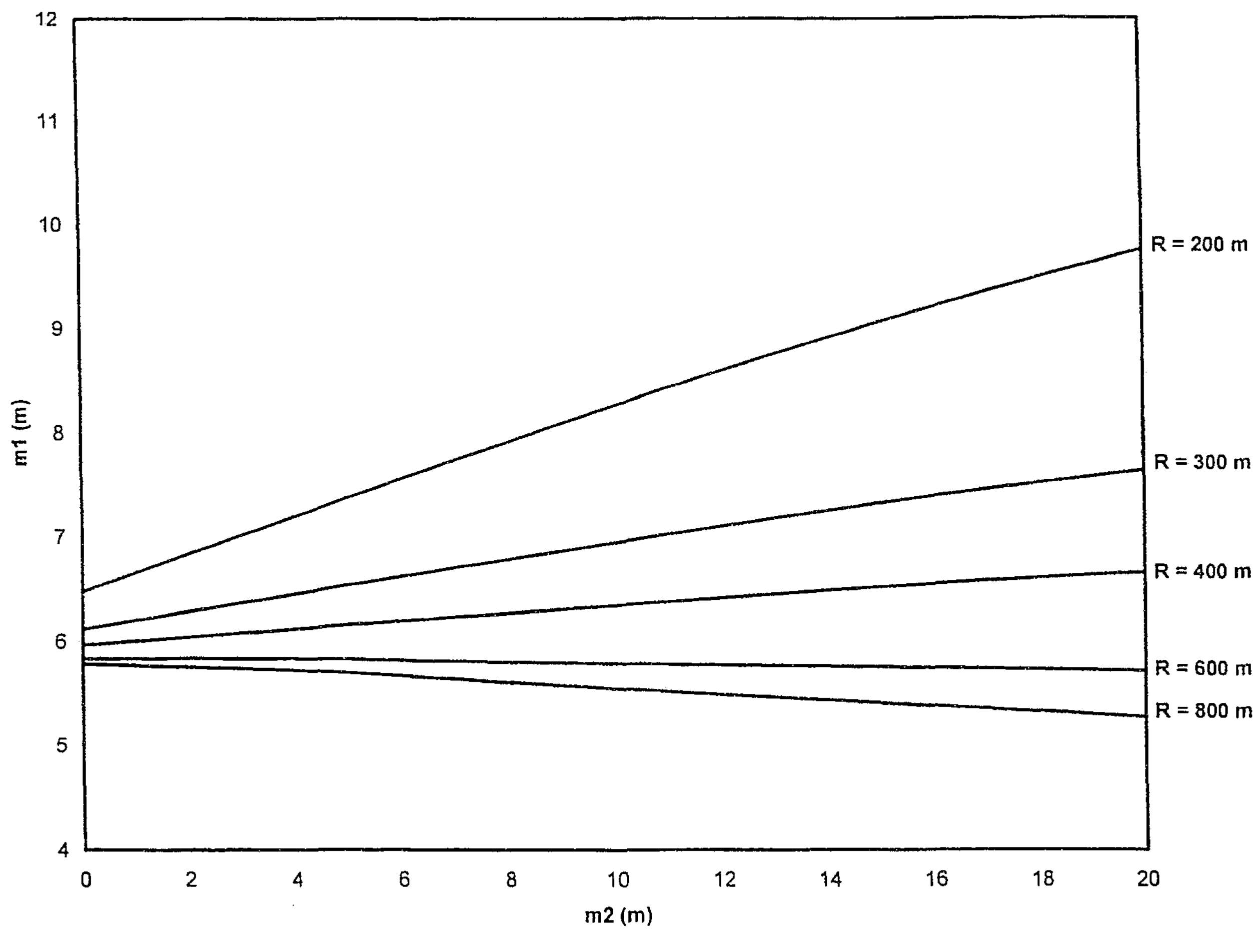

Figure D4.14 Design graph for P_f = 1% and CV = 10% for all variables (design speed = 60 km/h)

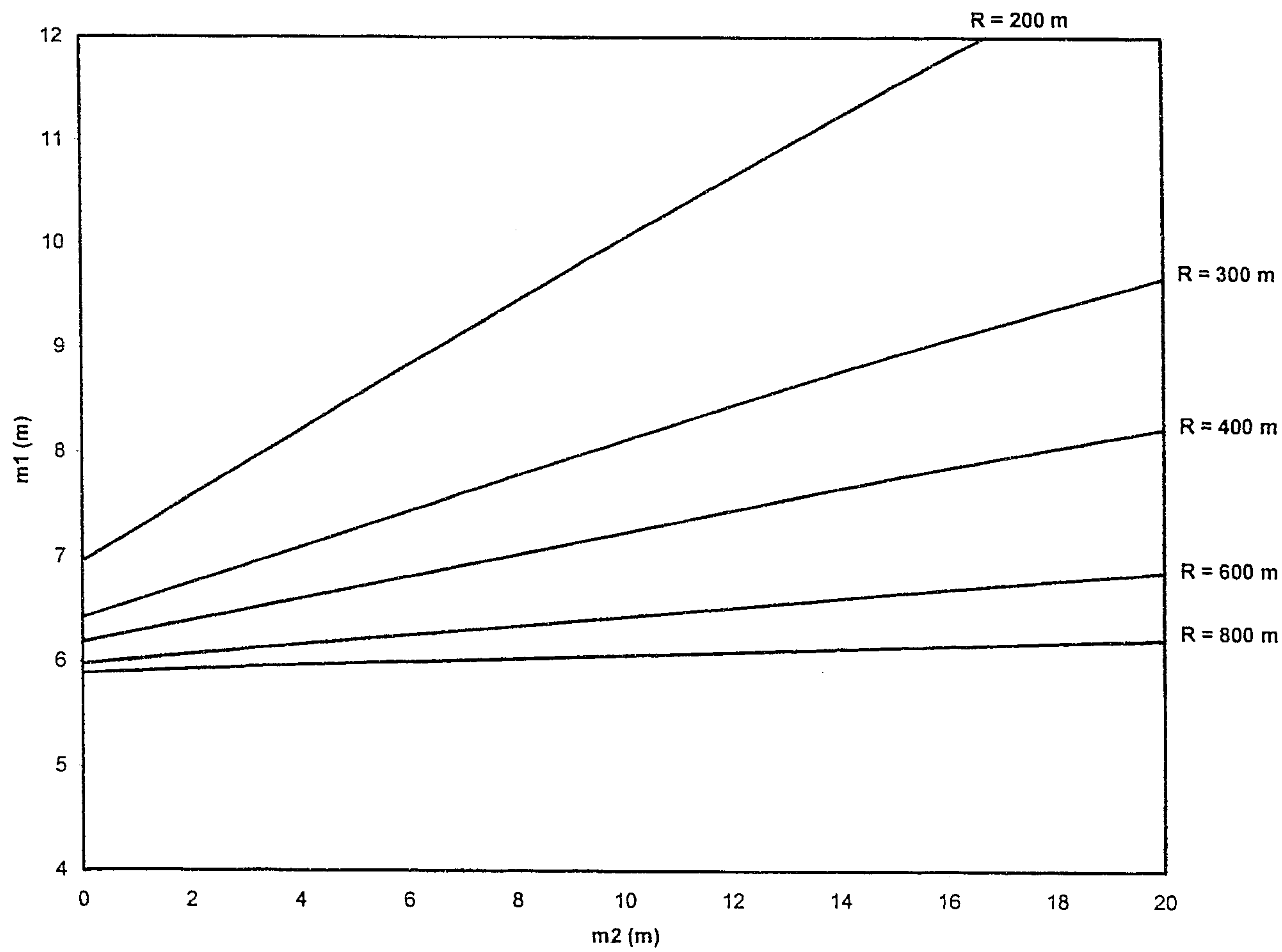

Figure D4.15 Design graph for P_f = 1% and CV = 10% for all variables (design speed = 80 km/h)

