Procedures for Evaluating Traffic Capacity and Improvements to Road Geometry

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PROCEDURES FOR EVALUATING TRAFFIC CAPACITY
AND IMPROVEMENTS TO ROAD GEOMETRY

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ABSTRACT

The geometric standards of a road, such as width, minimum curve radius and maximum grades, can have a major effect on the costs of road construction and maintenance, and the speed, safety and vehicle operating costs experienced by those who travel on the road. This report investigates methods for determining appropriate levels of investment in road geometry. These may then be compared with pavement construction and maintenance requirements to make best use of scarce road funds.

A number of models are available for predicting speeds and operating costs for isolated vehicles as a function of road geometry. Of these the World Bank's Highway Design and Maintenance Model HDM-III was found to have some advantages. This report concentrates on the development of simple models for incorporating the effects of increasing traffic flow into the road evaluation process.

A macroscopic speed-flow model was derived for use with HDM-III, making use of the detailed free speed-geometry relationships already available. In its simplest form, this requires the estimation of only one additional parameter to predict the decline in mean free speed for each vehicle type with increasing traffic flow. The effect of road width on total transportation cost was also investigated. It was concluded that road width may have very little effect on speeds and operating costs at low traffic volumes (depending on sight distance), but that increasing traffic flow on a narrow road leads to reduced speeds, increased road deterioration and increased 'effective roughness' experienced by vehicles travelling partly on the road shoulder. Simple and approximate predictions of these effects were derived.

The proposed models of traffic volume and road width effects were incorporated into the HDM-III model and applied to case studies for India and Costa Rica. These demonstrated that increasing standards of road width and alignment can be economically justified as traffic volumes increase. The appropriate standard for a given volume, however, varies with the difficulty of construction, traffic composition, unit costs in a particular region, and the base case (null alternative) being considered.

The report provides recommendations for parameter estimation in new countries or regions. Further research is needed to test the new models against observed traffic behavior, and to extend and refine the models in several areas. In particular, the effects of overtaking opportunities may be important in the analysis of wide two-lane roads and the effects of sight distance, shoulder condition, and edge damage must be considered in the evaluation of narrow roads.
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I. INTRODUCTION

1.01 The geometric characteristics of a road describe the longitudinal alignment and cross-section, including the roadside features on either side of the travelled way. In the design of new roads and the upgrading of existing roads, standards are required for maximum grades, minimum curve radius and sight distance, and appropriate selection of road width, number of lanes, shoulder width, crossfall and many other parameters.

1.02 The choice of geometric standards can have a major influence on the costs of construction and maintenance, and the costs and benefits experienced by road users and others affected by the road.

1.03 This paper investigates the effects of geometric improvements on the overall costs and benefits of a road project. The main objective of the paper is to develop a framework for evaluating changes in geometric characteristics, so that appropriate standards can be established for particular cases. Appropriate standards are likely to vary considerably from one region or country to another, depending on terrain, climate, traffic volume and composition, and regional cost structures for vehicle operation, road construction and maintenance.

1.04 A further objective of this paper is to review and assess methods for predicting vehicle operating costs, and the extent to which these methods take account of road geometry characteristics. While various models are available for predicting these costs for free-flowing traffic on relatively wide roads, the modelling of traffic interaction and narrow road widths is currently inadequate. Hence, much of this paper is devoted to developing approximate procedures for taking account of these effects. The paper also considers the estimation of marginal construction costs with changes in geometric standards, and the sensitivity of predicted geometric standards to uncertainties in the input parameters.

A. Framework for Evaluation of Geometric Characteristics

1.05 The geometric characteristics which are of interest to this study include:

- Vertical alignment;
- Horizontal alignment;
- Road width and number of lanes;
- Shoulder width and type;
- Sight distance; and
- Roadside characteristics

The evaluation of geometric standards should consider the effects of changes in any of the above characteristics on the following aspects of road transport costs and benefits:

- Construction costs;
- Maintenance costs;
It is convenient to discuss all of these impacts in economic terms, so that the objective of an optimum design is to minimize total transportation costs. The requirement for this evaluation is then to take any combination of geometric characteristics in the first list and to predict the impacts in the second list. Information on road surface type, roughness, cost parameters and traffic volume and composition are also required, since these could have a large effect on these relationships. The process is illustrated in Figure 1. In practice, the evaluation of geometric standards cannot be discussed in isolation. The range of candidate improvement options will vary considerably with terrain, climate, traffic volume, economic constraints and the standards and condition of existing roads.

**FIGURE 1 Requirements for a Road Evaluation Procedure**
At very low traffic volumes, design decisions are mainly concerned with appropriate alignment and road surface standards. As traffic volume increases, vehicle interactions result in increased delays, vehicle operating costs and road deterioration, especially at the edges of narrow seals. Decisions are then required on provision of adequate shoulders and widening of single-lane and narrow two-lane roads. At yet higher volumes, traffic on two-lane roads may experience delays because of inability to overtake slower vehicles. In these situations the provision of overtaking opportunities—whether by improved alignment, passing lanes or ultimately widening to four lanes—may become the major road improvement requirement. Special treatments for turning vehicles and roadside disturbances also become increasingly important at higher traffic volumes.

At all traffic volumes, appropriate standards of alignment and road surface quality must be assessed. Appropriate safety standards should also be evaluated, since these are closely related to geometry and surface characteristics. These standards generally increase with traffic volume, since the benefits from a given investment increase as more users realize the savings in accident and vehicle operating costs.

This paper is generally concerned with the evaluation of geometric characteristics at an aggregate level, using measures such as aggregate curvature and rise and fall, or overall design speed and maximum grade. The details of design standards and the interrelationship of road elements are discussed only where these have a significant influence on total cost.

**B. Macroscopic Evaluation Models**

There are several macroscopic road evaluation models which perform many of the tasks illustrated in Figure 1, including benefit-cost analysis over a twenty to thirty year evaluation period. Three examples are the World Bank's HDM-III model (Watanatada, et al. 1987a), the British RTIM2 model (Parsley and Robinson 1982) and the Australian NIMPAC model (NAASRA 1984). All of these models provide mechanisms for evaluating transportation costs on existing roads, discounting future costs back to base-year values, and programming improvement projects within budget constraints.

However, none of the models cover all of the items shown in Figure 1 to a sufficient level of detail. This is not surprising, since these models are designed for broad planning evaluations, rather than a more detailed analysis of small increments of geometric standards. Much of their logic is directed towards construction and maintenance strategies, which are not of interest in this study.

Of the three evaluation models, only NIMPAC incorporates accidents and speed-volume relationships. However, the methods used for modeling road geometry and traffic volume effects are quite approximate, and are not well validated at this stage. The model does not appear to have been applied outside Australia. Specific components of this model are discussed further in Chapters III and IV.
1.13 The RTIM2 and HDM-III models are similar in many respects, since both grew out of previous research initiated by the World Bank and Massachusetts Institute of Technology (Moavenzadeh 1972). However, HDM-III has a major advantage for the purposes of this study, in that it provides two alternative approaches for modelling geometric effects on speed:

+ Linear additive regression models as used in RTIM2; and
+ Limiting speed models based on mechanisms of vehicle and driver behavior.

1.14 As will be shown in the following section, the limiting speed concepts represent an important advance in the modelling of speed-geometry relationships, both at the macroscopic and microscopic levels. Because of its flexibility and its increasing application in project evaluations, HDM-III is adopted as a major component of this study. The procedures developed here, however, could also be incorporated into other macroscopic evaluation models. Before proceeding with this study, it is important to identify the range of applicability of HDM-III, its assumptions and limitations, and the aspects of Figure 1 which are not modelled and require further information.
II. GEOMETRIC RELATIONSHIPS IN THE HIGHWAY DESIGN AND MAINTENANCE STANDARDS MODEL (HDM-III)

A. Linear Additive Speed-Geometry Regression Equations

2.01 The HDM-III includes linear speed-geometry equations developed from field studies in Kenya (Hide et al. 1975), the Caribbean (Hide 1982, Morosiuk and Abaynayaka 1982) and India (CRRI 1982, Chesher 1983). Equations of this type are also used in the RTIM2 model (Parsley and Robinson 1982). A typical equation for Kenya (Hide et al. 1975), for passenger cars on a paved road more than 5 m in width, is:

\[ V_C = 102.6 - 0.372 \times RS - 0.076 \times FL - 0.111 \times C - 0.049 \times A \]  (1)

where:
- \( V_C \) = mean speed of cars (km/h);
- \( RS \) = average rise in m/km;
- \( FL \) = average fall in m/km;
- \( C \) = aggregate curvature in degrees/km; and
- \( A \) = altitude in m above sea level

A full set of these equations is given in Watanatada, et al. (1987a) and these are discussed and compared in Chesher and Harrison (1987), and in Bennett (1986).

2.02 Regression equations of this type have two fundamental problems. First, there is very little consistency of coefficients across a number of studies of this type. McLean (1982) and Bennett (1985) give graphic illustrations of this inconsistency. In Table 1, for example, McLean (1982) compared simulated vehicle performance over a range of specific trial alignments in Tasmania with the regression predictions of Duncan (1974), Hide et al. (1975) and Brewer et al. (1980). McLean concluded that:

"These [results] indicate the substantial spread of the predictions and the general lack of consistency between the prediction methods, suggesting that empirical speed-geometry relations are of doubtful validity outside of the particular region and road geometry configurations for which they were developed."

2.03 This inconsistency arises in part because road characteristics are often correlated; for example curves and grades are often found together. When this occurs the regression procedure is not able to isolate the effects of curves and grades separately, and may incorrectly allocate the effects of one road characteristic to the coefficient of the other.
Table 1 Comparison of Journey Speed Predictions

<table>
<thead>
<tr>
<th>Trial Alignment</th>
<th>Parameter Values</th>
<th>Predicted Car Speeds (km/h)</th>
<th>Predicted Truck Speeds (km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1(1)F</td>
<td>79.4</td>
<td>0</td>
<td>33.6</td>
</tr>
<tr>
<td>1(1)R</td>
<td>79.4</td>
<td>0</td>
<td>33.6</td>
</tr>
<tr>
<td>1(3)F</td>
<td>87.7</td>
<td>0</td>
<td>31.2</td>
</tr>
<tr>
<td>1(3)R</td>
<td>87.7</td>
<td>0</td>
<td>31.2</td>
</tr>
<tr>
<td>1(5)F</td>
<td>96.0</td>
<td>0</td>
<td>26.4</td>
</tr>
<tr>
<td>1(5)R</td>
<td>96.0</td>
<td>0</td>
<td>26.4</td>
</tr>
<tr>
<td>2(1)F</td>
<td>87.7</td>
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<td>140</td>
<td>54.9</td>
</tr>
<tr>
<td>2(2)R</td>
<td>83.5</td>
<td>140</td>
<td>54.9</td>
</tr>
<tr>
<td>2(3)F</td>
<td>75.2</td>
<td>174</td>
<td>52.5</td>
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<td>3(2)R</td>
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<td>29.4</td>
</tr>
<tr>
<td>3(3)R</td>
<td>91.9</td>
<td>103</td>
<td>29.4</td>
</tr>
</tbody>
</table>

Source: McLean (1982)

2.04 The second problem is that the regression equations do not behave well as they move towards extreme conditions. This occurs because the equations are derived from curve-fitting procedures rather than an understanding of the underlying traffic behavior. Many such equations yield negative speed values for reasonable combinations of road characteristics (Watanatada, et al. 1987a). Futhermore, the additive nature of the equations implies that each geometric characteristic has a constant effect regardless of the value of other geometric features. This means, for example, that speeds on a steep upgrade could be increased by making the surface smoother, when in fact the gradient is the speed-limiting factor.

2.05 Watanatada, et al. (1987a) refer to this as "lack of asymptotic consistency" in that linear regression models fail to asympotote towards a limiting value as one geometric constraint becomes dominant. They note that this could be partly overcome using non-linear regression with interaction terms, but this would only increase the problems of reliability and transferability discussed above.

B. Minimum Limiting Speed Approach

2.06 An alternative method of speed-geometry modelling was developed by Watanatada, et al. (1987a), using extensive field data collected in Brazil (GEIPOT 1981). Briefly, the model assumes that each vehicle has a set of limiting speeds for open roads (VDESIR), curves (VCURVE), upgrades (VDRIVE), downgrades (VBRAKE) and rough surfaces (VROUGH). At a given
point, the speed of the vehicle will be the lowest of these five limiting values. That is, the driver will attempt to maintain his desired speed \( V_{\text{DESIR}} \) subject to the other four constraints. The actual speed \( V \) can then be expressed as:

\[
V = \min (V_{\text{DRIVE}}, V_{\text{BRAKE}}, V_{\text{CURVE}}, V_{\text{ROUGH}}, V_{\text{DESIR}})
\]  

(2)

2.07 This equation applies to a single vehicle in the traffic stream. In order to determine the average speed for a given class of vehicles, some assumptions are required about the distribution of these constraining speeds across all vehicles over a homogeneous road section.

2.08 The model then goes through three further stages of development to derive a macroscopic form which can be fairly readily calibrated and applied at an aggregate planning level:

- The limiting speeds are assumed to be randomly and independently distributed, so that an expression can be developed for the mean of the minimum speeds as a function of the component distribution means.
- Component modelling of the five limiting speeds attempts to relate these speeds to vehicle and road characteristics.
- Model aggregation of model concepts from individual homogeneous road sections to hypothetical aggregate sections allows an evaluation of the errors introduced by this process.

2.09 The aggregate model thus provides an equation for mean speed on a road link as a function of five limiting mean speeds. The component equations relate limiting speeds to road geometry and roughness characteristics. In addition to basic data on vehicle mass, area and drag coefficients, the model requires estimation of five vehicle/driver parameters and a shape parameter \( \beta \) which is related to the variance of the component speed distributions. Watanatada, et al. (1987a) used regression analysis to estimate these parameters from field data.

2.10 The limiting speed model is conceptually more satisfactory than simple linear additive concepts, since it is more firmly based on actual vehicle and driver behavior. This should overcome the problems of asymptotic inconsistency discussed in the previous section. The form of the model is more complex than the linear equations, but this presents no problem for computer-based evaluation procedures. However, the model makes use of many assumptions to express vehicle behavior in simple forms, and the validity of these must be examined.

2.11 The use of regression to estimate model parameters could lead to the same problems of inconsistent coefficients as discussed for linear models in para. 2.02-2.05. This can only be overcome by the use of data sets in which road characteristics are not highly correlated, or by estimating some parameters by other methods. However, the use of a model based
on sound principles of driver behavior provides much greater opportunity for testing the reasonableness of regression results and substituting more "reasonable" estimates if appropriate.

2.12 In practice, road characteristics in a given country or region often tend to be highly correlated, and it is important to select a wide range of conditions if regression results are to be applied to other regions. Where parameters are estimated by other methods (e.g. controlled studies of desired speeds or braking performance), the overall model predictions must still be calibrated against field data.

Model Formulation

2.13 In order to derive a simple expression for the mean minimum speed, the model assumes that each of the constraining speeds has a distribution across all vehicles of a given type, that these distributions are mutually independent and all have the same variance. Watanatada, et al. (1987a) then use the Gumbel distribution to derive the following expression for the mean speed of vehicles on a given road section.

\[
\bar{V} = E \left[ (V_{\text{DRIVE}} + V_{\text{BRAKE}} + V_{\text{CURVE}} + V_{\text{ROUGH}} + V_{\text{DESIR}} ) \right] ^{-\beta}
\]

where:
- \( \bar{V} \) = mean speed on a given road section;
- \( V_{\text{DRIVE}}, V_{\text{BRAKE}}, V_{\text{CURVE}}, V_{\text{ROUGH}}, V_{\text{DESIR}} \) = mean constraining speeds for the road section;
- \( \beta \) = shape parameter related to the (constant) variance of the constraining speed distributions; and
- \( E \) = bias correction factor to allow for logarithmic conversions used in model estimation.

2.14 If \( V \) follows a lognormal distribution (usually a reasonable assumption for speeds), and the standard derivation of the log of \( V \) is \( \sigma \), then it can be shown that

\[
\frac{E}{\beta} = \exp \left( \frac{1}{2} \sigma^2 \right)
\]

and the coefficient of variation of \( V \) is \( \sigma \).

2.15 In the estimation of \( V \) or the model parameters from field data, it should be noted that the correct value of \( \sigma \) will depend upon the level of aggregation of the data being used. Clearly the value of \( \sigma \) for speeds at a given site, or mean speeds at many sites, will be less than \( \sigma \) for the entire data set.

2.16 Watanatada, et al. (1987a) estimate \( \beta \) empirically in the regression analysis rather than relating it to the variance of observed speeds.
When $\beta$ is significantly greater than zero (reflecting some spread of speeds about the mean) and more than one constraint is affecting speeds, then the predicted value of $V$ may be ten to fifty percent lower than the minimum constraining mean speed (Watanatada, et al. 1987a). This "drawdown" is to be expected if the constraining speeds are truly independently distributed, since it can be shown that the mean of minimums of random variables will always be less than the minimum of the means. However, the magnitude of the drawdown in some cases seems rather large.

2.17 In practice, we would expect some positive correlations between desired speeds of individual vehicles and their maximum speeds on curves, grades and rough surfaces. It also seems unlikely that all constraining speeds would have the same standard deviation, especially on road sections where a given constraint is not effective (e.g., curve speed constraint on straight roads). Of course some statistical assumptions are essential for estimation of this nature, but it is not clear what biases may be introduced by this process. The empirical estimations of $\beta$ may well overcome some of these difficulties for a given set of conditions, but this estimate may not be readily transferrable to other conditions.

2.18 In applying this model to new situations it is important to understand the role of $\beta$ and its associated assumptions about the distributions of constraining speeds. If $\beta$ were zero, the mean speed for a given vehicle type and road section would be equal to the lowest of the mean constraining speeds for those conditions. However model calibration has yielded $\beta$ values of 0.24–0.31 for Brazil and 0.59–0.68 for India, suggesting a wide variability in constraining speeds.

2.19 While the physical reasons for these high values (especially in the case of India) are not clear, a practical consequence is that predicted speed is well below the minimum mean constraining speed in many cases. To balance this effect, the estimation procedure appears to produce unrealistically high values of desired speed. Users of the model should be aware of the influence of $\beta$ when comparing parameters such as $V_{DESIR}$ with data from other sources.

Component Modelling

2.20 Watanatada, et al. (1987a) develop a set of equations for modelling each of the five limiting speeds in this model, in terms of road curvature, rise and fall, superelevation and roughness. In order to keep these equations in a simple form requiring few parameters, a number of assumptions are required, and these are presented in Appendix A. It is important that these assumptions are documented, so that they may be re-evaluated in new applications of the model, and any limitations on the range of model validity can be noted. Two limitations of particular interest to this study are:

- The mean desired speed $V_{DESIR}$ is assumed to be unaffected by road geometry characteristics. This conflicts with research by McLean (1980) showing that desired speeds are strongly affected by aggregate curvature and sight distance.
The model also assumes that minimum curve speed \( V_{\text{CURVE}} \) is unaffected by overall road curvature, and that minimum downhill speed is not affected by local curvature. At the micro level, these assumptions could lead to substantial inaccuracies (e.g., McLean 1979). However, they should cause little difficulty in macroscopic modelling, at least for the types of terrain combinations used in model calibration.

2.21 These assumptions, as well as the others listed in Appendix A and the rest of this section, mean that the prediction of speed using this model is not exact. The calibration and validation of model parameters against observed behavior provides some compensation for these uncertainties. However, the calibration process allows the possibility of "coefficient sharing" where the effects of one parameter are incorrectly assigned to another. Because of this, the model should be used with caution when applied outside the region and conditions for which it was calibrated.

2.22 A more important limitation for the purposes of this study is the lack of any modelling procedure to take account of road width and traffic volume. Techniques for evaluating these effects within the HDM model structure are discussed in Chapters III and IV.

**Aggregate and Microscopic Modelling**

2.23 In the study of the Brazil free speed data, Watanatada, et al. (1987a) applied the limiting speed concept at three levels of aggregation:

- The **Micro Transitional Model** divided a road link into short homogeneous sections, and used a process of backward and forward recursion to take account of transitional effects from one section to the next. The transitional process showed that speeds on many sections remained higher or lower than the predicted steady-state speeds, because of the influence of upstream speeds and short section lengths.

- The **Micro Non-Transitional Model** still used short homogeneous road sections, but ignored transition effects and assumed that steady-state speeds were applicable on each section.

- The **Aggregate Model** treated two-way travel on a road link in terms of just two road sections; one uphill and one downhill. Geometric characteristics were aggregated so that each extended road section could be described in terms of a single measure each of curvature, rise and fall, roughness etc.

2.24 Field data from six road sites were used to test the validity of the first and third models. Watanatada et al. (1987a) found that the Micro Transitional Model provided close fits to the data, with typical \( R^2 \) values of 0.64 for cars and 0.72 for medium trucks, and standard errors of 1.4 and 1.5 m/s respectively. The predictions of the Aggregate Model were almost
as good, indicating that very little error was introduced by the processes of aggregation and ignoring transitional effects.

2.25 It should be noted that the test sections of road were quite short (2 to 4 km) without severe values of curvature, roughness or aggregate grade, or combinations of these. The testing of aggregation for these sites may not be applicable to all situations.

2.26 To overcome this problem, Watanatada, et al. (1987a) used a further five field sites with more severe geometry and 10 km lengths (but without field data) to compare the three models with each other. This theoretical exercise confirmed that the aggregate model gives predictions very close to those of the micro transitional model, while the micro non-transitional model introduces a small downward bias. It appears that the aggregate model introduces a further bias in the opposite direction, so that the two effects partially cancel out. A further application of the aggregate model to 41 bus routes in Brazil confirmed its ability to provide reasonable fits to observed traffic data over route lengths of 50 to 700 km. The finding that aggregation of road characteristics introduces very little error is unexpected, but quite useful for aggregate modelling of traffic operations. Further validation of this result under different conditions is warranted.

2.27 The micro-transitional model of Watanatada, et al. (1987a) is not used in HDM-III, but provides a very useful procedure which may be valuable for other macroscopic models. The model uses a process of "backward recursion" to determine the maximum allowable speed on each road segment, and "forward recursion" to predict the actual speed profile. It may be possible to develop a procedure of this type for use with macroscopic road evaluation models such as the U.S. Highway Capacity Manual (TRB 1985).

C. Other Geometry-Dependent Relationships

2.28 As illustrated in Figure 1, changes in road geometry characteristics can affect total transportation cost in a number of ways. This section reviews the ability of HDM-III to predict road geometry effects on:

- Fuel consumption;
- Costs of vehicle ownership and maintenance;
- Traffic accidents;
- Road construction costs; and
- Road maintenance costs

**Fuel Consumption**

2.29 The HDM-III predicts overall fuel consumption on a road as a function of average speed, road rise and fall and roughness, and vehicle mass. Vehicle power-to-weight ratios and aerodynamic drag are also
considered in some formulations. Road curvature and width are assumed to have no effect on fuel consumption, except through their effect on speed.

2.30 As with speed prediction, HDM-III offers a choice of simple regression models developed for Kenya, India and the Caribbean, or mechanistic models developed for Brazil. Both approaches use regression analysis to fit a non-linear fuel consumption equation to observed data, and both yield U-shaped relationships giving minimum fuel consumption for an optimum speed range. The difference is that the simple models relate fuel consumption directly to speed and geometry parameters, while the Brazil models relate fuel use to operational parameters such as engine speed and power. These in turn can be related to vehicle and road characteristics using simple physical equations. The two approaches are illustrated in Figure 2.

2.31 The inclusion of known physical (or mechanistic) relationships in the Brazil model should improve its explanatory power, and the high $R^2$ values reported for unit fuel consumption appear to confirm this. However, the non-linear regression stage of both modelling approaches in Figure 2 introduces a substantial non-mechanistic element which reduces the advantage of the Brazil method.

FIGURE 2 Two approaches to fuel consumption modelling.
2.32 The application of the Brazil unit fuel consumption model to overall fuel prediction (Watanatada, et al. 1987a) introduced two further non-mechanistic assumptions:

- For calibration purposes, engine speed was assumed constant all for vehicle speeds. This appeared to be satisfactory for heavy vehicles, which have many gears to choose from, but gave poor $R^2$ values for cars and light trucks. In a vehicle with few gears, the fuel consumed at a given power demand must be expected to vary as engine speed varies above and below its optimum level.

- Comparisons between controlled tests and fleet operation that the latter used about 15 percent more fuel. This was presumed to be due to a combination of older vehicles, poorer state of engine tune, cold starts and other "real-world" factors, and multiplicative correction factors of 1.15 and 1.16 were derived for practical use, for trucks and cars respectively.

2.33 For the purpose of road geometry evaluations, both types of model should be capable of reflecting changes in fuel consumption due to speed and road rise and fall. Because of its mechanistic basis, the Brazil model should be expected to provide more accurate measures of these effects. At the aggregate level, neither type of model takes account of the extra fuel consumption arising from speed variations along a route. Watanatada, et al. (1987a) argue that these should only increase fuel use when power fluctuates from positive to negative, that is when vehicles accelerate and slow down. Applying their model at both an aggregate and microscopic level of road analysis, they demonstrate that results are quite similar for both cases over the range of conditions tested. Nevertheless some increase in fuel consumption must be expected where acceleration/deceleration cycles do occur, such as on inconsistent road alignments or in vehicle interactions involving large speed differences.

Costs of Tires, Parts, Vehicle Maintenance, and Oil

2.34 The simple regression equations used in HDM-III for Kenya and the Caribbean express vehicle operating costs in terms of the following parameters:

- Tire consumption: road roughness and (for trucks) vehicle weight.

1/ Engine speed is difficult to predict for real traffic situations because drivers have a choice of gears at a given speed. Watanatada, et al. (1987a) investigated an alternative calibration approach in which drivers were assumed to choose gears so as to minimize fuel consumption. Since both approaches gave similar results, the simpler model was adopted.
• Parts consumption: road roughness and vehicle age (measured by the cumulative km travelled at the half-life of an average vehicle).

• Labor cost: parts consumption and road roughness.

2.35 The Indian study provided similar relationships for labor cost, but related tire and parts consumption directly to a wide range of characteristics including road width, roughness, curvature and rise and fall, and vehicle cumulative kilometers travelled. Many of these relationships applied cutoff points for various parameters and emphasized the limited range over which regression equations could be considered valid.

2.36 The Brazil models express parts consumption and labor costs in terms of the same parameters used for Kenya and the Caribbean. However, no direct effect of roughness on labor cost was found in most cases, and both equations assume multiplicative effects with power and exponential terms, as opposed to the fairly simple linear and polynomial forms used in most other cases. An advantage of the Brazil formulations is that coefficients generally have practical meanings, and could be estimated for given conditions. This appears to be a difficult process, however, and it seems likely that many users would accept the default values derived for Brazil.

2.37 The Brazil tire consumption model for cars is a simple linear equation with roughness, for a narrow range of roughness values. For trucks, however, a complex model has been developed in terms of:

• Tread wear, expressed as a function of tread volume and forces acting on the tire; and

• Carcass wear, expressed as the number of retreads, which reduces exponentially with increasing road roughness and curvature.

2.38 Assuming no retreads, this reduces to a fairly simple linear expression in tire force ratios which reflect vehicle mass and grade. An additional term for lateral force on curves was considered important, especially on roads with high curvature (Watanatada et al. 1987a), but could not be included because of a lack of super elevation information in the Brazil study. The model is based on a number of important assumptions, in particular, that:

• Tread wear is not affected by road roughness; and

• Retreading practice is rational, and based on accurate assessments of tire carcass quality.

2.39 Watanatada et al. (1987a) report that the ratio of retread to new tire costs is typically only 15 percent in Brazil, compared with 40 to 60 percent in Costa Rica and India. They also report Brazilian data showing retread lives are of the order of 75 percent of new tire lives, regardless
of number of retreads. In contrast, retread lives in India were found to decrease with the number of retreads (Bennett 1985). With low retread cost ratios, the assumption of rational retreading practice seems reasonable, and is well supported by the data reported from Brazil. However, as retread cost ratio begins to approach the life ratio of 75 percent, this assumption becomes highly questionable. The validity of the roughness and curvature effects on retreading practice is also uncertain under these conditions.

2.40 Despite these reservations, the Brazil tire consumption model provides a considerable conceptual advance over earlier models which took no account of the mechanisms of tire wear. The inclusion of a geometric tread wear term based on tire forces is a considerable advantage for the purposes of this study, although the lack of information on lateral force effects is unfortunate. Care should be exercised in applying the model in countries or regions with different retread cost ratios. Comparisons presented by Watanatada et al. (1987a) showed some substantial differences between the mechanistic Brazil tire consumption models and those developed for other countries. Variations with rise and fall showed particular differences from other models. The comparisons do not provide sufficient information to judge which approach is more accurate.

2.41 The cost of oil or other lubricants is considered in the HDM model to be a linear function of road roughness. The roughness coefficient remains constant for all vehicle types, even though the basic lubricant consumption per 1,000 vehicle kilometers varies from 1.6 litres for cars to 5.2 litres for articulated trucks.

Time, Depreciation, Interest, and Overhead Costs

2.42 The remaining costs of vehicle operation considered by HDM-III fall into three groups:

(a) Cost of crew, passenger and cargo time are assumed to be directly related to travel time, and hence inversely related to journey speed. Values of crew time, passenger time, number of passengers and cargo holding time (per vehicle hour) must be specified by the user.

(b) Depreciation and interest charges applicable to a given journey depend upon the life of the vehicle and its annual utilization. The HDM model provides several options for predicting these:

- Vehicle life may be considered constant, or may reduce with increasing speed, such that total lifetime kilometers increases less than proportionally with speed.

- Annual kilometers driven may be considered constant, directly proportional to speed, or somewhere between these two extremes as determined by an "hourly use ratio."
Overhead costs of vehicle operation may be considered either as a proportion of all other vehicle operating costs excluding passenger and cargo time, or as a fixed annual cost which is proportioned across the annual kilometers driven.

Road Construction Costs

The HDM model allows for construction cost information to be provided by the user at various levels of detail, from a single cost for a whole project down to specific quantities and unit costs for each element of the construction process, as shown in Figure 3. HDM-III also incorporates prediction models for a number of the construction quantities, based on road rise and fall (RF), ground rise and fall (GRF) and road width (RW). These are derived from a study of 52 projects in 28 developing countries (Markow and Aw 1983), and include the following:

- **Site preparation quantities** are based on a fitted equation with exponential terms in GRF and GRF x RW. The model is very sensitive to road width, but costs increase less than proportionally with width.

- **Earthwork quantities** are derived from the area of the construction trapezoid given by road width, "effective height" and embankment slope. An equation for effective height was estimated for the available projects, as a linear function of GRF and (GRF - RF). This provided an intercept of 1.41 m embankment height in level terrain which, was considered reasonable for the project data available (Watanatada, et al. 1987b). However, HDM permits a user-specified embankment height for roads in level terrain as an alternative to this calculation.

- **Pipe culvert lengths** are expressed as a complex power and exponential function of GRF and RW, which increases with width but decreases with increasing GRF. A further multiplier reduces relative lengths for the middle range of GRF values. This reduction in pipe length with increasingly severe terrain possibly reflects a "tendency to minimize construction cost in steep terrain" (Watanatada et al. 1987b), or the difficulties which sometimes arise in providing drainage in flat terrain.