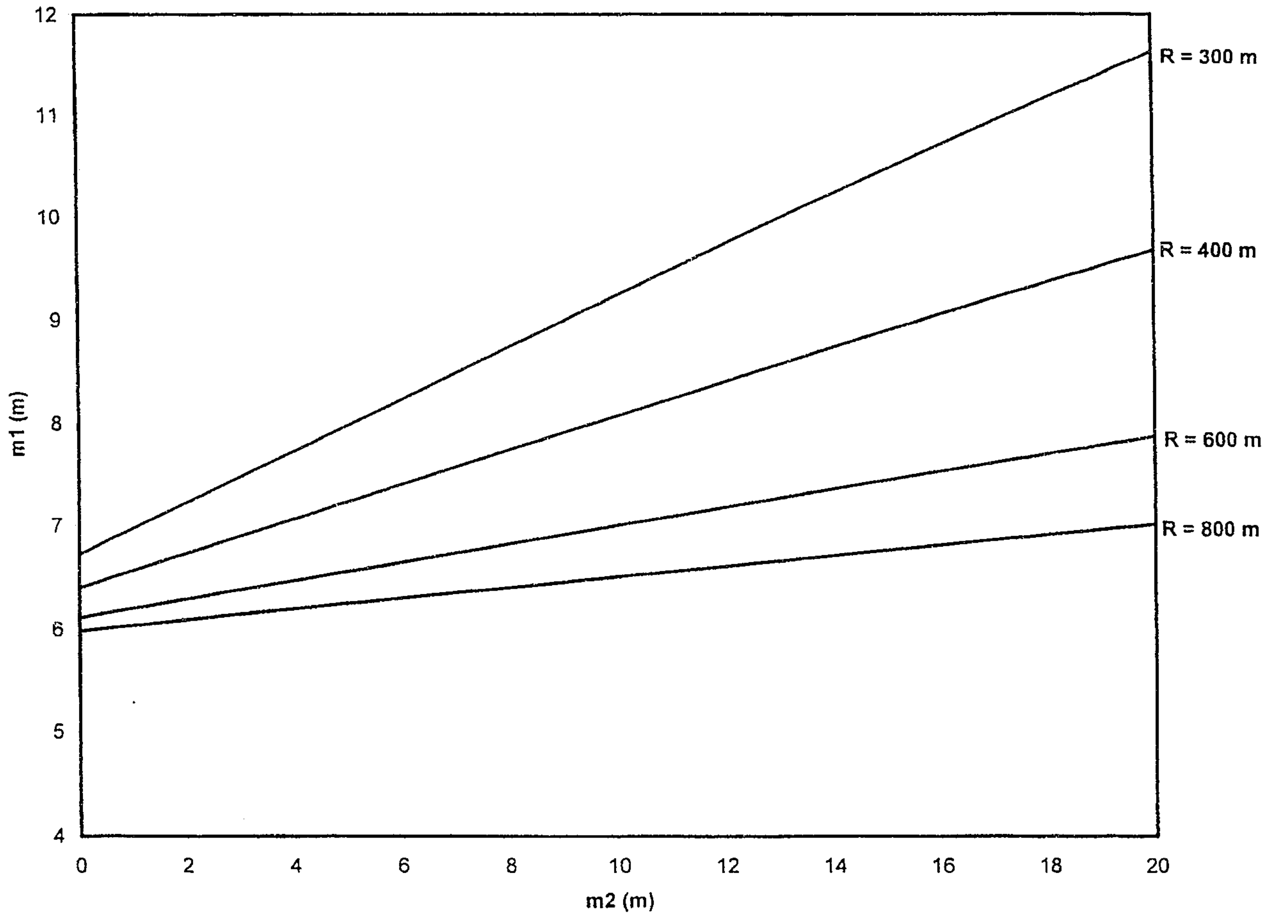

Figure D4.16 Design graph for P_f = 1% and CV = 10% for all variables (design speed = 100 km/h)

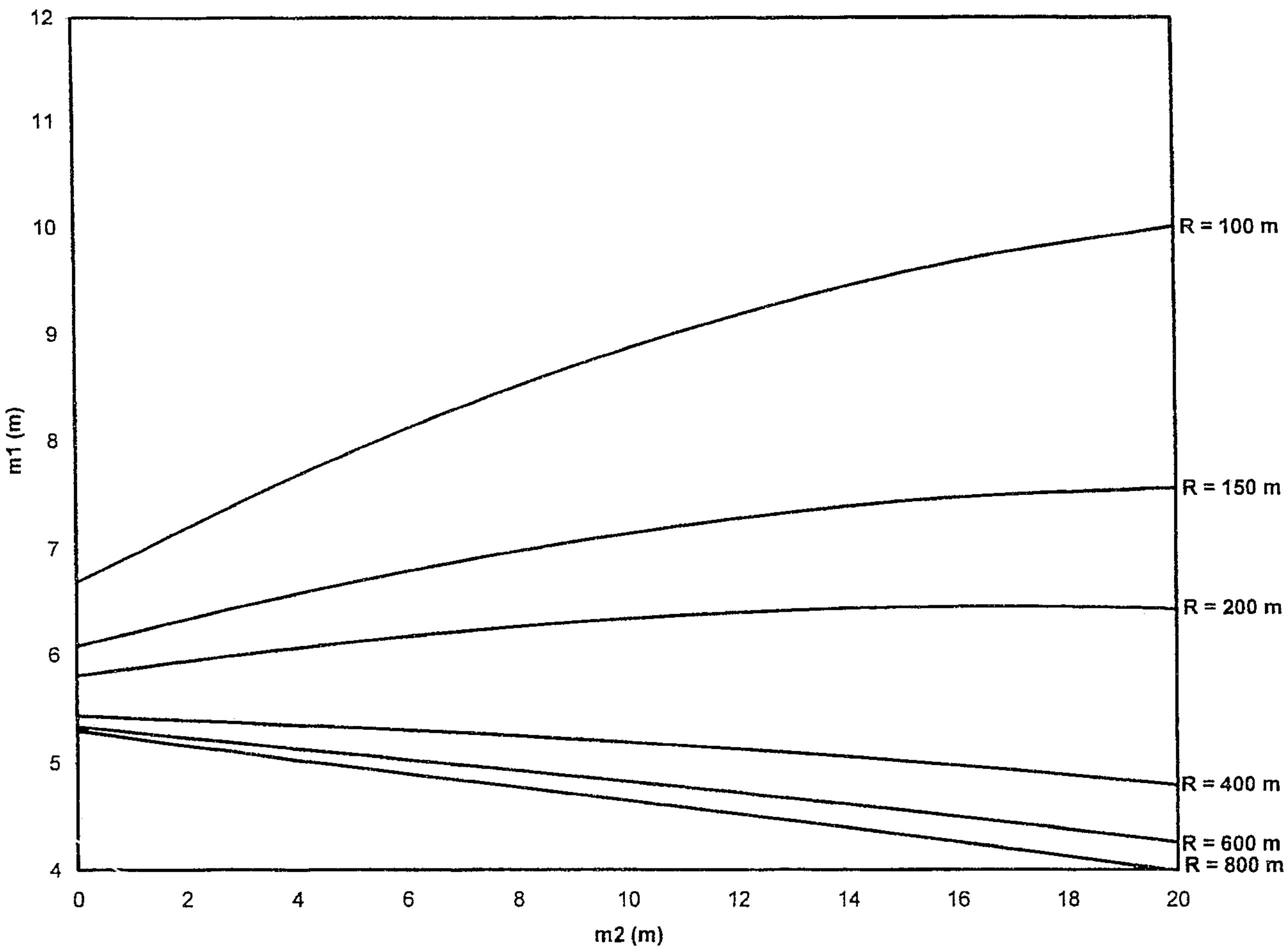

Figure D4.17 Design graph for P_f = 5% and CV = 5% for all variables (design speed = 40 km/h)

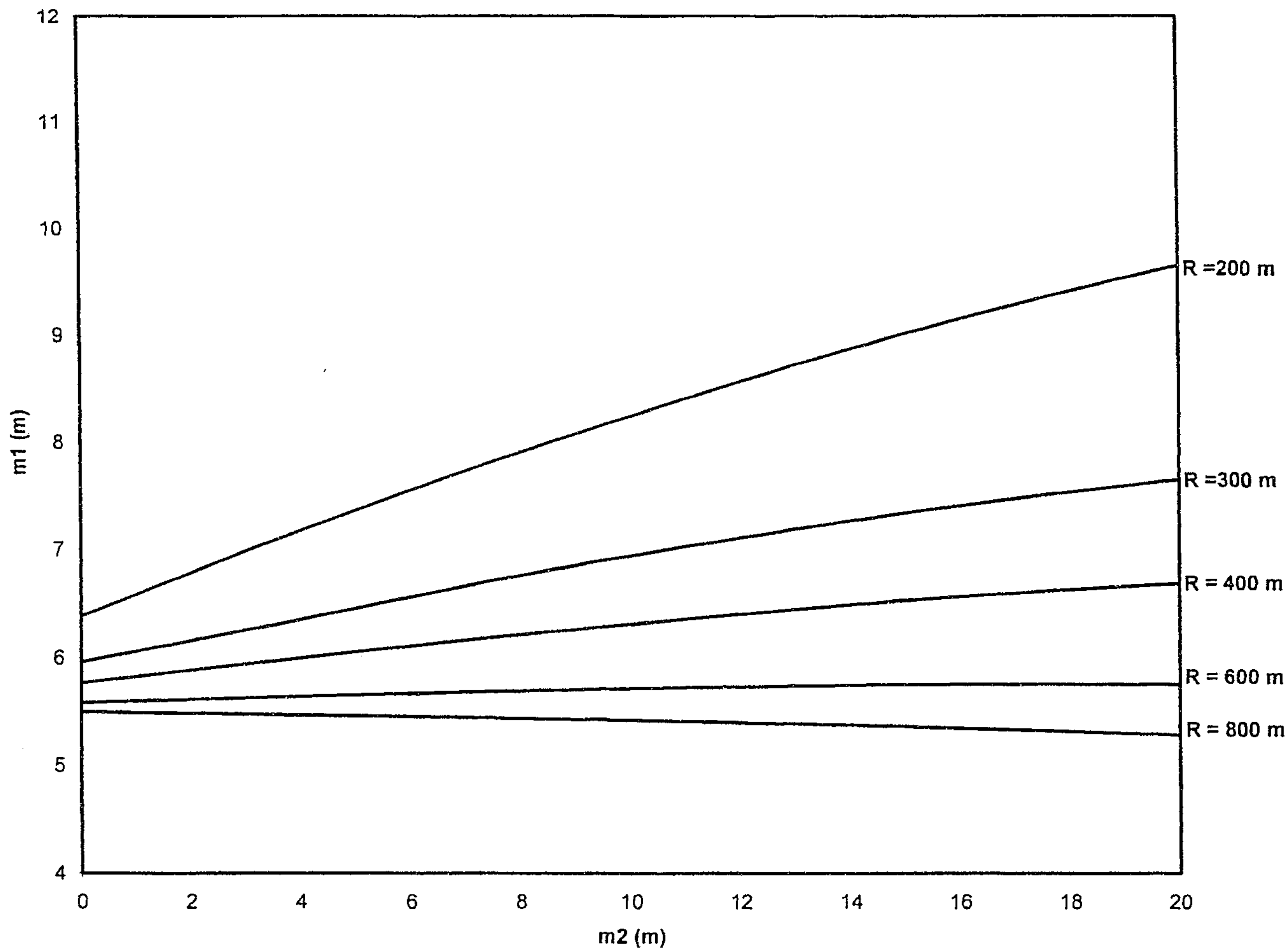

Figure D4.18 Design graph for P_f = 5% and CV = 5% for all variables (design speed = 60 km/h)

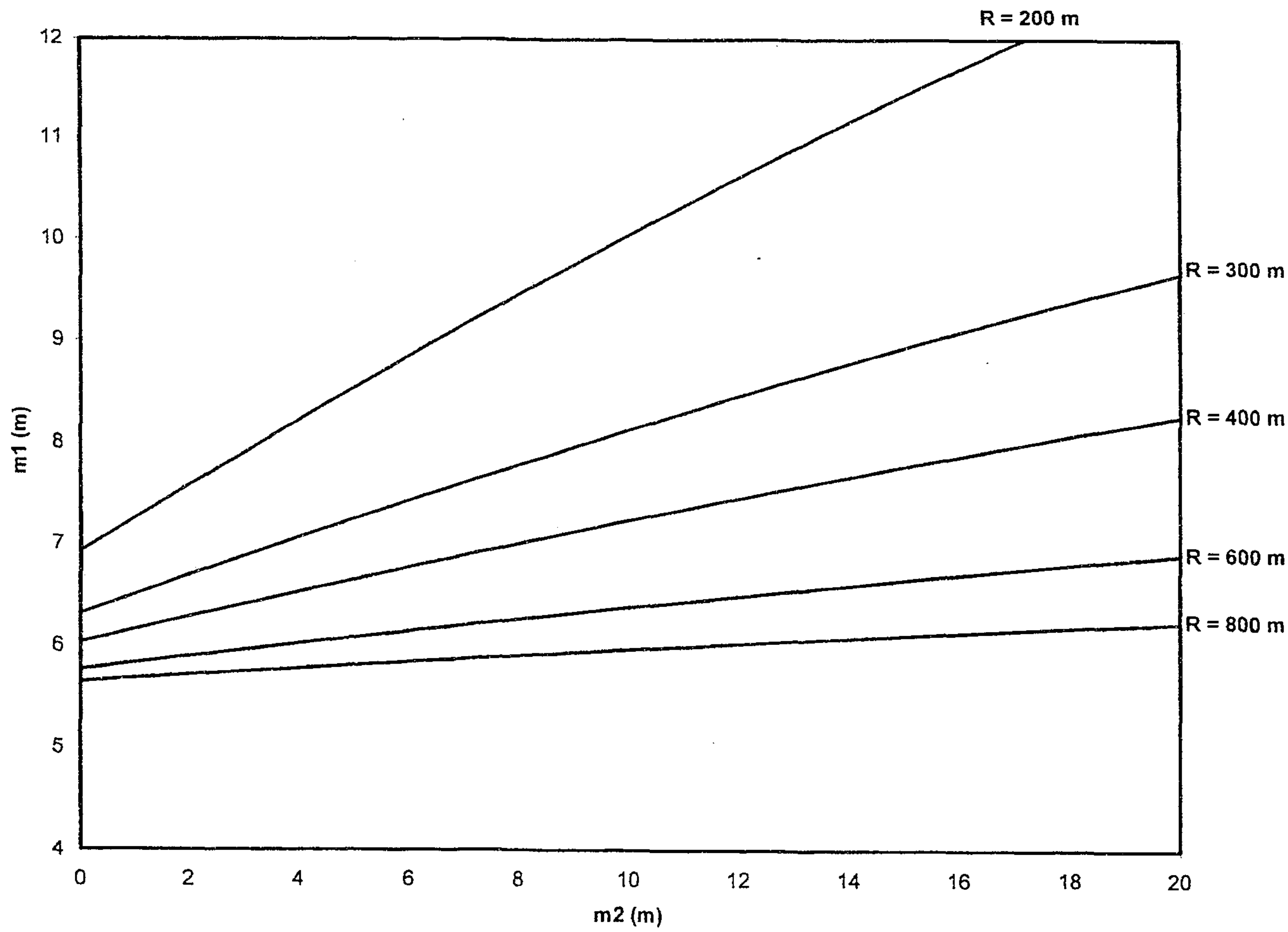

Figure D4.19 Design graph for P_f = 5% and CV = 5% for all variables (design speed = 80 km/h)

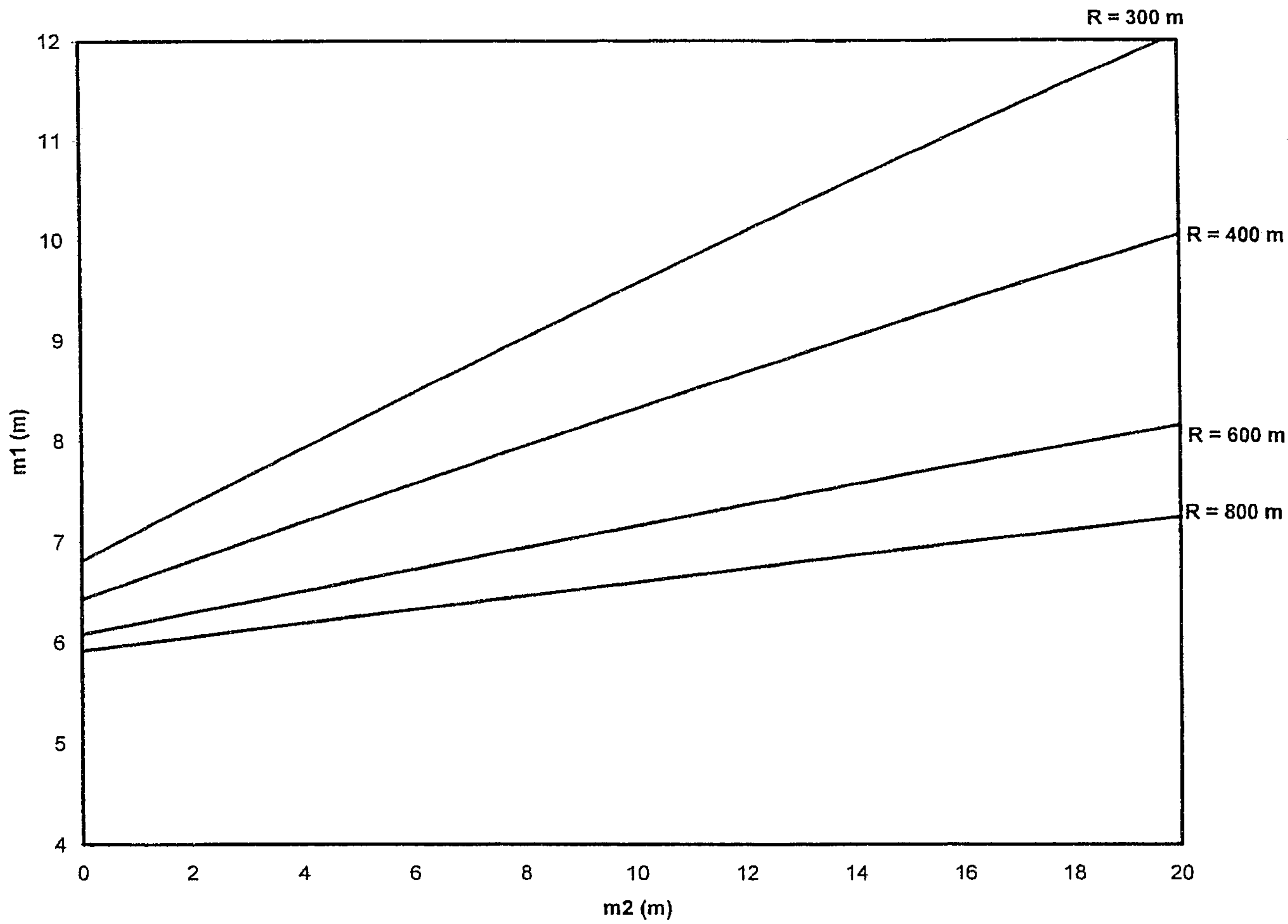

Figure D4.20 Design graph for P_f = 5% and CV = 5% for all variables (design speed = 100 km/h)