- **The number of box culverts and small bridges per kilometer** are simply estimated as constants for each of three terrain types (defined by ranges of GRF) as shown in Table 2. It is difficult to observe any meaningful pattern from these values. In the absence of any clear trends, it may be that these results are simply a reflection of the particular projects chosen for evaluation.
FIGURE 3 Construction costs and availability of models for predicting construction quantities
TABLE 2 Predicted number of box culverts or small bridges per kilometer

<table>
<thead>
<tr>
<th>Range of ground rise plus fall (GRF m/km)</th>
<th>0-10</th>
<th>10-40</th>
<th>40-100</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small box culverts per km</td>
<td>0.27</td>
<td>0.72</td>
<td>0.62</td>
</tr>
<tr>
<td>Small bridges per km</td>
<td>0.217</td>
<td>0.104</td>
<td>0.091</td>
</tr>
<tr>
<td>Number of observations</td>
<td>43</td>
<td>26</td>
<td>16</td>
</tr>
</tbody>
</table>

2.44 For road widening projects, the HDM procedures calculate the difference in site clearance, earthworks and pipe culvert quantities between wide and narrow roads. For box culverts, the same equations are used, and the user must take care to specify reduced unit costs for these items in a road widening project.

2.45 For an alignment improvement project, the HDM relationships appear to be based on completely new site clearance, earthworks and pavement quantities which take no account of the existing road. These assumptions would be quite inaccurate for most real projects, which utilize existing right-of-way, alignment or even road surface for substantial sections of a realigned road.

2.46 Since the aim of this study is to evaluate changes in geometric standards, the estimation of construction costs is of great importance. Of particular interest is the marginal cost of a small change in width, alignment or other road characteristics. Depending upon the terrain, cost parameters and the type of project being considered, marginal costs may vary from a fraction of average costs to many times the average. For example:

- It may be very costly to widen an existing bridge or a cement-stabilized road base;
- It may be marginally inexpensive to incorporate the same types of widening in a new construction project.

2.47 The relationships presented by Watanatada, et al. (1987b) and the underlying study of Markow and Aw (1983) provide essential information on the elements of construction cost and their relationships with geometric factors. Considerable care is required, however, in their application. In this study construction costs are calculated outside the HDM-III model using a separate program HDCOST. This program uses some of the HDM-III calculations described here, but permits greater flexibility in cost estimation. It is described in detail in para. 7.09-7.12.

**Road Deterioration and Maintenance Costs**

2.48 A major component of the HDM model deals with the prediction of road deterioration and the effects of alternative maintenance strategies.
This aspect of HDM-III is of little interest to the current study, except in those areas where it interacts with road geometry evaluations. Three types of interactions may be considered:

- Deterioration and maintenance are affected directly by road width and traffic loading.
- Vehicle operating costs and their relationships with road geometry are affected by road roughness, which in turn depends upon deterioration and maintenance factors.
- The magnitude of maintenance and reconstruction costs relative to vehicle operation costs may affect the evaluation of road geometry standards.

2.49 HDM-III assumes that deterioration of paved roads is related to traffic loading, but not to traffic speed or operational characteristics. Traffic loadings are measured by the number of vehicle axles per lane (YAX) and equivalent 80 kN standard axles per lane (YE4). The "effective number of lanes" (ELANES) may be specified by the user, or take the default values given in Table 3 for various ranges of road width. Road width is also used in various measures of deterioration and maintenance quantities, such as pothole development and patching. These two parameters appear to be the only geometric factors influencing paved road deterioration in HDM-III.

**TABLE 3 Effective number of lanes (ELANES) for various road widths: default values**

<table>
<thead>
<tr>
<th>Road width (m)</th>
<th>&lt; 4.5</th>
<th>4.5-6.0</th>
<th>6.0-8.0</th>
<th>8.0-11.0</th>
<th>11.0-14.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>ELANES</td>
<td>1.0</td>
<td>1.5</td>
<td>2.0</td>
<td>3.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

2.50 Longitudinal geometry (road curvature and rise plus fall) and shoulder width are considered in the prediction of unpaved road deterioration in HDM-III. These affect roughness progression, material loss and erosion predictions, and the quantities required for regravelling works. Unpaved roads, however, are not be considered in this study.

2.51 If the model is to be used to evaluate alternative width standards, a much closer examination of width effects is required. In moving from a wide (7.0-7.5m) two-lane paved road to a narrower formation, three types of effects may be observed:

- The effective number of lanes for traffic may be reduced. On narrower roads, paths for the two directions will overlap, and some vehicles will drive in the center of the road, especially
where sight distance is good or no oncoming traffic is in view.

- **Wind turbulence effects** from large vehicles near the edge of the pavement may cause loss of shoulder material and accelerated edge cracking and potholing.

- **Edge crossings** will increase in frequency as road width reduces and traffic volume increases, especially when the proportion of heavy vehicles is high. Repeated edge crossings are likely to result in significant pavement damage. Without prompt maintenance, edge crossing damage could lead to abrupt edge drop-offs and irregular pavement edgelines, resulting in accelerated deterioration and reduced pavement life. Edge crossings also have a direct effect on traffic speeds and vehicle operating costs, as discussed in Chapter IV.
III. SPEED REDUCTIONS DUE TO TRAFFIC VOLUME

3.01 The HDM model in its present form is based on free speeds, and takes no account of the effects of interactions between vehicles. This section reviews current knowledge of speed-volume relationships, with the objective of incorporating these effects into the HDM model. In the limited time scale of this study, such a relationship could only be derived from the results of existing research, and may not be of the same standard of reliability and validation as other elements of the HDM model. Nevertheless the quality of HDM predictions should be improved by including these effects, rather than ignoring them. Their inclusion also permits some estimation of the error introduced by ignoring vehicle interaction effects, and the range of conditions for which the assumption of free flow is reasonable.

3.02 There are two broad approaches which may be taken to the modelling of speed-volume relationships. The first is to model the mechanisms by which vehicle interactions reduce traffic speeds, such as the supply and demand of overtaking opportunities. The second approach is to express speed as a direct function of traffic volume, with other terms to account for the effects of road geometry, traffic composition and other features. The following sections provide a review of previous research on speed-volume relationships and describe a proposed new subroutine VOLUME which could take account of these effects in HDM-III.

A. Mechanisms of Speed Reduction Due to Traffic Volume

3.03 As traffic volume increases, journey speeds may be affected by three types of interaction between vehicles. First, overtaking delays arise when a vehicle catches up to a slower vehicle and is unable to overtake immediately. Second, crossing delays may be caused by interactions between vehicles travelling in opposite directions. These are negligible on wide two-lane or multi-lane roads, but can be substantial on narrow roads. Finally, intersections, turning vehicles, and roadside activities can also cause delays and disrupt smooth traffic flow. This last group is not considered in detail in this study, but is referred to as roadside "friction."

3.04 Overtaking Delays: Because vehicles travel at different speeds, faster vehicles continually catch up to slower ones. Wardrop (1952) found that the demand for overtaking (in other words the catch-up rate for unconstrained traffic) could be given by:

\[ p = \frac{1}{\sqrt{\pi}} \frac{Q^2 \sigma}{V^2} \]  

(4)

where:  

- \( p \) is the overtaking demand per kilometer per hour, desired speeds are assumed to be normally distributed with mean \( V \) and standard deviation \( \sigma \)
\( \text{km/h} \), and \( Q \text{ (veh/h)} \) is the one-way traffic volume. This implies that overtaking demand is proportional to the spread of the speed distribution (across all vehicle types) and the square of the traffic volume, for a given mean desired speed.

3.05 The overtaking demand given by Equation 4 is, however, rarely satisfied except at very low traffic volumes. Normann (1942) found that observed overtaking rates were approximately half of the demand at traffic volumes up to about 1000 veh/h, and declined to zero as traffic volume increased towards capacity. Hoban (1980) found similar results using traffic simulation, and showed that actual overtaking rates varied greatly with terrain and sight distance, and the provision of overtaking lanes.

3.06 The supply of overtaking opportunities on two-lane roads depends upon the availability of sufficient gaps in the oncoming traffic stream, and of sufficient sight distance to see that such a gap exists. The size of the required time gap increases with the length and speed of the overtaken vehicle (or group of vehicles) and also increases with decreasing road width (Troutbeck 1981, 1982).

3.07 Numerous attempts have been made to model overtaking supply and demand in mathematical or empirical terms, with limited practical success. McLean (1981b, 1982) and Hoban (1984) provide useful reviews of these studies. Macroscopic models by Werner and Morrall (1984) and McLean (1983a) have shown considerable promise. McLean (1983a) proposed that traffic platooning (the percentage of vehicles following slower leaders) could be modelled as a function of overtaking opportunities and the overtaking demand given by Equation (4). Overtaking opportunity was expressed as the joint probability of a gap in the opposing traffic stream, and sufficient sight distance to make use of that gap. While the concept was demonstrated using simulated traffic data, no attempt was made to estimate model parameters.

3.08 A similar approach was developed by MTC (1975) and Werner and Morrall (1984). They predicted "Net Passing Opportunities" from the proportion of time with gaps greater than a certain size and the proportion of road with adequate sight distance. These were then related to Wardrop's overtaking demand (Equation 4), to determine the "Unsatisfied Passing Demand" (Werner 1986). Neither of these two models is fully developed to predict traffic bunching for a given set of conditions, and both rely on very simplified assumptions about overtaking gap requirements. A particular deficiency is their inability to distinguish between overtaking opportunities on passing lanes or multi-lane roads, (which are virtually unlimited), and those on two-lane roads which only provide for some proportion of the required gaps. This highlights the need for a simple macroscopic measure of overtaking opportunity which is easy to measure and is able to reflect these differences. It seems likely that these problems will be overcome with further research on this topic.

3.09 Overtaking supply and demand are not highly sensitive to absolute speeds, although the spread of speeds is important. Because of this, and
the mechanistic basis of these models, they may offer a useful approach to macroscopic speed-volume modelling in the future. The models could be developed to provide estimates of the percentage of journey time spent following slower vehicles, for each vehicle type. Simple assumptions may then be used to estimate following delays from the known free speed distribution for each lead vehicle type. Aggregate measures of time spent following (or 'percent time delay') are now being used as the major criterion for level of service on two-lane roads in the U.S. Highway Capacity Manual (TRB 1985). Models of this type may in the future provide valuable techniques for incorporating speed-volume effects into a macroscopic model framework. A procedure for taking account of 'Net Passing Opportunities' in a speed-flow relationship is discussed in para. 3.73 to 3.75.

B. Direct Speed-Volume Relationships

3.10 A number of studies have attempted to relate speeds directly to traffic volume and a range of road and traffic characteristics. Two well known works in this area are the 1965 U.S. Highway Capacity Manual (HRB 1965) and a major British study by Duncan (1974). These have greatly affected subsequent research on speed-volume relationships, yet both have substantial flaws which limit their accuracy and reliability, especially outside their countries of origin. This section reviews these studies in some detail, and briefly discusses a number of other approaches to speed-volume modelling. A simple speed-volume model for use with HDM-III is then presented.

The 1965 Highway Capacity Manual

3.11 The 1965 U.S. Highway Capacity Manual, or HCM, (HRB 1965) presented a set of speed-volume relationships for two-lane roads with various sight distance conditions. Traffic volume was expressed as a proportion of road capacity, and capacity was given by:

\[ C = 2000 \text{WT} \]  

(5)

where:

- \( C \) is the total capacity in both directions (veh/h), with an ideal value of 2000 passenger cars per hour on two-lane roads;

- \( W \) is an adjustment factor for lane and shoulder width; and

- \( T \) is an adjustment factor for trucks and buses.

The truck adjustment factor is based on a system of passenger car equivalents, or PCE's, where each heavy vehicle is treated as being equivalent to a certain number of cars. Different PCE values were derived for various terrains, grades, and volume/capacity ratios. The reduction factors \( W \) and \( T \) in equation (5) are quite severe. For example a 20 ft. paved road with 2 ft. lateral clearance on both sides, carrying 20 percent trucks in rolling terrain has a predicted capacity of 694 veh/h, or about one third of the ideal value. Capacities below 20 percent of ideal could occur in more constrained conditions.
3.12 The HCM speed-volume relationships have been evaluated by a number of authors (e.g., Rorbech 1972, OECD 1972, Morrall and Werner 1981, Yagar et al. 1982), who have reported higher capacities and higher speeds at given volumes than those predicted by the HCM. McLean (1980) provides a detailed review of the origins and derivations of the HCM relationships. The speed-volume curves appear to be based on a mathematical model of catch-up and following behavior (not empirical observations as the manual seems to imply), but the procedure and its assumptions have not been published. McLean expresses a number of concerns about the apparent assumptions and limitations of the underlying model. The wavy form of the HCM speed-volume curves appears to be an artifact of the numerical calculations, and the method seems very sensitive to changes in its underlying assumptions.

3.13 McLean (1980) found the HCM width reduction factors "highly questionable". He could find no evidence to support these relationships, and quoted several studies from the same period which appeared to contradict them. McLean also noted that the HCM passenger car equivalents were derived on the basis of overtaking rates, and argued that these were not consistent with an evaluation procedure based on operating speeds. He further stated that:

"Both the method, and the concept of passenger car equivalence, tend to overstate the impedance effects of slow vehicles in the stream."

The 1985 Highway Capacity Manual

3.14 The new Highway Capacity Manual (TRB 1985) retains many of the features of the 1965 Manual for two-lane roads, but with a number of significant changes. Roads are still evaluated on the basis of volume-capacity ratios, but these have been derived as limiting volumes from a set of 'percent time delay' criteria using traffic simulation (Messer 1983). Speed criteria are also presented, apparently as a secondary evaluation measure. Only idealized speed-volume relationships are given, but these show much smaller reductions with volume than the 1965 manual. The new manual uses truck PCE values which are similar to those in the 1965 HCM for general terrains, but much lower for steep grades. New PCE's for buses and recreational vehicles are also introduced. The 1985 road width factors are in many cases higher than those of the 1965 manual (producing smaller capacity reductions), but the source of these new values is not discussed by TRB (1985) or Messer (1983). The 1985 manual also uses a substantially higher base capacity of 2800 passenger cars per hour, on two-lane roads and incorporates an additional capacity reduction for unequal directional splits. Predicted service flows are generally lower, however, because of the redefinition of level of service criteria.

British Speed-Volume Studies

3.15 Duncan (1974) used multiple regression analysis to investigate speed-volume relationships in Britain. Only the findings for two-lane
roads are considered here. The analysis was conducted in two stages. First, a preliminary analysis was undertaken to investigate the effects of road layout on speeds at low volumes. This considered about four hours of traffic data from each of 17 sites, and produced the following results:

\[
\begin{align*}
V_L &= 86.5 - 16.7 \frac{Q}{1000} - 12.8 \frac{H}{100} - 14.5 \frac{B}{100} \\
V_H &= 69.5 - 4.1 \frac{Q}{1000} - 19.3 \frac{H}{100} - 8.6 \frac{B}{100}
\end{align*}
\]

where:
- \(V_L\) = mean speed of light vehicles (km/h);
- \(V_H\) = mean speed of heavy vehicles (km/h);
- \(Q\) = two-way traffic volume (veh/h);
- \(H\) = average hilliness (m/km); and
- \(B\) = average bendiness (degree/km).

While detailed regression results were not reported, Duncan noted that "most of the regressions explain about three-quarters of the between-sites variance in speed," and most of the coefficients were significant. Small but insignificant width effects were found, and so this parameter was omitted from the final regression results. Actual widths ranged from 6.1 to 7.3 m. Duncan described this preliminary stage as a "low flow" analysis, although observed volumes ranged from 150 to 990 veh/h, with a mean of 450 veh/h. He considered that the high coefficients for traffic volume could partly be due to "differences in layout or traffic behavior between the busy roads and the quieter ones," and not entirely a true speed/flow effect.

Duncan then made two major assumptions in applying these results to overall speed prediction. First, he noted "at least some indications that wider roads were faster," and decided to express traffic volumes as flows per standard (3.65 m, or 12 ft) lane. This implies an inverse linear relationship between flow and width, so that a ten percent width reduction is equivalent to a ten percent increase in flow. Secondly, Duncan recognized the problems involved in defining free speed in field traffic studies. Since it is not practicable to measure speeds at zero flow, most studies use the speeds of isolated or unimpeded vehicles as surrogates for free speeds. These are not easy to define, and may not reflect the true distribution of desired speeds, since some vehicles are more likely to be followers or platoon leaders. To overcome this, Duncan defined "free" speed as that predicted by equations (6) and (7) at a flow of 550 veh/h. This is based on a nominal free flow of 300 veh/std. lane, and an average carriageway width of 6.7 m, or 1.83 standard lanes.

Conceptually, this approach gives a more clear definition of "free" speed. In practice, however, the nominal free flow seems arbitrary, and results from this analysis are not compatible with other free speed studies. This formulation also gives the appearance of suggesting a
speedflow "plateau," that is, no variation in speed with flow, at two-way volumes below 500 to 600 veh/h.

3.19 In the second stage of the analysis, Duncan (1974) developed equations for the slope of the speed-flow relationship for various combinations of road type and geometry, and traffic composition. He found substantial scatter in the data, and considerable effort was required to develop a form for these equations which was reasonable and consistent. In all, some 200 alternative models were tested, and the final model explained only 27 percent of the variance ($R^2=0.27$). Duncan (1974) also concluded that the effects of heavy vehicles relative to light vehicles in the traffic stream could not be expressed in terms of passenger car equivalency (PCE) factors. His reasons were not based on the technical issues raised in para. 3.32 to 3.34 but on the instability of regression coefficients for this stage of the analysis. His alternative formulation was to incorporate traffic composition effects into the expression for speed-volume slope. The analysis showed that heavy vehicles travelling in the opposing direction had a smaller effect on speeds than heavy vehicles travelling in the same direction as the vehicles under observation. However, this effect was not incorporated into the final models, since it could not be detected in the raw data.

3.20 While Duncan (1974) provides some valuable information on speed-volume relationships, his final models can not be applied with confidence outside the range of road types and traffic conditions for which they were established. In particular, the models are not appropriate for investigating the effects of small changes in road width and alignment on roads in other countries.

3.21 Brewer, et al. (1980) studied 40 two-lane and three-lane sites in Britain to extend and review the results of Duncan (1974). The new study covered a considerable range of geometric standards, and investigated the effects of verge width, frontage development, sight distance, weather and time of day. They reported a number of important findings with regard to the earlier work of Duncan (1974). First, Duncan's relationships "did not provide a wholly satisfactory explanation of vehicle speeds at any of the [first 20] sites," and showed poor agreement with the data across all sites. It was further noted that:

- Upgrade hilliness (or rise in m/km) provided most of the explanatory power of grade effects. Downgrade hilliness (i.e., fall in m/km) had much smaller effects, and the combined hilliness term was similar to that for upgrade hilliness.

- There was very little difference in the performance of three different volume measures: total two-way volume, volume per standard lane, and volume per nominal lane.

- There was some evidence of a flatter speed-volume slope at volumes below 300 veh/h/std. lane, but the difference was
not significant and a single coefficient was adopted for all volumes.

- Sight distances (measured by the harmonic mean visibility in m) was the single most important explanatory variable in the speed prediction equations. Sight distance was found to have a complex non-linear relationship to other geometric parameters, and Brewer et al. argued that its inclusion enhanced the effects of these parameters. However, its large explanatory power does not seem intuitively reasonable, and this term almost certainly incorporates the effects of other geometric characteristics in the regression.

3.22 The equations also showed a substantial effect of road frontage development, a small width effect and very small wet-weather effect, and a small non-linear contribution due to verge width.

3.23 The final results of Brewer et al. (1980) have two limitations in common with many studies of this type. First, the regression coefficients were not stable, and varied substantially from one stage of analysis to the next. Second, the equations cannot be extrapolated outside the range of the data, and in fact give unrealistic predictions for reasonable conditions. Truck speeds, for example, can be predicted as higher than car speeds in some circumstances. Other disadvantages are the uncertain treatment of width in the expression for volume, the unreasonably large visibility effect and the rather complicated expression for verge width which does not appear to have a very sound basis.

Other Speed-Volume Studies

3.24 Numerous other speed-volume relationships are reported in the technical literature, and it is beyond the scope of this report to review these in detail. Underwood (1964), Casey and Tindall (1966), Rorbech (1972), OECD (1972, 1983), McLean (1980), Krumins (1981), Morrall and Werner (1981) and Yagar, et al. (1982) provide examples for Australia, Europe and Canada. Overall, these show higher capacities and flatter speed-volume slopes than the 1965 HCM relationships, and demonstrate that a given vehicle's speed is affected more by traffic volume in the direction of travel than by opposing traffic volume.

3.25 Multiple and simple linear regression analyses were used in the Indian Road User Cost Study (CRRI 1982) to investigate speed-volume relationships on rural roads in a developing country with very heterogeneous traffic. Many equations were developed, but these generally showed a large scatter of data points about the fitted regression lines, and $R^2$ values generally less than 0.5. The speed-volume slope coefficients showed considerable variation with vehicle type, road width and terrain, but no clear patterns emerged. Slopes of 10 to 84 km/h per 1000 veh/h were reported for car speeds in different road conditions.

3.26 A number of studies have used microscopic traffic simulation models to predict speed-volume relationships for specific road and traffic
conditions. Numerous examples are reported by Gynnerstedt, et al. (1977), St. John and Kobett (1978), Hoban (1982a), Marwah (1983), Palaniswamy (1983), Botha and May (1981) and Morales and Panati (1984). Simulation models provide an ideal method for developing detailed speed-volume relationships for use in macroscopic traffic models. Because they deal with individual vehicle interactions they are less constrained by mean values and simplifying assumptions required in other models. However, they usually have more parameters and are therefore more difficult to validate than macroscopic models.

**Capacity**

3.27 The capacity of a two-lane road represents the upper limit of the speed-volume relationship. If the relationship is linear, then capacity, minimum speed and free speed are directly related to the speed-volume slope, and any one parameter can be determined from the other three. However, capacity is extremely difficult to measure on rural roads. Most roads operate well below capacity, and when congestion does occur it is often related to towns, intersections or bottlenecks which are not typical of the overall alignment. It is sometimes difficult to determine whether peak recorded volumes on high-standard roads represent capacity, or are simply limited by the demand. On the other hand, peak volumes on non-ideal roads may not appear particularly high, and are difficult to identify as capacity.

3.28 Where capacity is constrained by upstream or downstream bottlenecks, it will not be representative of the true steady-state condition for this road section, and is hence inappropriate for predicting traffic operations at lower volumes. On the other hand, many of the highest volumes are recorded on sections which are themselves bottlenecks, with wide roads upstream and downstream. Capacities recorded in these cases (e.g., bridges, tunnels, short two-lane sections) overestimate equilibrium conditions and hence do not have wide applicability.

3.29 McLean (1982) and OECD (1972, 1983) report highest recorded volumes of 2000 to 3000 veh/h on two-lane roads, in many cases including substantial proportions of trucks. Yagar (1982) postulated capacities of up to 3600 veh/h under ideal conditions. However, much of the data from these studies represent bottleneck conditions as described above, and hence overestimate the steady-state capacity. TRB (1985) has adopted a capacity of 2800 cars/h on ideal two-lane roads, with reduction factors for road width and directional split. However, the width factors are not supported by any published research on speed-volume effects. OECD (1972) also describes Swedish capacity estimates of 2250 PCE/h for ideal conditions, with reductions for reduced lane and shoulder width. The reductions are much less severe than the Highway Capacity Manual values, which give a minimum capacity of 1600 cars/h for a two-lane road with 8 ft. lanes and no shoulders.

3.30 CRRI (1982) estimated capacities for Indian roads carrying very mixed traffic, using subjective assessments of maximum volume and expected
speed at capacity. Estimated capacities were 75-230 veh/h on one-lane roads, 450 to 600 veh/h on intermediate (5.5 m) roads and 500-1500 veh/h on two-lane roads. Volumes are for mixed traffic, where the lowest values apply for curvy roads in hilly terrain and the highest for fairly straight level roads. CRRI (1982) implied that PCE volumes (i.e., equivalent ideal volumes in cars/h) would be about double these values. It should be noted that the Indian traffic contained only 5 to 30 percent cars.

3.31 The capacity of a two-lane road also appears to be strongly influenced by the availability of overtaking opportunities. While little research has been undertaken on this subject, its effect can be demonstrated in four ways:

- The capacity of a two-lane road is well below that of two lanes on a multi-lane road. This reduction occurs because "slower moving vehicles create gaps that can be filled only by passing maneuvers" (HRB 1965). As flow approaches capacity on a two-lane road, overtaking becomes more difficult, and these gaps become larger.

- The bottleneck locations described earlier are generally characterized by short lengths and good upstream and downstream opportunities for overtaking. The gaps in front of slow vehicles in this case do not have time to develop, and higher capacities are recorded.

- Two-lane roads with overtaking lanes have speed-volume relationships which are well above those for normal two-lane roads (Hoban 1982a). While research studies have not investigated these roads at capacity, it seems likely that the overtaking lanes should lead to increased capacity, since they provide a method for filling in the gaps of unused road space ahead of slow vehicles.

- The "Net Passing Opportunities" (para. 3.08) provided by a two-lane road often reduce to zero at capacity, since gaps in the oncoming traffic stream are small. Because of this, the effect of passing opportunities on capacity has been overlooked in many studies. However, "Net Passing Opportunities" have a strong effect on speed-flow relationships below capacity, and may be greater than zero at capacity where there are overtaking lanes or high directional splits, or downstream of a long no-passing zone.

Traffic Composition Effects

3.32 In relating speeds to traffic volume, some account must be taken of the composition of that traffic. This is generally achieved through the use of passenger car equivalents (PCEs) by which mixed traffic volumes are expressed as an equivalent number of passenger cars. The major difficulty
in this approach is the selection of an equivalency criterion. The relative impacts of a truck on speed, capacity, overtaking, platoon formation and other traffic characteristics may lead to quite different estimates of its PCE value.

3.33 Extensive discussion and analysis of this subject are given by many authors, including Werner and Morrall (1976), St. John (1976), Linzer et al. (1979), Craus, et al. (1980), Cunagin and Messer (1983), OECD (1983), Van Aerde and Yagar (1984) and Roess and Messer (1984). For two-lane roads, these studies collectively show that truck PCEs based on overtaking delays can range from 2 to 12 on general terrain sections, and possibly much higher on individual long steep grades. There is some disagreement as to the appropriate values for more extreme conditions and these are quite sensitive to vehicle power and speed assumptions. Estimated PCE values based on non-speed criteria such as capacity and platoon formation generally take much lower values, of the order 1 to 3.

3.34 Duncan (1974) argued that PCE values could not be derived from his regression studies, and incorporated traffic composition directly into his predictive relationships. As with other speed-based approaches, this produced an "effective PCE" with high values in steep terrain. The Indian regression study of CRRI (1982) considered several methods of deriving PCEs. However, in this case the differences between alternative criteria were considerably smaller, with truck PCEs, for example, taking values of 2 to 5 on two-lane roads.

3.35 Two other aspects of PCE factors may be noted. First, many analysis procedures implicitly assume that traffic operations in a given direction are equally affected by flow in the same direction and that in the opposing direction (i.e., the PCE of a car in the opposing direction is 1). Numerous studies (Normann 1939, Casey and Tindall 1966, Duncan 1974, Van Aerde and Yagar 1983) have demonstrated that this is not the case. The 1985 Highway Capacity Manual (TRB 1985) has accounted for this effect through a multiplicative factor for directional split, rather than varying PCE values by direction. Secondly, there is evidence that PCE values vary with traffic volume and the proportion of trucks in the traffic stream. St. John (1976), for example, found that the incremental effect of the first ten percent trucks in a traffic stream is greater than that of an additional ten percent.

The NIMPAC Speed-Volume Model

3.36 The NIMPAC macroscopic road planning model developed in Australia (NAASRA 1984) includes a simple procedure for estimating speed-volume effects, which has some similarities with the methods of the Highway Capacity Manual (HRB 1965) and Duncan (1974). The model is illustrated in Figure 4, and requires the following input information for a given set of road and traffic conditions:

- Free speed for each vehicle type: NIMPAC uses a family of look-up tables based on road type, width, grade and curvature, (Both and Bayley 1976).
Volume-capacity ratio (v/c): The method used by NIMPAC is to specify a set of 21 road types ranging from a narrow dirt track to a multi-lane freeway, with ideal capacities of 800 to 12000 passenger cars per hour as shown in Table 4. Vehicle equivalency factors from HRB (1965) are used to convert actual traffic conditions to passenger car equivalents, depending upon road gradient.

Speeds at capacity are also specified for each road type in Table 4. The table specifies "car operating speeds," but the formulation of the model in Figure 4 seems to require average speeds. Of course the average speeds for each vehicle type should be much the same at capacity, so this distinction may be of little importance. The model assumes that speed at capacity is not affected by traffic composition or road alignment.

A minimum stop-start speed for over-saturated conditions. The "capacity" condition in the NIMPAC model is assumed to be an upper bound of normal traffic interactions, beyond which behavior is more severely affected by minor disturbances. The line BC in Figure 4 attempts to take account of this behavior.

A volume-capacity ratio below which vehicle speeds may be assumed to be unaffected by traffic interactions. In fact there is little if any evidence that such a region exists, but errors are likely to be small if the upper volume limit is not too high, and the assumed v/c of 0.1 for two lane roads in NIMPAC is probably satisfactory. The concept of a free-speed zone is a useful modelling convenience, since speed-volume effects may be bypassed in the evaluation of low-volume roads.
X = 0.1 for one and two-lane roads
- 0.2 for three and four-lane undivided roads
- 0.3 for divided roads
- 0.4 for freeways

Queuing speed 8 km/h

RATIO OF HOURLY VOLUME TO ABSOLUTE HOURLY CAPACITY

FIGURE 4 NIMPAC Speed-Flow Model
Source: Both and Bayley (1976)

<table>
<thead>
<tr>
<th>Standard Road State</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>16</th>
<th>17</th>
<th>18</th>
<th>19</th>
<th>20</th>
<th>21</th>
<th>22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seal or gravel width (m)</td>
<td>—</td>
<td>3.7</td>
<td>5.5</td>
<td>3.7</td>
<td>4.9</td>
<td>5.5</td>
<td>6.1</td>
<td>6.1</td>
<td>6.7</td>
<td>7.3</td>
<td>7.3</td>
<td>11</td>
<td>11</td>
<td>12</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>15</td>
<td>22</td>
<td>22</td>
</tr>
<tr>
<td>Formation width (m)</td>
<td>—</td>
<td>7.9</td>
<td>7.9</td>
<td>7.9</td>
<td>7.9</td>
<td>7.9</td>
<td>8.5</td>
<td>8.5</td>
<td>10.4</td>
<td>12.2</td>
<td>13.4</td>
<td>17</td>
<td>17</td>
<td>18</td>
<td>21</td>
<td>27</td>
<td>27</td>
<td>27</td>
<td>34</td>
<td>34</td>
<td>34</td>
<td>34</td>
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<tr>
<td>Road type†</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
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<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Un</td>
<td>Div</td>
<td>Div</td>
<td>Div</td>
</tr>
<tr>
<td>Absolute hourly capacity (p.c.u./hr)</td>
<td>800</td>
<td>800</td>
<td>1000</td>
<td>1400</td>
<td>1200</td>
<td>1400</td>
<td>1520</td>
<td>1620</td>
<td>1620</td>
<td>1760</td>
<td>2000</td>
<td>2000</td>
<td>4000</td>
<td>4000</td>
<td>7120</td>
<td>8000</td>
<td>8000</td>
<td>8000</td>
<td>8000</td>
<td>12000</td>
<td>12000</td>
<td>12000</td>
</tr>
<tr>
<td>Car operating speed at capacity (km/h)</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
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<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Basic accident rate</td>
<td>1.37</td>
<td>1.37</td>
<td>1.5</td>
<td>1.5</td>
<td>1.87</td>
<td>1.75</td>
<td>1.62</td>
<td>1.37</td>
<td>1.37</td>
<td>1.25</td>
<td>1.12</td>
<td>1.12</td>
<td>1.12</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
<td>0.94</td>
</tr>
</tbody>
</table>

* NS = natural surface, E = earth fill, G = gravel pavement, B = bitumen, BC = bituminous concrete or cement concrete
† Un = undivided, Div = divided carriageways, F = freeway
§ All numbers per million vehicle-kilometers. This rate is multiplied by (1.0 – 50 × 10^-4 AADT) to allow for increase in the accident rate as traffic volume increases.

Source: Both and Bayley (1976)
3.37 The NIMPAC procedure provides a very simple mechanism for modeling of free flow, traffic interaction and congested flow, assuming linear volume effects in each region. Given this framework, the model has two conceptual flaws. First, the speed at capacity is assumed to be independent of road alignment and is expressed as a "car operating speed." It would be preferable to express the speed at capacity as a function of the slow vehicle speed on that alignment. On an infinitely long road section with no overtaking average speed asymptotes towards the speed of the slowest vehicle. A reasonable approximation for typical road section lengths might be the 15th percentile slow vehicle (e.g., truck) free speed.

3.38 The second flaw is that speeds of slower vehicles (e.g., trucks) are assumed to be unaffected by volume until they intersect with the declining car speeds, and average speeds for all vehicle types are assumed equal at all subsequent volumes (Figure 4). This is unrealistic. Traffic interactions cause delays to trucks as well as to cars, and even when interaction is considerable, cars spend some time travelling unconstrained, and thus have a higher mean speed than trucks. A better model formulation would be to assume separate linear speed reductions for each vehicle type, going from free speed at low (or zero) volume to a common low speed at capacity.

3.39 The over-capacity volume region in Figure 4 is an attempt to model traffic flow disturbances which reduce speeds below even the lowest free speed of vehicles in the stream. Such effects may be caused by minor within-traffic disturbances, which can be magnified by the reactions of following drivers. These are sometimes known as "shock waves." On two-lane roads, however, flow disturbances are usually external to the traffic stream, caused by intersections, driveways and roadside activities. Such disturbances can occur at volumes well below capacity, and the clear distinction between 'interaction' and "congestion" (implied by the lines AB and BC in Figure 4) is difficult to identify in practice.

3.40 The NIMPAC model can be used to take some account of external disturbances, in two ways. First, the capacity parameter could be renamed 'interaction capacity,' taking the same value as the present model on roads without any roadside disturbances, and progressively lower values as roadside friction increases. Effectively, this reduces the range of volumes over which interactions may be assumed to be undisturbed by external effects. Second, the minimum speed at capacity could be reduced to take account of external effects. The estimation of interaction capacity and the modelling of congested behavior clearly require further research.

3.41 The NIMPAC model also includes a simple but effective method for relating hourly volume to the annual average daily traffic (AADT). Both and Bayley (1976) present a flow-frequency histogram for a year in which hourly volumes are expressed as percentages of AADT. Six frequencies are given for 4-percentile ranges of AADT, ranging from 4865 hours with an average hourly volume of 2 percent of AADT, to 15 hours with a volume of 22 percent of AADT. Van Every (1982) used a similar approach, giving flow
frequency histograms for five rural highways in Australia. Van Every used fixed frequencies rather than fixed percentiles of AADT, as shown in Figure 5. With these frequencies, traffic volume effects can be analyzed separately for very low, medium and peak hourly volume conditions, and the results are then accumulated for all traffic over a year.

![Graph showing hourly volume frequencies for five highways in Australia.](image)

**Table: Approximate Percentage AADT in Each Hourly Group for Some Rural Highways in Victoria**

<table>
<thead>
<tr>
<th>Freeway/Highway</th>
<th>A%</th>
<th>B%</th>
<th>C%</th>
<th>D%</th>
<th>E%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calder</td>
<td>13.5</td>
<td>10</td>
<td>8</td>
<td>6</td>
<td>1.5</td>
</tr>
<tr>
<td>Hume</td>
<td>12</td>
<td>9</td>
<td>7</td>
<td>5</td>
<td>2.5</td>
</tr>
<tr>
<td>Princes (East)</td>
<td>14</td>
<td>10.5</td>
<td>8</td>
<td>5</td>
<td>2.0</td>
</tr>
<tr>
<td>Princes (West)</td>
<td>16</td>
<td>10</td>
<td>8</td>
<td>5</td>
<td>2.0</td>
</tr>
<tr>
<td>Western</td>
<td>19</td>
<td>11</td>
<td>8</td>
<td>5.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>

**FIGURE 5** Hourly Volume Frequencies for Five Highways in Australia

Source: Van Every (1982)
The Existing HDM-III Congestion Subroutines

3.42 A subroutine CONGST was written for HDM-III some years ago to provide for estimation of volume effects. It does not appear to have been used in practical applications of the model. The subroutine is called after free speeds are calculated for each vehicle type for a given year and road link alternative. If congestion effects are present, it returns modified values for speed and fuel consumption for each vehicle type.

Briefly, the early CONGST subroutine is based on the following assumptions:

- Average daily traffic may be modelled as a period of congested flow (PKHRS h) and a period where volume effects may be ignored (XNHIRS h).

- Peak speed and fuel consumption are determined from free flow values using the following equations:

\[
\begin{align*}
SFAC & = 1 - 0.0025 \text{ QPE} \\
FFAC & = 1 + 0.00167 \text{ QPE}
\end{align*}
\]

where: SFAC and FFAC are speed and fuel multiplying factors for all vehicle types; and QPE is the two-way hourly traffic volume in PCE/h.

- The overall modified speed and fuel consumption are then taken as the weighted average of peak and non-peak values; this weighting is done by number of hours rather than by number of vehicles.

3.43 The CONGST routine provides a useful basic structure for modelling speed-volume effects. However, the present formulation has three limitations which make it inappropriate for wider application. First, the division of a day into peak and non-peak periods must be fairly subjective, and could overlook important interactions at lower volumes, especially on hilly roads with high proportions of slow vehicles. The flow-frequency diagram described in the previous section provides a more rigorous and reliable estimate of volume effects which does not require subjective judgements. Alternatively, the appropriate hourly volume for the non-peak hours could be calculated (since the sum of hourly volumes must add to the daily volume). Actual numbers could then be used to obtain weighted averages. Secondly, the CONGST routine uses a single speed-volume coefficient and a single PCE value for each vehicle type, so that no account is taken of geometric effects on the speed-volume relationship. The origins of Equations 8 and 9 or the range of their applicability are not documented. Finally, the fuel consumption adjustment is not based on the revised speed estimate, and the estimation of other speed-dependent parameters (e.g., vehicle life) is not recalculated.
3.44 A second version of the CONGST subroutine was written in 1984 for specific application to a case study for India. The new version uses a flow-frequency distribution with three regions (peak, non-peak and the remainder) which can be input by the model user, and are used to give overall weighted results. The core of the new subroutine consists of four polyno-

\[ X = a + bQ + cQ^2 + dQ^3 \]  

(10)

where: \( X \) may be uphill or downhill speed or fuel consumption;

\( Q \) is the hourly traffic volume for each flow-frequency region; and

\( a, b, c, \) and \( d \) are coefficients which may be different for each equation and vary with road segment and vehicle types (car or truck).

3.45 The derivation of appropriate values for these parameters is not documented, but it was felt that these could be derived in the long term from traffic simulation for various terrains and road classes. In the current formulation, the user is required to specify 32 coefficients for each road section. Uphill and downhill speed and fuel consumption are calculated separately and appropriate averages are taken.

3.46 Unlike the earlier version, the 1984 CONGST routine is called before free speed and fuel consumption are calculated, and is called separately for each vehicle type. Predicted free speed and fuel consumption are simply replaced by the CONGST predictions. In the current release of HDM-III, the CONGST values of uphill and downhill speed are also used in the calculation of tire wear in the Brazil user cost model, and the revised speeds are used in the calculation of vehicle life and utilization. These procedures are only incorporated for the Brazil and India road user cost subroutines in HDM-III, but could be added for other regions with very little effort.

3.47 This formulation overcomes some of the limitations of the earlier version, by incorporating an improved flow-frequency distribution and using revised speed estimates in the calculation of vehicle life, utilization and tire wear. However, where the previous version had only two fixed coefficients for hourly traffic volume, the new model requires 32 coefficients for each road section being analyzed. Since the derivation of these coefficients is not documented, it is not known whether there is some mechanistic basis for their estimation, or whether they are related to road geometry characteristics. Clearly, different values might be required for each country or region in which the model is to be applied.

3.48 Apart from the obvious problem of estimating so many model parameters, a major limitation of this procedure is the lack of any ties to the free speed and fuel prediction methods. In fact, it seems likely that this model would produce discontinuities between free-flow and congested conditions, since the polynomial coefficients could not reasonably be reesti-

mated for all road geometry combinations. Considerable research has been
undertaken to relate free speeds to road characteristics, and it would be most desirable to make use of this information in predicting speeds and fuel consumption at higher traffic volumes.

3.49 The problem could be partly overcome by including free-flow predictions as coefficient 'a' of each of the equations (10). This would require some reordering of the calculations in HDM-III so that CONGST was called after the free speed (and perhaps fuel) calculations, but before the calculation of other vehicle operating costs.

3.50 A further consideration is that speed-volume effects for one vehicle type are dependent upon the free speeds of other vehicle types. For example, the speed reduction experienced by a car depends very much on the extent to which heavy vehicles are slowed down by grades. In general, the slope of the speed-volume relationship is directly related to the spread of free speeds across all vehicle types. It is therefore not appropriate to calculate speed-volume effects separately for each vehicle type without regard to the speeds of other vehicle types. An alternative model formulation which relates speed-volume effects to the predicted free speeds is presented in para. 3.54 to 3.61.

Summary of Speed-Volume Relationships

3.51 Figure 6 presents a comparison of a number of speed-volume relationships derived from field studies, mathematical models, and simulation as discussed in the preceding sections. The shapes of these relationships confirm many of the points made in those discussions:

- Speeds decrease with volume over the full range of volumes observed. There is little if any evidence of a free-speed "plateau" at low volumes.
- The slope of the speed-volume relationship appears to vary with many factors, but a linear approximation seems reasonable for most cases.
- Road width affects both free speeds and speeds at higher traffic volumes.
- Sight distance and overtaking opportunities affect the slope of the speed-volume relationship.
FIGURE 6 Speed-Volume Relationships from various Sources.

3.52 The capacity of a two-lane road has been estimated at 2000 to 3000 vehicle per hour under ideal conditions. Some of the higher values, however, represent non-equilibrium bottleneck conditions (e.g., bridges and tunnels) which would overestimate steady-state conditions for real roads.

3.53 Speed-volume relationships are generally expressed in terms of standardized passenger car (or PCE) volumes. This means that the effective speed-volume slope is steeper as the proportion of slow vehicles increases, especially on grades. While PCE values based on speed and delay are quite variable and sensitive to road and vehicle characteristics, those based on capacity and platoon formation generally take lower and more stable values.

C. Specification of a New HDM Speed-Volume Model

3.54 This section describes a proposed new speed-volume relationship for the HDM-III model. This includes some of the features of the NIMPAC model approach and the existing HDM CONGST subroutines. The new approach is formulated as a three-zone linear model similar to the NIMPAC model, which reduces to a simple linear model with appropriate parameter values. For convenience, the simple model is presented first.

The Simple Linear Model

3.55 In the simple speed-volume model, the actual speed for each vehicle type is assumed to decrease linearly from free speed $S$ km/h at zero volume to a minimum speed $SCAP$ km/h at a nominal capacity of $QCAP$ veh/h, as shown in Figure 7. The main characteristics of the model are:

- Free speeds for each vehicle type on a given road section are estimated using current HDM procedures.
- Unlike the NIMPAC relationship (Figure 4), the proposed model assumes that speeds for all vehicle types decrease linearly with volume from free speed to $SCAP$. This formulation is considered to be a much more realistic representation of vehicle interaction effects with increasing volume.
- Mean speed at capacity $SCAP$ is assumed to be the 15th percentile speed of the slowest vehicle type (usually trucks), given approximately by the mean steady state speed for this type minus its standard deviation. This may be reduced by a factor $XFRI$, which takes a value of 1.0 on undisturbed roads and smaller values where roadside friction affects traffic interactions.

The modified speed $SQ$ (km/h) for a given hourly volume $QHR$ veh/h is thus given by:

$$SQ = S - (S - SCAP) \times QHR/QCAP$$ (11)
Since $S$ and SCAP are determined from the HDM free speed calculations, the only additional parameters to be specified are the friction factor $XFRI$ and the nominal capacity QCAP; the latter parameter is considered nominal for two reasons:

- Capacity is difficult to measure for most two lane roads and in this model it is calibrated from the slope of the observed speed - volume relationship.
Capacity may be affected by overtaking demand and supply.

The Three-Zone Linear Model

3.57 A slightly more complex model formulation allows much greater user flexibility, while retaining the simple model as a special case. This is also illustrated in Figure 7, and incorporates three regions representing free flow, traffic interaction and traffic congestion. The parameter QCAP now represents the "interaction capacity," or the volume below which traffic interactions are not subject to severe internal or external disturbances. The model then allows for higher volumes at which speeds decline to crawl or stop-start conditions. At the other end of the volume scale, the model allows for a free flow range where volume effects may be ignored. The plateau AB in Figure 7 is optional. In this study it is removed by setting QO=0, but if QO is retained its value should not exceed (0.1* QCAP) for two-lane roads.

3.58 This model requires the specification of several additional parameters. These are defined with ranges and default values in Table 5. The model is then given by the following equations:

\[ SQ=S \quad QHR < QO \quad (12) \]
\[ SQ=S-(S-SCAP)*(QHR-QO)/(QCAP-QO) \quad QO < QHR < QCAP \quad (13) \]
\[ SQ=SCAP- (SCAP-SJAM)*(QHR-QCAP)/(QJAM-QCAP) \quad QCAP < QHR < QJAM \quad (14) \]

where: \( SQ \) = modified speed (km/h);
\( QO \) = maximum volume for free flow = \( QMAX*XQCUT \) (PCE/h)
\( QJAM \) = maximum congested flow = \( QMAX*XQJAM \) (PCE/h)
\( QHR \) = hourly volume for mixed traffic (PCE/h)
\( SCAP \) = 15th percentile slow vehicle speed (km/h) multiplied by XFRI. XFRI and the other model parameters are specified in Table 5.

3.59 With XQCUT=0 and XQJAM=1.0, this reduces to the simple model described earlier. If both XQCUT and XQJAM=1.0, on the other hand, the model becomes a free-flow model which ignores all volume effects. In the current study, greatest interest is focused on low-volume conditions, and the congested region of this relationship is of little concern. It is nevertheless retained to ensure that some form of congestion effects will be incorporated if volumes are high on a given road type.
TABLE 5 Parameters of the Proposed Speed-Volume Model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
<th>Range</th>
<th>Units</th>
<th>Default</th>
</tr>
</thead>
<tbody>
<tr>
<td>QMAX</td>
<td>Capacity for an ideal road</td>
<td>0-3000</td>
<td>PCE/h</td>
<td>2500</td>
</tr>
<tr>
<td>XQCUT</td>
<td>Maximum free-flow volume as a proportion of QMAX</td>
<td>0-1</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>XQJAM</td>
<td>Maximum congested volume as a proportion of QMAX</td>
<td>1.0-1.5</td>
<td>-</td>
<td>1.25</td>
</tr>
<tr>
<td>SJAM</td>
<td>Minimum crawl speed at congestion</td>
<td>5-40</td>
<td>km/h</td>
<td>8</td>
</tr>
<tr>
<td>XFRI</td>
<td>Reduction of minimum speed due to roadside friction</td>
<td>0-1</td>
<td>-</td>
<td>1</td>
</tr>
</tbody>
</table>

3.60 The three-zone linear model is therefore incorporated in a new subroutine VOLUME for use in HDM-III. In most cases, it is recommended that XQCUT and XQJAM be set to the values (0 and 1 respectively) so that the equations operate as a simple linear model. The effects of roadside friction could then be handled by the parameter XFRI. Details of the new subroutine and associated modifications to the HDM-III program are given in Appendix B.

3.61 Once the model has calculated a modified speed to take account of traffic volume effects, other speed-dependent vehicle operating parameters must also be modified. For simplicity, this is done within the new VOLUME subroutine in the present formulation. However, it may be appropriate to rearrange this component if the speed-volume procedures are to be retained in future versions of HDM-III model.

Determination of Hourly Volume

3.62 The appropriate hourly volume for this calculation is estimated using a flow-frequency histogram similar to that of the NIMPAC model. The histogram for the Hume Highway in Figure 5 is adopted here, since it is typical of a rural highway which is not dominated by recreational peak traffic. Using this histogram, the model calculates modified speeds, fuel and vehicle operating cost quantities for each flow-frequency category, and accumulates these to give overall estimates for an "average" day.

3.63 The averaging of speeds and other parameters over different flow-frequency regimes requires further explanation. The area under each bar of the histogram is a measure of the number of vehicles travelling at that volume in a given time period. Since the model predicts performance of
groups of vehicles travelling the same road at different times, the appropriate overall average is the arithmetic mean of the prediction for each volume, weighted according to the number of vehicles. Since the flow parameter in this histogram converts daily to hourly volume, its units are d/h. The new procedure therefore scales the frequency parameters so that they sum to 24 (h/d), and then checks that the area under the histogram sums to 1.0.

3.64 The most trivial flow-frequency assumption would be an even distribution of traffic over, say, 12 hours, with zero traffic over the remaining 12 hours. The volume coefficient in this case would be 1/12, or 0.083. A slightly more advanced model might assign 3 regions, such as 4 hours of peak traffic at 0.10 of AADT, 12 hours of comparatively low volume and 8 hours of zero volume. In this case the low-volume coefficient must assume a value of 0.05 i.e., (1.0-0.40)/12 to ensure that total volume is preserved. While the user may subjectively consider the lower volume insignificant, the model requires this information to give correct weighting to the performance measures calculated for each volume.

3.65 It would be a simple exercise to extend this formulation to include different traffic compositions for each flow-frequency case, so that incremental effects of particular vehicle types in peak or off-peak conditions could be accounted for. However, data are not generally available at this level of detail, so this feature is not currently included in the model.

Parameter Estimation

(i) Capacity

3.66 The ideal capacity of a two-lane road is of the order of 2200 to 2800 passenger cars per hour. There is some suggestion that the upper values may be associated with non-equilibrium bottleneck situations, and the middle value of 2500 PCE/h is therefore adopted.

(ii) Passenger Car Equivalents

3.67 A major advantage of the proposed new speed-volume model is that all of the speed effects of different traffic compositions are already accounted for. Both the free speeds and the speed at capacity are determined from the HDM relationships which take account of grades, curves and road surface roughness. The slope of the speed-volume relationship is therefore automatically increased or decreased according to the spread of free speeds between fast and slow vehicles.

3.68 As discussed in para. 3.32 - 3.35, the passenger car equivalent, PCE, of a slow or heavy vehicle may be considered to have two components: the extra space taken up by the vehicle, and the extra delay caused by slower speed and greater difficulty in overtaking. The space component should represent a lower bound PCE for capacity conditions on level roads where speeds are uniformly low and overtaking is generally not possible.
Since speed differences are already accounted for, the lower-bound PCE is the appropriate value for use in this model. Table 6 compares a range of minimum PCE estimates from various studies. These have been derived by various methods on two-lane and multi-lane roads, and where appropriate represent high-volume conditions in level terrain.

3.69 The current HDM CONGST subroutine includes a PCE matrix with values of 2.0 for medium and heavy trucks and buses, and 1.5 to 1.7 for other non-car vehicle types for all four regions in which the model may be applied. While the source of these values is not known, they are of the same order as the values in Table 6, and are therefore adopted for the new volume subroutine. Since the speed-volume relationship derived here is assumed to be linear, PCE's are taken as constant for all volumes. CRRI (1982) also indicate that similar values may be used on one-and two-lane roads.

<table>
<thead>
<tr>
<th>Source</th>
<th>TRUCK</th>
<th>RIGID</th>
<th>ARTIC</th>
<th>BUS</th>
<th>RECREATIONAL VEHICLE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ALL</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HRD (1965)</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>2.0</td>
<td>-</td>
</tr>
<tr>
<td>Linzer et.al. (1979)</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Werner &amp; Morrall (1976)</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>1.6</td>
<td>1.6</td>
</tr>
<tr>
<td>Seguin &amp; Crowley (1982)</td>
<td>-</td>
<td>1.6</td>
<td>2.0</td>
<td>1.6</td>
<td>-</td>
</tr>
<tr>
<td>Cunagin &amp; Messer (1983)</td>
<td>-</td>
<td>1.3-2.3</td>
<td>1.8-2.4</td>
<td>2.8</td>
<td>-</td>
</tr>
<tr>
<td>CRRI (1982)</td>
<td>2.4</td>
<td>-</td>
<td>-</td>
<td>2.3</td>
<td>-</td>
</tr>
<tr>
<td>Branston (1977)</td>
<td>-</td>
<td>2.0-2.1</td>
<td>2.4-2.9</td>
<td>1.3-1.9</td>
<td>-</td>
</tr>
<tr>
<td>OECD (1983) Sweden</td>
<td>2.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>OECD (1983) Germany</td>
<td>1.7-2.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Van Aerde &amp; Yagar (1984)</td>
<td>1.2-1.8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.1-1.6</td>
</tr>
<tr>
<td>Roess &amp; Messer (1984)</td>
<td>1.7</td>
<td>-</td>
<td>-</td>
<td>1.5</td>
<td>1.6</td>
</tr>
</tbody>
</table>

(iii) Speed-Volume Slope

3.70 The validity of capacity estimates in this model can be checked against speed-volume slopes predicted by various studies. In the 1965 Highway Capacity Manual, this slope ranged from 46 km/h per 1000 PCE/h on
fast narrow roads to 8 on wide slow roads. Typical values are of the order of 20 to 30. Duncan's (1974) preliminary analysis gave a slope of 16.7 km/h per 1000 veh/h for cars in mixed traffic. His final results ranged from 0 to 39 for the range of conditions specified, but gave unreasonable results for no trucks or more difficult terrain. Brewer et al. (1980) reported slopes of 112 and 81 km/h per 1000 veh/h for cars and trucks respectively in mixed traffic. Both of these are too high to give meaningful results for speed-volume relationships. OECD (1972) presents further speed-volume slopes for Europe, Ireland and Japan which range from 10 to 22 km/h per 1000 veh/h in mixed traffic.

(iv) Minimum Speed at Capacity

3.71 The parameter SCAP in Equations 10 to 13 is assumed to be the 15th percentile speed of the slowest vehicle type. For a normal distribution this is approximately one standard deviation below the mean. Thus an estimate is required for the standard deviation of speeds for the slowest type. The distribution of vehicle speeds is usually close to normal, with a fairly stable coefficient of variation (or CV, the ratio of the standard deviation to the mean) of the order of 0.08 to 0.18 (e.g. McLean 1982). If no other information is available, a CV of 0.12 is adopted here. This means that the minimum speed at capacity will be (1-0.12) or 0.88 times the mean speed of the slowest vehicle type.

3.72 In the Brazil model the coefficient of variation of free speed for a given vehicle type was found to be equal to \( \sigma \), the standard deviation of the logs of the constraining speeds (para. 2.13 to 2.19). Watanatada, et al. (1985) defined this as the standard deviation of the log of the error term in the constraining speed due to unmeasured road and vehicle characteristics. The values of \( \sigma^2 \) in HDM-III range from 0.0289 to 0.0724 for vehicle types. These imply CV values of 0.17 for cars and 0.21 to 0.27 for other vehicle types. The values for slower vehicles seem too large when compared with data from other sources, and all values could be biased by the modelling assumptions and estimation procedures used to develop the free speed predictions. Since this parameter is fairly stable over a wide range of studies, the lower CV of 0.12 is adopted as the default value in this study.

(v) Overtaking Opportunities

3.73 The speed-volume model as presented here takes no account of the availability of overtaking opportunities. This simplification was adopted for practical reasons, since information on overtaking opportunities is difficult to measure, and often not available in macroscopic road planning studies. On the other hand, the provision of overtaking opportunities is perhaps the major method of overcoming congestion effects on two-lane rural roads. Road realignment, the construction of overtaking and climbing lanes, and widening to four or more lanes are all methods for improving overtaking opportunities, with major impacts on traffic operations and overall road economics.
3.74 A simple but effective method for incorporating overtaking opportunities in this model would be to adjust the nominal capacity QCAP depending upon the "Net Passing Opportunities" (NPO) provided by a road (para. 3.3 to 3.9). Since NPO varies with traffic volume, this introduces a non-linear component into the model. In other words, passing opportunities may have a large effect on speed predictions at moderate volumes but little or no effect at low and high volumes. This agrees with the relationships presented in the 1965 and 1985 Highway Capacity Manuals (HRB 1965; TRB 1985). In the case of overtaking lanes and multilane road sections, the increased NPO should also affect speeds at high traffic volumes.

3.75 The development of a detailed relationship between QCAP and NPO is beyond the scope of this report. A preliminary analysis of simulated speed-flow results given by TRB (1985) indicates that a strong relationship exists but that its form does not appear to be linear. Since the current study is mainly concerned with low volume roads, the modelling of overtaking provision was not considered a high priority. However, if traffic congestion effects are to be seriously modelled over a range of volumes, and if the model is to be sensitive to the main policy options for improving overtaking, this feature must be accounted for in future enhancements to HDM-III.
IV. EFFECTS OF ROAD WIDTH

4.01 On a wide two-lane road, vehicles may be assumed to travel at their free speeds until they catch up to vehicles travelling in the same direction. Overtaking then requires some interaction with vehicles travelling in the opposite direction. As road width reduces, there is greater interaction between vehicles travelling in both the same and opposite directions, and vehicles may need to use the road shoulders for crossing and overtaking maneuvers. Reductions in road width can thus lead to four types of effects on overall road economics:

- Reductions in free speeds;
- Increased speed reductions due to traffic interactions, leading to reduced road capacity;
- Increased "effective roughness" experienced by a vehicle due to edge crossings and shoulder travel; and
- Increased road deterioration due to edge crossings and shoulder travel.

4.02 Each of these effects involves some transition between two-lane and one-lane road operations. The following sections will attempt to quantify this transitional behavior separately for each case, and these will be compared in para. 4.60 - 4.65. It should also be noted that the prediction of narrow road impacts may be somewhat cyclic. Shoulder driving, for example, may increase the effective roughness experienced by a vehicle, resulting in lower speed, which in turn affects the amount of shoulder driving. These inter-relationships will be discussed to ensure that cyclic effects are not overlooked, and that width effects are not double-counted.

A. Speed-Width Relationships for Free-Flowing Traffic

4.03 The earlier discussion of free speed models (para. 2.01 to 2.27) did not take any account of speed variations due to road width. In the Brazil equations in HDM-III, these were incorporated by reducing the desired speed $V_{DESIR}$ on narrow roads using factors derived by Hide et al. (1975). More recently, the model has adopted separate desired speeds for wide and narrow roads based on Indian data. This section discusses the mechanisms by which road width affects free speeds, and briefly reviews the results of research on this topic. A new speed-width relationship is then proposed which allows the magnitude of width effects to be varied, so that sensitivity of model predictions to this effect can be tested.

Mechanisms for Free Speed-Width Effects

4.04 A major difficulty in isolating width effects on free speeds is their interaction with sight distance, roadside friction, and traffic volume. Consider an isolated vehicle on a 4.0 m smooth paved road with excellent sight distance, clear flat roadsides, and no oncoming traffic.
This vehicle would have little reason to travel any slower than on a wide road in the same conditions.

4.05 On a road with limited sight distance, however, a driver must consider the possibility of an oncoming vehicle or other traffic disturbance just out of sight. As road width reduces, the risk associated with such an event, and the severity of avoiding action which would be required, increases considerably. Hence the sight distance availability and the expectation of intersections or roadside "friction" must be important parameters in relating desired speed to road width. Road roughness and roadside safety characteristics might also affect the safety margin required for vehicle maneuvering, and hence the desired speed of travel.

4.06 The effects of road width on traffic speeds are therefore closely related to sight distance, and to a lesser extent to friction and safety characteristics. In practice, sight distance and roadside characteristics are difficult to measure, and surrogate measures such as "good" "fair" and "poor" levels of these parameters might be required for macroscopic modelling purposes. It is assumed in this discussion that these factors affect desired speed only, and have no effect on the limiting speeds on grades, curves and rough surfaces. In view of the "draw-down" effect discussed in para. 2.13 to 2.19 when two constraints are of a similar order, this is probably a satisfactory assumption for macroscopic modelling purposes.

TRRL Free Speed-Width Studies

4.07 Several studies have been conducted by the U.K. Transport and Road Research Laboratory (TRRL); Duncan (1974) studied "low-flow" traffic operations on 17 two-lane road sites in Britain. Pavement widths ranged from 6.1 to 7.3 m, with a mean of 6.7 m. Traffic volumes were 150-990 veh/hr with 9-35 percent heavy vehicles in the traffic stream, and the speeds of all vehicles (not just "free" vehicles) were recorded for about four hours at each site. Multiple linear regression analysis showed that width had a small effect on speed, with coefficients of +0.90 and -0.63 km/h/m respectively for light and heavy vehicles. Both of these coefficients were less than their standards errors, and the negative coefficient for heavy trucks implies that speeds decrease with increasing road width. All that can be concluded from these results is that the effect of road width is small, and its magnitude could not be determined.

4.08 Duncan (1974) felt that there was some evidence of increasing speeds with road width, and assumed that traffic volumes could be expressed as flows per unit width. This implies a linear effect of width on all speed reductions due to traffic volume. While this assumption was not supported by the data, subsequent analysis by Brewer et al. (1980) showed that it was no better or worse than other modelling approaches for aggregate speed prediction.

4.09 Abaynayaka, et al. (1974) investigated speeds on 108 paved road sections in Kenya. Widths ranged from 3.0 to 7.0 m; however, the mean of 6.6 m and standard deviation of 0.6 m suggest that the great majority of
these sites must have been clustered between 6.0 and 7.0 m. Multiple linear regression analyses showed a significant width coefficient of 5.45 km/h/m for light vehicles, but no significant width effect for heavy vehicles. Speeds were also analysed on 78 unpaved road sites with widths of 4.0 to 7.9 m. The mean of 6.2 m and standard deviation of 1.2 m indicate that these were less clustered than the paved sites. In this case the multiple regression analyses produced the opposite effects; the width coefficient was not significant for light vehicles but 3.17 km/h/m for heavy vehicles.

4.10 Simple single-parameter regressions produced width coefficients of 4.32 and 5.47 km/h/m for light and heavy vehicles respectively on unpaved roads, and 7.31 km/h/m for light vehicles on paved roads. The coefficient for heavy vehicles on paved roads was not significant. Surprisingly, while the inter-correlation of geometric road features was discussed by Abaynayaka et al. (1974), width was not included in the discussion.

4.11 Hide, et al. (1975) conducted a similar analysis on 49 paved two-lane road sites in Kenya. Road widths ranged from 5.6 to 7.5 m, with a mean of 6.3 m and values well spread across this range. Traffic volumes ranged from quite low values up to 1500 veh/h. Multiple linear regression analysis showed that width had no significant effect on speed in any of twelve equations (four vehicle types by three surface types) on paved roads. Simple single-parameter linear regressions showed that width effects were not significant for trucks and buses, but yielded significant coefficients of 4.2 and 3.3 km/h/m respectively for cars and light goods vehicles. A similar analysis on 42 unpaved road sections with widths of 6.0 to 10.0 m again showed no width effects in the multiple regressions, and no significant coefficient in the single regressions except for buses.

4.12 To account for the effects of narrower road widths, Hide, et al. (1975) drew upon the earlier results of Abaynayaka, et al. (1974). Hide et al. assumed that free speed could be modelled as being unaffected by width on roads over 5.0 m wide, and linearly dependent on width when it is less than 5.0 m. While the assumption for wide roads is supported by the results of Hide et al. (1975), the assumption for narrow roads and the choice of a 5.0 m cutoff appear to be quite arbitrary. Hide, et al. (1975) argue that:

'The distribution of road widths in developing countries tends to be bi-modal with two distinct groups of "narrow" and "wide" roads.'