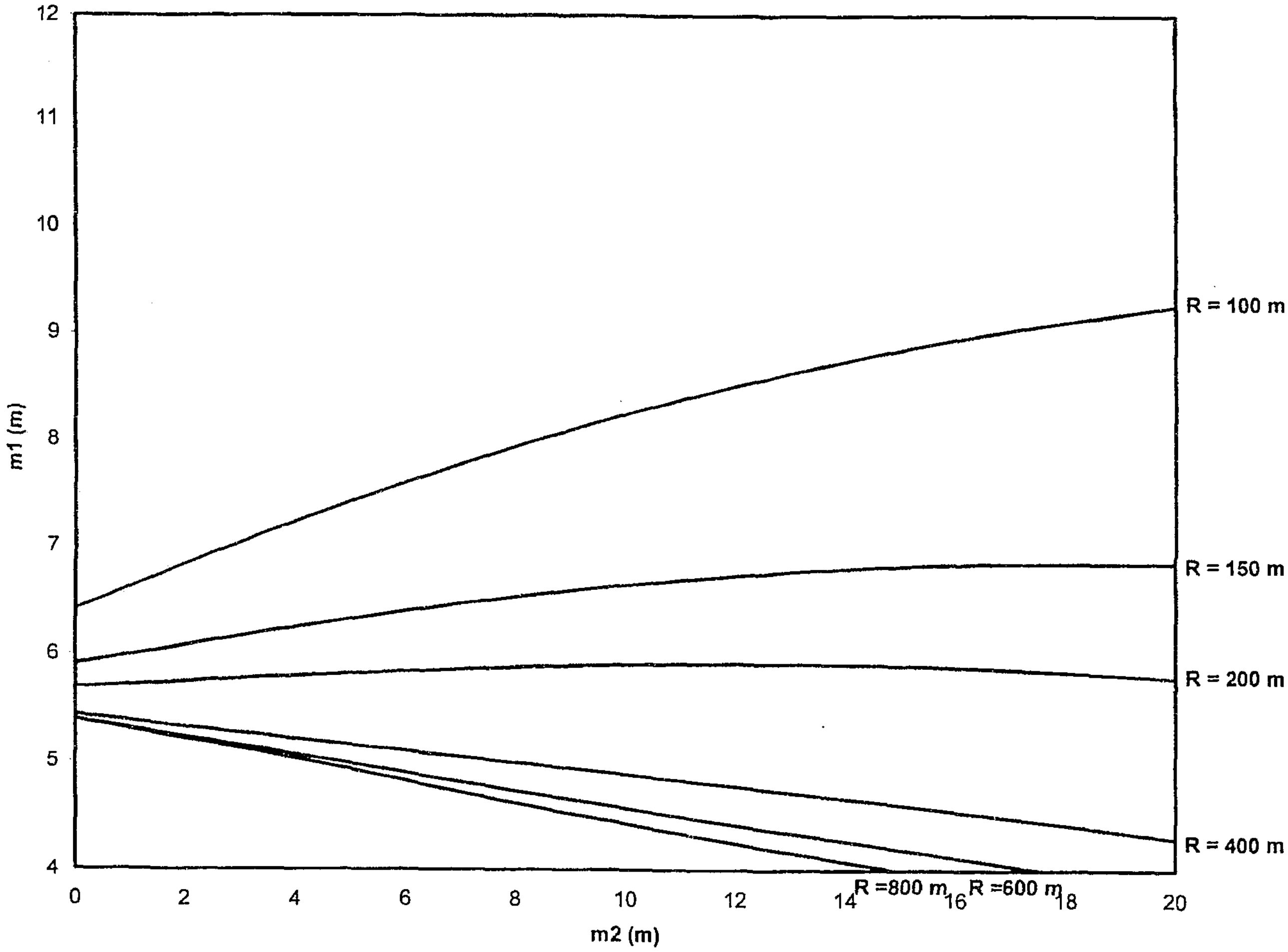

Figure D4.21 Design graph for P_f = 5% and CV = 10% for all variables (design speed = 40 km/h)

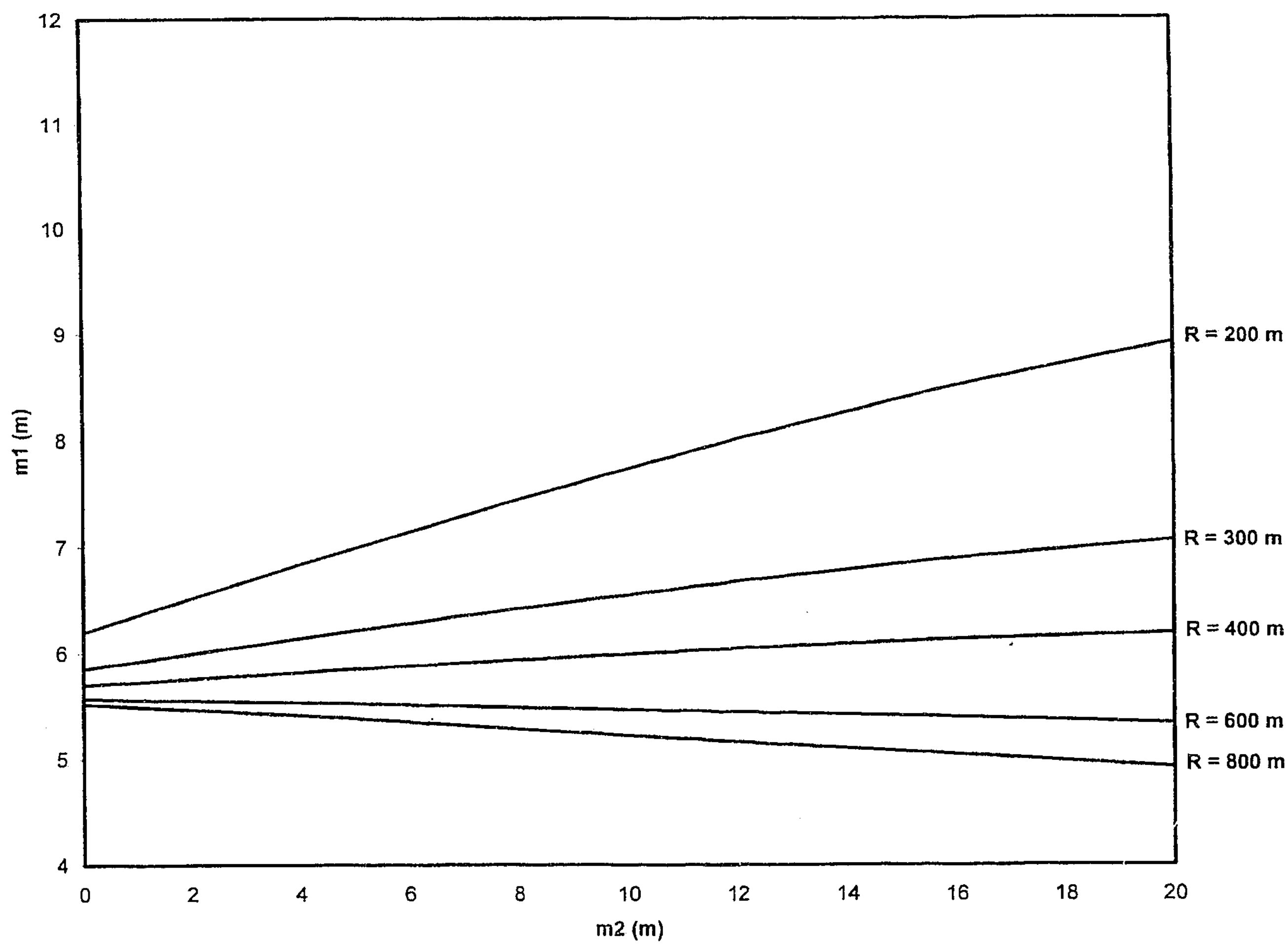

Figure D4.22 Design graph for P_f = 5% and CV = 10% for all variables (design speed = 60 km/h)

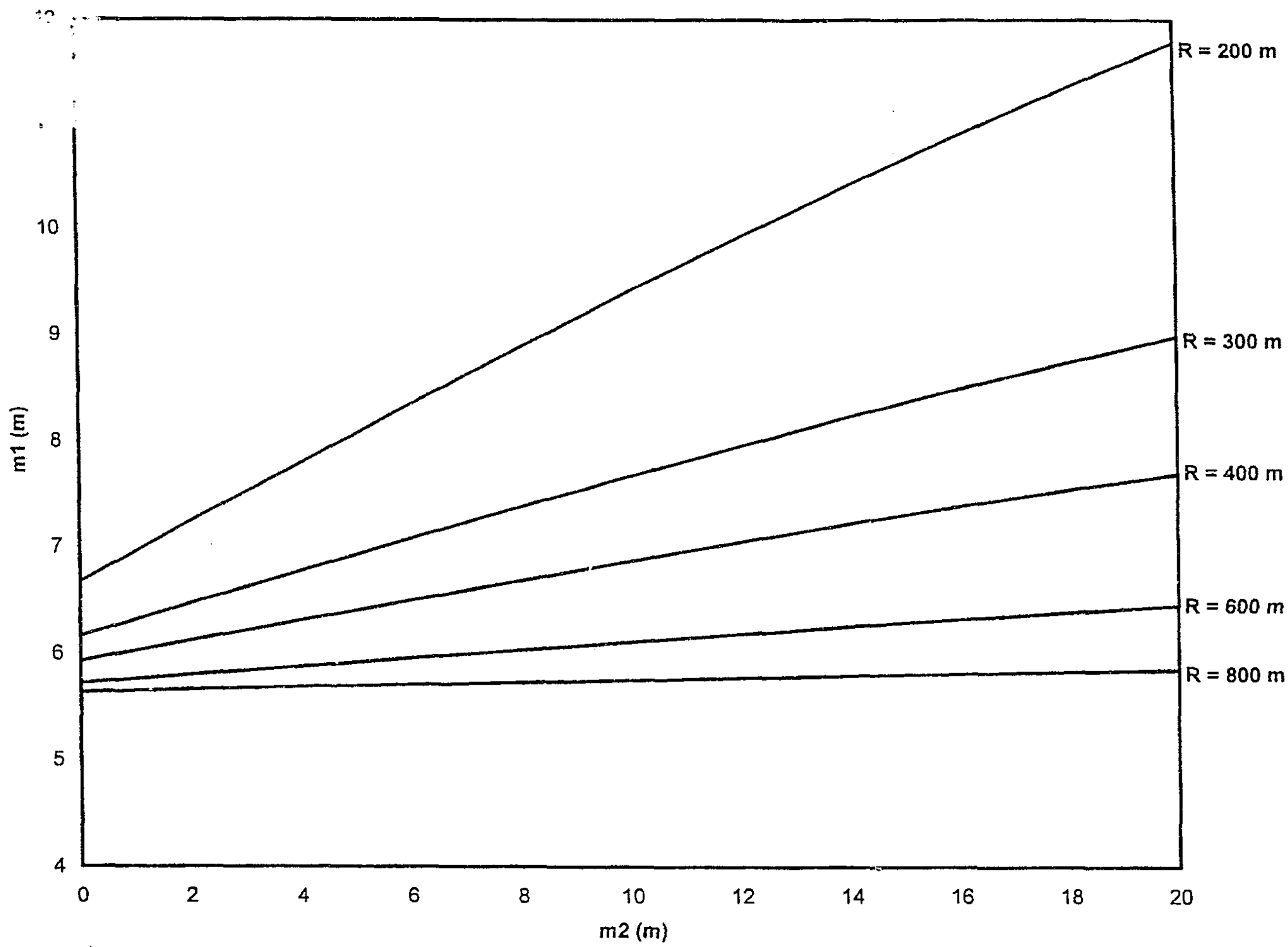

Figure D4.23 Design graph for P_f = 5% and CV = 10% for all variables (design speed = 80 km/h)

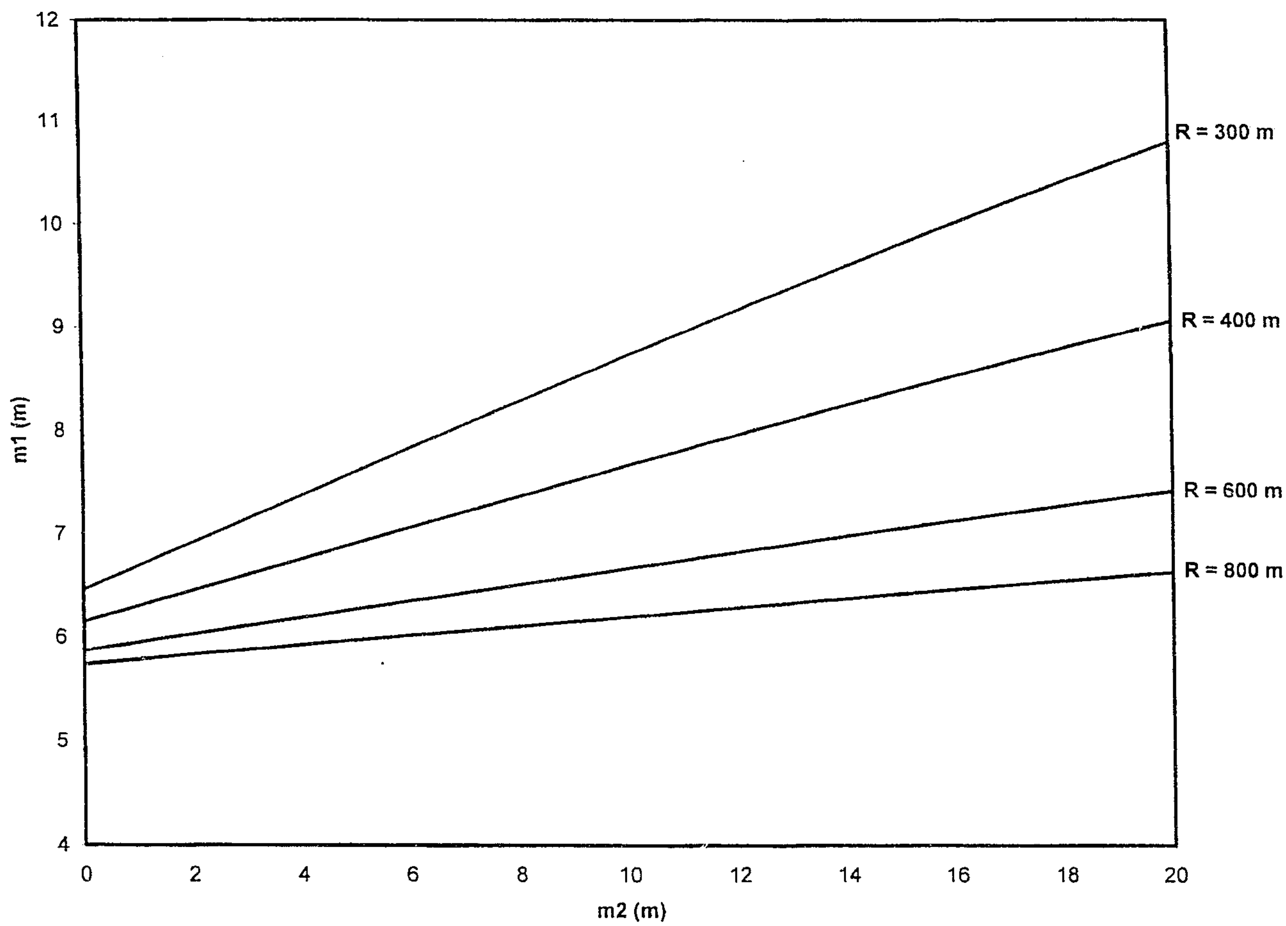

Figure D4.24 Design graph for P_f = 5% and CV = 10% for all variables (design speed = 100 km/h)

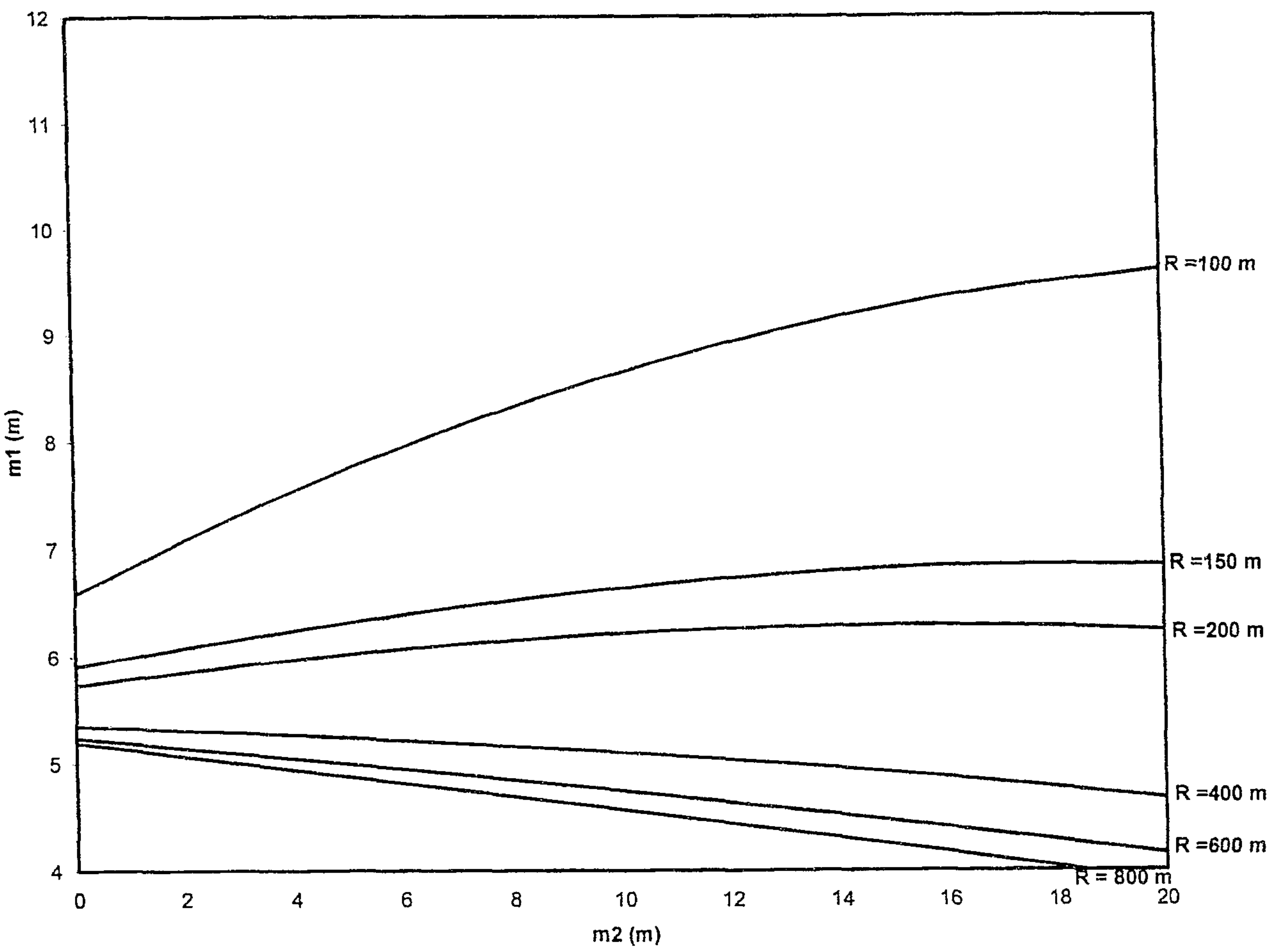

Figure D4.25 Design graph for P_f = 10% and CV = 5% for all variables (design speed = 40 km/h)