This argument would seem to support a simple step function of width, rather than a linear function for narrow roads. Hide, et al. (1975) claim that the coefficients selected for speed reductions on narrow roads were derived from the results of Abaynayaka, et al. (1974). However, only one of the coefficients is directly taken from the earlier report, and its new application appears to be out of the context for which it was derived.
4.13 The Hide, et al. (1975) model of zero width effect for widths over 5.0 m and a linear reduction below 5.0 m was subsequently adopted for use in the RTIM2 model (Parsley and Robinson 1982), an earlier version of the HDM-III model (Watanatada, et al. 1985b), and the Caribbean road user cost study (Morosiuk and Abaynayaka 1982).

4.14 Morosiuk and Abaynayaka (1982) investigated speeds of about 38,000 vehicles on paved roads in the Caribbean island of St. Lucia. The 28 sites had road widths of 4.3 to 8.5 m. Of these at least 5 and possibly up to 13 sites (all on the West Coast Road) had widths between 4.3 and 5.0 m. Road curvature, roughness and width were found to be highly correlated. Multiple regression analysis indicated strong width effects, with coefficients of 3.0 to 3.9 km/h/m. Morosiuk and Abaynayaka argued, however, that:

'the effect of road width on speed is not continuous through the spectrum of road widths as the equations suggest,'

although no evidence was presented to support this argument. They did demonstrate that observed speeds were different on "wide" and "narrow" roads, even when segregated by grade and curvature categories. Separate regressions were therefore conducted for "wide," "narrow" and "all" roads, with the width parameter excluded. These showed that narrow roads produced speed reductions of 19.9 km/h for cars, 13.5 km/h for light vehicles and 18.0 km/h for trucks with a power to weight ratio of 16.0.

4.15 Morosiuk and Abaynayaka (1982) then made a decision to adopt the speed prediction equations for "all" roads, and add a speed reduction adjustment factor for narrow roads. The validity of this approach seems questionable. While the single equation may be convenient, and has the advantage of including a small roughness effect, it already incorporates most of the width effects in the observed data through the highly correlated surrogate measures of curvature and roughness. The overlaying of further width effects (and only for narrow roads) appears to take these regression equations outside the range of conditions for which they were established. The coefficients used for width adjustment appear to have been derived from simple single-parameter regressions using the full range of road widths. If this is the case, their application as width adjustment factors using a modified parameter (5.0-W) also seem dubious.

4.16 Despite the lack of roughness coefficients and the variability of other coefficients between wide and narrow roads, the separate equations developed by Morosiuk and Abaynayaka (1982) would appear to be the most valid and useful of their results. These analyses, however, make no attempt to predict the effects of width within the wide and narrow road groupings. It would have been quite reasonable to derive parallel equations for wide and narrow roads which differ only in the free speed coefficient.
Other Free Speed-Width Studies

4.17 Road width effects were also investigated in the Indian Road User Cost Study (CRRI 1982). Only "free" speeds were recorded, with observers subjectively rejecting vehicles which appeared to be affected by other nearby traffic in either direction. 38 paved road sites were used, with widths (w) clustered in three groups:

- one lane: \( W=3.7-3.8 \) m
- intermediate: \( W=5.5-5.6 \) m
- two-lane \( W=6.6-7.0 \) m

Weighted multiple linear regression analysis gave width coefficients of 0.61 to 1.06 km/h/m, assuming a continuous linear width effect over the full range of observed widths, from 3.7 to 7.0 m. It should be noted that widths were substantially correlated with other road characteristics, especially roughness.

4.18 In calibrating the Brazil model for Indian conditions, however, Viswanathan (1985) used a different approach. He assumed that the effects of width could be modelled using a separate desired speed \( V_{\text{DESIR}} \) for each of the three width groupings studied by CRRI (1982). After estimating most of the HDM-III speed parameters from separate sources, Viswanathan calibrated the three \( V_{\text{DESIR}} \) values and the spread parameter \( \beta \) from the CRRI data.

4.19 The results of this analysis gave \( V_{\text{DESIR}} \) values shown in Table 7. These indicate a large speed reduction for cars in moving from 7.0 m to 5.5 m road widths, with a smaller reduction in moving to a 3.7 m width. For trucks and buses, very little difference is apparent between 7.0 and 5.5 m, but a large reduction occurs between 5.5 and 3.7 m. Viswanathan (1985) also reported \( \beta \) values considerably larger than those found for Brazil by Watanatada et al. (1985a). He found that the HDM-III model predictions were highly sensitive to this parameter. The higher value gives larger drawdown effects when several constraints are of a similar order, which frequently occurs on the Indian roads studied.

<table>
<thead>
<tr>
<th>Road Type</th>
<th>Width (m)</th>
<th>Desired Speed (km/h) and (t-statistic)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Car</td>
</tr>
<tr>
<td>Two-lane</td>
<td>3.7-3.8</td>
<td>90.6 (10.6)</td>
</tr>
<tr>
<td>Intermediate</td>
<td>5.5-5.6</td>
<td>70.3 (3.3)</td>
</tr>
<tr>
<td>Narrow</td>
<td>3.7-3.8</td>
<td>64.0 (1.5)</td>
</tr>
</tbody>
</table>

Source: Viswanathan (1985)
4.20 Watanatada, et al. (1987a) subsequently re-analyzed the data of Viswanathan (1985) to estimate new values of VDESIR and h for Indian conditions. Because of some inconsistencies in speeds on intermediate-width roads, these were combined with wide two-lane roads to produce two width classes. The analysis gave results similar to those of Viswanathan (1985), with large values of h, high desired speeds and substantial differences between desired speeds on wide and narrow roads. The Brazil free speed relationships incorporated into the HDM-III model were all derived from data collected on 6.0 m or wider roads, and hence could not incorporate width effects. The speed-width effects based on India data were subsequently incorporated into the HDM-III model to provide some mechanism for responding to road width.

4.21 Brodin and Carlsson (1985) describe a non-linear relationship between width and free speed as used in the Swedish traffic simulation model for two-lane roads. The relationship indicates that the free speed is affected by road width at all widths up to 12 m (taken as a 7 m seal with two 2.5 m paved shoulders), and that the slope of this effect increases continuously with decreasing road width. For road widths less than 6 m, the median speed is given by:

\[
\frac{1}{V_1} = \frac{1}{25.15} + \frac{a}{W-2.5} - \frac{a}{4.5} \tag{15}
\]

where:  
\(V_1\) = median speed (m/s)  
W = road width (m)  
a = calibration constant (0.042)

When W=7.0 m, this predicts a free speed of 25.15 m/s or 90.5 km/h. Brodin and Carlsson (1985) give no information on the source of this relationship or the availability of supporting data.

Summary of Width Effects on Free Speed

4.22 Table 8 summarizes some of the results of various free speed-width for cars and trucks on paved roads. The table suggests that the observed speed difference between one-lane and two-lane roads is of the order of 20 km/h, but attempts to derive speed-width coefficients from regression analyses have produced mixed and inconclusive results.
TABLE 8  Summary of Speed-Width Relationships on Paved Roads

<table>
<thead>
<tr>
<th></th>
<th>Simple Linear Regression Coefficient km/h/m</th>
<th>Multiple Linear Regression Coefficient km/h/m</th>
<th>Wide-Narrow Road Speed Difference km/h</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cars</td>
<td>Trucks</td>
<td>Cars</td>
</tr>
<tr>
<td>Duncan (1974)</td>
<td>-</td>
<td>-</td>
<td>0.90</td>
</tr>
<tr>
<td>Abaynayaka, et al. (1974)</td>
<td>7.31</td>
<td>n/s</td>
<td>5.45</td>
</tr>
<tr>
<td>Hide, et al. (1975)</td>
<td>4.2</td>
<td>n/s</td>
<td>n/s</td>
</tr>
<tr>
<td>Morosuik and Abaynayaka (1982)</td>
<td>8.1</td>
<td>6.2</td>
<td>3.9</td>
</tr>
<tr>
<td>CRRI (1982)</td>
<td>-</td>
<td>-</td>
<td>1.05</td>
</tr>
<tr>
<td>Viswanathan (1985)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Watanatada et al. (1987a)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

n/s = no significant effect

a/ assumed to be derived from simple regression: derivation not explained.

4.23 Figure 8 presents a number of alternative speed-width models in graphic form. Depending upon which model is adopted, the predicted effects of a small change in width could vary enormously. The models are:

- a linear relationship over the full range of widths. This is the basis of all of the regression coefficients in Table 8, but is clearly unrealistic for wide roads.

- a step function with separate values for wide, narrow and perhaps intermediate roads. This reflects the speed difference results in Table 8 and is the most appropriate model when width values are highly clustered, since it does not attempt to extrapolate outside the range of observed conditions.

- some form of relationship which asymptotes to an upper limit on wide roads. This is close to the "true" effect of width on speed, but little is known about the value of width at which speeds begin to be affected, and the shape of the lower part of the curve.
the step-linear model of Hide, et al. (1975). While this represents an attempt to come closer to true behavior, it gives the appearance of a sensitivity to width changes which seems to have little foundation. This is because the choice of 5.0 m cutoff appears arbitrary, and the slopes adopted by various studies have actually been derived from a type (a) relationship.

![Figure 8: Some Alternative Speed-Width Relationships](image)

**FIGURE 8 Some Alternative Speed-Width Relationships**

4.24 From the preceding discussion, current knowledge of width effects on free speed can be briefly summarized as follows:

- There are substantial differences between observed vehicle speeds on wide and narrow roads.

- Within the separate wide and narrow road groupings, there is evidence of a variation in free speed with changes in road width. On two-lane roads, it seems probable that width effects are not linear, that is, width changes between about 6.5 and 7.5 m probably have little or no effect on speeds while widths below about 6.5 m are more likely to reduce speeds substantially.

- No information is available on the effects of width changes on one-lane roads. Intuitively, the reduction in free speeds on narrow roads should vary with sight distance, road roughness and shoulder quality, but these effects have not been quantified.
An Alternative Speed-Width Model for Free Flow

4.25 One of the objectives of this study is to evaluate the effects of alternative road width standards. Since information on width effects is limited, it will be necessary to test a range of assumed width effects. The current HDM-III width effects based on India data are not sufficiently general for the close scrutiny required in this application.

4.26 Analogous to the HDM-III model, the alternative model assumes that road width affects desired speed only, and has no effect on the limiting speeds due to curves, grades and road roughness and that the desired speed on a one-lane road (say 3.8 m wide) is a known constant VDIFF km/h below that on a wide two-lane road, VDESIR. A default value of 20 km/h for VDIFF is suggested for all vehicle types, where no other information is available. The appropriate value could vary considerably with shoulder roughness and sight distance, possibly reducing to near zero in ideal conditions. It should be noted that some of the change in VDIFF could be due to roughness (as discussed in para. 4.43 to 4.49), and care should be taken to avoid double-counting. The modified desired speed VDESIRW is then assumed to be given by:

\[
V_{DESIRW} = V_{DESIR} - V_{DIFF} \times W_{FFACT} \quad (16)
\]

The multiplicative factor WFFACT is assumed to vary in a non-linear fashion with road width, reflecting a transition from two-lane to one-lane operation. It is proposed that WFFACT should be zero for widths above about 6.5 m, and increase more rapidly for widths between about 5.0 and 6.0 m. While changes for widths below about 4.5 m are unknown, these could be attributed to non-width characteristics such as sight distance and surface quality. Based on this limited information a distribution of values for WFFACT is postulated in Table 9. Values for intermediate widths may be obtained by linear interpolation.

<table>
<thead>
<tr>
<th>Road Width (m)</th>
<th>7.4</th>
<th>6.7</th>
<th>6.1</th>
<th>5.5</th>
<th>4.9</th>
<th>3.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>WFFACT</td>
<td>0.0</td>
<td>0.0</td>
<td>.04</td>
<td>.50</td>
<td>.96</td>
<td>1.0</td>
</tr>
</tbody>
</table>

B. Width Effects on Traffic Interactions

Overtaking Effects

4.26 In the previous section it was noted that overtaking gap requirements increase as road width decreases. Troutbeck (1982), for example, found that overtaking times on a 6.0 m wide road were 8 to 12 percent higher than equivalent values for a 7.4 m road.
4.27 The mechanism of overtaking behavior on two-lane roads is also described in para. 3.01 to 3.09. On roads which are considerably wider or narrower, different mechanisms apply:

- On multi-lane roads or two-lane roads with auxiliary lanes, a driver wishing to overtake needs only to check for space in the adjacent lane, and that he can travel faster in that lane. The supply of overtaking opportunities is very much greater on these road types.

- On two-lane roads with wide paved shoulders, some (but not all) slow vehicles pull aside onto the shoulder to be overtaken, though they may not do this immediately on being caught up. Some fast vehicles also attempt to overtake when gaps are inadequate, knowing that other vehicles have room to move aside (Ahman 1968). Such roads have significantly higher overtaking supply than two-lane roads (Morrall 1984) but there is generally some inconsistency and uncertainty in driver behavior.

- On narrow two-lane or one-lane roads the overtaking maneuver requires both a gap in the oncoming traffic and a yielding by the slower vehicle to one side to allow the faster vehicle to pass. One or both vehicles may need to travel partly on the road surface and partly on the shoulder, and in some cases where the slow vehicle does not yield, the overtaking vehicle may travel completely on the shoulder or verge. Assuming some sort of driveable shoulder exists, the opportunities for such maneuvers are essentially the same as those for two-lane roads. However, the speed of overtaking and the gap required to complete it are likely to be entirely different.

4.28 On wide roads, crossing vehicles have no interaction beyond a slight lateral shift away from the centreline. On narrower roads, however, crossing vehicles must move to the edge of the road surface, or partially or even fully onto the shoulder. To allow for edge crossing, rougher surfaces on the shoulder, and perhaps reduced lateral clearance between vehicles, speeds may be reduced. The extent of this speed reduction varies with road width, shoulder type, and the surface quality of pavement, shoulders and their common edge. In the extreme case of a one-lane bridge or mountain pass, one vehicle may have to pull completely off the road or back up to make way for the other.

4.29 The frequency of vehicle crossings increases with the square of traffic volume. Consider $Q_1$ veh/h travelling at a uniform speed of $V_1$ km/h, in one direction, and $Q_2$ veh/h at speed $V_2$ in the opposite direction. In one hour a vehicle in direction 1 would travel $V_1$ km. It would meet $Q_2$ opposing vehicles crossing a fixed point and a further $Q_2/V_2$ vehicles per km over its journey of $V_1$ km, or a total of $Q_2 + V_1 (Q_2/V_2)$. 
The total number of meetings per hour for all vehicles travelling in direction 1 is therefore:

$$Q_1 Q_2 (1 + V_1 / V_2)$$  \hspace{1cm} (17)

This expression can be used to find crossing rates for any two groups of vehicles. If $V_1$ and $V_2$ are equal and $Q_1 = Q_2 = Q/2$ then the total crossing rate per hour is given by $Q^2/2$ and the crossing rate per kilometre per hour (CRATE) is

$$\text{CRATE} = \frac{Q^2}{2V}$$  \hspace{1cm} (18)

where: $Q$ is the two-directional flow in veh/hr and $V$ is the average speed in km/hr.

4.30 Palaniswamy (1985) assumed that crossing speed could be expressed as a function of free speed and road width, reducing linearly from 100 percent of free speed on a 7.0 m road to zero on a 3.0 m road. Palaniswamy further assumed linear transitions between free and crossing speeds, as defined by a "point of deceleration" and a "point of conclusion of crossing." The method of determination of these points, however, is not documented, so it is not clear whether they are based on fixed time, fixed distance, fixed acceleration or some other assumptions.

4.31 The transitional changes in vehicle behavior with respect to road width may be described by an alternative model. Let us assume that the reduction in crossing speed, when all vehicles use the road shoulder, can be expressed as a proportion of free speed for each vehicle type. This proportion PVF may be expected to vary with the quality of road shoulders; for simplicity it may be considered constant for all vehicle types.

4.32 For roads of intermediate width, the appropriate speed reduction will lie somewhere between zero and PVF, since not all vehicles use the road shoulder. The average crossing speed could be given by:

$$V_{\text{CROSS}} = V_{\text{FREE}} (1 - PVF \times WVFACT)$$  \hspace{1cm} (19)

where: WVFACT represents the proportion of the total speed reduction which is appropriate for a given width. It seems reasonable to assume that WVFACT will have an S-shaped distribution, with no effect on wide roads, an increasingly strong effect as more vehicles are required to use the shoulder, and little or no variation with varying width on one-lane roads. Variations in this range would depend mainly on shoulder quality rather than seal width. The distribution should reflect the proportion of vehicles using the shoulder, plus some speed reductions due to crossing on very narrow seals. A typical distribution of WVFACT is postulated in Table 10, where intermediate values may be obtained by linear interpolation.
At very low traffic volumes, crossing delay could be estimated in the following manner. Assume that the crossing maneuver requires \( CTIME \) sec., divided equally before and after the point of meeting, and assume that vehicles decelerate smoothly to crossing speed and immediately accelerate back to free speed. From the area under a speed-time plot, each travels \( \frac{1}{2} \) \((\text{VFREE} - \text{VCROSS}) \times CTIME \) meters less in this maneuver than it would have travelled in the same period at free speed. Crossing delay (\( \text{CDELAY} \)) sec. may then be defined as the extra time required to cover this distance at free speed. This is given by:

\[
\text{CDELAY} = \frac{(\text{VFREE} - \text{VCROSS}) \times CTIME}{2 \times \text{VFREE}}
\]

Substituting from equation (19) then gives:

\[
\text{CDELAY} = 0.5 \times \text{WVFAC} \times \text{PVF} \times CTIME
\]

This gives a maximum delay of 2.5 s when a vehicle slows to a complete stop and accelerates back to free speed in a period of 5 s. Similar expressions could be developed for \( \text{CDELAY} \) assuming a constant acceleration/deceleration rate.

These equations can be used to predict overall delay from the overall mean desired speed. Separate calculations for each vehicle class could improve this estimate, since slower vehicles have more crossings in a given period of time (Equation 18).

As traffic volumes increase, traffic bunches form and some crossings involve more than two vehicles. Speed reductions and probability of shoulder travel for a given maneuver could increase in these cases, but the extra delay caused by the additional crossing would still be smaller than that for a single crossing. If the proportion of following vehicles is \( \text{PFOLL} \), then the number of bunches crossed by a given vehicle (that is, the number of potential shoulder travel maneuvers) can be determined by rewriting Equation 17 as:

\[
Q_1 \times Q_2(1-\text{PFOLL})(1+V_1/V_2)
\]

so that equation (18) for bunch crossings becomes:

\[
\text{CRATE} = \frac{Q^2(1-\text{PFOLL})}{2V}
\]
Each maneuver, however, would require extra travel at the (potentially) slower speed VCROSS. For a typical following headway of 2.0s, two vehicles with the same speed would cross in 1.0 s, causing an additional delay of 1.0 x (VCROSS-VFREE)/VFREE s, or 1.0 x WVFACT x PVF for each following vehicle. In this case the average delay in Equation 21 could be rewritten as:

\[ C_{\text{DELAY}} = WVFACT \times PVF \times (PFOLL + 0.5 \times CTIME) \] (24)

As traffic volumes increase further, an increasing number of vehicles would not be travelling at their free speeds, and drivers might modify their behavior to allow for frequent crossing maneuvers. These equations are therefore only applicable for low volume roads.

The models presented here provide simple approximate procedures for predicting crossing delays on narrow roads, where little information is otherwise available. In the current study, these relationships will not be used to predict speed reductions, but they will be used to estimate the effects of crossing maneuvers on road deterioration and vehicle operating costs (see following Sections C and D).

Capacity

Current mechanistic models for speed-volume relationships are not presently sufficiently well developed for use in the current study. The alternative approach of a direct speed-volume model was therefore adopted, and a suitable model form was proposed. In this approach the effects of road width on overtaking and crossing delays are assumed to be reflected in reduced road capacity, which effectively increases the speed-volume slope. This section discusses the effects of width on road capacity.

While there is evidence that capacity reduces with road width, there is virtually no reliable quantitative data available on this relationship. The Highway Capacity Manual relationships (HRB 1965, TRB 1985) offer no supporting evidence, and their validity has been questioned by McLean (1980). The simple proportional relationship of Duncan (1974) also lacks any firm support. The basis of the Swedish relationships reported by OECD (1983) is unknown to this author. The Indian values of CRRI (1982) have some basis in observed data, but were derived using extremely subjective criteria, and must be regarded as highly approximate. Both and Bayley (1976) present a range of assumed capacities for various road widths. While the upper values appear to be derived from the Highway Capacity Manual, the source of the values adopted for narrower roads is unknown. Table 11 compares values derived from all of these studies.

The frequency discussion of crossing and overtaking mechanisms gives some indications on the expected form of a width effect on speed reductions due to vehicle interactions. As width reduces from a wide to a medium two-lane road, it has very little effect on speeds. However, as vehicles need to slow down — either for very narrow lanes or driving on the shoulder — much greater reductions may be expected. Once a vehicle has to
TABLE 11 Capacity Reduction Factors for Road Width

<table>
<thead>
<tr>
<th>Source</th>
<th>Estimated Capacity Reduction for Roadway Width (m) of</th>
<th>7.4</th>
<th>6.7</th>
<th>6.1</th>
<th>5.5</th>
<th>4.9</th>
<th>3.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sweden (OECD 1983)</td>
<td></td>
<td>1.0</td>
<td>.96</td>
<td>.92</td>
<td>.80</td>
<td>.71</td>
<td></td>
</tr>
<tr>
<td>TRB (1985)</td>
<td></td>
<td>1.0</td>
<td>.94</td>
<td>.83</td>
<td>.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Duncan (1974)</td>
<td></td>
<td>1.0</td>
<td>.92</td>
<td>.83</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CRRI (1982)</td>
<td></td>
<td>1.0</td>
<td></td>
<td></td>
<td>.40</td>
<td>.15</td>
<td></td>
</tr>
<tr>
<td>Both &amp; Bayley (1976)</td>
<td></td>
<td>1.0</td>
<td>.88</td>
<td>.81</td>
<td>.76</td>
<td>.70</td>
<td>.60</td>
</tr>
<tr>
<td>Adopted WCFACT</td>
<td></td>
<td>1.0</td>
<td>.98</td>
<td>.90</td>
<td>.80</td>
<td>.70</td>
<td>.60</td>
</tr>
</tbody>
</table>

a/ Swedish and TRB factors assume shoulder widths of 2 m on 7.4 and 6.7 m road; 1 m on 6.1 m roads; and zero on narrower roads.

b/ Figures in brackets are estimated capacities in PCE/h, except for the Duncan (1974) results which are veh/h assuming 15 percent heavy vehicles.

travel partly on the shoulder for overtaking or crossing, it is difficult to identify any further need for speed reduction with reducing width. These mechanisms suggest three zones of width effects:

  * Little if any width effect on wide to medium two-lane roads.
  * Severe width effects as increasing proportions of the traffic need to travel on the shoulder for crossing and overtaking.
  * Little if any width effect once all crossing maneuvers require use of the shoulder.

4.42 The transition from on-pavement to off-pavement crossing behavior probably occurs mainly between widths of 5.0 and 6.0 m. It seems reasonable small width effect for widths outside this range. In practice, the shoulder type and quality have an enormous effect on this relationship, and these are almost certainly correlated with road width (i.e. narrow roads are likely to have worse shoulders). From the limited information available here, a set of assumed capacity reduction factors for width (WCFACT) is given in Table 11. These values must be regarded as highly approximate, and particularly sensitive to the width, type, and quality of shoulders available. These latter effects will be partly accounted for by the use of an "effective roughness" parameter as described in the following action.
The nominal capacity QCAP in Section 6.3.1 is then given by:

\[ QCAP = 2500 \times WCFAXT \quad (25) \]

C. Effective Roughness

4.43 When crossing or overtaking maneuvers require travel on the road shoulder, the surface roughness experienced by a vehicle is that of the shoulder and the pavement edge crossing rather than that of the pavement. The "effective roughness" for the whole journey is thus a combination of pavement, edge and shoulder roughness values, weighted according to the proportion of travel on each. This could be substantially higher than the pavement roughness alone, affecting speed, fuel consumption and other vehicle operating costs. An approximate estimate of effective roughness could be obtained from the following equation:

\[ RE = STIME \times RSE + (1- STIME) \times RP \quad (26) \]

where \( RE \) = effective roughness;

\( STIME \) = proportion of travel time spent on the road shoulder;

\( RSE \) = shoulder/edge roughness; and

\( RP \) = pavement roughness.

Pavement roughness (\( RP \)) is the current HDM-III parameter (in QI, BI or IRI, depending upon the measurement scale used).

4.44 While the roughness of the shoulder and pavement edge crossing (\( RSE \)) could be measured in the same way as surface roughness, it would often be a subjective assessment, taking particular account of potholes, surface drop-off and irregularity of the pavement edge. While the ratio of shoulder travel time per edge crossing could vary, a single roughness estimate is a satisfactory approximation. \( RSE \) will generally be greater than \( RP \), and sometimes much greater. A default value of double \( RP \) is used in this study.

4.45 The proportion of travel time spent on the shoulder will depend upon road width, frequency of crossing and overtaking maneuvers, and duration of those maneuvers. At very low volume on a one-lane road, the value of \( STIME \) may be approximately derived as follows:

- Average journey time per kilometer for each vehicle is 3600/V s.
- Frequency of crossing maneuvers = \( Q^2 / 2V \) per km per hour (Equation 18).
- Frequency of overtaking maneuvers = \( (CV/\pi) \sigma^2 / V \) per km per hour, (Equation 4) where \( CV \) is the coefficient of variation \( (\sigma/V) \) for the free speed distribution, which generally takes a value of about 0.12.
Average duration of shoulder travel for crossing is CTIME s; this is estimated as 5 s assuming vehicles maintain a clear headway of 2.5 s before and after crossing.

Duration of shoulder travel for overtaking is OTIME s. Troutbeck (1982) reported overtaking times of 5 to 20 s with a median of 10.5 s for overtakings around a car travelling at 60 km/h on a 6.0 m road. A typical value of 12 s is adopted here.

Each crossing or overtaking maneuver is assumed to require shoulder travel by both vehicles.

The estimated proportion of shoulder travel for these conditions (one lane road and very low volume) is therefore given by:

\[
\text{STIME} = \frac{2 \times \text{CTIME} \times Q^2/2V + 2 \times \text{OTIME} \times (CV/\sqrt{\pi}) \times Q^2/V}{Q \times 3600/V}
\]  

which reduces to:

\[
\text{STIME} = (\text{CTIME} + 2 \times \text{OTIME} \times CV/\sqrt{\pi}) \times Q/3600
\]  

where Q is the actual hourly traffic volume in veh/h, rather than the PCE volume. Using the default values assumed here, the following expression is obtained:

\[
\text{STIME} = 6.6 \times Q/3600
\]  

As traffic volume increases, not all overtaking demand is satisfied, and some crossings involve bunches of vehicles. If it is assumed that half of the overtaking demand is satisfied, and crossing time is modified to account for the proportion of following vehicles (PFOLL; see Equations 22 to 24), then Equation 28 becomes:

\[
\text{STIME} = (\text{CTIME} \times (1 - \text{PFOLL}) + \text{PFOLL} + \text{OTIME} \times CV/\sqrt{\pi}) \times Q/3600
\]

Using PFOLL = 0.20 with the previous default parameters gives:

\[
\text{STIME} = 5.0 \times Q/3600
\]

This equation is adopted here as an approximate prediction for shoulder travel time on narrow roads at low volumes. On wider roads, not all vehicles will travel on the shoulders. For both crossing and overtaking maneuvers, the proportion of vehicles with one set of wheels on the shoulder (PSH) is assumed - as in the previous sections - to take an S-shaped distribution, with very few edge crossings on widths above 6.0 m and all vehicles using the shoulder for widths below 4.0 m. A typical distribution is postulated in Table 12, and comparisons with other transitional distributions is given in Section E (para. 4.60 - 4.65).
TABLE 12 Proportion of Vehicles Using Road Shoulder vs. Road Width

<table>
<thead>
<tr>
<th>Width (m)</th>
<th>7.4</th>
<th>6.7</th>
<th>6.1</th>
<th>5.5</th>
<th>4.9</th>
<th>3.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>PSH</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.3</td>
<td>0.6</td>
<td>1.0</td>
</tr>
</tbody>
</table>

4.49 In practice, heavy vehicles are less likely to use shoulders than light vehicles, so that separate predictions could be made for each vehicle type. In view of the approximate nature of this prediction, however, a single distribution is used for all vehicle types. Values of PSH for other road widths may be linearly interpolated from Table 11. Equation 31 can now be generalized for all road widths, as:

\[ \text{STIME} = 5.0 \times \text{PSH} \times \frac{Q}{3600} \]  

(32)

This equation can now be used with Equation 26 to predict overall effective roughness for a given road section. Because these equations were derived for low-volume conditions, STIME is assumed to have an upper limit of 40 percent of total travel time. Higher values of STIME could be expected to cause substantial changes in driver behavior and speed, so that the models presented here could no longer apply.

D. Edge Deterioration

4.50 The edge of a road pavement experiences a considerable loading each time a crossing or overtaking maneuver requires travel on the shoulder. While some damage is caused by air turbulence of heavy vehicles travelling near the pavement edge, it is convenient to assume that edge deterioration is directly proportional to the number of edge crossings. It is further assumed that total crossing rate is an adequate parameter. In practice heavy vehicle crossings involve more wheels and probably do more damage, but these vehicles are more likely to stay on the sealed pavement than light vehicles.

4.51 The frequency of edge crossings can be estimated using similar assumptions to those of Equations 27 to 32. If each maneuver involves two vehicles, each making two edge crossings, then the overall rate of edge crossings (ERATE per km per hour) is given by:

\[ \text{ERATE} = 4 \times \text{PSH} \times \left( \frac{(1-\text{PFOLL})}{2} + \frac{\text{CV}}{\pi} \right) \frac{Q^2}{V} \]  

(33)

Assuming that CV = 0.12 and only half of the overtaking demand is satisfied (para. 3.01 to 3.09), results in:

\[ \text{ERATE} = 2 \left( 1.07 - \text{PFOLL} \right) \times \text{PSH} \times \frac{Q^2}{V} \]  

(34)

4.52 Observed road maintenance requirements could be used to relate edge deterioration to the parameter ERATE. Paterson (1987) presents useful data from Hide and Keith (1979) which could be used in this manner. The
data gave annual road patching requirements for a set of roads 4.0 to 5.0 m wide in the Caribbean Island of St. Lucia. The worst three roads required 45 m$^3$/km/yr patching, while the best three required 20 m$^3$/km/yr, and the annual traffic volumes were 760 and 170 veh/d respectively. No information was given on differences in width or pavement strength for the two cases.

4.53 Assuming that half of the extra 25 m$^3$/km/yr is edge damage directly attributable to the additional traffic for that case and that PFOLL = 0, PSH = .75 (i.e., width = 4.5 m), V= 75 km/h, and the appropriate hourly traffic volume is 0.10 of average daily traffic (i.e., volume equally distributed for 10 hours per day) then the two values of ERATE are therefore 123.6 and 6.2 edge crossings per km per hour for the high-volume and low-volume cases respectively. This means that 12.5 m$^3$/km/yr of patching is caused by 428,500 crossings/km/yr (117.4 x 10 hours x 365 days), or the average patching requirement due to edge crossings is approximately 30 m$^3$ per million edge crossings.