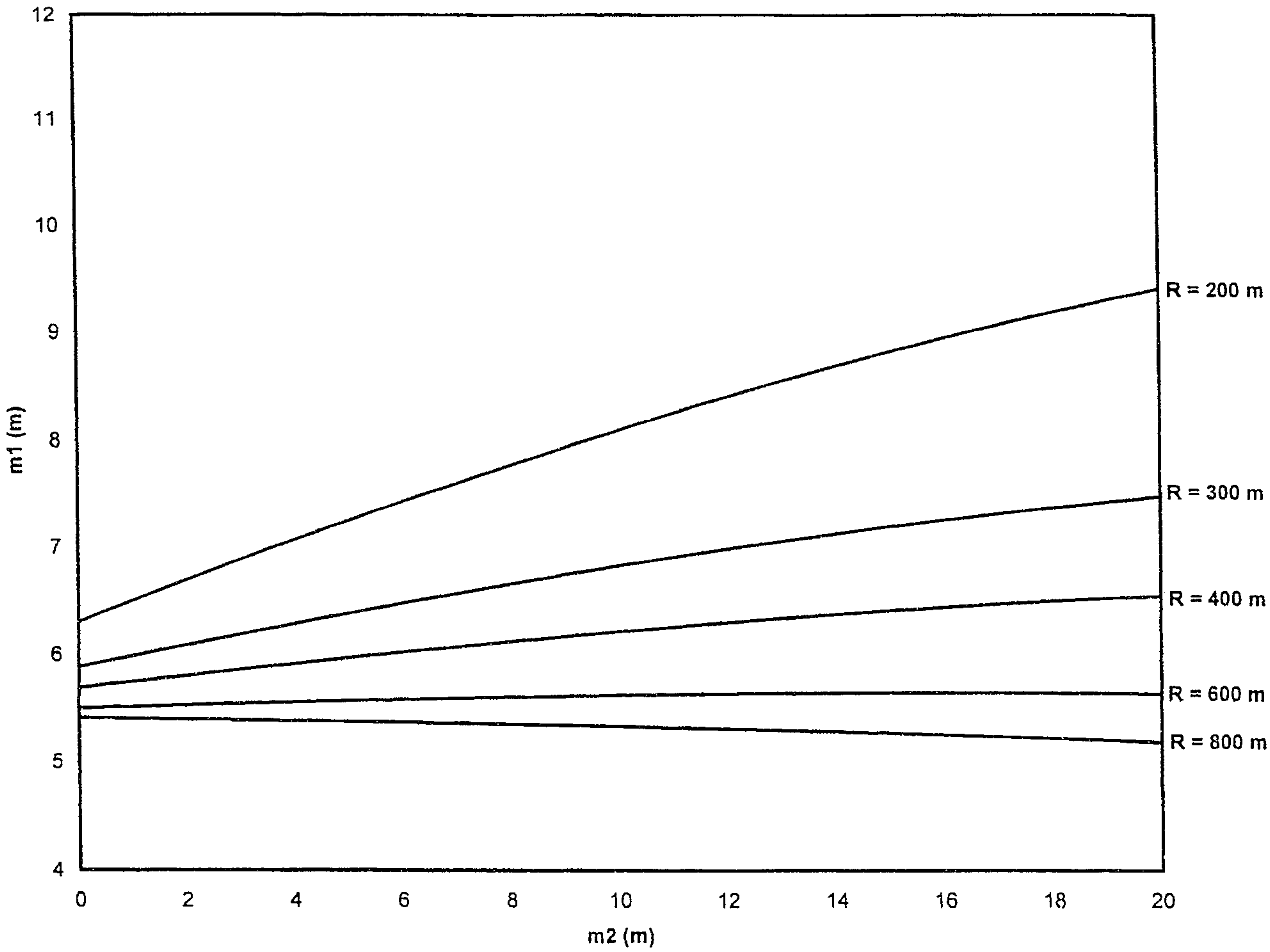

Figure D4.26 Design graph for P_f = 10% and CV = 5% for all variables (design speed = 60 km/h)

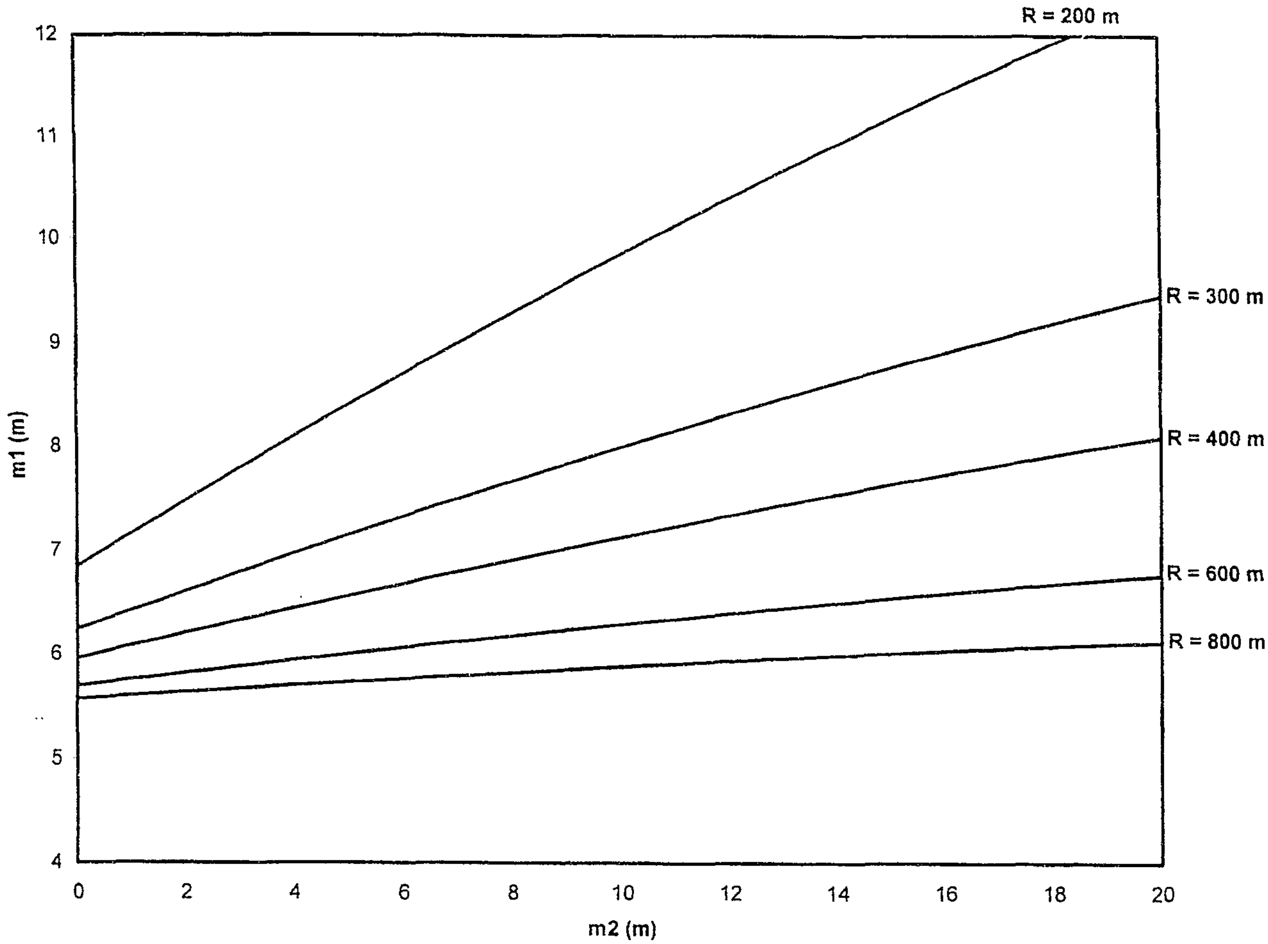

Figure D4.27 Design graph for P_f = 10% and CV = 5% for all variables (design speed = 80 km/h)