4.54 This formulation gives an approximate procedure for taking account of edge deterioration. The constant of 30 m$^3$ per million crossings could vary greatly with the strength of pavement and shoulders and the extent of material loss adjacent to the pavement edge. The prediction has the desirable property of increasing with the square of hourly traffic volume. The prediction is also inversely proportional to average speed, since this affects meeting rates; however this ignores possible increases in edge damage at higher speeds.

4.55 A major failing of the procedure is that it does not take account of maintenance or existing condition, and does not accelerate in the absence of maintenance. Nevertheless it provides some estimate of the extra costs associated with narrow roads at increasing traffic volumes, and is therefore retained in this study.

4.56 An upper limit may be imposed on edge deterioration to keep it in the scale of other maintenance costs, and to relate it to the surface quality and maintenance of the existing road. It will be assumed here that total edge patching requirements will not exceed 15 percent of the total pavement area. They are also assumed to be limited to 20 percent of the pavement edge area in a given year, based on an average pothole depth of 80 mm (Paterson 1985) and an edge width of 400 mm on each side of the road. This latter criterion places an upper limit of (.20x1000x.08x.40x2) = 12.8 m$^3$ per km of edge patching per year, which is similar to that of the example presented earlier in this section.

4.57 While pothole development in the HDM-III model does not commence until cracking and other forms of deterioration are well established, it is assumed in this model that edge deterioration can begin as early as two years after pavement resurfacing. It is further assumed that half of the edge deterioration can be added to pothole area for the purposes of calculating pavement roughness. This will be in addition to the effective roughness of shoulder travel described in the preceding section, and this component of the roughness could affect road maintenance decisions.
4.58 The edge deterioration model described here presents a very simplified procedure for making the HDM-III model sensitive to the maintenance impacts of road width standards. Since road widening projects may depend greatly on the maintenance and vehicle operating costs due to edge damage, it is important to incorporate some ability to reflect these effects.

4.59 Several parameters have been assumed in this discussion, including the volume of patching required per million vehicle crossings, the average pothole depth, upper limits on edge deterioration, and the proportion of edge damage which contributes to pavement roughness. Little substantive evidence is available to support these assumptions, and appropriate values are likely to vary with such factors as pavement and shoulder quality. Any analysis using this procedure should therefore use a range of parameter values to test the sensitivity of model predictions to these assumptions.

E. Summary of Width Effects

4.60 The reviews and discussions in this chapter have shown that there is very little quantitative information available on the effects of road width on overall road economics. Nevertheless it is clear that road width affects vehicle free speeds and interactions, the roughness experienced by the vehicle and the deterioration of the shoulder and pavement edge. Some modelling of these effects is therefore necessary in any evaluation of alternative width standards on rural roads.

4.61 The models discussed in Sections A to D of this chapter are not presented as accurate and reliable predictions of road width effects. They are intended rather as rough estimates of the magnitude of these effects, whose sensitivity can be tested by varying some of the assumed model parameters. Where the effects are found to be small or insensitive to model assumptions, the components may be simplified or ignored. Where effects are found to be significant, on the other hand, greater attention should be paid to model refinement and parameter estimation.

4.62 A number of width effects appear to be characterised by a distinct difference between two-lane and one-lane road parameters, and some form of non-linear transition between the two. In particular it is clear that width effects on traffic operations are negligible for widths above about 6.5 m, and accelerate as road width reduces from 6 to 5 to 4 m.

4.63 From a consideration of behavioral characteristics and very limited empirical evidence, four different transitional distributions were postulated in the preceding sections. These are compared in Table 13. While all of these distributions are very approximate, it is difficult to derive a common distribution which would serve all purposes. The four separate distributions are therefore retained at this stage of the analysis, and the sensitivity of predicted user costs to these values will be investigated at a later stage of this study.
4.64 The magnitude of differences between two-lane and one-lane road parameters is also fairly uncertain. Estimated differences may be partly due to differences in unmeasured road characteristics such as sight distance and shoulder quality, or differences in traffic composition and trip purpose or the road "speed environment" (Chapter II; para. 2.01-2.27).

**TABLE 13 Comparing Proposed Parameters for Width Effects**

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Parameter</th>
<th>Proportion of Change Occurring For Widths(m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>7.4</td>
</tr>
<tr>
<td>Free speed</td>
<td>WFFACT</td>
<td>0.0</td>
</tr>
<tr>
<td>Crossing speed</td>
<td>WVFACT a/</td>
<td>0.0</td>
</tr>
<tr>
<td>Capacity</td>
<td>WCFACT</td>
<td>0.0</td>
</tr>
<tr>
<td>Shoulder use</td>
<td>PSH</td>
<td>0.0</td>
</tr>
</tbody>
</table>

a/ Note that this table shows the changes in WCFACT, and not its actual values.

4.65 Since four types of road width effect are considered, it is important to avoid double-counting of these effects. In particular, speed reductions due to edge and shoulder roughness will be modelled by the edge deterioration and effective roughness calculations, and should therefore be excluded from free speed and capacity analyses. It is not clear whether this has been satisfactorily achieved in the preceding discussions. While the capacity reduction proposed in Table 11 is less than those reported elsewhere, the value of VDIFF derived from Table 8 could well incorporate some double-counting of other effects. To provide a conservative estimate of width effects for good roads with very good shoulders, a reduced value of 15 km/h is arbitrarily adopted for VDIFF in this study.
V. ACCIDENTS

5.01 Road geometry standards can have a major effect on road safety. Extensive research has been undertaken on this subject, and a detailed review is beyond the scope of the current study. Useful summaries are provided by FHWA (1980, 1982), Solomon (1964), Jorgenson (1966), RRL (1964), Sinclair Knight and Partners (1973), Boughton (1975) and Armour and McLean (1983).

5.02 While there is some divergence among the various research studies, they generally indicate that accidents can be reduced by the following types of road improvement:

- Realignment of road sections with sharp curves and steep grades, especially where these are below the standard of adjacent road sections;
- Increasing lane width, at least up to 3.4 m, or adding additional lanes;
- Adding auxiliary lanes;
- Controlling access to a road;
- Providing gentler side slopes and removing fixed objects, or installing guard fence to protect hazards; and
- Providing special facilities for runaway trucks on steep downgrades.

5.03 Road safety improvements are sometimes considered as a relatively low-cost alternative to major realignment of a problem section of road. McLean (1983b) reviewed a number of research studies, and expressed the results as accident reduction factors for various geometric improvements. The resultant reductions for travelled-way width, shoulder width, and total roadway width are presented in Figures 9 and 10. McLean derived base accident rates for wide two-lane roads with wide shoulders which were of the order of one accident per million vehicle kilometers. These appear to be mainly injury accidents, but the bases for these rates varied or were not specified in the studies reviewed.

5.04 McLean also presented some information on accident relationships with grade and curvature. Table 14 presents some early German factors quoted by HUFSM (1971), while Figure 11 gives an accident-curvature relationship derived by Shrewsbury and Summer (1980). The effects of grades and curves depend upon the consistency of geometric standards along the road. Table 15, for example, shows a much higher accident rate for sharp curves on a fairly straight alignment than for similar curves in more curvy alignment.
FIGURE 9 Accident Rate Adjustments for Lane and Shoulder Width

Source: McLean (1983b)
FIGURE 10 Accident rate adjustment factor vs. roadway width relations (13.2 m roadway width base condition).

Source: McLean (1983b)

TABLE 14 Accident Rate Adjustment Factors for Grade and Curvature

<table>
<thead>
<tr>
<th>Grade (per cent)</th>
<th>Adjustment Factor $K_G$</th>
<th>Curve Radius (m)</th>
<th>Adjustment Factor $K_C$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 1.9</td>
<td>1.00</td>
<td>&gt; 4000</td>
<td>1.00</td>
</tr>
<tr>
<td>2.0 - 3.9</td>
<td>1.04</td>
<td>300 - 4000</td>
<td>1.20</td>
</tr>
<tr>
<td>4.0 - 5.9</td>
<td>3.27</td>
<td>200 - 300</td>
<td>1.33</td>
</tr>
<tr>
<td>6.0 - 8.0</td>
<td>3.83</td>
<td>100 - 200</td>
<td>1.78</td>
</tr>
<tr>
<td></td>
<td></td>
<td>&lt; 100</td>
<td>2.13</td>
</tr>
</tbody>
</table>

Source: HUFSM (1971)
FIGURE 11 Accident rate vs curve radius relation obtained by Shrewsbury and Summer (1980).

TABLE 15 Non-Junction Injury Accident Rates on Straights and Curves

<table>
<thead>
<tr>
<th>Average curvature (degrees per mile)</th>
<th>Accidents per million vehicle-miles (and numbers of accidents)</th>
<th>STRAIGHTS and bends of radius more than 5000 ft</th>
<th>BENDS</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>radius 5000 ft-2000 ft</td>
<td>radius 2000 ft-1000 ft</td>
<td>radius less than 1000 ft</td>
</tr>
<tr>
<td>0-40</td>
<td></td>
<td>1.2 (284)</td>
<td>1.2 (33)</td>
<td>1.0 (4)</td>
</tr>
<tr>
<td>40-80</td>
<td></td>
<td>0.9 (142)</td>
<td>0.9 (37)</td>
<td>0.9 (23)</td>
</tr>
<tr>
<td>80-120</td>
<td></td>
<td>0.7 (69)</td>
<td>0.5 (11)</td>
<td>0.9 (16)</td>
</tr>
<tr>
<td>Over 120</td>
<td></td>
<td>0.4 (15)</td>
<td>0.5 (3)</td>
<td>1.0 (19)</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>1.0 (510)</td>
<td>0.9 (84)</td>
<td>1.0 (62)</td>
</tr>
</tbody>
</table>

Source: Road Research Laboratory (1965)
Hoban (1982b) also attempted to derive accident reduction factors for application to specific case study evaluations. The following accident cost reductions were estimated for four types of improvement:

- Improving isolated unsafe alignment features: 40–90%;
- Overall realignment from moderate to high standard: 10%;
- Construction of auxiliary lanes: 15–25% over the length of the improvement; and
- Widening to four lane divided standard on improved alignment: 40%.

Road safety is also related to overall alignment consistency, as well as signing and delineation. The accident reduction estimates presented in this section may be used to predict approximate accident cost reductions for project evaluation.
VI. SUMMARY OF MODELLING CAPABILITIES

6.01 The evaluation of road geometry standards requires reliable procedures for predicting changes in total transportation costs as geometric characteristics are changed. In this context the total transportation cost includes the costs of construction, maintenance, vehicle operation, travel time and accidents on a given road. This report has reviewed available information and modelling techniques for all of these costs, with particular reference to the Highway Design and Maintenance Model HDM-III developed by the World Bank (Watanatada et al. 1987). The main findings of this review are briefly summarized here.

A. Construction Costs

6.02 The costs of improving road width and alignment can vary enormously from one case to another, depending upon terrain, constraints and existing standards. At one extreme, the marginal cost of improved geometry in new construction works are sometimes quite small. At the other extreme, these improvements could require major earthworks, new bridges, and/or increased route length. The final analysis of appropriate geometric standards is quite sensitive to these costs, so that appropriate standards could vary markedly in different cost environments.

6.03 The construction cost prediction routines in the HDM-III model provide some estimates of the costs of resurfacing, widening, and complete reconstruction on a new alignment, based on the data of Markow and Aw 1983). However, the model is quite unsatisfactory for predicting costs of incremental alignment improvements, which use a combination of existing and new alignment. A simple method for overcoming this problem is to characterize each realignment proposal as being equivalent to some percentage of the cost of total reconstruction which makes no use of the existing road alignment. This procedure is still quite approximate, since there is little if any information available for estimating this percentage, or deciding the relative cost of, say, grade and curvature improvements. The procedure is incorporated into a separate program HDCOST, discussed in para. 7.09 to 7.12.

B. Maintenance Costs

6.04 Road surface standards and maintenance costs affect overall road economics and influence road geometry evaluations. Road roughness also has a direct effect on speeds and therefore influences the speed effects of road geometry changes. From the point of view of road geometry evaluations, however, the main impact on maintenance costs relates to edge damage on narrow roads.

6.05 Pavement edge loading appears to increase with the square of hourly traffic volume, and edge maintenance costs are often the major reason for the widening of narrow roads in rural areas. These costs are not currently considered by HDM-III. A simple and very approximate
procedure for estimating edge deterioration was developed in Section D of Chapter IV, for inclusion in HDM-III. The procedure is based on low-volume traffic behavior assumptions, so a conservative upper limit is placed on this effect. The coefficient may be estimated from local data and should be varied to reflect pavement strength and the support provided by the road shoulders.

C. Accident Costs

6.06 Changes in road geometry standards can have a major effect on accidents. Where existing accident rates and estimated cost data are available, these should be included in road evaluations using the accident adjustment estimates given in Chapter V.

D. Vehicle Operating Costs

6.07 A number of modelling procedures are available for predicting vehicle free speeds as a function of road geometry and roughness, and vehicle operating costs as a function of speed, roughness and other characteristics. These models have been developed from extensive research and large data bases collected in a number of countries.

6.08 Nevertheless, two general limitations should be noted. First, many of the models use simple additive linear equations, which suggest that the effects of each road characteristic are independent of those of other characteristics. This could give misleading results when used to assess incremental changes in geometric standards. Second, model parameters are often estimated using regression analysis techniques. Because road characteristics are generally correlated, this can lead to misallocation of effects between coefficients. This misallocation appears as an instability of coefficients between comparable studies, or between repeated analyses within a given study.

6.09 The use of mechanistic models for vehicle speed and performance predictions should improve their ability to reflect marginal changes, and reduce parameter instability. The use of meaningful mechanistic parameters also allows for estimation from other sources, although overall model calibration is still required. The review noted a number of relatively minor aspects of vehicle operating cost predictions which require closer scrutiny. These included the effect of overall alignment standard (or "speed environment") on desired speed and the speed adopted on individual sharp curves, and the limitations of assumed fixed engine speed in the fuel consumption predictions. The major deficiencies of the vehicle operating cost models, however, were their inadequate treatment of road width and their inability to take account of traffic interaction and congestion effects. A substantial part of this research was therefore devoted to reviewing available information and developing approximate techniques for modelling these effects.
E. Modelling Speed-Volume Relationships

6.10 The delays caused by traffic interactions on wide two-lane roads are basically related to overtaking demand and supply. While mechanistic demand/supply models show some promise for macroscopic predictions in the future, none is sufficiently developed for this purpose at the present time.

6.11 The alternative approach is to relate speed directly to traffic volume. Following a review of models currently available, a simple linear model was proposed. This makes use of free speed predictions to determine the spread of free speeds on a given road section, which affects the slope of the speed-volume relationship. The only parameter to be estimated is the capacity for a given road width. The capacity used in this model is for undisturbed traffic conditions, so that reduced values should be used for roads with significant turning movements or roadside activities. The main limitation of the model is its failure to take account of overtaking opportunities. These are very important in the evaluation of road alternatives at higher traffic volumes, but may require a non-linear model form. A second obstacle to modelling overtaking effects is the need for a simple macroscopic measure of overtaking opportunity. Because of this limitation, the model as presented here is most appropriate for low-volume road evaluations.

F. Modelling Road Width Effects

6.12 Four types of road width effects were identified in Chapter IV. The effect on edge deterioration has already been discussed here. This and the proportion of journey time spent in shoulder travel influence the "effective roughness" experienced by a vehicle, which in turn influences speed and vehicle operating costs. Width also affects the estimation of free speed and the change in speed with increasing traffic volume.

6.13 Following a review of the fairly limited information available on these relationships, four characteristics were identified which appeared to have an S-shaped transition between wide and narrow road behavior. These were free speed, capacity, crossing speed and the proportion of vehicles travelling on the road shoulder. Approximate distributions were postulated for each parameter, and these were found to be sufficiently different that they could not be combined into a single general distribution. Care was taken to ensure that width effects were not "double-counted." It was noted that road width effects may be closely related to sight distance and shoulder quality, which were not generally measured or accounted for in the studies reviewed. In HDM-III applications these factors may be accounted for by adjustments to the free speed reduction VDIFF and the shoulder/edge roughness RSE.

G. Proposed Changes to the HDM-III Model

6.14 A modified HDM-IIIa model was developed for the evaluation of road geometry standards in this study. The proposed new subroutines are
The changes affect the current HDM-III predictions in the following ways:

- The alternative model of width effect on free speed (Section A, Chapter IV) replaces the current modification of VDESIR in the Brazil user cost subroutine USEREB. Existing prediction methods are retained in the HDM-III user cost routines for other regions.

- The subroutine CONGST has been replaced with a completely new VOLUME subroutine which calculates speeds at a given traffic volume as a function of free speeds and road width (Sections C, Chapter III and Section B, Chapter IV). Because speeds affect fuel consumption and other vehicle operating costs, these are recalculated by VOLUME and tested against predicted free-flow values.

- Effective roughness is calculated in subroutine BIEFF, taking account of shoulder travel on narrow roads (Section C, Chapter IV). This is used in the calculation of vehicle operating costs, but has no effect on deterioration and maintenance calculations.

- Deterioration and maintenance predictions in subroutine RDMPV are modified to include extra pothole development due to edge crossings, as described in Section D, Chapter IV. This influences road roughness and maintenance requirements, but does not directly interact with other road deterioration predictions.

Details of the program modifications are given in Appendix B. In the short time frame of the current study, these changes have been arranged so as to minimize the interference with existing HDM-III computations, and to minimize programming changes. Appendix B discusses more elegant and efficient ways to incorporate some of these modifications.

H. Parameter Estimation Requirements

The new features introduced in HDM-IIIa include a number of additional parameters to take account of traffic interaction and road width effects. While recommended default values have been derived in this report, it would be preferable to obtain local estimates of these parameters wherever possible.

Free Speed vs. Width

The HDM-III model recognizes that desired speeds are substantially lower on one-lane roads than on two-lane roads. However, if incremental changes in road width are being considered, it is important to determine the transition between wide and narrow roads. There is little information available on this, and a possible form of this transition is postulated for the parameter VFFACT (see para. 4.25 - 4.26). If field studies are under-
taken to estimate this parameter, attention should also be given to previously unmeasured road characteristics, in particular the sight distance and shoulder quality in narrow roads.

**Speed-Volume Relationship**

6.18 The simple linear model (para. 3.55 to 3.56) depends primarily on the free speeds predicted for a given road section, and the nominal capacity QCAP. Traffic composition effects are incorporated through a set of passenger car equivalents (PCE's) for each vehicle type. QCAP may be estimated from the slope of the observed speed-flow relationship. This will clearly vary with road width, and a parameter WCFACT was derived (para. 4.39 to 4.42) to take account of this effect. The default value of QCAP is:

\[
Q_{\text{CAP}} = 2500 \times \text{WCFACT}
\]

for a given road width, and this may be tested against field data.

6.19 On some roads the 'friction' caused by roadside disturbances tends to reduce speeds below that of the lowest free speed. Two approaches are available for modelling this effect:

- If roadside disturbances are present at all volumes, then the minimum speed used for speed-flow predictions may be reduced by the proportion XFRI, which takes a value of 1.0 on undisturbed roads, and would need to be estimated from local data for varying degrees of roadside friction.

- If low speeds are only observed at high volumes, then the three-zone linear model described in para. 3.57 - 3.61 may be used. This allows for separate zones of 'traffic interaction' without external disturbances and 'congestion' where speeds tend down to a low 'jam' value of 10-20 km/h. However very little is known about speed-flow relationships in this region, and accurate estimation of the parameters would be difficult.

6.20 The three-zone linear model also offers the capability of a low-volume 'plateau' where no speed-flow effects exist. However there is little evidence that such a zone exists, and the savings in computation are small, so its use is not recommended. If local values of passenger car equivalents (PCE's) are to be substituted for the default values in subroutine VOLUME, care should be taken to ensure that they reflect only the non-speed aspects of vehicle equivalence, as discussed in para. 3.32 to 3.34. The speed-flow model also requires a flow-frequency distribution FFREQ to convert AADT to a range of hourly flows over a year. Default values (see para. 3.41 and para. 3.62 - 3.65) could be replaced by local data. This parameter is required in subroutine VOLUME, and also in subroutines BIEFF and RDMPVE which deal with effective roughness and edge damage, respectively.
Effective Roughness

6.21 On narrow roads, crossing and overtaking maneuvers may involve travel on the road shoulder, so that the roughness experienced by a vehicle is not just that of the road surface. The 'effective roughness' depends on three factors:

° the frequency and duration of crossing and overtaking maneuvers; these are discussed in Chapter IV, and estimates are derived for low-volume conditions.

° the probability of shoulder travel (PSH) as a function of road width; a distribution of PSH is postulated in Chapter IV, Section C and some simplifying assumptions are made with regard to different vehicle types.

° edge and shoulder roughness (RSE); while this could possibly be measured by conventional means, it will generally be estimated relative to the surface roughness RP. (A default value of RSE = 2 x RP is assumed here).

Values of RSE should be estimated for each section of existing and improved roads. More detailed research is necessary to estimate values for PSH and to validate the crossing and overtaking time models proposed here.

Edge Deterioration

6.22 The modelling of edge damage on narrow roads makes use of the edge crossing frequencies and probabilities discussed in the previous section. Only one additional parameter is required. This is known as the edge pothole volume (EPV), expressed as cubic meters of patching per million edge crossings. A default value of 30.0 was derived in Chapter IV, Section D. This should vary with the strength of the pavement and shoulder material.

6.23 Edge damage and maintenance predictions are also affected by a number of constraints assumed in the model formulation, regarding timing of edge damage after resurfacing, and maximum limit in a given period. The model could be improved by incorporating closer links with maintenance strategies and pavement condition. Research is required to estimate EPV for various road types, and to validate and refine the form of this model component.
VII. EVALUATION PROCEDURE

7.01 In most applications of the HDM model, the existing road condition or 'base case' is known, and various alternative maintenance and improvements strategies are evaluated. The current study, however, has a broader perspective with the objective of finding appropriate improvement strategies for a wide range of initial conditions, traffic volumes and construction cost levels.

7.02 It was therefore decided to use the HDM model to calculate "Total Operating Cost" for each road standard, without any reference to construction costs or the base case from which an improvement might have been provided. Total operating cost is defined here as the total cost of vehicle operation, travel time, road maintenance, accidents and other costs of road use, but excluding construction cost.

7.03 Construction costs are calculated separately outside the HDM program, for any improvement from a specified base width and alignment to a better geometric standard. The HDM results can then be used to evaluate any type of road geometry improvement, for a variety of base cases and different construction cost levels.

A. HDM Analysis Framework

7.04 The basic framework for HDM runs in this study is as follows. First, the base case is taken as a single road link with severe horizontal and vertical geometry, having a number of sections with different widths. A typical 'poor' pavement condition and 'standard' maintenance policy for that country or region are assumed, and considered to be the same for all terrains, traffic volumes, base cases and improvement options under study. While this might be unrealistic, it is convenient for the purpose of isolating geometric effects.

7.05 A range of alternative construction options are then considered, all assumed to be completed in the first year of the evaluation period. These include resurfacing only, and various combinations of vertical and horizontal geometry improvement, as shown in Table 16. All geometric improvements are assumed to be accompanied by resurfacing of the full road length, and all improvements are constructed to the same width as the existing road section. Traffic composition, vehicle characteristics and cost parameters in the HDM data were chosen to suit the country or region under study. While maintenance costs were included in this analysis procedure, the construction costs for pavement resurfacing and geometric improvements were all set to zero. The HDM model thus calculated the total operating cost (total cost excluding construction) for each option, at a range of traffic volumes.

7.06 Series K of the HDM input data specifies the combinations of maintenance, construction and traffic characteristics to be evaluated. For each road link and traffic flow, ten construction options are considered, including standard maintenance in the years after construction. The "do-nothing" option, i.e., standard maintenance only, is also considered.
TABLE 16 Conditions Simulated in HDM Runs

A. EXISTING ROAD CONDITIONS

- Poor alignment: rise and fall RF = 80 m/km;
  curvature CV = 500 deg/km
- Four links of length L = 10 km
- Seal widths W = 7.4, 6.5, 5.5, 3.8 m
- Typical Shoulder width*
- Typical 'poor' pavement characteristics*

B. MAINTENANCE

- Typical 'standard' maintenance*

C. CONSTRUCTION (same width as existing road link)

1. Repave only: RF = 80 CV = 500
2. Realign and repave to:
   RF = 80 CV = 300
   RF = 80 CV = 120
4. " " " RF = 50 CV = 300
5. " " " RF = 50 CV = 120
6. " " " RF = 50 CV = 50
7. " " " RF = 20 CV = 300
8. " " " RF = 20 CV = 120
9. " " " RF = 20 CV = 50
10. " " " RF = 10 CV = 15

D. VEHICLE CHARACTERISTICS

- Typical local values*

E. TRAFFIC CHARACTERISTICS

- Typical mix of vehicle types*
  Annual traffic volumes ranging from 100 to 3000 vpd.

* These parameters are selected as typical for a given country, region and type of road, and are assumed constant for all cases in a particular study.

7.07 It is useful to consider the road curvature and rise and fall in Table 16 in terms of more familiar road characteristics such as grade, curve radius and design speed. While there is no exact relationship between these parameters, typical comparisons are shown in Table 17. For convenience, it is assumed in these comparisons that steep grades and sharp curves represent 80-100 percent of road length in difficult terrain and 50-60 percent of length in moderate terrain. The table shows that the options range from a low-speed winding road in very steep terrain to a fairly straight and level road with a much higher speed standard. This broad range should be kept in mind in considering the evaluation of alternative geometric standards in the following chapters.
TABLE 17  HDM Road Alignment Measures Related to Typical Design Characteristics

<table>
<thead>
<tr>
<th>HDM ALIGNMENT OPTIONS</th>
<th>TYPICAL DESIGN CHARACTERISTICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rise &amp; Fall RF (m/km)</td>
<td>Curvature CV (deg/km)</td>
</tr>
<tr>
<td>1 80</td>
<td>500</td>
</tr>
<tr>
<td>2 80</td>
<td>300</td>
</tr>
<tr>
<td>3 80</td>
<td>120</td>
</tr>
<tr>
<td>4 50</td>
<td>300</td>
</tr>
<tr>
<td>5 50</td>
<td>120</td>
</tr>
<tr>
<td>6 50</td>
<td>50</td>
</tr>
<tr>
<td>7 20</td>
<td>300</td>
</tr>
<tr>
<td>8 20</td>
<td>120</td>
</tr>
<tr>
<td>9 20</td>
<td>50</td>
</tr>
<tr>
<td>10 10</td>
<td>15</td>
</tr>
</tbody>
</table>

B. Construction Cost Estimation

7.08 A major difficulty in a theoretical study of this type is the estimation of construction costs. Costs can vary a great deal between different countries, terrains and types of road construction, so that the selection of appropriate design standards should also vary. Of particular importance are the marginal costs of small changes in road width and alignment, and the relative costs of different levels of geometric improvement on an existing road. Since little information is available on these differences, it was necessary to make a number of assumptions which may affect the resultant guidelines on geometric standards. Some sensitivity analyses are presented to show the effects of higher and lower cost assumptions.

The HDCOST Program

7.09 A small computer program HDCOST was written to calculate construction costs and economic evaluation measures for a set of alternative road options. The program has three main components:

- Data on unit costs, base cases, construction options, traffic volumes and HDM 'total operating costs' are specified by the user.

- Construction costs are calculated using some of the equations contained within the HDM-III model, for site clearance, earth-
works, pavement layers, per kilometer costs and overhead. Some variations from the HDM calculations are described later in this section.

For each base condition, the relevant improvement options, including 'standard maintenance only,' are evaluated in terms of Net Present Value and Benefit-Cost Ratio. The program then selects the option with the highest Net Present Value for each base case and traffic volume.

7.10 In the calculation of construction costs, the main difference from the HDM-III procedure is the concept of a "Percent Reconstructed" (XPR) in the HDCOST program. The current HDM-III cost equations implicitly assume that any alignment improvement project is constructed completely from the natural ground surface. In practice this is rarely the case, and most realignment projects can make some use of existing road sections. Each improvement in HDCOST is therefore treated as having two elements:

- XPR percent of total length constructed from the natural ground surface; and
- (100-XPR) percent of length which requires only resurfacing over the old road.

7.11 Of course the true project will have many sections which fall between these two extremes, so that the value chosen for XPR is notional rather than exact. The HDCOST program calculates all earthworks, clearance, base and sub-base and per kilometer costs only over this percentage of the total length. Values of XPR could range from zero percent for a "resurface-only" project to 100 percent for a major deviation. These must be estimated by the user of HDCOST and included in the input data.

7.12 Two other assumptions in the HDCOST program relate to the effect of road width on construction costs:

- Widening unit costs are arbitrarily assumed to be 20 percent higher than those for complete construction, due to the difficulties of working with small quantities along the roadside.
- Per kilometer costs for narrow roads are assumed not to reduce proportionally with width, but to reduce in proportion to half of the width reduction from 7.4 m. The average drainage cost for a 6.0 m road, for example, is taken as 90 percent of that for a 7.4 m in road, rather than the proportional reduction to about 80 percent.

A full listing of the HDCOST program is provided in Appendix C of this report.
VIII. CASE STUDY - COSTA RICA

8.01 The evaluation procedure described in the previous chapter was used in a case study of hypothetical road improvements for Costa Rica.

A. Road and Traffic Conditions

8.02 The road and traffic characteristics for this analysis were largely drawn from a previous World Bank study (Bhandari 1984). Samples of the HDM input files for this study are presented in Appendix D. The existing road sections were assumed to be surface treated with 1.0 m shoulders and a roughness of 55 QI. Improved sections also had 1.0 m shoulders and surface treatment, with a roughness of 45 QI. A uniform maintenance policy was adopted, calling for 100 percent patching of severely damaged areas plus resealing when damaged area exceeds 20 percent. From the vehicle and traffic characteristics in Appendix B, it can be seen that the traffic composition included 60 percent cars, and 40 percent buses and trucks.

B. Analysis of Total Operating Costs Using HDM-IIIa

8.03 With these input data, the HDM-IIIa model (incorporating the features described in Chapter III; Section G) was used to analyze a range of road geometry standards, as described in Chapter VI; Section A and Table 16. Since no construction costs were specified, these produced estimates for Total Operating Cost (vehicle operation, travel time and road maintenance) for each road standard and traffic volume.

8.04 Typical results for 300 vehicles per day (vpd) average daily traffic (ADT) are presented in Figure 12. The figure shows that total operating costs are most strongly affected by road rise and fall (RF), with large jumps in cost between RF values of 80, 50 and 20 m/km. By comparison, the effects of road width and curvature are quite small. Similar results are presented in Figure 13 for a traffic volume of 3000 vpd, showing larger variations in operating cost with road width. It is interesting to note in both figures that the effects of all three geometric parameters tend to reduce as they approach a high standard of road geometry.

8.05 The major element in all of these estimates is vehicle operating cost, which represents 90-96 percent of Total Operating Cost, depending on road width and traffic volume. The remainder is made up of road maintenance and travel time costs, with the latter representing an increasing component as volume increases. The relative magnitudes of these costs are very much dependent on the assumed unit costs provided as input to the HDM model, and could vary from one case to another.
FIGURE 12 Costa Rica - Total Operating Costs at 300 vehicles per day

FIGURE 13 Costa Rica - Total Operating Costs at 3000 vehicles per day
C. Construction Cost Estimation

8.06 Unit costs for road construction were estimated from information provided by the U.S. Federal Highway Administration, for 16 farm-to-market road projects in Costa Rica. For the purposes of this study, costs were grouped into five categories, as shown in Table 18.

**TABLE 18 Unit Construction Costs - Costa Rica (1984).**

<table>
<thead>
<tr>
<th>Item</th>
<th>Typical cost per km (Million CR Colones)</th>
<th>Unit Cost (CR Colones)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthworks</td>
<td>.07 - .47</td>
<td>122/m³</td>
</tr>
<tr>
<td>Sub-base</td>
<td>.28 - 1.4</td>
<td>348/m³</td>
</tr>
<tr>
<td>Base</td>
<td>1.0 - 1.7</td>
<td>1225/m³</td>
</tr>
<tr>
<td>Surface</td>
<td>.43 - .65</td>
<td>2935/m³</td>
</tr>
<tr>
<td>Other</td>
<td>.13 - 1.3</td>
<td>0.5/km</td>
</tr>
</tbody>
</table>

8.07 The roads considered here generally had cement-stabilized base and asphalt surface treatment. The "other" category in Table 18 includes drainage, structures, site clearance, guard rail and signs and markings, as well as other minor items. The magnitude of these individual costs varies considerably from one project to another, and even the total "other" cost per kilometer varies by a factor of 10 across the sixteen projects considered. A foreign exchange rate of 45 Costa Rica Colones per U.S. Dollar was used.

D. Evaluation of Geometric Standards

8.08 The results of the HDM-IIIa analysis were then combined with unit cost information and provided as input to the HDCOST program Chapter VII; Section B. This program calculates the change in total operating cost which results from an improvement from one geometric standard to another, and compares this with the estimated construction cost for this improvement. This analysis yields a Net Present Value (NPV) for each improvement option at a given traffic volume, and the option with the highest NPV may be considered as the optimal design standard. Supplementary information on marginal benefit-cost ratios could be used to select improvements under constrained budget conditions, but this was not done in the current study. Results of this type were obtained for a total of 210 combinations of road alignment and width, for five levels of average daily traffic volume (ADT). Typical examples are presented in Figures 14 and 15.
8.09 Figure 14 gives NPVs for ten alignment options, where the existing base alignment is in mountainous terrain and both base and improved road widths are 6.0 m. For these conditions, optimal improvements are given by the highest NPV at each traffic volume in vehicles per day (vpd) as follows:

- **100 vpd**: do nothing
- **300-1000 vpd**: modest realignment to Option 4 (RF=50, CV=300);
- **3000 vpd**: major realignment to Option 8 (RF=20, CV=50).

8.10 Figure 15 presents a second example representing a base condition of hilly terrain and narrow pavement, and an improved road width of 7.4 m. Since the base alignment for this case is Option 4, only alignments equal to or better than this standard are considered.

8.11 In a similar way, optimum improvement standards can be derived for each base road alignment and width. These are shown in Table 19, and may be summarized as follows:

- **100 vpd**: No improvement is warranted for any base road condition.
- **300 vpd**: Some minor widening and realignment is warranted on roads of 6.0 m width or less.
- **500 vpd**: Minor widening or realignment is warranted in most cases. Alignment 4 (RF=50) is the optimum standard except where the existing alignment is better.
- **1000 vpd**: Modest widening and realignment warranted for all but the highest base condition. Steep alignments (RF=50 and 80) should be regraded to the next standard (RF=20 and 50 respectively).
- **3000 vpd**: All roads should be widened to 7.4 m and realigned to a high standard of vertical and horizontal geometry.
FIGURE 14 Costa Rica - Net Present Value of Selected Improvement Options in Mountainous Terrain; 60 m width before and after.

FIGURE 15 Costa Rica - Net Present Value of Selected Improvement Options in Hilly Terrain; 3.8 m width before to 7.4 m after.
TABLE 19 Costa Rica Case Study: Optimum Road Geometry Improvements

<table>
<thead>
<tr>
<th>BASE</th>
<th>OPTIMAL IMPROVEMENT AT</th>
<th>W GEOM</th>
<th>100 VEH/D</th>
<th>300 VEH/D</th>
<th>500 VEH/D</th>
<th>1000 VEH/D</th>
<th>3000 VEH/D</th>
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<td>W G NPV</td>
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</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS

E. Sensitivity to Construction Cost Estimates

8.12 The construction costs estimated by the HDCOST model in this analysis varied from $27,000 to $790,000 per kilometre. Because the program considered some very extreme terrain conditions, these costs were in some cases many times higher than those reported in para. 8.06. In practice, construction costs may vary considerably from one case to another, and so it is useful to evaluate higher and lower cost conditions.

8.13 Table 20 shows the optimum road geometry improvements when construction costs are halved. The results are similar to those of Table 19, except that more alignment improvements are called for at 300-1000 vpd, and more widening is appropriate at 1000 vpd. The optimal alignment at 1000 vpd in Table 20, for example, is 7 or 8 (RF=20, CV=300 or 120) regardless of base conditions, whereas many cases in Table 19 had an optimal alignment of 4 or 5 (RF=50, CV=300 or 120). At 3000 vpd, the optimal improvements in Table 20 are the same as those in Table 19, but yield higher NPVs.

8.14 The effects of increasing construction costs by 50 percent above the original estimates are shown in Table 21. This suggests that no alignment improvements are justified at 100 and 300 vpd in very difficult terrain, and even at 500 and 1000 vpd in severe terrain. This is slightly
offset by more widening in some cases. The variations in Tables 20 and 21 show that optimum road geometry standards are sensitive to construction costs, but a similar trend in recommended standards is evident across all three tables.

F. Comparison with the Standard HDM-III Model

8.15 All of the evaluations presented so far in this chapter are based on the HDM-IIIa model. This model has been modified from the standard HDM-III to take account of the effects of traffic volume and road width. However, the modifications are at this stage only theoretical and approximate, and have not yet been validated against observed conditions. It is therefore useful to compare the HDM-IIIa predictions with those of the standard HDM-III model.

8.16 Table 22 compares Total Operating Costs as predicted by both models, at 300 and 3000 veh/d. The values for HDM-IIIa are the same as those plotted in Figure 12 and 13. The table shows that at 300 veh/d there is very little difference between the two models, and that, as shown in Figure 12, the effect of pavement width is small. At 3000 veh/d, a small difference is evident for the 7.4 m road width, but a much larger difference arises for narrow roads.

### Table 20 Costa Rica Case Study: Optimum Geometry Improvements When Construction Costs are Halved

<table>
<thead>
<tr>
<th>BASE</th>
<th>OPTIMAL IMPROVEMENT AT 100 VEH/D</th>
<th>OPTIMAL IMPROVEMENT AT 300 VEH/D</th>
<th>OPTIMAL IMPROVEMENT AT 500 VEH/D</th>
<th>OPTIMAL IMPROVEMENT AT 1000 VEH/D</th>
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</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS
TABLE 21 Costa Rica Case Study: Optimum Geometry Improvements When Construction Costs Increase by 50 Percent

<table>
<thead>
<tr>
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<th>OPTIMAL IMPROVEMENT AT</th>
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</thead>
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<td>BASE</td>
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<td>W G</td>
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</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS

TABLE 22 Costa Rica: Total Operating Costs Predicted by HDM-IIia and HDM-III

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<tr>
<th>ADT</th>
<th>300 vpd</th>
<th>3000 vpd</th>
</tr>
</thead>
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<tr>
<td>Width</td>
<td>3.8 m</td>
<td>7.4 m</td>
</tr>
<tr>
<td>Road Alignment Option</td>
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<td>B^b/</td>
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<tr>
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<tr>
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<td>.34</td>
</tr>
<tr>
<td>10</td>
<td>.35</td>
<td>.34</td>
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</tbody>
</table>

^a/ Modified HDM-IIia model
^b/ Standard HDM-III model
Testing Traffic Volume Effects

8.17 The comparisons for the 7.4 m road surface width demonstrate the effects of the VOLUME subroutine in HDM-IIIa. The only difference between the standard and modified HDM runs for wide roads is the inclusion of a traffic volume effect on speeds. This subroutine reduces vehicle speeds as traffic volume increases, and the reduction depends upon the spread of free speeds on the particular road section under study. Speed reductions in turn affect the calculation of fuel consumption and other operating costs.

8.18 At the highest traffic volume of 3000 vpd, the total operating costs predicted by HDM-IIIa in Table 22 are only about one percent higher than those predicted by the standard HDM-III model, for wide roads. Differences are even smaller at lower traffic volumes. This suggests that traffic interaction effects are almost negligible on wide roads over the full range of traffic flows considered in this study.

8.19 There are two factors which contribute to this result. First, the assumed value of time savings in this study is quite low, so that vehicle operating costs represent 90-96 percent of total operating costs. These are not particularly sensitive to speed over the range of conditions considered here. The second factor to be considered is that the VOLUME subroutine uses a weighted average of typical hourly traffic flows over a whole year, so that effects at peak hours are 'diluted' by off-peak operating conditions. The flow-frequency distribution used in this study is equivalent to an hourly traffic flow of 5.4 percent of the average daily traffic (ADT) volume, or 163 veh/h at 3000 veh/h. This leads to only a small reduction in average speed under most conditions.

Testing Road Width Effects

8.20 While the HDM-III includes a small effect of road width on vehicle speeds, the modified HDM-IIIa uses several different mechanisms to estimate width effects. These include some quite approximate predictions of the effects of road width on total operating costs. The results of these modifications can be examined in Table 22, using the 3.8 m road surface width. At 300 vpd, the HDM-IIIa model predicts total operating costs of the order of 2.5 percent higher than for the HDM-III model. At 3000 vpd, this increases to about 12-18 percent, due to increased edge crossings caused by passing and crossing maneuvers. This results in increased maintenance, travel time, and vehicle operating costs.

8.21 The HDM-IIIa model thus introduces a small road width effect at 300 vpd, which rises to a significant increase in total operating costs at 3000 vpd. A more detailed examination of the HDM outputs shows how this cost is divided between maintenance, time and vehicle operating costs, and how it varies over a range of road widths and traffic flows.

8.22 The magnitude of these road width effects is highly approximate at this stage, and empirical studies are needed to verify and calibrate the component models. The different total operating costs produced by the
standard HDM-III model were used with the HDCOST program to give alternative estimates of optimum road geometry standards for this case study. The results are presented in Table 23.

8.23 Comparing this table with Table 19, it can be seen that the estimates of optimum alignment standard are almost identical for the two models, but the standard HDM-III model calls for very little road widening at any traffic volume. The HDM-III model also yields lower net present values for the optimum improvements in all cases, and especially on narrow roads. This comparison highlights the observation made in Chapter IV that the standard HDM-III model does not adequately account for the effects of road width, and is not particularly sensitive to changes in width.

TABLE 23 Costa Rica Case Study: Optimum Road Geometry Improvements Predicted by the Standard HDM-III Model

<table>
<thead>
<tr>
<th>BASE W GEOM</th>
<th>100 VEH/D W G NPV</th>
<th>300 VEH/D W G NPV</th>
<th>500 VEH/D W G NPV</th>
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<td>3.8 7. 0.</td>
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<td>3.8 7. 0.</td>
</tr>
</tbody>
</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS

G. Case Study Conclusions

8.24 The major result of this case study is the set of optimum road width and alignment standards presented in Table 19. For the particular road, traffic and construction cost conditions investigated here, it shows that progressively higher geometric standards are warranted as average.
daily traffic (ADT) volumes increase from 100 to 3000 vpd. Briefly, the table shows that no improvements are warranted in this study at 100 vpd, while a high standard wide road is warranted at 3000 veh/d for all base conditions. Between these two extremes, increasing degrees of widening and realignment are called for. The case study included a sensitivity test of very wide variables in construction cost, from 0.5 to 1.5 times the estimated values. Optimal road geometry standards were sensitive to these changes, as expected, indicating that optimal road width and alignment will vary for road projects with different cost conditions.

8.25 The modified HDM-IIIa model was also compared with the standard HDM-III model, to assess the effects of additional traffic volume and width components developed in this report. The comparison showed that the traffic interaction effects in this case study were quite small at volumes up to 3000 vpd. This is partly because high hourly traffic flow effects are averaged out with many hours of lower flow, and partly because of the low values of travel time assumed in this study. The comparison also showed a much larger difference in road width effects between the two models, especially at higher traffic flows. While the effects included in the modified HDM-IIIa model are rather approximate, the derived optimum road width standards in Table 19 appear much more realistic than those determined in Table 23 using the standard HDM-III model.
IX. CASE STUDY - INDIA

9.01 A second case study was conducted to investigate alternative road width and alignment standards for some hypothetical road projects in India. The study followed the procedure described in Chapter 7, using the same road widths, alignments and traffic volumes as in the Costa Rica case study presented in the previous chapter.

9.02 An important difference in this case study is the use of the India road user cost relationship in the HDM model. These use linear additive equations to relate free speeds to road characteristics, instead of the minimum limiting speed approach. The linear additive form of the India equations could have a strong effect on the analysis of small changes in geometric standards. The equations will predict, for example, that a width increase from 4.0 to 5.0 m will have the same effect as an increase from 6.5 to 7.5 m.

9.03 The HDM-IIIa model allows the selection of several alternative forms of vehicle operating cost equations, as does the standard HDM-III model. However in the modified model, the free speed-width adjustments are not utilized with the India equations, since the latter already include a free speed-width effect. All of the other road width and traffic volume effects in the HDM-IIIa model are still utilized with the India equations.

A. Road and Traffic Conditions

9.04 Samples of the HDM input files for this case study are presented in Appendix E. All road sections were assumed to be surface treated, with a shoulder width of 2.5 m. Existing road sections had a roughness of 3200 BI (58 QI) and improved sections had 2500 BI (45 QI). Existing and improved strength parameters SN were 3.3 and 3.5 respectively.

9.05 The road maintenance policy consisted of routine maintenance with patching of 80 percent of the severely damaged area, plus resealing when the damaged area exceeded 70 percent, though not more often than every 7 years or less often than every 12 years. Some problems were encountered in selecting pavement strengths and maintenance policies for the wide range of traffic volumes considered in this study. Since the case study is not concerned with the detail of pavement deterioration and maintenance, it would be desirable to assume a fixed set of conditions for all traffic volumes. However the combination of weak pavements and high axle loads found in India quickly lead to excessive deterioration at high volumes, unless stronger pavements are provided.

9.06 As a simple expedient in this study, the axle loads for trucks were set to zero, so that road pavement deterioration was not fully accounted for. Since the study is concerned with differences between alternative geometric standards, the implicit assumption is that construction and maintenance costs due to pavement deterioration will be similar for different widths and alignment at the same traffic volume. Clearly this is not entirely true, at least for road width, but the effect is assumed to be small relative to other geometric effects on total costs.
The traffic composition used in the India case study consisted of 60 percent trucks, with 20 percent cars and 20 percent buses. This is very different from the 60 percent car composition used in the Costa Rica case study.

B. Analysis of Total Operating Costs Using HDM-IIIa

9.07 Following the procedure of Chapters VII and VIII, the HDM-IIIa model was used to estimate total operating cost (vehicle operation, travel time, road maintenance, but not construction) for a range of road geometry standards and traffic volumes.

9.08 Figures 16 and 17 present typical results for 300 and 3000 vpd, respectively. These show a similar pattern to the Costa Rica results in Figures 12 and 13, but several differences may be noted:

- Total operating costs for India are substantially lower than those for Costa Rica for all road conditions and traffic volumes.
- At low flows, the India results indicate a much greater sensitivity to road width and a much smaller sensitivity to road rise and fall, compared with the Costa Rica relationships.
- At 3000 vpd, the India results are relatively insensitive to rise and fall on wide roads, but the effect of rise and fall increases as road width decreases.

As in the Costa Rica case study, the India results are dominated by vehicle operating costs.

C. Construction Cost Estimation

9.09 Unit costs for road construction were drawn largely from data published by the Indian Ministry of Shipping and Transport (MOST 1984). These are presented in Table 24. A foreign exchange rate of 11.4 Indian Rupees per U.S. Dollars was used.

D. Evaluation of Geometric Standards

9.10 The HDCOST program (para. 7.12 - 7.15) was used to investigate a wide range of road geometry improvements for this case study. This program calculates the construction cost for a given improvement based on the initial and final road geometry characteristics, and compares this with the change in total operating costs between the two cases. The result is an estimated Net Present Value (NPV) for each option, for a given traffic volume.

9.11 Figures 18 and 19 show typical results for selected conditions, using the same examples as in the previous case study. The option with the highest NPV is highlighted at each traffic volume. The relationships for
India are of a very similar order to those for Costa Rica (Figures 14 and 15), except that there are many more positive results for the India case study at low and moderate traffic volumes. As a result, a higher geometric standard is recommended for India in most cases.

300 veh/day

![Graph showing total operating costs at 300 veh/day for India.]

Figure 16 India - Total Operating Costs at 300 veh/d.

3000 veh/day

![Graph showing total operating costs at 3000 veh/day for India.]

Figure 17 India - Total Operating Costs at 3000 veh/d.

<table>
<thead>
<tr>
<th>Item</th>
<th>Unit Cost (Indian Rupees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Right of way</td>
<td>180,000 /km</td>
</tr>
<tr>
<td>Site preparation</td>
<td>1.0 /m²</td>
</tr>
<tr>
<td>Earthwork</td>
<td>6.0 /m³</td>
</tr>
<tr>
<td>Sub-base</td>
<td>26.0 /m³</td>
</tr>
<tr>
<td>Base</td>
<td>65.0 /m³</td>
</tr>
<tr>
<td>Surface treatment</td>
<td>690.0 /m³</td>
</tr>
<tr>
<td>Bituminous macadam</td>
<td>625.0 /m³</td>
</tr>
<tr>
<td>Asphaltic concrete</td>
<td>960.0 /m³</td>
</tr>
<tr>
<td>Culverts</td>
<td>55,000 /km</td>
</tr>
<tr>
<td>Bridges</td>
<td>160,000 /km</td>
</tr>
<tr>
<td>Other construction costs</td>
<td>60,000 /km</td>
</tr>
</tbody>
</table>

9.12 Results of this type can be used to select optimal improvement standards for a range of base road alignments and widths. These are shown in Table 25. The optimal improvements may be briefly summarized as:

- **100 vpd**: No alignment improvements warranted but all roads should be widened to 7.4 m.

- **300 vpd**: For level to hilly base conditions, results are the same as for 100 vpd. For mountainous terrain, however, it becomes preferable to improve road alignment to the standard of option 5 (RF=50, CV=120), generally without widening.

- **500 vpd**: Widening to 7.4 is recommended for all cases. Mountainous and hilly alignments should generally be improved substantially to the standard of options 5 (RF=50, CV=120) and 8 (RF=20, CV=120) respectively.

- **1000 vpd**: All cases call for widening to 7.4 m and substantial realignment to a high geometric standard.

- **3000 vpd**: Maximum widening and realignment is appropriate for all base road conditions.

9.13 The results in Table 25 show only the road options with the highest NPV for each base road condition. They do not indicate whether alternative alignments are almost as good, or much worse. Some of the relationships in Figures 18 and 19, for example, are quite flat, suggesting that differences between alternative road geometry improvements are small. This information can only be obtained from a more detailed examination of the case study results.
Figure 18 India - Net Present Values for Selected Improvement Options in Mountainous Terrain; 6.0 m width before and after

Figure 19 India - Net Present Values for Selected Improvement Options in Hilly Terrain; 3.8 m width before to 7.4 m width after.
TABLE 25 India Case Study: Optimum Road Geometry Improvements

<table>
<thead>
<tr>
<th>BASE W GEOM</th>
<th>OPTIMAL IMPROVEMENT AT 100 VEH/D W G NPV</th>
<th>OPTIMAL IMPROVEMENT AT 300 VEH/D W G NPV</th>
<th>OPTIMAL IMPROVEMENT AT 500 VEH/D W G NPV</th>
<th>OPTIMAL IMPROVEMENT AT 1000 VEH/D W G NPV</th>
<th>OPTIMAL IMPROVEMENT AT 3000 VEH/D W G NPV</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.4 1.</td>
<td>7.4 1. 0.</td>
<td>7.4 5. 32.</td>
<td>7.4 5. 91.</td>
<td>7.4 8. 278.</td>
<td>7.4 10. 1175.</td>
</tr>
<tr>
<td>7.4 4.</td>
<td>7.4 4. 0.</td>
<td>7.4 4. 0.</td>
<td>7.4 8. 13.</td>
<td>7.4 10. 98.</td>
<td>7.4 10. 499.</td>
</tr>
<tr>
<td>7.4 7.</td>
<td>7.4 7. 0.</td>
<td>7.4 7. 0.</td>
<td>7.4 7. 0.</td>
<td>7.4 10. 26.</td>
<td>7.4 10. 180.</td>
</tr>
<tr>
<td>6.0 1.</td>
<td>7.4 1. 4.</td>
<td>7.4 5. 74.</td>
<td>7.4 5. 122.</td>
<td>7.4 8. 369.</td>
<td>7.4 10. 1634.</td>
</tr>
<tr>
<td>6.0 4.</td>
<td>7.4 4. 2.</td>
<td>7.4 4. 22.</td>
<td>7.4 8. 26.</td>
<td>7.4 10. 150.</td>
<td>7.4 10. 760.</td>
</tr>
<tr>
<td>6.0 7.</td>
<td>7.4 7. 1.</td>
<td>7.4 7. 7.</td>
<td>7.4 7. 14.</td>
<td>7.4 10. 62.</td>
<td>7.4 10. 372.</td>
</tr>
<tr>
<td>5.0 1.</td>
<td>7.4 1. 8.</td>
<td>5.0 5. 59.</td>
<td>7.4 5. 138.</td>
<td>7.4 8. 432.</td>
<td>7.4 10. 1908.</td>
</tr>
<tr>
<td>5.0 4.</td>
<td>7.4 4. 3.</td>
<td>7.4 4. 18.</td>
<td>7.4 8. 36.</td>
<td>7.4 10. 184.</td>
<td>7.4 10. 906.</td>
</tr>
<tr>
<td>5.0 7.</td>
<td>7.4 7. 2.</td>
<td>7.4 7. 13.</td>
<td>7.4 7. 26.</td>
<td>7.4 10. 86.</td>
<td>7.4 10. 478.</td>
</tr>
<tr>
<td>3.8 1.</td>
<td>7.4 1. 14.</td>
<td>3.8 5. 53.</td>
<td>7.4 5. 177.</td>
<td>7.4 8. 541.</td>
<td>7.4 10. 2463.</td>
</tr>
<tr>
<td>3.8 4.</td>
<td>7.4 4. 6.</td>
<td>7.4 4. 19.</td>
<td>7.4 4. 61.</td>
<td>7.4 10. 244.</td>
<td>7.4 10. 1218.</td>
</tr>
<tr>
<td>3.8 7.</td>
<td>7.4 7. 3.</td>
<td>7.4 7. 22.</td>
<td>7.4 7. 45.</td>
<td>7.4 10. 130.</td>
<td>7.4 10. 702.</td>
</tr>
</tbody>
</table>

W=PAVEMENT WIDTH (M); G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS

E. Sensitivity to Construction Cost Estimates

9.14 Tables 26 and 27 illustrate the effects of varying construction cost estimates in this analysis. Table 26 presents the case where construction costs are halved. This leads to recommendations for more road geometry improvements at lower traffic volumes. Even at 100 vpd, the analysis suggests realignment of the worse base conditions and widening to 7.4 in many cases. Major widening and realignment are recommended at daily volumes as low as 300 to 500 vpd. When construction costs are increased to 1.5 times the assumed values, the optimal geometric standards are as shown in Table 27. This scenario still calls for major widening at volumes as low as 100 vpd, but recommended alignment improvements at 300 to 1000 vpd are somewhat less than those indicated in Table 25.

F. Comparison with the Standard HDM-III Model

9.15 Table 28 compares some total operating costs predicted by the HDM-IIIa model with those predicted by the standard HDM-III model, using the India VOC relationships. At 300 vpd, the estimates provided by the two models are almost identical on both wide and narrow roads. At 3000 vpd, there is very little difference between the two models for wide roads, except for some small differences in options 1 to 3 (RF=80). For narrow roads, however, the differences are quite substantial at 3000 vpd. The width effects predicted by HDM-IIIa are at least double those given by the standard HDM-III model, for all road alignment options.
9.16 These results indicate that traffic volume effects are very small over the full range of traffic volumes considered here. Two additional factors in the India study are lower average speeds and a higher percentage of trucks. The first of these would tend to reduce overall speed-volume effects, while the second could explain the larger variation on steep alignments.

**TABLE 26 India Case Study: Optimum Geometry Improvements When Construction Costs are Halved**

<table>
<thead>
<tr>
<th>BASE W G</th>
<th>100 VEH/D W G NPV</th>
<th>300 VEH/D W G NPV</th>
<th>500 VEH/D W G NPV</th>
<th>1000 VEH/D W G NPV</th>
<th>3000 VEH/D W G NPV</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.4 1.</td>
<td>7.4 4. 3.</td>
<td>7.4 5. 60.</td>
<td>7.4 8. 140.</td>
<td>7.4 10. 351.</td>
<td>7.4 10. 1275.</td>
</tr>
<tr>
<td>7.4 4.</td>
<td>7.4 4. 0.</td>
<td>7.4 8. 13.</td>
<td>7.4 10. 49.</td>
<td>7.4 10. 147.</td>
<td>7.4 10. 548.</td>
</tr>
<tr>
<td>7.4 7.</td>
<td>7.4 7. 0.</td>
<td>7.4 7. 0.</td>
<td>7.4 10. 13.</td>
<td>7.4 10. 51.</td>
<td>7.4 10. 205.</td>
</tr>
<tr>
<td>6.0 1.</td>
<td>6.0 4. 8.</td>
<td>6.0 8. 110.</td>
<td>7.4 8. 177.</td>
<td>7.4 10. 447.</td>
<td>7.4 10. 1733.</td>
</tr>
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<td>6.0 4.</td>
<td>7.4 4. 2.</td>
<td>6.0 10. 43.</td>
<td>7.4 10. 66.</td>
<td>7.4 10. 200.</td>
<td>7.4 10. 811.</td>
</tr>
<tr>
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<td>7.4 7. 1.</td>
<td>6.0 10. 14.</td>
<td>7.4 10. 25.</td>
<td>7.4 10. 89.</td>
<td>7.4 10. 399.</td>
</tr>
<tr>
<td>5.0 1.</td>
<td>5.0 4. 12.</td>
<td>6.0 8. 91.</td>
<td>7.4 8. 197.</td>
<td>7.4 10. 514.</td>
<td>7.4 10. 2007.</td>
</tr>
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<td>7.4 4. 3.</td>
<td>6.0 10. 37.</td>
<td>7.4 10. 80.</td>
<td>7.4 10. 237.</td>
<td>7.4 10. 959.</td>
</tr>
<tr>
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<td>7.4 7. 2.</td>
<td>6.0 10. 18.</td>
<td>7.4 10. 34.</td>
<td>7.4 10. 115.</td>
<td>7.4 10. 507.</td>
</tr>
<tr>
<td>3.8 1.</td>
<td>3.8 5. 18.</td>
<td>6.0 8. 80.</td>
<td>7.4 8. 241.</td>
<td>7.4 10. 627.</td>
<td>7.4 10. 2562.</td>
</tr>
<tr>
<td>3.8 4.</td>
<td>7.4 4. 6.</td>
<td>6.0 10. 36.</td>
<td>7.4 10. 104.</td>
<td>7.4 10. 298.</td>
<td>7.4 10. 1272.</td>
</tr>
<tr>
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<td>7.4 7. 3.</td>
<td>6.0 10. 25.</td>
<td>7.4 10. 51.</td>
<td>7.4 10. 161.</td>
<td>7.4 10. 733.</td>
</tr>
</tbody>
</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS
### TABLE 27 India Case Study: Optimum Geometry Improvements When Construction Costs Increase by 50 Percent

<table>
<thead>
<tr>
<th>BASE W GEOM</th>
<th>100 VEH/D</th>
<th>300 VEH/D</th>
<th>500 VEH/D</th>
<th>1000 VEH/D</th>
<th>3000 VEH/D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W G NPV</td>
<td>W G NPV</td>
<td>W G NPV</td>
<td>W G NPV</td>
<td>W G NPV</td>
</tr>
<tr>
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<td>7.4 1.</td>
<td>0.</td>
<td>7.4 4.</td>
<td>9.</td>
<td>7.4 5.</td>
</tr>
<tr>
<td>7.4 4.</td>
<td>7.4 4.</td>
<td>0.</td>
<td>7.4 4.</td>
<td>0.</td>
<td>7.4 4.</td>
</tr>
<tr>
<td>7.4 7.</td>
<td>7.4 7.</td>
<td>0.</td>
<td>7.4 7.</td>
<td>0.</td>
<td>7.4 7.</td>
</tr>
<tr>
<td>6.0 1.</td>
<td>7.4 1.</td>
<td>4.</td>
<td>7.4 1.</td>
<td>50.</td>
<td>6.0 5.</td>
</tr>
<tr>
<td>6.0 4.</td>
<td>7.4 4.</td>
<td>2.</td>
<td>7.4 4.</td>
<td>22.</td>
<td>7.4 4.</td>
</tr>
<tr>
<td>6.0 7.</td>
<td>7.4 7.</td>
<td>1.</td>
<td>7.4 7.</td>
<td>7.</td>
<td>7.4 7.</td>
</tr>
<tr>
<td>5.0 1.</td>
<td>7.4 1.</td>
<td>8.</td>
<td>5.0 4.</td>
<td>37.</td>
<td>5.0 5.</td>
</tr>
<tr>
<td>5.0 4.</td>
<td>7.4 4.</td>
<td>3.</td>
<td>7.4 4.</td>
<td>18.</td>
<td>7.4 4.</td>
</tr>
<tr>
<td>5.0 7.</td>
<td>7.4 7.</td>
<td>2.</td>
<td>7.4 7.</td>
<td>13.</td>
<td>7.4 7.</td>
</tr>
<tr>
<td>3.8 1.</td>
<td>7.4 1.</td>
<td>14.</td>
<td>3.8 4.</td>
<td>34.</td>
<td>5.0 5.</td>
</tr>
<tr>
<td>3.8 4.</td>
<td>7.4 4.</td>
<td>6.</td>
<td>7.4 4.</td>
<td>19.</td>
<td>7.4 4.</td>
</tr>
<tr>
<td>3.8 7.</td>
<td>7.4 7.</td>
<td>3.</td>
<td>7.4 7.</td>
<td>22.</td>
<td>7.4 7.</td>
</tr>
</tbody>
</table>

W=PAVEMENT WIDTH (M); GEO OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS

### TABLE 28 India: Total Operating Costs Predicted by HDM-IIIa and HDM-III

<table>
<thead>
<tr>
<th>ADT</th>
<th>300 veh/d</th>
<th>3000 veh/d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width</td>
<td>3.8 m</td>
<td>7.4 m</td>
</tr>
<tr>
<td>Road Alignment Option</td>
<td>a/ b/ A B</td>
<td>A B</td>
</tr>
<tr>
<td>Option</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>.44 .44</td>
<td>.42 .41</td>
</tr>
<tr>
<td>2</td>
<td>.42 .41</td>
<td>.39 .39</td>
</tr>
<tr>
<td>3</td>
<td>.40 .40</td>
<td>.38 .38</td>
</tr>
<tr>
<td>4</td>
<td>.36 .36</td>
<td>.34 .34</td>
</tr>
<tr>
<td>5</td>
<td>.35 .34</td>
<td>.33 .33</td>
</tr>
<tr>
<td>6</td>
<td>.34 .34</td>
<td>.32 .32</td>
</tr>
<tr>
<td>7</td>
<td>.33 .32</td>
<td>.30 .30</td>
</tr>
<tr>
<td>8</td>
<td>.32 .32</td>
<td>.29 .29</td>
</tr>
<tr>
<td>9</td>
<td>.32 .31</td>
<td>.29 .29</td>
</tr>
<tr>
<td>10</td>
<td>.31 .31</td>
<td>.28 .28</td>
</tr>
</tbody>
</table>

a/ Modified HDM-IIIa model  
b/ Standard HDM-III model, using India VOC relationships
The comparisons in Table 28 also illustrate the differences in road width effects in the two versions of the HDM model. Since both versions use the same equations for free speed, and the traffic volume effect is small, the differences are almost entirely due to the modelling of edge crossings in HDM-IIIa. These are assumed to lead to increased "effective roughness" experienced by the traffic, and increased road deterioration.