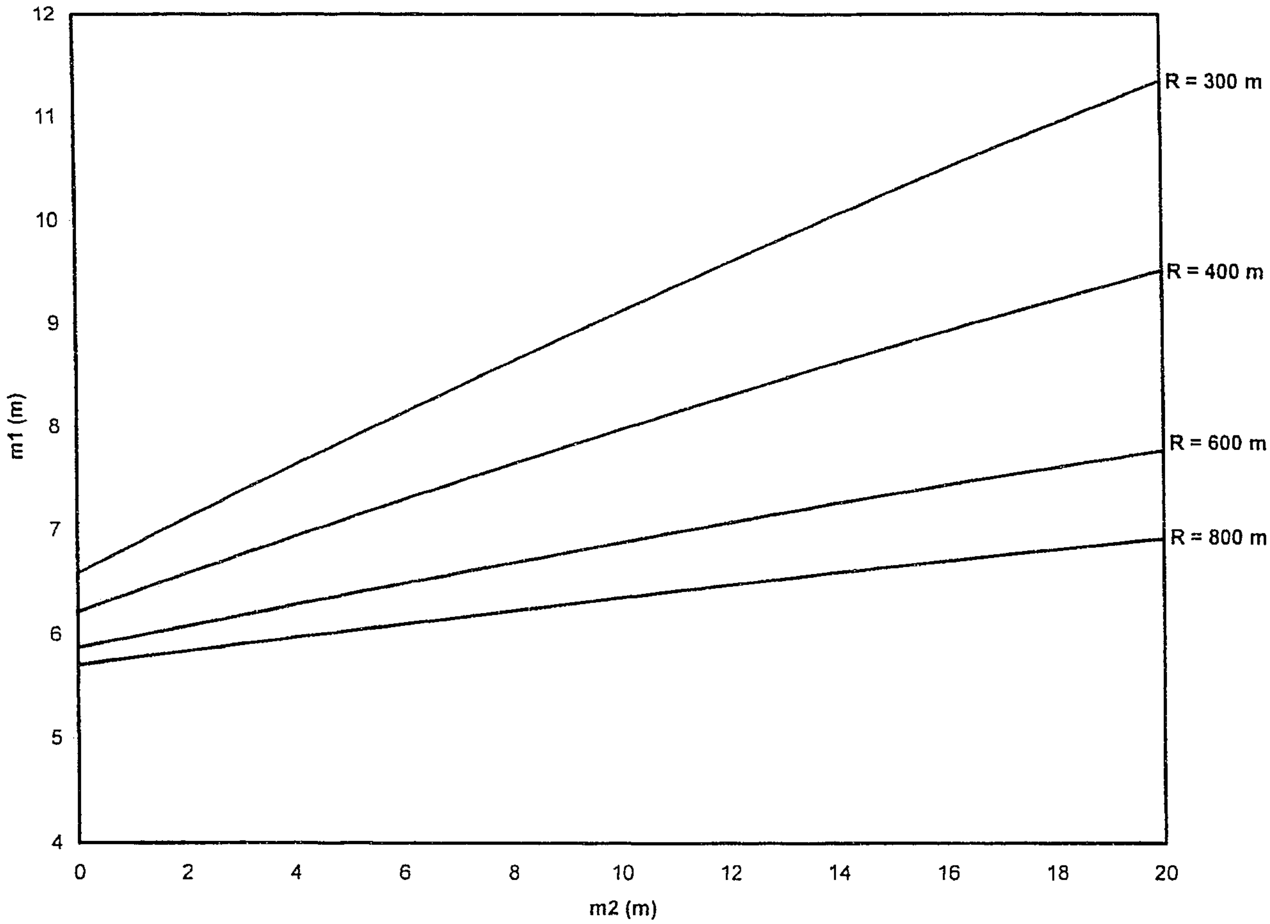

Figure D4.28 Design graph for P_f = 10% and CV = 5% for all variables (design speed = 100 km/h)

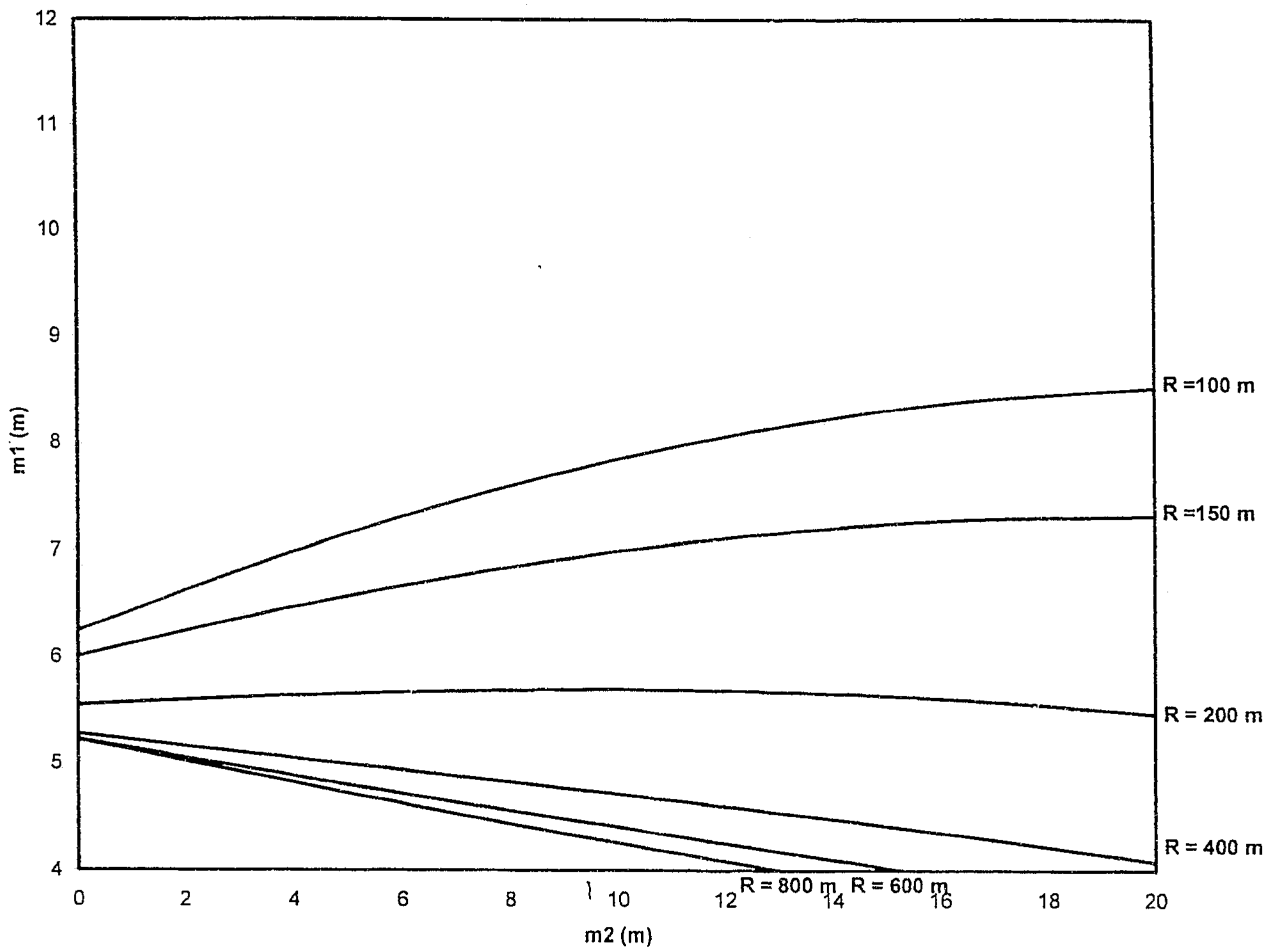

Figure D4.29 Design graph for P_f = 10% and CV = 10% for all variables (design speed = 40 km/h)