Table 29 presents the optimum road geometry improvements determined using the unmodified HDM-III model. The recommended standards are almost identical to those determined using HDM-IIIa, as shown in Table 25, except for a few cases where a narrower road width is indicated. However the Net Present Values in Table 29 are generally less than those in Table 25; the differences are small for wide roads, but very large on narrow roads.

### Table 29
India: Optimum Road Geometry Improvements Predicted by the Standard HDM-III Model

<table>
<thead>
<tr>
<th>BASE</th>
<th>OPTIMAL IMPROVEMENT AT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>W</td>
</tr>
<tr>
<td>W</td>
<td>G</td>
</tr>
<tr>
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<td>1</td>
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<td>1</td>
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<td>4</td>
</tr>
<tr>
<td>5.0</td>
<td>7</td>
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</tbody>
</table>

W=PAVEMENT WIDTH (M); GEOM OR G=ROAD GEOMETRY OPTION (1 TO 10); NPV=NET PRESENT VALUE IN MILLIONS OF U.S. DOLLARS
X. CONCLUSIONS AND RECOMMENDATIONS

10.01 This paper provides a summary of the capabilities of existing models for evaluating traffic capacity and road geometry improvements, and the development of new models to take account of traffic volume and width effects. These models have been incorporated into a modified computer program version known as HDM-IIIa, so that it can be used to evaluate geometric alternatives for road improvements.

10.02 In applying the model to this type of evaluation it is recommended that first, all costs exclusive of construction costs be estimated for each road alternative. Construction costs may then be estimated separately for each improvement option. This procedure provides great flexibility in identifying the relative benefits of alternative improvement types and comparing these for various levels of estimated construction cost. A program HDCOST written for this purpose is prescribed in Chapter VII.

10.03 The results of a general evaluation may also be used to formulate guidelines for economic assessments of specific geometric improvement schemes in a homogeneous region. When real construction cost data are available, the general results can be used to estimate overall project economics and the marginal economics of various parts or stages of a major scheme.

10.04 The procedures described here have been applied to two specific case studies for India and Costa Rica. These demonstrate that increasing standards of road width and alignment can be economically justified as traffic volumes increase. However the appropriate standard at a given volume is not fixed for all conditions, but varies with traffic composition, difficulty of construction, and the unit costs of construction, vehicle operation, travel time and other factors. Many of these factors were quite different for the two case studies considered in Chapters VIII and IX. As a result the India case study recommended geometric improvements at much lower traffic volumes than were found for Costa Rica.

10.05 Two interesting features of the analysis procedures should be noted. First, the case studies considered a number of different base conditions for both alignment and width. This led to a range of appropriate design standards at a given traffic volume, rather than a single result. Improvement recommendations were sensitive to the base condition, suggesting that this approach is valid - though perhaps inconvenient - in establishing design standards. Second, road width and alignment were treated as independent improvement variables in the analysis. This led to inconsistencies where one base condition would call for widening and another for realignment at the same traffic volume, or where a given base condition would favour widening at one volume, and then drop this for realignment at a higher traffic volume. These inconsistencies can also arise in practice, where various combinations of widening and realignment can yield similar results.
10.06 The procedures and results in Chapters VIII and IX may be used to develop optimum road geometry standards for particular countries and road and traffic conditions. However, the case studies indicated a need for more detailed analysis of the HDM results. In particular, the analysis must take account of marginal differences in moving from one standard to another, and a rational progression of cumulative road improvements. Since such detailed analysis is beyond the scope of the current study, it would be premature to draw conclusions regarding appropriate geometric standards for particular cases.

10.07 The procedures developed here are fairly robust and could be incorporated into applications of the HDM model to specific studies. Because of the uncertain validity of many model assumptions, any immediate applications should include some sensitivity tests and comparisons with the standard HDM-III model. Parameter estimation requirements for such applications are given in Chapter VI, Section H. In the short term, field data should be used to obtain local estimates of the parameters QCAP, FFREQ, PSH, RSE and EPV.

10.08 The needs for future research may be divided into two categories. First, empirical research is required to quantify the effects of traffic congestion and road width. Studies could aim to test, refine and validate some of the concepts and models presented in this report. Second, further theoretical research is required in specific areas which could not be finalized in the time available for this study. Two such areas are the effects of overtaking supply on speed-volume relationships, and the establishment of evaluation criteria for determining appropriate road geometry standards. It is recommended that future research efforts in the area of traffic capacity and road width modelling should focus on the following topics:

(i) Testing The Speed-Flow Model: The validation of the speed-flow relationships is considered as the top priority. It would be most useful to develop a procedure for fitting the proposed HDM speed-flow model, and testing it against field traffic data on speed, flow, platooning, and vehicle types at rural road sites. The procedure should be designed so that it can be repeated in other countries in order to provide a measure of the reliability of the proposed model and the magnitude of speed-flow effects.

(ii) Overtaking Opportunities: It appears that speed-flow slopes on busier two-lane roads will be substantially affected by overtaking opportunities, whether provided by sight distance or passing lanes. This can be most effectively studied using traffic simulation to control other variables, and using observed traffic data for confirmation. Research on this topic should lead to a refinement of the simple macroscopic measure of overtaking opportunity. Improved modelling in this area should encourage road designs which maximize overtaking opportunities, or use passing and climbing lanes as alternatives to expensive realignment and provision of multi-lane facilities.
(iii) Factors Affecting Road Width: Very little is know about factors that could influence the selection of an optimal road width for a particular road project and further research would be highly desirable. This can be sub-divided into several areas:

- **Vehicle interactions on narrow roads**: Approximate models of speed reductions and edge crossings resulting from vehicle interactions on narrow roads are included in this study, but it would be valuable to investigate this behavior under real traffic conditions. As a starting point, a moving vehicle using video and radar equipment could be used to record the initial and minimum speeds, and the probability and duration of shoulder travel for each overtaking or crossing maneuver. Calibrated marks on the windsccreen of the test car would serve to measure road width and lateral movement of vehicles, based on triangulation from a fixed camera position as well as the quality of the pavement, shoulder and edge conditions. The study would be somewhat exploratory in nature, but should lead to the development of a procedure for specific applications.

- **Free speed and speed-flow studies**: This research should be carried out on narrow roads in a number of countries and regions and should take account of factors which are often unmeasured in studies of this type, for example, sight distance, road roughness, and the quality, roughness, width, and safety of road shoulders or the roadside environment. The presence of other road users such as pedestrians or cyclists may also be important. Where possible, field studies should aim to record the same traffic travelling on road sections of differing width standard along a given route, provided the sections are of sufficient length to ensure equilibrium operation.

- **Edge deterioration and maintenance**: This appears to be a major factor in the selection of appropriate road width, and damage could be much greater in magnitude on a narrow road than normal pavement deterioration predicted by HDM-III. In view of the severe lack of information in this area, a survey of road authorities with one-lane and narrow two-lane roads would be needed initially, including collection of data on traffic flows and maintenance expenditures. Such a survey could provide information on the magnitude of edge damage problems as a function of road width and traffic volume and composition. Information would be needed on the magnitude of edge maintenance relative to that for the rest of the road pavement, and estimates of maintenance expenditure for various conditions. Results are likely to be largely qualitative, but the survey could produce quantitative data which would provide a basis for more substantial research.
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Assumptions Used in Component Speed Models

Modelling of the five limiting speeds requires a number of assumptions which should be reassessed in order to provide more robust estimates of vehicle operating costs in new environments. They include:

- Maximum used drive power on upgrades (HPDRIVE) is assumed to be independent of speed. In practice, the maximum power delivered by most vehicles varies with engine speed. This is a particular problem for cars, which have only a few gear ratios available.

- Maximum speed on downgrades is assumed to be controlled by a maximum used braking power (HPBRAKE) which is independent of speed. In practice, higher downgrade speeds may be observed towards the bottom of a grade, or where the downgrade is straight and has good sight distance, or where a downgrade is followed by an upgrade.

- Air resistance effects are assumed to be insignificant in the range of speeds for which HPBRAKE is a constraint.

- Maximum perceived friction ratio (FRATIO) is assumed to be independent of speed. McLean (1983) found that used side friction ratios on sealed pavements in Australia varied from 0.35 at 50 km/h design speed to 0.11 at 130 km/h design speed.

- FRATIO was also found to be relatively insensitive to roughness in the Brazil data. It may be appropriate to re-test this assumption in other countries which have rougher roads and/or worse vehicle suspensions.

- The term \( SP_g \cdot \left( \frac{V^2}{g \cdot RC_g} \right) \) in the equation for FRATIO was assumed to be very much smaller than 1, so that it could be neglected, permitting a simple solution for FRATIO. If we take \( RC=50m, V=54km/h \ (15m/s), g=9.81, SP_g=0.10 \) then this term gives 0.10 \( (15^2/500)=0.045 \). In other words, the assumption introduces an error of about 5 percent in this case.

- The maximum "average rectified velocity" of a vehicle axle relative to the body (ARVMAX, measured in mm/s) is assumed to be independent of vehicle speed and road roughness. While this assumption was found to be quite reasonable for the Brazil data, it should be re-tested in new environments.
The mean limiting speed on straight flat smooth roads (VDESIR) is assumed to be a constant for all roads in a region. However, McLean (1980) has shown that desired speed as defined here varies with the aggregate curvature on adjacent road sections, and to a lesser extent, with the sight distance available over an extended road length. In effect, overall curvature and sight distance levels were found to create a "speed environment" which influenced not only desired speed, but also the speeds adopted on individual curves (McLean 1979). Sight distance is likely to be of greater importance on narrow roads where the risks associated with unseen vehicles and obstructions are greater (see Chapter IV).
This Appendix documents changes to existing HDM routines and the source listing of the new subroutines RDMPVE, VOLUME, USERS2 and USERI2 which take account of traffic interaction effects in the HDM model. The new subroutines have been written so as to minimize changes to the existing HDM program. This is useful in the short term for development and debugging purposes but it means that the changes are not as elegant as might be desired for integration into the HDM system. If the subroutine is found to be acceptable for general application, it is recommended that its incorporation into HDM be modified in the following ways:

- The subroutines USERES, USEREB, USEREI and USEREC should each be broken into two parts, the first calculating free speed only and the second calculating user cost quantities.

- The subroutine MODUSE should call the speed calculation routine first, then the VOLUME subroutine if required, then the routines for calculating user cost quantities.

- Data input and checking procedures in the new routines should be integrated into the standard HDM procedures.

In addition to the creation of three new subroutines, a small number of changes are required in existing HDM routines to accommodate the new subroutines. These changes are as follows:

MODUSE:

11605900 DIMENSION RCON(11),RES(11) COSTS (14,3), GRRES(9,11)
11610810 CALL BIEFF (RCON, RADT)
11611100 DO 600 J=1,11
11611700 CALL VOLUME (GRRES,IS,RCON)

USERES:

16203810 C (10) - UPHILL SPEED (KM/H)
16203820 C (11) - DOWNHILL SPEED (KM/H)
16209700 REAL RES(11), RCON (11)
16238300 IF (IREGN.EQ.3) CALL USEREB (IV,RCON,FUEL,OIL, TIRES, SPARES RMNTLB,S,VU,VD,0)
16244110 RES (10)= VU*3.6
16244120 RES (11)= VD*3.6
USEREB:

15600000 SUBROUTINE USERES (IV, RCON, FUEL, OIL, TIRES, SPARES, RMNTLB, S, VU, VD, IR)

15607820 C WHEN IR = 0 RECALCULATE FUEL & TIRES ONLY
15607830 C
15607850 IF (IR.GT.0) GOTO 49
15615450 49 CONTINUE
15618450 C
15618460 IF (IR.GT.0) GOTO 700
15624110 IF (IR.GT.0) RETURN
APPENDIX B

CHANGES TO EXISTING SUBROUTINES:

*** SUBROUTINE MODUSE ***
-----------------

DIMENSION RCON(11), RES(11), COSTS(14,3), GRRES(9,11)
CALL BIEFF(RCON,RADT)
DO 600 I=1,11
   DO 600 I=1,11
   CALL VOLUME(GRRES,IS,RCON)
*** SUBROUTINE RDMPV ***
----------------

I1
SGCR2A(10), SGCR4A(10), SGRAVA(10), SGEDGA(10),
E
VBNE2(10,3,9), AEDGA(10,3),
AEDGA(IS,ISB)=SGEDGA(IS)
C
C EXTRA POTHOLING DUE TO EDGE CROSSINGS
IF(IAGE1(IS,ISB).GT.2) CALL RDMPVE(SGRW(IS), RADT, DAEDGD)
AEDGB=AMIN1(15., AEDGA(IS,ISB)+DAEDGD)
ACRAB=AMIN1(100.-APOTB-AEDGB, ACRABA(IS,ISB)+DACRAD-XX)
ACRWB=AMIN1(100.-APOTB-AEDGB, ACRWA(IS,ISB)+DACRWDX, ACRAB)
ARAVA=AMIN1(100.-APOTB-AEDGB-ACRAB, ARAVA(IS,ISB)+DARAVID-ZZ)
A + .0798*DCMDR + .8*SGRW(IS)*(DAPOTD+0.5*DAEDGD)) * SGDROP(IS)
DAEDGM=0.
DAEDGM=-AEDGB
IF (ACRAB+ARAVA+APOTB+AEDGB .LE. RMRESE(ISTD,3))
720 DASP=AMAX1((ACRW-20.)/10., 0.) + APOTB + AEDGB
DAEDGM=-AEDGB
QIP=AMAX1(.72*SGRW(IS)*(DAPOTM+0.5*DAEDGM), -60.)
Y=ACRW+ARAVA+APOTB+AEDGB
IF(RMSE(ISTD,1).EQ.3) DASP=(RMSE(ISTD,2)/100.)*(APOTB+AEDGB)
DAEDGM=AMIN1(AEDGB, DASP)
Y=DASP+DAEDGM
DAPOTM=AMIN1(APOTB, Y)
Y=Y+DAPOTM
A SGRW(IS)*(DAPOTM+0.5*DAEDGM)
AEDGA(IS,ISB)=AEDGB+DAEDGM

*** SUBROUTINE USECST ***
-----------------

DIMENSION RES(11), COSTS(14,3)
*** SUBROUTINE USEREB ***
-----------------------

SUBROUTINE USEREB(IV,RCON,FUEL,OIL, TIRES, SPARES, RMNTLB, S, VU, VD, IR)

REAL WFFACT(6,2)
DATA WFFACT /0.0, 0.0, .04, .50, .96, 1.0,
X 7.4, 6.7, 6.1, 5.5, 4.9, 3.8/
NFFACT=6

C WHEN IR \textgreater 0, RECALCULATE FUEL \& TIRES ONLY;
C THIS REPEAT IS CALLED BY SUBROUTINE VOLUME (HOBAN-AUG85)

IF (IR.GT.0) GOTO 49
W=RCON(8)
VDIFF=15.0
XX=1.0
IF (W.GT.WFFACT(1,2)) GOTO 44
DO 43 I=1,NFFACT
IF (W.LT.WFFACT(1,2)) GOTO 43
XX=WFFACT(I,1)+(WFFACT(I-1,1)-WFFACT(1,1))*(WFFACT(I-1,2)-W)
A /(WFFACT(I-1,2)-WFFACT(1,2))
GOTO 44
43 CONTINUE
44 IF (RCON(1).LT.1.5) VPSYCH=VPSYP(IV)-VDIFF*XX
IF (RCON(1).GE.1.5) VPSYCH=VPSYU(IV)-VDIFF*XX
C-DBG WRITE(99,45)VPSYCH,VPSYP(IV),VDIFF,XX,W
45 FORMAT(37H USEREB: VPSYCH,VPSYP(IV),VDIFF,XX,W=,5F7.2)
49 CONTINUE

C

*** SUBROUTINE USERES ***
-----------------------

C (10)- UPHILL SPEED (KM/H)
C (11)- DOWNHILL SPEED (KM/H)
REAL RES(11), RCON(11)
IF(IREGN.EQ.5) CALL USEREB(IV,RCON,FUEL,OIL, TIRES, SPARES, RMNTLB, S, VU, VD, 0)
RES(10) = VU*3.6
RES(11) = VD*3.6
SUBROUTINE BIEFF(RCON,RADT)

C **********************************************************************
C BIEFF : CALCULATES EFFECTIVE ROUGHNESS DUE TO SHOULDER TRAVEL
C **********************************************************************
C CALLED BY MODUSE AT LINE 11610810
REAL PSH(6,2),FFREQ(5,2),RCON(11),RE,RP,RSE,QHR,RADT,XPSH,STIME
REAL STIMI,FFNV,FSUM
INTEGER NFFREQ,NPSH
LOGICAL DBG
DATA PSH / 0.0, 0.0, 0.0, 0.3, 0.6, 1.0,
X 7.4, 6.7, 6.1, 5.5, 4.9, 3.8/
DATA FFREQ/ .12, .09, .07, .05, .025,
X .03, .03, .11, .29, .54/
NPSH = 6
NFFREQ = 5
W=RCON(8)
RP=RCON(2)
RSE=-RP*2
DBG=.TRUE.

C FIRST CALCULATE PROPORTION OF VEHICLES CROSSING EDGE (XPSH)
C FOR THIS ROAD WIDTH.
C
XPSH = 0.0
IF (W.GT.PSH(1,2)) GOTO 200
DO 100 J=2,NPSH
IF (W.LT.PSH(J,2)) GOTO 100
IF (PSH(J,1).EQ.0.0) GOTO 200
XPSH=PSH(J,1)+(PSH(J-1,1)-PSH(J,1))
X *(PSH(J-1,2)-W)/(PSH(J-1,2)-PSH(J,2))
GOTO 200
100 CONTINUE
200 IF (XPSH.EQ.0.0) RETURN
APPENDIX B

LOOP OVER THE FREQUENCY DISTRIBUTION OF HOURLY FLOWS;

SHOULDER TRAVEL TIME IS A FUNCTION OF XPSH*QHR;

STIME IS WEIGHTED AVERAGE ACCORDING TO NUMBER OF VEHICLES.

FFNV=0.0
FSUM=0.0
STIME=0.0
DO 20 I=1,NFFREQ
20 FSUM = FSUM + FFREQ(1,2)
DO 30 I=1,NFFREQ
QHR=RADT*FFREQ(I,1)
STIMI=XPSH*QHR/720
STIME=STIME+STIMI*FFREQ(I,2)
30 FFNV = FFNV + FFREQ(I,1) * FFREQ(I,2) * 24. / FSUM

WHEN FREQUENCIES SUM TO 24, AREA SHOULD SUM TO 1.0

IF(FFNV.GT.0.96.AND.FFNV.LT.1.04) GOTO 40
WRITE(99,35) FFNV,FSUM,NFFREQ,FFREQ
35 FORMAT(48H **STOP** FFREQ MATRIX AREA DOESN'T ADD UP TO 1:
X 25H FFNV,FSUM,NFFREQ,FFREQ= ,2F6.2,16(5X,5F6.1/))
STOP
40 CONTINUE

EFFECTIVE ROUGHNESS IS A WEIGHTED AVERAGE OF RSE AND RP.

NOTE STIME HAS A MAXIMUM VALUE OF 40 PERCENT.

STIME=AMIN1(.40,STIME/FFNV)
RE=RSE*STIME + RP*(1.-STIME)
RCON(2)=RE
C-DBG IF (DBG) WRITE(99,905) RP,RSE,RE,STIME,XPSH,W
905 FORMAT(32H BIEFF: RP,RSE,RE,STIME,XPSH,W= ,6F8.2)
RETURN
END
SUBROUTINE RDMPE(W,RADT,DAPOTE)
C ***************************************************************
C R D M P V E : EXTRA POTHOLING DUE TO EDGE CROSSINGS
C ***************************************************************
C CALLED BY RDMPE LINE 12439250
REAL PSH(6,2),FFREQ(5,2),EPV,SPEED,W,RADT,APOTE,APOT1,DAPOTE
REAL RNECRS,SNECRS,QHR,XPSH,XX
INTEGER NPSH,NFFREQ
LOGICAL DBG
DATA PSH / 0.0, 0.0, 0.0, 0.3, 0.6, 1.0, 7.4, 6.7, 6.1, 5.5, 4.9, 3.8/
DATA FFREQ/ .12, .09, .07, .05, .025, .03, .03, .11, .29, .54/
NPSH = 6
NFFREQ = 5
EPV=30.0
SPEED=70.0
DBG=.TRUE.
DAPOTE=0.0
C
C FIRST CALCULATE PROPORTION OF VEHICLES CROSSING EDGE (XPSH)
FOR THIS ROAD WIDTH.
C
XPSH = 0.0
IF (W.GT.PSH(1,2)) GOTO 200
DO 100 J=2,NPSH
IF (W.LT.PSH(J,2)) GOTO 100
IF (PSH(J,1).EQ.0.0) GOTO 200
XPSH=PSH(J,1)+(PSH(J-1,1)-PSH(J,1))
X=(PSH(J-1,2)-W)/(PSH(J-1,2)-PSH(J,2))
GOTO 200
100 CONTINUE
200 IF (XPSH.EQ.0.0) RETURN
LOOP OVER THE FREQUENCY DISTRIBUTION OF HOURLY FLOWS;
NUMBER OF EDGE CROSSINGS IS A FUNCTION OF XPSH*QHR*QHR.
NUMBER OF HOURS PER YEAR IS FFREQ(1,2)*8760/FSUM.

FSUM=0.
FFNV=0.
SNECRS=0.0
DO 20 I=1,NFFREQ
20 FSUM = FSUM + FFREQ(1,2)

DO 30 I=1,NFFREQ
QHR=RADT*FFREQ(I,1)
RNECRS=(2/SPEED)*XPSH*QHR*QHR*FFREQ(1,2)*8760/FSUM
SNECRS=SNECRS+RNECRS
30 FFNV = FFNV + FFREQ(I,1) * FFREQ(1,2) * 24 / FSUM
WHEN FREQUENCIES SUM TO 24, AREA SHOULD SUM TO 1.0
IF(FFNV.GT.0.96.AND.FFNV.LT.1.04) GOTO 40
WRITE(99,35) FFNV,FSUM,NFFREQ,FFREQ
35 FORMAT(47H RDMPVE: SNECRS,APOT1,APOTE,DAPOTE,XPSH,W,RADT=
 12505300 12505400 12505500 12505600 12505700 12505800 12505900
X F9.1,5F6.2,F9.1)
RETURN
END

EDGE POTHOLE VOLUME IS EPV CUBIC M PER MILLION CROSSINGS.
CONVERT TO % AREA ASSUMING 400 MM EDGE WIDTH EACH SIDE
AND AVERAGE DEPTH 80 MM, APOTE=100*VOL/(1000.*.8*.08).
ASSUME MAXIMUM POTHOLING OF 20 PERCENT OF EDGE AREA PER YR.
DAPOTE CONVERTS APOTE TO PERCENTAGE OF TOTAL ROAD AREA.

APOT1=EPVSNECRS/640000
APOTE=AMIN1(20.,APOT1)
DAPOTE=APOTE*0.8/W

IF(DBG)WRITE(99,905)SNECRS,APOT1,APOTE,DAPOTE,XPSH,W,RADT
905 FORMAT(47H RDMPE: SNECRS,APOT1,APOTE,DAPOTE,XPSH,W,RADT=
 12506500 12506600 12506700 12506800 12506900 12507000 12507100 12507200
X F9.1,5F6.2,F9.1)
RETURN
END
SUBROUTINE USER2(IV,RCON,FUEL,S)

C ********************************************************************
C USER12: TO RECALCULATE FUEL FOR MODIFIED SPEED
C COPYED FROM USEREI; MAKE SURE THESE MATCH!
C*******************************************************************
C CALLED BY VOLUME
COMMON /VEH/ NVEH, VNAME(9,2), IVTYPE(9), IVFUEL(9), VESAL2(9),
A VHP(9), VWGHT(9), VESAL4(9), VPASS(9), VESALX(9),
B VCVEH(9,3), VCTIRE(9,3), VCINT(9,3), VCPOL(9,3),
C VCCREW(9,3), VCMLAB(9,3), VCTIME(9,3),
D VCARGO(9,3), VCPCT(9,3), VCOVHD(9,3), ICHESH,
E VLEY(9), VYRKM(9), VUTIL(9), IVKM(9), IVDEP(9),
F VLOAD(9), VDRAG(9), VFRONT(9), VHPUP(9), VHPDOW(9),
G VRATOP(9), VRATIP(9), VRATIUP(9), VRATIUP(9), VPSY(9),
H VPSYU(9), VCRPM(9), VARVMX(9), VCPART(9,3), VHUR(9),
I VPD(9), VNIDUS(9), NVTIME(9), VRETRC(9), VTVOL(9),
J , VTNR(9), VTTW(9), VTCW(9), VPCON(9), VPRGH(9),
K VPLIM(9), VCON(9), VLEXP(9), VLRGH(9),
L
DIMENSION RCON(11)
C
C ASSIGN FREQUENTLY USED VARIABLES:
B1=RCON(2)
RI=RCON(5)
F =RCON(6)
RF=RI+F
CKM=(VYRKM(IV)*VLFYR(IV))/2.
IVT=IVTYPE(IV)
IF (IVT .GE. 4) PWR=AMIN1(30.,VHP(IV)/VWGHT(IV))

GOTO (100,200,300,400,500,600,700) IVT

100 FUEL=1.16*(49.8+319./S+.0035*S*S+.0019*B1+
A .132*RF)
RETURN

200 FUEL=1.16*(10.3+1676./S+.0133*S*S+.0006*B1+
A .178*RF)
RETURN

300 FUEL=1.16*(-30.8+2260./S+.0242*S*S+.0012*B1+.356*RF)
RETURN

400 FUEL=1.15*(85.1+3900./S+.0207*S*S+
A .0012*B1+.776*RF-.59*PWR)
RETURN

500 FUEL=1.15*(266.+2520./S+.0362*S*S+
A .0066*B1+.764*RF-.46*PWR)
RETURN

600 FUEL=1.15*(85.1+3900./S+.0207*S*S+
A .012*B1+.776*RF-.459*PWR)
RETURN

700 FUEL=1.15*(266.+2520./S+.0362*S*S+
A .0066*B1+.764*RF-.46*PWR)
RETURN
END
SUBROUTINE USERS2(Iv,AKM,ADEP,AINT,S) 16250000

C ****************************************** 16250010
C USER S 2 : RECALCULATE AKM AND LIFE FOR MODIFIED SPEED 16250020
C COPIED FROM USERS; MAKE SURE THEY MATCH! 16250030
C ****************************************** 16250040
C CALLED BY VOLUME 16250050
COMMON /VEH/ NVEH, VNAME(9,2), IVTYPE(9), IVFUEL(9), VESAL2(9), 16250060
A VHP(9), VWGHT(9), VESAL4(9), VPASS(9), VCPOL(9,3), 16250070
B VVEH(9,3), VCINT(9,3), VCMLAB(9,3), VCMLAB(9,3), 16250080
C VCSSR(9,3), VITIRE(9,3), VCTYR(9,3), 16250090
D VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250100
E VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250110
G VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250120
H VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250130
I VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250140
J VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250150
K VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250160
L VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250170
M VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250180
N VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250190
O VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250200
P VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250210
Q VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250220
R VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250230
S VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250240
T VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250250
U VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250260
V VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250270
W VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250280
X VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250290
Y VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250300
Z VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), VCSSR(9,3), 16250310

C ----- COMPUTE AVERAGE ANNUAL KILOMETERAGE 16252000
50 IVD=IVDEP(Iv) 16252020
IVK=IVKM(Iv) 16252030
GO TO (51,52,53), IVK 16252040

C CONSTANT KILOMETERAGE METHOD 16252500
51 AKM=VYRKM(Iv) 16252520
GO TO 60 16252530

C CONSTANT HOURLY UTILIZATION METHOD 16252900
52 AKM=S*VUTIL(Iv) 16252920
GO TO 60 16252930

C MAXIMUM UTILIZATION ON FIXED ROUTES METHOD 16253400
53 OPH=VUTIL(Iv)/VHUR(Iv) 16253420
AKM=OPH/(((OPH-VUTIL(Iv))/VYRKM(Iv)) + (1./S)) 16253430

C ----- COMPUTE DEPRECIATION AND INTEREST COMPONENTS 16253900
60 GO TO (63,64), IVD 16254100

C DEWEILLE VARYING VEHICLE LIFE METHOD 16254300
63 SZERO = VYRKM(Iv)/VUTIL(Iv) 16254400
XLF = (SZERO/S + 2.)/3. 16254500
XLF = AMIN1(XLF,1.5) 16254600
ALIFE = XLF*VLFYR(Iv) 16254700
GO TO 65 16254800

C CONSTANT VEHICLE LIFE METHOD 16255000
64 ALIFE = VLFYR(Iv) 16255100
65 ADEP = 1./ALIFE 16255200
AINT = .005 16255300
RETURN 16255400
END 16255700
SUBROUTINE VOLUME(GRRES,IS,RCON)
C******************************************************************
C VOLUME : MODIFY SPEED AND OTHER VOC FOR VOLUME EFFECTS
C******************************************************************
C CALLED BY MODUSE
C CALLS USERS2 AND USER12 OR USEREB FOR RECALCULATION OF VOC.
C
COMMON /CONTRL/ IBEGIN, IHORIZ, ISPRSS(7), IREGN, IRUNTI, IRUNTC,
A RCONF1, RCONF2, REGN(4), UNI(3), BITOQI, BASENM, RHORIZ
B COMMON /VEH/ NVEH, VNAME(9,2), IVTYPE(9), IVFUEL(9), VESAL2(9),
C VEH(9,3), VCTIRE(9,3), VCINT(9,3), VCPOL(9,3),
D VCCREW(9,3), VCMMLAB(9,3), VCTIME(9,3),
E VCARGO(9,3), VCPCT(9,3), VCOVHD(9,3), ICHESH,
F VLFYR(9), VYRKM(9), VUTIL(9), LWKM(9), LDPEP(9),
G VLOAD(9), VDRAG(9), VFRONT(9), VHPUP(9), VHPDOW(9),
H VRATOP(9), VRATOU(9), VRAT1P(9), VRAT1U(9), VPSYP(9),
I VPSYU(9), VCRP(9), VARYMX(9), VCPART(9,3), WHUR(9),
J WIDP(9), WIDU(9), NVTIRE(9), VRETRC(9), VTVOL(9),
K VTVR(9), VTTW(9), VTC(9), VPCON(9), VPGRH(9),
L VPLIM(9), VLCON(9), VLEXP(9), VLRGH(9),
COMMON /TRAF/ NTRAF, TRID(20), TRDES(20,4), TRTYPE(20),
A LTRGRO(20), NTRGRO(20), LTRCON(20), NTRCON(20),
B LTRGRO(200), NTRGRO(200), LTRCON(200), NTRCON(200),
C ITRGYR(200), ITRMOD(200), TRGRO(200,9),
D ITRPYR(200), ITRPKH(200), ITRP(200), TRPFAC(200),
E ICONG, QPE
COMMON /DATA01/ IRSEG(10), RLLEN, RSLLEN(10), RSDAMG(10,3,23),
A IRMYRS(10,2), RMMPOL, RSMSTD(10), RLMCST(12),
B RLMQT(12), RSMQT(10,12), RSMQT(10),
C RADT, RESAL2, RESAL4, RESALX, RLETRF(9), RLGTRF(9),
D RLESA2(9), RLESA4(9), RLESA9(9), RVNE2(10,3,9),
E RSOCE(10,9,11), RSSPED(10,9), RSSURF(10), RSSSTYP(10),
F RSSPD(10,9), RSSPD(10,9), RFSCA(10,9),
G RFSLCP(10,9), RFSLC(10,9), RSLPC(10,9,3,8),
H RVNE4(10,3,9), RVNX(10,3,9), RUNE4(10,3,9),
I RVNE4(10,9,10), RVLTR(10,9,10)
REAL GRRES(9,11), PCU(4,9), FFREQ(5,2), WFACT(6,2), RCON(11)
REAL QMAX, QCAP, QOCUT, QSAM, SMCM, CV, XX, QQ, QFAM, FSU
REAL FFNV, SCAP(2), CUMSP(9), CUMF(9), CUMT(9), SQ(2), SF(9,2)
INTEGER ILOOP, NFREQ, NFFACT
LOGICAL DBG
SET PARAMETERS; LATER VERSIONS MAY READ SOME IN AS INPUTS.

NFFREQ = 5
NWFACT = 6
QMAX = 2500.
XQJAM = 1.25
SJAM = 8.
CV = 0.15
W = RCON(8)
DBG = .TRUE.

CHECK THAT FFREQ MATRIX HAS AN AREA OF 1; THE AREA FFNV WILL BE USED LATER IN FINDING WEIGHTED MEAN SPEED. IF FREQUENCIES ADD TO 24 HOURS, AREAS SHOULD ADD TO 1 DAY

FSUM=0.
FFNV=0.
DO 20 I=1,NFFREQ
20 FSUM = FSUM + FFREQ(I,2)
DO 30 I=1,NFFREQ
30 FFNV = FFNV + FFREQ(I,1) * FFREQ(I,2)
IF (FSUM.NE.24.) FFREQ(I,2)=FFREQ(I,2)*24./FSUM
WRITE(99,35) FFNV,FSLUM,NFFREQ,FFREQ=,2F6.2,16/2(5X,5F6.1/)
STOP
40 CONTINUE
C-DBG IF (DBG) WRITE(99,45) 1,FSUM,FFNV
45 FORMAT(3H **,9F6.1/)
C
FIND REDUCED CAPACITY FOR A GIVEN ROAD WIDTH

XX=-1
IF (W.LT.WFACT(1,2)) GOTO 100
XX=1.0
GOTO 120
100 DO 110 I=2,NWFACT
110 IF (W.LT.WFACT(1,2)) GOTO 110
XX=WFACT(1,1)+(WFACT(1-1,1)-WFACT(I,1))*(WFACT(I-1,2)-W)/WFACT(I-1,2)-WFACT(1,2))
GOTO 120
110 CONTINUE
120 IF(XX.GT.O.AND.XX.LE.1.0) GOTO 130
WRITE(99,125) W,XX,WFACT=,2F6.2,16/2(5X,5F5.1/)
XX=1.0
130 QCAP = QMAX * XX
QO = QCAP * XQCUT
QJAM = QCAP * XQJAM
C-DBG IF (DBG) WRITE(99,45) 2,W,XX,QMAX,QCAP,QO,QJAM,XQCUT,XQJAM
APPENDIX B

C CALCULATE PCE VOLUME FOR MIXED TRAFFIC
C
QPE = 0.0
DO 220 I=1,NVEH
RLPCU(I)=PCU(IREGN,I) * (RLETRF(I)+RLGTRF(I))
QPE = QPE + RLPCU(I)
C-DBG IF (DBG) WRITE(99,215)1,RLPCU(I),RLETRF(I),RLGTRF(I),QPE,RADT
215 FORMAT(32H 1,RLPCU,RLETRF,RLGTRF,QPE,RADT=,13,F5.1,4F8.1)
220 CONTINUE
C
C FIND SPEED AT CAPACITY; SCAP = SMIN - S.D.
C FOR BRAZIL MODEL, NEED TO DO THIS FOR UP AND DOWN HILL
C ALSo ZERO COUNTERS CUMSP,CUMFL,CUMTI
C
NLOOP=1
IF (IREGN.EQ.3) NLOOP=2
DO 260 ILOOP=1,NLOOP
SMIN = 100.0
DO 250 I=1,NVEH
IF (IREGN.NE.3) S=GRRES(I,6) 
IF (IREGN.EQ.3.AND.ILOOP.EQ.1) S=GRRES(I,10)
IF (IREGN.EQ.3.AND.ILOOP.EQ.2) S=GRRES(I,11)
IF (SMIN.GT.S) SMIN=S
SF(I,ILOOP)=S
C-DBG IF (DBG) WRITE(99,45) 4,NLOOP,I,SF(I,ILOOP),SMIN
250 CONTINUE
SCAP(ILOOP)=SMIN * (1-CV)
260 CONTINUE
DO 270 I=1,NVEH
CUMSP(I)=0.0
CUMFL(I)=0.0
CUMTI(I)=0.0
270 CONTINUE
C-DBG IF (DBG) WRITE(99,285)W,XX,QCAP,QPE,SMIN
285 FORMAT(20H W,XX,QCAP,QPE,SMIN=,5F7.1)
LOOP OVER EACH FLOW-FREQUENCY REGIME IN FFREQ.
FIND MODIFIED SPEED SQ FOR EACH VEHICLE TYPE;

DO 600 I=1,NFFREQ
QHR = QPE * FFREQ(I,1)
DO 500 J=1,NVEH
NLOOP=1
IF (IREGN.EQ.3) NLOOP=2
DO 390 K=1,NLOOP
FREE SPEED=S, MODIFIED SPEED=SQ
S=SF(J,K) SQ(K) = S
IF (QHR.LE.QO) GOTO 340
IF (QHR.LE.QJAM) GOTO 320
WRITE(99,305) QHR,QPE,I,FFREQ(I,1)
SQ(K) = SJAM
GOTO 340
320 IF (QHR.LE.QCAP) SQ(K) = S-(S-SCAP(K))*(QHR-QO)/(QCAP-QO)
IF (QHR.GT.QCAP) SQ(K) = SCAP(K)-(SCAP(K)-SJAM)*
X *(QHR-QCAP)/(QJAM-QCAP)
340 IF (SQ(K).GE.SJAM.AND.SQ(K).LE.S) GOTO 360
WRITE(99,345) SQ(K),S,SJAM,QHR,QCAP,QO,QJAM
360 CONTINUE
IF(DBG.AND.J.EQ.1.AND.K.EQ.1.AND.I.EQ.1)WRITE(99,365)SQ(K),S,QHR,
X SJAM
365 CONTINUE

IF BRAZIL RECALCULATE FUEL AND TIRES; USE USEREB WITH IR=1

IF (IREGN.NE.3) GOTO 420
VU=SQ(1)/3.6
VD=SQ(2)/3.6
S=7.2/(1./VU + 1./VD)
CALL USEREB(J,RCON,FUEL,OIL,TIRES,SPARES,RMNTLB,S,VU,VD,1)
GOTO 440

IF INDIA, RECALCULATE FUEL ONLY; USE NEW ROUTINE USER2

IF (IREGN.NE.4) GOTO 480
S=SQ(1)
CALL USER2(J,RCON,FUEL,S)
C Now add up weighted averages over each FFREQ

C

440 WEIGHT=FFREQ(1,1)*FFREQ(1,2)
CUMSP(J)=CUMSP(J) + S*WEIGHT
CUMFL(J)=CUMFL(J) + FUEL*WEIGHT
CUMTI(J)=CUMTI(J) + TIRES*WEIGHT
GOTO 500

480 WRITE(99,485)IREGN,S,SQ(1)
485 FORMAT(50H **WARNING; VOLUME ROUTINE NOT SET UP FOR REGIONS
X 10 OTHER THAN/35H BRAZIL AND INDIA: IREGN,S,SQ(1)=,17,2F7.1)

500 CONTINUE
600 CONTINUE

C End of loops over veh types and FFREQ regimes

C Now check speed, fuel etc, and enter into GRRES.

C

DO 700 I=1,NVEH
S=CUMSP(I)/FFNV
IF(DBG.AND.(I.EQ.1.OR.I.EQ.4))WRITE(99,605)1,S,CUMFL(1),CUMTI(1),
X GRRES(1,6),GRRES(1,1),GRRES(1,3),FFNV
605 FORMAT(13,21H: SPEED,FUEL,TIRES=,7F6.1)
IF (S.LE.GRRES(1,6).AND.S.GE.SJAM) GOTO 620
WRITE(99,615)1,S,SJAM,GRRES(1,6)
615 FORMAT(52H **WARNING; MODIFIED SPEED OUTSIDE RANGE, CHANGE NOT,
X 6H MADE:/24H 1,S,SJAM,GRRES(1,6)=,17,3F6.1)
GOTO 640
620 GRRES(1,6)=S
640 FUEL=CUMFL(I)/FFNV
IF (FUEL.GT.50.AND.FUEL.LT.1000) GOTO 650
WRITE(99,645)FUEL,GRRES(1,1)
645 FORMAT(51H **WARNING; MODIFIED FUEL OUTSIDE RANGE, CHANGE NOT,
X 6H MADE:/21H 1,FUEL,GRRES(1,1)=17,2F6.1)
GOTO 660
650 GRRES(1,1)=FUEL
660 IF (IREGN.NE.3) GOTO 670
TIRES=CUMTI(I)/FFNV
IF (TIRES.EQ.0.0) GOTO 670
GRRES(1,3)=TIRES
670 CONTINUE
NOW CALL USERS2 TO CHECK ON AKM AND LIFE, AND REVISE RSVORE

CALL USERS2(1, AKM, AINT, ADEP, S)
ADEP = ADEP / (AKM / 1000)
AINT = AINT / (AKM / 1000)
IF (DBG.AND.I.EQ.1) WRITE(99, 685) GRRES(1, 7), GRRES(1, 8), GRRES(1, 9),
X ADEP, AINT, AKM

685 FORMAT(17H ADEP, AINT, AKM=, 2(2F6.3, F7.0))
GRRES(1, 7) = ADEP
GRRES(1, 8) = AINT
GRRES(1, 9) = AKM

CORRECT STORED RESOURCES PER VEH-KM FOR LATER REPORTING

RSVORE(IS, 1, 1) = FUEL/1000.
RSVORE(IS, 1, 3) = TIRES/1000.
RSVORE(IS, 1, 6) = GRRES(1, 7)/1000.
RSVORE(IS, 1, 7) = GRRES(1, 8)*VCINT(1, 2)/1000.
RSVORE(IS, 1, 8) = 1./S
RSVORE(IS, 1, 9) = VPASS(1)/S
RSVORE(IS, 1, 10) = 1./S

700 CONTINUE
RETURN
END
APPENDIX C

SOURCE LISTING OF THE HDCOST PROGRAM

```
1 PROGRAM HDCOST
C CALCULATES A CONSTRUCTION COST FOR A ROAD IMPROVEMENT PROJECT INVOLVING REALIGNMENT, WIDENING AND/OR RESURFACING, USING EQUATIONS SIMILAR TO THOSE IN THE HDM-III MODEL. THEN CALCULATES ECONOMIC MEASURES SUCH AS NPV AND BCR FOR EACH PROJECT USING DIFFERENCES IN TOTAL OPERATING COSTS OBTAINED FROM HDM RUNS.
C
COMMON /AA/ CROWK,CDRNK,CBRIK,COTHK,CSITU,CEWVU,XPOVH COMMON /BB/ CPVU1,CPVU2,CPVU3,TPV1,TPV2,TPV3,XCON COMMON /CC/ CASE(4,3),OPTION(5,10),WIDTH(4),COST(4) COMMON /DD/ HDMC1(5,12),HDMC2(5,12),HDMC3(5,12),HDMC4(5,12) COMMON /EE/ RLENF,CPKM COMMON /FF/ OPCOST(12,6,4),XNPV(5),XBCR(5),FLOW(5) COMMON /GG/ HDCST1(5,12),HDCST2(5,12),HDCST3(5,12),HDCST4(5,12) REAL XMBCR(5),XCOST(12),STACK(5,4) CPKM = (CROWK+CDRNK+CBRIK+COTHK) DO-80 14=1,12 DO 70 15=1,5 OPCOST(14,15,1)=HDCST1(15,14) OPCOST(14,15,2)=HDCST2(15,14) OPCOST(14,15,3)=HDCST3(15,14) OPCOST(14,15,4)=HDCST4(15,14) 70 CONTINUE 80 CONTINUE WRITE(21,105) 105 FORMAT(/10X,** ESTIMATED CONSTRUCTION COSTS **,/6X, # INITIAL FINAL CONSTRUCTION COSTS ($1000) /,6X, # WIDTH CASE WIDTH OPTION NEW W'DEN REPAY TOTAL HDM/) WRITE(23,115) 115 FORMAT(° BASE°,25X,°OPTIMAL IMPROVEMENT AT°,/,° W GEOM°,5X, ° 100 VEH/D°,6X,° 300 VEH/D°,6X,° 300 VEH/D°,5X,° 1000 VEH/D°, ° 5°,° 3000 VEH/D°,15X,° 1000 VEH/D°,° W G NPV °/) WRITE(20,125) 125 FORMAT(° II-14 COST°,19X,°NPV°,22X,°BCR°, ° /,8X,°$1000/KM°,14X,°$1000/KM°) WRITE(21,*)GRF,RFO,CVO WRITE(20,*)GRF,RFO,CVO 125 DO 570 13=1,4 570 CONTINUE WRITE(21,105) 105 FORMAT(/10X,** ESTIMATED CONSTRUCTION COSTS **,/6X, # INITIAL FINAL CONSTRUCTION COSTS ($1000) /,6X, # WIDTH CASE WIDTH OPTION NEW W'DEN REPAY TOTAL HDM/) WRITE(23,115) 115 FORMAT(° BASE°,25X,°OPTIMAL IMPROVEMENT AT°,/,° W GEOM°,5X, ° 100 VEH/D°,6X,° 300 VEH/D°,6X,° 300 VEH/D°,5X,° 1000 VEH/D°, ° 5°,° 3000 VEH/D°,15X,° 1000 VEH/D°,° W G NPV °/) WRITE(20,125) 125 FORMAT(° II-14 COST°,19X,°NPV°,22X,°BCR°, ° /,8X,°$1000/KM°,14X,°$1000/KM°) WRITE(21,*)GRF,RFO,CVO WRITE(20,*)GRF,RFO,CVO 125 DO 590 11=1,4 590 DO 580 12=1,3 580 DO 570 13=1,4
```
IF (13.LE.11) THEN
PWN=WIDTH(13)
DO 560 14=IBAS,10
RFN=OPTION(1,14)
CVN = OPTION(2,14)
IF (RFN.LE.RFO.AND.CVN.LE.CVO) THEN
XPR=OPTION(12+2,14)
CALL CALGST(PWO,PWN,RFO,RFN,GRF,XPR)
IF (14.EQ.IBAS) COST(4)=0.0
XCOST(14)=COST(4)
DO 550 15=1,5
!
C FIND NPV AND BCR USING COST=HDM TOTAL-CONSTRUCTION
C DCOST, COST(4) AND NPV ARE IN UNITS OF $1000/KM
DCOST=(OPCOST(IBAS,15,11)-OPCOST(14,15,13))*1000./RLENF
XNPV(15)=(DCOST-COST(4))
IF (COST(4).GT.0.0) THEN
XBCR(15)=DCOST/COST(4)
ELSE
XBCR(15)=0.0
ENDIF
!
C FIND IMPROVEMENT WITH THE HIGHEST NPV
IF (13.EQ.1.AND.14.EQ.IBAS) THEN
STACK(15,1)=WIDTH(1)
STACK(15,2)=14
STACK(15,3)=XNPV(15)
STACK(15,4)=XBCR(15)
ELSE IF (XNPV(15).GT.STACK(15,3)) THEN
STACK(15,1)=WIDTH(13)
STACK(15,2)=14
STACK(15,3)=XNPV(15)
STACK(15,4)=XBCR(15)
ENDIF
!
550 CONTINUE
WRITE(21,555)11,12,13,14,(COST(1),1=1,4),OPCOST(14,5,13)
555 FORMAT(417,2X,5F8.0)
WRITE(20,556)11,12,13,14,COST(4),
# (XNPV(I),1=1,5),(XBCR(1),1=1,5)
556 FORMAT(312,13,F6.0,2X,5F6.0,2X,5F5.1)
ENDIF
560 CONTINUE
WRITE(21,565)
WRITE(20,565)
565 FORMAT(/)
ENDIF
570 CONTINUE
WRITE(23,576)WIDTH(11),CASE(4,12),
# (STACK(15,1),1=1,3),15=1,3)
576 FORMAT(F4.1,F4.0,1X,5(F5.1,F4.0,F6.0))
580 CONTINUE
WRITE(23,565)
590 CONTINUE
STOP
END
## Appendix C

<table>
<thead>
<tr>
<th>Block Data BDA</th>
<th>00012200</th>
</tr>
</thead>
<tbody>
<tr>
<td>Common /AA/ CROWK, CDRNK, CBR1K, COTHK, CSITU, CEWVU, XPOVH</td>
<td>00012400</td>
</tr>
<tr>
<td>Common /BB/ CPVU1, CPVU2, CPVU3, TPV1, TPV2, TPV3, XCON</td>
<td>00012500</td>
</tr>
<tr>
<td>Common /CC/ CASE(4,3), OPTION(5,10), WIDTH(4), COST(4)</td>
<td>00012600</td>
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<tr>
<td>Common /DD/ HDMC1(5,12), HDMC2(5,12), HDMC3(5,12), HDMC4(5,12)</td>
<td>00012700</td>
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<tr>
<td>Common /EE/ RLENF, CPKM</td>
<td>00012800</td>
</tr>
<tr>
<td>Common /FF/ OPCOST(12,6,4), XNPV(5), XBCR(5), FLOW(5)</td>
<td>00012900</td>
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<tr>
<td>Common /GG/ HDCST1(5,12), HDCST2(5,12), HDCST3(5,12), HDCST4(5,12)</td>
<td>00013000</td>
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<tr>
<td>Data CROWK, CDRNK, CBR1K, COTHK /0.0, 0.0, 0.0, 500.0/</td>
<td>00013100</td>
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<tr>
<td>Data CPVU1, CPVU2, CPVU3 /2935, 1225, 348/</td>
<td>00013200</td>
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<tr>
<td>Data CSITU, CEWVU /0.0, 122/</td>
<td>00013300</td>
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<tr>
<td>Data XCON /0.02246/</td>
<td>00013400</td>
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<tr>
<td>Data RLENF, XPOVH /10.0, 0.0, 15/</td>
<td>00013500</td>
</tr>
<tr>
<td>Data TPV1, TPV2, TPV3 /0.025, 0.170, 0.200/</td>
<td>00013600</td>
</tr>
</tbody>
</table>

### Specify Three Existing Road Cases

| Data Case /85.0, 80.0, 500.0, 1.0, 55.0, 50.0, 300.0, 4.0, 25.0, 20.0, 300.0, 7.0/ | 00013700 |

### Specify Ten Road Improvement Options

| Data Option /80.0, 500.0, 0.0, 0.0, 0.0, 80.0, 300.0, 10.0, 0.0, 0.0, 80.0, 120.0, 20.0, 0.0, 0.0, 50.0, 300.0, 30.0, 0.0, 0.0, 50.0, 120.0, 40.0, 20.0, 0.0, 20.0, 300.0, 60.0, 40.0, 0.0, 20.0, 120.0, 70.0, 50.0, 20.0, 20.0, 50.0, 80.0, 60.0, 40.0, 10.0, 15.0, 100.0, 70.0, 50.0, 0.0, 0.0, 0.0, 0.0, 0.0, 0.0, 0.0/ | 00014000 |

### Specify Road Widths and Flows

| Data Width /7.4, 6.0, 5.0, 3.8/ | 00015200 |
| Data Flow /100.0, 300.0, 500.0, 1000.0, 3000.0/ | 00015300 |

### HDMIII-G Results for Costa Rica

<table>
<thead>
<tr>
<th>Data HDCST1/</th>
<th>00015400</th>
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<tbody>
<tr>
<td># 1.851, 5.357, 8.869, 17.673, 53.284, 00015401</td>
<td></td>
</tr>
<tr>
<td># 1.784, 5.156, 8.534, 17.001, 51.214, 00015404</td>
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<tr>
<td># 1.737, 5.016, 8.300, 16.532, 49.789, 00015405</td>
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</tr>
<tr>
<td># 1.450, 4.152, 6.859, 13.648, 41.088, 00015406</td>
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<tr>
<td># 1.413, 4.043, 6.677, 13.281, 39.966, 00015407</td>
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<td># 1.800, 5.205, 8.619, 17.178, 51.860, 00015414</td>
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</table>
C************************* C**************************** 001 5600
C CALCULATE COSTS FOR PROJECTS INVOLVING RECONSTRUCTION OVER 001 5700
C XPR PERCENT OF TOTAL LENGTH, AND RESURFACING AND POSSIBLY 001 5800
C WIDENING OVER THE REMAINDER. 001 5900
C***************************************************************************
C--------------------------------------------------------------------------
C SUBROUTINE CALCST(PWO,PWN,RFO,RFN,GRF,XPR) 001 6000
C***************************************************************************
COMMON /AA/ CROWK,CDRNK,CBRIK,COTHK,CSITU,CEWVU,XPOVH 001 6100
COMMON /BB/ CPVU1,CPVU2,CPVU3,TPV1,TPV2,TPV3,XCON 001 6200
COMMON /CC/ CASE(4,3),OPTION(5,10),WIDTH(4),COST(4) 001 6300
COMMON /DD/ HDMC1(5,12),HDMC2(5,12),HDMC3(5,12),HDMC4(5,12) 001 6400
COMMON /EE/ RLENF,CPKM 001 6500
COMMON /FF/ OPCOST(12,6,4),XNPV(5),XBCR(5),FLOW(5) 001 6600
COMMON /GG/ HDCST1(5,12),HDCST2(5,12),HDCST3(5,12),HDCST4(5,12) 001 6700
C***************************************************************************
C***************************************************************************
SW = 1.00 001 6800
XPR = XPR/100. 001 6900
RWO = PWO+SW*2. 001 7000
RWN = PWN+SW*2. 001 7100
RWD = RWN-RWO 001 7200
C***************************************************************************
C***************************************************************************
C COMPLETE RECONSTRUCTION 001 7300
XWFN = (7.4-0.5*(7.4-PWN))/7.4 001 7400
X1 = EXP(0.0278*GRF) 001 7500
X2 = 1.61 * EXP(-0.0114*GRF) 001 7600
CSITN = (1.77*X1 + RWN*X2) * CSITU * 1000. 001 7700
XH = 1.41 + 0.129*(GRF-RFN) + 0.0139*GRF 001 7800
CEWVN = (RWN + 0.731*XH) * XH * CEWVU * 1000. 001 7900
CPKMN = CPKM * XWFN * 1000. 001 8000
CPVMN=(CPVU1*TPV1*PWN + CPVU2*TPV2*PWN + CPVU3*TPV3*RFN)*1000. 001 8100
C***************************************************************************
C***************************************************************************
C RESURFACING ONLY (FOR UNCONSTRUCTED ROAD LENGTH) 001 8200
CPVMO=(CPVU1*TPV1*PWO) * 1000. 001 8300
C***************************************************************************
C***************************************************************************
C WIDENING 001 8400
XWFW = 0.5*RWD/7.4 001 8500
IF (RWD.GT.0.0) THEN 001 8600
CSITW = X2*RWD*CSITU * 1000. 001 8700
CEWVW = XH*RWD*CEWVU * 1000. 001 8800
CPKMW = CPKM*XWFW * 1000. 001 8900
CPVMW=(CPVU1*TPV1*RWD + CPVU2*TPV2*RWD + CPVU3*TPV3*RWD)*1000. 001 9000
ELSE 001 9100
CSITW = 0.0 001 9200
CEWVW = 0.0 001 9300
CPKMW = 0.0 001 9400
CPVMW = 0.0 001 9500
ENDIF 001 9600
C***************************************************************************
C***************************************************************************
C TOTAL COSTS IN THOUSANDS PER KM 001 9700
COST(1)=(CSITN+CEWVN+CPKMN+CPVMN)*XPR*XCON/1000. 001 9800
COST(2)=(CSITW+CEWVW+CPKMW+CPVMW)*((1-XPR))*XCON/1000.*1.20 001 9900
COST(3)=(CPVMO*1.1-XPR)*XCON/1000. 002 0000
COST(4)=(COST(1)+COST(2)+COST(3))*((1+XPOVH)) 002 0100
WRITE(22,205)CSITN,CSITW,CEWVN,CEWVW,CPKMN,CPKMW,CPVMN, 002 0200
# CPVMW,CPVMO,XPR,XWFN,XWFW 002 0300
205 FORMAT(12F10.1) 002 0400
RETURN 002 0500
END 002 0600
| DATA HDCST2/ |  |
| # | 1.841, 5.320, 8.929, 18.008, 55.707, | 00015415 |
| # | 1.774, 5.120, 8.595, 17.532, 53.618, | 00015416 |
| # | 1.728, 5.031, 8.362, 16.862, 52.165, | 00015417 |
| # | 1.441, 4.169, 6.924, 13.977, 43.440, | 00015418 |
| # | 1.406, 4.064, 6.748, 13.624, 42.372, | 00015419 |
| # | 1.397, 4.035, 6.700, 13.528, 42.080, | 00015420 |
| # | 1.251, 3.597, 5.970, 12.060, 37.616, | 00015421 |
| # | 1.224, 3.517, 5.835, 11.791, 36.812, | 00015422 |
| # | 1.218, 3.497, 5.803, 11.725, 36.614, | 00015423 |
| # | 1.193, 3.422, 5.678, 11.473, 35.840, | 00015424 |
| # | 1.770, 5.144, 8.524, 17.008, 51.389, | 00015425 |
| # | 1.803, 5.258, 8.741, 17.617, 54.609, | 00015426 |

| DATA HDCST3/ |  |
| # | 1.832, 5.370, 8.949, 18.142, 56.513, | 00015427 |
| # | 1.766, 5.171, 8.616, 17.465, 54.407, | 00015428 |
| # | 1.720, 5.032, 8.383, 16.994, 52.944, | 00015429 |
| # | 1.433, 4.177, 6.947, 14.110, 44.216, | 00015430 |
| # | 1.398, 4.067, 6.773, 13.762, 43.165, | 00015431 |
| # | 1.389, 4.039, 6.726, 13.667, 42.876, | 00015432 |
| # | 1.243, 3.599, 5.992, 12.191, 38.402, | 00015433 |
| # | 1.216, 3.520, 5.859, 11.927, 37.619, | 00015434 |
| # | 1.210, 3.501, 5.827, 11.863, 37.424, | 00015435 |
| # | 1.185, 3.426, 5.702, 11.610, 36.656, | 00015436 |
| # | 1.764, 5.160, 8.586, 17.328, 53.647, | 00015437 |
| # | 1.804, 5.287, 8.810, 17.850, 55.747, | 00015438 |

| DATA HDCST4/ |  |
| # | 1.825, 5.390, 9.026, 18.480, 59.376, | 00015439 |
| # | 1.759, 5.192, 8.694, 17.801, 57.195, | 00015440 |
| # | 1.714, 5.055, 8.464, 17.330, 55.680, | 00015441 |
| # | 1.428, 4.197, 7.030, 14.448, 49.953, | 00015442 |
| # | 1.395, 4.097, 6.863, 14.113, 45.929, | 00015443 |
| # | 1.386, 4.069, 6.818, 14.021, 45.641, | 00015444 |
| # | 1.238, 3.626, 6.076, 12.529, 41.213, | 00015445 |
| # | 1.213, 3.549, 5.949, 12.279, 40.478, | 00015446 |
| # | 1.207, 3.531, 5.918, 12.217, 40.291, | 00015447 |
| # | 1.182, 3.456, 5.792, 11.964, 39.542, | 00015448 |
| # | 1.753, 5.160, 8.609, 17.479, 54.592, | 00015449 |
| # | 1.809, 5.345, 8.949, 18.327, 59.336, | 00015450 |

END 00015500
APPENDIX D

SAMPLES OF INPUT DATA FOR THE COSTA RICA CASE STUDY

Note that data are samples only from the HDM-III input data files

A. EXISTING ROAD CONDITION

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<tr>
<th>LINK</th>
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B. CONSTRUCTION

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C. MAINTENANCE:

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## D. VEHICLES:

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<td>39.89</td>
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**FINANCIAL**

| VEHICLE | 17778 | 40000 | 86667 | 35556 | 77778 | 102222 |
| TIRE | 37.84 | 50.78 | 313.82 | 202.22 | 261.53 | 415.89 |
| MAINT LAB | 1.446 | 1.446 | 1.446 | 1.446 | 1.446 | 1.446 |
| CREW | 0 | 0 | 2.652 | 2.652 | 2.652 | 2.652 |
| TIME | 0 | 0 | 0 | 0 | 0 | 0 |
| STANDING | 15 | 15 | 15 | 15 | 15 | 15 |
| INTEREST | 22 | 22 | 22 | 22 | 22 | 22 |
| PETROL | 0.533 | 0.422 | 2.433 |

**ECONOMIC**

| VEHICLE | 5820 | 9700 | 58200 | 14938 | 55290 | 87300 |
| TIRE | 32.09 | 43.04 | 265.96 | 171.58 | 221.64 | 352.44 |
| MAINT LAB | 1.446 | 1.446 | 1.446 | 1.446 | 1.446 | 1.446 |
| CREW | 0 | 0 | 2.652 | 2.652 | 2.652 | 2.652 |
| TIME | 0.406 | 0.406 | 0 | 0 | 0 | 0 |
| STANDING | 12 | 12 | 12 | 12 | 12 | 12 |
| INTEREST | 15 | 15 | 15 | 15 | 15 | 15 |
| PETROL | 0.273 | 0.258 | 1.244 |

**FOREIGN**

| VEHICLE | 5820 | 9700 | 58200 | 14938 | 55290 | 87300 |
| TIRE | 19.25 | 25.82 | 159.58 | 102.95 | 132.98 | 211.46 |
| PETROL | 0.246 | 0.232 | 1.120 |

| DEPRECIATE | 1 | 1 | 1 | 1 | 1 | 1 |
| UTILIZE | 1 | 1 | 3 | 3 | 3 | 3 |
| KM Driven | 14000 | 14000 | 80000 | 40000 | 55000 | 60000 |
| VEH Life | 11 | 13 | 8 | 8 | 8 | 10 |
| HR Driven | 400 | 350 | 2000 | 1600 | 1800 | 2000 |
| PASS | 3 | 4 | 54 | 2.5 | 1.8 | 1.8 |

**END SERIES**
E. TRAFFIC:

TRAFFIC ET01 N
DESCRIPTION 100 VEH/H
NEW ADT 1985 1 24 46 6 15 4 5
NEW ADT 1986 3 3 3 4 4 4 4
END TRAFFIC

K. RUN CONTROL:

RUN TITLE GEOM: COSTA RICA BRAZ 1985 2004
DATE 125 8 19
CURRENCY U.S. DOLLARS U.S. DOLLARS 1.0000
ROUGHNESS Q1 Q1

ALTERNATIVE EL01AL00
TRAFFIC ET011985
MAINTENANCE MT091985

ALTERNATIVE EL01AL01
TRAFFIC ET011985
CONSTRUCTION CO11985
MAINTENANCE MT091985
APPENDIX E

SAMPLES OF INPUT DATA FOR THE INDIA CASE STUDY

Note that data are samples only from the HDM-III input data files

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C. MAINTENANCE

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D. VEHICLES

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END SERIES
E. TRAFFIC

TRAFFIC E101  N
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NEW ADT  1986 3 5 5 5

K. RUN CONTROL

RUN TITLE GEOMETRY: INDIA IND1 1985 2004
DATE 85 8 23
CURRENCY INDIAN RUPEES U.S. DOLLARS 0.0880
ROUGHNESS BI

ALTERNATIVE E101AL00
TRAFFIC ET011985
MAINTENANCE MT021985

ALTERNATIVE E101AL01
TRAFFIC ET011985
CONSTRUCTION C011985
MAINTENANCE MT021985