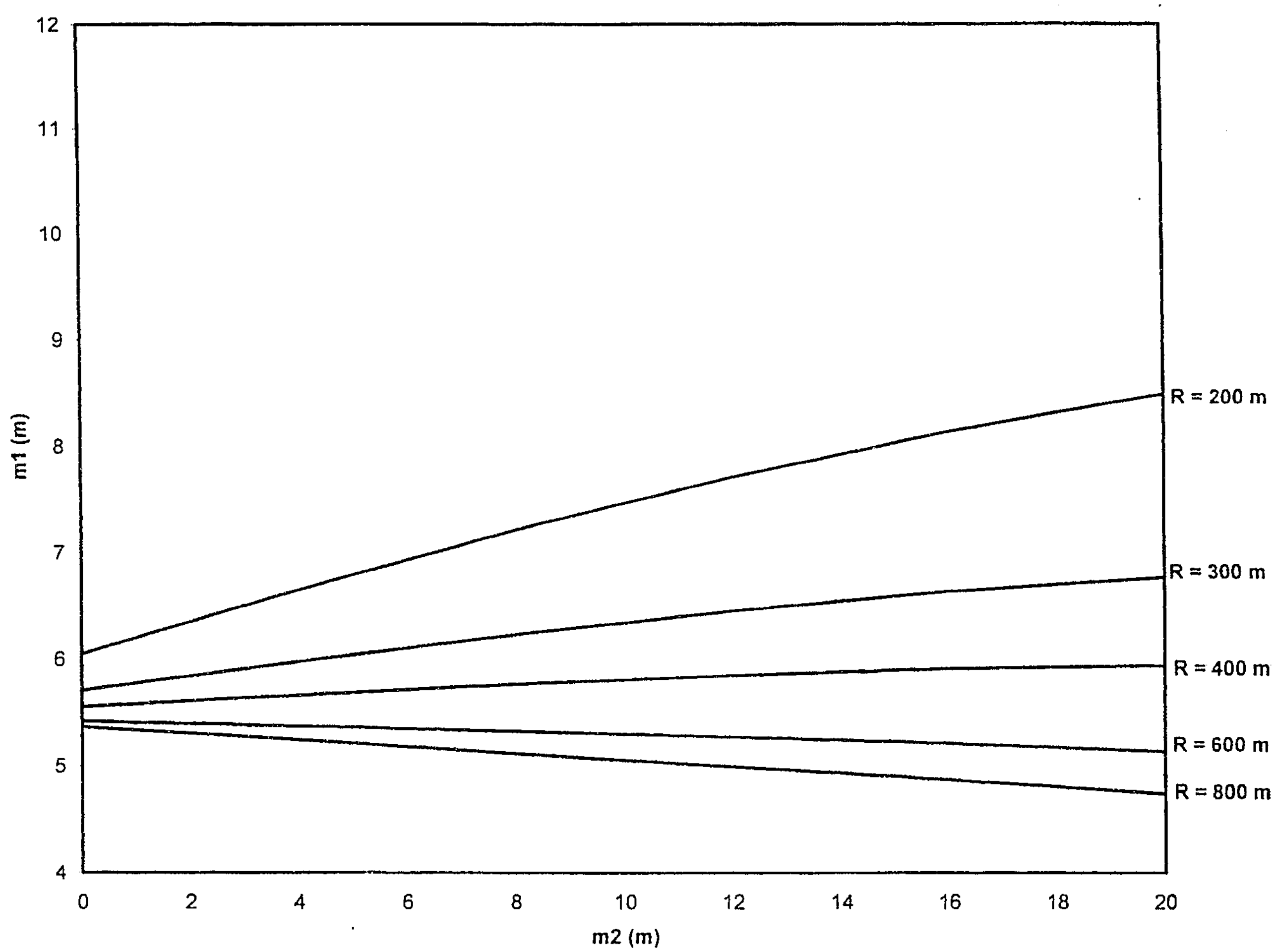

Figure D4.30 Design graph for P_f = 10% and CV = 10% for all variables (design speed = 60 km/h)

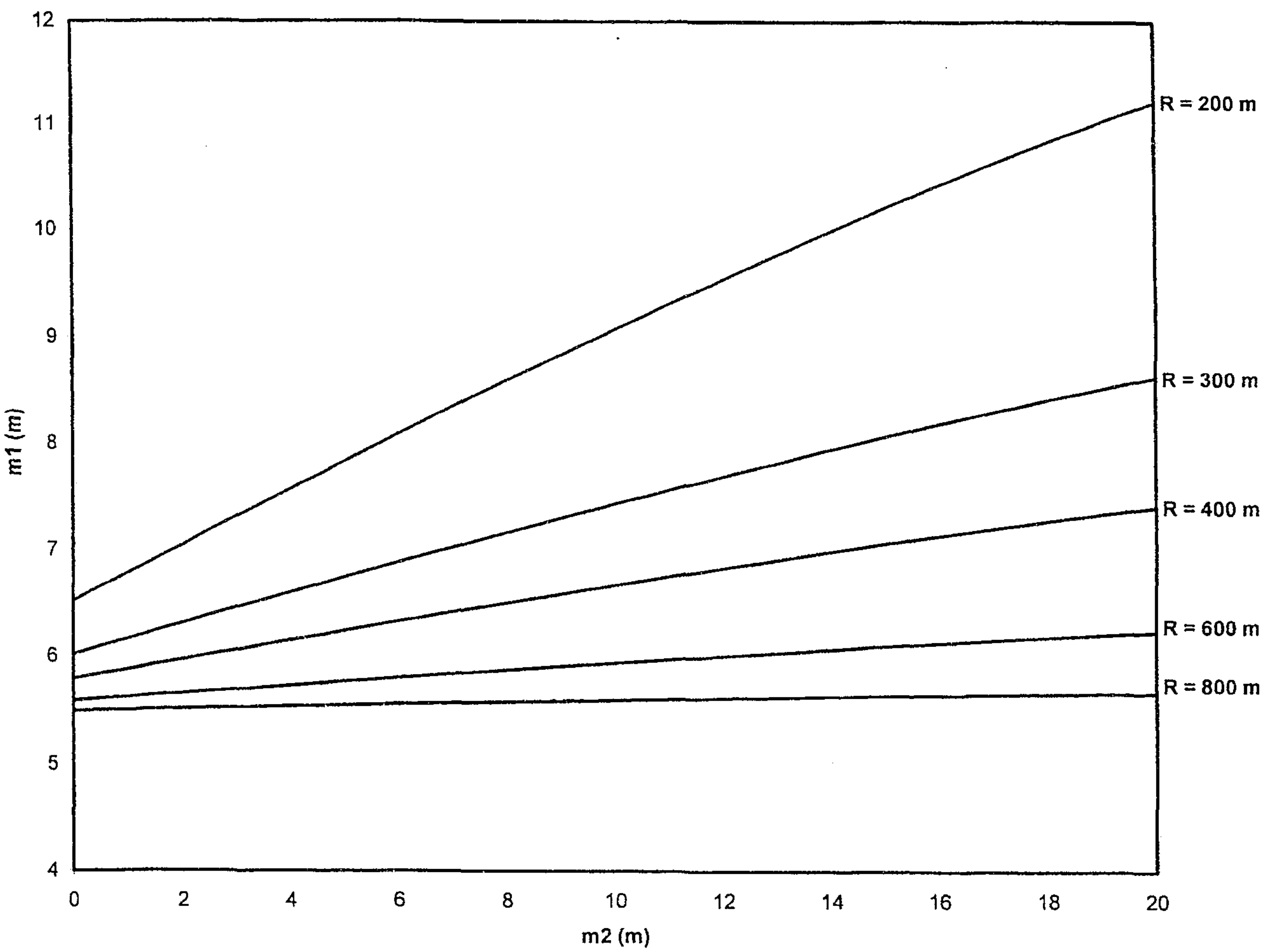

Figure D4.31 Design graph for P_f = 10% and CV = 10% for all variables (design speed = 80 km/h)

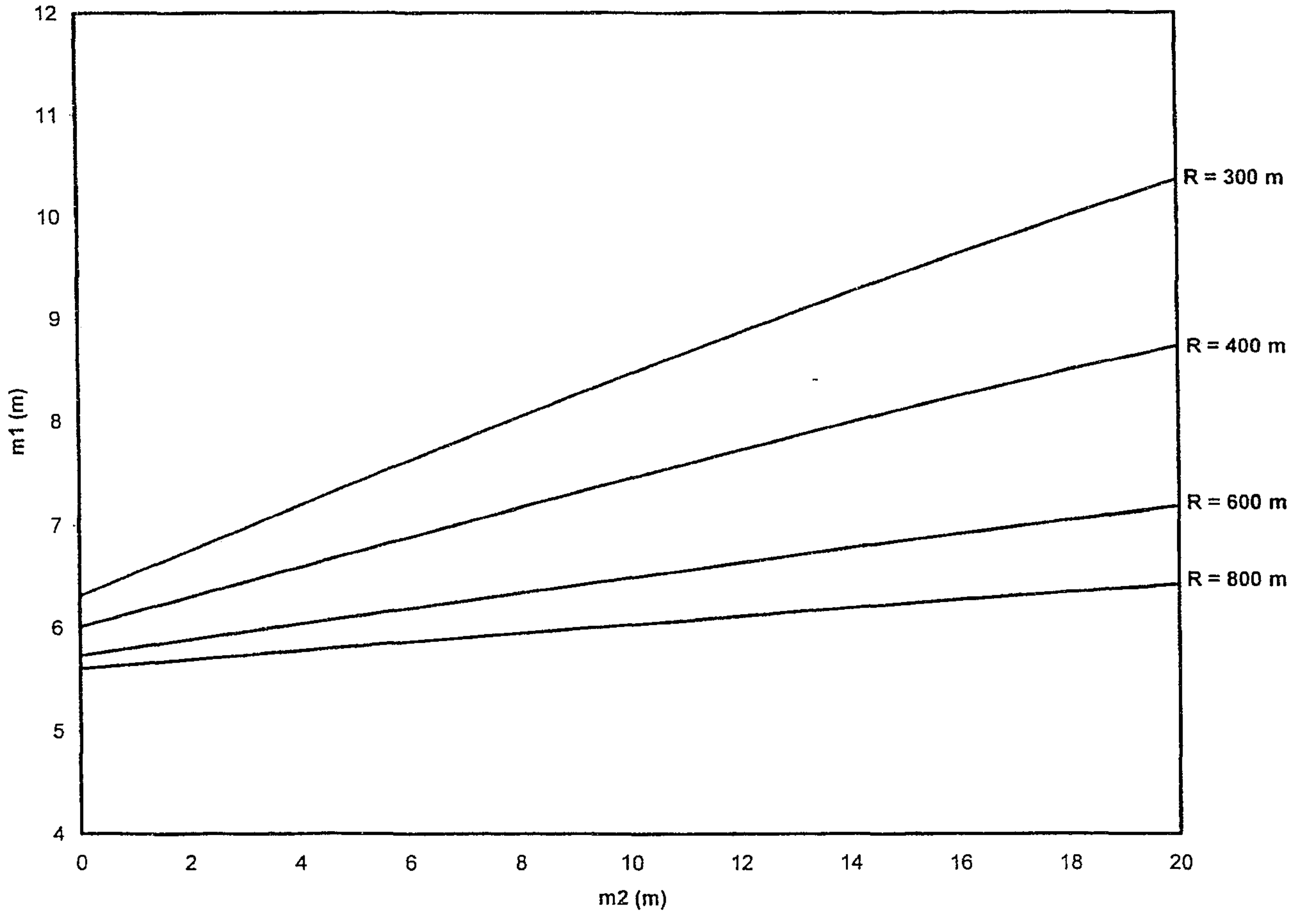

Figure D4.32 Design graph for P_f = 10% and CV = 10% for all variables (design speed = 100 km/h)